TOWN OF EXETER, NEW HAMPSHIRE

DRAFT

PHASE III INFILTRATION AND INFLOW EVALUATION

January 14, 2013

Prepared by:
Underwood Engineers, Inc.
Portsmouth, New Hampshire
FILE NO. 1542

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Exeter, NH, Flow Monitoring Report, April – June 2009, Flow Assessment Services, LLC, Bedford, NH

Exeter, NH, Flow Isolation Report, April 2009, Flow Assessment Services, LLC, Bedford, NH

Exeter, NH, Manhole Inspection Report, April – June 2009, Flow Assessment Services, LLC, Bedford, NH

Exeter, NH, Building Inspection Report, October 2009, Flow Assessment Services, LLC, Bedford, NH

Numeric Nutrient Criteria for the Great Bay Estuary, NHDES, June 2009

DRAFT – Preliminary Watershed Nitrogen Loading Thresholds for Watersheds Draining to the Great Bay Estuary, NHDES, October 30, 2009

1. EXECUTIVE SUMMARY

This study and report is the third phase of Infiltration and Inflow (I/I) investigations for the Town of Exeter, NH. The first two I/I studies were performed in the early 1990s and the field investigations performed as part of this study built on the recommendations of those reports. This report also serves to provide the Town with strategies for a long term control plan to mitigate Exeter's CSO.

1.1 Summary of I/I Investigations and Findings

The major field investigations performed as part of this study and findings include:

- Public Education and Outreach (pamphlets, web postings, questionnaires, and public meetings)
- Infiltration Investigations:
  - *Continuous Flow Monitoring* of 3 sewer basin "Pilot Areas" was performed from April 8 to June 18, 2009:
    - West Side Drive Pilot Area (~5,500' of sewer, ~99 homes). Flow monitoring in this area showed signs of relatively high inflow assumed to be from sump pumps.
    - Downing Court Pilot Area (~6,500' of sewer, ~76 homes). Flow monitoring in this area showed some signs of dry weather and wet weather infiltration, but limited signs of inflow, and the lowest I/I levels of the three pilot areas.
    - Jady Hill Pilot Area (~5,900' of sewer, ~93 homes). Flow monitoring indicated severe dry weather infiltration, wet weather infiltration, and inflow. This area exhibited the highest I/I levels of the three pilot areas leading to the sewer rehabilitation project currently underway in this area.
  - *Night-Time Flow Isolation* of approximately 75% of the system was performed from April 13 to May 15, 2009 (~144,000' of sewer in Sewer Basins A, B, C, D, F, F1, G, H, I). A total of approximately 750,000 gpd of I/I was observed during flow isolation. Total I/I included: flow measured in the mains which included flow from the unobserved services in the main (72%), flow from services into manholes (22%), manhole leakage (6%).

- *Manhole Inspections* of 651 manholes in Sewer Basins A, B, C, D, F, F1, G, H, I from April 13 to July 14, 2009. A total of approximately 50,000 gpd infiltration was observed during manhole inspections.

- *Internal CCTV Sewer Inspection* of approximately 38,500' of sewer in Sewer Basins A, B, C, D, F, G, I was performed from March 8 to May 18, 2010. The twenty two (22) CCTV areas were selected based on the areas with the highest I/I observed during flow isolation. Approximately 200,000 gpd of I/I was observed during CCTV, approximately 60% of which was observed coming from private service connections. Approximately 440,000 gpd of I/I was observed in these 22 CCTV areas during flow isolation.

- Inflow Investigations

  - *House Questionnaires* were mailed to all 3,200 sewer users on September 8 & 11, 2009. Eighteen percent (18%) of the surveys were returned and approximately 8% of respondents indicated that they had a sump pump that discharged to the sewer or unknown.

  - *House-to-House Inspections* was performed on 243 homes from October 6 to October 28, 2009 in the three "Pilot Areas" indicated above. Twenty one percent (21%) denied entry and 25% of homes that allowed entry were found to have a sump pump that discharged to the sewer or unknown.

  - *Town Performed Smoke Testing* was performed by the Town and the findings were relayed to UE for inclusion in this report. Drainage from portions of Phillips Exeter Academy (PEA) campus, the US Post Office and limited street drainage on Front St. were found to be connected to the sanitary sewer.

- Diversion Structure Investigations

  - A level survey and evaluation of the diversion structures was performed. Subsequently, additional flow monitoring instrumentation was installed in the two CSO diversion structures to better measure CSO discharges and whether the Squamscott River "back-flows" into the sewer.

1.2 Summary of Conclusions

- The Town has taken many steps to improve the operation and maintenance of the wastewater collection system to mitigate SSO and CSO discharges. In particular, operations modifications to reduce the Town water treatment plant waste discharges and more frequent cleaning of the dual 8-inch inverted siphons beneath the Squamscott River should help mitigate future SSO discharges in the vicinity of "Duck Point".

- The Town has significant I/I especially during storm events where 16 mgd peak flow (main pumping station plus CSO) has been observed since improved pumping station metering was installed in the spring of 2010 and improved CSO metering was installed in December 2010.

- Historical CSO metering, though not reliable, suggests peak flows of 30 mgd (main pumping station plus CSO) during severe flooding events. However, there is evidence that Squamscott River water "back-flowed" into the system during some of these flooding events and it is unclear whether the 30 mgd flows were real or a function of false flows measured in the system due to CSO tailwater effects. New CSO metering is now in place to evaluate future CSO tailwater effects.

- Approximately 60% of the I/I observed during I/I field investigations appeared to be from private sources. Private I/I sources include sump pumps, foundation drains, leaking services, roof leaders, etc. Future projects aimed at I/I reduction must include targeting private I/I mitigation to achieve any significant I/I removal.

- New CSO flow metering has revealed that significant direct inflow sources still appear to be connected to the wastewater collection system and that these direct inflow sources contribute to the CSO events because they generate high peak flows in response to rain. Many of these direct connections are believed to be private roof leaders with traps that prevent identification through smoke testing (as was the case for some buildings in PEA). Identification and removal of these suspected direct connections assist with the reduction of CSO events in the future.

- The Town has aging infrastructure that must be replaced over time to maintain the current level of service, and some of the private I/I mitigation approaches that the Town used for the Jady Hill project may be appropriate to implement in future infrastructure projects (pending the measured success of the Jady Hill Project).

- Certain collection system improvements are needed to maintain the current level of serviced regardless of the long-term CSO strategy that is selected. This is due to the age and condition of the existing sewers.

- Using flow measurements since 2010, the main pumping station peak discharge is approximately 7 mgd. This is slightly less than the WWTF permitted peak design flow of 7.5 mgd, so there is limited opportunity to increase main pumping station pumping rates without reevaluation of WWTF permitted design flows and/or improvements to the facility.

- Using flow measurements since late 2010, it may be possible to pump, equalize, and treat peak flows during storm events to reduce CSO discharges. However, significant capital improvements will be necessary. It may be appropriate to complete those improvements when long-term WWTF needs are identified. Since I/I improvements are necessary, it provides an opportunity for the Town to reduce flows prior to the major capital investments. Also, evaluation of pumping and CSO flow records over a longer time frame is required to refine long term design flows.

- A new WWTF is likely in Exeter due to more stringent permit limits. Reducing I/I prior to the new WWTF will reduce costs.

- Preliminary estimates of the anticipated WWTF upgrade and CSO mitigation efforts capital and O&M costs may result in annual sewer bills above 2% MHI, a common benchmark for affordability. Therefore, a well-managed approach is needed to balance the needs of the projects with affordability.

- The most cost effective approach is to complete I/I improvements to reduce peak flows until the needs for the new WWTF are determined. Confirmation of the success of I/I projects should be evaluated every 2-years and adjusted when necessary. Alternative CSO mitigation strategies (such as pump, equalize and blend) should be re-evaluated when the WWTF is needed (Figure 14-1).

1.3 Summary of Recommendations

<u>Flow Monitoring and Measurements</u>

- Provide improved metering at headworks and main pumping station so data can be easily compared with CSO and rainfall data (portions already implemented).

- Add wetwell level to the Flow Assessment Services web-based, pumping station and CSO flow monitoring system so more complete hydraulic evaluations can be performed in the future.

- Provide additional CSO flow monitoring (already implemented in 2010)

- Measure and evaluate the success of the Jady Hill Pilot Project to determine if adjustments in approach are needed.

<u>Additional Evaluations</u>

- Complete remaining items on 'to-do' list from CDM report (Table 2-2)

- Finish evaluating possible private inflow sources as identified in the CDM report (Appendix, Volume 1, A-7)

- Continue to monitor flows on a daily basis to assess the success of I/I projects and to provide design flows for future WWTF projects.

- Develop a policy for dealing with 'private' infiltration and inflow and update SUO appropriately to tackle removal of private I/I in the system. More proactive enforcement actions by the Town in the future may help eliminate some of the private I/I in the system.

- Perform additional inflow investigations starting in Sewer Basins C & I where the Spring St. Diversion Structure flows indicate significant inflow remains, but ultimately pursue private I/I removal system wide.

- Work with homeowners in pilot areas to remove identified and suspected sump pumps and other sources of inflow identified during the house-to-house study (Figures 5-5, 5-6, 5-7). This should be completed in conjunction with completion of the remaining 2 pilot area projects.

- Further evaluate 'suspect' cross country sewers that cross streams and low-lying areas for inflow during spring high groundwater and heavy rainfall. These are locations where ponded surface water or flooded streams/rivers could submerge manholes without being easily visible, since they are not on a street. Examples include basin F from MH 228 to MH 201, basin E between Court St and Linden St.

- Evaluate private pump stations for direct connections to inflow sources. Update SUO to require that private pumping stations provide operation, maintenance, and flow records to the Town.

- Perform visual inspections of manholes during wet weather flow. For example, the cross-country interceptor in basin F has been identified as suspect by DPW personnel, and was reported to have significant inflow entering the manholes from MH 210 to 201 by the TV inspection crew. Visual inspections should be made during wet weather to investigate this situation.

- The Town should CCTV inspect 20% of the collection system annually and incorporate findings into a long-term sewer asset management plan.

- The Town should continue smoke and dye testing to identify direct inflow sources and evaluate CSO outfall modifications to mitigate the risk of Squamscott River "back-flow" into the system.

Capital Projects (See attached CIP Schedule)

- Complete the balance of the Pilot Projects. Two areas remain. The pilot areas will further refine the Town's approach with private I/I.

- Begin annual budgeting for sewer manhole rehabilitaiton. Manholes should be repaired as prioritized by the manhole inspections (Table 9-1 & 9-2). However, manhole rehabilitation should also be coordinated with routine sewer main evaluations so that rehabilitated manholes are not replaced as part of sewer infrastructure management projects. We have included a $300,000 allowance to address manhole deficiencies identified in this report.

- Complete I/I Improvements in a prioritized system to reduce I/I (and CSOs).

- Provide capital budgeting for ongoing sewer collection system improvements. We have included a $26,000,000 allowance to address I/I peak flows and sewer deficiencies identified in this report. Once I/I projects are no longer being pursued or needed, the Town should budget $500,000 to $650,000 per year to maintain the current level of service. This $500,000 to $650,000 per year budgetary estimate is based on the approximate 48.5 miles of Exeter wastewater gravity collection system and an assumed replacement metric of approximately $1,000,000 to $1,300,000 per mile of gravity sewer divided over 100-years. However, an asset management plan would refine these figures and help prioritize projects. Please note that this $500,000 to $650,000 per year budgetary figure only includes mainline upgrades to maintain the current level of service and does not include private sewer separation required to effectively remove the private I/I in the system. Projects that include comprehensive improvements and private sewer separation, such as the Jady Hill Project, can cost $3,000,000/mile.

- Reassess the recommendations of this report at a frequency of no less than every 2-years and when a new WWTF is needed. When the WWTF is needed consider designing the new WWTF with equalization storage to accommodate storm flows. It may be appropriate at that time to construct a 'high-flow' pumping station and new force main with flow equalization. Schedule to be defined by affordability and the schedule of the anticipated WWTF upgrade (Figure 14-1).

FIGURE 14-1

Exeter I/I and CSO

Decision Matrix

February 9, 2010

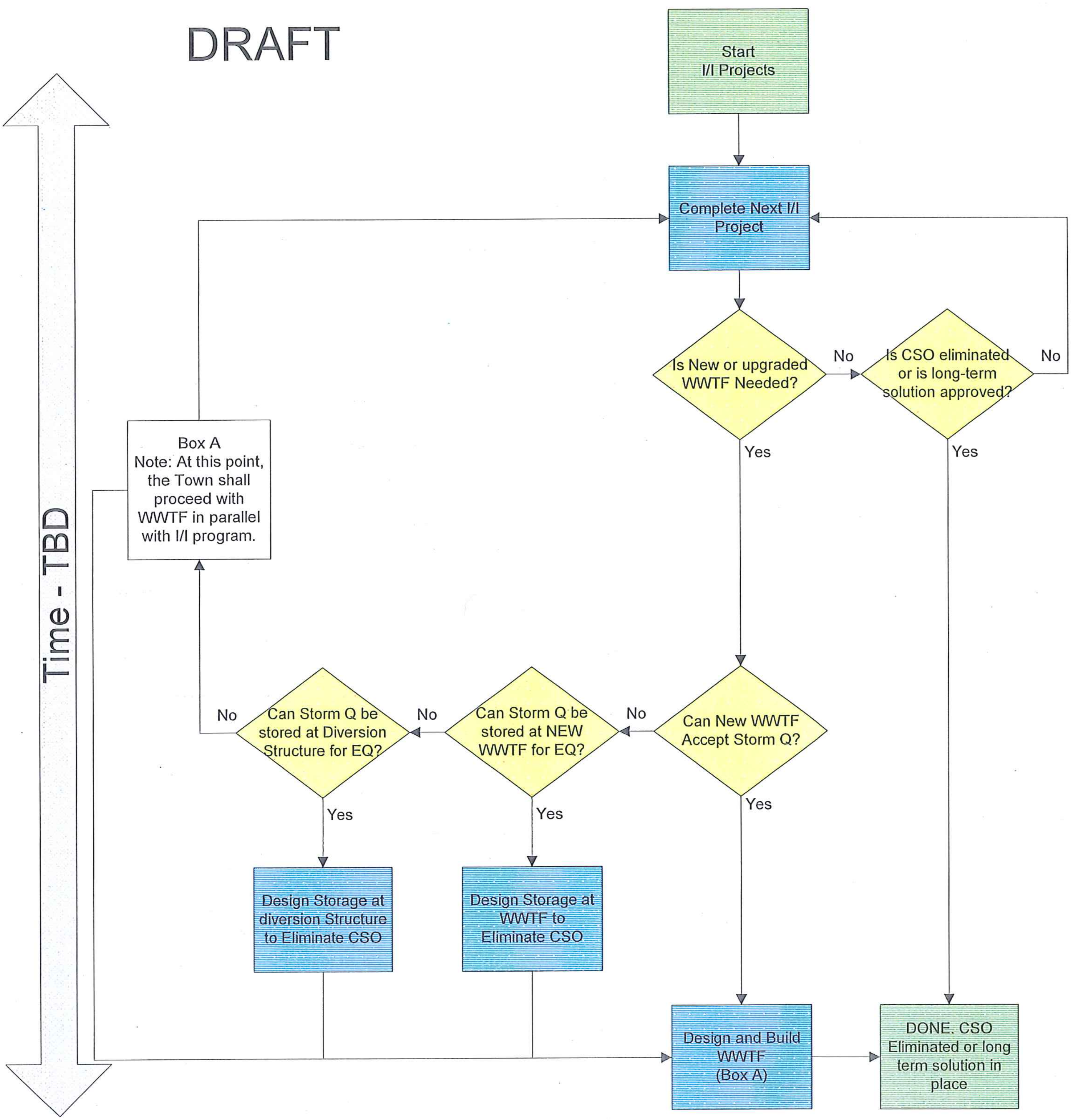


Table 14-1

Suggested CSO LTCP Sewer Implementation Schedule and Cash Flow - 5-Year Plan

Sewer Improvement Project/Program	Total Budgetary Cost [3,4,5]	Project Year 2013	2014	2015		2016	2017		2018	2019	2020	2021	2022	2023	2024	2025	2026	2027
WWTF Improvements [2]																		
Facility Plan	$375,000	$375,000																
Design	TBD		TBD	TBD														
Construction	TBD					TBD	TBD											
Phase I On-Line (8 mg/L) [9]	TBD								*									
Non-point Nitrogen Evaluations and Controls [9]	TBD	TBD	TBD	TBD		TBD	TBD		TBD	TBD	TBD	TBD	TBD	TBD				
Phase II On-Line (3 mg/L)- If Necessary, TBD [9]	TBD																	
Long Term CSO Control Plan					Confirmation			Confirmation										
Submit Report		*																
Jady Hill Project [1,6]																		
Construction	$3,436,000	$3,436,000																
Evaluation/Assessment	$20,000		$20,000															
Additional Evaluations/Monitoring/TV/Implementation	$515,000		$265,000			$250,000												
Manhole Rehabilitation			$60,000	$40,000		$11,000	$11,000											
Downing Ct./Westside Drive [1,8]																		
Design	$40,000			$40,000														
Construction/Implementation	$500,000			$500,000														
Evaluation/Assessment	$40,000					$40,000												
Subtotal Additional I/I Projects LTCP Driven		***$3,436,000***	***$345,000***	***$580,000***		***$301,000***	***$11,000***											
Sewer Collection CIP [7]																		
Portsmouth Avenue Sewer	$940,000	$940,000																
Lincoln Street Sewer	$196,000		$196,000															
Sewer Line Replacement	$1,700,000			$850,000			$850,000											
Subtotal Existing CIP Sewer Projects		***$940,000***	***$196,000***	***$850,000***		***$0***	***$850,000***											
ANNUAL TOTAL LTCP AND EXISTING SEWER CIP (WWTF COSTS NOT INCLUDED)		***$4,376,000***	***$541,000***	***$1,430,000***		***$301,000***	***$861,000***		***$TBD***	***$TBD***	***$TBD***	***$TBD***	***$TBD***	***$TBD***	***$TBD***	***$TBD***	***$TBD***	***$TBD***
		5-YEAR LTCP COMMITMENT (I/I) $3.34M Jady Hill + $1.24M Additional							***10-YEAR PHASE II LTCP Costs TBD if needed***									

Notes:

1 Pilot areas should be done initially to further refine private I/I approach.
2 A new WWTF may be needed due to revised permit limits. The schedule for this new facility is not known at this time. The above schedule should be reviewed/adjusted when the schedule and cost of the new WWTF is known.
3 All expenditures and projects indicated above are pending Town authorization through voting.
4 Reassessment of affordibility and approach of the program should be performed at a minimum of every 2-years and during critical milestones such as pilot area implementation, WWTF upgrade, and main pumping station improvements.
5 Budgetary project costs are present day and have not been escalated for the time value of money.
6 Jady Hill Project costs includes sewer related expenses only.
7 Sewer Collection CIP is a draft plan only.
8 Assumes enforcement only in Westside Drive.
9 Schedule is based on US Environmental Protection Agency (EPS) draft Administrative Compliance Order (ACO).

