WASTEWATER FACILITIES PLAN
for the
TOWN OF EXETER, NEW HAMPSHIRE

Preliminary Draft
October 2014

WRIGHT-PIERCE
Engineering a Better Environment

TOWN OF EXETER, NEW HAMPSHIRE

WASTEWATER FACILITIES PLAN

PRELIMINARY DRAFT
OCTOBER 2014

Prepared By:

Wright-Pierce
230 Commerce Way, Suite 302
Portsmouth, NH 03801

TOWN OF EXETER, NEW HAMPSHIRE

WASTEWATER FACILITIES PLAN

TABLE OF CONTENTS

TABLE OF CONTENTS (CONTINUED)

TABLE OF CONTENTS (CONTINUED)

TABLE OF CONTENTS (CONTINUED)

TABLE OF CONTENTS (CONTINUED)

TABLE OF CONTENTS (CONTINUED)

LIST OF TABLES

LIST OF FIGURES

TABLE OF CONTENTS (CONTINUED)

LIST OF APPENDICES *[NOT INCLUDED IN DRAFT]*

Section 1

SECTION 1

EXECUTIVE SUMMARY

1.1 INTRODUCTION

The Town of Exeter owns and operates a wastewater collection, treatment and disposal system which serves the Town of Exeter as well as small portions of the Towns of Stratham and Hampton. The collection system includes 9 pumping stations and approximately 48 miles of sewers. There are approximately 3,600 wastewater accounts.

The wastewater treatment facility (WWTF) is an aerated lagoon facility with disinfection that was constructed in 1964 and comprehensively upgraded in 1988. The WWTF discharges effluent into a tidally-influence segment of the Squamscott River (Class B), upstream of the Great Bay. The WWTF outfall has a dilution factor of 25:1. The effluent must meet standards set forth in state and federal water quality legislation, including the Clean Water Act. The WWTF effluent quality requirements are contained in a National Pollutant Discharge Elimination System (NPDES) permit which is issued by the US Environmental Protection Agency (EPA).

EPA issued a new NPDES permit to the Town in December 2012, which included requirements that the existing WWTF is not able to accomplish. EPA then issued an Administrative Order on Consent (AOC) to the Town in June 2013. The AOC provides a framework and schedule for the Town to achieve compliance with the NPDES permit requirements.

1.2 PURPOSE AND ORGANIZATION OF REPORT

The purpose of this report is to provide a technical basis upon which to make wastewater management decisions necessary to comply with the AOC and NPDES permit. This report is divided into the following sections:

- Section 1: Executive Summary
- Section 2: Wastewater Flows, Loads and Effluent Standards
- Section 3: Evaluation of Existing Facilities

- Section 4: Town-Wide Nitrogen Management
- Section 5: Evaluation of Alternatives
- Section 6: Recommended Plan
- Section 7: Project Costs and Financing

A list of commonly used acronyms and abbreviations is provided in **Table 1-1**.

1.3 CONCLUSIONS

Based on the work completed as a part of this project, the following conclusions are provided:

1. The WWTF has provided reliable service since the late 1980s; however, many of the equipment and building systems are reaching the end of their useful life and will require comprehensive upgrades in order to provide continued reliable service for the planning period. In addition, the WWTF will require significant modifications in order to meet the AOC requirements (i.e., less than 8 mg/l effluent total nitrogen) and/or the NPDES permit requirements (i.e., less than 3 mg/l effluent total nitrogen). Refer to **Section 3** for additional information.

2. Future flows projections were prepared based on input from the Public Works Department and Planning Department and are consistent with the Town Master Plan. Future flows are projected to be less than the NPDES permit flow limit (3.0-mgd) at "Build-Out" for the Town of Exeter alone and to be less than the NPDES permit flow limit at the "Planning Horizon" (2040) if the Towns of Stratham and Newfields were connected to the Exeter WWTF. Refer to **Section 2** for additional information.

3. Based on the Town's evaluative criteria, the recommended approach is to upgrade the existing facility to achieve 5 mg/l effluent total nitrogen. The Town will utilize either On-Site Alternative No. 2 (Bardenpho) or On-Site Alternative No. 3 (SBR). In the future, if required, this system can be upgraded to achieve 3 mg/l effluent total nitrogen. Refer to **Section 5** for additional information.

TABLE 1-1
LIST OF COMMONLY USED ACRONYMS AND ABBREVIATIONS

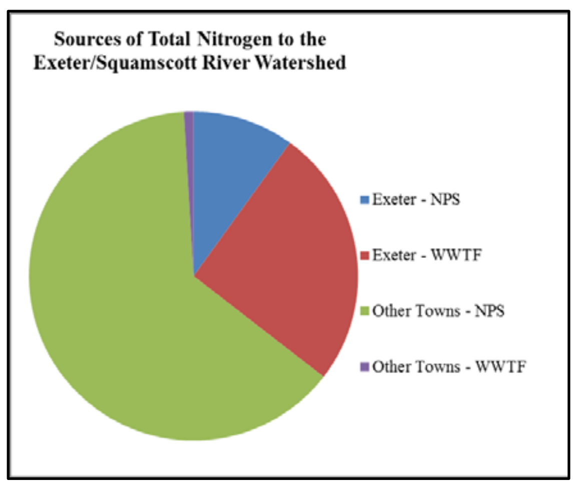
AO	Administrative Order
AOC	Administrative Order on Consent
BMP	Best Management Practice
BOD	Biochemical Oxygen Demand
BOS	Board of Selectmen
CAPE	Climate Adaption Plan for Exeter
CMOM	Capacity, Management, Operations and Maintenance (for sewer collection system)
COD	Chemical Oxygen Demand
CSO	Combined Sewer Overflow
Current	Covering the dates 2011 to 2013, applied to population, wastewater flow or nitrogen load conditions
DO	Dissolved Oxygen
Future	Referring to population, wastewater flows or nitrogen loads, expected at Planning Horizon (2040)
GIS	Geographic Information System
gpd	Gallons Per Day
gpd/sf	Gallons Per Day Per Square Foot
IDDE	Illicit Discharge Detection and Elimination
I/I	Infiltration and Inflow
lb/day, lb/yr	Pounds Per Day, Pounds Per Year
mgd	Million Gallons Per Day
mg/l	Milligrams Per Liter
MS4	Municipal Separate Storm Sewer System
NHDES	New Hampshire Department of Environmental Services
NPDES	National Pollutant Discharge Elimination System
NPS	Non-Point Source
PH	Planning Horizon
ppm	Parts Per Million
PREP	Piscataqua Region Estuaries Partnership
SRF	State Revolving Fund (administered by New Hampshire Department of Environmental Services)
SSO	Sanitary Sewer Overflow
TBA	Total Buildable Area
TBO	Theoretical Build-Out
TDN	Total Dissolved Nitrogen
TKN	Total Kjeldahl Nitrogen
TMDL	Total Maximum Daily Load
TN	Total Nitrogen
TP	Total Phosphorous
USEPA	U.S. Environmental Protection Agency
USGS	United States Geologic Survey
WISE	Water Integration for Squamscott-Exeter
WWFP	Wastewater Facilities Plan
W&SAC	Water & Sewer Advisory Commission

4. The AOC requires significant efforts by the Town to quantify, track, account for and reduce non-point sources of total nitrogen to the Squamscott River and Great Bay. Non-point sources include storm drainage, fertilizer management, septic system nitrogen management and animal waste management. Per the AOC, the Town needs to fund and develop a town-wide Nitrogen Control Plan by September 2018. Refer to **Section 4** for additional information.

5. It is critical for the Town to establish a river monitoring program, in collaboration with other towns and NHDES, in order to establish baseline information. While there is a relatively long-term record of data in Great Bay, such data does not exist for the Squamscott River or the Exeter WWTF. The upcoming Great Dam removal and WWTF upgrade will introduce major changes in the data record for the river. The Town should establish a robust monitoring program, based on sound science, as well as a calibrated water quality model, in order for the Town, NHDES and EPA to properly assess the environmental benefits resulting from these significant capital expenditures. Refer to **Section 4** for additional information.

6. Based on the NHDES Great Bay Nitrogen Non-Point Source Study (June 2014, Appendix H), septic systems which are located greater than 200 meters from shore provide for approximately 74% nitrogen removal from the source to the estuary based in large part on natural attenuation. This is roughly equivalent to nitrogen removal from a WWTF designed to achieve effluent total nitrogen of 8 mg/l. Refer to **Section 4** for additional information.

7. The AOC and NPDES permit require the Town to remove more than its "fair share" of nitrogen from the Exeter River/Squamscott River watershed. Exeter represents approximately 35% of the total nitrogen load to the Exeter River/Squamscott River watershed. The Town should aggressively pursue a watershed funding source for additional point source and non-point source nitrogen controls. The Town should partner with other "point source communities" through the Great Bay Municipal Coalition and/or the Southeast Watershed Alliance. Refer to **Section 4** for additional information.



Source: NHDES-GBNNPS, June 2014

8. There are two on-going studies which will provide information, analysis and conclusions that are essential to the Town's decision making process with regard to the WWTF and its regional upgrade options. Reports for these projects (i.e., the WISE project and the Pease Alternative) must be in-hand in October/November 2014 in order to support the Town's decision making process. Refer to **Sections 4 and 5** for additional information.

9. There is a clear downward trend in peak system flows based on the infiltration/inflow reduction efforts initiated in the late 1990's and continued to present. There is also a downward trend in average system flows. This is a result of the Town's considerable infiltration/inflow removal efforts. This trend should be re-assessed in Spring 2015 to incorporate the results of the on-going and recently efforts with private inflow removal from Phillips Exeter Academy and the Jady Hill neighborhood. Refer to **Section 2** for additional information.

10. The Town's influent sampling program indicates that there is a relatively small data set with relatively large variability. The detailed supplemental sampling program should be continued until there is a sufficient body of data on which to base the design of its upgraded wastewater treatment facilities. In addition, the Town should investigate the impacts of the Exeter Water Treatment Plant discharge as well as potential impacts of industrial user discharges to the variability of the influent concentrations. This topic represents significant uncertainty in terms of the cost of the recommended plan. Refer to **Section 2** for additional information.

1.4 PROJECT COSTS AND FINANCING

The recommended plan, and its estimated cost, is described in detail in **Section 6**. The funding and financing implications are described in detail in **Section 7**. The recommended facilities are estimated to cost approximately $50,700,000 to design/construct and $1,150,000 annually to operate, expressed in 2014 and 2018 dollars, respectively. It is important to note that these estimates <u>do not</u> include costs associated with the non-point source nitrogen reductions or other AOC related compliance items described in **Section 4**.

These project costs are significant and will have a significant impact on the average sewer user rate. Based on the funding assumptions described in **Section 7**, the total annual Sewer Enterprise Fund will increase to approximately $6,520,000 (*with no State Aid Grant*). This results in a 166% increase in the Sewer Enterprise Fund annual budget. If the State of New Hampshire re-establishes the State Aid Grant program, the total annual Sewer Enterprise Fund will increase to approximately $5,310,000 and will result in a 117% increase in the existing Sewer Enterprise Fund annual budget. The Town should also review and revise, as appropriate, all of its other fees.

In order to mitigate these impacts to the sewer user rates, the following grant funding sources should be aggressively pursued: **NHDES State Aid Grant (SAG) and SAG Plus grants; US Economic Development Administration grants; and Unitil grants**.

It is important to note that DES has issued a moratorium on new SAG and SAG Plus grant applications as of July 1, 2013. We recommend that the Town get involved with the New Hampshire Municipal Association's on-going effort to maintain this important grant program. We also recommend that the Town get involved with efforts to create a State Water Trust Fund, which was recommended by the SB60 Joint Legislative Study Commission created to study water infrastructure sustainability funding. The Town should also begin the process of contacting grant agencies and assembling grant application materials.

1.5 PROJECT IMPLEMENTATION

The Administrative Order on Consent (AOC Docket No. 13-010) puts forth a specific implementation schedule, as described in greater detail in **Section 4**. In order to meet the AOC requirement for initiating construction of the new facility, the Town will need to initiate the design phase in January 2015. Accordingly, the following key implementation steps are recommended:

1. Meet to discuss the Town's questions and comments on the draft report.
2. Review the WISE report, CAPE report and Pease Regional Evaluation report when they are issued. Determine whether they modify as conclusions identified herein.
3. Update the report and submit to NHDES and EPA. Address comments received.
4. Engage neighboring communities if the Town intends to serve as a regional host facility.

5. Engage grant funding agencies including NHDES, EDA and Unitil. Complete grant funding applications for portion(s) of the project which are eligible and supported.
6. Review sewer user fees, as well as all other fees, and determine whether revisions are appropriate.
7. Formalize rate increases based on the final project financing scenario.
8. Implement the recommended upgrades in accordance with the approved project schedule.
9. Continue with monitoring, study, planning and implementation of non-point source nitrogen management (refer to **Section 4** of this report).

A **draft** implementation schedule for the recommended plan is presented in **Table 1-2**.

TABLE 1-2
IMPLEMENTATION SCHEDULE

Item	Milestone Dates
Planning	
Draft Report to Town	October 2014
Draft Report to NHDES and EPA	November 2014
Public Informational Meetings	November 2014
Final Report to Town, NHDES and EPA	December 2014
Develop and Submit Grant Applications	January 2015 to March 2015
Design, Bidding & Award	
Design	January 2015 to March 2016
Bidding & Award	April 2016 to June 2016
Town Meeting Funding Authorizations	
Design Funding	Completed March 2014
Construction Funding	March 2015 to March 2016
Construction	
Initiate Construction (AOC)	June 30, 2016 **
Substantially Complete Construction (AOC)	June 30, 2018 **
Meet Interim TN NPDES Permit (AOC)	June 30, 2019 **
Other	
TN Annual Reports (on-going)	2014 to 2018
Squamscott River Monitoring (on-going)	2014 to 2018
Review regulations, ordinances and bylaws (e.g., stormwater, fertilizer control, nitrogen management, etc.)	2015 to 2016
Total Nitrogen Control Plan (AOC)	September 30, 2018 **
Nitrogen Reduction Projects	To be determined
Nitrogen Engineering Evaluation (AOC)	December 31, 2023 **

** AOC deadline

Section 2

WRIGHT-PIERCE
Engineering a Better Environment™

SECTION 2

WASTEWATER FLOWS, LOADS AND EFFLUENT LIMITS

2.1 INTRODUCTION

This report summarizes current land use, population trends and wastewater flows and loadings for the Exeter Wastewater Treatment Facility (WWTF). Daily data has been collected and analyzed from the past seven years of plant operations. This data will be used as the baseline for the projected future flows and loadings. A summary of the current permit requirements as well as potential future permit requirements are also presented in this section.

2.2 LAND USE AND POPULATION DATA

Land use and zoning information presented herein is based on information contained in the Town Master Plan (2002, with selected updates) and the 2013 GIS database information supplied by the Town. The Town has 19 different zoning districts. **Figure 2-1** depicts a simplified zoning map where all similar zoning districts have been consolidated (e.g., R-1, R-2, R-3, etc., consolidated to Residential). **Table 2-1** summarizes the total land area and remaining developable land area, as presented in the Town Master Plan.

The Town Master Plan indicates several key items related to potential future development:

- There is relatively limited buildable acreage in the Industrial, Office and Commercial Districts (page LU-6)
- there is a fair amount of buildable acreage in Residential Districts (page H-34)
- The Town does not plan to extend the sewer service area (page LU-30) and future residential development outside the sewered area will rely on septic systems (page LU-12)

Since the development of the Town Master Plan, there have been discussions with Stratham regarding potentially extending sewer service into Stratham to a designated area along Route 108 and there has been some consideration of potentially extending sewer service to the High School in the future if septic system maintenance and replacement becomes problematic.

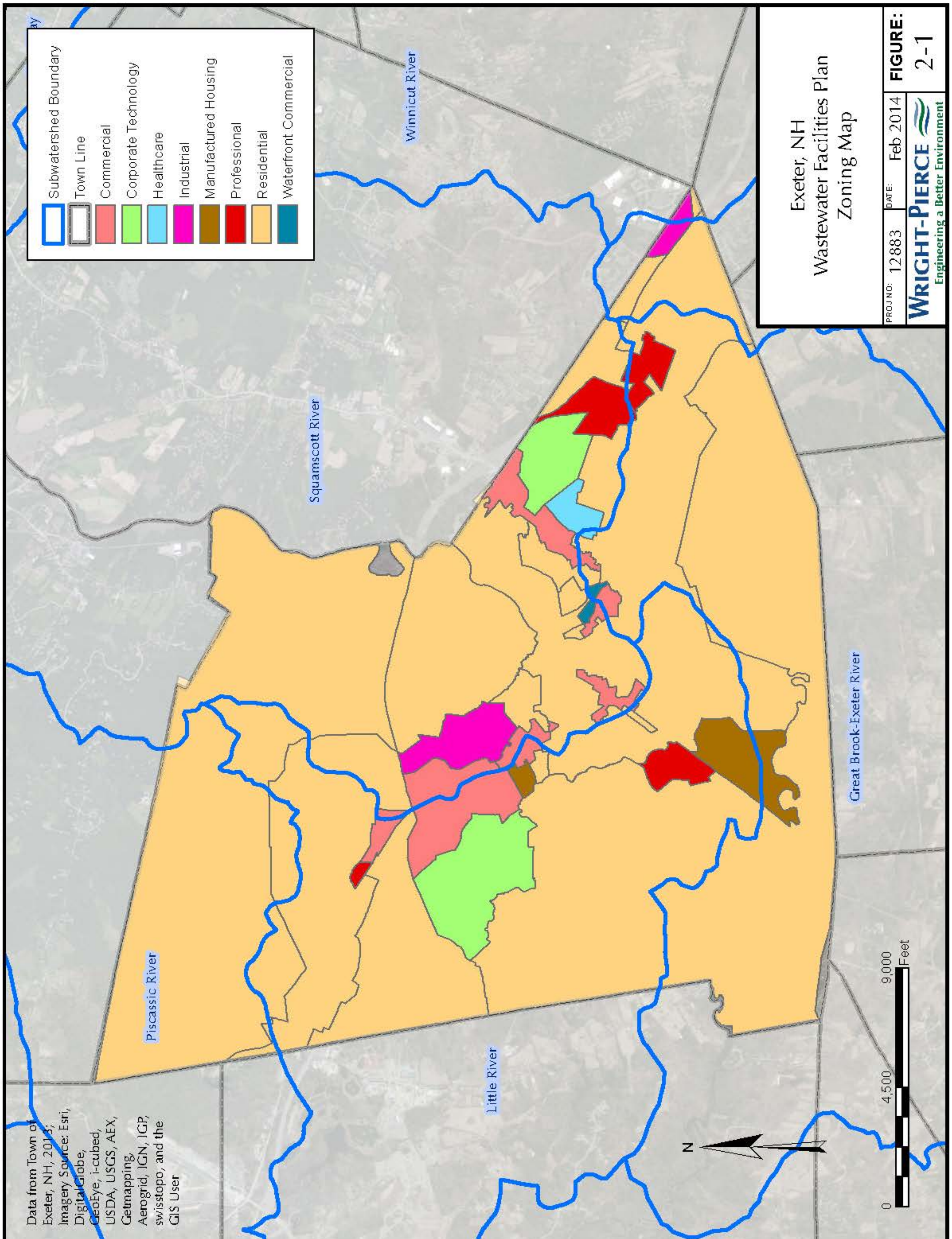

Subwatershed Boundary
Town Line
Commercial
Corporate Technology
Healthcare
Industrial
Manufactured Housing
Professional
Residential
Waterfront Commercial
Winnicut River
Squamscott River
Great Brook-Exeter River
Piscassic River
Little River
Data from Town of Exeter, NH, 2013; Imagery Source: Esri, DigitalGlobe, GeoEye, i-cubed, USDA, USGS, AEX, Getmapping, Aerogrid, IGN, IGP, swisstopo, and the GIS User
N
0
4,500
9,000
Feet
Exeter, NH
Wastewater Facilities Plan
Zoning Map
PROJ NO: 12883
DATE: Feb 2014
FIGURE: 2-1
WRIGHT-PIERCE
Engineering a Better Environment

TABLE 2-1
SUMMARY OF LAND USE AND BUILDABLE ACRES

Development Zone	Total Land Area (acres)	% of Total Land Area	Total Land Area Remaining as Developable[1] (acres)	% of Total Land Area Remaining as Developable [1]
C-1 Central Area Commercial	65.0	0.5%	0.0	0.0%
C-2 Highway Commercial	173.6	1.4%	46.5	26.8%
C-3 Epping Road Hwy Comm.	269.0	2.1%	112.7	41.9%
NP Neighborhood Professional	136.7	1.1%	16.9	12.4%
WC Waterfront Commercial	9.4	0.1%	0.0	0.0%
CT Corp Technology Park	145.0	1.1%	61.9	42.7%
CT-1 Corp Technology Park 1	333.7	2.6%	80.6	24.1%
PP Professional Tech Park	98.4	0.8%	28.4	28.8%
I Industrial	488.9	3.9%	135.6	27.7%
H Healthcare	44.6	0.4%	2.2	5.0%
RU Rural	2,836.3	22.4%	952.6	33.6%
R-1 Single Family	5,388.4	42.6%	1,544.1	28.7%
R-2 Single Family	2,150.2	17.0%	270.6	12.6%
R-3 Single Family	70.1	0.6%	2.3	3.3%
R-4 Multi-Family	157.0	1.2%	25.1	16.0%
R-5 Multi-Family/ Elderly	33.7	0.3%	1.3	3.8%
R-6 Retirement Community	45.2	0.4%	32.4	71.5%
M Mobile Home Park	180.5	1.4%	1.8	1.0%
MS Mobile Home Subdivision	19.7	0.2%	0.2	1.1%
TOTAL	**12,646**	**100%**	**3,315**	**26%**

Source:
1) *Town Master Plan (2002, 2010), Table H-11 – Land Area and Developable Land by Zone*.

According to the 2010 US Census, Exeter had a population of approximately 14,306 residents. Population growth in Town was significant from the 1970s to 2000; however, population growth has slowed considerably since 2000. Two previous population projections were developed for the Seacoast region – one by the New Hampshire Office of Energy and Planning (NHOEP) and the other by a consultant which incorporated input from NHOEP and Rockingham Planning Commission. A summary of past and projected future population is presented in **Table 2-2**.

TABLE 2-2
SUMMARY OF PAST AND PROJECTED FUTURE POPULATION

Date	US Census	Projected by NH OEP[1]	Projected in Seacoast Study[2]
1970	8,892	-	-
1980	11,024	-	-
1990	12,654	-	-
2000	14,098	-	14,098
2010	14,306	-	-
2020	-	14,187	-
2025	-	14,499	17,280
2040	-	14,851	-
2055	-	-	20,161

Source:
1) *New Hampshire Population Forecast by Municipality:2013.* NH Office of Energy and Planning (2013).
2) *New Hampshire Seacoast Region Wastewater Management Feasibility Study*. AECOM (2005).

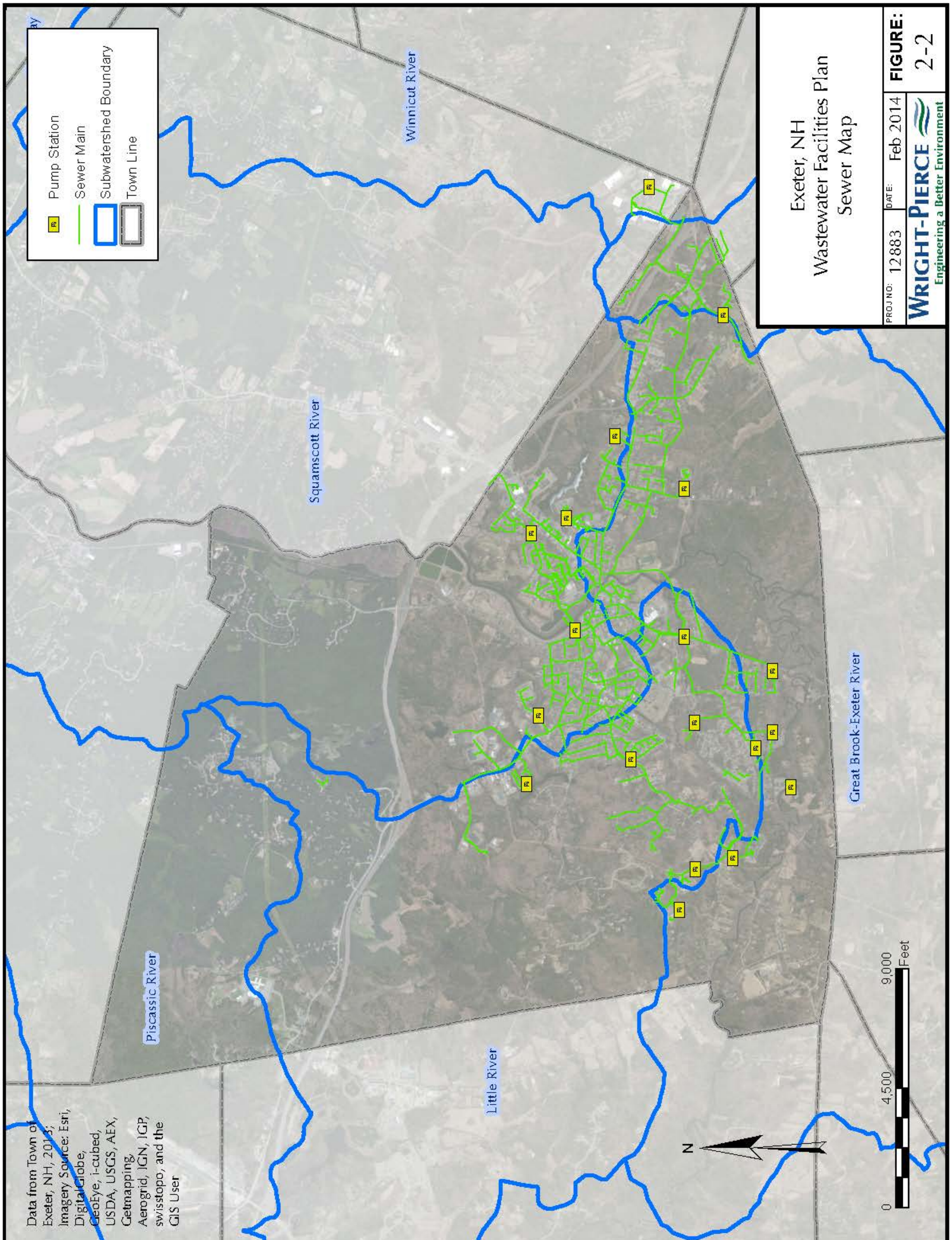
2.3 SEWER SERVICE AREA

The existing sewer service area is presented on **Figure 2-2**. Based on information contained in the Town Master Plan as well as water and sewer account information provided by the Town, approximately 85% of the housing units are served by public sewer. Additional information is summarized in **Table 2-3**.

TABLE 2-3
ESTIMATE OF SEWERED VERSUS NON-SEWERED POPULATION

	Town Master Plan (1990 Census)	Current Estimate
Total Population	12,654	14,306*
Total Housing Units	5,346	6,422*
Persons per Household	2.3	2.2*
Wastewater Accounts	Unknown	3,600 **
Housing Units Served by Public Sewer	4,522	5,000 **
% of Total Housing Units	85%	78%
Estimated Population Served by Public Sewer	10,400	11,000 **
% of Total Population	82%	77%

Note: "*" indicates 2010 Census data; "**" indicates estimated based on Town data


Pump Station
Sewer Main
Subwatershed Boundary
Town Line
Winnicut River
Squamscott River
Great Brook-Exeter River
Piscassic River
Little River
Exeter, NH
Wastewater Facilities Plan
Sewer Map
PROJ NO: 12883
DATE: Feb 2014
FIGURE: 2-2
WRIGHT-PIERCE
Engineering a Better Environment
Data from Town of Exeter, NH, 2013; Imagery Source: Esri, DigitalGlobe, GeoEye, i-cubed, USDA, USGS, AEX, Getmapping, Aerogrid, IGN, IGP, swisstopo, and the GIS User
N
0
4,500
9,000
Feet

2.4 CURRENT WASTEWATER FLOWS AND LOADINGS

Exeter's wastewater is generated from two general sources: *sewage flow* from residential, commercial, and industrial sources; and *infiltration and inflow (I/I)*, which is water from extraneous sources such as storm drains, cellar drains and roof leaders and is generally associated with rainfall or ground water. The Town does not currently accept *septage*, which is highly concentrated sludge from septic tanks or boat pump-outs. The current treatment process does not have any recurring *recycle* flows or loads.

Influent flow data is measured by a magnetic flow meter installed on the influent forcemain (from the Main Pump Station) in August 2010. Prior to that time, influent flow data was measured by an area-velocity insert-type flow meter in the 24-inch influent pipe in the Grit Building. Influent samples are collected just downstream of the manual bar rack by a composite sampler that was permanently installed in January 2014 (time-based composite samples). Prior to that time, influent data is based on grab samples collected from influent channel just upstream of the manual bar rack.

Effluent flow data is measured by a Parshall flume with ultrasonic flow element. Effluent samples are collected upstream of the Parshall flume just before the ultrasonic level by a composite sampler that was permanently installed in July 2013 (time-based composite samples). Prior to that time, effluent data is based on grab samples collected from the same location.

2.4.1 Data Analysis

The key flow and load conditions that have been utilized as the basis of the evaluation for unit processes are identified and defined as follows:

- Annual Average: This is the average of daily data for the study period. The average flows and loadings are important benchmarks, but capacity is typically controlled by other design criteria.
- Maximum Month: This is the maximum 30-day running average for the study period which is calculated for each parameter independently (i.e. the maximum TSS loading condition may not have occurred at the same time as the maximum month BOD loading

condition). The maximum month conditions are an important measure of sustained capacity. Note that this data is not available for nitrogen and phosphorus loadings as samples are only taken quarterly.

- Maximum Day: This is the maximum single day that occurs for each parameter during the period and, similarly to the maximum month condition, each parameter is calculated independently. The single maximum day values for the data set are reported along with the 98th percentile values. Typically, unit processes are designed to handle the peak recorded flow rate (i.e. 100th percentile) and the 98th percentile loading rates. This is done to eliminate any outliers in the data set.
- Peak Hour: This is the peak instantaneous recorded value during any one day and is only determined (and available) for flow. The peak hour flow is an important hydraulic consideration for the design of unit processes. Sufficient hydraulic capacity is typically provided for the peak recorded flow rate to prevent overtopping of channels and structures. However, individual unit processes would typically be sized for the 98th percentile flow rate.
- Minimum Day: This is the minimum recorded value during any one day and is only determined for flow. The minimum hour flow is an important hydraulic consideration for the design of unit processes to ensure that velocities are adequate to prevent solids deposition and that the unit processes are not oversized.

A review of current flows and loadings for the WWTF was conducted by analyzing data from Monthly Operation Reports (MOR) from 2007 through 2013. Flow and loadings information is presented below, summarized in **Table 2-4,** and depicted on **Figures 2-3 through 2-7**. Additional nutrient-related data was obtained from supplemental sampling conducted by WWTF as well as by third party groups (e.g., PREP). Additional "Influent Characterization" sampling was completed in 2010 and in 2014 and is presented in **Section 2.4.5**.

TABLE 2-4

SUMMARY OF WWTF INFLUENT FLOWS AND LOADS (2007 to 2013)

Parameter	Flow[1]		Influent TSS		Influent BOD	
	MGD	P.F.	mg/L	lb/day	mg/L	lb/day
Average for Individual Years						
2007	1.88	-	138	2,116	168	2,574
2008	2.34	-	127	2,407	148	2,806
2009	2.13	-	142	2,483	233	4,009
2010	2.13	-	186	3,037	164	2,809
2011	1.93	-	175	2,706	139	2,127
2012	1.58	-	185	2,423	174	2,259
2013	1.63	-	183	2,460	156	2,018
Summary for 2007 to 2013						
Average	1.95	-	161	2,522	172	2,756
Minimum Month	1.09	0.6	87	1,215	58	890
Maximum Month	4.23	2.2	291	3,955	367	6,137
Maximum Day[3,4]	4.64	2.4	321	4,688	411	7,286
Peak Hour[5]	6.42	3.3	-	-	-	-
No. Data Points	2,557	-	340	-	303	-
Summary for 2011 to 2013						
Average	1.71	-	180	2,544	158	2,138
Minimum Month	1.18	0.7	88	1,215	75	890
Maximum Month	2.88	1.7	273	3,632	239	3,484
Maximum Day[3,4]	3.75	2.2	349	4,376	325	4,210
Peak Hour[5]	5.65	3.3	-	-	-	-
No. Data Points	1,096	-	133	-	100	-

Notes:

1. Flows are recorded by area-velocity insert flow meter from 2007 to August 2010.
2. Flows are recorded by magnetic flow meter on influent forcemain from August 2010 to present.
3. Maximum Day values for BOD and TSS are based on 98th percentile of collected data
4. Maximum Day Flow is based on 99th percentile of collected data.
5. Peak hour flow is not recorded. Peak hour flow is estimated by on TR-16 peaking factor of 3.3.
6. All data is based on Grab samples.

FIGURE 2-3
WWTF INFLUENT FLOWS (MGD)

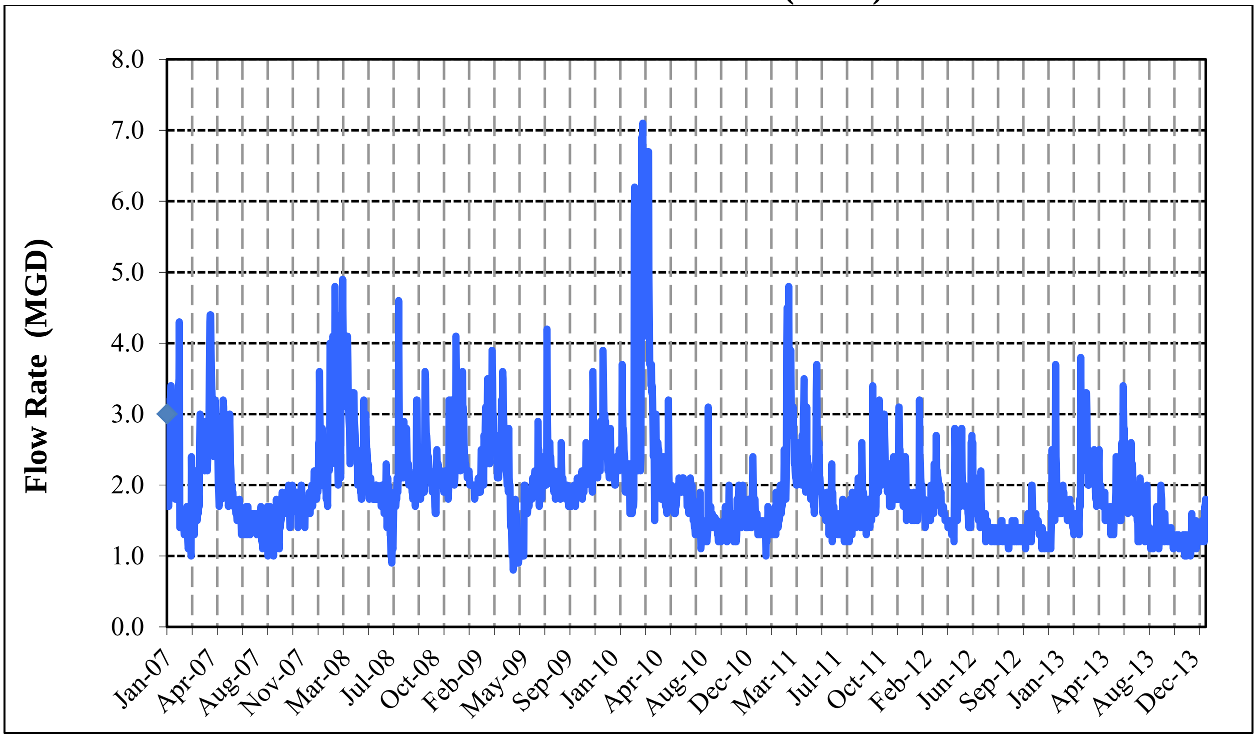


FIGURE 2-4
INFLUENT FLOW – EVENT FREQUENCY

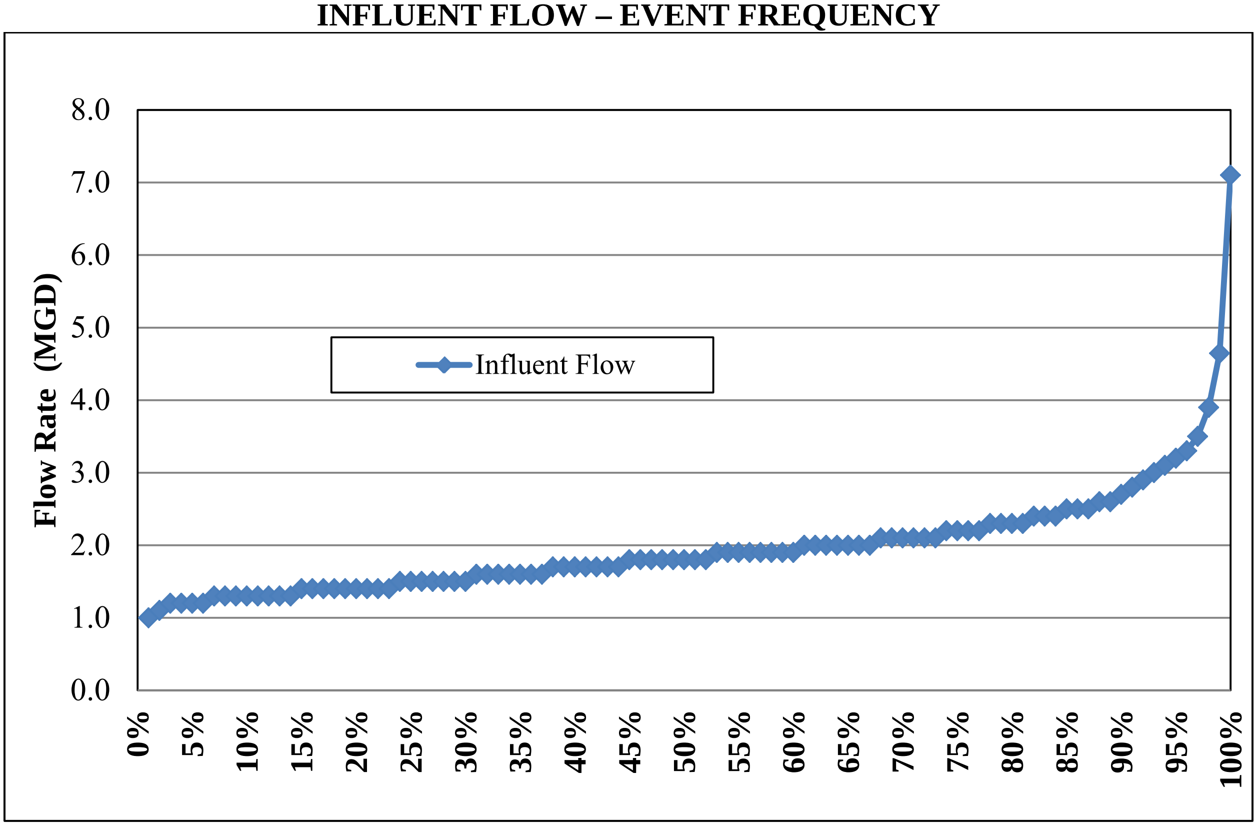


FIGURE 2-5
INFLUENT BOD & TSS MASS LOADINGS

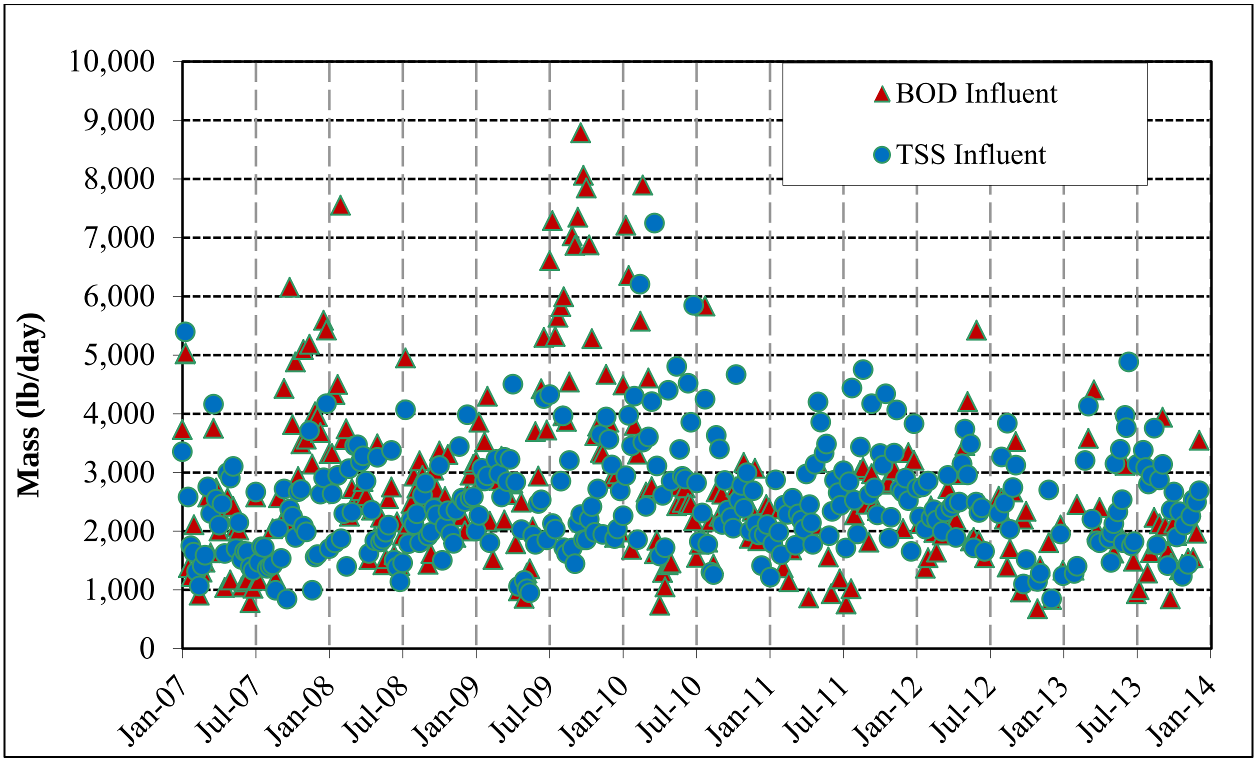


FIGURE 2-6
INFLUENT BOD & TSS MASS LOADINGS – EVENT FREQUENCY

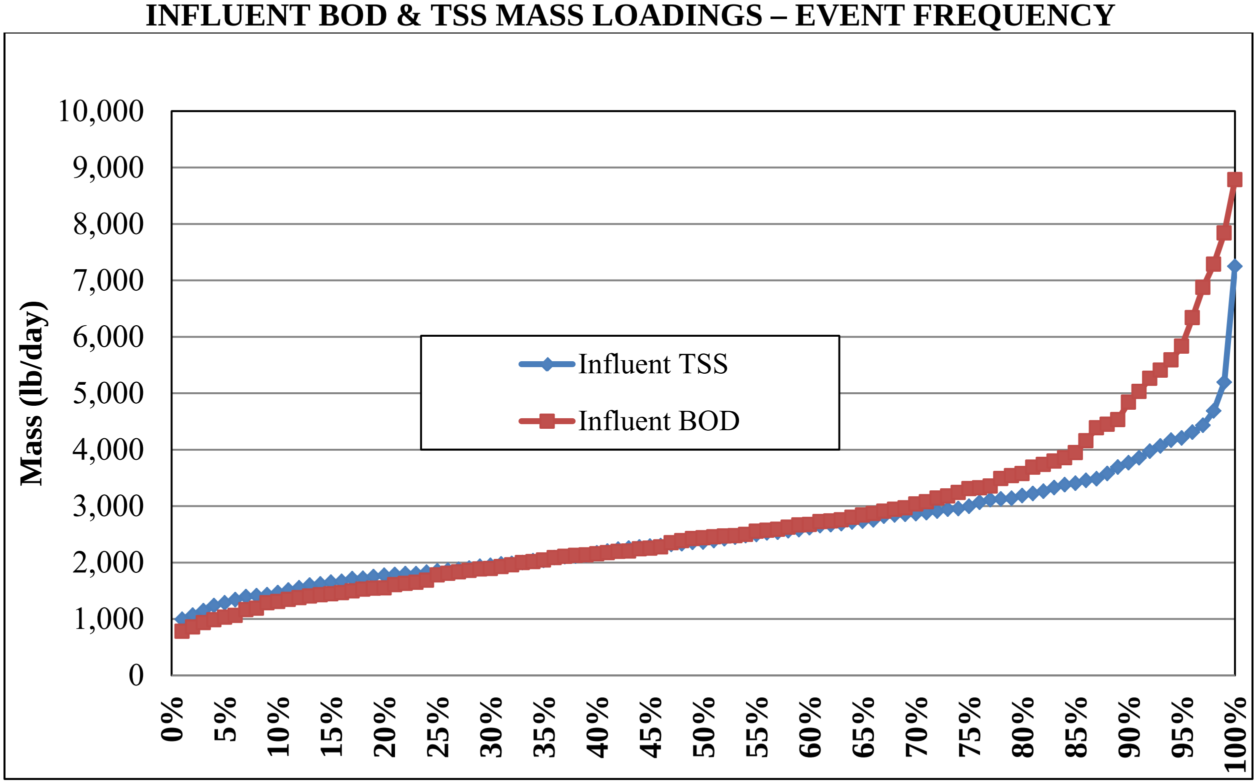
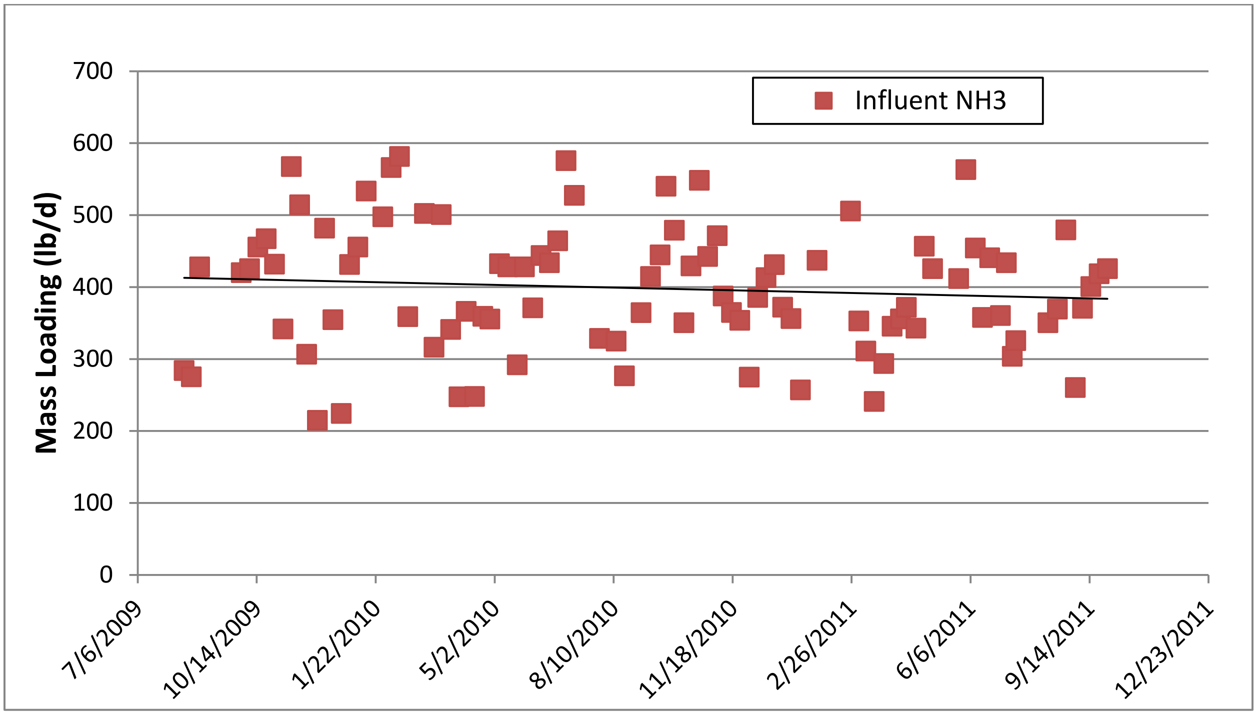


FIGURE 2-7
INFLUENT AMMONIA MASS LOADINGS



2.4.2 Industrial Users and Industrial Pretreatment Program

The Town's Sewer Use Regulations define industrial waste as "any process waste which is distinct from sanitary waste". Major industrial users are required to obtain an Industrial Discharge Permit (IDP) through the Town's Industrial Pretreatment Program (IPP). The definition of a major industrial user is discussed in the Sewer Regulations, but generally includes facilities with design flows over 10,000 gpd or with the requirement to install pretreatment in accordance with Federal standards. A summary of the industries which currently have an IDP is presented in **Table 2-5**. A summary of typical IPP permit limits is included in **Table 2-6.**

TABLE 2-5
SUMMARY OF MAJOR INDUSTRIAL USERS

Name	Permitted Annual Average Flow Rate (gpd)
Exeter Hospital	48,500
Phillips Exeter Academy	7,055
Lindt	6,000
Chemtan	1,770
Cobham Defense	12,477
OSRAM	5,685
Total	81,487

Note: The Town is currently in negotiations with Lindt regarding increasing its permit from 6,000 to 30,000 gpd.

TABLE 2-6
TYPICAL INDUSTRIAL DISCHARGE PERMIT LIMITS

Parameter	Typical Limit
Annual Average/Daily Maximum Flow (gpd)	Based on Expected Flow
BOD (mg/l)	276
TSS (mg/l)	306
Oil/Grease (mg/l)	100SL/350L
pH	5.5-11.5
Temperature (°F)	150
Chromium (mg/l)	1.7
Cyanide (mg/l)	0.08
Ammonia N (mg/l)	20
Total Kjeldahl Nitrogen (mg/l)	Monitor only
Chloride (mg/l)	1500
Sulfate (mg/l)	150, 1500
Sulfide (mg/l)	1
Arsenic (mg/l)	0.004
Cadmium (mg/l)	0.001
Copper (mg/l)	0.12
Lead (mg/l)	0.013
Mercury (mg/l)	0.00004
Nickel (mg/l)	0.02
Selenium (mg/l)	0.003
Silver (mg/l)	0.038
Zinc (mg/l)	0.42

2.4.3 Inflow/Infiltration

The Town has completed numerous infiltration/inflow (I/I) studies in the past to address significant I/I flows in the system. The most recent study encompassed approximately 75% of the collection system and determined that in some areas, infiltration accounted for 20-70% of total dry weather flows and over 90% of peak wet weather flows (Underwood Engineering, 2013). The 2013 report estimated that peak I/I accounted for 63% of total system flows. I/I flows tend to be highest when the groundwater is high (spring) which can be observed in **Figure 2-3** and **Figure 2-4.**

The Town has recently completed projects focused on reducing I/I, including private inflow and groundwater infiltration. A listing of I/I projects completed by the Town from 2011 to 2013 is provided below.

- Jady Hill Utility Replacement Project Phase I and Phase II
    - Project Dates: October 2011 through August 2013
    - 8-inch diameter sewer: 5,500 lf
    - 4-inch diameter sewer service in right-of-way: 1,650 lf
    - 4-inch diameter sewer service out right-of-way: 3,500 lf
    - 15-inch diameter storm drain: 3,540 lf
    - 18-inch diameter storm drain: 460 lf
    - 24-inch diameter storm drain: 1,065 lf
    - 4-inch diameter storm drain service in right-of-way: 2, 780 lf
    - 4-inch diameter storm drain service out right-of-way: 2,500 lf

- Water Street Sewer Interceptor Improvement Project
    - Project Dates: November 2011 through November 2012
    - 24-inch diameter sewer: 204 lf
    - 30-inch diameter sewer: 63 lf
    - 36-inch diameter sewer: 43 lf
    - New CSO Structure installed
    - Disconnected storm drain system from CSO structure

  - Re-lined 300 lf of 18-inch diameter sewer

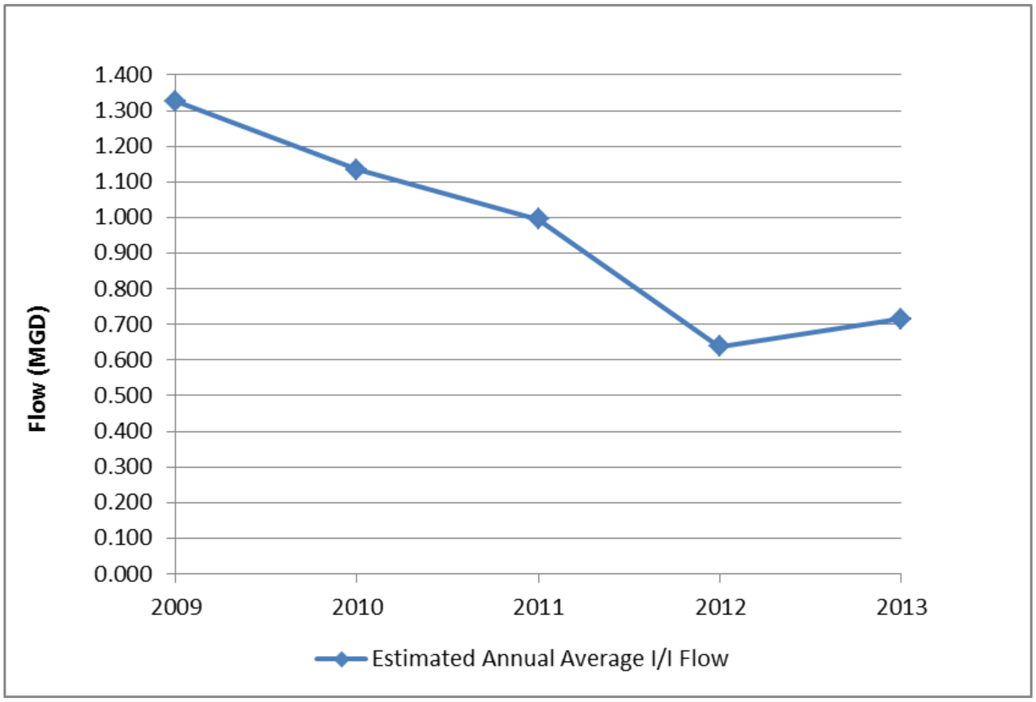
- Water Street / Main Pump Station Sewer Manhole Rehabilitation
  - Project Date: November 2012
  - Chemically sealed and grouted SMH-902, SMH-937 and SMH-938

- Phillips-Exeter Academy and Spring Street I/I Removal
  - Project Date: August 2013
  - Removed Langdon Merrill Dining Hall sump pump and roof leaders from sewer
  - Removed two catch basins from sewer

- Portsmouth Avenue Water and Sewer Improvement Project
  - Project Dates: November 2013 through June 2014
  - 8-inch diameter sewer: 2,550 lf
  - 10-inch diameter sewer: 250 lf
  - 6-inch diameter sewer service: 1,350lf

Figure 2-8 shows the estimated I/I flows based on a review water use records and estimated sanitary flows. In 2009, I/I represented approximately 60% of influent flows to the WWTF; whereas by 2013, I/I represented approximately 35% of the influent flows to the WWTF. The Town continues to make improvements to reduce I/I flows through regular O&M and sewer main repair/replacement projects.

FIGURE 2-8
ESTIMATED INFILTRATION AND INFLOW TRENDS



2.4.4 Septage

The Exeter WWTF does not currently accept septage flows. It is estimated that the non-sewered buildings in Exeter generate approximately 650,000 gallons per year of septage; which is currently disposed of at the Hampton WWTF (Seacoast Region Wastewater Management Study, 2005).

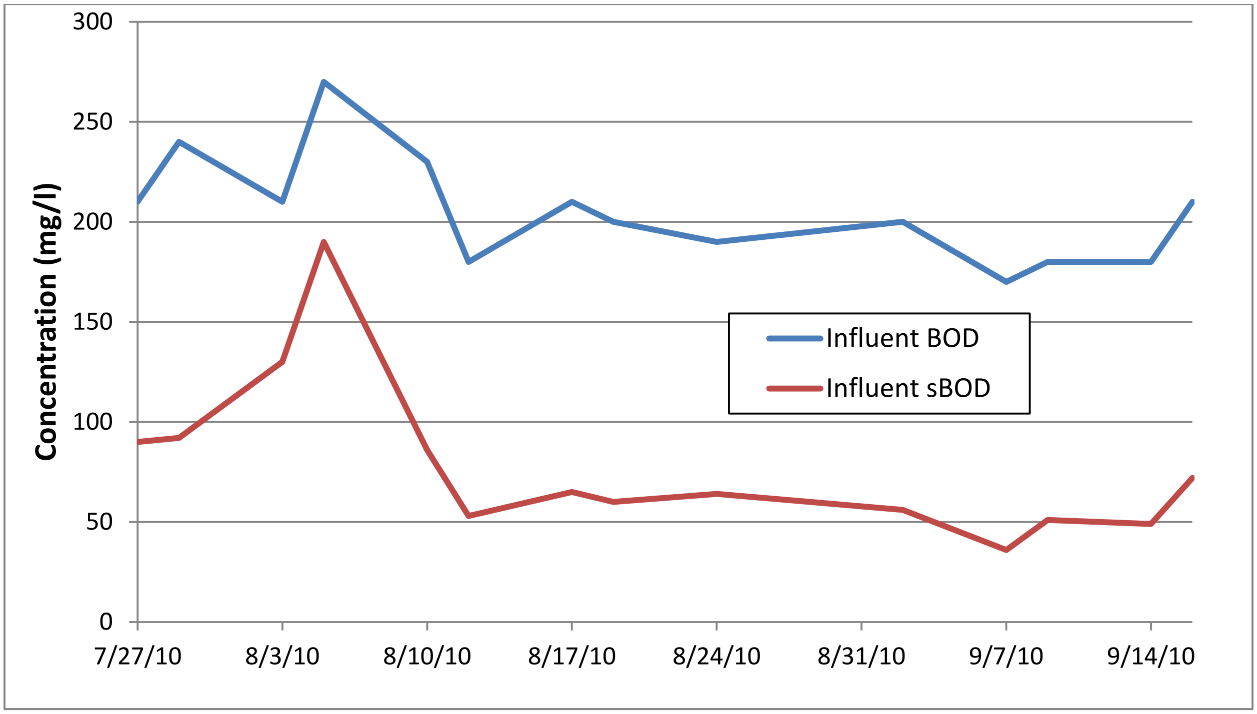
2.4.5 Supplemental Sampling Program

To gather sufficient data for a wastewater facility plan for a WWTF facing nutrient limits, a supplemental influent wastewater characterization program was implemented between July 2010 and January 2011. This data and is summarized in **Table 2-7** and was used to populate **Figures 2-9, 2-10**, **2-11** and **2-12.** The samples were time-based composites collected at the influent sampler from the influent channel. The supplemental sampling program provided composite samples necessary to determine typical influent characteristics.

TABLE 2-7
INFLUENT COMPOSITE SAMPLING DATA

Compound	Average (mg/l)	Maximum (mg/l)	Minimum (mg/l)	No. of Data Points
July 2010 to January 2011				
Total Kjeldahl Nitrogen	28	37	16	43
Ammonia Nitrogen	22	26	13	43
Organic Nitrogen	6	13	1	43
Total Suspended Solids	217	256	174	13
Volatile Suspended Solids	161	234	62	13
Biochemical Oxygen Demand (BOD)	201	263	110	18
BOD, Soluble	78	174	36	14
Chemical Oxygen Demand (COD)	226	302	150	45
COD, Soluble	150	211	86	45
Total Phosphorus	3.9	5.3	2.0	11
Ortho Phosphorus	1.9	2.6	1.1	11
BOD:TKN Ratio	7.0	9.1	5.0	14
BOD:TP Ratio	47.8	79.9	34.0	8
BOD:SBOD Ratio	3.0	4.7	1.4	14
VSS:TSS	0.74	0.95	0.27	13
January 2014 to June 2014				
Total Kjeldahl Nitrogen	24	38	13	29
Ammonia Nitrogen	21	33	12	29
Organic Nitrogen	5	13	0	22
Total Suspended Solids	311	880	120	24
Volatile Suspended Solids	280	840	116	24
Biochemical Oxygen Demand (BOD)	237	390	120	29
BOD, Soluble	58	110	36	29
Chemical Oxygen Demand (COD)	379	720	140	29
COD, Soluble	139	260	27	29
Total Phosphorus	3.7	6.9	2.3	29
Ortho Phosphorus	2.1	4.4	1.0	29
BOD:TKN Ratio	10.1	17.5	5.8	29
BOD:TP Ratio	67.9	134.5	37.5	29
BOD:SBOD Ratio	4.1	6.1	1.9	29
VSS:TSS	0.90	0.99	0.54	24
Alkalinity as CaCO3	152	220	55	28

FIGURE 2-9
INFLUENT BOD AND SBOD CONCENTRATIONS



Note: Influent BOD and sBOD samples were only taken from 7/27/2010 to 9/14/2010

FIGURE 2-10
INFLUENT COD AND SCOD CONCENTRATIONS

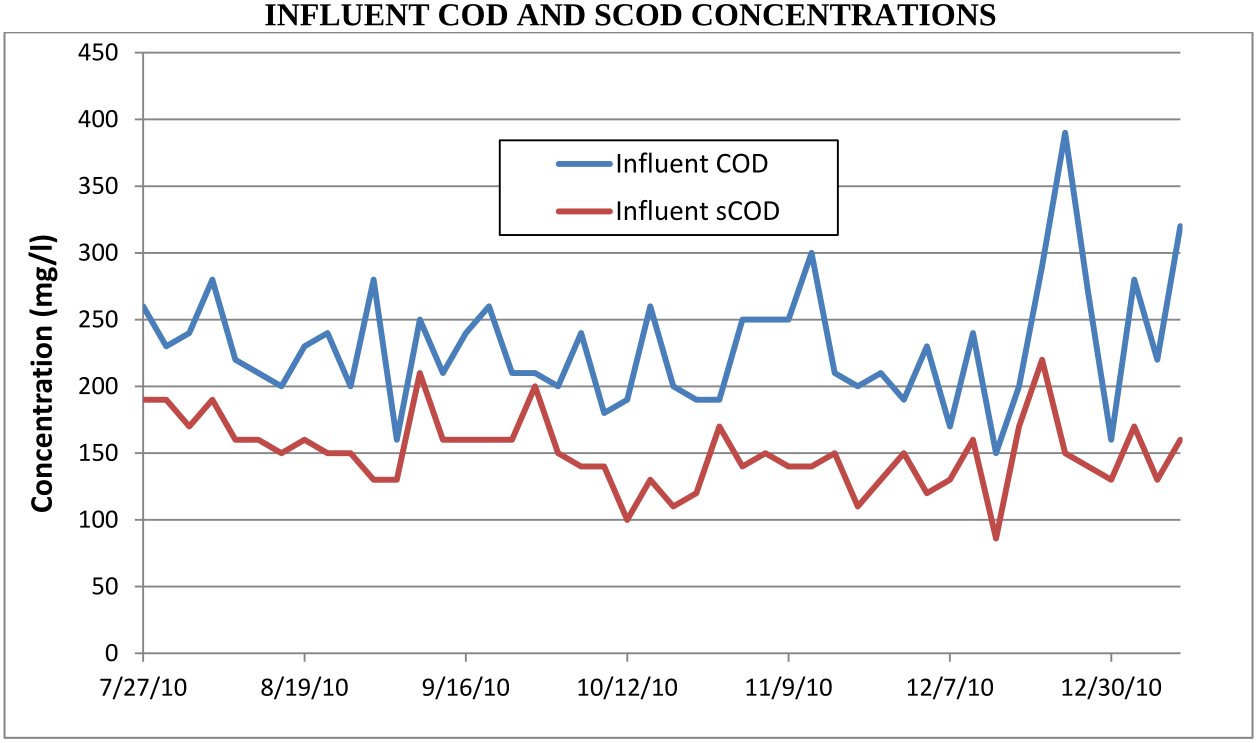
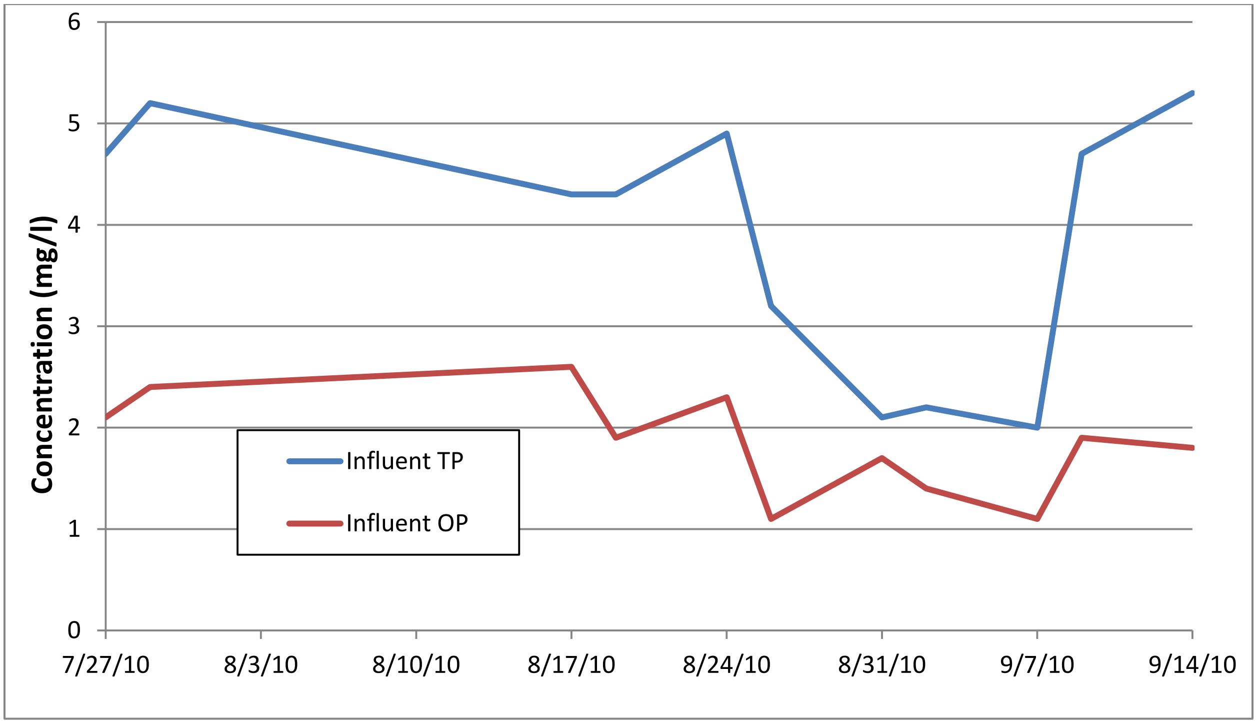
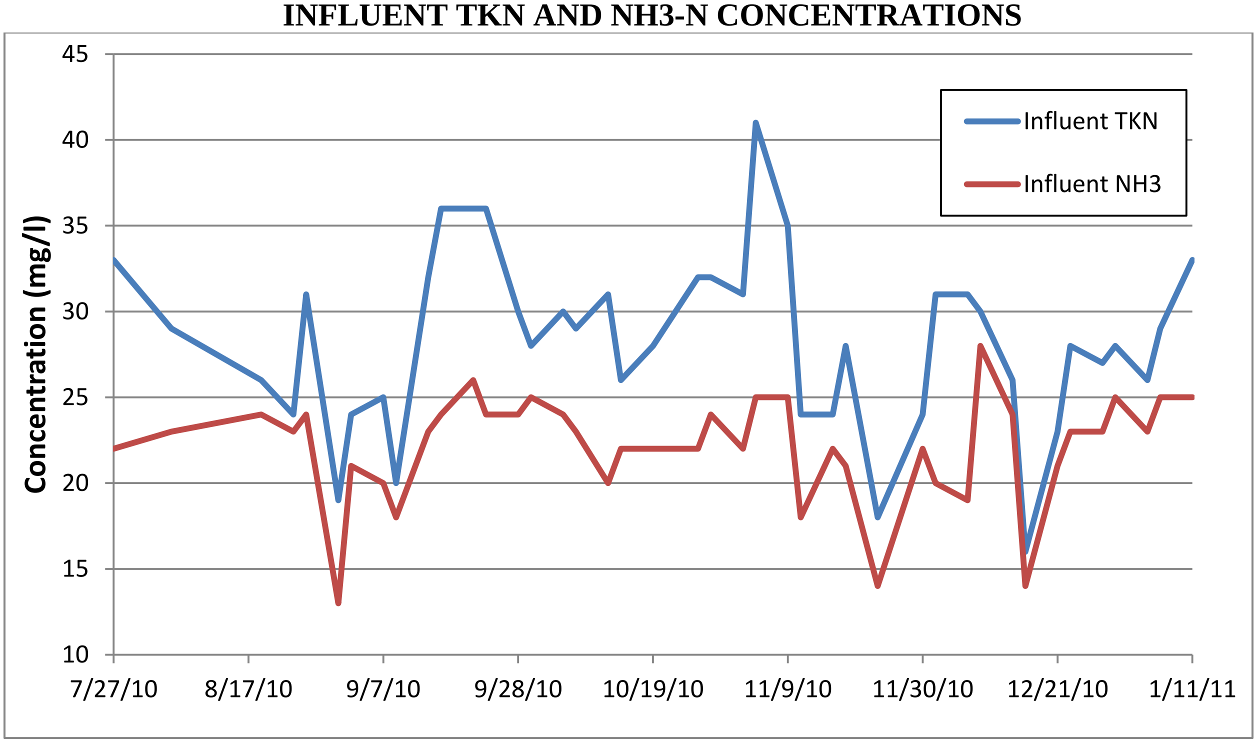


FIGURE 2-11
INFLUENT TP & OP CONCENTRATIONS



Note: Influent TP and OP samples were only taken from 7/27/2010 to 9/14/2010

FIGURE 2-12
INFLUENT TKN AND NH3-N CONCENTRATIONS



2.4.6 Combined Sewer and Sanitary Sewer Overflows

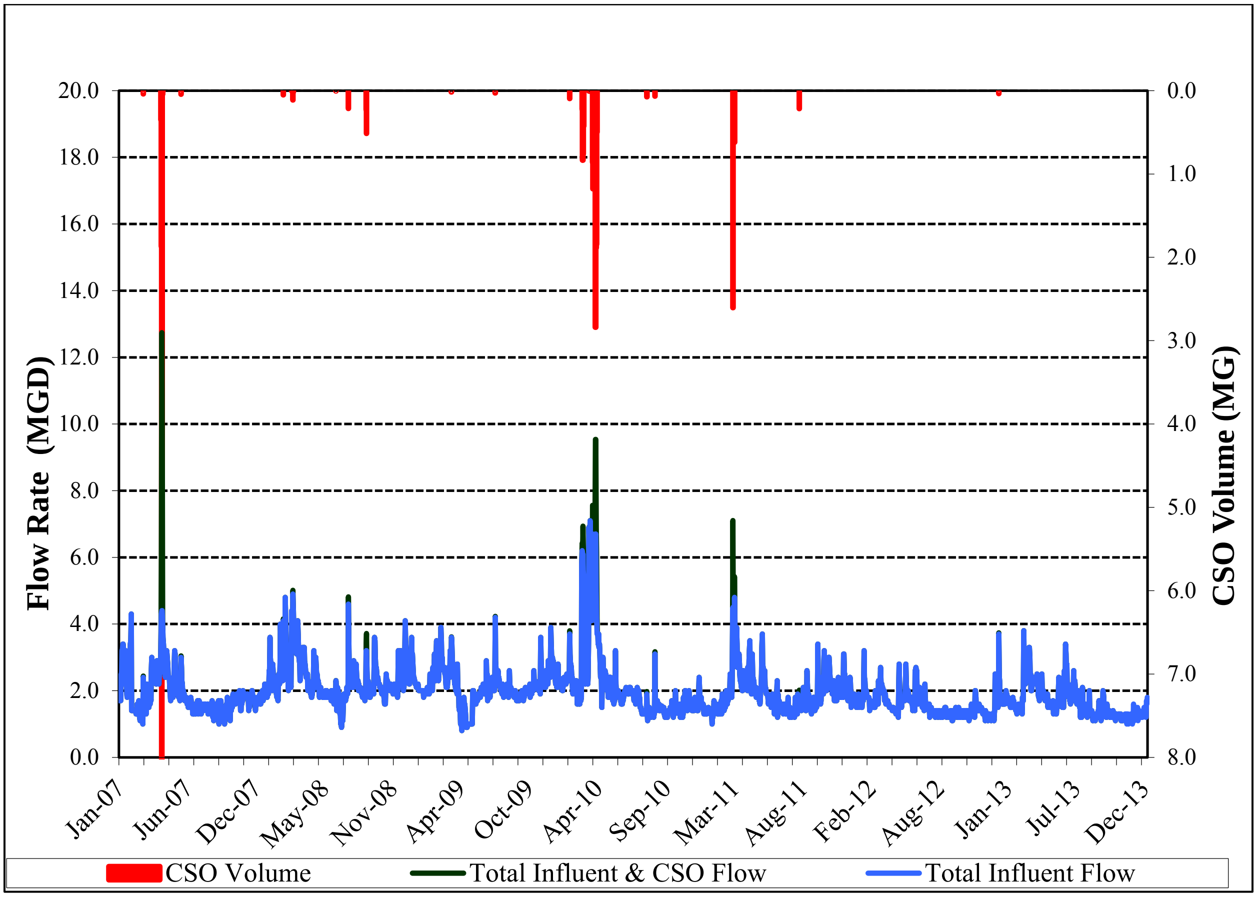
The Town has approximately 49 miles of separated gravity sewer lines, portions of which were originally constructed as combined sewers. The system still contains two diversion structures and one licensed CSO discharge (Outfall #003, located at Clemson Pond and controlled by an outlet weir and tide gates). A summary of CSO events is shown in **Table 2-7**. **Figure 2-13** depicts WWTF flows, CSO flows and CSO volumes from 2007 through 2013. The graph also portrays the "theoretical peak system flow" if all flow were captured and directed to the WWTF. In 2007, the theoretical peak daily system flow was approximately 13.0 mgd; however, the theoretical peak daily system flow has been less than 10.0 mgd since that time. Clearly, the I/I removal work completed by the Town over the past 5 years has significantly decreased rates and volumes of CSOs in the system.

Sanitary sewer overflows (SSO) occur when wastewater exits the collection system at an unlicensed location (e.g., manhole). SSOs often occur due to undersized piping, excessive I/I, lack of O&M and lack of standby power. In Exeter's case, the most common reason for a reported SSO was a surcharged line and pipe blockages. SSO record keeping is essential to making adjustments to the Town's collection system operational procedures. **Table 2-8** summarizes the SSOs that have occurred since 2007. **Figure 2-14** depicts the location of the SSOs and frequency of occurrence.

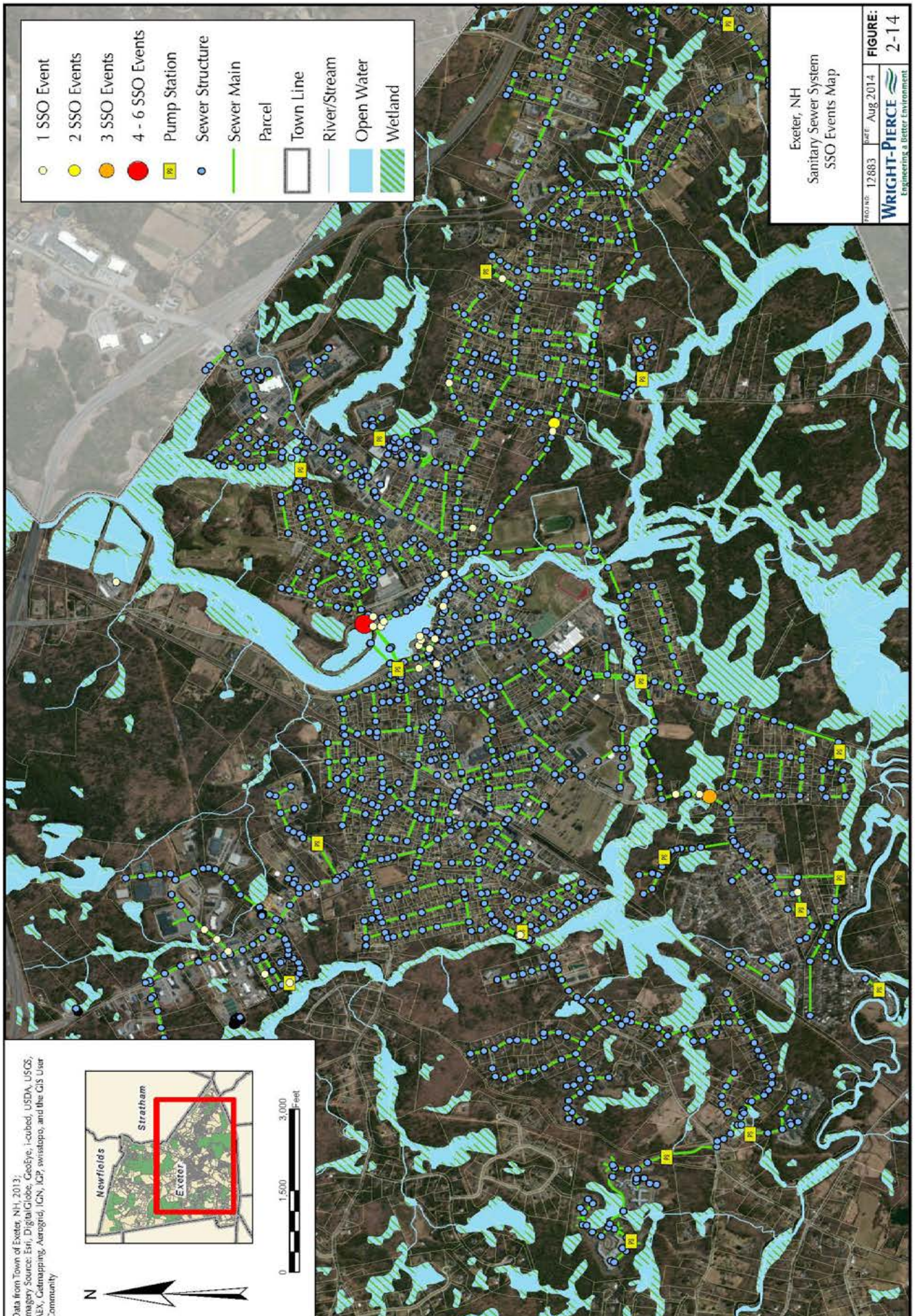
TABLE 2-8
SUMMARY OF CSO AND SSO EVENTS

Year	Annual CSO Events	Annual CSO Volume (MG)	Annual WWTF Volume (MG)	Annual Wastewater Volume (MG)	% of Annual Wastewater Volume as CSO	Total SSO Events	Dry Weather SSO Events
2007	8	17.2	693.5	710.7	2.4%	3	3
2008	8	1.1	839.5	840.6	0.1%	3	3
2009	2	0.05	766.5	766.5	<0.1%	6	6
2010	23	17.0	777.5	794.5	2.1%	11	0
2011	3	3.4	693.5	696.9	0.5%	2	2
2012	1	0.04	576.7	576.7	<0.1%	4	4
2013	0	0	595.0	595.0	0%	5	5

FIGURE 2-13
WWTF INFLUENT AND CSO FLOWS



There is a clear downward trend in peak system flows based on the infiltration/inflow reduction efforts initiated in the late 1990's and continued to present. There is also a downward trend in average system flows. This is a result of the Town's considerable infiltration/inflow removal efforts. This trend should be re-assessed in Spring 2015 to incorporate the results of the on-going and recently efforts with private inflow removal from Phillips Exeter Academy and the Jady Hill neighborhood.


1 SSO Event
2 SSO Events
3 SSO Events
4 - 6 SSO Events
Pump Station
Sewer Structure
Sewer Main
Parcel
Town Line
River/Stream
Open Water
Wetland
Exeter, NH
Sanitary Sewer System
SSO Events Map
PROJ NO. 12883
DATE Aug 2014
FIGURE: 2-14
WRIGHT-PIERCE
Engineering a Better Environment
Data from Town of Exeter, NH, 2013;
Imagery Source: Esri, DigitalGlobe, GeoEye, i-cubed, USDA, USGS, AEX, Getmapping, Aerogrid, IGN, IGP, swisstopo, and the GIS User Community
Newfields
Stratham
Exeter
0
1,500
3,000
Feet
N

2.4.7 Groundwater Discharge Flows

The existing WWTF treatment lagoons are un-lined; therefore, there is a potential for seepage from the lagoons into the groundwater. There are three monitoring wells located down gradient and one up gradient of the lagoons for groundwater sampling and monitoring. See **Section 2.5.4** for a summary of the Groundwater Discharge Permit monitoring requirements.

2.4.8 Summary of Current Flows and Loadings

The majority of the influent sampling record is from grab sample results. While this method is consistent with the NPDES permit requirements and is acceptable for a lagoon plant, it is not sufficient for a non-lagoon plant. Starting in January 2014, the Town began collecting composite influent sampling. Starting in June 2014, the Town converted to *flow-proportional* composite samples. The table below summarizes the differences between the composite sampling data and the grab sampling data for various time periods.

Dates	**Sample Type**	**Avg Flow (mgd)**	**Avg BOD (mg/l)**	**Avg TSS (mg/l)**	**Avg BOD (lb/d)**	**Avg TSS (lb/d)**	**No. of Samples for BOD**
2010/ July to Dec	Composite	1.52	201	217	2,550	2,750	18
2010/ July to Dec	Grab	1.52	185	204	2,350	2,590	21
2011/ July to Dec	Grab	1.83	152	197	2,320	3,010	26
2012/ July to Dec	Grab	1.39	176	200	2,040	2,320	13
2013/ July to Dec	Grab	1.38	164	215	1,890	2,480	22
2014/ Jan to Aug	Grab	1.67	155	145	2,160	2,020	32
2014/ Jan to Aug	Composite	1.67	237	311	3,300	4,330	29

From this data, the following conclusions can be reached:

- The 2010 data set compares reasonably well (i.e., grab to composite, ±5% to 10%); however, the 2014 data set does not compare well (i.e., grab to composite, ±35% to 55%). Initial investigations by Town staff indicate that the Water Treatment Plant discharges to the sewer on the composite sampling day. The Town should review whether there have been any operational changes at the Water Treatment Plan in 2014 which may be causing this. The Town should also investigate whether there are any industrial users which may be contributing to this differential.

- In general, the grab sampling results appear to be lower than the composite sampling results. Composite sampling results are more representative than grab sampling; therefore, the composite sampling results should be given more weight.

- There is a relatively small data set of composite sampling results; therefore, there is some uncertainty related to the appropriate concentrations to utilize as the design basis. The Town should continue its detailed supplemental sampling program until there is a sufficient body of data on which to base the design of its upgraded wastewater treatment facilities.

2.4.9 Summary of Baseline Effluent Nitrogen Loadings

Since the early 2000s, there has been increased interest and attention in total nitrogen in the Great Bay estuary environment. Various groups have collected WWTF effluent samples for nitrogen analysis over the years, including the Piscataqua Region (PREP), HydroQual and the Town. Most of the earlier sampling efforts were grab samples collected monthly; while the more recent sampling efforts have been weekly time-based composite samples. A summary of the annual total nitrogen concentrations and loads is presented in **Table 2-9. Figure 2-15** through **Figure 2-17** depict the effluent total nitrogen concentrations and loads, from the various sampling efforts.

TABLE 2-9
EFFLUENT TN VALUES TO SQUAMSCOTT RIVER

Period	NH3-N Avg. Concentration (mg/l)	Total Nitrogen Avg. Concentration (mg/l)	Estimate of Annual Total Nitrogen Load (tons/yr)	Notes
2008	11.7	14.4	46.28	1
2011	14.8	14.7	49.11	2
2012	16.0	19.0	44.29	3
2013	21.5	22.4	48.85	4
2014/ Jan to June	n/a	21.1	TBD	5

Notes:

1. For 2008, the Town collected 54 grab samples for NH3-N and PREP collected 10 grab samples for TN. Annual load estimated by PREP (2008).
2. For 2011, the Town collected 51 grab samples for NH3-N and Hydroqual collected 2 grab samples for TN.
3. For 2012, the Town collected 50 grab samples for NH3-N and 6 grab samples for TN.
4. For 2013, the Town collected 10 grab samples for NH3-N and 12 grab and 27 composite samples for TN.
5. For 2014 (through June), the Town collected 0 samples for NH3-N and 28 composite samples for TN.
6. The estimate of annual TN was generated by multiplying the average nitrogen load/day by 365 days/year.
7. The TN Annual loads for 2012 and 2013 were based on estimates for months with no available data.