2. INTRODUCTION

2.1 Background and Objectives

The Town of Exeter, New Hampshire (Town) is located in Rockingham County in the southeast corner of New Hampshire as shown on Figure 2-1. The Town of Exeter owns and operates a municipal Wastewater Treatment Facility (WWTF) and a wastewater collection system. The Main Sewage Pumping Station, located near Swazey Parkway, pumps wastewater from the collection system to the WWTF. The WWTF discharges treated effluent to the Squamscott River, a tidal river of the Great Bay estuary.

The Exeter sewer collection system was originally a combined system with combined sewer overflow (CSO) structures. Since the construction of the municipal Wastewater Treatment Facility (WWTF) in the mid-1960s, the Town has worked to reduce the amount of untreated wastewater discharged to the Squamscott River. As such, only two diversion structures and one permitted CSO structure remains.

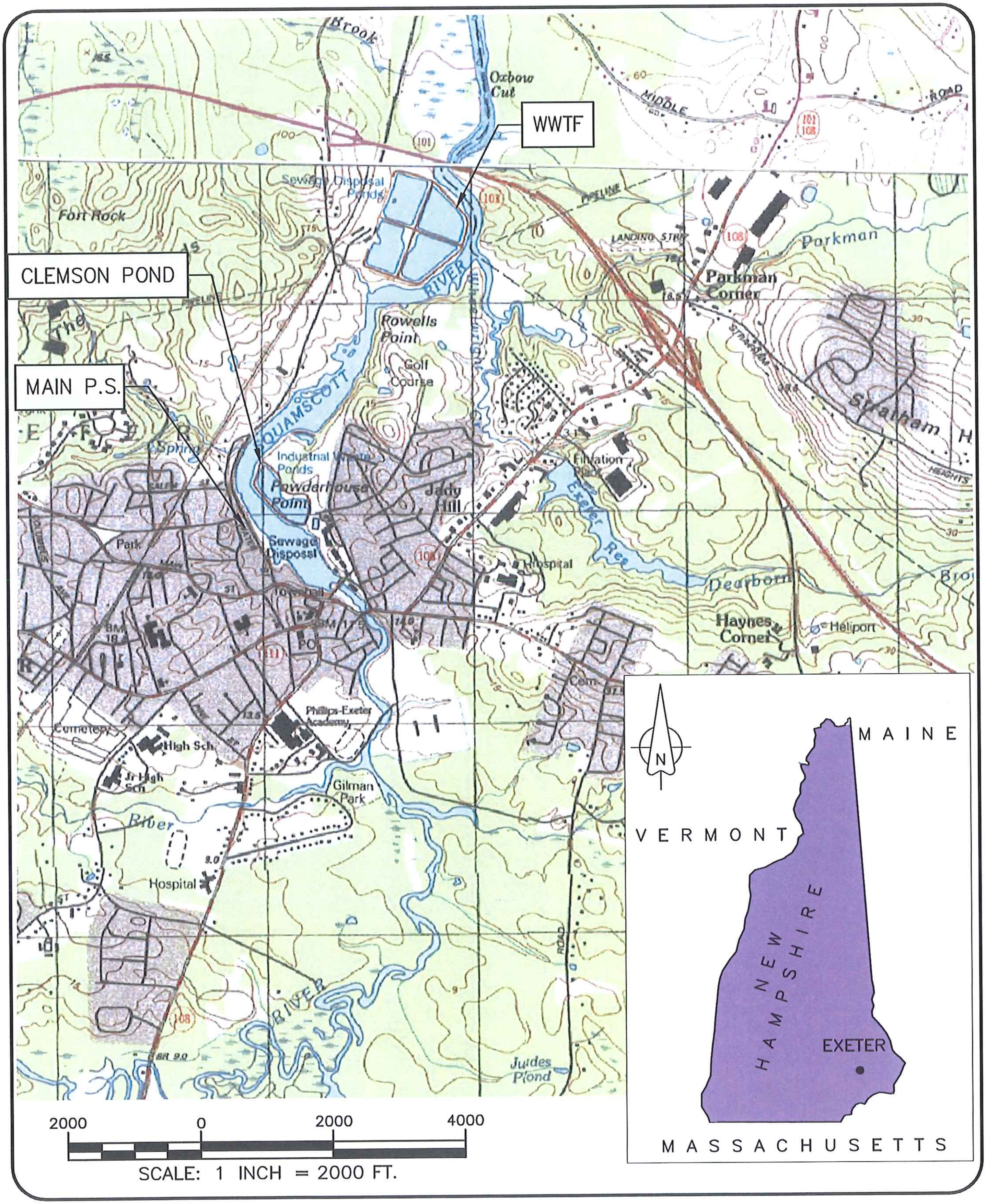
Over the last 10-20 years, the Town has performed extensive studies to identify the sources of I/I to target wastewater collection system improvements in attempt to reduce CSO events. These large capital projects have primarily focused on the removal of inflow through the separation of public drainage systems. The Town believes that public sewer separation is complete, yet CSO events, although reduced, still occur during certain rainfall events. The Town is therefore continuing efforts to reduce remaining I/I with an emphasis on infiltration and private services. As required by EPA's 1994 CSO Control Policy, permittees with CSOs must ultimately comply with the requirements of the Clean Water Act (CWA) which means the discharges must meet Water Quality Standards (WQS).

In 1997 a two phase I/I study was conducted by Camp Dresser and McKee (CDM). This study included continuous flow monitoring of all 9 sub-basins to evaluate the volume of I/I in each basin. The study also included smoke testing to identify specific inflow sources for targeted removal. The CDM study recommended specific separation projects, removal of specific inflow sources, and further studies to evaluate infiltration in the system. The third phase planned at the time of the 1997 study was an infiltration study, but this work was not completed at that time. The basis of the current study is to complete the infiltration investigation portion of the previous I/I study, and develop a Capital Improvement Plan (CIP) for the Town.

The primary objectives of this study are follows:

- Build on the findings of previous I/I studies
- Perform field investigations to further evaluate/quantify infiltration previously identified as the next major step to reduce wet weather flows

- Identify cost effective projects to reduce I/I
- Identify cost effective CSO control strategies
- Develop a 20-year Capital Improvement Plan for removing I/I and reducing/eliminating CSOs.
- Establish pilot study areas so that the Town can evaluate the effectiveness of small scale I/I removal projects after construction.
- Establish recommendations for the control of private inflow sources.
- Frame all recommendations within the ultimate goal of meeting WQS for the long-term or to eliminate CSOs at some point in future.
- Develop a process to evaluate and balance CSO mitigation with the need for the anticipated new WWTF, if possible and if cost-effective.



Location Map

I&I Evaluation Report
Exeter, New Hampshire

UNDERWOOD engineers
25 Vaughan Mall, Portsmouth, N.H. 03801
Tel. 603-436-6192 Fax. 603-431-4733

DATE 7/30/10

FIGURE 2-1

2.2 Scope of Work

The purpose of this study was to provide evaluation and cost effective recommendations for infiltration and inflow (I/I) removal projects for the Town of Exeter, including a comprehensive 20 year planning period Capital Improvements Plan (CIP) identifying projects for removal of I/I and project phasing. This work was intended to build off of previous I/I studies completed for the Town.

The following tasks were included in this study and are described in more detail in Appendix A-1:

Task 1 – Information Collection

Task 3 - Public Education and Outreach

- Pamphlets
- Web postings
- Mailing with questionnaire (See Task 4)
- Public Meetings (See Task 2)

Task 2 – Meetings and Work Sessions

Task 4 – Field Investigations

<u>Infiltration Investigations</u>

- Continuous Flow Monitoring
- Flow Isolation
- Manhole inspections
- Internal CCTV Inspection

<u>Inflow Investigations:</u>

- House questionnaires
- House to House Inspections

<u>Diversion Structure Investigations</u>

- Evaluation and calibration
- Level survey

<u>WWTF Drainage Investigation</u>

Task 5 – Engineering Evaluation

- System Evaluation
- I/I Reduction Strategies
- Overall CSO Strategies

- Planning Level Design

Task 6 – Budgeting and Planning

Task 7 – Rate Study - deleted by contract Amendment #1 on December 27, 2008

Task 8 – Funding

Task 9 – Engineering Report

2.3 Definitions

The definitions related to infiltration/inflow and CSOs, unless otherwise noted, are provided from Metcalf & Eddy, *Wastewater Treatment and Reuse*, Fourth Edition, 2003, Federal Register, and the Commonwealth of Massachusetts, Department of Environmental Protection, *Guidelines for Performing Infiltration/Inflow Analyses and Sewer System Evaluation Survey*, January 1993 and are presented in Appendix A-2. A few key definitions are repeated here:

Infiltration: (Extraneous) (Ground)water entering a collection system from a variety of entry points including service connections and from the ground through such means as defective pipes, pipe joints, connections, or access port (manhole) walls.

Total Inflow: The sum of the direct inflow at any point in the system plus any flow discharged from the system upstream through overflows, pumping station bypasses, and the like (is assumed to also include the entry of stormwater into the system from direct connections, roof leaders, sump pumps, catch basins, leaky manhole covers, etc.).

Private Infiltration/Inflow Source: An infiltration/inflow source emanating from private property and discharging to the public sewers (The Exeter Sewer Use Ordinance considers all services from the main to the building as private).

Combined Sewer Overflows (CSOs):

1.) "CSOs are flows from a combined sewer in excess of the interceptor or regulator capacity that are discharged into a receiving water without going to a publically owned treatment works (POTW). CSOs occur prior to reaching the headworks of a treatment facility and are distinguished from bypasses which are 'intentional diversions of waste streams from any portion of a treatment facility'"(54 FR 37370, September 1989)

2.) "A CSO is the discharge from a combined sewer system (CSS) at a point prior to the POTW Treatment Plant." (Federal Register, Volume 59, No. 75, April 1994) An SSO occurs if a discharge is reported during dry weather anywhere in the system, including permitted CSO structures or during wet weather if not occurring at the CSO structure.

Sanitary Sewer Overflows (SSOs): "Untreated or partially treated sewage overflows from a sanitary sewer collection system" (EPA, NPDES glossary)

Combined Sewer: A sewer intended to serve as both a sanitary and storm sewer.

Combined Sewer Systems (CSS): "A wastewater collection system owned by a state or municipality (as defined by section 502(4) of the CWA) which conveys sanitary wastewaters (domestic, commercial, and industrial wastewaters) and storm water through a single-pipe system to a Publically Owned Treatment Works (POTW) Treatment Plant (as defined in 40 CFR 403.3(p))." (Federal Register, Volume 59, No. 75, April 1994)

Storm water: "Storm water means storm water runoff, snow melt runoff, and surface runoff and drainage." (40 CFR 122.26(b)(13))

2.4 Prior Reports, Studies and Plans

2.4.1 Recent Reports

The following is a list of previous reports on the Town's sewer system:

- Wastewater Treatment Investigation, Town of Exeter, January 1972, Weston and Sampson Engineers.
- Facilities Plan for Wastewater Collection and Treatment, Town of Exeter, NH, March 1975, by Wright, Pierce, Barnes & Wyman
- Water Quality Benefits, Proposed Combined Sewer Overflow Control Projects, January 1985, by New Hampshire Water Supply & Pollution Control Commission
- Proposed Sewage Collection and Treatment Improvements, September 1985, by Jones and Beach Engineers, Inc.
- Wastewater Treatment Alternatives, Exeter, New Hampshire, June 1987, by Hoyle, Tanner & Associates, Inc.
- Phase I Infiltration/Infow Study, Sewer System Evaluation Survey and Combined Sewer Overflow Study, Town of Exeter, New Hampshire, October 1997, Camp Dresser & McKee (CDM)
- Draft Phase II Infiltration/Infow Study, Sewer System Evaluation Survey and Combined Sewer Overflow Study, Town of Exeter, New Hampshire, October 1997, CDM

2.4.2 Status of 1997 (CDM) Report Recommendations

The Phase I and II Infiltration/Inflow Studies by CDM (1997) were the most recent I/I studies performed for the Town of Exeter. At that time of that study, three phases were planned:

Phase I -	Sewer System and CSO Evaluations – completed 1997
Phase II -	Inflow Studies and Recommended Improvements – completed 1997
Phase III -	Infiltration Study – this report

Phases I and II were completed by CDM in 1997, but Phase III was not authorized until now. This current study is intended to complete Phase III of the planned I/I work.

Continuous flow monitoring was conducted in the Exeter collection system by CDM as part of the Phase I study in 1997. The collection system was divided into 9 sub-basins identified as A through I (Figure 2-2). As a result of this continuous flow monitoring, CDM concluded that sub-basins B, D and G had the highest infiltration rates. CDM recommended that sub-basins F, G and H should be further investigated for infiltration. A total of nearly 2 mgd of infiltration into the system was estimated by CDM.

Smoke testing was recommended and conducted in the entire system (basins A through I) to identify inflow sources that required removal. Additional dyed water testing of specific potential inflow sources was conducted. A total of nearly 10 mgd of inflow into the system was estimated.

A specific recommendation of the CDM report was to initially pursue an infiltration program for sub-systems F, G and H, which exhibited high rain-dependent infiltration problems, to achieve control of CSOs during a five-year storm. The Phase I report (section 5.4) identified the following excessive quantities of dry-weather infiltration:

Subsystem F:	4,000 gpdim
Subsystem G:	11,111 gpdim
Subsystem H	6,364 gpdim

The report recommended subsequent infiltration investigations of subsystems B, C, D and I.

The 1997 Phase I/II report by CDM examined the following five (5) alternatives "*to eliminate CSO discharges and to have acceptable levels of surcharging during a five-year design storm*". It is understood that CSO events would still occur during rain events with a 5-year intensity even if all the projects were complete.

- ***Alternative 1*** *– Remove identified inflow sources and rehabilitate manholes*
- ***Alternative 2*** *– Remove identified inflow sources, perform sewer separation and rehabilitate manholes*
- ***Alternative 3*** *– Remove identified inflow sources, rehabilitate manholes and provide storage instead of sewer separation*
- ***Alternative 4*** *– Remove all inflow sources, perform sewer separation and rehabilitate manholes*

- ***Alternative 5*** *– Remove identified inflow sources, perform sewer separation, rehabitate manholes, and conduct infiltration program in selected subsystems*

CDM recommended ***Alternative 5*** which included certain specific projects. The status of those projects is summarized below and is based on the November 27, 2002 letter to NHDES from the Town and the Town's 2008 308 response (Appendix A-10):

- ***Full separation of the Court, Elm and Center Street area***

  Status: In the November 27, 2002 letter to NHDES, the Town (in their 308 response) indicated that this work was completed in 2000.

- ***Removal of field-identified inflow sources.*** *CDM summarized the location of public and private inflow sources in Tables 2-3 and 2-4 of the Phase II report.*

  Status: In the November 27, 2002 letter to NHDES (and 2008 308 Response) the Town indicated that public inflow sources were eliminated. However, private inflow sources remain. The Town is still working on investigating a lengthy list of potential private inflow sources (mainly roof leaders).

- ***An infiltration program for subsystems F, G and H.*** *CDM identified that subsystems F, G, and H exhibited high rain-dependent infiltration as well as excessive quantities of dry-weather infiltration, and therefore recommended prioritized flow isolation/rehabilitation for these subsystems. In addition, CDM recommended that flow isolation/rehabilitation subsequently be performed in subsystems B, C, D and I.*

  Status: UEI performed the recommended flow isolation as part of this study and the results are discussed in later sections of this report.

- ***Pipe Capacity Improvements.*** *CDM recommended the following pipe capacity improvement projects:*

  - *Pipe enlargements from a 12-inch to an 18-inch diameter pipe downstream of the Water Street diversion structure*

  - *Elimination of reverse-slope pipes in the vicinity of SMH 244 and SMH 377 around High Street.*

  - *Enlarge pipes from 8-inch to 18-inch on Industrial Drive and Epping Road as industrial development occurs (although this is not an I/I driven project).*

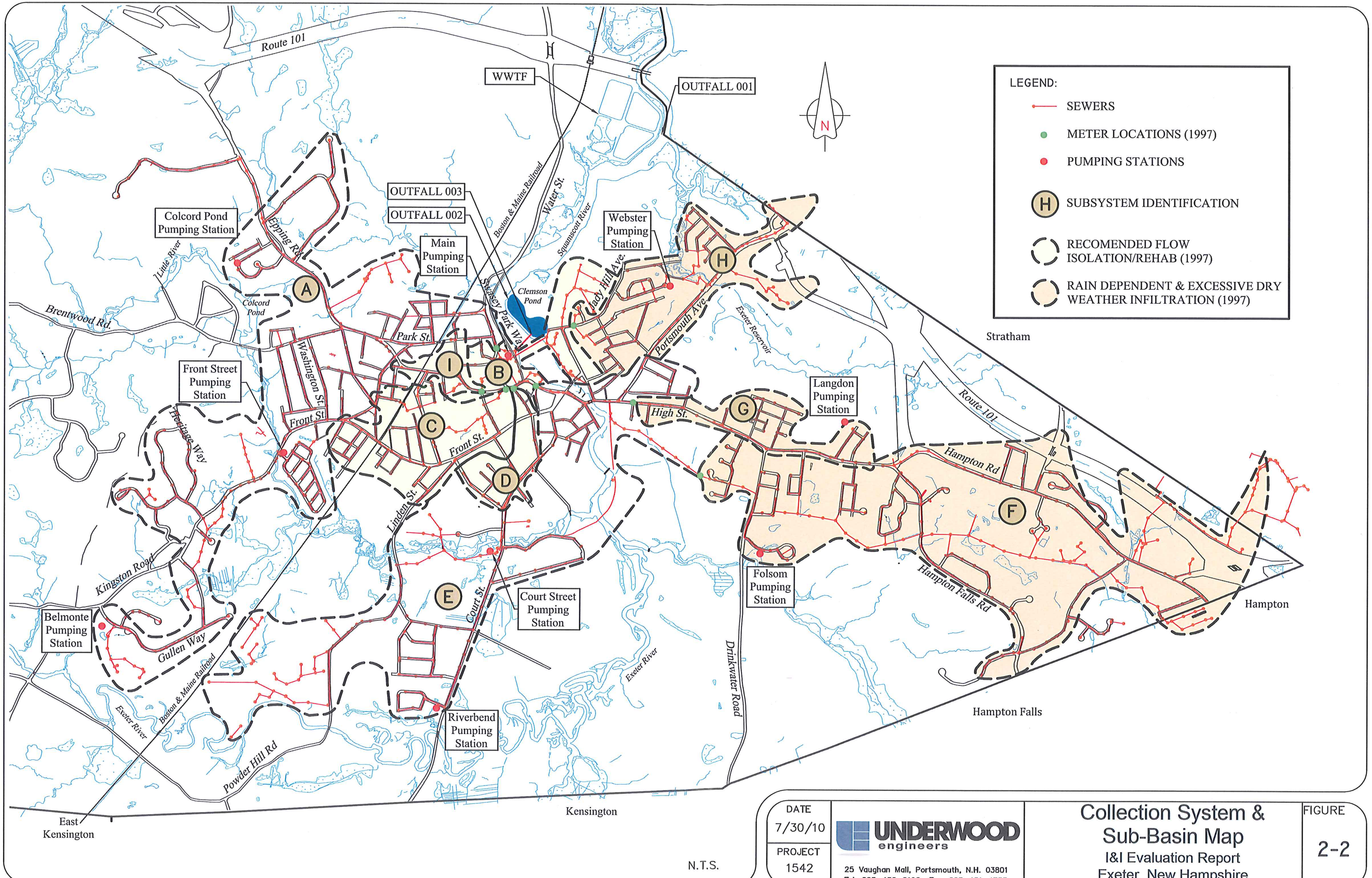
  Status:

  - The pipe enlargement downstream of the Water Street diversion structure has been designed but has not been constructed. The Town authorized the funding of the work through the American Resource and Recovery Act (ARRA), but is currently waiting on confirmation from the NHDES since the funds have not been made available yet.

- In the November 27, 2002 letter to NHDES the Town indicated that the reversed slope pipe in the vicinity of SMH 377 was replaced in 2002 as part of a bridge replacement project that year, and was replaced with an 18" PVC pipe as recommended.

- Based on a June 29, 2010 discussion with Town personnel the reverse pipe slopes in the vicinity of SMH 244 has been eliminated.

- In the November 27, 2002 letter to NHDES the Town indicated that pipe enlargements on Epping Road was completed in 1998. Based on the Town GIS it appears that pipe enlargements have occurred on Industrial Drive as required, though 15-inch and 12-inch pipe was installed.



DATE	PROJECT
7/30/10	1542

UNDERWOOD engineers
25 Vaughan Mall, Portsmouth, N.H. 03801
Tel. 603-436-6192 Fax. 603-431-4733

Collection System & Sub-Basin Map

I&I Evaluation Report
Exeter, New Hampshire

FIGURE 2-2

As noted above, the projects recommended by CDM (Alternative 5) were made based on eliminating CSO events up to a 5-year storm with the understanding that CSO events will still occur in conjunction with rainfall events more intense than the 5-year storm. However, January 8, 2003 correspondence from NHDES to the Town (Appendix A-10), indicates that "additional CSO controls may be needed as EPA's CSO Control Policy requires that all CSO discharges, regardless of their frequency or volume, be in compliance with the State's Surface water Quality Regulations" (Appendix A-11). Therefore, control of CSO events for the 5-year storm may not be adequate to meet the Town's Long Term CSO control needs. Further discussion of EPA's CSO Control Policy is included in Section 2.7 of this report.

2.4.3 Additional Town Efforts

As part of the Town's response to the EPA's 308 request, two lists of projects completed to date were presented. One list included projects and work that were completed by the Town's Sewer Department. The second list included projects and work that were completed by private property owners. Complete lists of the projects are included (Appendix A-5).

The Town's 308 Response, Appendix II also included two lists of possible private inflow sources (primarily roof leaders), identified as Table 3-1 through 3-3 (original list) and Table 4-1 through 4-2 (most recent list). Also included in the Appendix is a letter from the Town to the NHDES dated November 27, 2002 referencing these tables and indicating that these private inflow sources remain. It appears that several sources appear to have been investigated and found not connected, or removed. If the remaining private inflow sources (approximately 70) have not yet been pursued, the Town should again begin investigation of these private inflow sources. The list identified as Table 4-1 and 4-2 from the 308 response appears to be the current 'working' list, and as such has been included (Appendix A-7) for reference.

2.4.4 Summary of Projects since 1995

Table 2-1 contains a summary of major efforts completed by the Town since 1997 to remove I/I from the system and reduce CSOs:

Table 2-1
Summary of Projects Completed Since 1995

Town Efforts/Sewer Work since 1995

Description	Approx. Year
• Service investigations (CDM Recommendations)	1998-2000
• TV work	On-Going
• C/B removal (CDM Recommendations)	1998-2000
• Raised Manhole @ Duck Point (Jady Hill)	2010
• Clean/Jet Problem Sewers	On-Going/Quarterly
• Identification of former High School Roof Leaders	On-Going
• CIP Lined Linden St. Sewer (SMH 447-446)	2010
• Clean/Jet Sewer Siphons	On-Going/2009
• Webster Upgrade	

Contract Work since 1995

Project	Description	Approx. Year
• Water Street	1,100' (8" Sewer) & Drainage	2000
• Court Street	Drainage & Minor Sewer	2000
• Epping Road Sewer	4,000' (15" Sewer)	1999
• High Street Sewer	830' (8" Sewer) & 270' (12" Sewer)	2002
• Portsmouth Ave. Sewer	2,500' (8" Sewer) & 280' (12" Sewer)	2005
• Portsmouth Ave. (2) – Pend.	2,400' (8" Sewer) & 300' (12" Sewer)	TBD
• Lantern Lane	Drainage and Minor Sewer	2006
• Meadowoods Sewer	1,200 (8" Sewer)	2008
• Belmonte/High Street PS	Pump Station Upgrade	2005
• Main Street PS	Pump Station Upgrade	2004

Since 1997, it is estimated that the Town has completed replacement sewers as follows:

- 5,630' (8" Sewer)
- 850' (12" Sewer)
- 4,000' (15" Sewer)

In doing these replacements and other specific separation projects, the Town has removed about 10 mgd of inflow (based on CDM estimates for a 1-inch rain event).

Recommended inflow removal projects were included in Tables 2-3, 2-4, 2-5 and 2-6 of the 1997 CDM study. Projects in these tables were compared to projects identified as completed by the Town in their response to the EPA 308 request, as noted above. In addition, projects that Town personnel indicated to us as complete were also removed from the list. Remaining projects that were recommended in the 1997 CDM report, but do not appear to have been completed to date based on the Town's response to EPS's 308 inquiry and discussions with Town personnel, are summarized below in Table 2-2. These projects were investigated by the Town, but either nothing was found, or there were extenuating circumstances why the issue could not be addressed. Given the estimated magnitude of inflow associated with the Water Street source, we recommend that area be smoke tested again, and perhaps investigated further.

Table 2-2
Inflow Sources Identified in CDM Report, 1997
but not completed as of August 2010

Source (CDM rpt)	Sub Basin	From M.H.	To M.H.	Location	Estimated Inflow (gpd)	Findings
Table 2.5	A	774	792	Epping Rd		Lt. to moderate smoke @ catch basin between 48 and 50 Epping Road (Town investigation could not locate issue. Town believes CB disconnected during Epping Road project)
Table 2.4	B	895	894	Water St.	890,244	Smoke from hole in ground with brick work present @ end of swale (Town investigation could not locate issue, so issue may still remain)
Table 2.4	F	115	114	Hampton Rd	104	Smoke from ground @ 1 Wayside Drive - possible open joint and/or broken house service (Town personnel investigated, private sewer service issue)
Table 2.4	H	325	324	Hall Place	12,018	Smoke from driveway drain @ 33 Hall Place - direct (Town investigation found no practical solution/outlet for drain, private drain, would require pumping stormwater)
					902,366	

These projects were recommended for inflow removal in the 1997 CDM report. Per CDM, the estimated inflow that would be removed from the system by these projects was about 10 MGD. According to the CDM report, this inflow was estimated using the rational formula with a rainfall intensity of 1-inch/hour and runoff coefficients of 0.9 for asphalt and rooftops and 0.3 for grass and native soils.

In summary, the CDM report recommended that the Town pursue removal of about 10 mgd of inflow and 2 mgd of infiltration (based on a 1.7-inch storm). The Town has completed nearly all of the public and private inflow projects that were recommended, which should have resulted in the removal of approximately 10 mgd of inflow. However, based on current flow records (February through June 2010), it appears that the Town may still be experiencing up to 25 MGD+ of I/I during significant rainfall events (greater than 1.7-inches). But, these recorded historical high CSO flows are suspect due to inaccurate metering and as a result of tailwater effects at the CSO weirs. The town installed new metering in 2010 to more accurately quantify future CSO flow as discussed in later sections of this report.

3. EXISTING WASTEWATER SYSTEM

3.1 Existing Wastewater Collection System

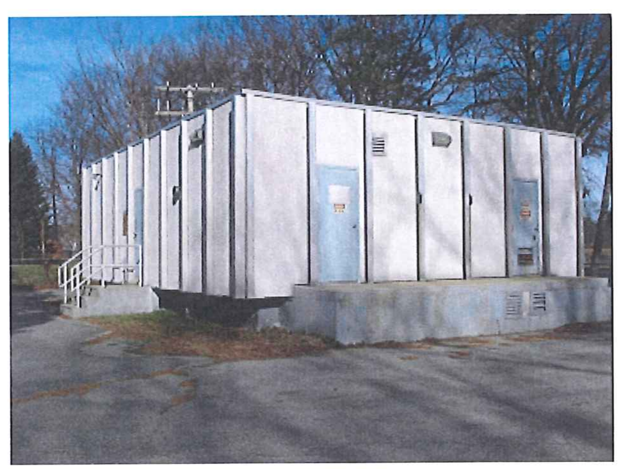
According to the Town's 308 response to EPA in April 2007, Exeter's wastewater collection system includes 48.5 miles of separate gravity sewer, 20,346 linear feet of force main, 9 publicly owned and operated pumping stations and 2,150 linear feet of 'combined' gravity sewer. This 2,150 linear feet of combined sewer is the piping between the two CSO diversion structures and Clemson Pond, since it carries combined flows during rain events. The wastewater collection system is composed of a combination of vitrified clay (VC), asbestos cement (AC), reinforced concrete (RC), cast iron (CI) and polyvinyl chloride (PVC). Generally, the VC pipes are believed to be over 100-years old in some areas, the AC pipes are generally believed to be 50 to 60 years old, and the PVC and RC pipes are less than 50 years old.

Wastewater from the entire Exeter sewer collection area, including some portions of Stratham and Hampton, is conveyed to the Main Pumping Station which is located between Water Street and Swazey Parkway. The Main Pumping Station pumps wastewater to the Exeter Wastewater Treatment Facility (WWTF) located on the Squamscott River north of Town.

See previous Figure 2-2 for a schematic of the wastewater collection system.

3.2 Existing Wastewater Treatment System

The existing Wastewater Treatment Facility (WWTF) is an aerated lagoon facility, providing secondary treatment prior to discharging to the Squamscott River. The plant was constructed in 1965 and underwent major upgrades in 1990. The plant is designed for an average daily flow of 3 mgd and a peak flow of 7.5 mgd.



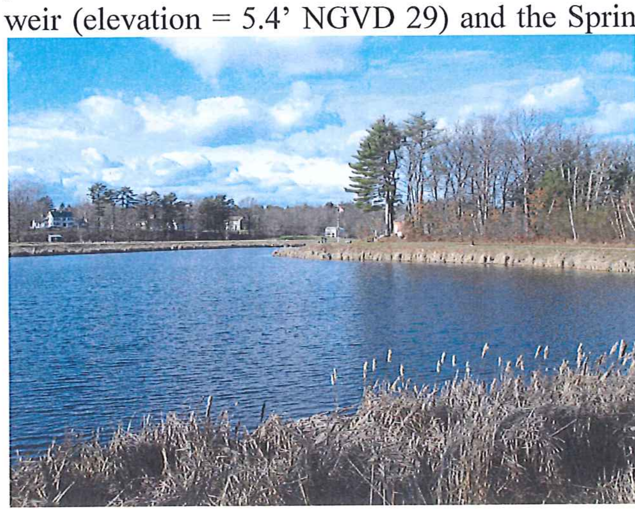
Wastewater from the Main Pumping Station enters the plant by flowing through a grit chamber before being conveyed to the aerated lagoons. Floating mechanical aerators are used to aerate the lagoon. Plant effluent is disinfected with sodium hypochlorite and dechlorinated before being discharged to the Squamscott River.

Influent flow data at the WWTF is generated from an area-velocity meter in the bottom of the influent channel, which is reportedly subject to wave action, and does not necessarily have a free-flowing condition. Therefore the accuracy of data from this meter is questionable. To remedy the inaccuracies of the area velocity influent meter, the Town installed strap-on Doppler meters at the main pumping station in February 2010, and a magnetic flow meter on the WWTF influent forcemain in August 2010 which is now used for WWTF influent flow meter reporting purposes.