FIGURE 2-15
EFFLUENT TN CONCENTRATIONS

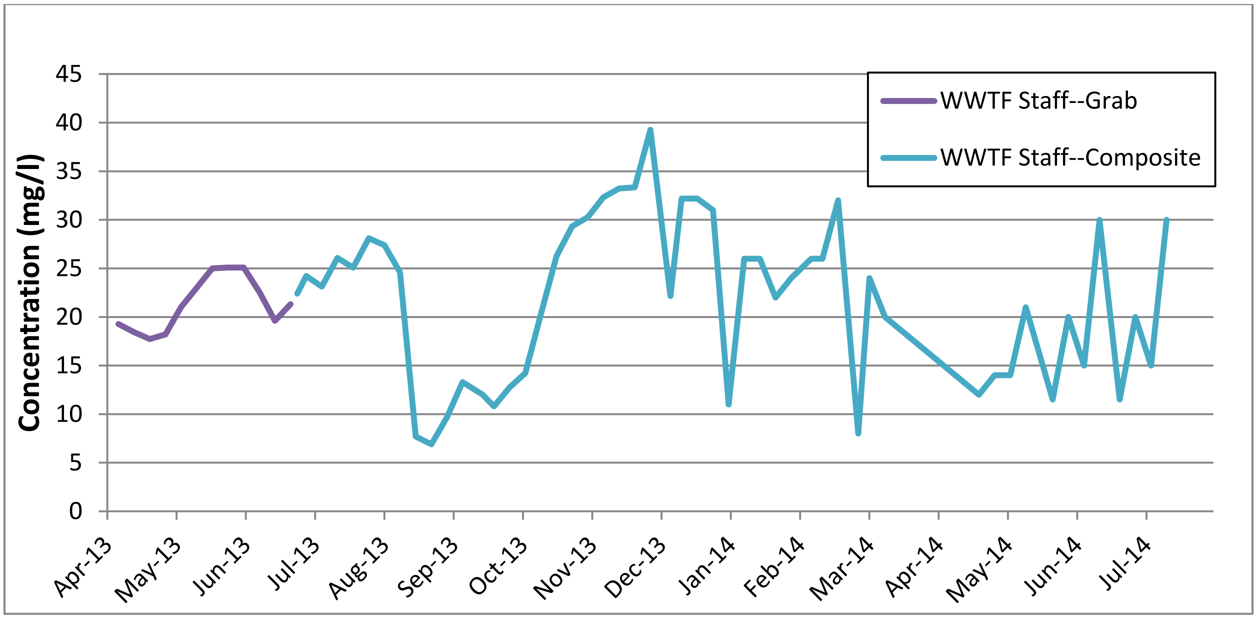


FIGURE 2-16
EFFLUENT TN MASS LOADINGS

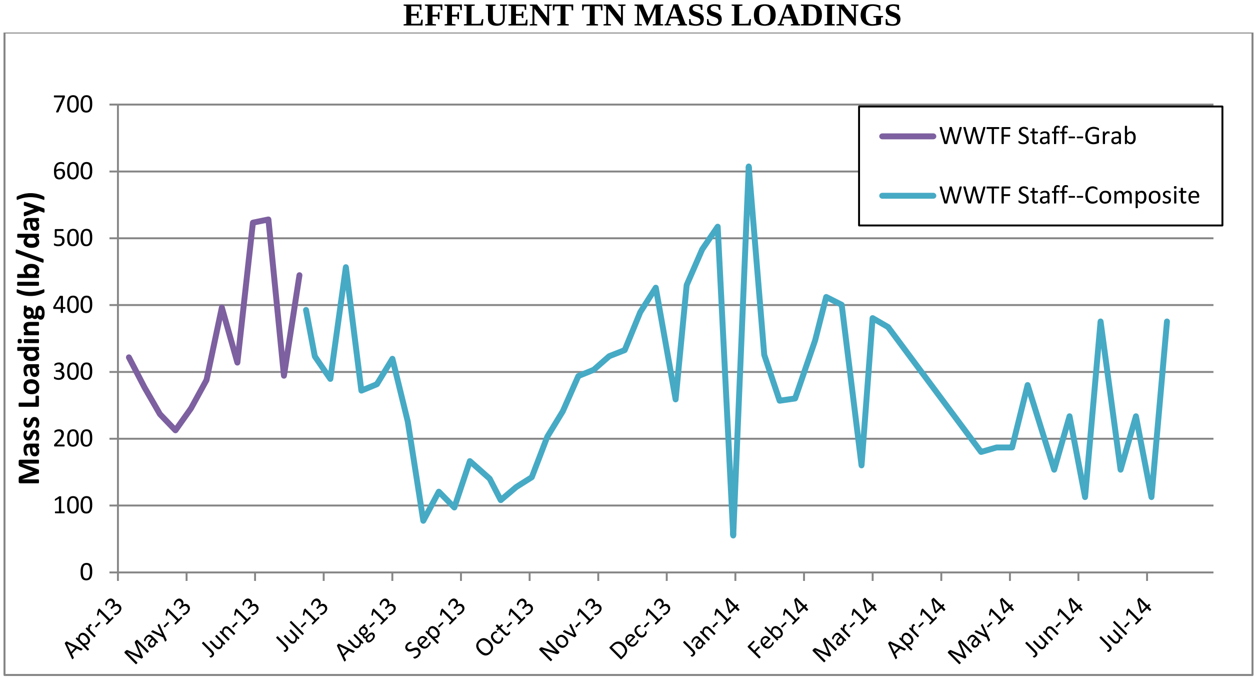


FIGURE 2-17
EFFLUENT TN CONCENTRATIONS FROM VARIOUS DATA SOURCES

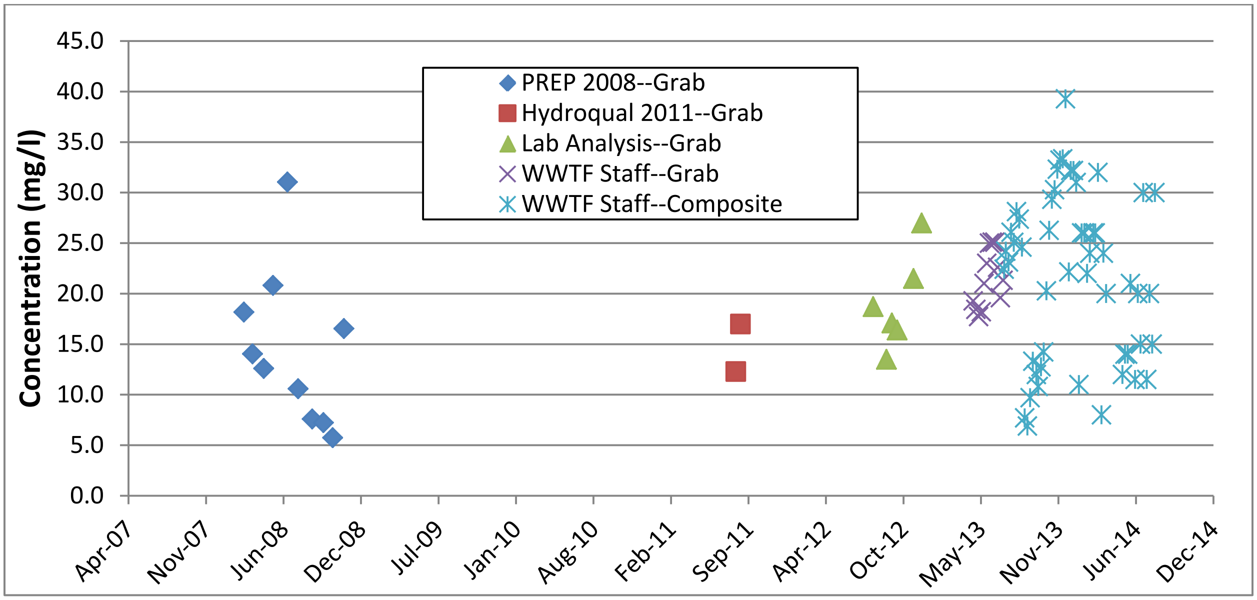
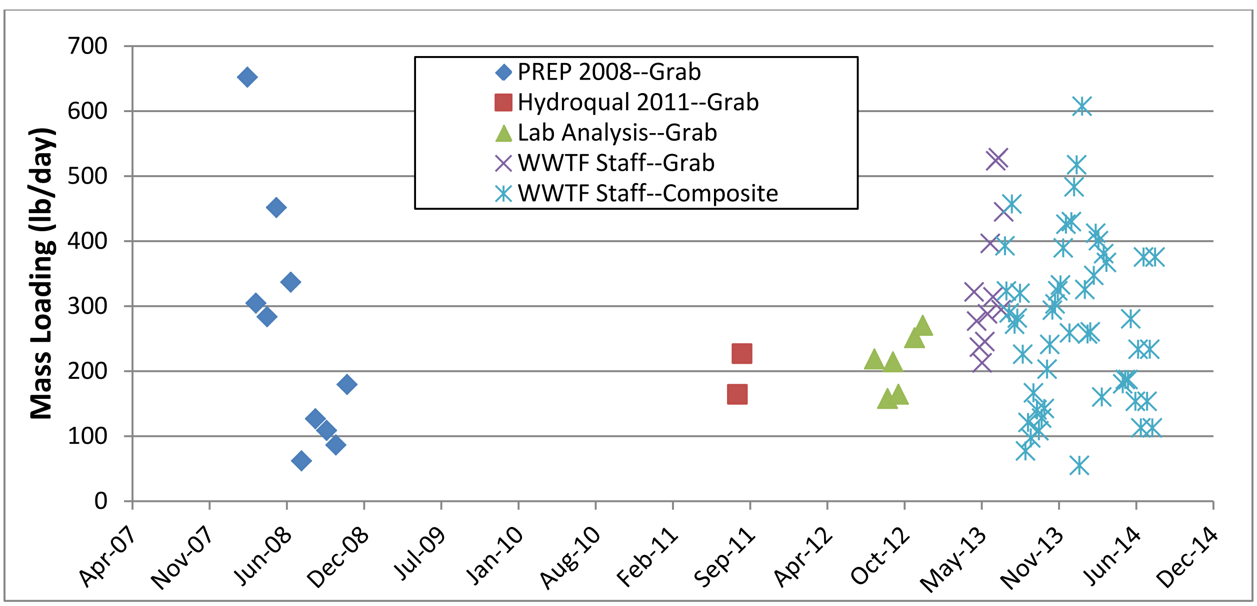


FIGURE 2-18
EFFLUENT TN LOADS FROM VARIOUS DATA SOURCES



2.5 FUTURE WASTEWATER FLOWS AND LOADS

Water resource management planning must consider both the current and future needs which will occur within the planning horizon. Future flows and loadings are a function of residential, commercial and industrial development within the existing sewered area, sewer extensions to existing or future development, redevelopment of existing properties and septage quantities to the WWTF. For the purposes of this study, wastewater volumes have been used as the "measure" of future growth. The estimates of town-wide wastewater flows are presented as annual average daily volumes.

2.5.1 Definition of Terms

"Future" conditions are defined as the conditions that will exist once additional development occurs. For the future conditions, the following terms apply to this discussion:

- *Planning Horizon:* A future population, level of development and an associated wastewater flow that will be the basis for analyzing and designing wastewater infrastructure. The design life of the mechanical components of wastewater facilities is typically 20 years; therefore, including time for planning and construction of recommended measures, a planning horizon should be 25 to 30 years into the future. The planning horizon for this study is 2040.

- *Theoretical Build-Out:* The population and commercial activity associated with the ultimate development to the fullest extent possible under current zoning and other regulation, regardless of economic issues.

- *Total Buildable Area:* The area of a parcel which excludes 100% of all water bodies, 75% of all wetlands and 10% of the total parcel area to account for roads and parking.

2.5.2 Methodology for Development of Future Growth Projections

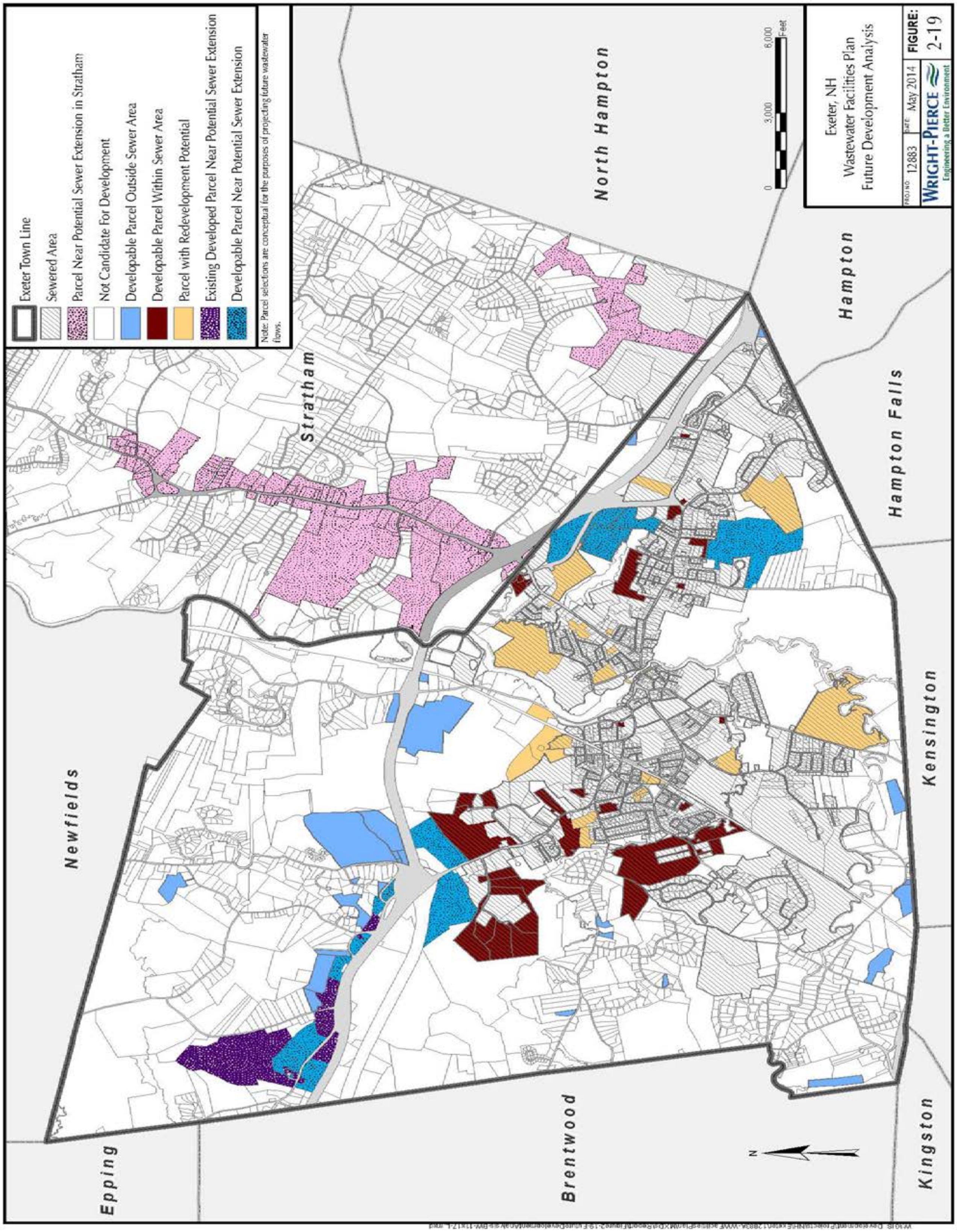
Summarized below is the methodology that was used in the development of future growth projections:

- The Town of Exeter Master Plan (2002 – 2010) was reviewed and analyzed for Town wide trends of development in the residential, commercial and industrial zoning districts.

- A meeting held on February 13, 2014 between Town staff (Jennifer Perry, Michael Jeffers, Matt Berube, Sylvia von Aulock, Doug Eastman) and Wright-Pierce (Ed Leonard, Andy Morrill) to discuss potential development scenarios within the existing sewer area, potential redevelopment scenarios within the existing sewer area as well as possible sewer extensions to serve existing and potential future development. A figure was developed to document the identified parcels. A follow-up meeting held on March 6th, 2014 between Town staff (Sylvia von Aulock, Kristen Murphy) and Wright-Pierce (Ed Leonard, Andy Morrill) to review and adjust the figure. **Figure 2-19** represents a summary of the discussions held during these two meetings.

- The amount of buildable land areas was estimated based on a visual review of the identified parcels on the Town of Exeter MapsOnline interactive website tool and the calculation basis described in **Section 2.5.1**.

- The wastewater generated from the estimated buildable area was estimated by zoning district. This spreadsheet tabulated the identified parcels which had the potential for development or redevelopment in five categories (Developable Parcel within Sewer Area, Parcel with Redevelopment Potential, Existing Developed Parcel near Potential Sewer Extension, Developable Parcel near Potential Sewer Extension and Developable Parcel Outside Sewer Area) and broken out per zoning districts. This information is summarized in **Appendix B**.



Exeter Town Line
Sewered Area
Parcel Near Potential Sewer Extension in Stratham
Not Candidate For Development
Developable Parcel Outside Sewer Area
Developable Parcel Within Sewer Area
Parcel with Redevelopment Potential
Existing Developed Parcel Near Potential Sewer Extension
Developable Parcel Near Potential Sewer Extension
Note: Parcel selections are conceptual for the purposes of projecting future wastewater flows.
Newfields
Stratham
North Hampton
Hampton
Hampton Falls
Kensington
Kingston
Brentwood
Epping
N
0
3,000
6,000
Feet
Exeter, NH
Wastewater Facilities Plan
Future Development Analysis
PROJ NO. 12883
DATE May 2014
FIGURE:
2-19
WRIGHT-PIERCE
Engineering a Better Environment

2.5.3 Residential

The theoretical build-out for residential zones was calculated by dividing the total residential buildable area by the minimum lot size. A wastewater flow allowance of 140 gallons per day per lot was provided. The planning horizon estimated flow was calculated by multiplying the theoretical build-out estimated flow by the probability of occurrence within the planning horizon (set at 50% probability). The rate of 140 gallons per day per lot was based on the Town of Exeter Water Use Data. **Table 2-10** summarizes the potential residential development in each residential zoning district. The development will result in an additional 1,126 people on sewer and an additional 145 people off-sewer.

TABLE 2-10
POTENTIAL RESIDENTIAL DEVELOPMENT

Residential Zoning Districts	Theoretical Buildout New Residential Lots	Theoretical Buildout Estimated Flow (gpd)	Planning Horizon New Residential Lots	Planning Horizon Estimated Flow (gpd)
R-1 (Low Density - Residential)	276	22,064	139	11,032
R-2 (Single Family - Residential)	642	59,192	322	29,596
R-4 (Multi-Family - Residential)	72	2,016	36	1,008
R-5 (Multi-Family District)	28	784	14	392
MS (Manuf. Housing Subdivision)	1	140	1	70
Totals	**1,019**	**84,196**	**512**	**42,098**
Outside the Sewer Area	132		66	
Totals	**1,151**		**578**	

2.5.4 Commercial and Industrial

The theoretical build-out for commercial and industrial zones was calculated by dividing the total commercial and industrial buildable area by the minimum lot size. A wastewater flow allowance of 1,500 gallons per day per buildable acre for commercial parcels and 2,000 gallons per day per buildable acre for industrial parcels was provided. The planning horizon estimated flow was calculated by multiplying the theoretical build-out estimated flow by the probability of occurrence within the planning horizon (set at 50% probability). **Table 2-11** summarizes the potential commercial and industrial development per each zoning district.

TABLE 2-11
POTENTIAL COMMERICAL AND INDUSTRIAL DEVELOPMENT

Commercial Zoning Districts	Total Buildable Area (acres)	Theoretical Buildout Estimated Flow (gpd)	Planning Horizon Estimated Flow (gpd)
C-1 (Central Area Commercial District)	8.30	2,491	1,245
C-2 (Highway – Commercial)	59.25	48,898	24,449
C-3 (Epping Road Highway – Commercial)	102.75	154,120	77,059
CT (Corporate / Technology Park)	60.55	90,868	45,434
CT-1 (Corporate / Technology Park – 1)	85.18	127,769	63,884
NP (Neighborhood Professional)	24.24	36,355	18,177
PP (Professional / Technology Park)	32.48	35,950	17,974
WC (Waterfront Commercial)	0.52	156	78
I (Industrial)	73.28	146,569	73,284
Totals	**446.55**	**643,176**	**321,584**

A vast majority of the commercial development could occur in commercial zoning districts C-3 (Epping Road Highway – Commercial) and CT-1 (Corporate / Technology Park – 1) which are located on both sides of Route 27 / Epping Road just before Exit 9 directly off of Route 101. The industrial zoning district is located east of Route 27 / Epping Road on both sides of Industrial Drive.

2.5.5 Redevelopment of Existing Structures or Parcels

In contrast to development of vacant lots, additional wastewater flows could be generated by the redevelopment of existing structures or parcels to a more intense use. A number of redevelopment possibilities were conceptualized with Town staff; however, none of these are firm development plans. Accordingly, a redevelopment allowance of 20% of existing sanitary flows was used as a placeholder.

2.5.6 Potential Sewer Extensions in Exeter

At present, the Town does not have any plans to extend the sewer area; however, one potential area that has been considered by the Town is the Route 27 corridor (north of Route 101) out to the High School. If a sewer extension was constructed to the High School in the future, some existing and potential development could be served by public sewer. These flows were developed using the methodologies described previously for commercial and residential parcels.

2.5.7 Inflow/Infiltration

The Town has invested considerable effort and funds aimed at reducing inflow/infiltration. The Town has implemented, and will continue to implement, inflow/infiltration removal projects including investigations, sewer and manhole rehabilitation, sewer replacement, sewer service work and storm drain service work, where applicable. For the purposes of this report, future inflow/infiltration is assumed to be held constant through the planning horizon, based on continued investment in the collection system over time. Based on observations of the Exeter WWTF dry weather flows, we estimated the inflow/infiltration to be approximately 700,000 gallons per day.

2.5.8 Septage

As noted previously, Exeter currently generates an estimated 650,000 gallons of septage per year which is generally disposed of at the Hampton WWTF. Based on potential residential development outside of the anticipated sewered area, an estimated 66 to 132 new residential lots would be served by septic systems at the planning horizon and theoretical build-out, respectively. This growth would generate approximately an additional 22,000 to 44,000 gallons of septage per year at the planning horizon and theoretical build-out, respectively.

2.5.9 Stratham

The Town of Stratham has expressed interest in constructing a sewer extension to serve the Route 108 area and connecting that sewer extension to the Town of Exeter wastewater infrastructure. Both Towns have engaged in numerous workshops in an effort to determine if this inter-municipal connection is viable. For the purposes of this study, we have carried 555,000 gallons per day and 660,000 gallons per day as the capacity requested at the planning horizon and at theoretical build-out, respectively. The planning horizon values represent the "Phase I" flow of 165,000 gallons per day and the "Phase II" flow of 390,000 gallons per day. ("Exeter/Stratham Inter-municipal Water and Wastewater Systems Evaluation Study", Kleinfelder, July 2012, Table 3-6).

2.5.10 Newfields

The Town of Newfields currently operates a WWTF with an annual average flow of approximately 50,000 gallons per day and is permitted for a flow of 117,000 gallons per day. At this time, the Town of Newfields has not requested service from the Town of Exeter; however, for the purposes of this study, we have included the Newfields' permitted flows in the future flow projections.

2.5.11 Future Wastewater Flow and Loading Projections

Future wastewater flow projections were developed by multiplying future population projections by current water use rates (for each user category – residential, commercial and industrial/institutional). Future maximum month and maximum day flows were developed by multiplying future annual average flows and current "peaking factors" based on the 2011 to 2013 influent flow data set. Future wastewater flow projections are summarized in **Table 2-12.**

Future annual average wastewater loads were developed by multiplying future wastewater flow projections by current average day wastewater concentrations obtained from the 2010 and 2014 influent characterization programs. Future maximum month and maximum day wastewater loads were calculated by multiplying future annual average loads and current "peaking factors" based on the 2010 and 2014 influent characterization programs. Future wastewater flows and loadings are summarized in **Table 2-13.**

TABLE 2-12
FUTURE WASTEWATER FLOW PROJECTIONS

Category	Current 2013	Future Planning Horizon 2014 to 2040	Future Theoretical Buildout 2040+
Existing Flows (gpd)			
Residential	490,000	-	-
Institutional	100,000	-	-
Commercial/Industrial	330,000	-	-
Sewer Only	80,000	-	-
Inflow/Infiltration	700,000	-	-300,000
Septage	0	-	-
Total – Existing Flows (gpd)	**1,700,000**	**1,700,000**	**1,400,000**
New Flows, within Sewer Area (gpd)	-	432,000	664,000
New Flows, Sewer Extension (gpd)	-	150,000	266,000
Septage (gpd)	-	3,000	3,000
Total – Exeter (gpd)	1,700,000	**2,285,000**	**2,333,000**
New Flows – Other Towns (gpd)	-	605,000	777,000
Total – with Regional (gpd)	1,700,000	**2,890,000**	**3,110,000**

TABLE 2-13
EXISTING AND PROJECTED WASTEWATER FLOWS AND LOADS

	Existing No Septage (Current)	Projected Without Septage (2040)	Projected With Septage (2040)
Flows (MGD)			
Annual Average (Note 3)	1.71*	3.00	3.00
Minimum Month	1.18*	1.60	1.60
Maximum Month	2.88*	5.10	5.10
Maximum Two-Week	3.09*	5.40	5.40
Maximum Day (99.5th Percentile)	3.75*	6.60	6.60
Instantaneous Peak Flow (100th Percentile)	5.65*	9.75	9.75
Biochemical Oxygen Demand (lbs/day)			
Annual Average	2,138*	5,400	5,600
Maximum Month	3,484*	6,800	7,100
Maximum Day	4,210*	7,900	8,200
Total Suspended Solids (lbs/day)			
Annual Average	2,544*	6,000	6,400
Maximum Month	3,632*	10,500	11,200
Maximum Day	4,376*	12,600	13,400
Ammonia-Nitrogen (lbs/day)			
Annual Average	265**	550	570
Maximum Month	320**	660	680
Maximum Day	360**	750	780
Total Kjeldahl Nitrogen (lbs/day)			
Annual Average	306**	690	710
Maximum Month	320**	910	940
Maximum Day	480**	1090	1120
Total Phosphorus (lbs/day)			
Annual Average	45**	110	120
Maximum Month	57**	140	150
Maximum Day	77**	190	210

Notes:
1) "*" denotes measured data for 2011 to 2013.
2) "**" denotes measured data for 2010 and 2014 only, limited data set.
3) Existing and projected conditions exclude on-site recycle flows & loads
4) Existing permitted flow and design flow is 3.0-mgd.
5) Future peak flows to WWTF will be increased in order to reduce or eliminate CSO activity in the collection system.

2.6 EFFLUENT STANDARDS

2.6.1 NPDES Permit and Administrative Order on Consent

The effluent discharge must meet standards set forth in state and federal water quality legislation. These standards establish minimum effluent discharge requirements which must be satisfied at all times. In accordance with Section 402 of the Clean Water Act, the plant's effluent quality requirements are contained in a National Pollutant Discharge Elimination System (NPDES) permit which is issued to the Town by the Environmental Protection Agency (EPA). A copy of the current NPDES permit (Permit No. NH0100871, issued December 2012) and related correspondence is contained in **Appendix A**.

The existing WWTF was not designed to remove nitrogen from wastewater and, therefore, cannot meet the NPDES permit requirements. Accordingly, EPA issued Administrative Order on Consent (AOC) Docket No. 13-010. A copy of the AOC is also included in **Appendix A.** The AOC provides the Town with an interim effluent Total Nitrogen limit of 8.0 mg/l and provides a compliance schedule to achieve numerous specific tasks, as summarized below:

- <u>June 30, 2016</u>: Initiate construction of the WWTF upgrade.
- <u>June 30, 2018</u>: Achieve substantial completion of the WWTF upgrade.
- <u>June 30, 2019</u>: Meet the interim WWTF effluent limit of 8 mg/l Total Nitrogen.
- <u>September 30, 2018:</u> Submit a "Nitrogen Control Plan" for implementing specific control measures for non-point source (NPS) and stormwater nitrogen loadings to the Great Bay Estuary (including Squamscott River) within the Town. The plan shall include a 5 year schedule for implementing the control measures.
- <u>December 31, 2023:</u> Submit an engineering evaluation with recommendations to achieve the NPDES TN discharge requirement of 3 mg/l or a justification for leaving the interim limit of 8 mg/l.
- <u>Annually (beginning January 2014):</u> Submit Total Nitrogen Control Plan Progress Reports to EPA and NHDES. The reports must include the following descriptions with sufficient information such that changes to Nitrogen loads within the watershed can be associated with individual sources of nitrogen. The required descriptions include: the pounds of Total

Nitrogen (TN) discharged from the WWTF during the previous calendar year; a description of the WWTF operational changes that were implemented during the previous calendar year; the status of the development of a TN NPS and stormwater point source accounting system; the status of the development of the NPS and stormwater point source Nitrogen Control Plan; a description and accounting of the activities conducted by the Town as part of its Nitrogen Control Plan; a description of all activities within the Town during the previous year that affect nitrogen loading to the Great Bay Estuary.

- <u>On-going:</u> Take action to reduce NPS and stormwater sources of total nitrogen to the Great Bay, including:
  - Track all activities within the Town that affect TN including new/modified septic systems, decentralized WWTFs, changes to impervious cover, and any new or modified BMPs.
  - Develop and utilize a comprehensive subwatershed-based tracking/accounting system for quantifying the TN loading changes associated with Town activities.
  - Develop a subwatershed community-based TN allocation, in coordination with NHDES.

2.6.2 Receiving Water Quality

The WWTF discharges into the Squamscott River, upstream of the Great Bay estuary. The Squamscott River is a Class B waterway, as designated by the New Hampshire Department of Environmental Services (NHDES). The NPDES permit provides for a dilution factor of 25.2:1 for the WWTF effluent discharge to the Squamscott River.

2.6.3 Current NPDES Effluent Limitations

The NPDES permit limits for the WWTF effluent (Outfall #001 to the Squamscott River) are summarized in **Table 2-14**. The mass limits for the WWTF are based on a design flow of 3.0-mgd. The NPDES permit limits for the permitted CSO (Outfall #003 to Clemson Pond) are summarized in **Table 2-15.**

TABLE 2-14
NPDES EFFLUENT LIMITS FOR WWTF

Parameter	Monthly Average	Weekly Average	Daily Maximum
Flow, mgd	Report	—	Report
BOD_5, mg/l	30	45	50
TSS, mg/l	30	45	50
pH, Std. Units	6.0-9.0	6.0-9.0	6.0-9.0
Fecal Coliform, #/100 mL	14	—	Report
Fecal Coliform, %	—	—	Report
Enterococci, #/100Ml	Report	—	Report
Total Residual Chorine, mg/L	0.19	—	0.33
Total Nitrogen, mg/l November 1 to March 31	Report	—	—
Total Nitrogen, mg/l (lb/d) April 1 to October 31, seasonal rolling average	3.0 (75)	—	—
Whole Effluent Toxicity - LC50; % effluent	—	—	100
Total Recoverable Metals, mg/L Aluminum, Cadmium, Chromium, Copper Nickel, Lead, Zinc	Report	Report	Report
Ammonia Nitrogen as N, mg/L	Report	Report	Report

Note:
1) The AOC requirement is for 8.0 mg/l effluent total nitrogen, from April 1 to October 31, seasonal rolling average.
2) The AOC states that supplemental carbon is <u>not required</u> at any time during the year.

TABLE 2-15
NPDES EFFLUENT LIMITS FOR CSO #003

Parameter	Each CSO Event
Volume	Report
Escherichia Coli, #/100 mL	1,000
Duration	Report
1-hr and 24-hr rain gauge data (in.)	Report

2.6.4 Groundwater Discharge Permit

The existing WWTF lagoons do not have impermeable liners. The NHDES recently issued the Town a Groundwater Discharge Permit to monitor the groundwater quality proximate to the lagoons (Permit No. GWP-198401079-E-001, issued January 2012). A copy of the Groundwater Discharge Permit is included in **Appendix A**. The sampling and monitoring requirements contained in the permit are summarized in **Table 2-16**.

TABLE 2-16
GROUNDWATER DISCHARGE PERMIT MONITORING REQUIREMENTS

Parameter	Sampling/Monitoring Frequency
WWTF Effluent Flow, mgd	Weekly
pH, Std. Units*	May and November, each year
Escherichia Coli, #/100 mL	May and November, each year
Arsenic, Boron, Chloride, Nitrate, Total Kjeldahl Nitrogen, Total Phosphorus	May and November, each year
Static Water Level (ft)	May and November, each year
Water Temperature	May and November, each year
Drinking Water Metals and VOCs by EPA 8260B (including 1,4-Dioxane)	November 2014, May 2017

2.6.5 Anticipated Future Effluent Limitations

The current NPDES permit and AOC are focused primarily on addressing concerns related to effluent total nitrogen. Over time, the Town may face more stringent effluent limits for other parameters. Each of these potential areas are described below.

2.6.6 Phosphorus

The WWTF discharges into a tidally-influenced and brackish section (<10 ppt, HydroQual, August 2011 data) of the Squamscott River. Given the location of the discharge (i.e., upgradient of an estuary), it is unlikely that phosphorus limits would be imposed on the WWTF in the near-term. However, as effluent nitrogen concentrations are substantially reduced in the near term, the regulators will look at in-stream nitrogen and phosphorus ratios to confirm that the nutrient values do not cause or contribute to water quality problems. It is appropriate to consider the implications of future phosphorus removal requirements in this planning effort.

2.6.7 Ammonia and Metals

The WWTF has a dilution factor of 25.2:1. This is a modest dilution factor which could result in future metals limits being imposed.

2.6.8 Compounds of Emerging Concern

Compounds of emerging concern (CECs) encompass a wide variety of compounds including endocrine disrupting compounds, pharmaceuticals, flame retardants, hormones, industrial solvents and surfactants, metals, pesticides, and personal care products. CECs have been found in wastewater for decades; however, they have recently reached the forefront of regulatory and public concern, and there is currently a great deal of research on CECs. One of the difficulties associated with addressing this topic is the large number and wide array of substances that can be classified as CECs. EPA and NHDES have not established effluent standards for CECs to date, and have not indicated any intention to regulate CECs in the near term.

Processes utilized at typical secondary wastewater treatment facilities provide for some CEC removal based on sorption and biodegradation. Longer solids retention time systems are more likely to remove more CECs. The technologies more frequently referenced for potential supplemental removal of CECs include reverse osmosis, adsorption (granular activated carbon, ion exchange), ultraviolet/peroxide; ozone; and coagulation/flocculation. Reverse osmosis is generally considered the most effective (and expensive) approach for a broad range of CECs

2.6.9 Staffing/License Classifications

The NPDES permit requires that the existing WWTF be operated by a Grade II operation, minimum. The WWTF is currently staffed by one Grade II operator, one Grade III operator and one full-time equivalent maintenance technician. Depending on the processes selected, the future WWTF may require a higher operator grade and may require additional staff.

Section 3

WRIGHT-PIERCE
Engineering a Better Environment

SECTION 3

EVALUATION OF EXISTING PROCESS SYSTEMS

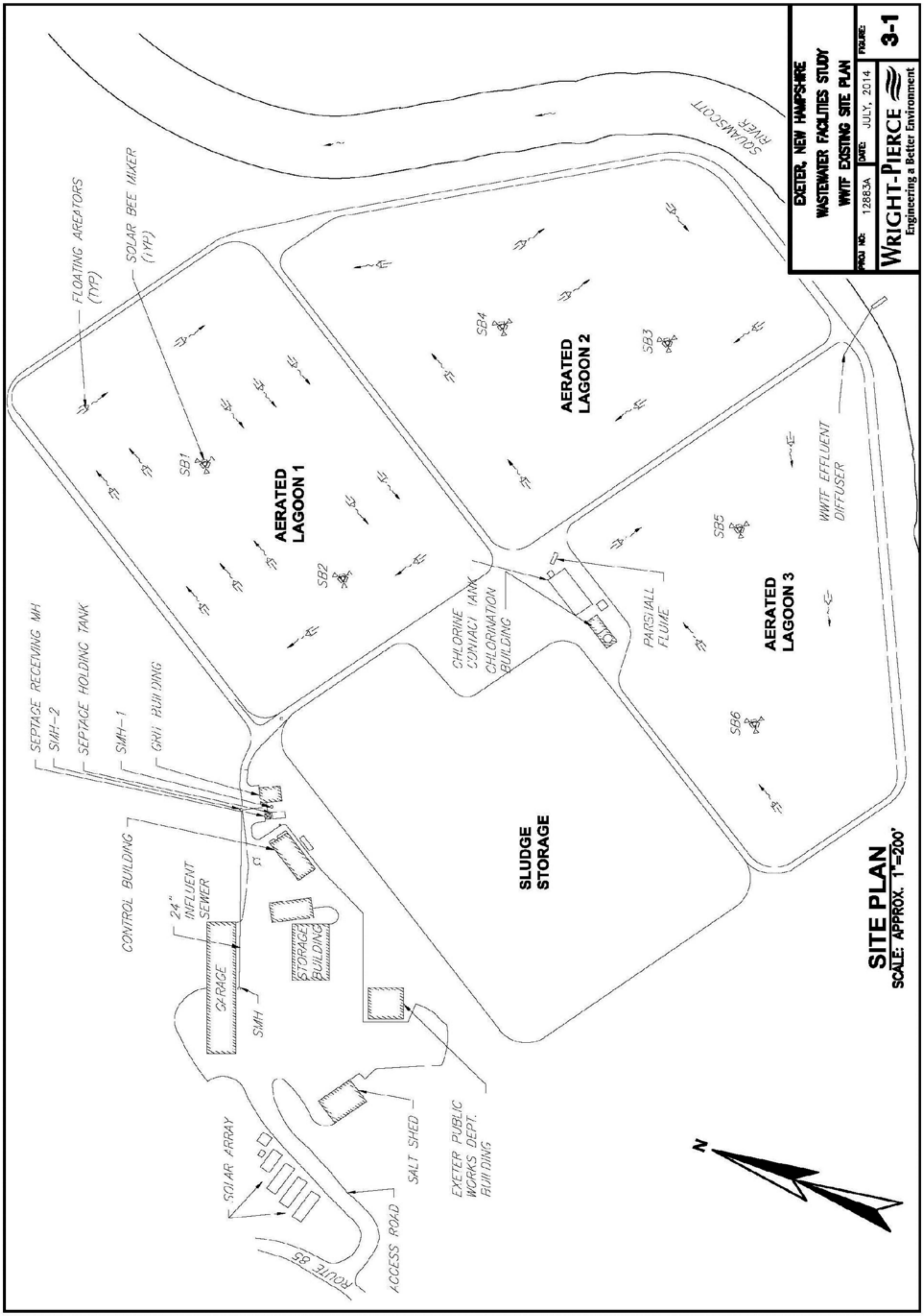
3.1 BACKGROUND

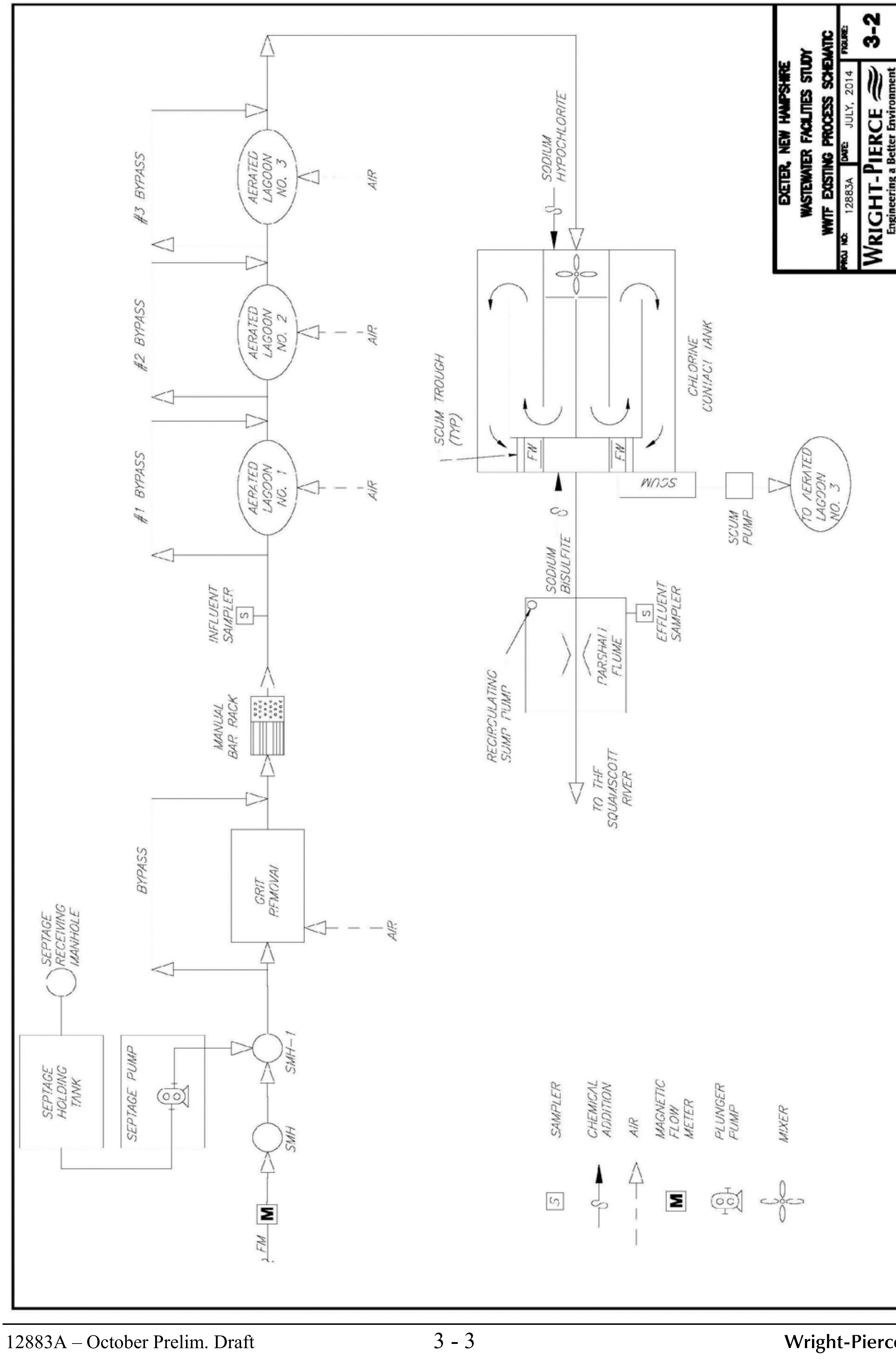
The purpose of this section of the report is to present background information on each unit process at the wastewater treatment facility (WWTF) and recommended improvements to individual unit processes. Each of these unit processes is discussed in greater detail below. The WWTF existing site plan is shown in **Figure 3-1**. The existing site process schematic is shown in **Figure 3-2.**

The Exeter WWTF consists of the following treatment processes:

- Main Pump Station and Forcemain
- Influent Flow Metering
- Septage Receiving
- Preliminary Treatment
- Secondary Treatment
- Disinfection
- Effluent Outfall
- Plant Wide Support Systems
- Biosolids Handling

In some cases, the recommended improvements presented herein are independent of the improvements which will be needed for advanced nutrient removal at the facility. Alternatives for WWTF upgrades are presented in **Section 5**.


EXETER, NEW HAMPSHIRE
WASTEWATER FACILITIES STUDY
WWTF EXISTING SITE PLAN
PROJ NO: 12883A
DATE: JULY, 2014
FIGURE: 3-1
WRIGHT-PIERCE
Engineering a Better Environment
SQUAMSCOTT RIVER
FLOATING AREATORS (TYP)
SOLAR BEE MIXER (TYP)
AERATED LAGOON 1
AERATED LAGOON 2
AERATED LAGOON 3
SB1
SB2
SB3
SB4
SB5
SB6
WWTF EFFLUENT DIFFUSER
PARSHALL FLUME
CHLORINE CONTACT TANK
CHLORINATION BUILDING
SEPTAGE RECEIVING MH
SMH-2
SEPTAGE HOLDING TANK
SMH-1
GRIT BUILDING
CONTROL BUILDING
24" INFLUENT SEWER
GARAGE
SMH
STORAGE BUILDING
SLUDGE STORAGE
SALT SHED
EXETER PUBLIC WORKS DEPT. BUILDING
SOLAR ARRAY
ACCESS ROAD
ROUTE 85
N
SITE PLAN
SCALE: APPROX. 1"=200'


EXETER, NEW HAMPSHIRE
WASTEWATER FACILITIES STUDY
WWTF EXISTING PROCESS SCHEMATIC
PROJ NO: 12883A
DATE: JULY, 2014
FIGURE: 3-2
WRIGHT-PIERCE
Engineering a Better Environment
SEPTAGE HOLDING TANK
SEPTAGE PUMP
SEPTAGE RECEIVING MANHOLE
FM
SMH
SMH-1
BYPASS
GRIT REMOVAL
AIR
MANUAL BAR RACK
INFLUENT SAMPLER
#1 BYPASS
#2 BYPASS
#3 BYPASS
AERATED LAGOON NO. 1
AERATED LAGOON NO. 2
AERATED LAGOON NO. 3
SODIUM HYPOCHLORITE
SCUM TROUGH (TYP)
FW
CHLORINE CONTACT TANK
SCUM
SCUM PUMP
TO AERATED LAGOON NO. 3
SODIUM BISULFITE
RECIRCULATING SUMP PUMP
PARSHALL FLUME
EFFLUENT SAMPLER
TO THE SQUAMSCOTT RIVER
SAMPLER
CHEMICAL ADDITION
MAGNETIC FLOW METER
PLUNGER PUMP
MIXER

3.2 MAIN PUMP STATION AND FORCEMAIN

The Main Pump Station and forcemain were constructed in 1964 and are located just off Swasey Parkway in downtown Exeter. The forcemain conveys all of Exeter's wastewater flow from the Main Pump Station to the Exeter Wastewater Treatment Facility (WWTF) on Newfields Road.

3.2.1 Main Pump Station

The Main Pump Station is a dry-pit, wetwell configuration with three vertical close coupled centrifugal pumps. In 1996 all three pumps were upgraded to 75-horsepower with variable frequency drives increasing the design capacity to 5.0 MGD. The pumps are operated in a lead-lag-standby configuration and each pump is alternated on a weekly basis. Presently, the pump station is capable of conveying approximately 4.4 MGD at 72 feet total dynamic head to the Exeter WWTF. The pump station was originally constructed with grit removal, which consisted of a grit collection sump, grit pump and classifier, but due to regular clogging of the classifier it was removed in the mid-1980's. Presently grit still collects in the grit collection sump and is removed monthly or when levels become problematic.

Wastewater enters the Main Pump Station through a 24-inch diameter influent sewer pipe where it is directed to two influent channels. Each influent channel has a grinder that operates continuously. Wet well level is monitored and controlled by an ultrasonic level sensor and has a float system as backup. Seal water to each pumps' split face mechanical seal is supplied by Town water. Each pump discharge has a strap-on type flow meter. A 200-kilowatt emergency generator serves the entire Main Pump Station and was installed in March 1999.

The mechanical, instrumentation and electrical components in the Main Pump Station have reached the end of their useful life and should be overhauled with any future upgrades to the facility. The Main Pump Station pumping capacity should be comprehensively upgraded to convey the peak flows so that CSO events can be avoided. The generator should be maintained for continued use.

3.2.2 Forcemain

The Main Pump Station forcemain is a 16-inch diameter cement-lined cast iron forcemain that is approximately 4,900 linear feet long. A portion of the forcemain was inspected by Wright-Pierce in August 2010, in the vicinity of the new flow meter and the forcemain invert was found to show considerable wear of the cement lining as well as the invert of the cast iron pipe (approximately 78% remaining). Forcemain velocities should be maintained at or above 2.0 ft/sec to ensure that solids do not collect in the forcemain, which would decrease the pumping capacities. During normal flow conditions, the velocity in the forcemain is approximately 3.4 ft/sec; during high flow conditions, the velocity in the forcemain is approximately 7.5 ft/sec. Due to the critical nature of this forcemain, it is recommended that the forcemain be rehabilitated or replaced within 5 to 10 years. Several options are listed below:

1. *Sliplining* the existing forcemain is a trenchless technology with minimal excavation, but would not allow for increasing the forcemain diameter/capacity and would require bypass pumping.
2. *Pipe bursting* the existing forcemain is another trenchless technology with minimal excavation that would allow for a modest upsizing of the forcemain for increased capacity and would require bypass pumping.
3. *Open cut replacement* of the existing forcemain would allow for upsizing the forcemain for additional capacity but would require bypass pumping and excavation along the entire route.
4. *Open cut construction of a seasonal parallel forcemain* would allow for upsizing the forcemain for additional capacity and would dramatically reduce the time bypass pumping would be needed but would require excavation along the entire route and may require modifications to existing easements if the forcemain crosses private property.

A combination of Option 1 and Option 4 is recommended.

The WWTF is not currently served by public water. A new 8-inch or 12-inch diameter ductile iron water main should be installed from the intersection of Water Street/Summer Street (approx. 5,000 feet) to provide potable water and fire protection to the WWTF and the Public Works Complex.

3.3 INFLUENT FLOW METERING

The influent flow meter vault was installed in August 2010 just off Newfields Road to the left of the entrance driveway to the Public Works Complex. It consists of an 8-foot diameter precast structure where a 16-inch diameter magnetic flow meter is housed. The influent flow meter isolation gate valves are located a few feet outside of the structure to provide upstream and downstream isolation. An offset 12-inch diameter bypass line was also installed and consists of two 12-inch diameter live-tapping tees and a 12-inch diameter forcemain with isolation valves. The influent flow meter is calibrated annually by A&D Instruments. In June 2014, the influent flow meter radio telemetry was upgraded by A&D Instruments and provides accurate influent flow data to SCADA. From August 2010 to June 2014, the WWTF operator needed to manually record the totalizer reading from the local panel because the value sent to SCADA was not accurate. No additional modifications are anticipated.