Effluent flow at the WWTF is measured at the parshall flume by an ultrasonic level transmitter. The effluent flow from the lagoons to the contact tank can be controlled by a motor operated valve, so the amount discharged is controlled by the operators and not necessarily by what comes into the plant. Lagoons 1, 2 and 3 each have volumes of 26 Million Gallons (MG), 28.7 MG and 22.8 MG, respectively. The total volume in all three lagoons is 77.5 MG. The existing average flow is approximately 2 MGD. There is a 30-40 day detention time through the lagoons to provide secondary treatment.

3.3 Existing Diversion Structures and CSO

Potential combined sewer overflows in the wastewater collection system are regulated by two diversion structures on Water Street, one located at the intersection of Spring Street (Spring Street Diversion Structure) and one located near the Main Pumping Station (Water Street Diversion Structure). During certain wet weather events, the diversion structures allow combined sewage to discharge over a weir to a CSO piping system that discharges to the permitted CSO at Clemson Pond. The Water Street Diversion Structure (MH 934) has an 8 ft weir (elevation = 5.4' NGVD 29) and the Spring Street Diversion Structure (MH 897/898) has two weirs, one 3.5 ft and one 3.7 ft in length (elevation = 5.8' NGVD 29). Flow Assessment Services, under subcontract to UEI, performed an inspection of the CSO structures and observed that the overflow from the smaller weir in MH 897 will eventually hydraulically interrupt the free discharge of the MH 898 weir. Flows over both diversion structure weirs were historically monitored by ultrasonic level sensors and flowrate was historically recorded on circular chart recorders at the Main Pumping Station and transmitted to the SCADA system. (However this monitoring system was upgraded in 2010.) The overflows pass under the Squamscott River to Clemson Pond via two (2) 36-inch diameter inverted siphons. See Appendix A-18 for inspection reports of the diversion structures.



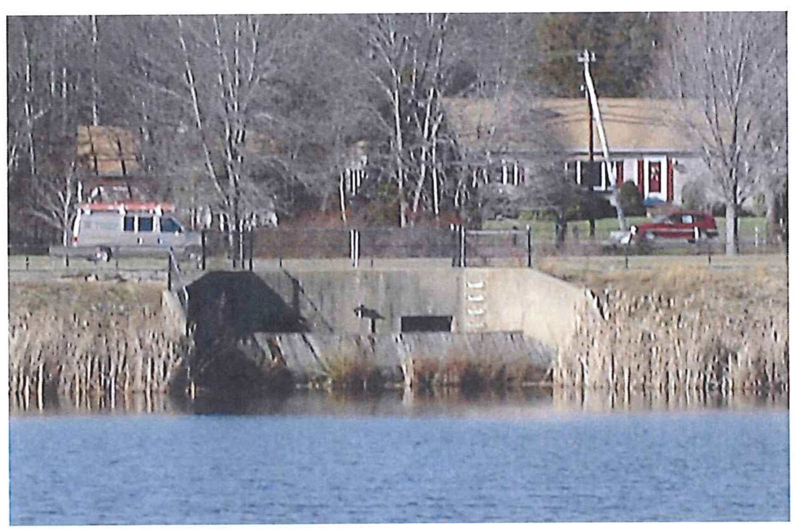
In 1966 Clemson Pond was constructed for treatment of industrial wastewater and CSO from the municipal wastewater collection system. Due to odor problems at Clemson Pond and depressed dissolved oxygen levels in the Squamscott River, the Clemson Automotive Fabric Company constructed a treatment facility (Clemson WWTF) in 1974 to treat textile wastes prior to discharge to Clemson Pond. Due to a change in ownership/use of the mills, the Town acquired ownership of the Clemson WWTF in the 1980s and abandoned/razed the WWTF. However, Clemson Pond remains in use for CSO discharge and storage.

It should also be noted that a localized storm drainage system in the vicinity of the Water Street Diversion Structure and in the vicinity of Clemson Pond also discharge directly to Clemson Pond. In the vicinity of the Water Street diversion structure, an 18" RCP drain pipe that conveys stormwater north along Water Street from the Green/Dewey Street area enters the CSO overflow side of the diversion structure to Clemson Pond confirmed through dye testing in July 2012. However, this 18" pipe was rerouted to the drainage system in October 2012. The Town GIS does not show any drainage infrastructure to discharge to Clemson Pond.

Clemson Pond has an estimated surface area of approximately 7.3 acres and about 2.4 million gallons per foot of storage. Based on an assumed low Clemson Pond stage of 1.2' (el.) Clemson Pond has the following maximum storage capacity:

- To Emergency Openings @ 4.6'(el) = 8.2 million gallons
- To Low Area on Embankment @ 8.6'(el) =17.8 million gallons

Clemson Pond has an outlet control structure, Outfall 002, which regulates pond depth and the discharge of water to the Squamscott River, a tidal waterway that is part of the Great Bay Estuary.



The Clemson Pond outlet structure is constructed of cast-in-place concrete and consists of two sets of 4'x4' dual-tide gates in series (invert elevation = -0.8' NGVD 29). In addition, there are two non-tide gated 1.5'x4' emergency openings (invert elevation = 4.6' NGVD 29). The water level in Clemson Pond is controlled by steel plates that serve as sharp crested weirs that maintains a pond elevation of approximately 1.2' NGVD 29 and reduces the effective primary pond outlet opening size to 2.2'x4'. Previous reports indicate the tidal Squamscott River in this area has a mean high water elevation (M.H.W.) of 4.3' and mean low water elevation (M.L.W.) of -3.0'.

The Clemson Pond outlet structure is configured so that Clemson Pond only discharges to the Squamscott River when the Clemson Pond water elevation is above that of the river and above that of steel plate weirs (elevation 1.2' NGVD 29). This means that CSO discharge from the Water Street and Spring Street CSO diversion structures (to Clemson Pond) does not necessarily correspond with a CSO event from Clemson Pond to the Squamscott River. In addition, the emergency (non-tide-gated) openings are protected by a concrete wall that does not allow back-flow of Squamscott River water into Clemson Pond until the Squamscott River stage rises to above 6.8' NGVD 29. However, when the Squamscott River rises above elevation 6.8', Squamscott River floodwaters can surcharge into the sewage collection system by flowing backwards over the diversion structure weirs. This "back-flowing" of Squamscott River water into Clemson Pond appears to have occurred in February 2010.

3.3.1 February 2010 Squamscott River Flooding

On February 26, 2010 Town personnel reported Squamscott River flooding in Swasey Parkway. At this time there was significant regional flooding as a result of over 5 inches of rain which occurred over several days throughout the region. Based on observations of Town personnel during February 2010 flooding, and high water evidence in Swasey Parkway observed by UEI, it appears that Squamscott River floodwaters rose to an elevation of approximately 7.5 to 8 feet (NGVD 29). This River flood stage would allow floodwaters to enter Clemson Pond over the outer tide gate concrete wall (top of wall elevation = 6.8 feet) and through the non-tide-gated emergency openings. Once Squamscott River floodwaters entered Clemson Pond, flooding could potentially enter the wastewater collection system by "back-flowing" out of Clemson Pond through the CSO siphons and backwards over the CSO weirs in the Spring Street and Water Street diversion structures. This may impact reported CSO flowrates and volumes since the weir plates are at elevation 5.4' (Water St) and 5.8' (Spring St.). No direct observations of this "backflowing" were made. However, the Town installed an ultrasonic transducer on the downstream side of the Water St. CSO weir in December 2010 to monitor tailwater effects and "backflow" events.

Based on approximate hydraulic calculations, it is estimated that 10 million gallons of Squamscott River water may have back-flowed into Clemson Pond through the emergency openings over a 1 to 2-hour period during the February 26, 2010 flood event. This would correspond to an approximate 2 to 4 foot rise in the stage of Clemson Pond and the tailwater effects of this "backflow" would approach the level of the CSO diversion weirs in the CSO piping. It is not believed that this "backflowing" is a regular occurrence since it has never been directly observed, and, only appears to occur when unusually high tides occur during large rainfall/river flood events such as occurred in February 2010.

3.4 Main Pumping Station

The Main Pump Station is located between Water Street and Swazey Parkway. Influent flow to the station enters through two channels with manual sluice gates. One channel has a grinder and the other has an aluminum bar rack. The station contains three variable speed pumps which discharge to a 5,300 lf, 16-inch diameter cast iron force main, and subsequently to a 24-inch gravity sewer that conveys wastewater to the WWTF. The pump station was designed for a lead-lag-standby operation for the three pumps. According to the 1997 CDM Phase I Report, the station has a rated peak capacity of 5,500 gpm, or 7.9 MGD. However, Section 3.5.2 of the same report indicates that a 1996 upgrade increased the capacity of the station from 3 mgd to 5 mgd. Prior to February 2010 there were no flowmeters installed on the pump discharge to measure flow and flow was calculated based on pump run-time.

In February 2010, three ultrasonic, Doppler-type flowmeters were installed at the station, one on each pump discharge. The maximum flowrate reported to date is about 7.1 MGD. Flow data recorded during the spring 2010 is illustrated on Figure 3-1. At high flows, the level in the wetwell rises and eventually the CSO diversion structures at Water Street and Spring Street are activated. In December 2010 new metering was added to the diversion structures to more accurately monitor CSO flow and the data is available 24 hours a day through an web-based service. Prior to the 2010 upgrade, the water level over the diversion structure weirs is measured by an ultrasonic level sensor, and the resulting flowrate was recorded on a circular chart recorder.

3.5 Existing and Historical Wastewater Flows and Loads

WWTF daily influent flow graphs with daily precipitation for the past 3 years (2007-2009) are included (Appendix A-21). Table 3-1 summarizes the annual average daily wastewater and CSO flows for the past 3 years.

Figure 3-1
Main Pump Station Flow
2010
Exeter, NH

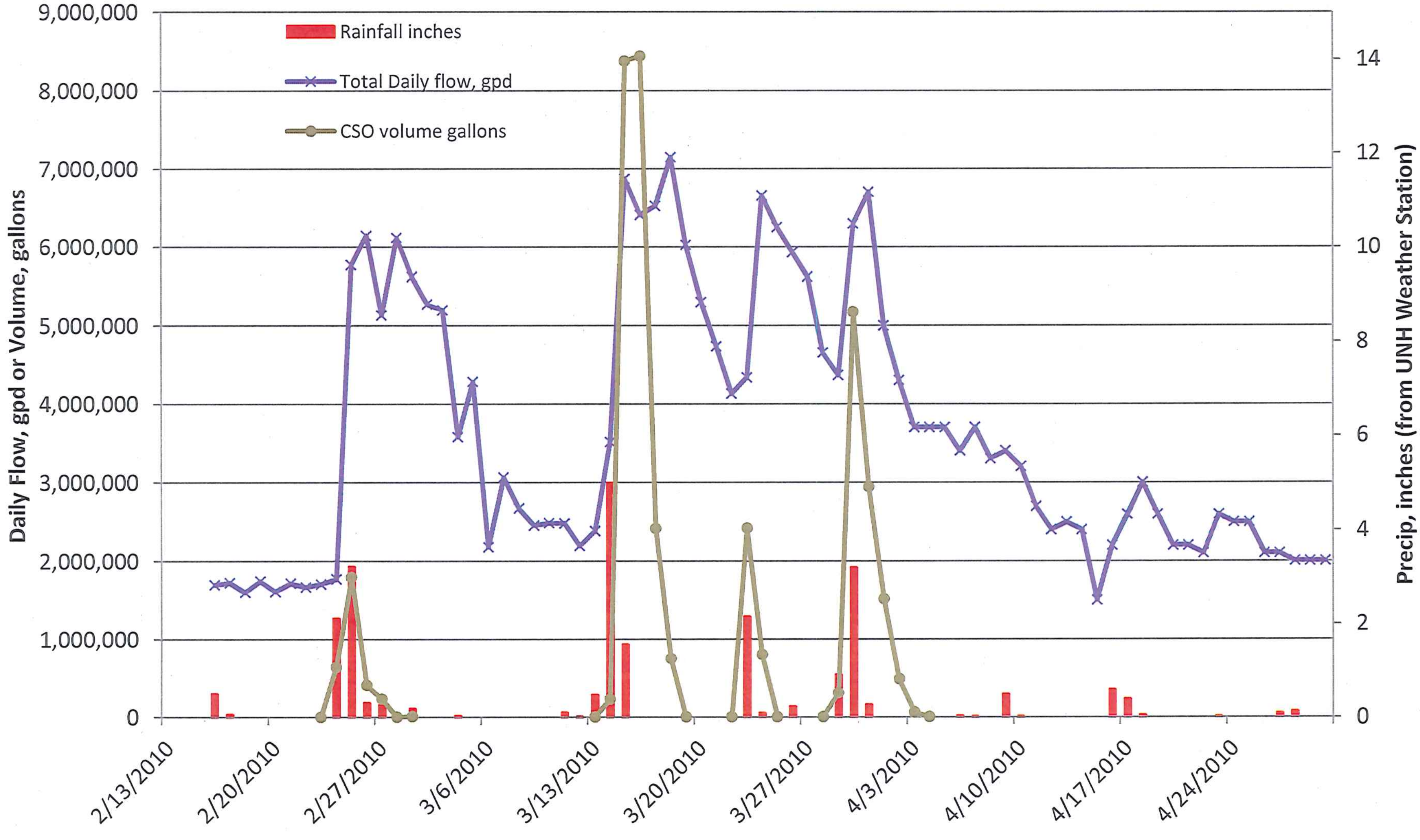


Table 3-1
Annual Average Daily Wastewater and CSO Flows
2007-2011

Year	2007	2008	2009	2010	2011	Average
Annual Average Sanitary Flow (Metered Water)	1.2	1.1	0.9	TBD	TBD	1.1
Annual Average WTP Gallons Water Treated and Pumped to Distribution System, mgd	1.2	1.1	1.1	1.1	0.9	1.1
Annual Average Wasted WTP to Sewer, mgd	0.4	0.3	0.3	0.3	0.3	0.3
Subtotal WTP Treated and Wasted to Sewer, mgd	1.6	1.4	1.5	1.4	1.2	1.4
Annual Average WWTF Influent, mgd	1.9	2.3	2.1	2.1	1.9	2.1
Annual Average CSO Flow, mgd	0.05	0.003	0.0001	0.05	0.009	0.02
Subtotal Annual Sewer and CSO Flow, mgd	1.95	2.30	2.10	2.15	1.91	2.08
Annual Average Extraneous Flow Treated (I/I), mgd	**0.4**	**0.9**	**0.6**	**0.7**	**0.7**	**0.7**
% I/I	**20%**	**38%**	**29%**	**33%**	**37%**	**32%**

Notes:
1. All values are in units of million gallons per day
2. WTP Water Treated and Pumped data from internal Town memos increased by 8% due to historical meter error reported by the Town
3. WTP wasted water to sewer (backwash) assumed based on 30% of WTP Water Treated and Pumped based on 2009 ratio provided by Town
4. Average annual daily WWTF influent based on values reported in the WWTF monthly reports
5. Average annual daily CSO flow based on total CSO volume indicated in CSO summary table provided by the Town

It is estimated that a total annual average daily flow of approximately 1.4 mgd of sanitary flow (WTP finished water and WTP backflow) entered the sewer collection system and 2.1 mgd annual average wastewater (WWTF influent and CSO overflow) were observed. From 2007-2009 approximately 0.7 mgd, or approximately 30% of the WWTF influent was extraneous flow (I/I). The average annual flows indicated in the table show the relative annual amounts of extraneous flow in the wastewater collection system. However, flows in the wastewater collection system have been reported to spike by an order of magnitude during large rainfall events as described in later sections of this report.

At the time of the 1997 CDM studies, the Spring Street CSO was reported to overflow when a rain storm of less than 0.3 inches occurred, and the Water street CSO reportedly overflowed during storms greater than 1-inch. At the time, there was also an active CSO at Center Street that occurred only during severe storms. The Center Street CSO has since been removed from the system due to separation work completed by the Town.

The Town provided EPA with wet weather and dry weather SSO discharge information for 2003-2008 and CSO discharge information for 2006-2007 in their 2008, Section 308 EPA response as required (Appendix A-12). The Town identified SSO events as uncontrolled sewer overflows that occurred at locations other than at the Water Street and Spring Street diversion structures during dry weather and/or wet weather. Dry weather events were typically caused by mechanical failures (pumps), pump control system failures, blockages in sewers, or failures on private property that were corrected immediately by the Town. Wet weather SSOs are summarized in Table 3-2. CSO events were controlled sewer overflows that occurred at the Water Street and Spring Street Diversion Structures and discharged to Clemson Pond during rain events. The Town has not reported any discharges to Clemson Pond during dry-weather.