3.4 SEPTAGE RECEIVING

The Septage Receiving Facility was constructed during the 1988 upgrade and is located between the Control Building and Grit Building. Septage is discharged from the truck into the septage dumping manhole where it flows by gravity into the Septage Holding Tank (approximately 10,500 gallon capacity). Septage is then conveyed through an inline commuter and one of two 7.5-hp plunger pumps, located in the basement of the Control Building, before being discharged in to SMH-1. Flow is measured through the use of a cycle counter on each pump, where each piston cycle is counted and then multiplied by the volume of the cylinder to calculate total flow.

The Exeter WWTF has never received septage since the administrative protocol to do so was never developed. Septage represents a source of revenue and should be considered in the WWTF upgrade plans. If septage will be received, the existing system should be upgraded including the addition of mechanical fine screening and flow metering. The existing septage holding tanks should receive concrete repairs.

3.5 PRELIMINARY TREATMENT

The Grit Building houses the preliminary treatment equipment which was constructed during the 1988 upgrade and is located northeast of the Septage Receiving Facility. Flow enters the Grit Building from SMH-1 on the east side of the building via a 24-inch diameter ductile iron sewer pipe. Flow is then conveyed through the manual bar rack and aerated grit chamber before exiting the building on the northeast corner via a 24-inch diameter ductile iron pipe.

3.5.1 Screening/Manual Bar Rack

Influent screening is achieved by the one coarse manual bar rack (1-inch spacing). Screenings are periodically manually raked by an operator and then placed in a five gallon bucket which is transferred into a hopper that is dumped into the storage container located east of the storage lagoon. The storage container holds all of the screenings, grit, spoils from cleaning pump station wet wells and sewer main construction debris. The contents of this container are periodically disposed of offsite. In 2012 and 2013, 12.5 tons and 16.5 tons of material, respectively, were disposed of at the Turnkey Landfill in Rochester, NH. The influent screenings should be upgraded with the addition of a new mechanical fine screen (1/4-inch to 3/8-inch spacing) with a screenings wash press and the coarse manual bar rack (1-inch spacing) should be replaced.

3.5.2 Grit Removal

After exiting the bar rack, wastewater flows to the aerated grit chamber, which is approximately 15.2-feet wide by 15.0-feet long by 13.1-feet deep and a volume of approximately 22,200 gallons. Per NHDES regulations and TR-16, ideal aerated grit chamber geometry has a length to width ratio of 3:1 to 8:1 and a width to depth ratio of 0.89:1. The existing aerated grit chamber has a length to width ratio of 1:1 and a width to depth ratio of 1.15:1. At the peak hourly flow rate, the detention time through the grit chamber is approximately 5.3-minutes, which is just outside the design standard of 2 to 5 minutes of detention time. The grit chamber is aerated by a series of coarse bubble diffusers, replaced in 2012, which are served from a 4-inch diameter air header. The air header is fed from two 5-hp positive displacement lobe blowers that are located in the basement of the Control Building. The aeration in the chamber creates a spiral roll pattern which promotes the grit to separate from organic matter and settle out at the bottom of the tank. A 12-inch diameter 15-foot long screw conveyor then collects the settled grit and conveys it to the grit sump where it is picked up by the elevator chain and bucket system. The buckets discharge the grit into the dewatering screw where the separated grit is deposited into a roll-off container for disposal and the organics are drained back into the grit chamber.

The existing aerated grit chamber does not conform to current design standards and all of the grit removal system equipment has reached the end of its' useful life. If the WWTF upgrades allow for the same hydraulic gradeline, the grit removal system could be upgraded to minimize cost. However, the grit removal efficiency could be improved with an upgraded configuration.

3.5.3 Influent Sampling

The influent composite sampler was recently installed in January 2014. It is located on the east side of the Grit Building in a prefabricated enclosure. The influent samples are taken from the effluent channel of the Grit Building just downstream of the manual bar rack. As of June 2014, the influent samples are flow paced composite samples. The influent sampler should be maintained for continued use.

3.6 SECONDARY TREATMENT

Secondary treatment is accomplished through the aerated lagoon system. Specific details concerning each component are presented below.

3.6.1 Aerated Lagoons

Three aerated lagoons are located behind the Control and Grit Buildings and were re-graded and re-configured during the 1988 upgrade. **Table 3-1** above summarizes key dimensional data associated with the aerated lagoons.

TABLE 3-1
AERATED LAGOON DATA

Dimensions		Lagoon No.1	Lagoon No.2	Lagoon No.3
Volume at Average Design Flow (MG)		26.0	27.0	23.4
Water Surface Area (acres)		9.01	9.30	8.22
Water Surface Elevation (ft)	Average Design Flow	25.40	16.27	15.28
	Peak Design Flow	25.60	16.50	15.72
Maximum Depth (ft)[1]		9.6	10.5	9.7
Bottom Elevation (ft)		16.0	6.0	6.0
Freeboard (ft)		2.4	1.5	2.3

Note: 1. Maximum depth calculated at Peak Design Flow.

All lagoon piping consists of 24-inch diameter ductile iron pipe, except for the outlet piping for Lagoon No. 3 which consists of 30-inch diameter ductile iron pipe. During normal flow conditions, flow goes from Lagoon No. 1, through Lagoon No. 2, through Lagoon No. 3 and then to disinfection. During high flow conditions Lagoon No. 1 and No. 2 have a bypass outlet structure to avoid overtopping of the embankments. Lagoon No. 1 utilizes fourteen 15-hp floating aerators, Lagoon No. 2 utilizes eight 10-hp floating aerators and Lagoon No. 3 utilizes five 7.5-hp floating aerators. The floating aerators in Lagoon No. 1 and No. 2 were replaced in 1995, while the floating aerators in Lagoon No. 3 are original. Each lagoon is also equipped with two solar powered 0.5-hp SolarBee circulators (six total) which were installed in 2000. Although the lagoons have never been drained, dewatering sumps exist to gravity drain the lagoons for routine maintenance. Lagoon No. 2 dewatering sump is presently inoperable due to the riser section having tipped over during a winter freeze and thaw cycle.

Algae blooms typically occur in both the spring and fall in Lagoons No. 2 and No. 3 but rarely in Lagoon No. 1. The Exeter WWTF has had six violations for TSS due to algae since 1989. When NHDES was consulted for solutions to the TSS violations due to algae, they suggested introducing daphnia into the lagoons. Since the NHDES recommendation has been implemented, there has been a noticeable decrease in algae and TSS violations.

The existing lagoons cannot be configured to reliably achieve the nitrogen removal requirements identified in the NPDES permit or the AOC (due to lower levels and specific calendar year time frames). The lagoons will need to be replaced by an activated sludge treatment system to meet these specified limits and timeframes.

3.7 DISINFECTION

Disinfection is the final treatment process and provides the means for removal of pathogens prior to discharge to the Squamscott River. Disinfection is accomplished in the Chlorine Contact Tanks which are located at the northwest corner of Lagoon No. 3 and were constructed during the 1988 upgrade.

3.7.1 Chlorine Contact Tank

The Chlorine Contact Tank is a “three-pass” serpentine channel configuration. Under normal flow conditions chlorinated wastewater is conveyed to one of two “three-pass” serpentine channels after passing through its respective slide gate. During peak flow conditions both “three-pass” serpentine channels are placed into service and are able to properly disinfect with no known issues. Each serpentine channel is approximately 233.5-feet long, 5.0-feet wide, with a maximum water depth of approximately 9.4-feet and has a volume of approximately 82,000 gallons (164,000 gallons total). Each chlorine contact train is equipped with a gutter drain that leads to a sump to facilitate draining the tanks for maintenance; however this drain system is not currently operational. Each Chlorine Contact Tank can be pumped down to Lagoon No. 3 for maintenance using a pump powered from the closest aerator in Lagoon 3 which is controlled through SCADA. There is a scum trough at the end of the last pass channel. The Chlorine Contact Tank has numerous cracks located throughout the tanks and should be inspected for structural damage.

Wastewater enters the Chlorine Contact Tank via a 4,000 gallon± mixing chamber where sodium hypochlorite is mixed using a 5-hp single speed mixer. The mixer operates continuously and the motor and gears have been replaced. As chlorinated wastewater passes over the effluent weir it enters a 3,000 gallon± mixing chamber; however, the Town removed the dechlorination mixer at some point in the past. Sodium bisulfite is now mixed via turbulence in the mixing chamber and a sump pump in the entrance of the effluent Parshall Flume.

At the design peak hourly flow rate, the contact time is approximately 26 minutes, which meets the NHDES design standard of 15 minutes at peak flow. Since there has been a good compliance record associated with disinfection, the Chlorine Contact Tank could be repaired and maintained for continued use.

3.7.2 Chlorination System

Sodium hypochlorite is added to the mixing tank through a 1.5-inch diameter CPVC pipe that is fed by three metering pumps located in the Chlorination Building. Process water can be added as carrier water if needed. The three sodium hypochlorite metering pumps are paced off influent flow through SCADA. Since the chlorine residual samples are taken from the end of the "second-pass" serpentine channel, the chlorine residual results are not used to trim the pacing of the sodium hypochlorite metering pumps. Seasonally the sodium hypochlorite metering pumps' strokes are adjusted by the operators based on operational experience. The sodium hypochlorite metering pumps are fed from a pumped loop system which is supplied from one of two 1/2-hp sodium hypochlorite recirculation pumps that take suction from and discharge back to a 1,000 gallon day tank located in the Control Building. Weekly the operators alternate the sodium hypochlorite recirculation pumps and cleanout the offline Y-strainer. The sodium hypochlorite pumped loop system has had two leaks since coming online in 1988 with the last incidence occurring in January 2014 just behind the Control Building. The day tank is filled by two sodium hypochlorite 1-hp transfer pumps which take suction from one of two 2,000 gallon bulk storage tanks. During normal operation, approximately 100 gallons of sodium hypochlorite (12.5% concentration) and 500 gallons of process water are used to fill the 1,000 gallon day tank (2.0% concentration) each week. However, during times of partial nitrification sodium hypochlorite use can be upwards of 400 gallons per day at which time Lagoon No. 3 is taken offline and the discharge from Lagoon No. 2 is directed to the Chlorine Contact Tank. The 1,000 gallon day tank was installed during 1988 upgrade, the 2,000 gallon bulk storage tank No. 1 was replaced in 2013 and the 2,000 gallon bulk storage tank No. 2 was replaced in approximately 2002. Each sodium hypochlorite tank is equipped with an ultrasonic level probe which is connected to SCADA and provides a low and high level alarm. The sodium hypochlorite feed pumps, 1,000 gallon day tank, transfer pumps, and both 2,000 gallon bulk storage tanks are all located in the Control Building.

All components of the chlorination system have reached the end of their useful life and should be replaced with any future upgrades to the facility.

3.7.3 Dechlorination System

Sodium bisulfite is added to the mixing tank through a 1.5-inch diameter CPVC pipe that is fed by two sodium bisulfite metering pumps located in the Chlorination Building. Process water can be added as carrier water if needed. Mixing in the sodium bisulfite mixing tank is accomplished through a submerged sump pump that locally recirculates the wastewater. The two sodium bisulfite metering pumps are paced off influent flow through SCADA and trimmed using the chlorine residual analyzer results. The chlorine residual samples are taken from the "second-pass" of the serpentine channel. Seasonally the sodium bisulfite metering pumps' strokes are adjusted by the operators based on operational experience. The sodium bisulfite metering pumps are fed from a pumped loop system which is supplied from one of two 1/2-hp sodium bisulfite recirculation pumps that take suction from and discharge back to a 1,000 gallon day tank. Weekly the operators alternate the sodium bisulfite recirculation pumps and cleanout the offline Y-strainer. The sodium bisulfite loop system has never had a leak since coming online in 1988. The day tank is filled by a 1-hp sodium bisulfite transfer pump that takes suction from the 4,000 gallon sodium bisulfite bulk storage tank. During normal operation, approximately 42 gallons of sodium bisulfite (38% concentration) and 600 gallons of process water are used to fill the 1,000 gallon day tank (2.5% concentration) each week. The 1,000 gallon day tank was installed during 1988 upgrade and the 4,000 gallon bulk storage tank was replaced in approximately 2006. The 1,000 gallon day tank and 4,000 gallon bulk storage tank is equipped with an ultrasonic level probe which is connected to SCADA and provides a low and high level alarm. The room which stores both sodium bisulfite tanks has a low room temperature alarm which is connected to SCADA. During normal operation in the winter months the chlorine residual is between 0.6 and 0.8 mg/L while in the summer months the chlorine residual is between 1.0 and 1.5 mg/L. The sodium bisulfite feed pumps, day tank, transfer pump, 1,000 gallon day tank and 4,000 gallon bulk storage tank are all located in the Control Building.

All components of the dechlorination system have reached the end of their useful life and will need to be replaced with any future upgrades to the facility.

3.7.4 Effluent Flow Measurement

Effluent flow measurement is accomplished through the 18-inch wide Parshall Flume located northeast of the Chlorine Contact Tank and was constructed as part of the 1988 upgrade. After wastewater flow leaves the dechlorination mixing tank via a 30-inch diameter ductile iron pipe it is conveyed in to the Parshall Flume. The depth of wastewater over the flume is measured by an ultrasonic sensor and then the depth measurement is converted into a corresponding flow rate. The ultrasonic sensor was replaced in approximately 2009.

The Parshall Flume insert has been compromised due to water infiltration between the fiberglass flume insert and the concrete that houses it. Due to freeze and thaw action, the throat of the flume has been restricted at the entrance to 17.25 inches wide and 16.75 inches wide at the exit. As a cross-check, the depth at the ultrasonic level sensor was measured at 1.05 feet which correspond to a flow of 4.18 MGD on the 18-inch Parshall Flume discharge table. The corresponding flow reading was recorded at 4.10 MGD, which is a difference of 0.08 MGD or approximately 1.9% difference. The Chief Operator indicated that Environmental Instrument Services (EIS) or A&D Instruments had adjusted the effluent flow signal to account for the restriction. However, when EIS and A&D Instruments were contacted in April 2014, they had no record or recollection of making any adjustments.

The 18-inch wide Parshall Flume is appropriately sized for the design flow rate of the WWTF; however, due to the damage to the throat of the Parshall Flume and possibility of further damage over time it is recommended to replace the 18-inch wide Parshall Flume fiberglass insert and grout fillet at a minimum with any future upgrades to the facility.

3.7.5 Effluent Sampling

The effluent sampler was installed in 2009 and is located on the north side of the Parshall Flume in a prefabricated enclosure. Effluent composite samples are automatically collected in the Parshall Flume before the ultrasonic sensor. The samples are time-paced, 24-hour composite samples. The effluent sampler is in good condition and should be calibrated and maintained for continued use. The sampler should be converted to flow-paced composite sampling as a part of any future upgrade.

3.8 EFFLUENT OUTFALL

The extended effluent outfall was constructed during the 2002 upgrade and is located in the Squamscott River, east of Lagoon No. 2 and just downstream of the confluence of Wheelwright Creek. After treated wastewater leaves the Parshall Flume it is conveyed to the effluent outfall via a 30-inch diameter ductile iron pipe which transitions to a 32-inch diameter HPDE SDR-17 pipe. The effluent outfall consists of eight 9.0-inch diameter diffusers which are spaced at 5.7-feet on center. The effluent outfall is inspected by divers every 2 years and dredged if the average depth to the bottom is less than 16.5-inches. The effluent outfall is in good condition and has no known issues and therefore should be maintained for continued use.

3.9 PLANT HYDRAULICS

The operation staff indicated that, prior to the 2002 Outfall Upgrade project, the Parshall Flume experienced a tail water condition during extreme high tides. The operations staff indicated that there no known hydraulic problems at the WWTF at this time. The NPDES permit requires periodic visual inspection of the outfall.

The 100-year flood elevation as defined by the FEMA Flood Insurance Rate Maps (Map No. 33015C0402E, May 2005) at Elevation 8.0 (NGVD 1929 datum). The 100-year flood elevation as defined by the *Preliminary* FEMA Flood Insurance Rate Maps (Map No. 33015C0402F, April 2014) at Elevation 7.0 (NAVD 1988 datum). The current and preliminary proposed flood elevation are essentially identical when expressed on the same datum. The current and preliminary FEMA flood elevations is lower than the aerated lagoon berms as well as the lowest hydraulic control point at the WWTF (i.e., the effluent parshall flume, invert Elevation 10, NGVD 1929).

The Town is currently participating in the Climate Adaptation Plan for Exeter (CAPE) project. The purpose of the CAPE project is to facilitate long-term adaptation planning as it pertains to existing zoning as well as existing stormwater infrastructure (and to a lesser extent wastewater infrastructure). As a part of the project, the CAPE project team has developed a computer model to assess flood elevations under a series of existing and future conditions. For the 100-year flood combined with the 100-year storm surge in the year 2070, the CAPE model is projecting flooding to Elevation 11 to 13 (NAVD 1988 datum). This is below the existing aerated lagoon berms but is <u>well above</u> the lowest hydraulic control point at the WWTF. The impact of these higher future flood elevations should be considered in the preliminary design phase of the project as it may impact the elevation of the new WWTF unit processes. It may also be appropriate to provide space on-site for a potential future effluent pump station. ***[NOTE: The CAPE final report has not yet been issued. This paragraph needs to be updated when that work is completed.]***

3.10 PLANT WIDE SUPPORT SYSTEMS

The ancillary plant wide support systems are described below.

3.10.1 Process Water System

The process water system was installed during the 1988 upgrade and is fed from the "second-pass" of both Chlorine Contact Tanks via an 8-inch diameter ductile iron pipe. The system capacity was identified as 200 gpm at 80 psi. The process water feed is pumped by one of two 10-hp process water pumps, located in the Chlorination Building, via a 4-inch diameter ductile iron forcemain to a 1,000 gallon hydro-pneumatic storage tank located in the Control Building. The process pumps were rebuilt in approximately 2011. The hydro-pneumatic storage tank is pressurized by a 3-hp air compressor also located in the Control Building which was replaced in approximately 2002 and had the motor replaced in approximately 2009. The process pump running status is sent to SCADA and alarms if the pump fails, but there are no controls associated with the pumps. Process water is supplied to the Septage Holding Tank, Grit Building, yard hydrants and as carrier water for the sodium hypochlorite and sodium bisulfite chemical systems. The operators indicated that the system capacity is sufficient for current demands. The process water system has reached the end of its useful life and should be replaced with any future upgrades to the facility.

3.10.2 Scum Removal

Scum removal is only accomplished at the end of the Chlorine Contact Tank. Scum is collected at the end of each serpentine channel via an 8-inch diameter scum trough and then conveyed through an 8-inch diameter ductile iron pipe into the approximately 180 gallons Scum Well. The scum is pumped from the Scum Well via a 1/2-hp scum pump via a 2-inch diameter PVC pipe which discharges into Lagoon No. 3. The scum pump operates by floats and is not configured to SCADA. Both of the scum troughs worm gears are difficult to operate and leak. The scum removal system has reached the end of its useful life and should be replaced with any future upgrades to the facility.

3.11 BIOSOLIDS PROCESSING

3.11.1 Aerated Lagoons No. 1, 2, and 3

Waste biosolids settle out from the wastewater and accumulate in the bottom of each aerated lagoon. The amount of biosolid accumulation decreases as the wastewater moves from Lagoon No. 1 to No. 2 and No. 3, therefore Lagoon No. 1 has the most accumulated biosolids and Lagoon No. 3 has the least amount of biosolids. The estimated waste biosolids volume is approximately 8.0 MG, based on the SolarBee data report dated October 26, 2013. These biosolids will need to be removed if the lagoons are to be decommissioned.

3.11.2 Sludge Storage Lagoon

The Sludge Storage Lagoon has never been used for its intended purpose of storing sludge from Lagoons No. 1, No. 2 and No. 3. Prior to becoming the Sludge Storage Lagoon, it was Lagoon

No. 1 and a Stormwater Holding Pond. Presently the Sludge Storage Lagoon has two ponds located in it that drain via two 8-inch diameter culverts under the access road to Aerated Lagoon No. 3.

3.12 BUILDING SYSTEMS

A site evaluation was conducted on July 15, 2014 by Wright-Pierce architectural and electrical engineers. A summary of their findings is presented below.

3.12.1 Architectural

Wastewater Treatment Facility Buildings

The buildings at the WWTF were constructed in 1988 and have not been significantly upgraded. The buildings consist of a Control Building, a Grit Building and a Chlorination Building. All three buildings are of similar construction type: single story split faced masonry exterior walls with wood framed shingle roofs. Any of the existing buildings that will be retained for continued use should have the following repairs and improvements:

- Repair the minor cracks in the exterior masonry walls.
- Clean the moss and organic growth at the base of the walls in various locations.
- Install new sealants at the control joints and around the perimeter of all wall penetrations.
- Replace the shingle roofing and eave flashing.
- Replace vinyl siding at gable ends.
- Replace deteriorated doors.
- Replace the wood trim at the overhead door in the Control Building, if it is to remain.
- Replace the existing windows.
- Repaint the interior spaces.
- Replace other interior finishes such as flooring and acoustical ceilings.
- Provide separation of electrical gear from process spaces in Chlorination Building.
- Maintain separation between “classified” Pump Room and “unclassified” upper floor in Control Building (NFPA 820).

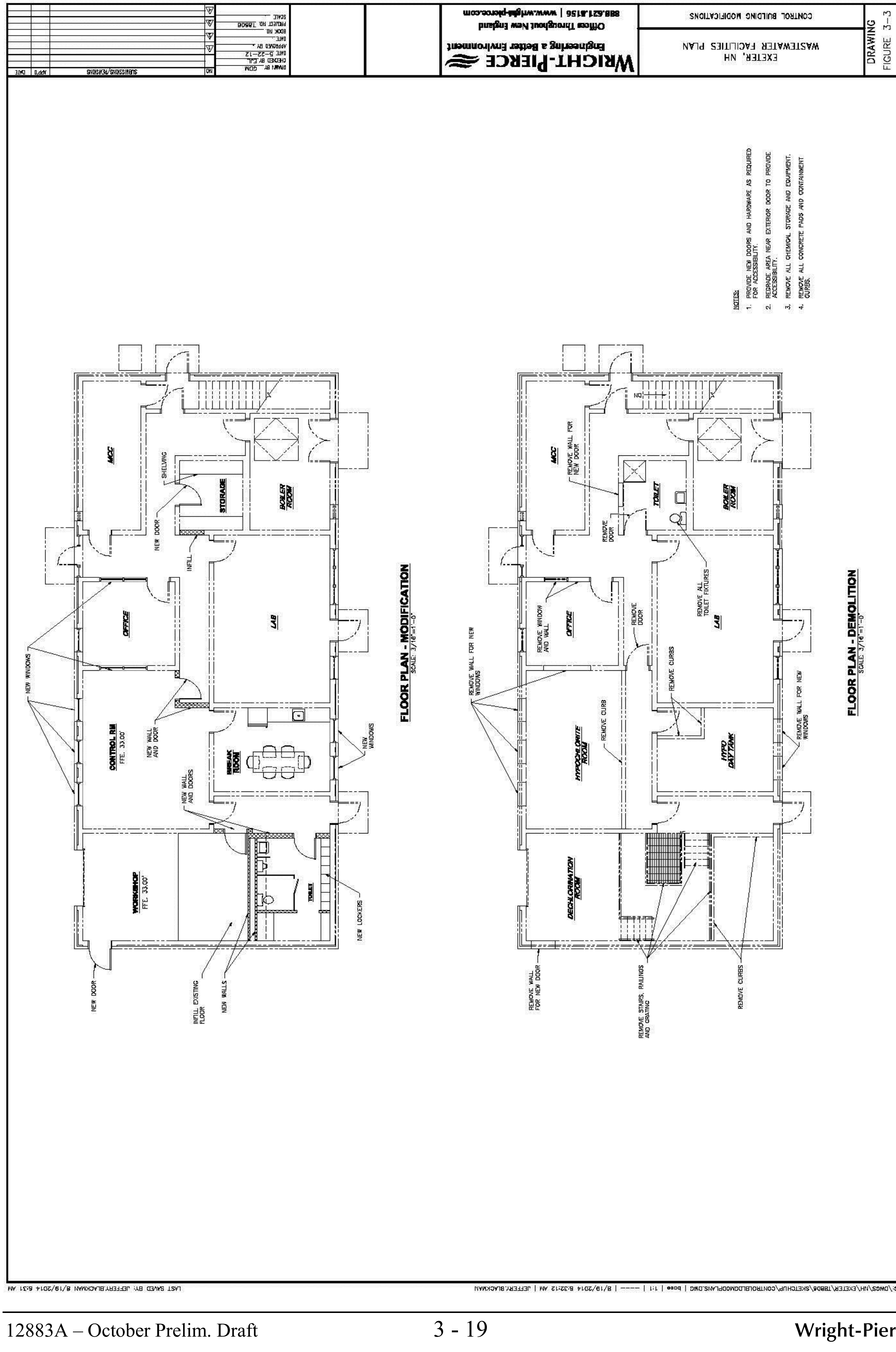
If a major upgrade is implemented at this facility, additional buildings would be constructed to meet the new treatment requirements. This would allow the chemical systems to be relocated out of the existing Control Building and would allow for the current chemical rooms to be converted to occupied functions (e.g., Meeting/Break Room, Control Room, Storage, Workshop and a handicapped accessible restroom) to better accommodate the needs of the current staff of four. Improvement required to implement these changes would include:

- Raising the depressed floor areas in the chemical rooms.
- New windows in the occupied spaces.
- Demolition of existing walls and construction of new walls
- New accessible rest room.
- New accessible door hardware.
- New interior finishes including paint, acoustical ceilings and flooring.
- New lighting.
- New HVAC systems.
- Re-grading at the building entry to make it accessible.
- Accessible parking.
- Add a small ramp or re-grade as required to provide a second accessible means of egress.
- Provide accessible signage.

A preliminary layout of the Control Building, indicating alternative space arrangements to address the identified space needs, is presented as **Figure 3-3**. This preliminary layout will need to be reviewed with the WWTF staff as well as the Code Enforcement Officer in greater detail in the preliminary design phase.

Main Pump Station Building

The Main Pump Station was constructed in 1964 and was upgraded in 1996. The building consists of single story building with a below-grade pump room and wetwell. The materials of construction are precast concrete tilt-up panels framed by aluminum “W” shapes installed vertically with base support plates to retain each panel. The aluminum frame is installed at the face of the slab with the wall cantilevered off the structure. The general condition of the building is fair to good, but there is evidence of movement of the building components. A gap is evident


DRAWING
FIGURE 3-3
EXETER, NH
WASTEWATER FACILITIES PLAN
CONTROL BUILDING MODIFICATIONS
WRIGHT-PIERCE
Engineering a Better Environment
Offices Throughout New England
888.621.8156 | www.wright-pierce.com
NOTES:
1. PROVIDE NEW DOORS AND HARDWARE AS REQUIRED FOR ACCESSIBILITY.
2. REGRADE AREA NEAR EXTERIOR DOOR TO PROVIDE ACCESSIBILITY.
3. REMOVE ALL CHEMICAL STORAGE AND EQUIPMENT.
4. REMOVE ALL CONCRETE PADS AND CONTAINMENT CURBS.
FLOOR PLAN - MODIFICATION
SCALE: 3/16"=1'-0"
MCC
STORAGE
BOILER ROOM
OFFICE
LAB
CONTROL RM
FFE. 33.00
BREAK ROOM
WORKSHOP
FFE. 33.00"
TOILET
SHELVING
NEW DOOR
INFILL
NEW WINDOWS
NEW WALL AND DOOR
NEW WALL AND DOORS
NEW LOCKERS
INFILL EXISTING FLOOR
NEW WALLS
FLOOR PLAN - DEMOLITION
SCALE: 3/16"=1'-0"
REMOVE WALL FOR NEW DOOR
REMOVE DOOR
REMOVE ALL TOILET FIXTURES
REMOVE WINDOW AND WALL
REMOVE CURBS
REMOVE CURB
HYPOCHLORITE ROOM
HYPO DAY TANK
DECHLORINATION ROOM
REMOVE WALL FOR NEW WINDOWS
REMOVE STAIRS, RAILINGS AND GRATING

between the loading dock and the wall panels and several of the base plated supporting the wall panels are deformed. Recommended improvements and repairs at this building should include:

- Repair the damaged base plates supporting the wall.
- Investigate further the cause of the gap between the wall panels and loading dock. This may be as result of simple settlement of the loading dock, but it should be further investigated.
- Replace the exterior doors.
- Provide separation between the "classified" and the "unclassified" spaces (NFPA 820).
- Replace the damaged stair nosings at the exterior stairs.
- The roofing system likely needs to be replaced.

Note that this building should be surveyed for lead and asbestos unless that has already been done as part of the previous upgrade.

3.12.2 Electrical

Wastewater Treatment Facility

The WWTF was constructed in its present form in 1988, and most of the electrical equipment dating from the initial construction is still in service. Electric service to the facility is provided by overhead utility primary conductors to riser pole #3736. From this pole, primary conductors feed an adjacent 500 kVA pad-mounted three-phase utility transformer located in front of the Control Building. Secondary conductors from the transformer supply electric service to the Control Building Main Circuit Breaker (Electrical Room) at 480 volts, three-phase, three-wire ungrounded, 800 amps. The aforementioned riser pole also supplies telephone and communications services to the Control Building. Also located adjacent to the riser pole and transformer is a diesel standby generator, built by Superior and rated 60 kW, located inside a walk-in enclosure which appears to be non-sound-attenuated. General observations are summarized below:

- The electric service disconnecting means (Main Circuit Breaker) is located inside the Control Building Electrical Room just off the building front entrance. The three-wire service appears to be ungrounded with no evidence of ground detection equipment. From the main circuit breaker switchboard, power is split with one branch feeding MCC#1 Normal Power Section (Aerators) and one branch feeding the Automatic Transfer Switch and MCC#1 Emergency Power Section. From MCC#1 Emergency Power Section, power is fed underground to the Grit Building (MCC#2) and the Chlorination (Lagoon) Building (MCC#3). The major electrical gear all appears to date from the original facility construction.

- Electrical components associated with a photovoltaic (PV) system are located outside the Control Building and are connected to a Photovoltaic Array located along the entrance to the site. This equipment is assumed to be connected into the Control Building electrical service at some point, although this could not be determined visually. The PV equipment is rated 50kW, 208 volts, 141 amps, with a 75 kVA dry-type transformer which appears to be provided for the purpose of stepping up the voltage from 208 volts to 480 volts. This equipment does not date from the original facility construction, but is of undetermined age.
- Power capacitors are located adjacent to, and connected to, MCC#1 Normal Power Section. These were reportedly provided to attempt to rectify some utility power problems and are not original to the facility construction.
- Standby power is supplied from the 60kW generator to all facility loads except for the Lagoon Aerators, which will not operate during an interruption of utility power to the facility. The Aerators are each fed by underground conductors from MCC#1 Normal Power section, to receptacle connection points located on the banks of the lagoons. Power is then carried aerially to each Aerator by power cables suspended on messenger cables.
- Lighting and single phase power in each of the buildings is provided from lighting panels and/or subpanels, with power to these panelboards being supplied from dry-type transformers.
- Interior lighting fixtures are either fluorescent or incandescent, depending on the location. Fixtures in the Control and Chlorination Buildings are enclosed and gasketed fluorescent with T8 lamps. Fixtures in the Grit Building are incandescent hazardous-location fixtures appropriate for that space. Exterior lighting fixtures are building-mounted HID wallpack fixtures. The fixtures are mostly functional, and appear to date from the original facility construction. No emergency battery lighting was observed in the facility, and exit lighting appeared to be inadequate in some areas.

- The facility presently has a SCADA system in place, with radio telemetry being received at the Control Building and signals being transmitted to the SCADA Panel MPU located in the Electrical Room. These controls are more recent than the original construction.
- The facility Fire Alarm System, GE ESL 1500 Series, appears original to the facility construction, and is reportedly functional and tested annually. The system covers the Control Building, Grit Building, and Chlorination (Lagoon) Building. It includes pull stations, smoke or heat detectors, notification appliances, and outdoor items at the Control Building (Gamewell box, red strobe, remote annunciator, and Suprasafe key box).
- Electrical equipment and systems in the facility are generally functional and in conditions consistent with their age and various locations. As expected, equipment in the Grit Building and nearest the different chemical systems is showing the greatest degree of corrosion.

Given the age and obsolescence of much of the electrical equipment and systems in the facility, it should be considered for replacement. Ultimately, however, it will depend on the final process configuration of the facility whether the electrical systems are completely or only partially replaced. If the present facility is replaced with a new activated-sludge treatment facility, then there would be a completely new electrical service with new standby generator, and new distribution equipment throughout the facility. Existing buildings would be upgraded with new electrical equipment and wiring to meet the new space requirements. If the present facility is to remain as it exists today as a lagoon plant, then more targeted electrical upgrades would be provided. The intent would be to replace degraded or obsolete equipment and wiring as necessary, and leave some newer functional equipment in place.

Main Pump Station

The Main Pump Station was constructed in 1964, and most electrical equipment in the station dating from the initial construction is still in service. Since that time, variable frequency drives have been provided for the present-day pumps, which were upgraded in 1996. Also, the original indoor standby generator was removed and replaced with a new outdoor, 200 kW Caterpillar diesel generator, installed in a sound-attenuated walk-in enclosure. This generator, installed within the past 12 to 15 years, has its fuel supplied from a dual-wall, sub-base tank located under the generator inside the enclosure. General observations are summarized below:

- Electric service to the station is provided by a pole-mounted three-phase utility transformer located adjacent to the station. Main service and distribution equipment consists of the original Clark Control motor control center, with transfer to standby power through the ASCO automatic transfer switch located in the Clark MCC. The main circuit breaker in the Clark MCC is not readily accessible from the station entry door, necessitating travel through the main floor of the station in order to shut off utility power to the station.
- The variable frequency drives provided as part of the 1996 pump upgrade are Cutler-Hammer SV9000 drives. The drives are located on the main floor level. There are no local safety disconnect switches on the lower level where the pumps are located.
- Interior and exterior lighting fixtures are a mix of incandescent (lower level and outdoors) and fluorescent (main floor level). The fixtures are mostly functional, and have likely been upgraded since the original construction.
- Telephone service exists in the station, but there is no fire alarm system present in the station.
- Pump controls have been upgraded since the original construction, with SCADA system panel RTU-800 providing control and data transmission to the Wastewater Treatment Facility Control Building via radio telemetry.

Given the age and obsolescence of much of the pump station electrical equipment, it is recommended that the station electrical equipment and systems be completely replaced, with the exception of the outdoor standby generator, which can remain in service. This will also provide an opportunity to bring the pump station into compliance with present National Electrical Code requirements regarding location of power disconnecting means, as well as other pertinent requirements.

3.12.3 Energy Efficiency/ Green Design

New buildings, as well as upgrades to existing buildings, will need to consider current building codes, energy efficiency guidelines and requirements and "green design" elements (where cost effective). Items that are typically considered for WWTF upgrades include the following:

- Natural and high efficiency lighting (with motion sensors in some locations);
- Solar walls;

- Effluent heat exchanger (to capture heat from WWTF effluent) and air-to-air heat exchangers and/or energy recovery ventilators (to capture heat from heated spaces);
- Building envelope improvements such as insulated walls, windows and roofs;
- White EPDM roofing for reduced solar gain; and
- Minimizing impervious surfaces and point source runoff.

Section 4

SECTION 4

TOWN-WIDE NITROGEN MANAGEMENT

4.1 INTRODUCTION

DES has been studying the Great Bay Estuary system for many years. A listing of the most relevant work prepared by DES is provided below.

- Numeric Nutrient Criteria for the Great Bay Estuary (June 2009)
- Preliminary Watershed Nitrogen Loading Thresholds for the Watersheds Draining to the Great Bay Estuary (October 2009)
- Review of Numeric Nutrient Criteria for the Great Bay Estuary (EPA funded review, Howarth, June 2010)
- Analysis of Nitrogen Loading Reductions for Wastewater Treatment Facilities and Non-Point Sources in the Great Bay Watershed (Draft, December 2010)
- Assessments of Aquatic Life Use Support in the Great Bay Estuary for Chlorophyll-a, Dissolved Oxygen, Water Clarity, Eelgrass Habitat, and Nitrogen (April 2012)
- Great Bay Nitrogen Non-Point Source Study (Draft, May 2013)
- Joint Report of Peer Review Panel for Numeric Nutrient Criteria for the Great Bay Estuary (Coalition funded review, Bierman, Diaz, Kenworthy, Reckhow, February 2014)
- Great Bay Nitrogen Non-Point Source Study (Final, June 2014)

Based on their studies, DES has determined that the nitrogen sources of concern are largely “man-made” (or anthropogenic) sources which come from “point sources” (e.g., WWTF) and from “non-point sources” (e.g., atmospheric deposition, stormwater drainage systems, fertilizer use, animal wastes, and septic systems). Further, DES has concluded that reductions in nitrogen are required from all communities within the Great Bay Estuary watershed in order to achieve the desired level of water quality improvements. On this basis, EPA issued the Town a NPDES permit and an Administrative Order on Consent (AOC). The AOC requires that the Town have a serious and long-standing commitment to monitoring, tracking, accounting and implementation for nitrogen management. The AOC is included in **Appendix A** of this report.

Key implementation elements of the AOC are summarized below.

- *"…the Town shall begin tracking all activities [that the Town should reasonably be aware of, e.g., activities that involve a Town review/approval process or otherwise require a notification to the Town] within the Town that affect the total nitrogen load to Great Bay Estuary. This includes, but is not limited to, new/modified septic systems, decentralized wastewater treatment facilities, changes to the amount of effective impervious cover, changes to the amount of disconnected impervious cover [including pavement and buildings], conversion of existing landscape to lawn/turf and any new or modified Best Management Practices." [Article D.1]*

- *"…the Town shall begin coordination with the NHDES, other Great Bay communities, and watershed organizations in NHDES's efforts to develop and utilize a comprehensive subwatershed-based tracking/accounting system for quantifying the total nitrogen loading changes associated with all activities within the Town that affect the total nitrogen load to the Great Bay Estuary." [Article D.2]*

- *"…the Town shall begin coordination with the NHDES to develop a subwatershed community-based total nitrogen allocation." [Article D.3]*

- *"By September 30, 2018, [the Town shall] submit to EPA and the NHDES a total nitrogen non-point source and point source stormwater control plan ("Nitrogen Control Plan"), including a schedule of at least five years for implementing specific control measures as allowed by state law to address identified non-point source and stormwater Nitrogen loadings in the Town of Exeter that contribute total nitrogen to the Great Bay Estuary, including the Squamscott River. … The Nitrogen Control Plan shall be implemented in accordance with the schedules contained therein."[Article D.4]*

- *"By December 31, 2023, the Town shall submit an engineering evaluation that includes recommendations for the implementation of any additional measures necessary to achieve compliance with the NPDES Permit, or a justification for leaving the interim discharge limit set forth in Attachment 1.a in place (or lower the interim limit to a level below 8.0 mg/l but still above 3.0 mg/l) beyond that date." [Article E.2]*

In addition to the above items, the AOC also requires the submittal of annual progress reports [Article E.1] on the status of the development of the nitrogen tracking/accounting system, status of the development of the Nitrogen Control Plan and a description of any activities that changed nitrogen loading.

4.2 BASELINE LOADINGS FROM EXETER TO GREAT BAY

In order to determine the source of nitrogen loadings to the Great Bay, DES has developed numerous technical reports over the past five years, including reports which estimate the amount of point source and non-point source nitrogen generated by each municipality. The most recent and comprehensive effort is the 2014 Final Great Bay Nitrogen Non-Point Source Study. This study provides a breakdown of non-point source loadings resulting from atmospheric deposition, chemical fertilizers, animal wastes, and human wastes (septic systems).

The Great Bay Nitrogen Non-Point Source Study describes the distinction between the "input load" to the watershed (i.e., the actual load generated by a particular source such as a roof, field, forest, parking lot, etc.) and the "delivered load" to the watershed (i.e., the load which ultimately reaches the receptor surface water after undergoing natural treatment processes along the transport pathway such as bacterial action, vegetative uptake, etc.). The delivered load is the most important parameter and is used exclusively herein.

The municipal boundaries of the Town of Exeter encompass four sub-estuary watersheds: Exeter/Squamscott River watershed; Lamprey River watershed; Winnicut River watershed; and Hampton Harbor watershed (refer to **Figure 4-1**). **Table 4-1** summarizes the demographics and delivered nitrogen loadings from Exeter to each of these sub-estuary watersheds. For example, Exeter has 30% of the total population that lives within the Exeter/Squamscott River sub-estuary watershed but has 10% of the total land area that falls within that watershed. **Table 4-2** summarizes Exeter's delivered nitrogen loadings to all four sub-watersheds by source type. **Table 4-3** summarizes the delivered nitrogen loadings to the Exeter/Squamscott River watershed by source type and by source town. Key conclusions from these tables include:

- The significant majority of Exeter's nitrogen loads are to the Exeter/Squamscott River watershed; the loadings to the Lamprey River, Winnicut River, and Hampton Harbor watersheds are relatively insignificant.
- 65% of the nitrogen load to the Exeter/Squamscott River watershed comes from other towns.
- 74% of the nitrogen load to the Exeter/Squamscott River watershed comes from non-point sources.

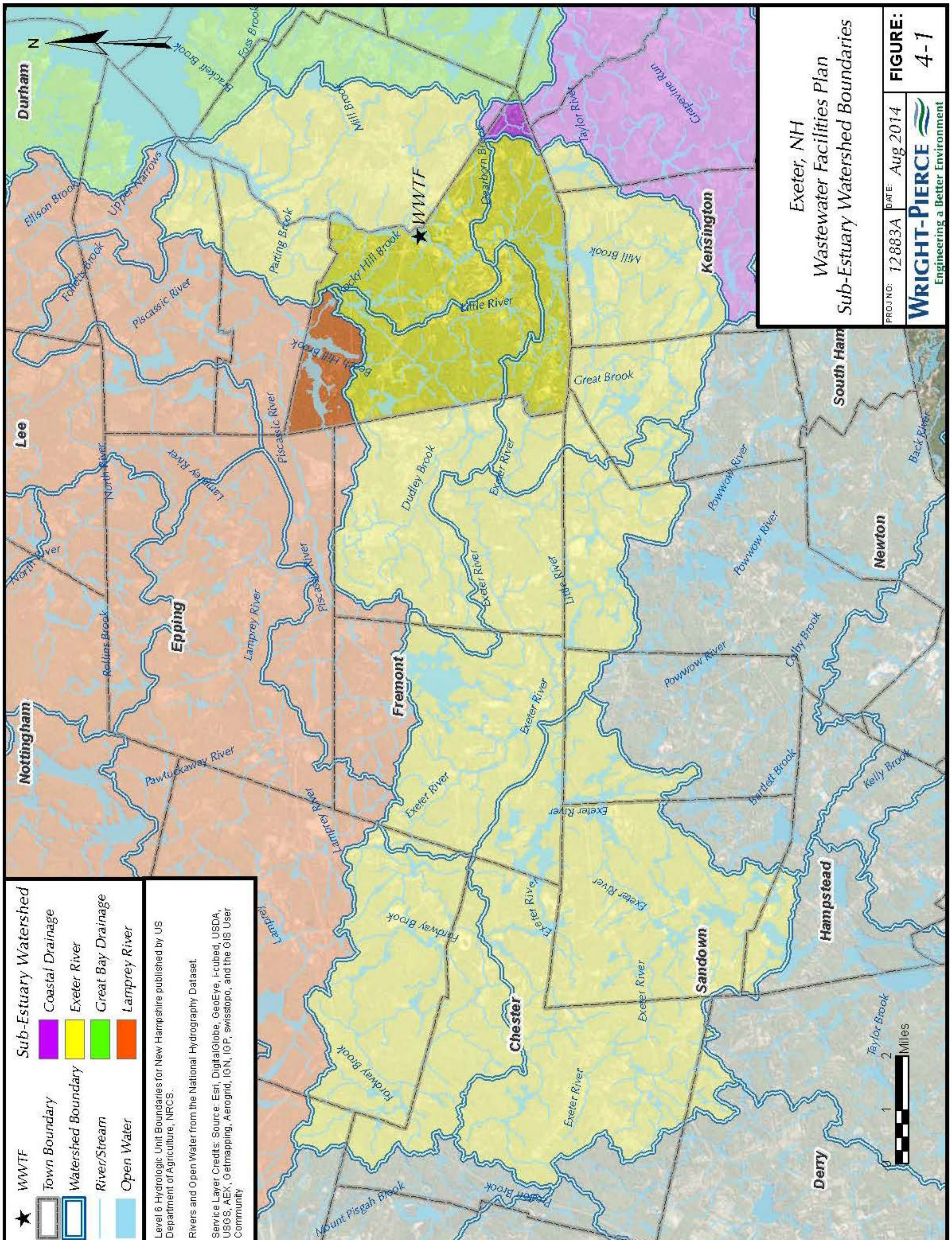

Exeter, NH
Wastewater Facilities Plan
Sub-Estuary Watershed Boundaries
PROJ NO: 12883A
DATE: Aug 2014
FIGURE:
4-1
WRIGHT-PIERCE
Engineering a Better Environment
Sub-Estuary Watershed
Coastal Drainage
Exeter River
Great Bay Drainage
Lamprey River
WWTF
Town Boundary
Watershed Boundary
River/Stream
Open Water
Level 6 Hydrologic Unit Boundaries for New Hampshire published by US Department of Agriculture, NRCS.
Rivers and Open Water from the National Hydrography Dataset.
Service Layer Credits: Source: Esri, DigitalGlobe, GeoEye, i-cubed, USDA, USGS, AEX, Getmapping, Aerogrid, IGN, IGP, swisstopo, and the GIS User Community
Miles
0
1
2
N
Durham
Lee
Nottingham
Epping
Fremont
Chester
Sandown
Kensington
South Ham
Newton
Hampstead
Derry
WWTF
Exeter River
Little River
Dudley Brook
Fordway Brook
Great Brook
Mill Brook
Parting Brook
Rocky Hill Brook
Beech Hill Brook
Dearborn Brook
Taylor River
Grapevine Run
Piscassic River
Lamprey River
North River
Pawtuckaway River
Rollins Brook
Powwow River
Colby Brook
Bartlett Brook
Kelly Brook
Back River
Taylor Brook
Mount Pisgah Brook
Ellison Brook
Follett's Brook
Upper Narrows
Bracket Brook
Foss Brook

TABLE 4-1
DELIVERED TN LOAD FROM EXETER – BY SUB-ESTUARY WATERSHED

Category	% of Category Resulting From Exeter			
	Exeter/ Squamscott River	Lamprey River	Winnicut River	Hampton Harbor
Population	30%	1.0%	0.4%	1.7%
Land Area	10%	1.1%	0.2%	6.7%
No. of Septic Systems	8%	1.3%	0.2%	2.4%
Point Source Nitrogen	96%	0%	0%	0%
Non-Point Source Nitrogen	14%	0.7%	0.5%	0.6%
Total Nitrogen	35%	0.5%	0.5%	0.6%

Source: Great Bay Nitrogen Non-Point Source Study (2014), WWTF effluent data (2009-2012).
Additional point source nitrogen loads from lagoon leakage, CSOs and SSOs are not quantified.

TABLE 4-2
DELIVERED TN LOAD FROM EXETER – BY SOURCE TYPE

Source Type	Nitrogen Load (tons/year)	% of Total	Rank
NPS-Atmospheric Deposition (incl. stormwater)	7.22	12%	2
NPS-Chemical Fertilizers	4.37	7%	3
NPS-Animal Waste	2.87	5%	5
NPS-Human Waste (septic systems)	4.17	6%	4
PS-WWTF	42.69	70%	1
Total	61.33	100%	

Source: Great Bay Nitrogen Non-Point Source Study (2014), WWTF effluent data (2009-2012).
Additional point source nitrogen loads from lagoon leakage, CSOs and SSOs are not quantified.

TABLE 4-3
DELIVERED TN LOAD TO EXETER RIVER WATERSHED
BY SOURCE TYPE & TOWN

Source Type	Nitrogen Load From Exeter (tons/year)	Nitrogen Load Total (tons/year)	% of Total from Exeter
NPS-Atmospheric Deposition (incl. stormwater)	6.38	41.36	15%
NPS-Chemical Fertilizers	4.00	19.43	21%
NPS-Animal Waste	2.77	16.82	16%
NPS-Human Waste (septic systems)	3.53	45.40	8%
PS-WWTF	42.69	44.27	96%
Total	59.37	167.28	35%

Source: Great Bay Nitrogen Non-Point Source Study (2014), WWTF effluent data (2009-2012).
Additional point source nitrogen loads from lagoon leakage, CSOs and SSOs are not quantified.

4.3 ESTABLISHMENT OF NITROGEN THRESHOLDS

A "threshold load" is the load below which water quality goals are presumed or expected to be met. Typically, a threshold load would be established by a Total Maximum Daily Load (TMDL) Report. To date, a TMDL Report has not been completed and is not being contemplated in the near term. Instead, the 2010 Analysis of Nitrogen Loading Reductions is the only document prepared by DES to date which identifies a threshold load. These threshold loads are based on the 2009 Numeric Nutrient Criteria document. The 2009 Numeric Nutrient Criteria document established 0.3-mg/l as the water column nitrogen concentration necessary to prevent loss of eelgrass habitat and 0.45-mg/l as the water column nitrogen concentration necessary to prevent occurrences of low dissolved oxygen. DES identified a threshold load for the Great Bay as well as each sub-estuary. The 2010 Analysis of Nitrogen Loading Reductions identifies the threshold loads for the Exeter/Squamscott River sub-estuary watershed as:

- 140.3 tons of nitrogen per year to prevent low dissolved oxygen conditions in the river;
- 87.8 tons of nitrogen per year to protect eelgrass in the sub-estuary; and
- 161.7 tons of nitrogen per year to protect eelgrass in the downstream subestuaries.

DES has indicated that there is no known eelgrass habitat within the Exeter/Squamscott River sub-estuary (personal communication, P. Trowbridge, NHDES, January 2014). Therefore, the governing threshold is that needed to prevent low dissolved oxygen conditions in the river, which totals 140.3 tons of nitrogen per year. Per **Table 4-3**, the current delivered load is 167.29 tons/year; therefore, approximately 16% (~27 tons of nitrogen per year) of the current delivered load is required to be removed to meet the threshold. It is important to note that, in order to maintain nitrogen loads below the threshold load, future growth must be fully offset (i.e., no net nitrogen increase resulting from growth).

These values will be used for planning purposes in this report. However, it is essential to note that the 2009 Numeric Nutrient Criteria document underwent a peer review by collaborative agreement between DES and the Cities of Dover, Rochester and Portsmouth. The results of the peer review are documented in a report entitled "*Joint Report of Peer Review Panel for Numeric Nutrient Criteria for the Great Bay Estuary, New Hampshire Department of Environmental*

Service, June 2009". On the basis of this peer review, DES and the Cities of Dover, Rochester and Portsmouth agreed that the DES will no longer use the numeric nutrient criteria in its Section 305(b) and 303(d) water quality assessment for the Great Bay Estuary (Settlement Agreement, Docket 2013-0119). Accordingly, the threshold values noted above and used herein should be considered guidance that may change in the future.

4.4 PRELIMINARY STRATEGY FOR NITROGEN MANAGEMENT

Given the context of the AOC, nitrogen management strategies should focus on reduction of both point sources and non-point sources. Point source reduction strategies are addressed in **Section 5** of this report and consist of upgrading the WWTF. Non-point source reduction strategies could consist of a host of options to manage the loads coming from the various categories included in the DES model. A general description of each category is provided below.

- <u>Atmospheric Deposition</u> - There is a growing body of data which indicates that atmospheric nitrogen deposition has been decreasing since the late 1990s (a result of the Clean Air Act and Clean Air Act Amendments). These trends in atmospheric deposition warrant inclusion in nitrogen management strategies. It is worth noting that the Long Island Sound TMDL Report (CTDEP, 2000) included an 18% reduction in atmospheric nitrogen deposition as a part of the required reductions. The CTDEP Long Island Sound Study Work Group is currently re-evaluating the TMDL and expects that atmospheric nitrogen deposition has been reduced more than the 18% value. In Appendix A of the DES Great Bay Non-Point Source Study, referencing EPA estimates, NHDES cites that nitrogen deposition could decrease by as much as 33% from the 2009 rates included in the report by 2020. In addition, the atmospheric deposition category includes non-point source loadings from stormwater. Best management practices (BMPs) will be required for over time which would be expected to further reduce the amount of nitrogen in stormwater. We suggest a target value of 30% reduction in atmospheric nitrogen reduction for the planning period (through 2040).

- <u>Chemical Fertilizers</u> - Nitrogen load resulting from chemical fertilizers could be reduced by BMPs, public education and community outreach. We suggest a target value of 20% reduction in the current loadings for this category.

- Animal Wastes - Nitrogen load resulting from animal wastes could be reduced by public education and community outreach. We suggest a target value of 10% reduction in the current loadings for this category.

- Human Wastes (Septic Systems) - Nitrogen load resulting from human wastes/septic systems could be reduced by connecting currently unsewered homes to public sewers or by requiring the installation of on-site denitrifying systems. We suggest a target value of 0% to 10% increase in the current nitrogen loadings for this category.

  It is important to note that the effectiveness and cost associated with control of nitrogen from septic systems should be carefully considered. For example, a home with a standard septic system located greater than 200 meter from surface water is estimated to reduce the "delivered load" to 26% of the original load from the home (i.e., 74% removal). This is more than twice as much nitrogen removal achieved in a typical secondary WWTF and similar to the percent removal resulting from a WWTF designed to produce effluent total nitrogen of 8 mg/l. The WISE project will provide cost information for the different wastewater management approaches. The effectiveness and cost associated with each approach should be carefully considered prior to setting policy. Refer to the table below for additional information regarding the effectiveness of various approaches.

Wastewater Management Approach	Assumed Input Load lbs/day	Resultant Delivered Load lbs/day	Effective Removal
Secondary WWTF	1	0.67	33%
Standard Septic System, <200m	1	0.6	40%
Denitrifying System, <200m	1	0.3	70%
WWTF with TN Removal to 8 mg/l	1	0.27	73%
Standard Septic System, >200m	1	0.26	74%
WWTF with TN Removal to 5 mg/l	1	0.17	83%
Denitrifying System, >200m	1	0.13	87%
WWTF with TN Removal to 3 mg/l	1	0.10	90%

Notes:
1. Delivery factors for standard septic systems are from NHDES GBNNPS Study (June 2014).
2. Delivery factors for denitrifying systems were adjusted by Wright-Pierce to account for improved TN removal by the on-site system.
3. WWTF TN removals were based on the typical Exeter influent TKN value of 30 mg/l.

Figure 4-2 provides a comparison of existing conditions (i.e., existing flows and existing effluent nitrogen concentrations) versus with three nitrogen management scenarios at the full NPDES permitted flow rate. All three scenarios utilize the non-point source strategies described above (16% aggregate value) plus:

- Scenario A – Upgrade the WWTF to 3.0-mgd flow at **8-mg/l** effluent total nitrogen;
- Scenario B – Upgrade the WWTF to 3.0-mgd flow at **5-mg/l** effluent total nitrogen; and
- Scenario C – Upgrade the WWTF to 3.0-mgd flow at **3-mg/l** effluent total nitrogen.

It is also important to consider how the nitrogen management strategies might play out over time. As noted in previous sections of this report, the annual average flow from the WWTF is considerably less than the permitted 3.0-mgd. Also, the WWTF upgrade and the non-point source management measures will take time to implement and for the benefits to be measureable.

FIGURE 4-2
COMPARISON OF NITROGEN MANAGEMENT STRATEGIES FOR EXETER/SQUAMSCOTT RIVER WATERSHED LOADS

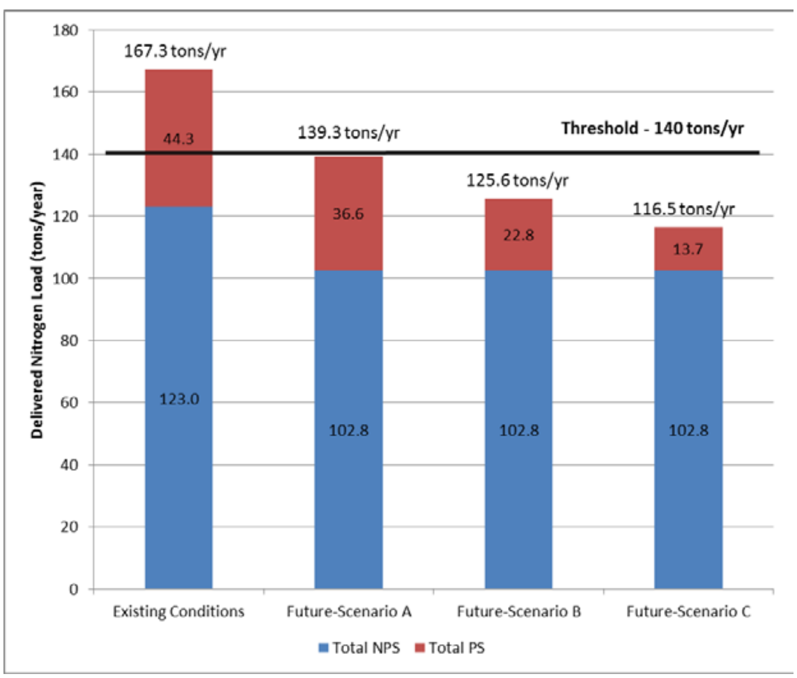
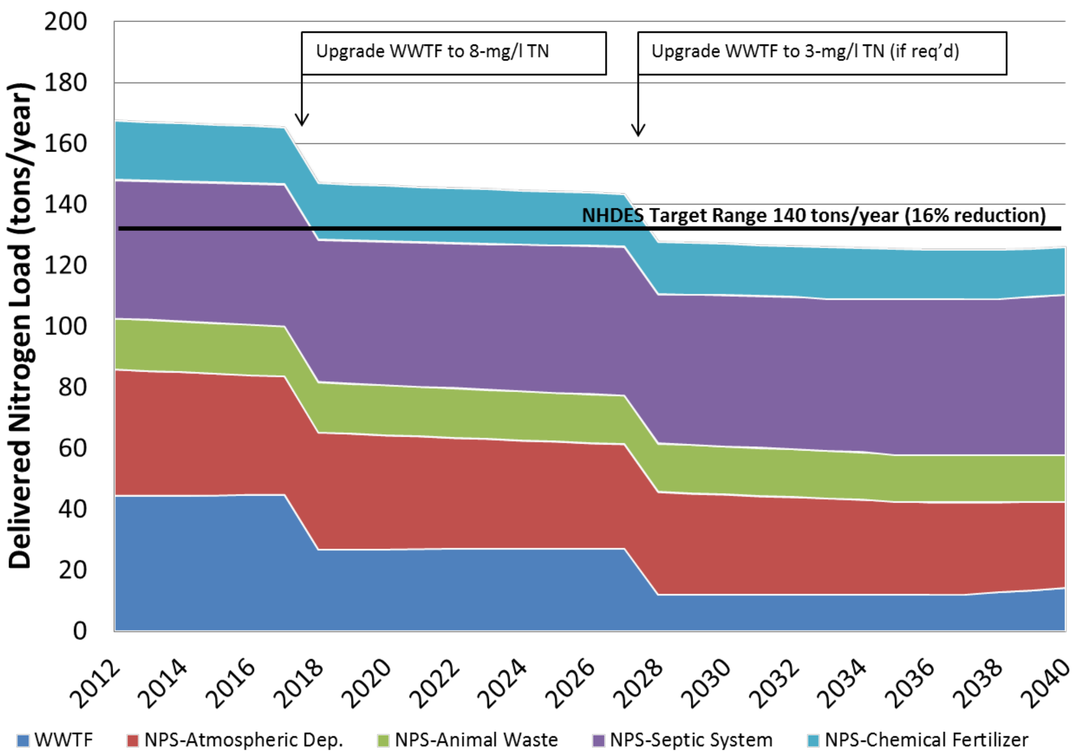


Figure 4-3 presents the delivered nitrogen load to the Exeter/Squamscott River watershed as it would vary over time from now through 2040 based on the assumptions listed below.

- Point Source – Loads are assumed to decrease overall based on Exeter flow increasing to 3.0 and Newfields flows increasing to 0.12 mgd (both permitted flows) and based on both WWTFs being upgraded to reduce effluent total nitrogen.
- Atmospheric Sources – Decrease 30% by 2040 from the Clean Air Act and Amendments as well as stormwater Best Management Practices.
- Chemical Fertilizers – Decrease 20% by 2040 via BMPs and public education.
- Animal Sources – Decrease 10% by 2040 via public education.
- Human Sources – Increase by 10% by 2040 due to population growth.

FIGURE 4-3
EXETER/SQUAMSCOTT RIVER WATERSHED LOADS OVER TIME



It is important to note, based on Figure 4-3, that 5 mg/l effluent nitrogen from the WWTFs appears to be sufficient to achieve the NHDES Target Range.

4.5 IDENTIFICATION OF RELEVANT WATERSHED STUDIES

Squamscott River August-September 2011 Field Studies

A field study of the Squamscott River was conducted in the August and September 2011. This work was documented in a technical memorandum prepared by HydroQual dated March 20, 2012. The study included two "spatial surveys" to collect representative samples for laboratory analysis of a suite of parameters along the river section between Great Dam in Exeter and Railroad Bridge in Stratham/Newfields. The study also included two datasondes deployed in the Squamscott River for approximately 45 days to provide continuous data for dissolved oxygen, chlorophyll-a, temperature and salinity. The technical memorandum indicates that the existing Exeter WWTF is a dominant factor is the dissolved oxygen levels in the river, in part because the WWTF is a source of nutrient as well as a direct source of chlorophyll-a to the river. The technical memorandum concludes that upgrade of the WWTF to an activated sludge-type treatment system, suitable to achieve 8-mg/l effluent total nitrogen, will result in substantial reduction in chlorophyll-a, and increase in dissolved oxygen. In addition, the technical memorandum concludes that decisions on further upgrades to the WWTF should be made based on a calibrated water quality model with data collected after the first upgrade.

Water Integration for the Squamscott-Exeter ("WISE")

The WISE project is funded by a grant from the National Estuarine Research Reserve System (NERRS). The purpose of the project is to establish a framework for inter-municipal collaboration for the Exeter/Squamscott River watershed and to provide certain tools for use by the towns. The project began in late 2013 and is on-going. Primary outputs from the project include items identified below:

- Analysis of a broad range of scenarios for non-point source nitrogen management such as green infrastructure, stormwater BMPs, fertilizer controls, low impact design zoning, "business as usual" zoning, etc.).
- Framework for the tracking and accounting system required by the AOC for use in the Nitrogen Control Plan.

- Input and technical assistance to evaluate and recommend the river monitoring locations and protocols for long-term AOC and MS4 (Municipal Separate Storm Sewer System) permit compliance.
- Macroalgae monitoring in the Squamscott River in 2014.
- Technical tools and guidance for stormwater BMPs, "illicit discharge detection and elimination" (IDDE) program, Water Quality Response Plan, mapping, etc.

Exeter River Great Dam Removal Project

The Town of Exeter has been studying the advantages, disadvantages and costs associated with removing Great Dam located on the Exeter River. Removal of the dam will likely improve water quality upstream and downstream of the dam. It is important to note that the Exeter WWTF is located downstream of the dam where river flow and depth characteristics are not expected to change. The Exeter River Great Dam Removal Feasibility and Impact Study (Vanasse Hangen Brustlin, Inc., 2013) indicates that removal of the dam would reduce thermal gain (smaller surface area for thermal absorption) and result in improved dissolved oxygen concentrations. Downstream water quality impacts/improvements will require additional data collection and analysis subsequent to the dam removal.

4.6 DEMONSTRATION OF FUTURE COMPLIANCE

EPA and DES have not specified a "conventional path" to demonstrate future compliance, but rather, have stated that they expect that information gathered and prepared by Exeter and other regulated communities (e.g., Newmarket, Durham, Dover, Rochester, Portsmouth) over the next five to ten years (through the AOC and other public studies) will inform this determination. Ultimately, EPA and DES will be looking for the Great Bay and its sub-estuaries to have an ecological and biological response that meets the water quality standards. This response may occur at nitrogen levels that are above or below the threshold criteria concentrations developed by DES. If an adequate response occurs with nitrogen levels higher than the threshold criteria, this would be justification to suspend implementation activities. Alternatively, if the response has not occurred and nitrogen levels are lower than the threshold criteria, additional efforts will likely be required.

Accordingly, the following specific items should be considered over the upcoming years:

- EPA and DES have indicated that groundwater travel time ("on the order of decades") and natural and seasonal variations will need to be taken into account in the demonstration of compliance over the long-term. This will place additional emphasis on the river monitoring program and on the ability of the tracking and accounting tools to project future conditions (i.e., when does the load arrive in the river or the bay).

- DES will review trends in the nitrogen concentrations in the Squamscott River, above and below the WWTF, and in Great Bay. Establishing a long-term data record for in-stream nitrogen concentration is critically important.

- Exeter should maintain a lead role in advocating for allocation of responsibility for nitrogen management based on percent contribution of delivered load. The methodology for allocation of responsibility for nitrogen loads is extremely important as it will determine how the cost burden is shared between sewered and non-sewered communities.

- Exeter should make efforts to evaluate its current practices with regard to the "nitrogen footprint" of future development (i.e., Town Master Plan, zoning, ordinances, conservation easements, etc.). Measures which have a low cost burden and/or a low "cost per pound of nitrogen removed" should be considered high priority measures for promulgation.

- Exeter nitrogen management program should provide for an adaptive and phased approach to implementation of both point source and non-point source management efforts. Efforts should be focused on measures that have the <u>least natural attenuation</u> as well as the <u>shortest travel time</u> to the Squamscott River and Great Bay.

- Exeter should strongly consider WWTF upgrade approaches that have the "lowest cost per pound of nitrogen removed" (versus just "lowest cost"), especially for approaches that provide additional nitrogen removal and minimal or modest incremental cost.

- Exeter should continue to monitor the progress of, and to collaborate with, the other regulated "point source" Great Bay communities. For example, significant point source load reductions will be implemented over the next four years. Specifically, upgrades to the five largest WWTFs are anticipated to occur as follows: Rochester (2015); Dover (2015); Durham (2015); Portsmouth Peirce Island (2017); Newmarket (2017); and Exeter (2018).

4.7 NON-STRUCTURAL AND NON-TRADITIONAL MEASURES

Nitrogen management can be accomplished through so-called structural, non-structural and non-traditional measures. Structural measures include "grey infrastructure" (e.g., sewers, treatment plants, etc.) and "green infrastructure" (e.g., source control through private I/I reduction, engineered wetlands for stormwater treatment, pervious pavement, rain gardens, etc.). Non-structural and non-traditional measures which could be used for nitrogen management include:

Non-Structural	Non-Traditional
Density controls	Permeable reactive treatment barriers
Fertilizer management	Aquaculture
Stormwater best management practices	Dredging and flushing enhancements
Public awareness campaigns	Alternative toilet systems
Septic system nutrient management	Integrated "grey" and "green" approaches

It is also essential to ensure that all Great Bay watershed communities participate and address their share of the attenuated load (i.e., the load that reaches the estuary). This will require that DES refine its point source and non-point source models to "allocate" responsibility among the Great Bay watershed communities. Implementation under this model could be accomplished through techniques such as cost sharing arrangements (e.g., Maryland's "Flush Tax") or watershed-based permitting and nutrient trading (e.g., Connecticut's Long Island Sound Program).

4.8 ADAPTIVE MANAGEMENT

In dealing with complex environmental problems, precisely determining the optimum solution can take many years and require very extensive study. At some point, sufficient information is available to embark on a solution, even though all aspects of the best solution have not yet been determined. Adaptive management is the formulation and implementation of a plan that begins to solve the problem while further information is gained to guide later phases toward the best overall solution. The basic elements of a successful adaptive management plan are:

- A solution that can be implemented in phases over time;
- Acquisition of data to show the effectiveness of the early phases of the solution; and
- A mechanism to re-assess the plan and adjust it to reflect the information gathered.

The data acquisition program must be directed at answering the question: "***What information is needed to determine the impacts of early phases of the project so that later phases can be modified if necessary?***" The data evaluation and "program re-assessment" must be well planned and must provide results that are approvable by the regulatory agencies.

Exeter's Adaptive Management Plan should address the following uncertainties:

1. How does the reduction in watershed nitrogen loading actually improve the water column nitrogen concentration in the impacted embayment? Is the water column concentration more or less sensitive to watershed load than predicted by the NHDES models?

2. How does the eelgrass or benthic community respond to the reduction in water column nitrogen concentration? Are the eelgrass and/or benthic communities more or less sensitive to water column nitrogen concentration than predicted in the NHDES models?

3. Has progress in other watershed communities occurred on schedule and, if not, how does that impact the decision making framework for Exeter?

4. Has growth followed the progression expected or is capacity needed sooner (or later) than planned?

5. Have any municipalities expressed interest in regional solutions?

6. Are the non-structural and non-traditional components of the plan more, or less, effective than assumed?

7. Have any pilot programs for non-traditional and/or non-structural measures conducted in the Great Bay watershed produced results which should be applied full-scale in Exeter? Have pilot programs for non-traditional and/or non-structural measures conducted in other areas of the United States produced results which could be applied in Exeter?

8. Have advanced on-site denitrifying treatment systems become available and should they be applied in less densely developed neighborhoods in lieu of sewer extensions? Should a nitrogen management ordinance be enacted within 200 meters of surface waters?

A data acquisition program should be developed such that these questions can be analyzed on an annual basis throughout the project. This review could be documented in an annual report which could be distributed to regulators, representatives of neighboring towns and interested watershed associations. A core group of these parties could meet annually to review the annual report and to provide input on possible modifications to the program.

Section 5

SECTION 5

EVALUATION OF ALTERNATIVES

5.1 INTRODUCTION

This section of the report presents the identification and evaluation of several wastewater treatment alternatives to address specific facility needs identified in **Sections 2 and 3** while acknowledging the town-wide nitrogen management considerations identified in **Section 4**.

5.1.1 Purpose of the Alternatives Analyses

In order to progress through a facilities planning process, numerous decisions must be made. The purpose of these alternatives analyses is to provide technical and cost information on which to base these decisions. Each of these decisions will serve as a “building block” towards the development of the recommended plan. We have made every effort to develop each analysis is such a way as to compare alternatives on an “apples to apples” basis. However, it is important to recognize that items which are “equivalent between alternatives” may not be included. It is also important to recognize that there will likely be cost saving opportunities as well as phasing opportunities, which will be explored in **Section 6**.

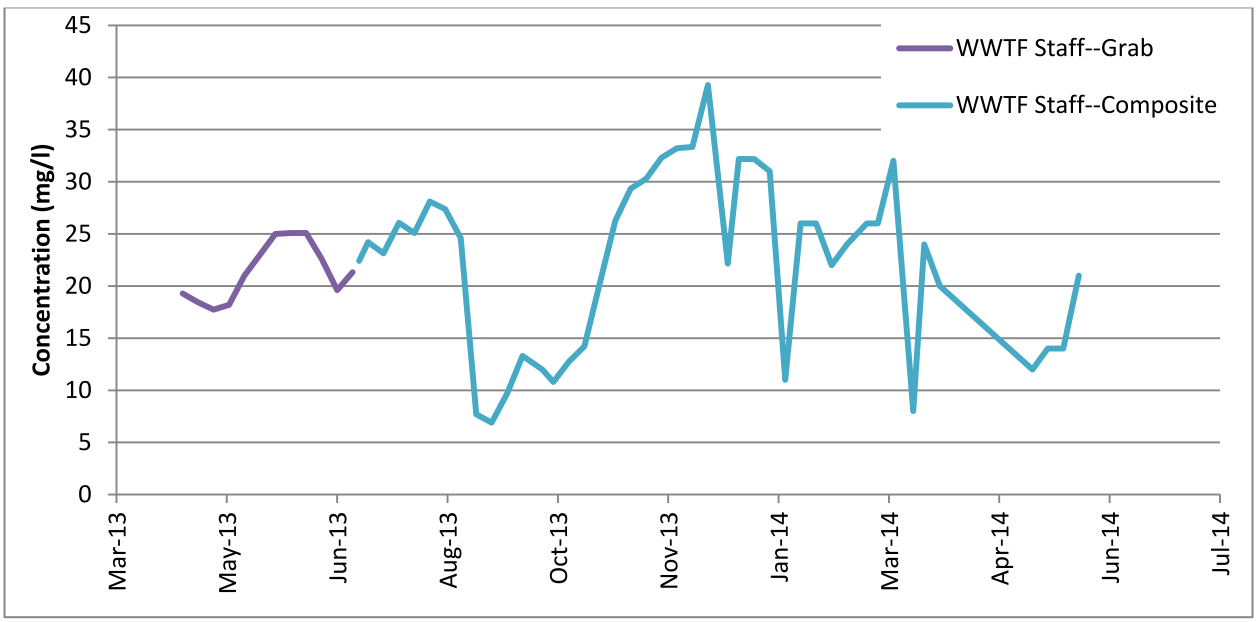
5.1.2 NPDES Permit and AOC Requirements

As described in Section 2, the NPDES permit provides the WWTF with a limit of 3.0 mg/l effluent total nitrogen based on a 214 day, seasonal rolling average from April 1 to October 31. The facility must “optimize the operation” of the facility for total nitrogen removal from November 1 to March 31, however, there is no effluent limit and no supplemental carbon is required in this non-summer period. The AOC provides the WWTF with an interim limit of 8.0 mg/l effluent total nitrogen based on a 214-day seasonal rolling average from April 1 to October 31. The facility must “optimize the operation” of the facility for total nitrogen removal from November 1 to March 31; however, there is no effluent limit during this non-summer period. In addition, the AOC states that no supplemental carbon is required at any time during the year.

5.1.3 Mechanisms of Nitrogen Removal at WWTFs

For *aerated lagoon* WWTFs, like that in Exeter, there are several mechanisms for nitrogen removal, including algal uptake, solids settling (sludge deposition), adsorption by bottom sediments and to lesser extents nitrification, denitrification and volatilization. Total nitrogen removal at aerated lagoon WWTFs is seasonal, limited in effectiveness and typically occurs between June and October when conditions are favorable (i.e., not able to be positively controlled to a specific timeframe). The effluent concentrations from Exeter's WWTF, as shown in **Figure 5-1**, are typical of a lagoon facility and are significantly higher than the levels required by the AOC and the NPDES permit.

FIGURE 5-1
EFFLUENT TN CONCENTRATIONS



For *nitrogen removal WWTFs*, total nitrogen removal is accomplished through the use of two primary biological processes: nitrification and denitrification. When coupled together, influent nitrogen is reduced through either converting the influent nitrogen to nitrogen gas or converting and capturing it as a biological solid and "wasting" it out of the system. Total nitrogen removal at conventional WWTFs can be designed to work on a year round basis.

As noted above, biological nitrogen removal is a two-step process: nitrification followed by denitrification. The conversion of ammonia to nitrate is referred to as nitrification. This first step requires oxygen and alkalinity and, depending on wastewater temperatures and treatment process configuration, can convert most of the ammonia to nitrate. The conversion of nitrate to nitrogen gas is referred to as denitrification. This second step requires a carbon source in order for the bacteria to convert nitrate to nitrogen gas. Typically, this carbon source comes from the sewage itself; however, depending on influent characteristics and treatment process configuration, supplemental carbon (e.g., methanol) is sometimes necessary.

Denitrification processes can be grouped into two general categories – *exogenous* and *endogenous*. *Exogenous* denitrification processes utilize either the soluble carbon in the influent sewage or an external carbon source (e.g., methanol). *Endogenous* denitrification processes utilize the carbon released from the normal cell decay of the activated sludge biomass. Individually, exogenous or endogenous denitrification processes can achieve effluent total nitrogen levels in the range of 6.5 to 8 mg/l. When combined, exogenous and endogenous denitrification processes can achieve effluent total nitrogen levels in the range of 3.5 mg/l to 4 mg/l. The application of exogenous and endogenous are determined through aeration tank sizing and configuration.

For effluent total nitrogen limits of 5 mg/l and below, the *non-biodegradable* nitrogen fraction becomes very important. The non-biodegradable nitrogen fraction is a characteristic of the influent wastewater. Total nitrogen is the sum of multiple nitrogen components including ammonia, organic nitrogen, nitrate and nitrite. The dissolved organic nitrogen (DON) fraction is of particular concern. Effluent DON is primarily due to recalcitrant or hard-to-degrade forms of the influent nitrogen which can pass through the treatment plant unchanged. Typical municipal recalcitrant DON (rDON) levels range from 0.5 - 2.0 mg/l.

The effluent rDON value is a function of the influent wastewater characteristics, not the specific process employed at the facility to remove nitrogen. The remaining nitrogen components of the effluent total nitrogen are ammonia and nitrate/nitrite. The levels of these components are directly affected by the operation of the biological process. Advanced *non-biological* processes (e.g., carbon adsorption) may be required to remove the non-biodegradable organic nitrogen portion if effluent TN levels of 3.0 mg/l or less are required.

5.1.4 Basis for Cost Estimates

Regardless of which alternatives are implemented, the Town will be faced with costs in two categories. The first category is "capital cost", which include the cost to design and construct the needed facilities, including technical, legal and administrative costs. The second category is "operation and maintenance costs" (O&M costs), which include the on-going annual expenses to run the facilities.

For the ***regional WWTF alternatives analysis*** presented in **Section 5.2** below, capital and O&M were develop using standard cost estimating procedures consistent with industry standards for conceptual estimates. Costs for conveyance piping are based on conceptual layouts and unit cost information. Costs for the treatment plants and pump stations are based on the identified flow rate and planning-level cost curves. Unit costs for treatment facilities were taken from the Barnstable County Cost Report ("*Comparison of Costs for Wastewater Management Systems Applicable to Cape Cod*", April 2010). Once basic construction costs were estimated, allowances were added for contingencies and technical services, legal and administrative services (40%). Land acquisition costs were not evaluated at this time. Annual O&M costs were developed for each plan for the purposes of comparison among the plans. These planning-level costs were developed using the anticipated wastewater flow rates for each plan based on the O&M costs from the Barnstable County Cost Report (April 2010). All cost information presented herein is in current dollars. These estimates have been developed primarily for determining whether a regional solution is advantageous to Exeter. Conceptual cost estimates are based on limited technical information and have a broad range of accuracy (+40% to -25%).

For the ***on-site regional WWTF alternatives analysis*** presented in **Section 5.3** below, capital and O&M costs were developed using standard cost estimating procedures consistent with industry standards for planning-level estimates. Costs were developed by utilizing concept site-specific tank and building layouts and unit cost information. Once basic construction costs were estimated, allowances were added for contingencies and technical services, legal and administrative services (40%). Land acquisition costs were not evaluated because the WWTF in on Town land. Annual O&M costs were developed for each plan for the purposes of comparison among the plans. All cost information is presented in current dollars. These estimates have been developed primarily for evaluating alternative solutions and are generally reliable for determining the relative costs of various options. Planning-level costs are based on a greater level of technical information and have a more narrow range of accuracy (+30% to -10%).

5.1.5 Evaluative Criteria

Alternatives were evaluated based on the following cost and non-cost criteria:

- Reliability – The selected alternative must be able to reliably and consistently achieve the effluent limits and seasonal time frames. Reliability is the primary selection criteria.
- Flexibility – The selected alternative should provide for flexibility in the operation and maintenance of the facility given the daily and seasonal variations in flows, loads and effluent limits. All systems were targeted to have a similar level of flexibility, including the ability to add tertiary if future effluent limits are imposed (e.g., TN less than 3 mg/l).
- Life Cycle Cost – Life cycle cost, as measured by a “present worth analysis”, is a standard economic tool that allows for the calculation of a single “cost” to represent the combination of capital cost and annual expenses for operation and maintenance. In essence, the present worth represents the amount of money that one would invest at the beginning of the project to pay for the capital costs and to allow periodic withdrawals to pay the annual expenses over a certain period at a given interest rate.
- Operational Complexity – The existing lagoon system is a very simple operation and, to the extent possible, the upgraded facilities should not be unnecessarily complex.
- Phase-ability – The ability to phase elements of construction can improve the affordability of an alternative. The extent to which a process alternative provides the ability to phase or to defer (e.g., in the case of processes which reliably remove nitrogen to 5 mg/l) construction will be considered advantageous. The extent to which the incremental cost to upgrade from 8 mg/l to 3 mg/l is minimized will also be considered advantageous.

5.2 REGIONAL WASTEWATER ALTERNATIVES

5.2.1 Identification of Alternatives

At the outset of this project, the Town posed the question: ***would regional WWTF alternatives be more cost-effective than constructing an Exeter-only facility in Exeter?*** In order to address this question, a conceptual analysis was conducted for the following three broad alternatives:

1. A regional WWTF located in Exeter with effluent disposal to the Squamscott River;
2. A regional WWTF located in Exeter with effluent disposal to the Atlantic Ocean via a regional outfall shared with the Hampton WWTF; and
3. A regional WWTF located in Portsmouth (at the existing Pease WWTF) with effluent disposal to the Piscataqua River.

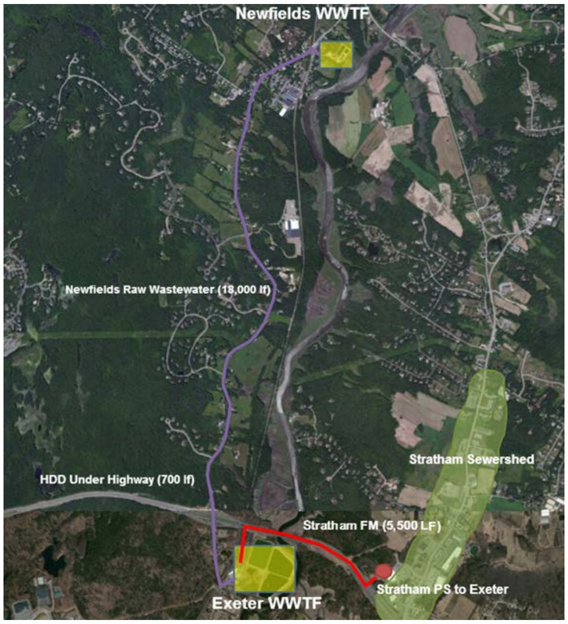
This analysis was completed in April 2014 and is reported herein to provide context for the remainder of this section.

In order to evaluate these alternatives, preliminary routing of conveyance piping (i.e., "transport to treatment" and "transport to disposal") was developed. Conceptual site figures for the regional alternatives are presented in **Figure 5-2**. Schematics of the regional alternatives are presented in **Figure 5-3**.

The sizing of conveyance, treatment and disposal systems were conceptualized based on projected wastewater flow rates from each community through the planning horizon (2040). The projected wastewater flows used in the analysis are summarized in **Table 5-1**, including source of the information. Actual sizing of treatment facilities could be tailored more closely to actual flow based on a phased construction approach and should be considered in more detail if one of these alternatives is selected.

FIGURE 5-2
LOCATION SCHEMATICS OF REGIONAL ALTERNATIVES

REGIONAL ALTERNATIVE 1



REGIONAL ALTERNATIVE 2

REGIONAL ALTERNATIVE 3

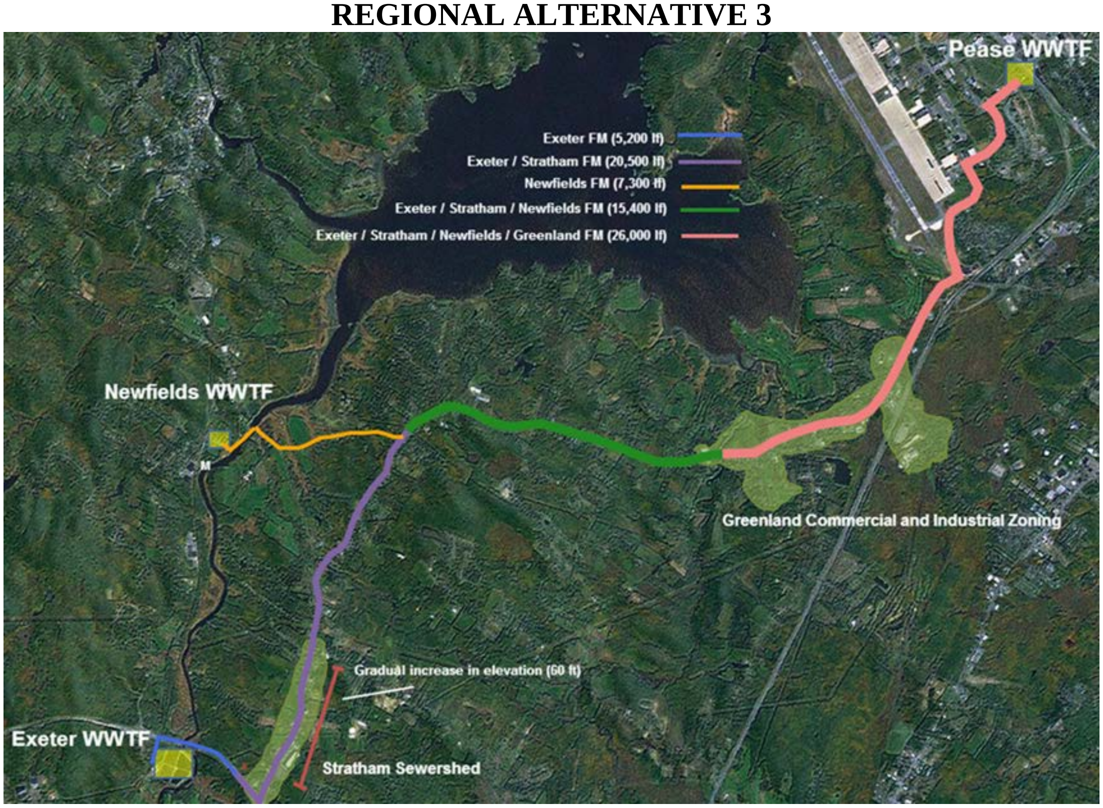


FIGURE 5-3
PROCESS SCHEMATICS OF REGIONAL ALTERNATIVES

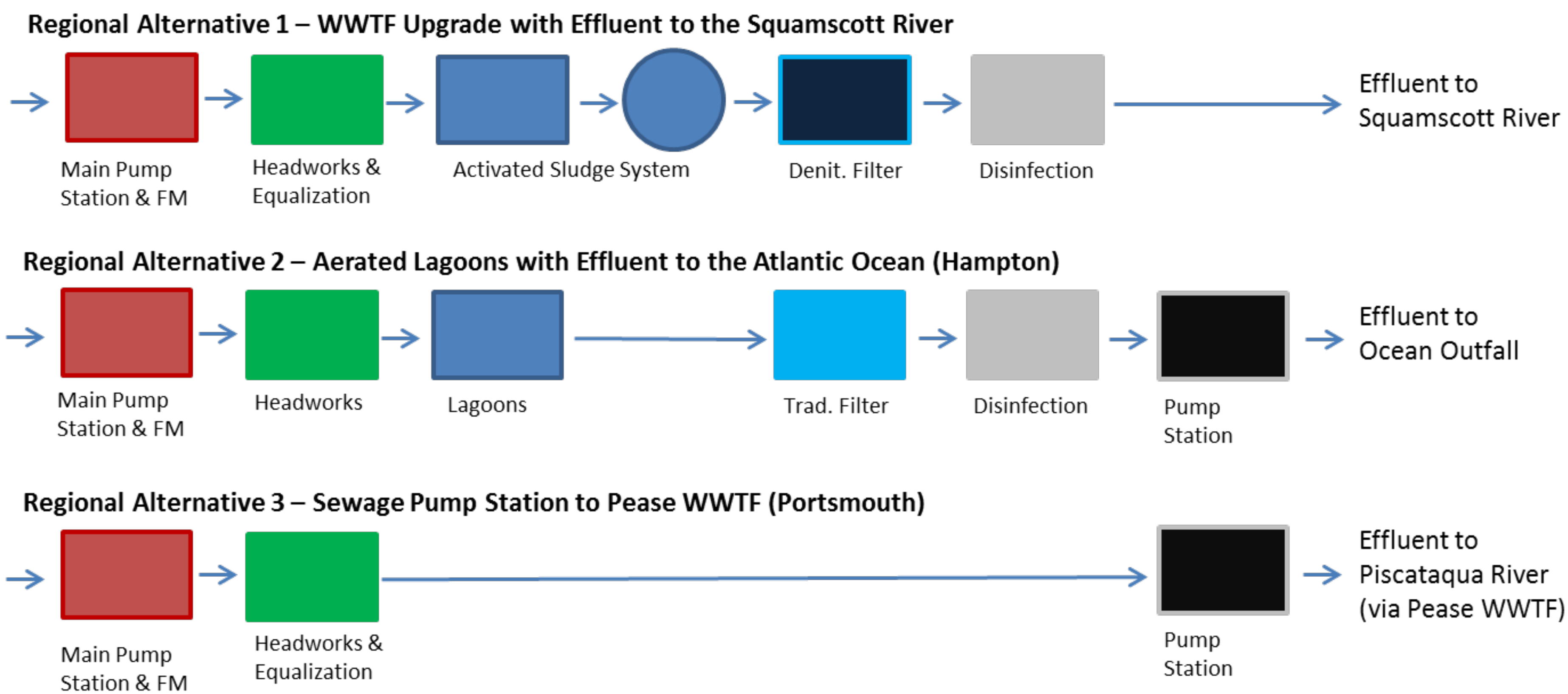


TABLE 5-1
SUMMARY OF PLANNING-LEVEL FLOWS BY TOWN

Town	Average Daily Flow (MGD)			Source
	Current	Planning Horizon	Build-out	
Exeter	1.70	2.40	2.60	Wright-Pierce (Section 2)
Stratham	0.17	0.55	0.66	Kleinfelder, 2012
Newfields	0.05	0.08	0.12	AECOM, 2005
Greenland	0.17	0.32	0.32	Portsmouth (B.Geotz, 2014)
Portsmouth/Pease	0.60	1.35	1.35	Portsmouth (B.Geotz, 2014)
Total – Alternative 1	1.92	3.03	3.38	
Total – Alternative 2	1.92	3.03	3.38	
Total – Alternative 3	2.69	4.70	5.05	

5.2.2 Regional Alternative 1: WWTF in Exeter with Effluent to Squamscott River

This alternative is summarized as follows:

- Communities involved: Exeter, Stratham, and Newfields.
- Collection system modifications:
  - Exeter: None.
  - Stratham: New pump station to Exeter WWTF.
  - Newfields: New forcemain from existing WWTF along Route 85 to Exeter WWTF.
- Exeter WWTF Modifications: Comprehensive upgrade including provisions for TN removal to 3-mg/l. Lagoon decommissioning would be required but is not included in this analysis.
- Newfields WWTF Modifications: Targeted upgraded to convert to a pump station. Lagoon decommissioning would be required but is not included in this analysis.
- Effluent Forcemain: None.
- Outfall: No modifications required.
- NPDES Permitting: Complete.