Based on the limited flow data available from the main pump station, average daily flow is approximately 2 MGD. This flow includes base-line I/I and sanitary flow. 'Wet weather' flows have been reported as high as 25-30 MGD, based on 6-7 MGD pumped by the main pump station and peak CSO rates of up to 10 MGD from each CSO diversion structure, as observed during significant rain storms in March 2010. However, at this time it is uncertain if these 10 mgd flows measured at the diversion structures are 'real'. It appears that if the water level in Clemson Pond reaches a certain elevation, the system may hydraulically back-up into the diversion structures and could cause an inaccurately high flow reading.

As previously mentioned, the Town installed additional and improved monitoring at the WWTF influent, Main Pumping Station, and Water St. and Spring St. Diversion Structures in 2010 and 2011 to better monitor flows in the system and to evaluate CSO "backflow" tailwater effects. However, there has only been three CSO events since the new metering was installed and no flooding to levels that could cause a "backflow" into the system.

Table 3-2
Wet Weather Overflows

Non CSO's Locations

Appendix 11A of 308 Response

April 1, 2004 - April 16, 2007 (25 Overflows)

Date	Location	Est. Rate	Time	Est. Flow	Cause
4/1/2004-4/3/2004	Bottom of Jady Hill smh 299 to Clemson Pd	50 gpm	30 hrs	90,000 gal.	5" rain in 48 hrs
4/2/2005	Bottom of Jady Hill smh 299 to Clemson Pd	5-10 gpm	<2 hrs		2" rain & snowmelt + 1.75 " rain
5/27/2005	Bottom of Jady Hill smh 299 to Clemson Pd				
10/15/2005	Bottom of Jady Hill smh 299 to Clemson Pd			600 gal.	2 days of >3" rain & excessive WTP flows
5/13/2006-5/18/2006	Bottom of Jady Hill (Duck Pt) to Squamscott River	50-100 gpm	116 hrs		9" rain
5/14/2006-5/16/2006	Fox Chapel off of Folsom St. smh 230 to Exeter & Little Rivers	5-10 gpm	44 hrs		9" rain
5/14/2006-5/15/2006	Thornton St. smh 151 into Dearborn Resevoir	10-20 gpm	21.5 hrs		9" rain
5/14/2006-5/15/2006	Sleepy Hollow smh 152 into Dearborn Resevoir	10-20 gpm	21.5 hrs		9" rain
5/14/2006-5/17/2006	Marlboro St smh 343 into Exeter & Little Rivers	10-20 gpm	73 hrs		9" rain
5/14/2006-5/16/2006	Front St. smh 566 into Exeter & Little Rivers	5 gpm	44 hrs		9" rain
5/14/2006-5/17/2006	smh 1089 behind downtown stores into Squamscott River	5-10 gpm	73 hrs		9" rain
5/15/2006-5/16/2006	Main Line at daycare smh 901 into Squamscott River	5-10 gpm	22.5		9" rain
5/15/2006-5/16/2006	Linden St. smh 455 into Little River	20-30 gpm	18.5		9" rain
5/15/2006-5/16/2006	Linden St. smh 456 into Little River	20-30 gpm	19.5		9" rain
5/15/2006-5/16/2006	Linden St. smh 457 into Little River	20-30 gpm	20.5		9" rain
5/15/2006-5/16/2006	Linden St. smh 458 into Little River	20-30 gpm	21.5		9" rain
5/15/2006-5/16/2006	Linden St. smh 459 into Little River	20-30 gpm	22.5		9" rain
6/8/2006	Bottom of Jady Hill smh 299 into Clemson Pd	50 gpm	8 hrs	24,000 gal.	2.54" rain
7/11/2006	Bottom of Jady Hill smh 299 into Clemson Pd	150 gpm			5.28" rain
7/11/2006	Linden St smh 455 into Little River	15-20 gpm			5.28" rain
7/12/2006	Linden St smh 456 into Little River	15-20 gpm			5.28" rain

3.6 Historical CSO and SSO Events

Based on recorded influent flows at the WWTF, dry weather flows average approximately 2 MGD, and wet weather peaks may be as high as 25-30 MGD, although the data for these peaks is suspect. The following sections discuss historical CSO and SSO events

3.6.1 Historical CSO Volumes

The 2006 & 2007 CSO information was provided by the Town's Section 308 Response (Appendix A-12). The Town reports that CSO events have occurred every year since the Town began recording CSOs in 1997. A summary of CSO events from 1997 to 2010 is included (Appendix A-23).

The annual total CSO volumes have ranged from a minimum of 0.045 million gallons in 2009 to greater than 60 million gallons in 2006. It should be noted that the Town reported a malfunction of the CSO meter during the severe flooding that occurred in May 2006, when the majority of the annual CSO volume occurred, so accurate records/measurements for 2006 are not available.

Figure 3-2 shows that there is a limited connection between total annual rainfall and total annual CSO volume. For example, as shown, 2008 had higher annual precipitaiton than 2007, but there was a much larger annual CSO volume in 2007 than 2008. This apparent discrepancy was due to the historical flooding/storm event that occurred in the April 2007 during which over 99% of the 2007 CSO volume ocurred.

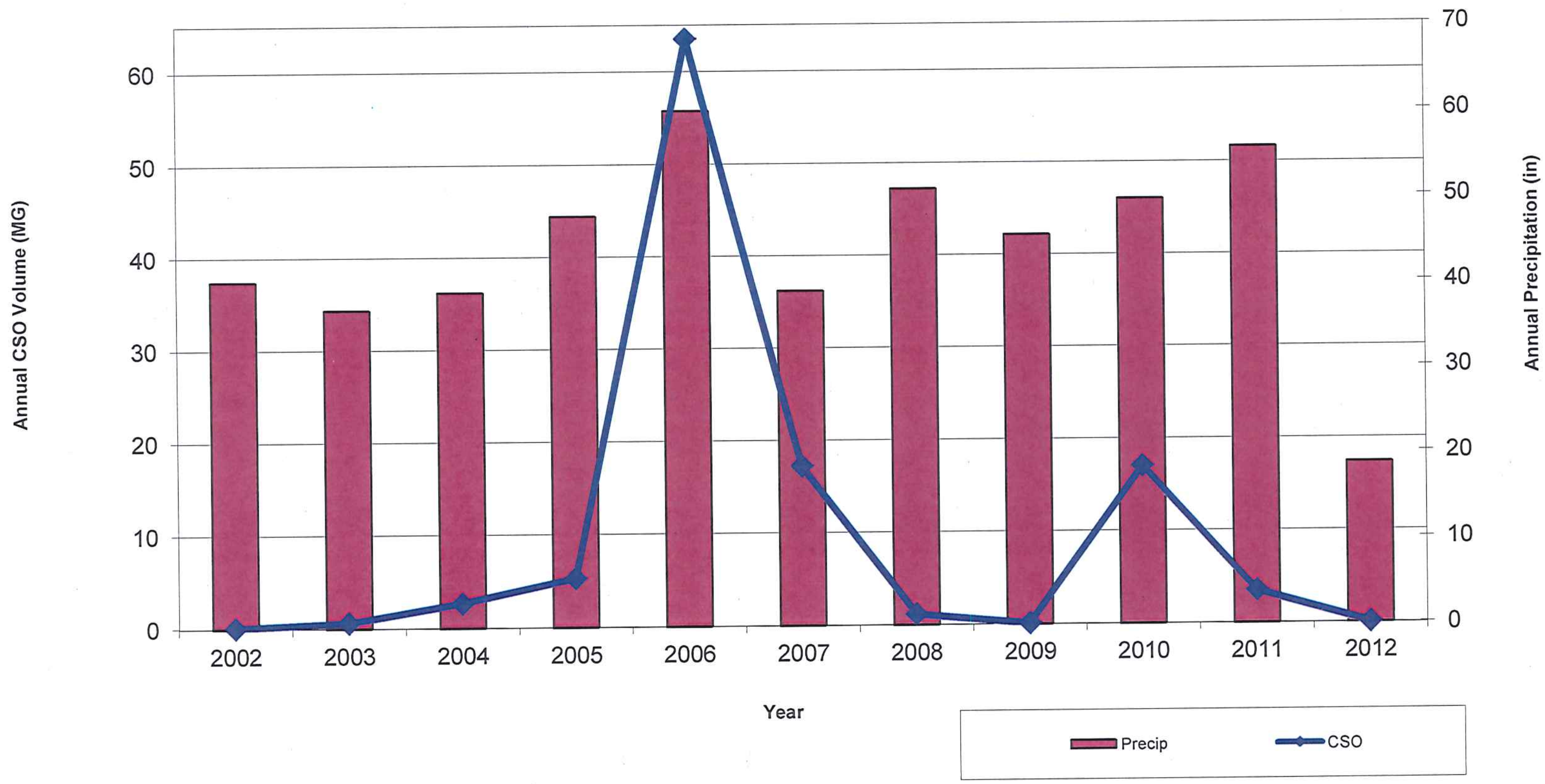
Table 3-3
Number of Annual Storm Events Causing CSO
2004-2012

Year	Total Precipitation (in.)	CSO Volume (MG)	Number of Storm Events Causing CSOs per Year
2004	39.0	2.6	3
2005	47.9	5.2	5
2006*	60.0	63.5*	8
2007	39.0	17.2	3
2008	50.8	1.1	6
2009	45.4	0.05	2
2010	49.6	17.0	5
2011	55.6	3.4	3
2012 (1/2 year)	18.7	0.0	0
Average	48	12	4

*Data is suspect

Notes:
1. 2004-2005 Total annual rainfall as reported by a weather station in Stratham, NH (strathamweather.com)
2. 2006-2010 Total annual rainfall as reported on daily WWTF operations reports
3. 2011-2012 Total annual rainfall as reported by Exeter weather station located on the Main Pumping Station.
4. Annual CSO volumes summed from a CSO summary spreadsheet provided by the Town
5. 2012 information is not complete and is through 7/24/2012

Figure 3-2
Total Annual CSO Volume 2002-2012



Notes:

1. 2002-2005 Total annual rainfall as reported by a weather station in Stratham, NH (strathamweather.com)
2. 2006-2010 Total annual rainfall as reported on daily WWTF operations reports
3. 2011-2012 Total annual rainfall as reported by Exeter weather station located on the Main Pumping Station.
4. Annual CSO volumes summed from a CSO summary spreadsheet provided by the Town
5. 2012 information is not complete and is through 7/24/2012
6. 2006 datais suspect due to meter malfunction

3.6.2 CSO Discharges and Rainfall

Because CSO events occur concurrently with and are caused by precipitation events, CSO events were evaluated with respect to rainfall. Additional CSO event information for years 2004 to 2010 as it relates to rainfall is included (Appendix A-24). It should be noted that CSO events that were reported by the Town to be influenced by snowmelt were not included in this analysis, so annual total CSO volumes may not match those reported in Appendix A-23.

2004-2009 CSO event information for the Spring Street and Water Street diversion structures provided by the Town were plotted on rainfall storm hydrographs provided by a *Davis Vantage Pro* weather station in Stratham, NH (strathamweather.com). The purpose was to determine at which storm interval CSO events have occurred. An example is included below (Figure 3-3) and complete 2004-2009 graphs are provided (Appendix Volume VII).

Figure 3-3
Example - October 25, 2005 Rainfall Hydrograph with CSO

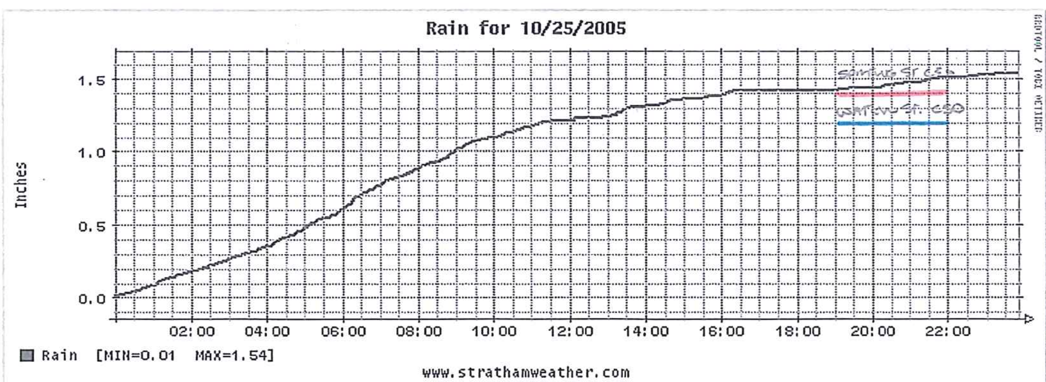
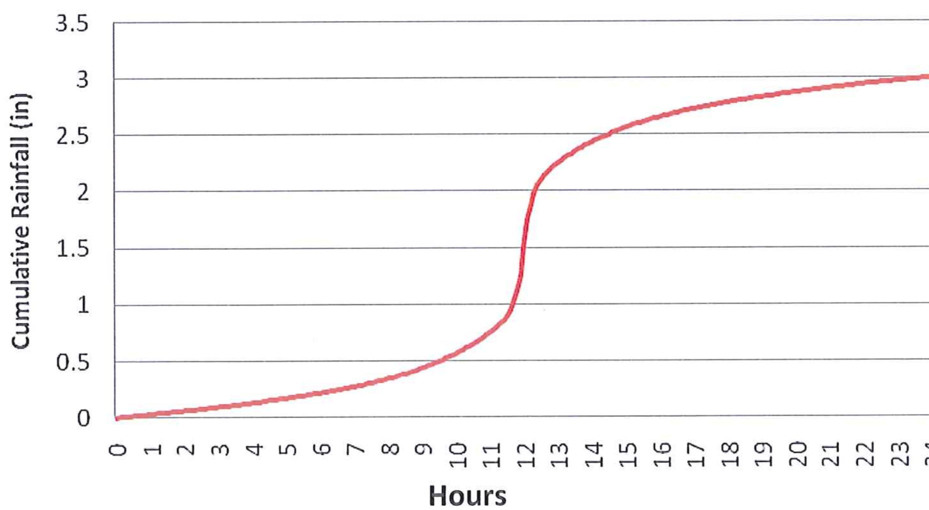


Figure 3-4
Typical SCS, Type-III, 24-Hour Unit Hydrograph (2-year recurrence interval)



The total storm rainfall as measured by the weather station during CSO events were compared to the Soil Conservation Service (SCS) Type III, 24-hour rainfall unit hydrograph as described in TR-55. It should be noted that actual rainfall hydrographs rarely occur over 24-hours or reflect the SCS Unit Hydrograph rainfall distribution shown (Figure 3-4). However, based on the guidance information provided in TR-55 the following recurrence intervals for 24-hour rainfall totals for the Town were used in the analysis:

- *2-Year Recurrence Interval = 3"*
- *5-Year Recurrence Interval = 3.8" (CDM Modeling = 3.71", 20-hour)*
- *10-Year Recurrence Interval = 4.5"*
- *25-Year Recurrence Interval = 5.2"*
- *50-Year Recurrence Interval = 5.7"*
- *100-Year Recurrence Interval = 6.5"*

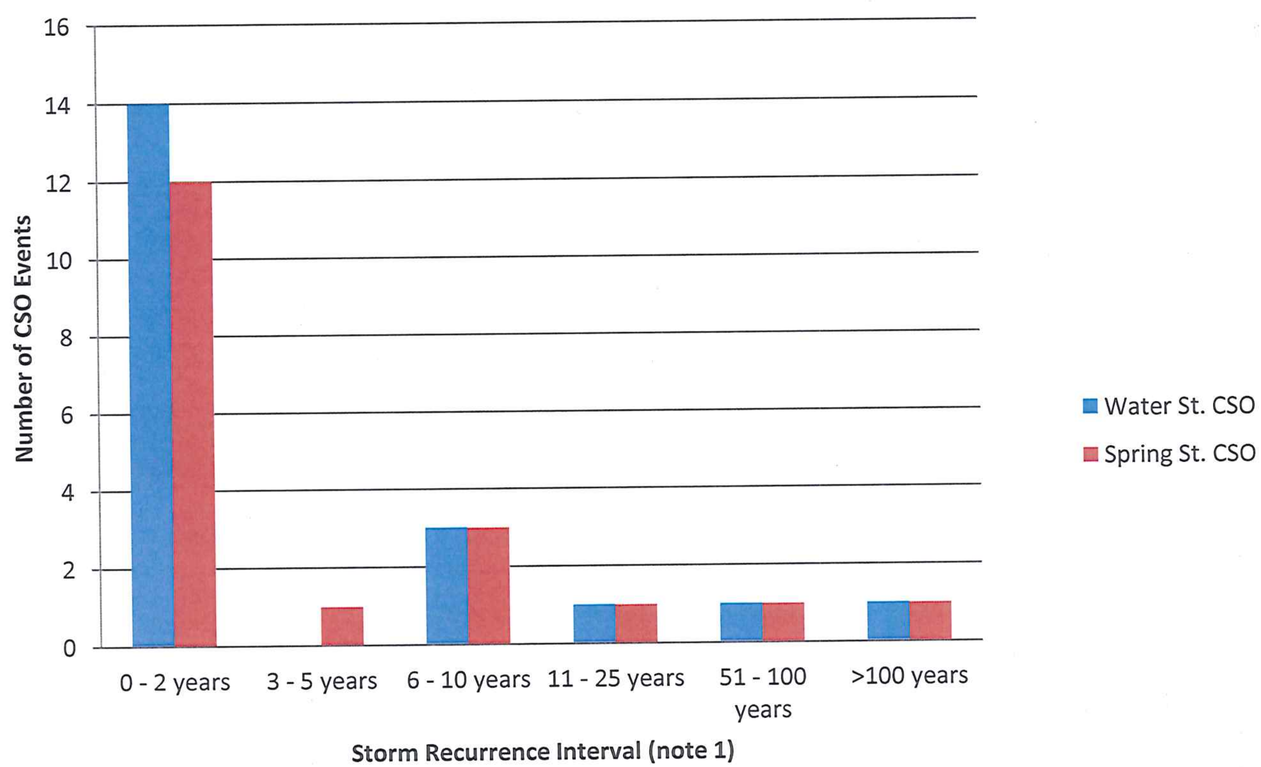
The above storm recurrence intervals used in the analysis is based on the total rainfall for a storm event that resulted in a CSO in 2004-2009 compared to SCS, 24-hour rainfall totals above to approximate an artificial recurrence interval. The actual rainfall for smaller storms generally occurred over a period of time less than 24-hours or and larger storms greater than 24-hours which may under-estimate the recurrence interval for smaller storms and over-estimate the recurrence interval for large storms. In addition, CDM's EXTRAN modeling used a hydrograph for a 20-hour, 3.71" total precipitation to simulate the 5-year storm.