5.2.3 Regional Alternative 2: WWTF in Exeter with Effluent to Atlantic Ocean

This alternative is summarized as follows:

- Communities involved: Exeter, Stratham, Newfields, and Hampton.

- Collection system modifications:
    - Exeter: None.
    - Stratham: New pump station to Exeter WWTF.
    - Newfields: New forcemain from existing WWTF along Route 85 to Exeter WWTF.
    - Hampton: None.
- Exeter WWTF Modifications: Targeted upgrade of Exeter's WWTF including provisions for Headworks and Effluent Filtration (to capture algae from the lagoons). Upgrades for TN removal are not included. Lagoon decommissioning would not be required.
- Newfields WWTF Modifications: Targeted upgraded to convert to a pump station. Lagoon decommissioning would be required but is not included in this analysis.
- Hampton WWTF Modifications: None included.
- Effluent Forcemain: New forcemain from Exeter east on Route 101 where Hampton's effluent forcemain would merge to share a new outfall in the Atlantic Ocean. Hampton's existing effluent piping would require modifications to connect to the new forcemain. Provisions to minimize bacterial growth are not included in this analysis.
- Outfall: A new outfall with diffusers would need to be constructed approximately 1,500 linear feet offshore in the Atlantic Ocean. Hampton's 201 Facilities Plan Update (2006) showed two potential outfall locations – one off Winnacunnet Road and another off of Route 101. The outfall location will need to be carefully reviewed with Hampton, EPA, CLF, PREP and other interested stakeholders.
- NPDES Permitting: This option would require a new NPDES permit for the combined ocean discharge from Exeter WWTF and Hampton WWTF. Since this option would involve eliminating two NPDES permits upstream of Great Bay (Exeter and Newfields) and would relocate one NPDES permit out of a sensitive tidal creek (Hampton), EPA could view this option as a significant improvement. Further, it is assumed that an ocean outfall would be issued a NPDES permit without any effluent TN requirements. If TN removal is required, the WWTF costs will increase significantly. It is unknown at this time whether CLF, PREP, DES, EPA and others would support or oppose this option. Significant opposition would likely be put forward by residents in the vicinity of the ocean outfall.

5.2.4 Regional Alternative 3: WWTF in Portsmouth with Effluent to the Piscataqua River

This alternative is summarized as follows:

- Communities involved: Exeter, Stratham, Newfields, Greenland, and Portsmouth.
- Collection system modifications:
    - Exeter: Convey untreated wastewater via a new forcemain to the Pease WWTF.
    - Stratham: Connect to the Exeter FM along Route 108 in Stratham.
    - Newfields: Connect to the Exeter FM via Squamscott Road at Route 33.
    - Greenland: Connect to the Exeter FM along Route 33.
    - Portsmouth (Pease service area): None.
- Exeter WWTF Modifications: Targeted upgrade to improve the Headworks. Lagoon decommissioning would be required but is not included in this analysis.
- Newfields WWTF Modifications: Targeted upgraded to convert to a pump station. Lagoon decommissioning would be required but is not included in this analysis.
- Pease WWTF Modifications: Comprehensive upgrade to accommodate the significant increase in flow with TN removal to 8 mg/l (see below).
- Effluent Forcemain: Not applicable.
- Outfall Modifications: The existing Pease WWTF outfall would need to be increased in diameter and expanded to provide additional diffusers. The Pease WWTF shares its outfall with the Newington WWTF and any potential impacts would need to be mitigated.
- NPDES Permitting: The Pease WWTF currently has a NPDES permit for 1.2 MGD. This option would require that the NPDES permit be reissued for 4.7 MGD. The anti-degradation provisions of the Clean Water Act may preclude this as an option. Since this option would involve eliminating two NPDES permits upstream of Great Bay (Exeter and Newfields), EPA could view this approach as a significant improvement which could pre-empt the anti-degradation provisions. Further, it is assumed that this location would be issued a NPDES permit with an effluent TN of 8 mg/l (as opposed to 3 mg/l). It is unknown at this time whether CLF, PREP, DES, EPA and others would support or oppose this option. It is possible that EPA could require that the existing outfall diffusers be relocated a significant distance down-river. Costs for outfall relocation are not included in this analysis.

5.2.5 Comparison of Regional Alternatives

Capital and annual O&M estimates costs were developed for each alternative in April 2014 and are summarized on **Table 5-2**. A summary of the advantages/disadvantages of the regional alternatives is presented in **Table 5-3**. As noted in **Section 5.2.4**, several elements have not been included in the cost presented below (e.g., lagoon decommissioning, Main Pump Station and forcemain upgrades, etc.).

TABLE 5-2
ORDER OF MAGNITUDE COSTS FOR REGIONAL ALTERNATIVES (APRIL 2014)

	Alternative 1 Exeter WWTF/ Squamscott River	**Alternative 2 Exeter WWTF/ Hampton WWTF/ Atlantic Ocean**	**Alternative 3 Pease WWTF/ Piscataqua River**
Project Capital Cost			
Construction - Transport to Treatment	$5,500,000	$5,500,000	$25,500,000
Construction – Treatment	$29,100,000	$10,300,000	$31,800,000
Construction - Transport to Disposal	$0	$21,200,000	$1,000,000
Contingency, Admin, Legal & Technical Services	$13,800,000	$14,800,000	$23,300,000
Total Capital Cost	**$48,400,000**	**$51,800,000**	**$81,600,000**
Total Annual O&M Cost	**$3,420,000**	**$3,760,000**	**$5,830,000**
50-Year Present Worth of O&M	$73,500,00	$80,800,000	$125,200,000
Total 50-Year Present Worth	**$121,900,000**	**$132,600,000**	**$206,800,000**
Exeter/Stratham/Newfields Share	**$121,900,000**	**$119,300,000**	**$144,600,000**

Notes:

1) ENR CCI 9700 (April 2014).
2) Transport to treatment costs include the items identified is Section 5.2 above including new pump stations in Exeter, Newfields, Stratham and Greenland. Treatment and transport to disposal costs include the items identified in Section 5.2. No cost was carried for outfall extension for Alternative 3.
3) Contingency and technical services are based on 40% of the Construction costs.
4) Annual O&M Costs are intended to represent the total Sewer Enterprise Funds costs (i.e., entire sewer system, transport to treatment, treatment, effluent transmission and disposal) and not just the WWTF costs.
5) Present worth calculated based on 4% interest for 50 years.
6) The Exeter/Stratham/Newfields share of the present worth was calculated as 100% of the transport to treatment costs and the prorated costs for the other categories, based on flow.

TABLE 5-3
ADVANTAGES AND DISADVANTAGES OF REGIONAL ALTERNATIVES

Options	**Advantages**	**Disadvantages**
Exeter WWTF with Effluent to Squamscott River	• Can be completed within the AOC schedule • Lowest total cost. • Middle "cost share for Exeter" • NPDES permitting is completed	• Even with the WWTF upgrade, some nitrogen remains in the Squamscott River and goes to Great Bay.
Exeter WWTF with Effluent to Atlantic Ocean (Hampton)	• Middle total cost • Lowest "cost share for Exeter" • 100% point source TN removal from Squamscott River (Exeter and Newfields WWTFs)	• Greatest difficulty inter-municipal hurdles. • Will likely have strong opposition from the public in Hampton. • Only cost effective if TN removal is not required in Exeter. • Will likely require AOC schedule modification • Multiple communities involved, potentially making consensus more difficult. • May not be able to secure a NPDES permit.
Pease WWTF with Effluent to Piscataqua River	• Inter-municipal hurdles can likely be surmounted if all parties are amenable. • 100% point source TN removal from Squamscott River (Exeter and Newfields WWTFs)	• Highest total capital cost. • Highest "cost share for Exeter" • Greatest number of inter-municipal hurdles. • Will likely require AOC schedule modification • Loss of "local control" for Exeter. • Multiple communities involved, potentially making consensus more difficult. • May not be able to secure a NPDES permit modification. May require costly outfall extension to Portsmouth Harbor (now or in the future) Future regulatory uncertainty related to 303(d) listing of Portsmouth Harbor.

5.2.6 Next Steps for Regional Alternatives

Based on this analysis, the most cost-effective approach was Regional Alternative 2 (Hampton); however, the Town decided that the technical, political and regulatory hurdles associated with this alternative were substantial and has decided not to pursue this alternative any further. The next most cost-effective approach was Regional Alternative 1 (Exeter), which also has the least political and regulatory uncertainty. It is possible that the Exeter WWTF may not need to achieve 3 mg/l total nitrogen at its WWTF, which would reduce the cost of this alternative. Regional Alternative 3 (Pease) has the highest cost for Exeter and has considerable technical, political, regulatory and cost uncertainty. It is possible that the Pease WWTF may need to achieve better than 8 mg/l total nitrogen or extend the outfall to Portsmouth Harbor, which would increase the cost of this alternative. Also, while Regional Alternative 3 would undoubtedly benefit Great Bay, it will have an as-of-yet unquantified impact on the Piscataqua River and Portsmouth Harbor (i.e., due to less natural attenuation).

It is important to note that there are three separate studies currently on-going which address regional wastewater management. These are identified below:

- The Town commissioned a separate study, initiated Spring 2014, to develop a more detailed analysis of Regional Alternative 3 (Pease). This separate study is expected to be completed in October 2014.
- The Town is participating in the WISE project, initiated Fall 2013, which is assessing the costs and benefits associated with non-point source nitrogen management. This separate study is also expected to be completed in October 2014.
- The City of Portsmouth recently commissioned a separate study, initiated September 2014, to analysis another regional alternative (i.e. upgrading the Pease WWTF to also incorporate all wastewater flow from the City, thereby increasing the target Pease WWTF flow to greater than 10 mgd). The City has commissioned a separate study to develop this alternative. This separate study is also expected to be completed in January 2015.

A final decision on the cost-effectiveness of regional alternatives should be made with these additional studies in-hand.

5.3 ON-SITE NUTRIENT REMOVAL ALTERNATIVES

5.3.1 General

As noted previously, the purpose of this analysis is to select the on-site nutrient removal alternatives for the WWTF upgrade. A number of items have been considered "baseline" or "common" elements between the alternatives. These items are summarized below.

- Influent Equalization – The existing aerated lagoons are large and offer a low cost opportunity to convert a portion of these lagoons to off-line influent equalization. This will allow the Town to increase the capacity of the Main Pump Station in order to convey higher peak flows from the collection system to the WWTF without increasing the size of the WWTF. Using "peak shaving" approach, flow will be diverted into the basin during high flow events and will be conveyed back into the process after peak flows subside. Based on our calculations, 2 million gallons of influent equalization volume will allow for the peak instantaneous flow for the WWTF to be reduced from 13 MGD to 6.6 MGD. We have utilized 6.6 MGD peak instantaneous and peak day flow for the each of the on-site nutrient removal alternatives.

- Primary Clarification – There are no definitive requirements in the NHDES design regulations or in TR-16 (Guides for the Design of Wastewater Treatment Works, NEIWPCC, 2011) regarding whether primary clarifiers should be provided for a facility of this size. For the purposes of this planning-level analysis, we have elected to not include primary clarifiers in the treatment process based on our experience with similar sized facility, on our biological process modeling (described later in this section), and on the desire to eliminate the additional complexity that comes with primary treatment (additional tanks, equipment and sludge waste stream). We have left space on the site and in the preliminary hydraulic profile to include two primary clarifiers in the future (if desired). This decision does not impact the cost-effectiveness of the various nutrient removal alternatives relative to each other. If primary clarifiers were included, the WWTF would be incrementally more complex to operate but the nutrient removal activated sludge components would be smaller. This decision can be revisited in the preliminary design phase.

- Number of Nutrient Removal Treatment Trains – There are no definitive requirements in the NHDES design regulations or in TR-16 regarding the number of treatment trains required for the activated sludge systems (aeration tanks and secondary clarifiers; SBRs). The NHDES design regulations do require that three independent secondary clarifiers be provided for facilities with design average daily flows greater than 5 MGD. For purposes of this planning-level analysis, we have selected three treatment trains based on our experience with similar sized facilities, on the stringent nitrogen limits (more treatment units will allow for more precise control and "turndown") and on the ability to construct the facility in phases (e.g., two treatment trains initially, with a third in the future). This allows for a phasing and/or cost saving opportunity if needed. This decision can be revisited in the preliminary design phase.

- Separate Stage Tertiary Nitrogen Removal - There are no definitive requirements in the NHDES design regulations or in TR-16 regarding the number of treatment trains required for separate stage nitrogen removal (denitrification filters). All of the treatment processes identified herein will require separate stage tertiary treatment to achieve the ultimate effluent limit of 3.0 mg/l identified in the NPDES permit, if or when required. There are two main types of tertiary filtration processes for consideration; (1) biologically active filters and (2) traditional, non-biologically active filters. The type of filter required is determined by the level of treatment that occurs upstream of the filters. A biologically active filter (referred to herein as "denitrification filters") is a generic term for solids separation/filtration process that also includes bacteria attached to the filtration media. These filters will remove solids as well as convert nitrate to nitrogen gas for further nitrogen removal. These filters are typically capable of reducing the effluent nitrogen of nitrified wastewater to 3.0 mg/l.

  A non-biologically active filter (referred to herein as a "traditional filter") removes solids and does not provide any biological treatment. A modest 0.5 mg/l nitrogen reduction is expected with this treatment system. In general, these filters are significantly less complicated and less expensive to construct and operate than biologically active filters, but have limited nitrogen removal capacity. These filters must be paired with an upstream biological process that fully nitrifies and denitrifies.

The decision regarding the need for, and type of, tertiary treatment approach will be best determined once the new WWTF is on-line, the upgraded effluent quality can be assessed and the range of tertiary treatment equipment systems can be pilot-tested, as necessary. The timing of this will be determined in accordance with the AOC.

Potential cost saving opportunities as well as phasing opportunities are identified where appropriate herein and will be explored in greater detail in **Section 6**.

5.3.2 Identification of Alternatives for Nitrogen Removal

A broad array of technologies has been used to perform nitrogen removal at municipal wastewater treatment facilities. Common and less common technologies are listed below.

More Common	Less Common
Modified Ludzack-Ettinger (MLE)	Moving Bed Bioreactor (MBBR)
Four-Stage Bardenpho	Biolac
Sequencing Batch Reactor (SBR)	BioMag
Oxidation Ditch	Rotating Biological Contactors (Aerobic/Anoxic)
Schreiber Cyclic Aeration	De-ammonification
Integrated Fixed Film Activated Sludge (IFAS)	Trickling Filters
Membrane Bioreactors (MBR)	Breakpoint Chlorination
Denitrification Filters	Air Stripping

In terms of identifying a shortlist of processes for evaluation for the Exeter WWTF, we used the key criteria identified earlier in this section. Several of these processes are eliminated due to high life cycle cost (e.g., air stripping and breakpoint chlorination) and reliability for stringent nitrification/denitrification limits (e.g., RBCs, trickling filters). Several of these processes were eliminated because they are high-rate processes that are typically only cost effective on space-constrained sites (e.g., IFAS, MBBR, BAF, MBR, BioMag). The oxidation ditch process requires a very large amount of space and is less flexible than the remaining processes.

The process configurations selected for facility-specific evaluation to achieve 3 mg/l effluent total nitrogen are as follows:

1. Modified Ludzack Ettinger with Denitrification Filter
2. Four-Stage Bardenpho with Traditional Filter
3. Sequencing Batch Reactors with Denitrification Filter
4. Biolac with Denitrification Filter

Each process configuration will be arranged to allow for phased implementation in order to achieve an effluent total nitrogen limit of 8 mg/l in the near-term and 3 mg/l, if required by EPA in the longer-term. Each configuration is described in the following subsections.

For the purposes of this analysis, we have included a supplemental alkalinity system and a supplemental carbon system for all process configurations. Upgrade items which are required for an effluent total nitrogen of 8 mg/l are indicated in regular font whereas upgrade items which are required for an effluent total nitrogen of 3 mg/l are indicated in *italic font*.

5.3.3 Biological Process Modeling

A "steady-state" computer process model was developed in BioWIN 3.1 in order to analyze two process alternatives: the Modified-Ludzack Ettinger (MLE) process (exogenous) and the Four-Stage Bardenpho process (exogenous/endogenous). The modeling effort used the following key inputs and assumptions:

- Since the MLE and Bardenpho processes do not currently exist at the Exeter WWTF, it is not possible to develop a calibrated model; accordingly, default kinetic and stoichiometric process parameters were utilized. In some cases, default parameters were adjusted based on experience. The model results are used primarily as a tool to analyze applicable upgrade options.
- The model incorporated site-specific influent flow and load data as well as site-specific process tank sizing and configurations. A long-term operational record of the influent wastewater temperature was not available; however, the influent wastewater temperature

was set at 10 degrees C to simulate spring conditions. The aerobic solids retention time was held at 12 days for each process configuration to provide for complete nitrification at 10°C.

- Typical dissolved oxygen levels were set at 2.0 mg/l under annual average and maximum month conditions, with a minimum value of 1.0 mg/l under peak day loads.
- Peak daily and peak hourly flows were capped at 6.6 MGD based on the assumption that influent equalization will be incorporated at the WWTF.
- The MLE process was sized to produce 8 mg/l effluent total nitrogen at design annual average flows and up to 9.3 mg/l effluent total nitrogen at maximum month conditions. The Bardenpho process was sized to produce 3.5 mg/l effluent total nitrogen at design annual average flows and up to 3.8 mg/l effluent total nitrogen at maximum month conditions.
- A separate stage denitrification filter will be required for the MLE process to reliably achieve the 3 mg/l effluent total nitrogen limit; whereas a separate stage traditional filter will be required for the Bardenpho process.

Key conclusions from this modeling effort include:

- Supplemental alkalinity is required for the MLE and Bardenpho processes.
- Supplemental carbon is required for the Bardenpho process to achieve 3.5 mg/l effluent total nitrogen with the 0.56 million gallons of post-anoxic tankage modeled. If the post-anoxic volume were increased to 1.15 million gallons, the Bardenpho process could achieve 5 mg/l without supplemental carbon. A cost-benefit analysis can be conducted to determine which approach is preferable during the preliminary design phase.

An abbreviated summary of the model outputs are shown in **Table 5-4 and Table 5-5**. A technical memorandum summarizing the modeling effort is included as **Appendix B**.

TABLE 5-4
PROCESS MODEL OUTPUT – MLE ALTERNATIVE

	Annual Average (2 Trains Online)	**Annual Average** (3 Trains Online)	**Max Month** (3 Trains Online)
Aeration Tanks			
No. of Trains	3	3	3
No. of Zones per Train	2	2	2
Total Volume (MG)	1.47	2.20	2.20
Pre-Anoxic Volume (MG)	0.37	0.55	0.55
HRT (hr)	11.74	17.60	10.56
Aerobic SRT (days)	8.00	12.00	12.00
MLSS (mg/l)	2920	1950	4140
Supp. Alkalinity (lb/d $CaCO_3$)	1,500	1,500	2,500
Supp. Carbon (methanol gpd)	0	0	0
Secondary Clarifiers			
Tanks Online	2	3	3
Diameter (ft)	75	75	75
Depth (ft)	16	16	16
Effluent Quality			
Effluent BOD5 (mg/l)	3.5	3.2	3.8
Effluent TN (mg/l)	8.0	8.0	9.3
Effluent TN (lbs/day)	197	197	384
Effluent P (mg/l)	3.1	3.1	2.6
Effluent TSS (mg/l)	7.7	7.2	9.5
Waste Activated Sludge			
WAS (lb/d)	3,352	3,360	4,753

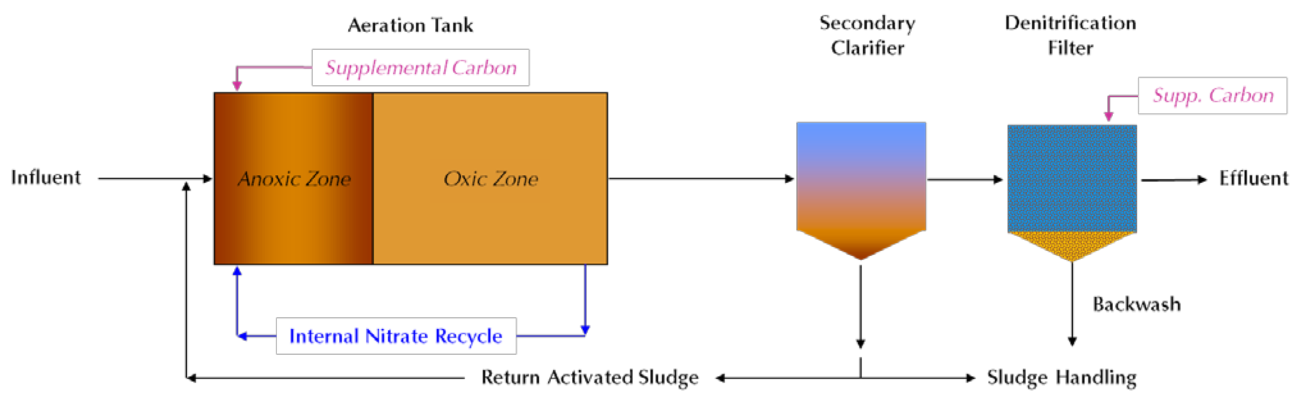
TABLE 5-5
PROCESS MODEL OUTPUT - FOUR-STAGE BARDENPHO ALTERNATIVE

	Annual Average (2 Trains Online)	**Annual Average** (3 Trains Online)	**Max Month** (3 Trains Online)
Aeration Tanks			
No. of Trains	3	3	3
No. of Zones per Train	4	4	4
Total Volume (MG)	1.86	2.78	2.78
Pre-Anoxic Volume (MG)	0.37	0.55	0.55
Post-Anoxic Volume (MG)	0.37	0.56	0.56
HRT (hr)	14.84	22.26	13.33
Aerobic SRT (days)	8.00	8.00	12.00
MLSS (mg/l)	3310	2020	4110
Supp. Alkalinity (lb/d $CaCO_3$)	1,750	1,750	2,550
Supp. Carbon (methanol gpd)	100	100	25
Secondary Clarifier			
Tanks Online	2	3	3
Diameter (ft)	75	75	75
Depth (ft)	16	16	16
Effluent Quality			
Effluent BOD5 (mg/l)	3.4	2.4	3.0
Effluent TN (mg/l)	3.5	3.5	3.8
Effluent TN (lbs/day)	74	74	155
Effluent P (mg/l)	3.3	2.9	2.6
Effluent TSS (mg/l)	8.1	4.5	9.4
Waste Activated Sludge			
WAS (lb/d)	3,380	3,538	4,699

5.3.4 On-Site Alternative 1: Modified Ludzack-Ettinger with Denitrification Filter

The MLE process is similar to a traditional activated sludge system but with anoxic zones preceding the oxic (aerobic) zones. Influent wastewater which provides organic carbon and return activated sludge (RAS) which provides biomass are fed into the anoxic zone. Internal mixing recycles wastewater from the aerobic zone to the anoxic zone. The process flow diagram is shown in **Figure 5-4**.

FIGURE 5-4
MLE PROCESS SCHEMATIC



To achieve biological nitrogen removal, ammonia must first be completely transformed to nitrate (via nitrification) in the aerobic zone of the activated sludge system. Nitrates produced in the aerobic zone are then recycled back to the anoxic zone through a pumped internal recycle system allowing them to come in contact with soluble BOD from the influent, thus creating an environment conducive for denitrification.

The limit of technology for the MLE process is typically considered between 6 to 10 mg/l of effluent total nitrogen. The effluent total nitrogen level achieved is highly dependent on the amount of influent substrate carbon available for the denitrification process. Increasing the influent carbon to nitrogen ratio typically results in improved performance. To achieve effluent TN less than 3.0 mg/l, a denitrification filter and supplemental carbon system are required.

This option would consist of the following major components:

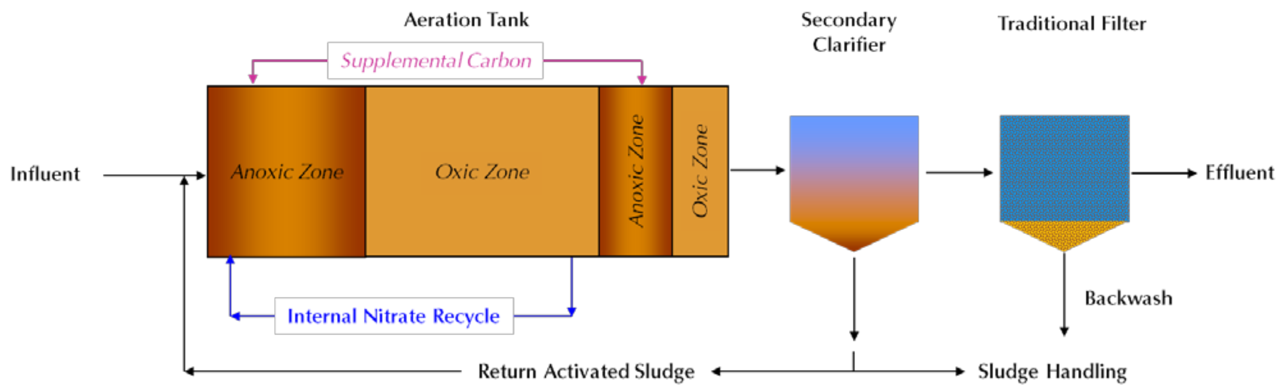
a. Flow splitter box to distribute flow between treatment tanks
b. Three concrete tanks for the activated sludge treatment process, with a total volume of 2.2 million gallons. Treatment tanks will be configured with an aeration tank component partitioned into anoxic and oxic zones. Anoxic zones will have submersible mixers. The oxic zones will have an internal recycle pump to recycle nitrate rich mixed liquor to the anoxic zone for denitrification.
c. Three 75-foot diameter secondary clarifiers and influent splitter box, with a total volume of 1.6 million gallons.
d. Supplemental alkalinity storage and feed system.
e. *Tertiary denitrification filter system and supplemental carbon storage and feed system (if an effluent TN limit of 5 mg/l or less is imposed).*

5.3.5 On-Site Alternative 2: Four-Stage Bardenpho with Traditional Filter

The 4-stage Bardenpho process is similar to the MLE process. It includes a primary anoxic zone, primary oxic (aerobic) zone, secondary anoxic zone, and reaeration zone in series as shown in **Figure 5-5**. The first anoxic zone and aerobic zone work the same as the MLE process. Nitrates are recycled from the effluent end of the first aerobic zone to the first anoxic zone. A second anoxic zone is provided after the aerobic zone for additional denitrification through biomass degradation to further reduce the effluent total nitrogen. The re-aeration zone at the end is provided to add dissolved oxygen to the wastewater prior to the secondary clarifiers. A supplemental carbon source is typically utilized in the second anoxic zone to provide sufficient substrate (carbon) to complete denitrification.

The limit of technology for the 4-stage Bardenpho process is considered to be 3.5 to 4.5 mg/L of effluent total nitrogen, depending on recalcitrant (non-degradable) organic nitrogen in the wastewater as well as effluent particulate nitrogen levels. To achieve a TN less than 8.0 mg/l, a supplemental carbon system is required. To achieve effluent TN less than 3.0 mg/l, a traditional filter system required.

FIGURE 5-5
4-STAGE BARDENPHO PROCESS SCHEMATIC



This option would consist of the following major components:

a. Flow splitter box to distribute flow between treatment tanks

b. Three concrete tanks for treatment process, with total volume of 2.8 million gallons. Treatment tanks will be configured with an aeration tank component partitioned into anoxic and oxic zones. Anoxic zones will have submersible mixers. The oxic zones will have an internal recycle pump to recycle nitrate rich mixed liquor to the anoxic zone for denitrification. Following the oxic zone is an additional anoxic zone to further denitrify and a reaeration zone to add oxygen to the tank effluent. Consider provisions for step feed of aeration tank influent to the secondary anoxic zone as a carbon source.

c. Three 75-foot diameter secondary clarifiers and influent splitter box, with a total volume of 1.6 million gallons.

d. Supplemental alkalinity storage and feed system

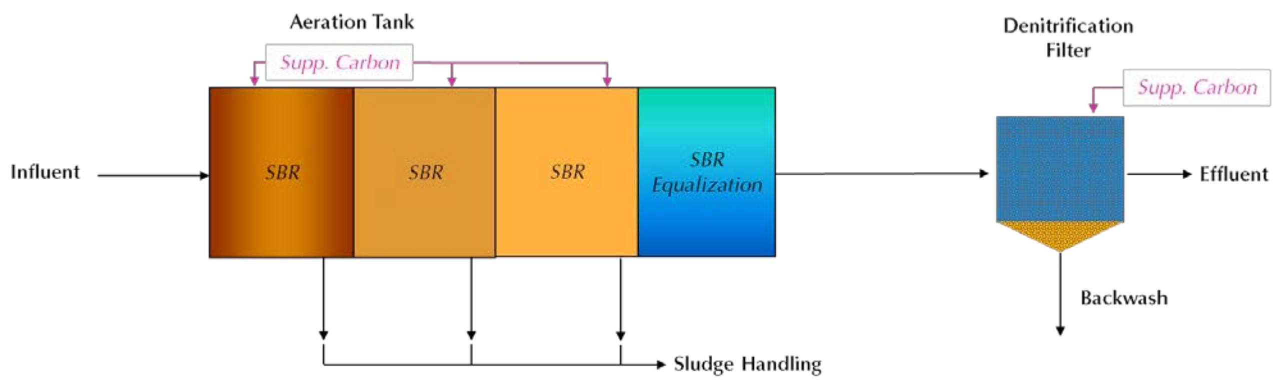
e. Supplemental carbon storage and feed system for the post-anoxic zone (if an effluent TN limit of 5 mg/l or less is imposed)

f. Traditional filter system (if an effluent TN limit of 3 mg/l or less is imposed)..

5.3.6 On-Site Alternative 3: Sequencing Batch Reactors with Denitrification Filters

The SBR activated sludge process utilizes a common tank for both aeration and clarification. SBR systems have five steps in common, which are carried out in sequence as follows: (1) fill, (2) react (aeration), (3) settle (sedimentation/clarification), (4) draw (decant), and (5) idle. Given the size of the facility, three SBRs are recommended to effectively treat influent wastewater and carryout nitrification/denitrification at the Exeter WWTF. Since aeration and clarification occurs in the same tank, the SBR process does not require secondary clarifiers; however, treated flows must be equalized after decanting to avoid the need to oversize downstream processes. To denitrify, the reaction stage alternates between aerobic and anoxic conditions by controlling the dissolved oxygen concentration within the SBR. A typical SBR process is shown in **Figure 5-6**.

FIGURE 5-6
SBR PROCESS SCHEMATIC



The limit of technology for the SBR process is considered to be 5.0 to 6.0 mg/L of effluent total nitrogen, depending on recalcitrant (non-degradable) organic nitrogen in the wastewater as well as effluent particulate nitrogen levels. To achieve effluent TN less than 3.0 mg/l, a denitrification filter and supplemental carbon system are required. SBR manufacturers indicate that 3.0 mg/l effluent nitrogen can be achieved with a traditional filter.

This option would consist of the following major components:

a. Flow splitter box to distribute flow between treatment tanks
b. Three concrete tanks for the SBRs, with a total volume of 5.3 million gallons. Treatment tanks will include installation of the SBR equipment including diffuser assemblies, mixers, transfer pumps, and decanters
c. Secondary equalization tank or basin (0.3 million gallons) and equipment including coarse diffusers and effluent transfer pumps.

d. Supplemental alkalinity storage and feed system.

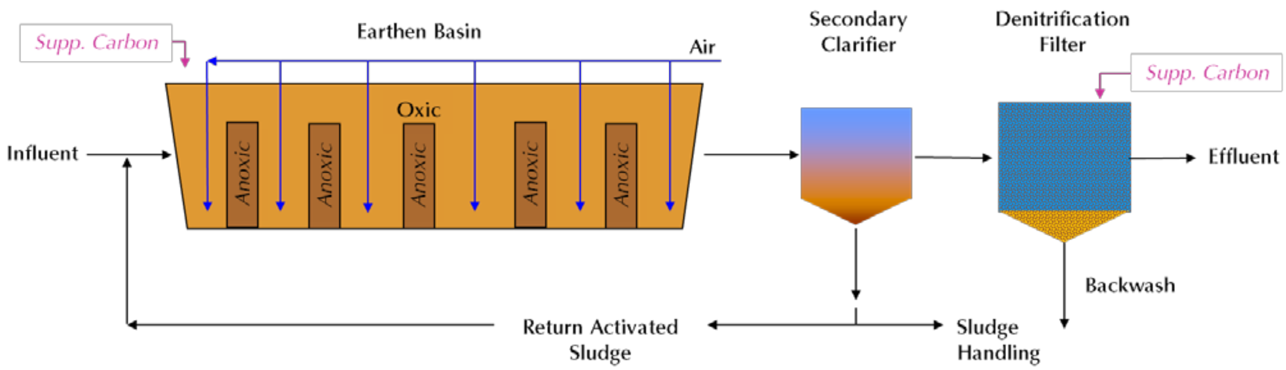
e. Supplemental carbon storage and feed system (if an effluent TN limit of 5 mg/l or less is imposed).

f. Tertiary treatment (denitrification or traditional) filter system (if an effluent TN limit of 3 mg/l or less is imposed).

5.3.7 On-Site Alternative 4: Biolac® with Denitrification Filters

Biolac® is an activated sludge system adapted for construction with an earthen basin. Oxygen is delivered to the wastewater via fine bubble membrane diffusers attached to diffuser assemblies and floating aeration chains. The aeration chains suspend the diffusers above the bottom of the basin without coming in contact with it. Mixing is provided by the diffuser assemblies which are moved back and forth from the force of the oxygen. Denitrification is achieved through a cyclic aeration process called Wave-Oxidation® (WaveOx) which alternates air flow distribution from the aeration chains creating multiple aerobic and anoxic zones within the treatment basin as shown in **Figure 5-7**.

FIGURE 5-7
BIOLAC® PROCESS SCHEMATIC



The limit of technology for the Biolac process is considered to be 8.0 mg/L of effluent total nitrogen. To achieve effluent TN less than 3.0 mg/l, a denitrification filter and supplemental carbon system are required.

This option would consist of the following major components:

a. Flow splitter box to distribute flow between treatment zones
b. New lagoon liner
c. Concrete walls (long axis) and earthen walls (short axes) to create three separate basins, with a total volume of 5.0 million gallons. Treatment basins will include installation of Biolac equipment including moving aeration chain, diffuser assemblies and controls.
d. Three 75-foot diameter secondary clarifiers and influent splitter box, with a total volume of 1.6 million gallons.
e. Supplemental alkalinity storage and feed system
f. Tertiary denitrification filter system and supplemental carbon storage and feed system (if an effluent TN limit of 5 mg/l or less is imposed).

5.3.8 Comparison of Regional On-Site Alternatives

A planning-level analysis was performed for each of the nitrogen removal options. Each option was developed to a consistent level of conservatism based on the future wastewater flows and loads presented in Section 3 of this report and based on the near-term effluent total nitrogen of 8 mg/l and future effluent total nitrogen of 3 mg/l. Each option was assumed to require a supplemental carbon system and tertiary denitrification filter to achieve a TN limit of 3.0 mg/l. A summary of the advantages and disadvantages of each option is presented in **Table 5-6**.

Planning-level capital cost and annual operations and maintenance cost estimates were developed for each of the options in the manner described in **Section 6** of this report. A summary of these costs are presented in current dollars in **Table 5-7**.

TABLE 5-6
ADVANTAGES AND DISADVANTAGES OF ON-SITE ALTERNATIVES

Alternatives	Advantages	Disadvantages
MLE with Denitrification Filters	• Common in cold-weather climate	• May have difficulty meeting 8 mg/l limit at future flows and loads if influent TKN trends higher. • More complex operation than Bardenpho or SBR if 5 mg/l limit is imposed.
Four-Stage Bardenpho with Traditional Filters	• Easily achieves TN less than 8 mg/l • Common in cold-weather climate • Tertiary upgrade may be avoided.	• Greatest complexity of the four alternatives
Sequencing Batch Reactors with Denitrification Filters	• Simplest operation if PLCs operational. • Greatest degree of automation • Does not require clarifiers (but does require secondary equalization) • Easily achieves TN less than 8 mg/l • Common in cold-weather climate • Tertiary upgrade may be avoided.	• Complex operation if PLC controllers fail.l
Biolac (Earthen Basin) with Denitrification Filters	• Similar to existing operation • Reuses existing lagoon basin	• Achieving 8 mg/l requires more operator awareness and input than Four-Stage Bardenpho or SBRs. • More likely to have foaming and filamentous issues than other processes evaluated • Relatively few installations of similar size with stringent TN limits in cold-weather climate • Requires an intermediate pump station between secondary clarifiers and denitrification filters because hydraulic gradeline is fixed at existing lagoon water surface. • More complex operation than Bardenpho or SBR if 5 mg/l limit is imposed.

TABLE 5-7
COSTS FOR ON-SITE NITROGEN REMOVAL ALTERNATIVES

Alternatives	**Total Nitrogen 8 mg/l**			**Total Nitrogen 3 mg/l**		
	Capital Cost ($$M)	Present Worth ($$M)	Present Worth per Pound TN Removed	Capital Cost ($$M)	Present Worth ($$M)	Present Worth per Pound TN Removed
1 – Modified Ludzack-Ettinger with Denitrification Filters	$19.7M	$25.3M	$231	$28.1M	$35.9M	$231
2 – Four-Stage Bardenpho with Traditional Filters	$22.8M	$29.3M	$200	**$27.1M**	**$35.0M**	**$225**
3 – Sequencing Batch Reactor with Denitrification Filters	$19.9M	$26.0M	**$190**	$28.3M	$36.0M	$233
4 – Biolac (Earthen Basin) with Denitrification Filters	**$18.8M**	**$25.0M**	$228	$29.3M	$37.8M	$243

Notes:
1) ENR CCI 9700.
2) Capital costs include an allowance for contingency and technical services (40%) for only the Biological Nutrient Removal portions of the project. Additional project costs associated with other portions of the WWTF (e.g., headworks, disinfection, biosolids, etc.) are not included. The remainder of the project costs are addressed elsewhere is Section 5 and in Section 6.
3) Present worth is based on 20 years at 4% interest and include only the “incremental annual O&M costs” for the BNR portions of the project. The remainder of the project costs are addressed elsewhere is Section 5 and in Section 6.
4) Present worth per pound of TN removed is based on the difference between the “baseline” (existing lagoon WWTF at 3.0-mgd and 20 mg/l effluent TN) vs each alternative at 3.0-mgd at an effluent TN concentration of 8 mg/l for MLE and Biolac, 5 mg/l for SBR and 4 mg/l for Bardenpho for the “Total Nitrogen 8 mg/l” columns. For the “Total Nitrogen 3 mg/l”, all alternatives utilized 3 mg/l effluent TN.
5) Bold indicates the lowest cost per column.

The alternatives were assessed qualitatively based on the evaluative criteria identified in Section 5.1.5. Key factors for each evaluative criterion were identified (e.g., present worth per pound of TN removed, similar installations, etc.). Since each of the alternatives was provided "a pathway to 3-mg/l effluent TN", phasing criteria were not included in the analysis. A value of "advantage" (A), "neutral" (N) or "disadvantage" (D) was assigned for each alternative and each criteria for each level of nitrogen removed. A summary of this analysis is presented below on **Table 5-8**.

TABLE 5-8
QUALITATIVE ANALYSIS OF ON-SITE NITROGEN REMOVAL ALTERNATIVES

	Cost			Reliability/ Flexibility			Operations			
Alternative	Capital Cost	Present Worth	Present Worth per Pound of TN Removed	Similar Installations (Size, Low TN & Cold Temps)	Flexibility	Ability to Add Primary Clarifiers in the Future	Complexity (Simplicity is an advantage)	Energy Use (less is an advantage)	Chemical Use (less is an advantage)	Requires Effluent Pump Station
For 8 mg/l TN (AOC Compliance)										
1 - MLE	A	A	N	N	N	N	N	N	N	N
2 - Bardenpho	D	D	N	N	N	N	N	N	N	N
3 - SBR	N	N	N	N	N	N	A	N	N	N
4 - Biolac	A	A	N	N	N	N	N	N	N	N
For 5-mg/l TN										
1 - MLE	Cannot reliably achieve 5-mg/l TN									
2 - Bardenpho	D	N	A	A	N	N	D	D	A	N
3 - SBR	N	N	A	A	N	N	A	N	A	N
4 - Biolac	Cannot reliably achieve 5-mg/l TN									
For 3 mg/l TN (NPDES Compliance)										
1 - MLE plus Denit Filter	N	D	D	A	N	N	N	N	N	N
2 - Bardenpho plus Traditional Filter	A	A	A	A	N	N	N	N	A	N
3 - SBR plus Denit Filter	N	N	N	A	N	N	N	N	A	N
4 - Biolac plus Denit Filter	N	D	D	D	N	N	N	N	N	D

5.3.9 Recommended On-Site Nitrogen Removal Alternative

The following general conclusions are indicated:

- The alternatives present a broad range of capital costs, but have relatively similar present worth values. The systems with the higher capital cost have a lower annual O&M cost.
- The MLE, Bardenpho and SBR processes are widely used in the United States for similar sized facilities with stringent nitrogen limits and in cold-weather climates.
- Biolac has relatively few installations in the United States for this size facility with stringent nitrogen limits in a cold-weather climate. Biolac would be expected to have a greater temperature drop through the treatment process (due to its large surface area) which could result in reduced reliability to achieve low TN effluent in colder weather or colder wastewater months (more of a concern due to April permit limit).
- The Bardenpho and SBR options can both reliably achieve less than 5 mg/l effluent TN for the same costs presented under the 8 mg/l column. This is identified in the “capital cost per pound of TN removed” column.
- For 8 mg/l effluent TN, the lowest capital cost is Biolac; whereas, the highest is Bardenpho. Similarly, the lowest present worth is MLE and Biolac; whereas, the highest is Bardenpho.
- For 3 mg/l effluent TN, the lowest and present worth is Bardenpho.
- The lowest capital cost per pound of TN removed is Bardenpho and SBR; whereas the highest capital cost per pound of TN removed is MLE.
- There is a low incremental cost to achieve 5 mg/l with Bardenpho and SBR.
- The highest ranked alternatives for 8-mg/l TN is MLE or Biolac. The highest ranked alternative for 5-mg/l TN is SBR. The highest ranked alternative for 3-mg/l is Bardenpho.

Based on our review of the applicable technologies, including advantages, disadvantages and conceptual capital and operational costs, the recommended option is either On-Site Alternative No. 2 (Bardenpho) or On-Site Alternative No. 3 (SBR). These options will be carried forward into the facility-wide recommended plan, and the higher costs of On-Site Alternative No. 2, will be presented in **Section 6**.

5.4 BIOSOLIDS MANAGEMENT ALTERNATIVES

The Exeter WWTF currently stores all biosolids in the three aerated lagoons. No biosolids have ever been processed or disposed of from the three aerated lagoons. For the purpose of this analysis, it is assumed that the Exeter WWTF will be upgraded to one of the activated sludge treatment processes identified previously in this section and will require either mechanical thickening with liquid disposal, mechanical dewatering with solids disposal, or mechanical thickening followed by mechanical dewatering with solids disposal. The sludge quantities used in this analysis are summarized in **Table 5-9** below.

TABLE 5-9
CURRENT AND PROJECTED FUTURE SLUDGE PRODUCTION

SLUDGE PRODUCTION	CURRENT (2014)		DESIGN (2040)	
	MIN MONTH	AVERAGE	AVERAGE	MAX MONTH
Biological (lbs/d)[(1)]	1,456	2,240	3,360	4,753
Tertiary (lbs/d)[(2)]	0	0	1,170	1,949
Total (lbs/d)	1,456	2,240	4,530	6,702

Notes:
1. Biological sludge quantities were estimated in the Biowin process model.
2. Tertiary sludge quantities were estimated based on input from manufacturers. Under current conditions, the tertiary process would not be constructed.

5.4.1 Biosolids Alternative 1: Mechanical Thickening with Liquid Disposal

In this alternative, the waste sludge is assumed to be 0.6 percent solids and would be collected in three 150,000 gallon (450,000 gallon total) sludge storage tanks (SST). The SSTs would have provisions for mixing and aeration of the waste sludge. The waste sludge would be pumped out of the SSTs to two rotary drum thickeners (RDT) via two sludge feed pumps. The RDTs would utilize dilute polymer to flocculate the waste sludge delivered via a polymer activation system which would improve thickening.

RDTs are routinely used to thicken waste sludge to approximately 5 to 7 percent solids. Waste sludge would be thickened to 6 percent solids which would be stored in two 25,000 gallon (50,000 gallon total) thickened sludge storage tanks (TSST). The TSSTs would have a mixing system installed to keep the thickened waste sludge homogenous. The thickened waste sludge would then be pumped to a tanker for disposal by two thickened sludge pumps. The thickened waste sludge would then be trucked to a processing facility for disposal. For this analysis, it was assumed that the thickened waste sludge would be hauled and disposed of by Synagro at the Woonsocket Thermal Conversion Facility in Woonsocket, Rhode Island. ($0.20/Gal, personal communications with Synagro, 07/10/2014).

5.4.2 Biosolids Alternative 2: Mechanical Dewatering with Cake Disposal

In this alternative, the waste sludge is assumed to be 0.6 percent solids and would be collected in three 150,000 gallon (450,000 gallon total) SSTs. The SSTs would have provisions for mixing and aeration of the waste sludge. The SSTs would also have a decanting system which would thicken the waste sludge to approximately 1.5 percent solids. The decanted waste sludge would then be pumped by two sludge feed pumps to two mechanical dewatering systems (such as a screw press, centrifuge or rotary press). Dewatering systems utilize dilute polymer to promote flocculation of the waste sludge.

The dewatered waste sludge would then be conveyed via a conveyor system in to a hauling trailer, while the filtrate would be directed back to the headworks. The dewatered sludge could be disposed of as a solid waste at a secure landfill or could be post-processed for beneficial reuse. There are numerous beneficial reuse options for biosolids which have been post-processed to meet either Class A or B biosolids criteria (e.g., land application, topsoil amendments, composting, pellet fertilizers, etc.); which are often accomplished at an off-site facility by a contractor. For the purpose of this analysis, it was assumed that the dewatered sludge would be hauled to an off-site location for post-processing by a contractor for beneficial reuse ($100/wet ton, personal communication with RMI, 7/15/2014).

5.4.3 Biosolids Alternative 3: Mechanical Thickening, Mechanical Dewatering with Cake Disposal

In this alternative, the waste sludge assumed to be 0.6 percent solids would be thickened in two RDTs to 6 percent solids which would be stored in two 25,000 gallon (50,000 gallon total) thickened sludge storage tanks (TSST). The TSSTs would have a mixing system installed to keep the thickened sludge homogenous. The thickened sludge would then be pumped via two sludge feed pumps to two dewatering systems to be dewatered. The dewatered waste sludge would then be conveyed via a conveyor system in to a hauling trailer, which would be routinely hauled away. The filtrate from the thickening and dewatering systems would be directed back to headworks. For the purpose of this analysis, it was assumed that the dewatered sludge would be hauled to an off-site location for post-processing by a contractor for beneficial reuse ($100/wet ton, personal communication with RMI, 7/15/2014).

5.4.4 Comparison of Biosolids Alternatives

Capital cost, operation and maintenance costs and present worth were generated for each alternative. These are summarized in **Table 5-10** below.

TABLE 5-10
COSTS FOR BIOSOLIDS MANAGEMENT ALTERNATIVES

	Alternative 1 Mechanical Thickening Liquid Disposal	**Alternative 2** Mechanical Dewatering Cake Disposal	**Alternative 3** Mechanical Thickening Mechanical Dewatering Cake Disposal
Construction Cost(1)	$4,606,000	$5,529,000	$6,299,000
Annual O&M Costs			
Energy Cost(2)	$27,400	$28,600	$62,800
Disposal Cost(3)	$664,000	$378,000	$378,000
Polymer Cost(4)	$29,000	$71,000	$99,000
Maintenance Cost(5)	$17,700	$18,500	$36,600
Total O&M Cost	$738,100	$496,100	$576,400
Net Present Worth(6)	$14,638,000	$12,272,000	$14,133,000

Notes:
(1) Installation and electrical costs estimated at 20% of equipment cost each.
(2) Energy cost based on connected horsepower; average run time per year estimated as annual solids per equipment throughput capacity; average energy cost estimated at $0.14/kW-hr.
(3) Disposal costs based on budgetary pricing provided by Synagro ($0.20/gal) and RMI ($100/wet ton).
(4) RDT usage 10 lb active/dry ton; Screw Press usage 25 lb active/dry ton; Polymer cost estimated at $3.40/lb.
(5) Mechanical equipment maintenance cost based on 25% of operating hours at a labor cost of $40/hr.
(6) Present Worth is based on 20-year at 4.0% interest.
(7) ENR CCI 9700.

The advantages of Alternative 1 include the following:

- SST volume provides for 5 days of storage at future annual average sludge production
- Least complex operation and lowest capital cost

The disadvantages of Alternative 1 include the following:

- Highest net present worth, annual O&M costs, capital cost and truck trips per year
- Limited local disposal options for liquid sludge

The advantages of Alternative 2 include the following:

- Mid-complexity operation with the lowest net present worth and O&M costs.
- SST volume provides for 5 days of storage at future annual average sludge production
- There are several local disposal options for dewatered cake

The disadvantages of Alternative 2 include the following:

- None

The advantages of Alternative 3 include the following:

- Second lowest net present worth and O&M costs
- There are several local disposal options for dewatered cake

The disadvantages of Alternative 3 include the following:

- Smaller SST heightens dependence on proper O&M of the mechanical thickening system
- Most complex operation

5.4.5 Recommended Biosolids Alternative

Alternative 2 is the recommended biosolids management option based on having the lowest net present worth, the security of large sludge storage tanks and multiple local options to dispose of dewatered cake.

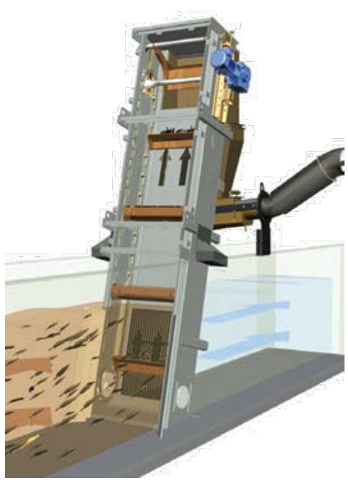
5.5 SCREENINGS AND GRIT REMOVAL ALTERNATIVES

The Main Pump Station was constructed during 1964 and included manual bar rack and detritor-type grit sump. The Town abandoned grit removal at the MPS in the mid-1980's due to regular clogging of the classifier. Grit still collects in the MPS grit collection sump and is removed monthly or when levels become problematic. Recently the MPS has been updated with two channel macerators, replacing the manual bar screen and previous channel macerator. The WWTF screenings and aerated grit system was constructed in 1998 and included manual bar rack and aerated grit removal. The WWTF aerated grit chamber and manual bar rack are still in operation but both systems have reached the end of their useful life and are recommended to be updated with any future upgrades to the facility. The purpose of this evaluation is to assess whether the Town should upgrade the screening and grit facility at the MPS or the WWTF. Two "location alternatives" were considered as part of the assessment for screenings and grit removal: constructing a new screenings/grit removal facility just upstream of the MPS; or constructing a new screenings/grit removal facility at the WWTF

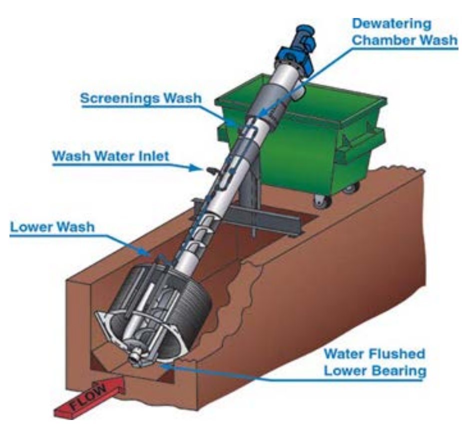
5.5.1 Screenings Equipment

Multiple equipment systems are applicable for either screening location. **Figure 5-8** below depicts typical screening systems.

FIGURE 5-8
SCREENING SYSTEMS



Vertical Screens



Rotary Drum Screens

Vertical Screens

There are numerous types of vertical screens, including climber screens; multi-rake screens and step screens. Climber screen employs a single rake arm connected to a cogwheel that rides up and down a pin rack located on the screen frame. Typically all moving parts are located above the waterline. Climber screens typically have 3/8” to 1/2”” bar spacing.

Multi-Rake Bar Screens have rakes attached to the dual chains to provide the screenings removal mechanism. A pair of sprockets are located in the bottom of the channel to provide for positive engagement of the rake to the bar screen. The chain rotates within the frame, reducing the overall size of the unit (height and length). Multi-rake screens typically have 1/4” to 3/8” bar spacing.

Step screens have a series of moving screen plates that rotate adjacent to a series of fixed screen plates. The moving screen plates move debris up the screen like an escalator. Typically all moving parts are located above the waterline. Step screens typically have 1/4” bar spacing.

Vertical screen systems typically discharge screenings into a screenings wash presses. The wash presses would wash out organics from the screenings to reduce odor potential and then be dewatered and compact the screenings. The dewatered screenings from each wash presses would be discharged through a discharge chute and into a screenings container.

Rotary Drum Fine Screen

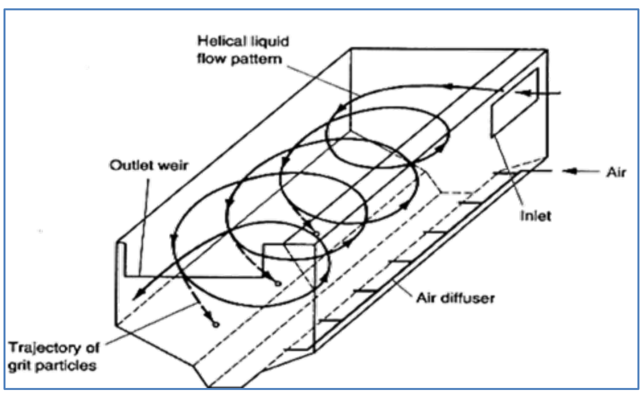
This in-channel, cylindrical bar screen will screen, wash, compact and transport screenings out of the influent wastewater. The angled installation minimizes the space requirements for required shallow installations. The screenings are removed from the cylindrical bars by a rotating rake that passed through the full depth of the bars. The entire unit would be constructed of 304 stainless steel.

As liquid flows through the screening basket the solids are trapped by the screen bars that form the circular basket. When the liquid rises to a predetermined level then the rake begins to rotate and clears the screen bars. When the rake reaches the top of the screen the captured material drops into the central screw conveyor and then the rake reverses to complete a cleaning pass. The central screw conveyor will wash and compact the collected material as it is transported to the discharge chute. Screenings are initially washed as they are deposited into the collection screw conveyor and then washed again in the upper section of the transport tube. The macerating action of the screw breaks down the large organic particles which are then washed back into the flow stream. A spray wash system in the dewatering chamber removes any collected material to ensure free drainage of water which is removed in the compaction process. The new screen would have perforations from 1-mm to 8-mm.

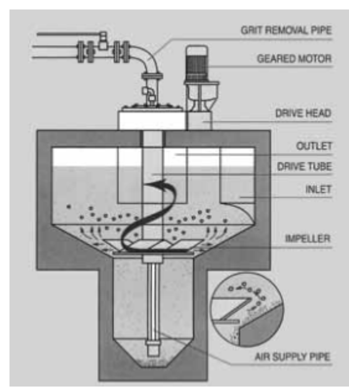
5.5.2 Grit Removal Equipment

For either location alternative, two grit removal technologies were considered: vortex grit removal and aerated grit removal. **Figure 5-9** depicts both technologies.

FIGURE 5-9
GRIT REMOVAL SYSTEMS



Aerated Grit Removal



Vortex Grit Removal

<u>Vortex Grit Removal</u>

Vortex grit removal is a well-established technology that uses centrifugal forces to separate the grit from the wastewater flow. The vortex can be generated with a paddle mixer ("forced vortex") or with hydraulic force ("induced vortex"). The grit slurry is pumped or drained to a hydrogritter (hydro-cyclone followed by a screw classifier), or to a grit washer for grit and organics separation prior to disposal into a roll off container. The grit washers are a smaller form of a vortex grit removal system that is used to wash away the organics from the grit and incorporates a dewatering screw for final transport.

The advantages of vortex grit removal include the following:

- Does not utilize aeration, and thus does not contribute oxygen to the flow that could hinder downstream biological nutrient removal processes
- Small footprint
- Lower operating and maintenance costs due to aeration blowers are not required
- Well-suited for odor control

The disadvantages of vortex grit removal include the following:

- Vortex grit removal is typically less effective than properly sized aerated grit removal
- High grit loadings during peak wet weather events can overload vortex systems resulting in clogging and compromised operation performance

Aerated Grit Chamber

Aerated grit chambers are designed to remove grit consisting of sand, gravel, cinders, or other heavy materials that have specific gravities or settling velocities generally greater than those of organic particles. In addition to these materials, grit contains eggshells, bone chips, seeds, coffee grounds, and large organic particles. The aerated grit chambers consist of a tank where positive displacement blowers provide air to diffusers on one side of the tank inducing a helical roll across the longitudinal forward flow. The helical roll pattern induced in the grit chamber causes grit to settle to the bottom of the chamber while keeping lighter organic material in suspension to be processed further downstream. If the velocity is too high, grit will be carried out of the tank; if it is too low, organic material will be removed with the grit. The turbulence in the tank also helps to scour organic material that has attached to the grit particles. Grit that is not well-washed and contains organic matter produces undesirable odors and attracts pests. The grit that settles at the bottom of the grit chamber is typically conveyed via a screw conveyor to a grit sump. The grit slurry could be pumped to a hydrogritter (hydro-cyclone followed by a screw classifier), or to a grit washer for separation prior to disposal into a roll off container.

The advantages of a new aerated grit removal system include the following:

- Properly sized aerated grit systems are more effective than vortex grit removal
- High grit loading during peak wet weather events can be stored in the grit chamber and processed as able

The disadvantages of a new aerated grit removal system include the following:

- Aerated grit removal technology contributes dissolved oxygen to the secondary influent, which would adversely affect the performance of nutrient removal process
- Higher operating and maintenance cost due to aeration blowers being required
- Not well-suited for implementing odor control

5.5.3 Locate Headworks at WWTF

As noted previously, screening and grit removal is currently performed at the WWTF. This option would consist of upgrading the existing facilities at the WWTF or of abandoning the existing systems and constructing a new screenings and grit facility at a hydraulic gradeline appropriate for the WWTF upgrade. There is ample room to construct either the vortex or aerated grit removal systems outside.

5.5.4 Locate Headworks at Main Pump Station

This option would consist of constructing a new screenings and grit removal facility at the Main Pump Station. Due to the hydraulic gradeline, the finished floor for screening and grit removal systems would be approximately 10-feet to 14-feet below grade and the bottom elevation would be approximately 18-feet to 24-feet below grade. Due to the proximity to adjacent public and private property, screening and grit removal systems would need to be enclosed in a building. Preliminary estimates indicate that the building would need to be approximately 25-feet by 45-feet. Due to limited parcel area, either option would require that the Town acquire property from an abutter or the gain the approval to use land within the Swazey Parkway. Contaminated soils are known to exist in the project area and therefore soils and groundwater generated from the site will require special handling and monitoring. The existing MPS would need to be renovated and brought up to current electrical, mechanical, fire and architectural codes.

5.5.5 Comparison of Headworks Alternatives

The advantages of constructing a new screenings and grit facility at the WWTF include:

- Space is not a limitation, public complaints related to odors and materials handling are unlikely, and the screening and grit facility can be built at the desired hydraulic gradeline.
- No known contaminated soils or groundwater.
- Grinders do an adequate job of minimizing pump clogging at the Main Pump Station.
- Flows from other communities (if connected) could be processed through the same screening and grit removal facility.

The disadvantages of constructing a new screenings and grit facility at the WWTF include:

- Continuing to pump raw sewerage from the MPS will continue to wear the pump and forcemain over time, resulting in increased O&M costs

The advantages of constructing a new screening and grit facility at the MPS include:

- Removing screenings prior to pumping will decrease incidents of pump clogging
- Removing grit prior to pumping will decrease the wear on the pumps, valves and forcemain

The disadvantages of constructing a new screening and grit facility at the MPS include:

- Excavation and dewatering would be challenging in the close vicinity of the Squamscott River due to high ground water levels
- Contaminated soils are known to be in the project area and would be expensive to monitor and dispose of during excavation
- The Town would need to acquire property (or construction/permanent easement) from an abutter or gain the approval to use land within the Swazey Parkway
- Screenings and grit disposal can be odorous and could result in public complaints. Odor control would be recommended
- Additional operator attention would be required at the MPS (which is not staffed)

5.5.6 Recommended Headworks Alternative

Based on the analysis above it recommended that the screenings and grit facility be constructed at the WWTF. With the limited space and contaminated soils at the MPS site, construction of the expansion would be very challenging and expensive.

5.6 DISINFECTION ALTERNATIVES

The most common means of disinfection at most wastewater facilities in New England is the addition of sodium hypochlorite to the effluent to chlorinate followed by the addition of sodium bisulfite to dechlorinate. An increasingly popular means of disinfection is ultraviolet (UV) light radiation. A discussion on the advantages and disadvantages of each system is presented below.

5.6.1 Chemical Disinfection

A chemical disinfection system consists of chemical fill station, bulk storage of sodium hypochlorite and sodium bisulfite with secondary containment, tank level sensors, tank vents and miscellaneous valves and piping; sodium hypochlorite and sodium bisulfite feed pumps such as peristaltic pumps or diaphragm pumps; Chlorine Contact Tank with miscellaneous gates and scum trough removal; sodium hypochlorite and sodium bisulfite carrier/dilution water; and a feed rate control system. Sodium hypochlorite addition rate is normally paced on effluent flow rate and trimmed on the chlorine residual taken at the upstream end of the Chlorine Contact Tank. Sodium bisulfite addition rate is normally flow paced and trimmed on the chlorine residual taken at the downstream end of the Chlorine Contact Tank. Mechanical mixers are commonly used at the points of chemical addition to provide positive mixing of effluent and chemical and the chlorine residual is measured with a free chlorine analyzer.

5.6.2 UV Disinfection

In order to provide effective UV radiation disinfection, the effluent needs to flow through open channels with multiple banks of UV light modules. A downstream level control device needs to be provided to maintain the adequate water level in the channel under low flow conditions and a recirculating sump pump may be necessary during extreme low flow conditions. UV radiation disinfection systems require provisions for measuring UV transmittance; a cleaning system to remove grease, dirt build-up and scaling on the lamps which minimizes disinfection performance; and a jib crane to perform routine maintenance such as bulb replacements. Per the State of New Hampshire Code of Administrative Rules, Env-Wq 700 Standards of Design and Construction for Sewerage and Wastewater Treatment Facilities, the UV radiation disinfection system must be enclosed in a ventilated building for year-round operation and pilot testing may be required to demonstrate effective disinfection.

UV light radiation systems have been gaining popularity in the past few years. For the most part, UV systems have been most commonly used in advanced wastewater treatment systems where suspended solids levels are consistently less than 30 mg/l and in places where chlorine residual would be a problem to groundwater or sensitive water bodies. However, improvements to UV disinfection systems such as different light intensities and bulb cleaning systems have led to increased use within secondary, activated sludge wastewater treatment systems. UV transmissivity is a critical parameter for the proper sizing of a UV disinfection system. If UV disinfection is selected transmissivity testing would need to be performed prior to design, preferably over several seasons.

5.6.3 Comparison of Disinfection Alternatives

Three options were considered for disinfection.

- *Chemical Disinfection Alternative A* consisted of relocating new sodium hypochlorite and sodium bisulfite bulk storage tanks and chemical pumps next to the existing Chlorination Building. This would require a building addition onto the existing Chlorination Building and reconfiguration of chemical piping. The Chlorine Contact Tank could be reused but would require crack repair.

- *Chemical Disinfection Alternative B* consisted of reusing the existing chemical disinfection system. This option would include reusing the existing sodium hypochlorite and sodium bisulfite bulk storage tanks, replacing the chemical recirculation pumps, chemical pumps and controls. The Chlorine Contact Tank could be reused but would require crack repair.

- *UV Disinfection* consisted of modifications to the existing Chlorine Contact Tanks with a new building prior to installing a UV disinfection unit. The UV disinfection system would need to be added to SCADA with provisions to stop discharging in the event of a power loss, in order to comply with Env-Wq 712.05.

Capital cost, operation and maintenance costs and present worth were generated for each alternative. These are summarized in **Table 5-11** below.

TABLE 5-11
COSTS FOR DISINFECTION SYSTEM ALTERNATIVES

	Chemical Disinfection Alternative A	**Chemical Disinfection Alternative B**	**UV Disinfection Alternative**
Capital Cost	$910,000	$420,000	$1,370,000
Annual O&M (Year 1)	$28,000	$28,000	$28,000
S. Hypochlorite (gallons)	17,500	17,500	0
S. Bisulfite (gallons	2,100	2,100	0
Electricity (kw-hrs)	Negligible	Negligible	104,000
Annual O&M (Year 20)	$91,000	$91,000	$64,000
S. Hypochlorite (gallons)	38,400	38,400	0
S. Bisulfite (gallons	3,700	3,700	0
Electricity (kw-hrs)	Negligible	Negligible	162,500
Net Present Worth	**$1,760,000**	**$1,240,000**	**$2,120,000**

Notes:
(1) Installation and electrical costs estimated at 20% of equipment cost each.
(2) Energy costs based on flow-proportional energy demand, current electrical cost and 3% per year inflation.
(3) Chemical costs are based on flow-proportional chemical use, current chemical costs and 3% per year inflation.
(4) Present Worth is based on 20-year at 4.0% interest.
(5) ENR CCI 9700

Advantages for Chemical Disinfection Alternative A include:

- Relocating the entire chemical disinfection system at the Chlorination Building would eliminate recirculating chemicals from the Control Building. Having all new components to the chemical disinfection system would improve reliability
- WWTF staff presently use chemical disinfection and are familiar with the process

Disadvantages for Chemical Disinfection Alternative A include:

- Second lowest net present worth
- Continue to utilize chemicals for disinfection

Advantages for Chemical Disinfection Alternative B include:

- Lowest net present worth
- WWTF presently use chemical disinfection and are familiar with the process

Disadvantages for Chemical Disinfection Alternative B include:

- Continue to utilize chemicals for disinfection

Advantages/disadvantages for UV disinfection alternative include:

- Exeter WWTF has expressed a strong interest in not storing hazardous chemicals onsite
- No toxic byproducts produced and discharged to the environment (water or air)
- No risk of overdosing
- No issues with chloramine formation due to partial nitrification

Disadvantages for UV Disinfection include:

- Still require a small hypochlorite system for filament and odor control

5.6.4 Recommended Disinfection Alternative

The least cost approach is to include a chemical disinfection system in the WWTF upgrade. However, the annual O&M cost associated with UV disinfection is lower than chemical disinfection over time. The significant reduction in the use of chemicals on-site is advantageous.

Either alternative meets NHDES regulations. The Town should discuss the advantages and disadvantages of each alternative.

5.7 LAGOON DECOMMISSIONING ALTERNATIVES

A meeting was held on July 3, 2014 with NHDES to discuss the acceptable methods for decommissioning the four lagoons at the Exeter WWTF. The meeting was attended by: NHDES (Mike Rainey, Stergios Spanos, Gloria Andrews, Lori Sommers, Dan Fenno); Town of Exeter (Michael Jeffers); and Wright-Pierce (Andy Morrill). Four methods were discussed as potentially viable. Each method is summarized below.

5.7.1 Decommissioning Method No. 1 – Cap and Monitor Lagoon

Method No. 1 would consist of sampling the sludge in Aerated Lagoons No. 1, No. 2, No. 3 and the Sludge Storage Lagoon. If the sludge is deemed acceptable by NHDES it would be hydraulically dredged/excavated from Aerated Lagoons No. 2, No. 3 and the Sludge Storage Lagoon into the portion of Aerated Lagoon No. 1 that will not be used for the two influent equalization basins. Aerated Lagoon No. 1 would then be drained and dewatered to have a soil cap installed over the stored sludge. Vents would be installed to monitor and relieve any released gases. The capped portion of Aerated Lagoon No. 1 could have end use restrictions depending on he contaminants found during sludge sampling. Aerated Lagoons No. 2, No. 3 and the Sludge Storage Lagoon would then require that the bottoms be tested free of sludge.

A NHDES approved Closure Plan would be required. The current Exeter WWTF Groundwater Discharge Permit would need to be amended and would likely require a hydrologic study to determine the proper groundwater well sampling points. The current groundwater sampling wells could be used if found suitable by NHDES. Groundwater sampling and gas monitoring would be required for a minimum of 30 years and would need to be bonded.

5.7.2 Decommissioning Method No. 2 – Dewater and Dispose of Sludge

Method No. 2 would consist of hydraulically dredging/excavating the sludge in Aerated Lagoons No. 1, 2, 3 and the Sludge Storage Lagoon. The sludge would then be dewatered and disposed of by a sub-contractor. The disposal of the dewatered sludge could either be through beneficial reuse as Class A or Class B biosolids or it could be deposited into a secure landfill (unclassified sludge). Class A dewatered sludge has the lowest disposal cost, followed by Class B dewatered sludge, while the unclassified dewatered sludge has the highest disposal cost. The landfill would require that the unclassified sludge be tested for contaminants and pass a paint filter test, which requires a total solid content of approximately 18%.

A NHDES approved Closure Plan would be required. To classify the sludge for disposal (i.e. Class A, Class B or unclassified) a Sludge Quality Certificate (SQC) needs to be obtained from NHDES. The SQC could be obtained by the Exeter WWTF or the Contractor. SQC testing requires that one composite sample be obtained for each lagoon to test for contaminants. The composite sample would consist of 20 to 40 grab samples throughout the lagoon. Once the sludge has been classified, dewatered and disposed, the bottom of each lagoon is required to be tested free of sludge.

5.7.3 Decommissioning Method No. 3 – Dry and Dispose of Sludge

Method No. 3 would consist of hydraulically dredging/excavating the sludge from Aerated Lagoons No. 1, No. 2, No. 3 and the Sludge Storage Lagoon to be dried and disposed of off-site. The sludge could be dried in geo-bags or an on-site drying bed. It takes a minimum of approximately two months for the sludge to dry before it can be disposed of properly. For best results, it is ideal if the sludge is able to dry through a winter freeze and spring thaw period. This method could be accomplished over several years, provided the intended procedure is outlined in the Closure Plan.

A NHDES approved Closure Plan and a SQC would be required. The dried sludge could be disposed of through beneficial reuse or deposited in a landfill, depending on the class of sludge determined in the SQC process. All lagoons would then require that the bottoms be tested free of sludge.

5.7.4 Decommissioning Method No. 4 – Keep Aerated Lagoons in Process

Method No. 4 would consist of keeping all aerated lagoons in the active process and would not require decommissioning of the lagoons. The sludge in the lagoons could then be removed and disposed of as required. Since the aerated lagoons cannot meet the NHDES permit and AOC as issued, this is not viable for the "on-site alternatives" described herein; however, it would be feasible for one of the "regional alternatives" described herein.

5.7.5 End Use of Decommissioned Lagoons

Once the lagoons are decommissioned there are three options for end use of the land: 1) fill the lagoons with clean water (i.e., not part of the treatment process); 2) fill the lagoons with backfill and reuse the site for municipal purposes (e.g., recreational uses, public works uses; etc.); or 3) removing all/portions of the lagoon embankments and restoring the area to flood plains and brackish wetlands for the Squamscott River.

A second meeting held with on October 8, 2014 with numerous agencies to discuss the potential for flood plain and wetlands restoration. This meeting was attended by: NHDES (Lori Sommers, Gloria Andrews, Tracey Wood, Mindy Bubier, Kevin Lucey, Frank Richardson); Inland Fisheries and Wildlife (Corey Riley, Cheri Patterson); Nature Conservatency (Ray Konisky); EPA (Joy Hilton, Mark Kern, Ed Reiner (EPA); UNH (Dave Burdick); Town of Exeter (Michael Jeffers, Jen Mates, Matt Berube, Jennifer Perry); and Wright-Pierce (Andy Morrill, Travis Pryor, Ed Leonard). Based on the discussions at the meeting, the general consensus was:

- This location represents a very good opportunity for a large flood plain and "low marsh" wetland restoration project (approx. 20 acres). From 1962 aerial photographs, it appears that the river meander was present prior to the construction of the lagoons.
- There are numerous phragmytes colonies in the area. If invasive species mitigation is not methodically done in advance, this location could serve as a seed area for phragmytes

propagation. UNH indicated that the Town should reach to numerous project partners for this work including NHDOT, NHDES, NRCS, Rockingham County Conservation Commission, the Town of Newfields and the Town of Stratham.

- NHDES indicated that the restoration project would likely rank high on competitive State and regional grant opportunities.

If any of the lagoons are restored to flood plains/wetlands for the Squamscott River, a Wetland Compensation Bank (WCB) could be utilized to offset decommissioning costs. Although the NHDES does not presently have WCB regulations in place, they would defer to the EPA and ACOE for guidance. If a WCB were established, the Town of Exeter would be compensated by other project proponents for placing its' wetlands into preservation. Drawbacks to establishing a WCB are that it could take several years for NHDES to consider the wetlands operational and it is unknown if there will be sufficient local projects requiring wetland mitigation.

Another option for offsetting the decommissioning costs would be the use of the Aquatic Resource Mitigation (ARM) Fund. The ARM Fund is a NHDES grant program where wetland mitigation compensation can be used for wetland restoration design, demolition, construction, legal fees and/or plantings. The restored wetlands would need to be placed in preservation for protection. Lori Sommers, NHDES Wetland Mitigation Coordinator, noted that there is a substantial amount of Seacoast Area grant funds that will be available in 2015 to 2016.

5.7.6 Comparison of Decommissioning Alternatives

Advantages/Disadvantages of Method No. 1:

- No sludge dewatering and disposal is required
- Sludge would be used as fill material in Aerated Lagoon No. 1
- Could be used with any new Exeter WWTF option
- Lagoons could be reclaimed or restored to flood plains/wetlands for the Squamscott River
- Groundwater and gas monitoring would be required for a minimum of 30 years
- Reuse of capped area could have restrictions depending on the sludge quality

Advantages/Disadvantages of Method No. 2:

- Quickest method of decommissioning
- Former lagoon areas would not have end use restrictions
- Could be used with most new Exeter WWTF options
- Lagoons could be reclaimed or restored to flood plains/wetlands for the Squamscott River

Advantages/Disadvantages of Method No. 3:

- Former lagoon areas would not have end use restrictions
- Could be used with most new Exeter WWTF option
- Lagoons could be reclaimed or restored to flood plains/wetlands for the Squamscott River
- Longest duration required to complete decommissioning

Advantages/Disadvantages of Method No. 4:

- No decommissioning tasks are required
- Lagoon areas could not be reclaimed or restored to flood plains/wetlands

Based on discussions with the Town, Method 1 and Method 4 are not desired or recommended. Method 2 and Method 3 are similar (i.e., both involve removing all biosolids) with the difference the time required to remove the sludge (i.e., Method 3 will take substantially longer). Since Lagoon No. 1 will be needed for influent equalization in the WWTF, Method 3 is not available for Lagoon No. 1 but could be used for Lagoon Nos. 2 and 3.

5.7.7 Recommended Decommissioning Method

A combination of Method 2 or Method 3 is recommended. The lagoon decommissioning cost depends greatly on the Sludge Quality Certificate, sludge volume and desired end use of the former lagoons. For the purposes of this study, it was assumed that Exeter would retain a contractor to dewater and dispose of the sludge as "unclassified waste" using Method 2. Preliminary construction cost estimates were developed for Method No. 2, including the three methods of end use. These costs are shown in **Table 5-12**.

[*NOTE: The quantity and quality of the sludge in the lagoons will be assessed in Fall 2014 as a part of this Study in order to refine cost estimates. The quantity and quality of the sludge will need to be re-assessed prior to securing a Closure Plan from NHDES.*]

TABLE 5-12
COSTS FOR LAGOONS DECOMMISSIONING ALTERNATIVES

	Method 2 with Reclaimed Land	**Method 2 with Wetlands Restoration**	**Method 2 with Open Water**
Permitting & Closure Plan	$50,000	$50,000	$50,000
Site Protection and Restoration	$300,000	$300,000	$300,000
Dewater and Dispose of Lagoon Solids	$1,800,000	$1,800,000	$1,800,000
Remove Lagoon Equipment & Structures	$150,000	$150,000	$150,000
Fill Lagoons	$10,000,000	-	-
Restore Area as Flood Plains	-	$1,000,000	-
Fill with Clean Water	-	-	$0
Contractor OH&P	$1,850,000	$500,000	$350,000
Contingency & Inflation	$2,120,000	$570,000	$400,000
Grant Funding	$0	($300,000)	$0
Total Construction Cost	**$16,270,000**	**$4,070,000**	**$3,050,000**

Note:
1. ENR CCI 9700
2. Unit costs based on Jaffrey NH and Peterborough NH Lagoon Closure project bids.
3. Lagoon biosolids volume estimates from Solarbee Service Report, October 2013 (total of 1,800 tons)
4. Biosolids disposal assumed to be unclassified ($1,000 per dry ton).
5. Lagoon earth fill estimated at 675,000 CY at $15/CY.
6. Wetlands and flood plain restoration cost is an allowance.
7. Contractor OH&P estimated at 15% of costs. Contingency and Inflation estimated at 15% of costs including Contractor OH&P.
8. Grant funding estimated based on discussions with NHDES Wetlands Mitigation Coordinator

5.8 SUMMARY OF ALTERNATIVES EVALUATIONS

The section presented the results of several alternatives evaluations. A number of these evaluations may be refined in the preliminary design phase of the project; however, the refinements would not be expected to change the recommendations.

- Final decisions on the regional wastewater management alternatives should be made when the results of three separate studies are in-hand (i.e., the Town-commissioned Pease WWTF Regional Study; the WISE project repor; and the City of Portsmouth commissioned Pease WWTF Regional Study). This separate studies are expected to be completed between October 2014 and January 2015.

- The recommended on-site WWTF approach is Alternative 2 (Bardenpho) or Alternative 3 (Sequencing Batch Reactor)

- The recommended biosolids management approach is Alternative 2 (Mechanical Dewatering with Cake Disposal).

- The recommended disinfection approach is UV disinfection>

- The recommended headworks approach is to construct new facilities at the WWTF.

- The recommended lagoon decommissioning approach is Method 2 (Dewater and Dispose of Solids) or Method 3 (Dry-in-place and Dispose of Solids) in combination with either wetlands restoration or open water.

The items will be carried forward into the recommended plan

Section 6

SECTION 6

RECOMMENDED PLAN

6.1 INTRODUCTION

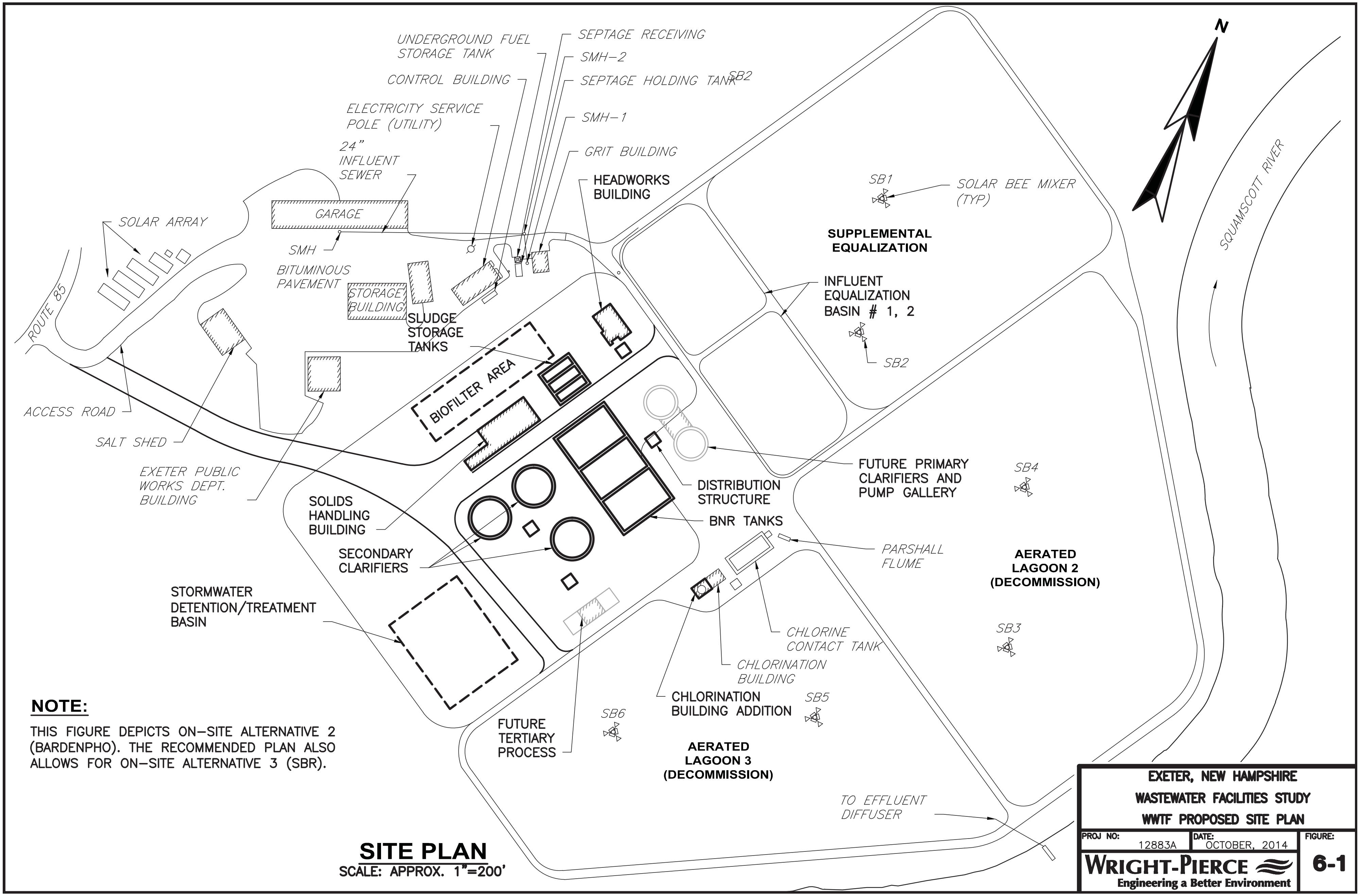
Section 5 of this report concluded that the on-site regional alternative was a cost-effective and practicable approach to addressing Exeter's NPDES permit and AOC. This section of the report presents the details of the recommended plan for the on-site regional alternative including phasing, estimated staffing requirements, estimated capital costs and estimated operations and maintenance costs. The details were developed for the purposes of quantifying the financial impacts of the project. Each of the details can be refined in the preliminary design phase of the project.

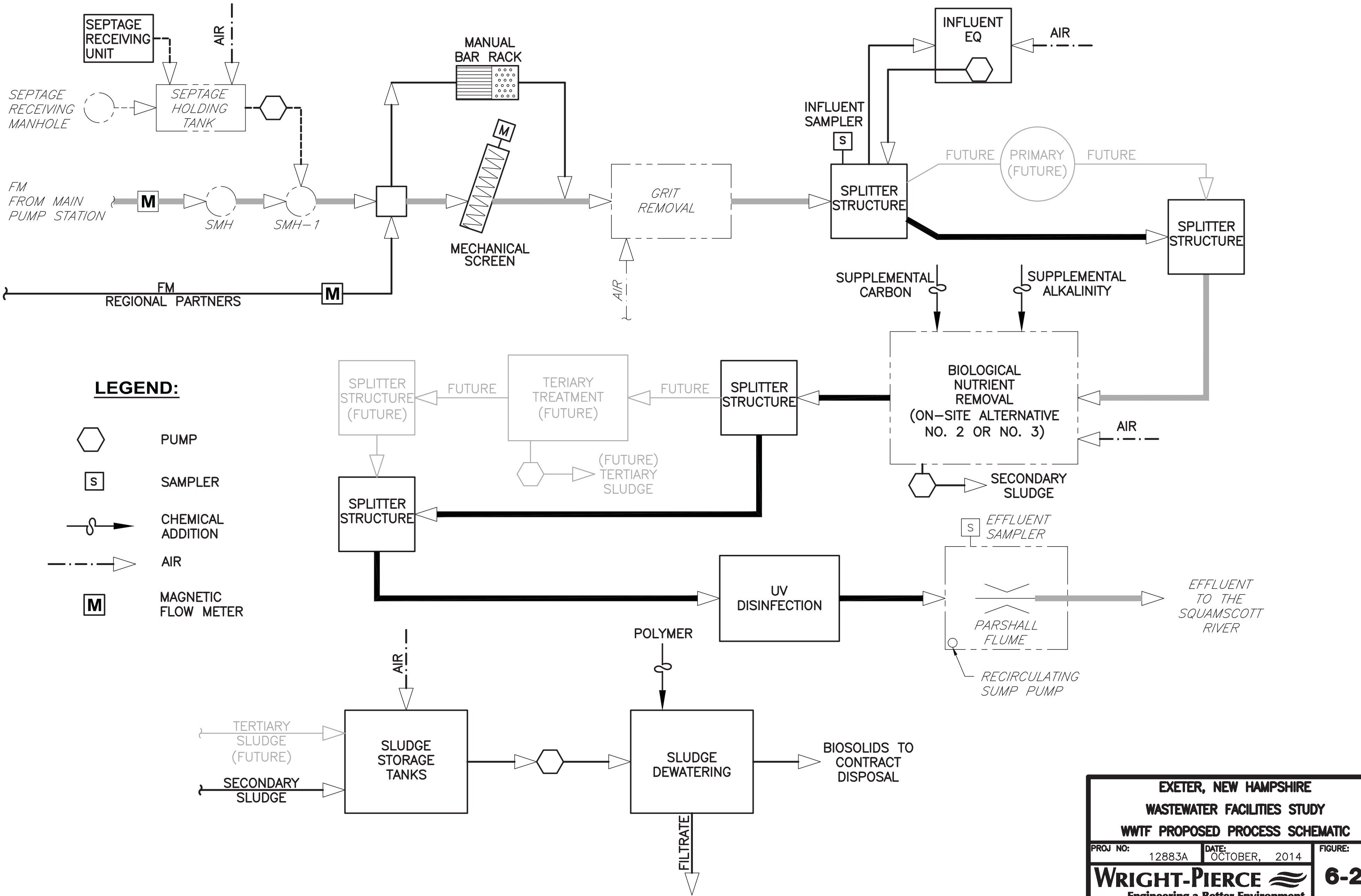
6.2 RECOMMENDED PLAN

The basis for the recommended treatment facility improvements are described via unit process and/or building system in **Sections 3 and 5** of this report. The components of the recommended plan are included for a variety of reasons, including being:

- Required to meet current and/or identified future permit limits
- Required for equipment or process reliability or to meet NHDES regulations
- Required to reduce or eliminate combined sewer overflows
- Required to address building or life-safety code-related issues
- Desired to improve energy efficiency/reduce operating costs
- Desired to increase revenues (e.g., septage receiving improvements, "customer communities")
- Desired to improve efficiency of operations/utilization of staff
- Desired to better utilize existing spaces

The proposed site plan and process schematic are presented as **Figure 6-1** and **Figure 6-2**, respectively. Phasing of project improvements is presented later in this section of the report.