Figure 3-5 shows that the largest number of 2004-2009 CSO events from the Water Street and Spring Street diversion structure occurred as a result of a rainfall total recurrence interval of less than 2-years. This observation is likely due to that rainfall events with a recurrence frequency of less than 2 years are more common than rainfall events with higher recurrence intervals.

However, Figure 3-6 shows that the cumulative total CSO volume for rainfall totals with recurrence intervals of less than 2 years, although more frequent, accounts for only a small fraction of the total CSO volume for 2004-2009. It should be noted that the one rainfall event with recurrence interval >100 years corresponds to the May 2006 storm event where it rained approximately 9.5 inches over 7 days, causing widespread flooding, and resulted in a large percentage of the CSO volume from 2004-2009. In addition, only rainfall events that resulted in a CSO were included in this analysis and there were many rainfall events that did not result in a CSO.

It should not be inferred that rainfall alone with total amounts less than a 2-year recurrence interval will always result in a CSO event. Table 3-3 shows that an average of 75% of the annual rainfall volume from 2004-2009 did not occur concurrently with a CSO event. In addition this analysis does not consider the influence of delayed inflow (sump pumps) and high infiltration. Many CSO events from 2004-2009 occurred in the spring during periods of high infiltration and delayed inflow and it is generally not rainfall alone, but a combination of these extraneous sewer flow sources that result in a CSO event.

Figure 3-5
Number of CSO events and Storm Recurrence Interval of Storm Contributing to CSO
2004-2009

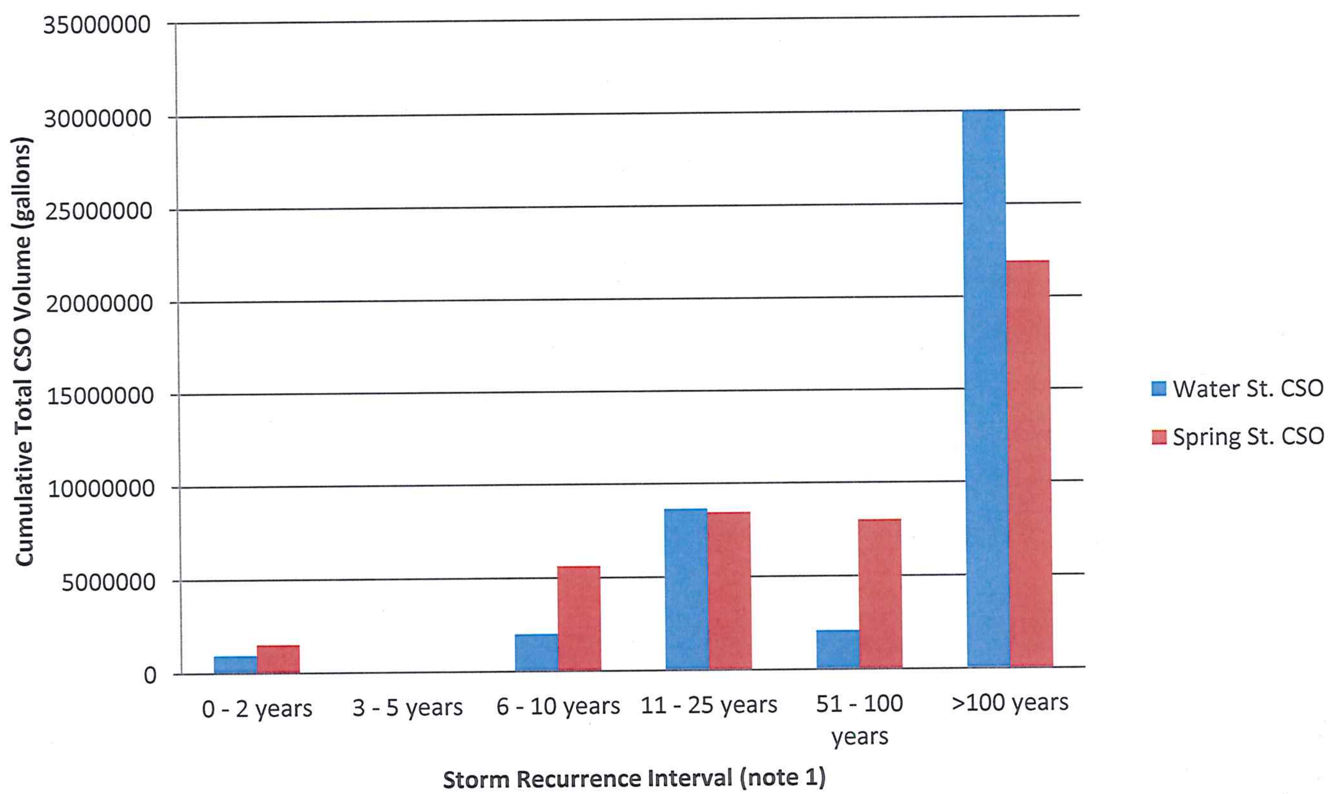


Notes:
1. Storm total rainfall was compared to 24-hour SCS recurrance interval rainfall totals. Since storms were rarely 24 hours in duration or followed the SCS rainfall intensity curves, it is only a rough approximation of recurrance frequency.

Figure 3-6

Total CSO Volume and Storm Recurrence Interval Contributing to CSO Event

2004-2009



Notes:

1. Storm total rainfall was compared to 24-hour SCS recurrance interval rainfall totals. Since storms were rarely 24 hours in duration or followed the SCS rainfall intensity curves, it is only a rough approximation of recurrance frequency.

Figure 3-7 shows that generally total storm rainfall less than 3.71" (assumed 5-year storm) peak total CSO flows are less than 10 MGD. Assuming that the main pumping station was pumping approximately 7 MGD at that time, a total of 17 MGD Main pumping station pumping capacity would be needed to convey the total flow to the WWTF to avoid CSOs. However, the reported peak CSO rates for total rainfall in excess of 3.71" results in much higher CSO flows. It should be noted that the CSO chart records do not record flows above 10 MGD for each diversion structure, so total CSO flows may be in excess of the 20 MGD indicated in Figure 3-7. It is estimated that a 30+ MGD pumping capacity would be required to convey total flows in beyond the 5-year recurrance interval level of control.

3.6.3 CSO Water Quality

It is our understanding that CSO monitoring is not required by the Town's NPDES permit, and as such, no water quality data is available at this time.

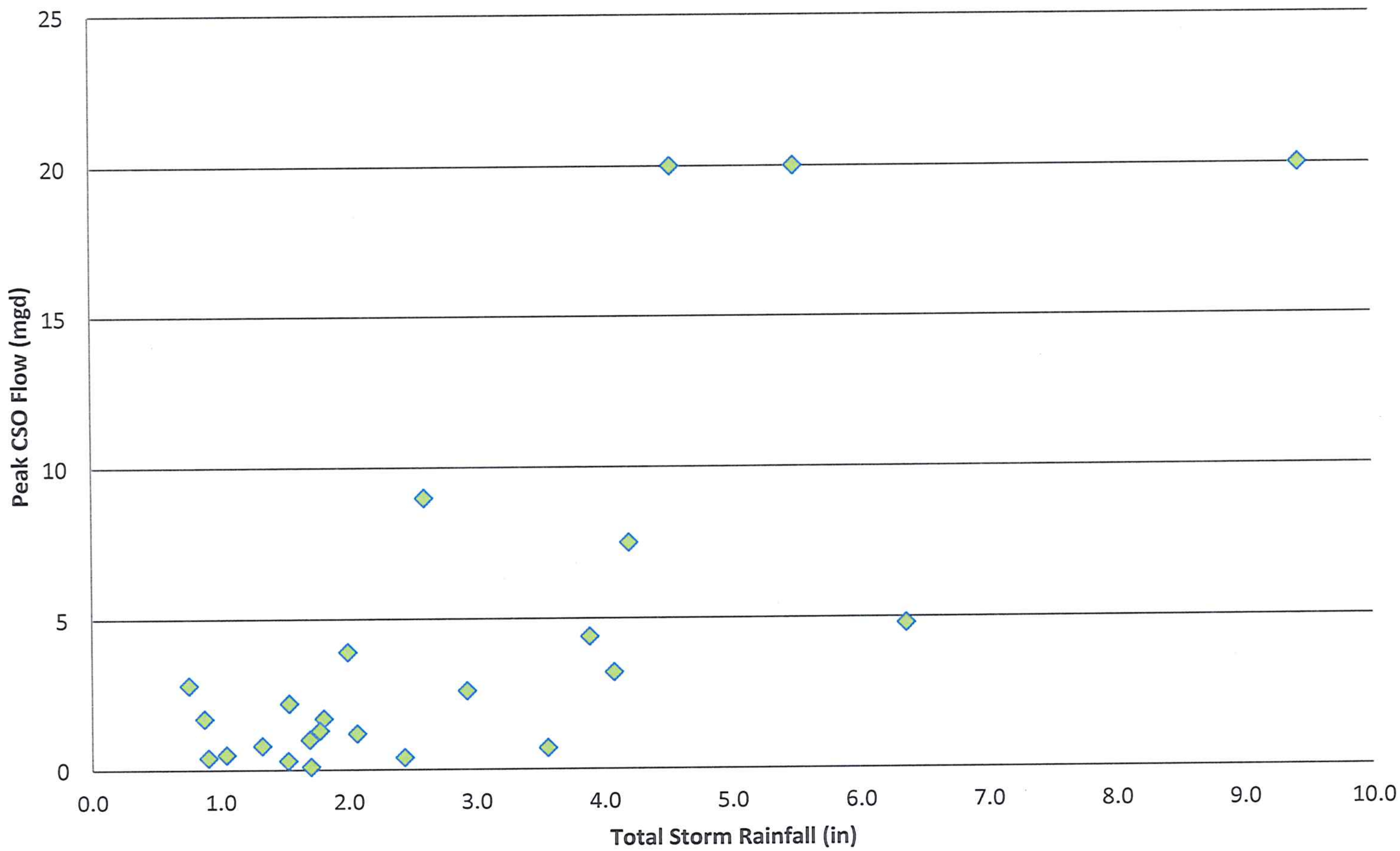
3.6.4 Dry Weather SSO Discharges

A total of eleven (11) dry weather SSO events were reported by the Town from 2003-2007. As shown in the table included in Appendix A-12, six (6) of these discharges were associated with private sewers and five (5) were associated with public sewers. As reported by the Town, dry weather SSO events were generally limited to less than 200 gallons and were associated with pump failures and blocked sewer lines. However, the WWTF outfall dry weather SSO event on July 15, 2003 was reported to discharge 234,390 gallons to the Squamscott River. This SSO event lead to improvement to the SCADA system to prevent future similar events.

3.6.5 Wet Weather SSO Discharges

A total of twenty five (25) Wet Weather SSO events were reported from 2003-2007. Although 25 SSOs were reported, they occurred on only eight (8) discrete occasions. Seventeen (17) of these events occurred simultaneously during the May 2006 and April 2007 storms that caused widespread flooding throughout the State. Most of the other Wet Weather SSO events occurred at a location locally known as "Duck Point" in the vicinity of Jady Hill, an area with known infiltration issues, and, coincidentally, the overflow from Jady Hill SSO events discharged to Clemson Pond, the receiving water for the Town's permitted CSOs. Discharges from "Duck Point" to Clemson Pond during very heavy rain events are a recurring problem, as an overflow in this area occurred during each of the eight (8) overflow occasions. However, the Town has taken several steps in the past several years to mitigate SSO discharges at "Duck Point" including: constructing a multi-million dollar sewer rehabilitation project around Jady Hill aimed to reduce public and private I/I in this area, modified Water Treatment Plant backflushing operations to reduce peak discharges to the sewer, and routinely cleans the inverted sewer siphons immediately downstream of "Duck Point". These improvements will reduce the likelihood of future SSOs from this area.

Figure 3-7
Combined Peak CSO Flow (Water & Spring St.) and Total Storm Rainfall
2004-2009



4. REGULATORY FRAMWORK AND WATER QUALITY STANDARDS

4.1 Regulatory Framework Background

The Town of Exeter is required to meet water quality limits established by State and Federal water quality legislation for all wastewater discharges, including combined sewer overflows (CSOs). In accordance with Section 402 of the Clean Water Act (as amended in 1977), and the national CSO control policy (as expanded in 1994), effluent requirements for CSOs are enforced through the National Pollutant Discharge Elimination System (NPDES) permitting program.

The Clean Water Act requires that states develop water quality standards for water bodies in the state that include:

- Define the "designated uses" for each water body
- Water quality "criteria" that specify the amounts of pollutants that may be present without impairing the designated uses for each water body
- Antidegradation provisions to protect high quality water bodies and maintain water quality necessary to protect existing uses

4.2 Water Quality Standards – Squamscott River

New Hampshire surface water quality regulations (Env-Wq 1700 – Appendix A-4) and State Statute (RSA 485-A:8) include standards for Class "A" and Class "B" receiving waters. The Squamscott River, the receiving water for WWTF effluent and the Clemson Pond outlet, is classified as a Class B tidal water.

The State has indicated that bacterial limits for the protection of shellfish applies to the Town of Exeter because the Squamscott River is a Class B tidal water. Table 4-1 summarizes the applicable water quality standards for the Town of Exeter.

Table 4-1
Water quality standards applicable to Town of Exeter

Criteria	**Class B Waters (Squamscott River)**
Uses	Fishing, swimming, water supply after adequate treatment. Potential shellfish growing waters.
Dissolved oxygen % saturation and mg/L	>75% saturation; > 5 mg/l
Total coliforms # colonies / 100 mL	< 70 Shellfish growing waters
Fecal coliforms # colonies / 100 mL	<14 Shellfish growing waters
E. coli # colonies / 100 mL	<126 geo. mean (>3 samples over 60 days) <406 maximum any sample

NHDES (1999). NH Code of Administrative Rules Env-Wq 1700, Surface Water Quality Regulations.

In April 2010 the Squamscott River in Stratham, NH, located downstream of the WWTF outfall and Clemson Pond outlet, was included on NHDES *Final Submitted to EPA – 2010 List of Threatened or Impaired Waters that Require a TMDL* (Section 303(d) List, Appendix) for several impairments. As shown on the 303(d) list for the Squamscott River, generally, multiple pollutants are listed to contribute to impairing each designated use of the Squamscott River. However, only the pollutant of Enterococcus, listed to impair Primary Contact Recreation and Secondary Contact, is indicated as a high TMDL priority with a TMDL schedule date of 2010. *Combined Sewer Overflows* and *Wet Weather Discharges* were listed as a source of Enterococcus contamination.

Additional impairments are listed with unknown sources of impairment. Two of concern to the Town include nitrogen and dissolved oxygen. Both are listed as low priority for development of a TMDL, with scheduled TMDL dates of 2021 and 2019, respectively.

Recent studies by the NHDES entitled *Preliminary Watershed Nitrogen Loading Thresholds for Watersheds Draining to the Great Bay Estuary, October 2009,* and *Numeric Nutrient Criteria for the Great Bay Estuary, June 2009* evaluate the water quality of various tributaries to the Great Bay, including the Squamscott River. For nearly every parameter evaluated, the Squamscott River is the worst contributor to Great Bay. In these reports, the State proposes an in-stream water quality standard for total nitrogen of 0.25-0.3 mg/l for protection of eelgrass, and a standard of 0.45 mg/l for dissolved oxygen. Existing nitrogen levels in the river (0.748 mg/l) exceed these proposed water quality standards.