N
UNDERGROUND FUEL STORAGE TANK
SEPTAGE RECEIVING
SMH-2
CONTROL BUILDING
SEPTAGE HOLDING TANK
SB2
ELECTRICITY SERVICE POLE (UTILITY)
SMH-1
24" INFLUENT SEWER
GRIT BUILDING
HEADWORKS BUILDING
SB1
SOLAR BEE MIXER (TYP)
SOLAR ARRAY
GARAGE
SMH
BITUMINOUS PAVEMENT
STORAGE BUILDING
SLUDGE STORAGE TANKS
SUPPLEMENTAL EQUALIZATION
INFLUENT EQUALIZATION BASIN # 1, 2
SB2
ROUTE 85
SQUAMSCOTT RIVER
BIOFILTER AREA
ACCESS ROAD
SALT SHED
EXETER PUBLIC WORKS DEPT. BUILDING
FUTURE PRIMARY CLARIFIERS AND PUMP GALLERY
SB4
DISTRIBUTION STRUCTURE
SOLIDS HANDLING BUILDING
BNR TANKS
SECONDARY CLARIFIERS
PARSHALL FLUME
AERATED LAGOON 2 (DECOMMISSION)
STORMWATER DETENTION/TREATMENT BASIN
CHLORINE CONTACT TANK
SB3
CHLORINATION BUILDING
CHLORINATION BUILDING ADDITION
SB5
SB6
FUTURE TERTIARY PROCESS
AERATED LAGOON 3 (DECOMMISSION)
NOTE:
THIS FIGURE DEPICTS ON-SITE ALTERNATIVE 2 (BARDENPHO). THE RECOMMENDED PLAN ALSO ALLOWS FOR ON-SITE ALTERNATIVE 3 (SBR).
TO EFFLUENT DIFFUSER
SITE PLAN
SCALE: APPROX. 1"=200'
EXETER, NEW HAMPSHIRE
WASTEWATER FACILITIES STUDY
WWTF PROPOSED SITE PLAN
PROJ NO: 12883A
DATE: OCTOBER, 2014
FIGURE: 6-1
WRIGHT-PIERCE
Engineering a Better Environment



EXETER, NEW HAMPSHIRE
WASTEWATER FACILITIES STUDY
WWTF PROPOSED PROCESS SCHEMATIC

PROJ NO:	DATE:	FIGURE:
12883A	OCTOBER, 2014	6-2

WRIGHT-PIERCE
Engineering a Better Environment

6.2.1 Main Pump Station

- Provide new influent sluice gate to wetwell.
- Maintain existing grit sump for periodic manual cleanout. Maintain existing channel grinders.
- Upgrade the existing three pumps to dry-pit submersible pumps sized to convey the peak flows to the WWTF in order to eliminate future combined sewer overflows (CSOs). Pumps will be provided with variable frequency drives (VFDs) for variable speed pumping.
- Provide miscellaneous process upgrade including new suction/discharge piping, new link-type seals on wet-to-dry well wall penetrations and pressure injection of wetwell/drywell wall cracks.
- Provide new PLC-based control panel with new instrumentation, including wetwell level, combustible gas, wastewater flow and CSO flow. Upgrade connectivity to the WWTF SCADA system.
- Comprehensively upgrade the electrical service, main power distribution and automatic transfer switch. Retain the existing standby generator for continued use. Provide local disconnects and ESTOPS at process equipment. Upgrade the remainder of the electrical systems to include energy efficient lighting (interior and exterior), emergency lighting/exit signs, receptacles and fire alarm system (if required by the Fire Chief).
- Comprehensively upgrade the building and building systems, including: repairing the damaged base plates supporting the wall panels; replacing the exterior doors; creating separation between the "classified" Pump Room and the "unclassified" upper level (NFPA 820); replacing the damaged stair nosings at the exterior stairs; replacing the roofing system; repainting the interior spaces; and upgrading the heating, ventilating and plumbing systems.

6.2.2 Main Pump Station Forcemain

- Construct a new 16-inch diameter forcemain from the Pump Station to the WWTF (approximately 5,000 feet). Reline the existing forcemain from the Pump Station to the

WWTF (approximately 5,000 feet). This will allow for additional capacity and improved longevity of the existing piping. Consider the cost-effectiveness of open cut installation of two forcemains.

6.2.3 Influent Flow Measurement and Sampling

- Maintain the existing flow meter for continued use.
- Relocate the existing influent sampler to the new Headworks Building.
- If “customer communities” are allowed to connect to the Exeter WWTF, provide the ability to meter and sample flows from those communities separate from Exeter’s influent.

6.2.4 Septage Receiving

- Provide a mechanical septage receiving unit to provide for fine screening (1/4”) and screenings washing/compaction. The septage receiving unit should be provided with a flow meter to measure the volume of septage received. The unit will be insulated and heat-traced and be suitable for an exterior installation.
- Upgrade the existing Septage Tank including pressure injecting concrete cracks and adding instrumentation for level measurement.
- Construct a second Septage Tank, of similar volume, to allow for equalization of this load. Consider using the existing Aerated Grit Chamber.
- Upgrade the two septage transfer pumps including a new septage flow meter

6.2.5 Screening and Grit Removal

- Abandon the existing Grit Building for its current process functions. Repurpose the structure for alternate uses. If repurposed, comprehensively upgrade the building and building systems, including: repairing the minor cracks in the exterior masonry walls; cleaning the moss and organic growth at the base of the walls; installing new sealants at the control joints and around the perimeter of all wall penetrations; replacing the shingle roofing and eave flashing; replacing vinyl siding at gable ends; replacing existing doors;

repainting the interior surfaces; and upgrading the heating, ventilating and plumbing systems.

- Construct a new Headworks Building. Similar to the existing building on-site, this building would consist of cast-in-place concrete foundation and block wall with split-face block finish.
- Provide a mechanical fine screen (1/4" preferred) with screenings wash press and by-pass manual bar rack. Provide two vortex grit removal systems to allow for proper sizing under average and peak conditions, including two grit pumps and two grit classifiers/washers. Screening and grit removal systems will be sized for the peak instantaneous flow to the WWTF including flows from "customer communities" (if applicable).
- Provide instrumentation, controls and SCADA connectivity.

6.2.6 Influent Equalization Basin

- Create two lined off-line influent equalization basins within a portion of former Aerated Lagoon No. 1. The basins will be 1.0 million gallons each. The intent is to size the basins to limit the peak instantaneous flow to 6.6-mgd.
- Provide a triplex influent equalization pump station with instrumentation (level, flow), controls and SCADA connectivity.

6.2.7 Primary Treatment

- Arrange the site plan and set the hydraulic gradeline for the possible future inclusion of primary treatment.

6.2.8 Advanced Secondary Treatment/ Nitrogen Removal

- Construct three trains of activated sludge/ nitrogen removal process, including mixers, pumps, blowers, fine bubble diffused aeration systems, instrumentation (air flow, dissolved oxygen, ORP, nitrate, ammonia, TSS), control systems, flow splitter structures and site piping.

- The Bardenpho configuration would include three aeration tanks, three internal recycle pumps, nine submersible mixers, three circular secondary clarifiers (75-foot diameter by 16-foot sidewater depth with rapid sludge removal withdrawal mechanism), secondary scum pump station, four return sludge pumps (three duty/one standby), two waste sludge pumps (one duty/one standby) and four aeration blowers (three duty/one standby). This equipment will be in the Solids Process Building, the Aeration Tanks and Clarifiers
- The SBR configuration would include three SBR tanks, three mixers, three waste sludge pumps and four aeration blowers (three duty/one standby), one post-equalization tank with diffused aeration system. This equipment will be in the Solids Process Building, the SBR Tanks and Post-Equalization Tanks

- Construct a supplemental alkalinity system to maintain pH for process control (nitrification/denitrification) and effluent pH compliance. This system will have a bulk liquid storage tank and two chemical feed pumps. This system will be housed in the Solids Process Building
- Construct a supplemental carbon storage and feed system to achieve 3-mg/l effluent TN. This system will have a bulk liquid storage tank and three chemical feed pumps suitable for use with methanol, MicroC® or similar products. This system will be an exterior installation.
- Construct a three train traditional filtration system (cloth disk or sand), including appurtenant pumping, chemical, instrumentation and control systems. This system will be housed in the Tertiary Building.

6.2.9 Disinfection

- Remove the existing chlorination and dechlorination systems from the Control Building and from the Chlorination Building. Rename the building “Chlorination Building” to the “Disinfection Building”.
- Provide a UV disinfection system retrofitted in the existing Chlorine Contact Tank. Rename the “Chlorine Contact Tank” to the “Disinfection Tank”. Repairs cracks in the Disinfection Tank.

- Per NHDES regulations, construct a ventilated building around the UV disinfection system for year-round operation. In lieu of a large uninterruptible power supply, maintain a portion of former Aerated Lagoon No. 1 as "supplemental influent storage" to provide a means to stop discharging in the event of a power loss until the UV system is back up to full intensity.
- Provide instrumentation (level, flow, turbidity), controls and SCADA connectivity for the UV disinfection system.
- Provide new electrical service and main power distribution to the Disinfection Building. Provide local disconnects and ESTOPS at process equipment. Upgrade the remainder of the electrical systems to include energy efficient lighting (interior and exterior), emergency lighting/exit signs, receptacles and fire alarm system (if required by the Fire Chief).
- Comprehensively upgrade the Disinfection Building and building systems, including: repairing the minor cracks in the exterior masonry walls; cleaning the moss and organic growth at the base of the walls; installing new sealants at the control joints and around the perimeter of all wall penetrations; replacing the shingle roofing and eave flashing; replacing vinyl siding at gable ends; replacing existing doors; repainting the interior surfaces; providing separation of electrical gear from process spaces; and upgrading the heating, ventilating and plumbing systems.
- Comprehensively upgrade the Control Building and building systems.

6.2.10 Effluent Flow Measurement and Sampling

- Upgrade the existing parshall flume insert and ultrasonic instrumentation.
- Maintain the existing effluent sampler for continued use. Add flow-pacing capability based on effluent flow rate.

6.2.11 Outfall

- No modifications anticipated within the planning period; however, note that the CAPE (Climate Adaptation Project Exeter) estimates a significant increase in flood elevation through the 21st century. At some point in the future, outfall modifications or an effluent pump station may be needed.

6.2.12 Sludge Processing Systems

- Construct a new Sludge Process Building with single sludge truck bay. Similar to the existing building on-site, this building would consist of cast-in-place concrete foundation and block wall with split-face block finish.
- Provide a sludge storage system including three Sludge Storage Tanks sized for 5 days of storage at design annual average conditions (i.e., 450,000 gallons total) with instrumentation (level), decanting and aeration systems. The decanting system is assumed to consist of telescoping valves. The aeration system is assumed to consist of three positive displacement blowers with diffused aeration grid (sized for 30 to 50 scfm per thousand cubic feet).
- Provide a mechanical sludge dewatering system including three sludge feed pumps (two duty, plus common standby), two dewatering machines (e.g., centrifuges or slow rotating presses), two polymer make-down systems, sludge conveyors and truck bay leveling conveyor.
- Provide instrumentation, controls and SCADA connectivity for the sludge processing systems.

6.2.13 Support Systems

- Upgrade the existing process water to allow for on-going use of effluent for on-site cleaning. The new system should reuse the existing hydropneumatic tank and should replace the existing pumps, air compressor, instrumentation and controls to match the duties required for the upgraded facilities.
- Upgrade the existing Disinfection Tank scum pump station and redirect scum to the new Sludge Storage Tanks.
- Per NHDES regulations, provide new sodium hypochlorite bulk storage tank and chemical metering pumps to allow for miscellaneous process/filament control and odor

mitigation uses. This is anticipated to be a 1,000 gallon tank with two chemical metering pumps. This system will be included in the Sludge Process Building.

6.2.14 Lagoon Decommissioning

- Abandon the existing Aerated Lagoons. Abandon/remove structures and piping.
- Conduct decommissioning of former Aerated Lagoon Nos. 1, 2 and the former Sludge Storage Lagoon in accordance with a NHDES-approved Closure Plan. Decommissioning is assumed to consist of hydraulically dredging, dewatering and disposal of the sludges as an "unclassified waste" by a construction contractor. **[*NOTE: The quantity/quality of the sludge in the lagoons will be assessed in Fall 2014 as a part of this Study. The quantity and quality of the sludge will need to be re-assessed prior to securing a Closure Plan from NHDES.*]**
- Repurpose the former Sludge Storage Lagoon as the location for the majority of the new WWTF tankage and buildings.
- Repurpose former Aerated Lagoon No. 1 to new influent equalization basins.
- Restore brackish flood plains and tidal wetlands within former Aerated Lagoons No. 2 and No. 3 to brackish flood plains/tidal wetlands. ***[NOTE: The scope of this effort is currently under discussion with NHDES.]***
- Pursue NHDES grants (e.g., the Aquatic Resource Mitigation (ARM) Fund) to offset restoration costs for design, demolition, construction, legal fees and/or plantings.

6.2.15 Civil-Site Improvements

- Construct a new 8-inch or 12-inch diameter water main from Summer Street to the Public Works Complex to provide potable water and fire protection flows (approximately 5,000 feet) for the Public Works Complex and WWTF.
- Construct a new access drive from Route 85 to the new facilities in order to minimize temporary construction traffic and permanent WWTF traffic on the existing Public Works facilities. WWTF related will increase over current, primarily due to biosolids hauling.

- Modify existing site to address parking and access for vehicles, maintenance activities, chemical deliveries, septage deliveries and biosolids hauling.
- Address stormwater management for new impervious areas, including stormwater harvesting for general purpose irrigation use. Stormwater detention ponds and/or rain gardens can be located within the footprint of the former Sludge Storage Lagoon and/or Aerated Lagoon No. 3.
- Construct new and/or upgraded site piping systems for raw sewage, equalization flows, activated sludge, return/waste sludge, scum and chemicals.
- Construct a new 12-inch water main from Water Street to the Public Works site and WWTF.

6.2.16 Architectural Improvements

- Construct new Headworks Building and Sludge Process Building, as described above.
- Renovate/repurpose the existing Grit Building and Disinfection Building ("Chlorination Building"), as described above.
- As noted above, comprehensively renovate the existing Control Building and building systems, including: repairing the minor cracks in the exterior masonry walls; cleaning the moss and organic growth at the base of the walls; installing new sealants at the control joints and around the perimeter of all wall penetrations; replacing the shingle roofing and eave flashing; replacing vinyl siding at gable ends; replacing existing windows and doors; repainting the interior surfaces; providing separation between the "classified" Pump Room and the "unclassified" upper floor (NFPA 820); and upgrading the heating, ventilating and plumbing systems. In addition, create new spaces in the Control Building to facilitate operations including converting the existing chemical rooms to occupied functions such as meeting/break room, control room, storage and a workshop and making the spaces ADA-accessible.

6.2.17 Instrumentation Improvements

- Upgrade the existing SCADA system to incorporate the WWTF upgrade instrumentation, monitoring, control and alarming systems. The new SCADA system will include three workstations – two in the Control Building and one in the Solids Process Building.

6.2.18 Electrical Improvements

- Comprehensively upgrade the electrical service and main power distribution. The preliminary sizing of the new service entrance is 2000 ampere.
- Provide a new stand-alone, diesel-engine, standby generator and automatic transfer switch housed in a sound-attenuated enclosure. The preliminary sizing of the unit is estimated at 750 kw.
- Upgrade the site duct bank system for power distribution to existing and new buildings and tanks.
- Provide exterior site lighting for new driveways, tankage and buildings.
- Provide local disconnects and ESTOPS at process equipment. Upgrade the remainder of the electrical systems to include energy efficient lighting (interior and exterior), emergency lighting/exit signs, receptacles and fire alarm system (if required by the Fire Chief).

6.2.19 Energy Efficiency/Green Design Improvements

The following types of energy efficient and green design elements will be evaluated and included where appropriate and cost effective.

- Natural and high efficiency lighting (with motion sensors in some locations);
- Solar walls;
- Effluent heat exchanger;
- Air-to-air heat exchangers;
- Energy recovery ventilators;
- Minimize impervious surfaces; and
- Light-colored roofing for reduced solar gain.

6.3 PHASING

The WWTF upgrades can be phased in any number of ways depending on the Town's goals. The purpose of phasing is generally to defer costs in order to moderate the rate impacts to users. Several examples of ways the WWTF upgrades could be phased include:

- By capacity (i.e., the initial phase could be sized for less than the licensed 3.0 mgd);
- By level of treatment (i.e., the initial phase would be sized to meet 8 mg/l effluent TN to meet the AOC versus 3 mg/l effluent TN to meet the NPDES permit); or
- By component (e.g., items such as decommissioning or disinfection could be deferred).

Figure 6-3 identifies the anticipated influent flow rates over the planning period. As described in Section 4, the AOC requires that the Town evaluate the effectiveness of its Nitrogen Control Plan in 2023 and determine whether additional WWTF upgrades are needed. If the Town elected to "phase by capacity", flows are anticipated to be 2.4 mgd with regional contributions and 2.1 mgd without regional contributions by 2023 to 2025. **Table 6-1** identifies several approaches to "phase by level of treatment" and "phase by capacity".

Based on discussions with the Town, the recommended phasing plan is presented in **Table 6-2** below. The goal of the recommended plan is to provide for nutrient removal greater than required by the AOC but less than that required by the NPDES permit and to defer any tertiary treatment components.

FIGURE 6-3
CONCEPTS FOR PHASING OF ON-SITE ALTERNATIVES

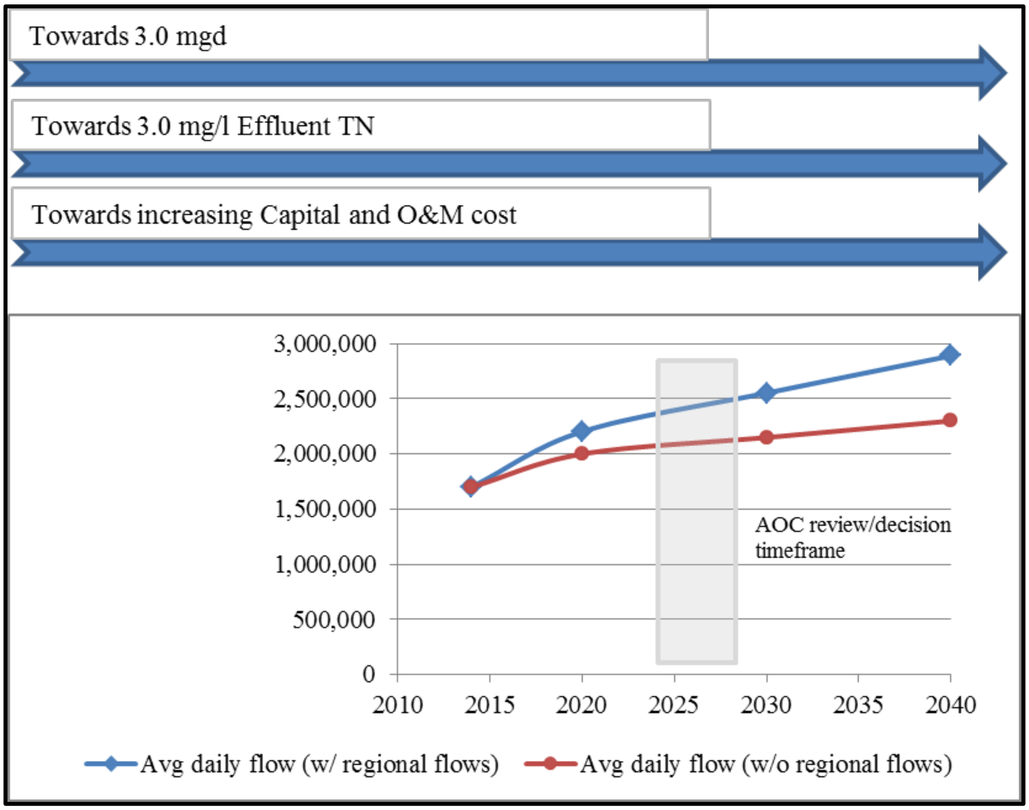


TABLE 6-1
POTENTIAL PHASING OPPORTUNITIES FOR ON-SITE ALTERNATIVES

Alternative	Phase 1	Phase 2
2A	Construct Bardenpho for 3.0-mgd	Add Filters for 3.0-mgd
2B	Construct MLE for 3.0-mgd	Expand to Bardenpho, add Filters for 3.0-mgd
2C	Construct Bardenpho for 2.25-mgd	Expand and add Filters for 3.0-mgd
2D	Construct MLE for 3.0-mgd	Add Primary Clarifiers, re-rate to Bardenpho for 3.0-mgd, add Filters for 3.0-mgd
3A	Construct SBR for 3.0-mgd	Add Denit Filter for 3.0-mgd
3B	Construct SBR for 2.25-mgd	Add 3rd SBR and Denit Filter for 3.0-mgd

TABLE 6-2
PRELIMINARY PHASING PLAN

Item	**Phase 1 (2014 to 2018)**	**Future Phase (2026 to 2030+)**
Main Pump Station	X	
Main Pump Station Forcemain	X	
Influent Flow Measurement & Sampling	X	
Septage Receiving	X	
Screening & Grit Removal	X	
Influent Equalization Basin	X	
Primary Treatment		X
Advanced Secondary Treatment/ Nitrogen Removal		
• 4-Stage Bardenpho & Secondary Clarifiers	X	
• Supplemental Alkalinity System	X	
• Supplemental Carbon System		X
• Tertiary Filtration System		X
Disinfection	X	
Effluent Flow Measurement & Sampling	X	
Sludge Processing Systems	X	
Support Systems	X	
Lagoon Decommissioning	X	
Architectural Improvements		
• Control Building	X	
• Grit Building	X	
• Chlorination Building ("Disinfection Building")	X	
• Headworks Building	X	
• Sludge Process Building	X	
• Tertiary Building		X
Civil-Site, Instrumentation & Electrical	X	X
Energy Efficiency/Green Design	X	X

6.4 STAFFING

Currently, three personnel operate and maintain the WWTF including one Grade III operator, one Grade II operator and one full-time equivalent maintenance mechanics (two mechanics, part-time, shared with Public Works). The existing WWTF is a Grade II plant. Using the criteria established by NHDES in ENV-WS 901.18 ("Classification and Reclassification of Wastewater Treatment Plants"), the upgraded WWTF would become a Grade III facility after the Phase 1 upgrade and a Grade IV facility after the Phase 2 upgrade. Using the criteria established by EPA

Publication MO-1 ("Estimated Staffing for Municipal Wastewater Treatment Facilities"), the upgraded WWTF is estimated to require five personnel after the Phase 1 upgrade and six personnel after the Phase 2 upgrade.

6.5 ESTIMATED CAPITAL COSTS

Planning-level project costs have been prepared for the recommended facilities and are presented in **Table 6-3**. The cost for upgrades to achieve 3 mg/l is provided for information purposes. The cost of the recommended plan is **$50,700,000**, which includes WWTF upgrade to achieve 5 mg/l, the Main Pump Station items and lagoon decommissioning items.

The planning-level costs were developed using standard cost estimating procedures consistent with industry standards utilizing concept layouts, unit cost information, and planning-level cost curves, as necessary. Total project capital costs include allowances of 40% of the estimated construction costs to account for construction contingency, design and construction engineering, permitting, as well as financing, administrative and legal expenses. Many factors arise during final design (e.g. foundation conditions, owner selected features and amenities, code issues, etc.) that cannot be definitively identified and estimated at this time. These factors are typically covered by the allowances described above; however, this allowance may not be adequate for all circumstances. The project cost information presented herein is in current dollars and is based on ENR Construction Cost Index 9846 (August 2014).

As described previously in this report, there are several areas of uncertainty related to existing conditions and this capital cost estimate. Specifically: influent sampling (refer to Section 2); lagoon decommissioning/reuse (refer to Section 5); process selection (refer to Section 5); and, phasing opportunities (refer to Section 6). These items should be resolved as soon as possible.

TABLE 6-3
ESTIMATED CAPITAL COSTS FOR RECOMMENDED PLAN COMPONENTS

PROJECT COMPONENT		**EST. COST** WWTF TN 3 mg/l	**EST. COST** WWTF TN 5 mg/l	**EST. COST** Main Pump Station FM & WM	**EST. COST** Lagoon Decommissioning	**NOTES**
CONSTRUCTION		$36,200,000	$31,400,000	$4,000,000	$4,600,000	1
CONSTRUCTION CONTINGENCY	5%	$1,810,000	$1,570,000	$200,000	$230,000	2
TECHNICAL SERVICES	20%	$7,240,000	$6,280,000	$800,000	$920,000	3
VALUE ENGINEERING		$100,000	$100,000	$0	$0	4
MATERIALS TESTING	0.25%	$90,000	$80,000	$10,000	$10,000	5
ASBESTOS AND LEAD PAINT ABATEMENT		$0	$0	$0	$0	6
DIRECT EQUIPMENT PURCHASE		$0	$0	$0	$0	7
LAND ACQUISITION/ EASEMENTS		$0	$0	$0	$0	7
LEGAL/ ADMINISTRATIVE		$10,000	$10,000	$10,000	$10,000	8
FINANCING	1%	$450,000	$390,000	$50,000	$60,000	9
ENGINEER'S ESTIMATE		$45,900,000	$39,830,000	$5,070,000	$5,830,000	10

Notes
1.) Construction cost estimate details provided in Appendices. Costs based on ENR CCI 9846.
2.) Construction contingency is an allowance at 5% of construction cost.
3.) Technical services is an allowance at 20% of construction cost.
4.) Value engineering is an allowance assuming two sessions.
5.) Materials testing is an allowance based on similar sized projects.
6.) Asbestos and lead paint is not anticipated at the WWTF site, but should be evaluated at the Main Pump Station site.
7.) None anticipated
8.) Legal/administrative costs are for bond counsel and project advertisements.
9.) Financing is an allowance based on assumed interim financing costs at 1%.
10.) DES estimate for 5 mg/l effluent TN for Exeter was $44M ("Analysis of Nitrogen Loading Reductions for WWTF and NPS in the Great Bay Estuary Watershed", Dec 2010, ENR 8660).

6.6 ESTIMATED ANNUAL OPERATION AND MAINTENANCE COSTS

The Town's operations and maintenance (O&M) budget for wastewater collection, treatment and disposal for the 2014 fiscal year was **$467,000**, excluding existing debt service. An O&M budget for the first year of operation (2018) of the upgraded WWTF was prepared based on the estimated increases and decreases for specific line items of the budget. The estimated first year O&M budget for the upgraded facility is **$1,150,000**, excluding debt service, for the Recommended Plan (WWTF Upgrade with Bardenpho for 3-mgd to 5 mg/l effluent TN plus appurtenant facility components. The current budget and the current flows and loads were considered the baseline. A summary of the annual O&M costs is presented in **Table 6-4**. This estimate is based on the assumptions listed below.

- Inflation was assumed to be 10% over the 4 years (2.5% per year) between now and 2018. This was applied equally to all labor, goods, services, chemicals and utilities. Biosolids disposal was assumed to $100/wet ton in current dollars.
- Flows and loads were assumed to increase by 5% over the 4 years between now and 2018.
- The Phase I upgrade is implemented and 2 new staff are hired.
- Major maintenance budgets were held constant (i.e., without inflation).

TABLE 6-4
ESTIMATED ANNUAL OPERATION & MAINTENANCE COSTS
(for 3.0-mgd to 5-mg/l Effluent TN, Bardenpho process, 3 treatment trains)

Category	2014 O&M Costs		2018 O&M Costs	
Salaries	3 staff	$124,000	5 staff	$227,000
Benefits	3 staff	$68,000	5 staff	$125,000
Buildings and System Maintenance	-	$49,000	-	$94,000
Chemicals, Licenses, Software				
Licenses, Software, etc	-	$54,000	-	$59,000
Hypochlorite	17,500 gal	$18,000	1,800 gal	$2,000
Bisulfite	3,250 gal	$6,000	0 gal	$0
Supp Alkalinity	n/a	$0	16,000 gal	$18,000
Supp Carbon	n/a	$0	7,500 gal	$21,000
Polymer	n/a	$0	8,000 gal	$44,000
Utilities				
Natural Gas	-	$11,000	-	$21,000
Electricity	1.1MW-hrs	$134,000	2.0MW-hrs	$260,000
Fuel	-	$2,000	-	$3,000
Gas Monitoring	-	$1,000	-	$1,000
Biosolids	n/a	$0	2,500 wet ton	$275,000
TOTAL		**$467,000**		**$1,150,000**

Section 7

WRIGHT-PIERCE
Engineering a Better Environment

SECTION 7

PROJECT COSTS AND FINANCING

7.1 INTRODUCTION

The recommended plan and its estimated costs are described in detail in Section 6. This section of the report identifies the potential funding sources, the recommended financing scenario as well as the implementation schedule. The recommended facilities are estimated to cost approximately $50.73 million (expressed in 2014 dollars, with inflation to mid-point of construction) to design/construct and will raise the "Treatment" portion of the Sewer Enterprise Fund from $467,000 to $1,150,000 annually to operate (expressed in 2018 dollars). The remainder of the sewer budget will remain unchanged. Therefore, the total annual sewer enterprise fund budget will increase from $2.45 million to $3.15 million, excluding new WWTF debt. The estimated annual debt repayment on a $50.73 million SRF loan is $3.54 million.

The project costs for the recommended plan described herein will have a significant impact on the average sewer user rate. Based on the funding assumptions made herein, the total annual costs associated with the recommended plan is approximately $6.57 million (with no State Aid Grant), which is approximately 168% higher than the current total annual budget for the wastewater collection and treatment system.

7.2 CAPITAL COST FUNDING SOURCES

There are several state and federal agencies from which the Town may be able to obtain financial assistance in the form of grants and/or low-interest loans. If the Town were to act as a regional host, additional funding sources may be available to incentivize a regional solution. These programs are described in the following paragraphs.

7.2.1 New Hampshire Department of Environmental Services

The New Hampshire Department of Environmental Services (NHDES) has several programs available to municipalities for the planning, design, and construction of wastewater infrastructure projects - the State Aid Grant (SAG) program, the SAG Plus program (also referred to as the House Bill 207 Septage Grant program), and the State Revolving Loan Fund (SRF) program. SAG grant funds are available in amounts of 20% of eligible project costs or, if sewer user fees are more than 20% above the state average, the grant amount increases to 30% of eligible costs. Based on the most recent NHDES Sewer User Charge Study (2010), the State average user fee was $575 and Exeter's average residential sewer user fee was $411; however, the projected average residential sewer user fee will be $1,092 with no SAG funding. Based on the above information, Exeter would *qualify* for a 30% grant for the proposed project.

The SAG Plus program provides for grants based on the costs associated with receiving and treating septage at the WWTF. The amount of grant depends on the number of communities served (i.e. 10% for the host municipality plus 2% per additional municipality served up to a maximum of 5 additional municipalities). It is anticipated that the Exeter would *qualify* for a 10% grant for the septage related aspects of the proposed project. Approximately $1,800,000 of the project cost is for septage related aspects of the project and should qualify for a SAG Plus grant. Exeter's septage is currently discharged primarily to the Hampton WWTF.

The SRF loan program provides low-interest loans for the planning, design, and construction of municipal wastewater projects. Loan interest rates vary depending on the repayment period. Currently, 20-year loans are at 3.392% interest. However, the interest rate should be updated soon, and representatives of NHDES have indicated that the new rate will be lower. It is anticipated that Exeter would qualify for and receive an SRF loan for this project.

The SAG and SAG Plus programs have been suspended since the fiscal issues in 2008. However, DES is hopeful that these grant programs will soon be reinstated. DES is accepting applications for the SRF loan program; however, DES issued a moratorium on new SAG and SAG Plus grant applications as of July 1, 2013. Accordingly, we have shown the project financing summary both with and without SAG and SAG Plus funds in Table 7-1at the end of this section.

7.2.2 Aquatic Resource Mitigation (ARM) Fund

One option for offsetting the lagoon decommissioning costs would be the use of the Aquatic Resource Mitigation (ARM) Fund. The ARM Fund is a NHDES grant program where wetland mitigation compensation can be used for wetland restoration design, demolition, construction, legal fees and/or plantings. The restored wetlands would need to be placed in preservation for protection. Lori Sommers, NHDES Wetland Mitigation Coordinator, noted that there is a substantial amount of Seacoast Area grant funds that will be available in 2015 to 2016.

7.2.3 New Hampshire Municipal Bond Bank

The New Hampshire Municipal Bond Bank (NHMBB) has a loan program which provides low-interest loans for the planning, design, and construction of municipal wastewater projects. Loan interest rates vary depending on the repayment period. Currently 20-year loans are at 4.5% interest. It is anticipated that Exeter would qualify for and receive a NHMBB loan for this project.

7.2.4 New Hampshire Community Development Finance Authority

The New Hampshire Community Development Finance Authority (formerly the Office of State Planning) administers the Community Development Block Grant (CDBG) program with funds allocated by the U.S. Department of Housing and Urban Development. Grants are available in several different categories, including public facilities implementation grants for water and wastewater projects. Grant funds of up to $500,000 are available for eligible projects; however, these grants are very competitive. Although Exeter likely qualifies, we have assumed no CDBG funding for this project because it would preclude Exeter from pursuing CDBG funds for other infrastructure projects. If the Town wishes to pursue CDBG funding, we recommended that the Town meet with the Community Development Finance Authority to discuss potential project financing. CDBG applications are due in late January and in late July.

7.2.5 U.S. Department of Agriculture

The U.S. Department of Agriculture also has a grant and loan program, administered by Rural Development, that is available for the planning, design, and construction of municipal wastewater infrastructure projects for communities with a population of less than 10,000. Grant amounts and loan interest rates vary depending on the availability of funds and the median household income of the municipality. The main eligibility criterion is median household income (MHI). Specifically, if the municipality's MHI is below the State average, then it qualifies for up to 45% grant funding; however, if the municipality's MHI is below 80% of the State average, then it may qualify for up to 75% grant funding. The State average MHI based on the 2008-2012 American Community Survey 5-Year Estimates is $64,925. Exeter’s MHI is $72,231. On this basis, the Town would not qualify for any USDA Rural Development grant funding. Since Exeter’s population according to the 2010 U.S. Census was 14,306, the Town would likely not qualify for loan funding from USDA Rural Development either.

7.2.6 U.S. Economic Development Administration

The U.S. Economic Development Administration (EDA) has a grant program for municipal infrastructure construction necessary to attract or increase commercial and/or industrial development. Grants of 50% of project cost, up to a maximum of $1,000,000, are available. One of the primary eligibility criteria is that the project must create or maintain employment opportunities in an economically disadvantaged area. EDA does consider household income when awarding grants. Since Exeter’s MHI is substantially higher than the state average, this will reduce the likelihood of receiving a grant. If the Town wanted to pursue EDA funding, it would need to present a compelling case that jobs would be created or maintained by this project. If the Town wishes to pursue EDA funding, we recommend a meeting with EDA to discuss potential project funding. At this time no EDA funding has been assumed in this analysis.

7.2.7 State and Tribal Assistance Grant

The State and Tribal Assistance Grant (STAG) is an appropriations-based grant for States, tribal and local governments for a variety of water and wastewater infrastructure projects. This grant is administered by the Environmental Protection Agency. This grant requires strong support by Town management, NHDES and the congressional delegation. Grants up to $2 million have previously been awarded, although a more typical grant award is $300,000 to $500,000. It is important to note that Congressional appropriations have recently come under fire, and STAG funding has been considerably reduced. The Town should consider applying for STAG funding; however, no funding has been assumed in this analysis.

7.2.8 Environmental Programs and Management Grant

The Environmental Programs and Management Grant (EPMG) is an appropriations-based grant for State and local governments for infrastructure projects. This grant is administered by the Environmental Protection Agency. This appropriations program has also come under fire recently and, based on conversations with NHDES personnel, grants have typically been reserved for State government agencies in recent history. On this basis, it is unlikely that the Town would receive any grant funding from this program; however, this program would be worth discussing with the US Congressional representative. No EPMG funding has been assumed in this analysis.

7.2.9 Unitil

Unitil provides energy rebate incentive grants for wastewater infrastructure projects. Depending on the design, Exeter should qualify for energy rebate grants for measures implemented to improve energy efficiency of new facilities. Based on our past experience with grants of this type, it is anticipated that the Town could qualify for and receive rebate grants in the range of $25,000 to $50,000. A Unitil grant of $50,000 has been assumed in this analysis.

7.3 SEWER USER FEES

The quarterly sewer user rate is currently $4.44 per thousand gallons for the first 29,999 gallons of water used; $5.23 per thousand for use between 30,000 and 194,999 gallons; and $5.62 for use over 194,999 gallons. In addition, all users pay a service charge of $28.00 per meter per quarter.

For the purposes of this analysis, we have assumed that sewer user rates will be utilized to pay for debt service and O&M costs.

The current annual sewer fee based on the NHDES criterion of 67,389 gallons per year is $411. The implementation of this project will result in approximately a 166% increase in the total annual wastewater collection and treatment budget and, therefore, about a 166% increase in the annual sewer user fee. This will result in an average annual charge of about $1,092 with no SAG or SAG Plus funds.

7.4 INDUSTRIAL PRE-TREATMENT PROGRAM FEES

Sewer users in the Industrial Pre-Treatment Program pay a $100.00 annual Pre-treatment License fee. In addition, industrial users who discharge higher concentrations of biochemical oxygen demand (BOD) or total suspended solids (TSS) than the amounts allowed in the Sewer Use Ordinance pay a surcharge of $17.57 per 100 pounds over the allowable concentrations. Those who discharge excess fats, oil, and grease pay an additional $37.60 per 100 pounds over the allowable concentration.

7.5 OTHER FEES

There are a number of existing and potential "other fees" which could be used to generate revenues for the necessary upgrades. These are presented below.

7.5.1 Existing Fees

Exeter currently has the following additional sewer related fees:

- Out of Town Service Surcharge – Usage Charge plus 15% as permitted by RSAs
- Sewer Hook-up Fee - $300.00
- Sewer Repair/Replace Existing Service - $100.00
- Line Repair/Grease Violations – Actual Costs
- Sewer Assessment Fee – $4.85 per Gallon

- Sewer Call-out Fees - $100.00 First Violation; $250.00 Second Violation; $500.00 Third Violation)
- Emergency Sewer Call-out (non-municipal problem) - $190.00

Exeter currently does not have septage hauler fees or septage disposal fees.

The Town may want to review its rate structure in advance of any WWTF upgrades. For example, the cost of treating biochemical oxygen demand and suspended solids will likely increase substantially with the proposed upgrades. Therefore, the Town should adjust the surcharge fees to those users who discharge pollutants in concentrations that exceed the allowances in the Sewer Use Ordinance to reflect the additional costs.

7.5.2 Potential Future Fees

The Town could consider implementing additional targeted fees, as described below.

- Regional Host Fees – If the Town served as a "host" for regional wastewater treatment and disposal, it could charge "host fees" to the "customer towns". These fees could be a flat fee or a variable fee and would be in addition to the user fees associated with actual flows and loads discharged to the treatment system. Note that the Town does currently charge an additional 15% to individual out of town users. Any additional wastewater flows received from customer towns could result in lower sewer user fees for Exeter users.

- Private Infiltration/Inflow Fees – Private I/I fees could be utilized as a cost-based incentive to have property owners remove private I/I sources from their property (e.g., roof leaders, sump pumps, etc.). If the property owner is unwilling or unable to remove the private I/I source, the Town would receive some additional revenue to account for the additional cost associated with these flows.

- Stormwater Fees – Stormwater fees could be utilized as a method to fund stormwater infrastructure and/or non-point source (NPS) nitrogen which results from stormwater on private property. These fees could be utilized to fund the NPS monitoring, study, tracking/accounting and implementation activities which are required by the AOC.

- Wetland Compensation Bank - If any of the lagoons are restored to flood plains/wetlands for the Squamscott River, a Wetland Compensation Bank (WCB) could be utilized to offset decommissioning costs. Although the NHDES does not presently have WCB regulations in place, they would defer to the EPA and ACOE for guidance. If a WCB were established, the Town of Exeter would be compensated by other project proponents for placing its' wetlands into preservation. Drawbacks to establishing a WCB are that it could take several years for NHDES to consider the wetlands operational and it is unknown if there will be sufficient local projects requiring wetland mitigation.

- Watershed Fees – As noted in Section 7, Exeter is one of 15 communities which contribute nitrogen to the Exeter-Squamscott River watershed. Based on the 2014 GBNNPS completed by NHDES, Exeter accounts for approximately 35% of the delivered load to the watershed. The Town should work with the watershed communities and the State to come up with an equitable methodology to address the costs and benefits associated with nitrogen management.

- Nitrogen Trading – Nitrogen trading is another avenue which is often discussed. The State of Connecticut has developed and implemented a successful Nitrogen Trading Program which has resulted in the removal of a significant amount of nitrogen from WWTFs from the waters of the State since the baseline year of 2000.

Each of the above fee types has advantages, disadvantages and challenges (e.g., public acceptance, administrative complexity, Town Meeting approval, etc.). Analysis of these factors is beyond the scope of this study but should be considered in greater detail prior to advancing towards implementation.

7.6 LOCAL PROPERTY TAXES

Local property taxes currently are not used to fund any portion of the debt for wastewater facilities. At this time no contribution from local property taxes has been assumed in this analysis. However, as was noted in Section 7.3, the sewer user fee will double or triple (depending on availability of SAG funds) as a result of this project if user fees are the sole source of revenue for debt repayment.

Because of this substantial increase, the Town may want to consider using property tax revenue to pay some portion of the debt service. As an example, repaying 25% of the debt with property taxes could possibly reduce the sewer user fee by about 13% while increasing the tax rate by about 2% if no SAG funding is received.

7.7 SEWER FUND

The Town's sewer fund has an unassigned balance (not audited) of $2,027,761 as of May 31, 2014. These funds are not reserved for any specific uses (e.g. unexpended contract commitments, collection system maintenance and repair, collection system inflow investigations, GIS mapping, budget shortfalls, etc.). These funds could be used to reduce the amount of project cost that needs to be borrowed or could be retained for future unanticipated costs. For the purposes of this evaluation, we have assumed that $1,000,000 from these funds will be applied to the project in order to reduce the required borrowing and minimize the financial impact on the ratepayers.

7.8 PROJECT FINANCING SCENARIO

Although there are no grant commitments in place and no guarantees that grant funding will be obtained to help defray the capital cost associated with the recommended facilities, the project financing scenario presented below is believed to be a probable financing plan based on our discussions with the funding agencies as well as our prior experiences. The project financing scenario is presented in **Table 7-1**. ***[NOTE: Items highlighted in yellow can be modified with input from the Town.]***

The most favorable means of securing a long-term note will be through the NHDES SRF program. The NHDES SRF rate is currently 3.392%. However, DES has recently stated that they hope to be able to reduce the SRF interest rate in order to remain competitive with commercial lenders. We have assumed that the project costs will be financed through the NHDES SRF program by 20-year loan at 3.25% interest.

We recommend that the Town begin raising the sewer rates now in order to minimize the immediate impact of such a large rate increase. Doing this will also start generating reserve funds that can be used to reduce any borrowing.

The Town should also recalculate all existing fees, including the BOD and TSS surcharges and develop a surcharge for total nitrogen. In addition, the Town should establish a septage disposal fee.

TABLE 7-1: PROJECT FINANCING SUMMARY

Item	Existing (2014)	With State Aid Grant (2018)	Without State Aid Grant (2018)
Total Project Capital Cost	$0	$50,730,000	$50,730,000
Project Capital Funding			
USDA Rural Development Grant Funds	$0	$0	$0
Community Development Finance Authority Grant Funds	$0	$0	$0
US Economic Development Administration	$0	$0	$0
State and Tribal Assistance Grant Funds	$0	$0	$0
Environmental Programs and Management Grant	$0	$0	$0
Unitil Grant	$0	$50,000	$50,000
Connection Fees	$0	$0	$0
Revenue from Sewer Fund	$0	$1,000,000	$1,000,000
SRF Loan Amount	$0	$49,680,000	$49,680,000
Total Project Funding	$0	$50,730,000	$50,730,000
Annual Budget			
Existing Debt Service	$731,000	$731,000	$731,000
Total Operating & Maintenance Cost	$1,722,000	$2,419,000	$2,419,000
Project Debt Service	$0	$3,420,000	$3,420,000
Less SAG Reimbursement (30% of Project)	$0	-$1,026,000	$0
Less SAG Plus Reimbursement (10% of Septage)	$0	-$180,000	$0
Less Revenue from Property Taxation	$0	$0	$0
Revenue - Septage	$0	$50,000	$50,000
Revenue - Industrial Pretreatment Program	$0	$0	$0
Total Annual Revenue Requirement	$2,453,000	$5,314,000	$6,520,000
Rates			
Average Residential Sewer User Charge	$411	$890	$1,092
% Increase in Residential Sewer User Fee	-	117%	166%
% of Median Household Income (MHI)	0.6%	1.2%	1.5%

Notes:

1. Assumes SRF loan for 20 years at 3.25% interest rate.
2. Assumes 30% State Aid Grant received annually at time of SRF loan payment.
3. Assumes 10% SAG Plus received annually at time of SRF loan payment. Based on 10% of septage related costs.
4. Average residential charge based on NHDES water use criterion of 67,389 gallons per year (90ccf per year).
5. Exeter median household income $72,231 (2008-2012 American Community Survey 5-year Estimates).
6. Septage revenue assumed at 500,000 gallons per year at $0.10 per gallon.
7. ENR Construction Cost Index 9846 (August 2014).

WRIGHT-PIERCE

Engineering a Better Environment