4.3 Water Quality Standards (WQS) – Clemson Pond

Ken Edwardson of NHDES indicated that Clemson Pond (NHIMP-60030806-08) is a Class B fresh water pond and will be listed on EPA's 305(b), Category 4(b) 2012 Surface Water report card in the future as impaired for E. coli primary contact. He indicated that Clemson Pond is not listed on EPA's 303(d) list of impaired waters requiring a TMDL, because water quality issues for Clemson Pond will be addressed by EPA's administrative order regarding CSO discharges so therefore does not require a TMDL study. However, the Town's current and draft NPDES permits include a CSO discharge limitation of 1,000 colonies/100 ml for E. coli, it is likely that more stringent discharge limitations will be imposed in the future.

Jeff Andrews of NHDES indicated that Clemson Pond is a freshwater pond with no dilution so long term CSO discharges must meet NHDES's *Freshwater Acute Aquatic Life Criteria.*

4.4 EPA CSO Control Policy

EPA's 1994 CSO Control Policy (1994 CSO Policy) requires that the Town, as a NPDES permittee, implement the nine minimum controls for operation of the system and develop and implement a long-term CSO control plan that will ultimately result in compliance with the Clean Water Act. Based on the Town's May 30, 2008 response to EPA Section 308 request, portions of this report (UEI) are intended to address CSO Long Term Control Plan requirements. The following is a brief discussion of some of the elements of the 1994 CSO Control Policy of importance to the Town.

4.4.1 Small System Considerations

Section I(D) of EPAs 1994 CSO Policy allows for a reduction of the CSO Control Plan requirements, at the discretion of the NPDES regulatory authority, for communities with populations less than 75,000. This reduction could be applied given the population of the Town. However, even under this provision, the Town must still comply with the following sections of the 1994 CSO Policy

- Nine Minimum Controls (Section II.B)
- Public Participation (Section II.C.2)
- Sensitive Areas (Section II.C.3)

The draft 2007 NPDES permit issued by EPA, but subsequently withdrawn indicated that the Town prepared a report *documenting compliance with the nine minimum controls* in April 1997. The Town was also required to employ a public participation process that actively involves the affected public (rate payers, industrial users, downstream residents, persons who use and enjoy downstream waters, and any other interested persons) to select the long-term CSO controls. In addition, because the CSO discharges to the Squamscott River, a tidal river within the Great Bay Estuary, provisions of "sensitive areas" described below apply.

4.4.2 Sensitive Areas

Section II.C.3 of the 1994 CSO Policy identifies shellfish beds as a sensitive area, which would likely apply to the Squamscott River, the receiving water for both CSO and WWTF discharge. The 1994 CSO Policy requires that the CSO Long Term Control Plan discharging to sensitive areas address the following:

- Prohibit new or significantly increased overflows
- Eliminate or relocate overflows where physically possible and economically achievable
- Where elimination or relocation is not physically possible and economically achievable, provide treatment to meet WQS for existing and designated uses for the receiving water (Squamscott River) and the level of control should not be less than those described in the *Evaluation of Alternatives* (Section II.C.4) described below.
- If the CSO remains discharging to sensitive areas due to physical and economic prohibitions, NPDES will re-review CSO removal achievability as part of each permit renewal cycle.

Chris Nash of the NHDES Shellfish Program indicated that it is NHDES current policy to temporarily close Great Bay shellfishing in response to CSO discharges greater than 100,000 gallons or as a result of approximately 1.5" of rain. He indicated that the Squamscott River, tidal portions of the Lamprey River, to their confluence in Great Bay to Brackett Point are permanently closed to shellfishing due to the presence of WWTF outfalls, and this permanent closure is unlikely to change if CSOs were eliminated. He indicated that it is unlikely that elimination of the CSO would increase shellfishing opportunities due to bacteriological contamination from non-point sources and WWTF outfalls within the Squamscott River/Lamprey River/Great Bay watershed.

4.4.3 Evaluation of Alternatives

EPA requires that the Long-Term CSO Control plan evaluate a range of alternatives necessary to control CSO events from zero to twelve per year or capture 100% to 75% for treatment. The Long Term CSO Control Plan should adopt either the "Presumption" or "Demonstration" approach.

4.4.3.1 "Presumption" Approach

Section II(C)(4)(a) indicates that a Long Term CSO Control Program could be presumed to meet the WQS of the CWA if one of the following criteria is met. However, it should be noted that consideration of sensitive areas (i.e. shellfish beds) is required.

- Section II(C)(4)(a)(i) requires no more than four overflow events per year, or
- Section II(C)(4)(a)(ii) requires capture of no less than 85% by volume during precipitation events on a system-wide annual average, or
- Section II(C)(4)(a)(i) requires elimination or removal of the mass of pollutants and all flow shall receive a minimum of: primary clarification, solids and floatable disposal, and disinfection as necessary to meet WQS

4.4.3.2 "Demonstration" Approach

Section II(C)(4)(b) indicates that a Long Term CSO Control Program that does not meet the presumptive approach above, it must demonstrate all of the following:

- Section II(C)(4)(b)(i) requires that the Long Term CSO Plan is adequate to meet WQS and uses unless the receiving water is already impaired due to natural conditions or pollution other than that from CSOs.
- Section II(C)(4)(b)(ii) requires that the Long Term CSO Plan apportion and allocate pollution loads for waters not currently meeting WQS and uses due to other sources than CSOs.
- Section II(C)(4)(b)(iii) requires that the Long Term CSO provide the maximum pollution reduction that is reasonably attainable.

Section II(C)(4)(b)(iv) requires that the Long Term CSO allow for future improvements to CSO control as required.

4.4.3.3 Applicability of CSO Policy to Exeter

As shown in Table 3-3, from 2004-2012 Exeter has generally met the requirements of the presumptive approach where the Town has averaged approximately 4 CSO events per year and captures an average of approximately 98% flow (14 million average annual CSO gallons versus 730 million WWTF average flow or 2 MGD). It should also be noted that the 2004-2012 time frame includes the unusually high flows from 2006, 2007, and 2010 where widespread flooding occurred throughout the region, skewing the results.

Because consideration of sensitive areas (i.e. shellfish beds) is required for the presumptive approach, and NHDES has listed the Squamscott River as impaired, it is unclear whether the presumptive approach is applicable to the Town. If the presumptive approach is determined to not apply to the Town, then the Town's options would be to fulfill the requirements of the demonstration approach or eliminate the CSO.

4.4.3.4 LTCP and NMC

The CSO policy contains four fundamental principles to ensure that the CSO LTCP will be cost-effective and meet local environmental objectives, as follows:

1. Clear levels of control to meet health and environmental objectives.
2. Flexibility to consider the site-specific nature of CSOs and find the most cost-effective way to control them.
3. Phased implementation of CSO controls to accommodate a community's financial capability.
4. Review and revision, as appropriate, of water quality standards during the development of CSO control plans to reflect the site-specific wet weather impacts of CSOs.

The CSO Policy also required immediate implementation of minimum technology-based controls referred to as the Nine Minimum Controls (NMCs), to achieve some optimization of system capacity as quickly as possible. The NMCs were intended to provide cost effective and easily implemented measures which could be pursued in the short term to help reduce the volume, pollutant load and frequency of CSO events. Each permittee was required to submit, as soon as practical, but no later than January 1997, a report documenting their implementation of the NMCs. They are summarized as follows:

1. Proper operation and regular maintenance programs for the sewer system and CSO outfalls.
2. Maximum use of the collection system for storage.
3. Review and modification of pretreatment requirements to ensure that CSO impacts are minimized.
4. Maximization of flow to the POTW for treatment.
5. Prohibition of CSOs during dry weather.
6. Control of solid and floatable materials in CSOs
7. Pollution prevention.
8. Public notification to ensure that the public receives adequate notification of CSO occurrences and CSO impacts.
9. Monitoring to effectively characterize CSO impacts and the efficacy of CSO controls.

The Town of Exeter submitted its Nine Minimum Controls report in April 1997.

4.5 Current/Future Storm Water Regulations

Separated storm water systems are also regulated through the NPDES permit program under EPA's Phase I and Phase II Rules, and are applicable to municipalities with separated storm systems serving populations of 10,000 persons or greater. Phase I of the storm water program was promulgated in 1990 under the Clean Water Act, and applied to medium and large municipalities, generally serving populations of 100,000 or more. Phase II of the storm water program was promulgated in December 1999 to extend the program requirements to the smaller municipal separate storm systems. Although outside the scope of this report, Exeter has a separate stormwater system and must comply with the Phase II requirements.

EPA's Storm Water Phase II Rule (EPA, 1999) extended control of pollutant loads from storm water runoff to small municipal separate storm systems (MS4s) including the Town of Exeter. Storm water discharges from urbanized areas are of concern due to common pollutants including street litter, sediment, road salt, oils, and other debris. The storm water program relies on the use of narrative, rather than numeric effluent limitations, requiring the implementation of best management practices (BMPs) applied to the "maximum extent practicable".

Small MS4 systems, of which Exeter is one, must implement the following six (6) minimum control measures as part of their storm water management program:

1. Public education and outreach
2. Public participation and involvement
3. Illicit discharge detection and elimination
4. Construction site runoff control
5. Post construction runoff control
6. Pollution prevention / good housekeeping.

These measures must be fully implemented by the end of the first permit term, typically 5 years. State standards for storm water treatment vary considerably between the states. Vermont and Massachusetts require net annual suspended solids (TSS) removal of 80% from separated storm water discharges, others, including New Hampshire, do not require additional standards beyond the federal rules.

The Town of Exeter submitted a Stormwater Pollution Prevention Plan (SWPPP) and Notice of Intent detailing its selection of storm water BMPs and measurable goals for evaluation of their effectiveness in 2003.

4.6 Exeter NPDES Permit

4.6.1 NPDES Permit Summary

Appendix A-9 contains a copy of the draft NPDES permit that was issued in October 2007 and subsequently withdrawn. This permit may give some insight as to some of the provisions that the Town may be subjected to when their revised permit is issued. Because of the withdrawn draft permit of 2007, the July 5, 2000 permit remains in effect. The Town is awaiting a revised draft permit. The WWTF is treated and discharged to the Squamscott River (Outfall 001). The CSO diversion structures which discharge to Clemson Pond (Outfall 003) are permitted for wet weather only. Clemson Pond subsequently discharges to the Squamscott River.

Effluent discharges from both facilities are regulated under NPDES Permit No. NH0100871. The following is a brief discussion of the CSO portions of the NPDES permits since 1995:

4.6.1.1 <u>1995 NPDES Permit (Section 4(H) – Combined Sewer Overflows)</u>

- The NPDES permit erroneously indicated that the holding pond (Clemson Pond) was constructed for stormwater runoff. (A January 1985 New Hampshire Water Supply & Pollution Control Commission report indicates that Clemson Pond was constructed for wastewater treatment, see Section 3.3 of this report for additional discussion)

- The NPDES permit identified that the permitted CSO Outfall was 002, the tide gate discharge from Clemson Pond to the Squamscott River.

- Required documentation from the Town by January 1, 1997 demonstrating implementing the nine minimum controls and the degree to which the controls achieve compliance with water quality standards.

4.6.1.2 <u>2000 NPDES Permit (Section B – Combined Sewer Overflows)</u>

- The NPDES permit identified that the only permitted CSO Outfall was 003, the outlet of the two siphon pipes into Clemson Pond and Clemson Pond was identified as the receiving water. This is a change from the previous NPDES Permit that identified the Clemson Pond tide gates (CSO 002) as the only permitted CSO outfall.

- Required that CSO discharges not cause violations of Federal or State Water Quality Standards.

- Required monthly inspections of CSO structures
- Required quantification of CSO discharges including: CSO duration, volume, and precipitation data
- Required that a sign be installed in the vicinity of Outfall 003.
- Required CSO sampling for E. coli at CSO 003 setting a discharge limitation of 1,000 colonies per 100 ml.

4.6.1.3 2002 NPDES Permit Modification

- Section B(ii) required that CSO discharges shall not "cause or contribute" to violations of Federal or State Water Quality Standards. The previous NPDES permit only indicated that CSO discharges shall not "cause" violations.

4.6.1.4 2007 Draft NPDES Permit Modification

The 2007 draft permit was subsequently withdrawn, but may provide some insight as to provisions that may be included in the revised permit. Provisions that were included in the 2007 draft permit that pertain to CSO's (Section IV.g) were as follows:

- Required quarterly sampling at the outlet of Clemson Pond (outfall 002) and flow into Clemson Pond (outfall 003) quarterly for the first year for:
    - Fecal Coliform
    - E. Coli
    - Salinity
    - Temperature
- Wet-weather CSO discharges must not cause violations to Federal and State Water Quality Standards and are subject to both water-quality based and technology-based NPDES permit requirements.
- Town must document compliance with the nine minimum controls by updating the April 1997 document prepared by the Town. The document shall include information which indicates the degree to which the controls have achieved compliance with water quality standards.

4.6.1.5 2011 Draft NPDES Permit Modification

The 2011 draft permit was issued in March 2011 and requires a 3 mg/L total nitrogen discharge limit on a seasonal basis which the existing WWTF can not reliably meet. A significant WWTF upgrade would be required to meet discharge limits included in this draft permit.

4.6.1.6 2012 Groundwater Discharge Permit

The Town was issued a groundwater discharge permit for the WWTF on January 23, 2012. The permit requires periodic groundwater sampling and analysis for monitoring wells located down-gradient of the WWTF. Only one round of groundwater sampling has been performed to date and future sampling and analysis will be used to evaluate whether and/or the extent to which the WWTF lagoons are impacting down-gradient groundwater quality.

4.7 EPA Section 308

Section 308(a) of the Clean Water Act authorizes EPA to request information to evaluate whether any person is in violation of any effluent or pretreatment standard promulgated under the Clean Water Act. Under this jurisdiction, EPA issued a letter to the Town dated January 7, 2008 requesting the following information relative to the wastewater collection and treatment facilities:

I. Dry-Weather Overflows

- This included submission of a Dry-Weather Overflow Report, which included a chronological listing, description and details of all discharges from the Spring Street and Water Street diversion structures (permitted CSO locations) during dry weather. Also included was a listing, description and details of all discharges occurring at non-permitted discharge locations.

II. Wet-Weather Overflows

- This included submission of a Wet-Weather Overflow Report, which included a chronological listing, description and details of all discharges occurring during wet-weather at non-permitted discharge locations.

III. Collection System Operation and Maintenance

- This included submission of maps of the collection system identifying areas where the Town has experienced structural integrity, maintenance or capacity problems since January 1, 1998, copies of documents associated with enforcement actions initiated against the Town by the NHDES, and progress the Town has made in meeting milestones established in these enforcement actions, staffing levels, annual O&M budgets, reports assessing I/I or SSES, adequacy of the collection and/or treatment facilities,

IV. Building/Private Property Sewer Backup Incidents

- This included submission of all know building/private backup incidents caused by malfunctions in Town-owned treatment facilities, the cause and actions taken to mitigate future incidents.

V. Wastewater Treatment Facility

- This included submission of engineering evaluations and reports prepared for the Town, standard operational procedures for managing high flows at the WWTF, listing and description of bypasses and correspondence between the Town and NHDES.

VI. Long-Term CSO Control Plan for CSO Abatement

- This included a summary of CSOs occurring at both the Spring St and Water St diversion structures, inspection reports and WWTF influent flow data, and references this report for CSO mitigation alternatives.

VII. Water Treatment Plant

- This included a listing of unauthorized/non-permitted water treatment plant spills, releases or discharges of raw or partially treated wastewater and summaries of reports prepared addressing capacity or maintenance issues at the WTP.

The Town issued a response to EPA's Section 308 request on April 7, 2008 and May 30, 2008. To date, it is our understanding that EPA has not commented on the Town's response.

5. FIELD INVESTIGATIONS AND RESULTS

5.1 Overview and Approach

The sewer system evaluation study included field investigations to further identify sources of infiltration and private inflow. This effort was recommended in the 1997 CDM report and is intended to assist in the development of cost-effective strategies to reduce I/I. As a result of the CDM work, subsequent phases of work, including flow isolation, manhole inspection, house-to-house inspections and TV inspection were conducted as part of this phase of work. Field investigations included the following:

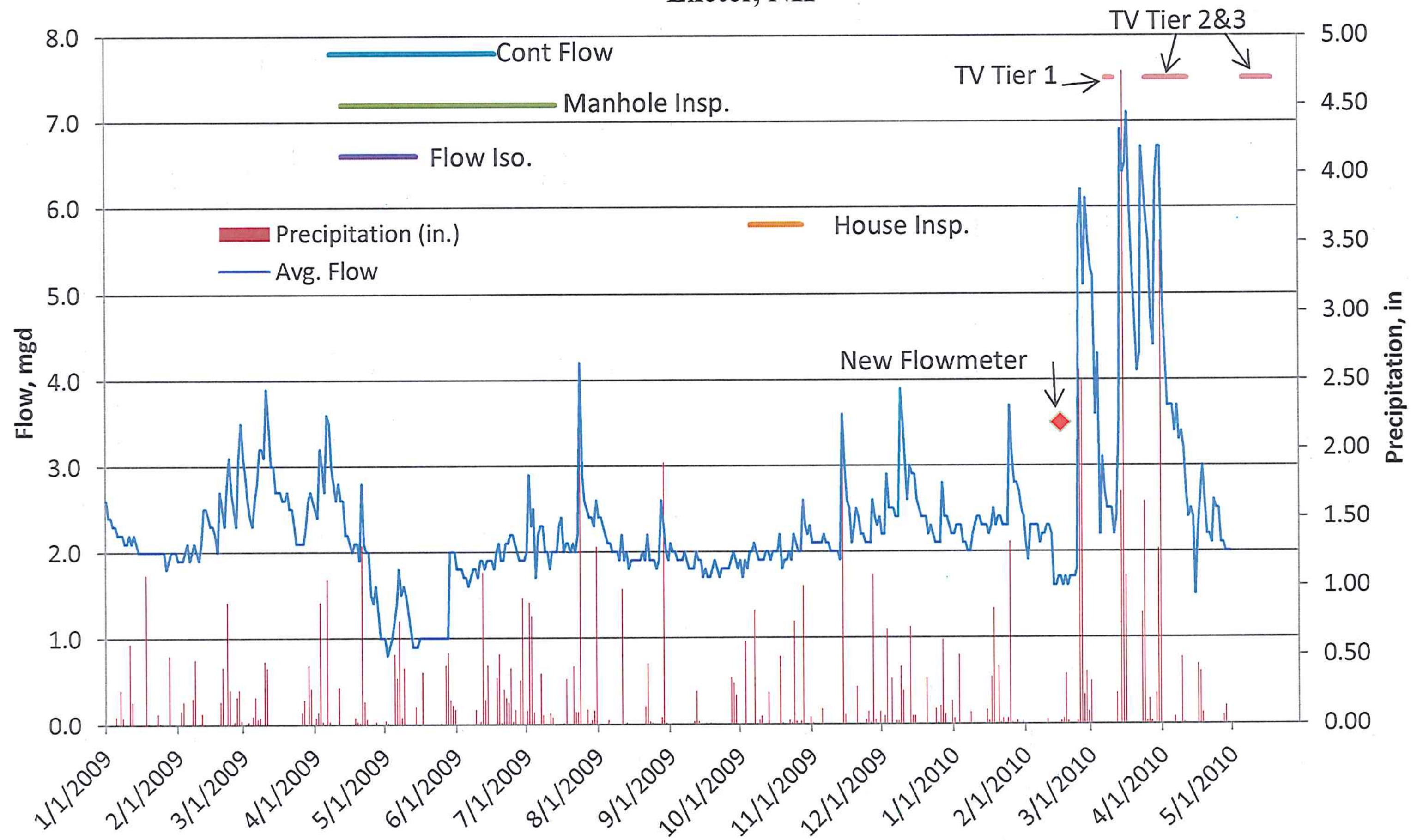
Table 5-1
Field Studies
2009-2010

Study Name	***Purpose***	***Date Completed***
Flow Isolation	Determine sewer reaches with excessive infiltration	April 2009
CSO Diversion Structure Investigation	Evaluate functioning of CSO diversion structures in the system	May 11, 2009
Continuous Flow Metering	Monitor changes in sewer flow due to rain events in select "pilot" areas	April – June 2009
Manhole Inspections	Identify manhole deficiencies and infiltration	April – June 2009
House-to-House Inspections	Identify houses with illicit connections in select "pilot" areas	October 2009
Customer Questionaires	Identify houses with illicit connections system-wide	September 2009
TV Inspection	Identify specific defects in sewers, quantify leakage at each defect, and evaluate possible repair stradegies	March 25- May 18, 2010
WWTF Drainage Evaluations	Evaluate drainage patterns at the WWTF and pathways for stormwater entering the WWTF	March 2010

Figure 5-1 shows the timing of the investigations and the flows and precipitation observed at the WWTF.

Generally, I/I investigation activities were targeted to be performed in the spring when groundwater levels and associated I/I is assumed to be high. However, based on the 2009 WWTF daily influent flow (Figure 5-1) and precipitation as reported based on monthly WWTF operations reports, the infiltration investigations may have only captured the "tail end" of the high spring groundwater and peak I/I conditions, as the high groundwater conditions occurred relatively early in 2009.

Figure 5-1
2009-2010 Average WWTF Influent flow, Daily Precipitation & Field Investigations
Exeter, NH



G:\REALNUM\1500's\1542 - Exeter - II Eng. Services\Report\Report Figures\Figure 5-1 Influent flow-field work

It should be noted that Town personnel indicated that WWTF influent flow data may not be accurate or reliable due to the configuration of the WWTF influent meter. Influent flow at the WWTF is measured using an area-velocity meter in the bottom of the pipe, but the water surface at this location is subject to wave action, as the pipe does not always experience free-flow characteristics. However, due to the long residence time of the WWTF (estimated by the Town to be 30-40 days), and the configuration of the WWTF effluent discharge (a manually operated valve that permits operators to control the discharge), the WWTF influent flow is the best indicator of relative system-wide flows available to show general relative seasonal fluctuations in I/I. There is currently no flow meter at the main pump station.

5.2 Continuous Flow Monitoring

Continuous flow monitoring is typically the first step in evaluating inflow and infiltration into a collection system. The collection system is divided into several sub-basins, typically of 20,000 linear feet of sewer or less, with key manholes identified where flow can be measured from the entire sub-basin during a wet-weather period, typically in the Spring. Rainfall data is also recorded during this time period at a location in close proximity to the collection system.

Although continuous flow monitoring of the collection system was performed in 1997 as part of the CDM study, three small pilot areas where I/I was suspected were identified for this study. Continuous flow monitoring was conducted from April 8, 2009 to June 18, 2009 in these pilot areas as part of this study to establish baseline flow data for each pilot area, and evaluate the effects of rainfall on flow in each area.

5.2.1 Pilot Areas

To provide information for comparison, and further investigate infiltration in smaller areas of sub-basins where high infiltration was suspected, three "pilot" areas were identified and additional continuous flow monitoring was conducted in these areas during the spring of 2009, concurrent with flow isolation and manhole inspections.

Continuous flow monitoring was conducted immediately downstream of three (3) pilot areas which were previously identified areas of suspected excessive I/I. Continuous flow monitoring pilot areas, meter location, and building count within each area were as follows (Figure 5-2):

1. West Side Drive Pilot Area - The West Side Drive continuous flow monitoring location (Site #1) was performed at SMH 535.
   House Count = Approximately 99
   Sewer Main Length = Approximately 5,500'
   Meter installed on 4/8/09 in 11.6-in PVC pipe

2. Downing Court Pilot Area - The Webster Street Pumping Station continuous flow monitoring location (Site #2) was performed at SMH 275.
   Building Count = Approximately 76
   Sewer Main Length = Approximately 6,500'
   Meter installed on 4/8/09 in 8-inch VCP

3. Jady Hill Pilot Area – Continuous flow monitoring was conducted on Haven Lane (Site #3) at SMH 301 (X-country/parking lot).
   House Count = Approximately 93
   Sewer Main Length = Approximately 5,900'
   Meter installed on 4/8/09 in 10-inch PVC.

The continuous flow monitoring for each pilot area was intended to serve as a check for the other I/I investigation measurements and to allow the Town to perform future flow monitoring to measure the effectiveness of future I/I removal programs.

Each pilot area is comprised of primarily detached, single family, residential homes.

5.2.2 Continuous Flow Monitoring - Results

Continuous flow monitoring was performed using area-velocity meters installed within the existing sewer pipe at each monitoring site noted above from April 8, 2009 through June 18, 2009. The area velocity meter at Site #1 (Westside Drive), was replaced after approximately 3 weeks (May 1, 2009) with a smaller Palmer Bowlus flume due to the decreasing flows observed at that location.

Rainfall data during the period was measured using a tipping bucket type rain gauge installed on the roof of the main pumping station. The flow meters recorded instantaneous flow in 15-minute intervals. Results of the continuous flow monitoring for each meter with rainfall data is included in Appendix A-13.

Water records were reviewed for each of the pilot areas. Water meters are read quarterly; however a "quarter" may range from 50 days to 132 days. Quarterly readings were therefore normalized to gallons per day for each "quarter". The average daily water use for the spring "quarter" of 2008 (1/31/08-4/30/08) and 2009 (3/12/09-5/27/09) were averaged to obtain average daily water usage for each pilot area. The average water use for the pilot areas ranged from 10-13,000 gpd, or 110 to 160 gpd/home. This usage is consistent with what UE has observed in other NH communities. For the purposes of this study, average sanitary use was assumed to be the same as average daily metered water use, as included in Table 5-2. In the Downing Ct and Jady Hill pilot areas, the minimum daily sewage flow recorded was typically much higher than the average daily water use (nearly double).

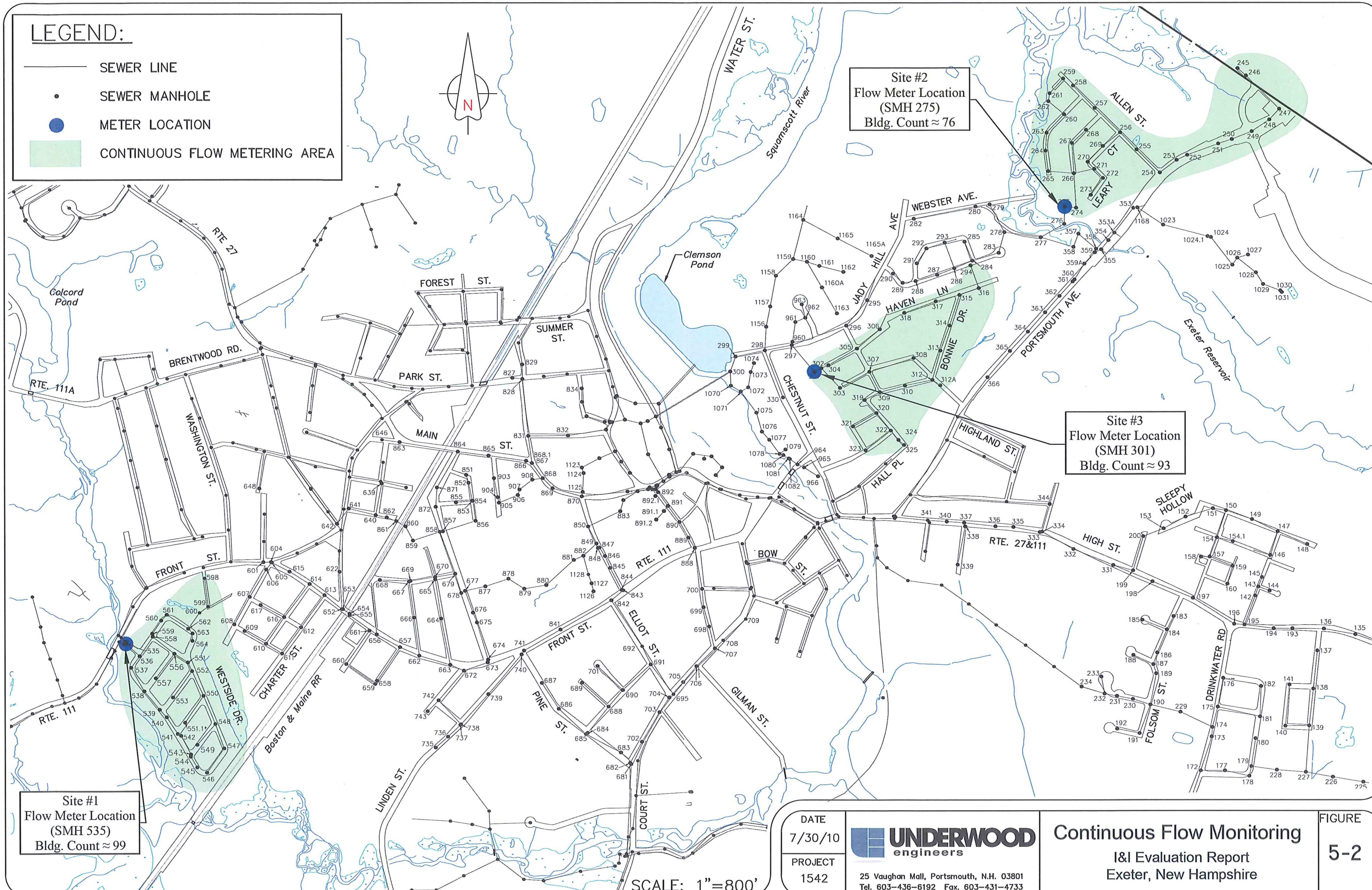
Sanitary flow for the entire system was estimated based on 350 gpd per building, assuming that there are about 3,000 users in the system. Obviously there are industrial users in the system that use much more than typical residential units. To account for this, a per building value on the higher side of the typical range was chosen.

Infiltration estimates were made based on graphical data collected from the flow monitoring. Estimates of sanitary flow dry and wet weather infiltration estimated from graphical flow monitoring data from the pilot areas from April 8, 2009 through June 18, 2009. It was assumed that minimum flows measured during the night, during dry periods, consisted of dry-weather infiltration. This assumes that sanitary use in the residential pilot areas should be minimal during the night. Wet weather infiltration was assumed to be minimum flows measured during the night during periods of rain, based on the similar assumption that sanitary flows are negligible during the night in these residential areas.

Peak flows observed at each metering site during the period are shown in Table 5-2. For areas 2 and 3, these occurred during a rain event on April 21, 2009. The peak flow for the system on this date was used for the system wide value, but was measured at the WWTF influent (rather than more accurate meters that have since been installed at the pump station). No CSOs were recorded during that rain event.

Peak infiltration was calculated by subtracting the sanitary flow from the peak flow measured during the continuous flow monitoring period.

Peak Inflow in each basin was estimated by comparing the magnitude of daily minimum and maximum flows during dry weather and during wet weather. For example, if the typical min:max flow rate change during dry weather was 10,000 gpd and during rain events the min:max flow rate change was 30,000 gpd, then the inflow to the system was assumed to be 20,000 gpd. Flows in pilot area #3 varied so largely in magnitude during rain events that is was difficult to estimate inflow separate from infiltration. Graphically estimated inflow rates are shown in Table 5-2.



H:\SDSKPROJ\1542\1542LDD\dwg\1542_FIG_FLOWM.dwg, FIG 5-2, 11/12/2012 2:31:44 PM, rla

Table 5-2
Continuous Flow Monitoring Results
Pilot Areas 1-3
April – June 2009

	Site #1 (Westside Dr)	Site #2 (Downing Ct.)	Site #3 (Jady Hill)	System Wide (approximate)
House/Building Count	99	76	93	3,000+/-
Sanitary Flow estimate, gpd	13,000	12,000	10,000	1,100,000
Sanitary Flow per building, gpd	130	160	110	~350
Est. Dry Weather Infiltration, gpd	5,000	20,000	30,000	NA
Est. Wet Weather Infiltration, gpd	30,000	30,000	155,000-160,000	NA
Peak flow observed, gpd	140,000	70,000	235,000	2,700,000
Peak I/I (calculated)	127,000	58,000	225,000	1,700,000
Peak I/I per building, gpd	1,280	760	2,420	550
Estimated Inflow, gpd	43,000	5,000	20,000	NA

- NA = information not available
- Peak Flows observed during rain event April 21, 2009. Peak flow for system-wide column is from WWTF influent flowmeter (accuracy is questionable)
- Estimated inflow determined graphically from continuous flow monitoring graphs = (wet weather daily max-daily min flow) – (dry weather daily max – daily min flow).

5.3 Infiltration Investigations

5.3.1 Flow Isolation

Flow isolation was conducted during the period from April 13 to May 15, 2009 in portions of sub-basins A, B, C, D, F, G, H and I during dry-weather flow at night. The field work was conducted by UEI subcontractor Flow Assessment Services (FAS). Flow Isolation is typically performed during the night-time hours, during dry weather, to measure infiltration and some private inflow (sump pumps) into the system caused by high groundwater. Since night-time flows are typically very low, in the absence of known night-time dischargers (3rd shift at a processing factory, etc.) all flows measured during flow isolation are assumed to be from infiltration.

To start, large (1,000 LF) sections of pipe were isolated by plugging the upstream end of the reach and measuring flow at the downstream end of the reach. Based on the flow measured in the reach, an approximate infiltration rate for the reach was calculated in the field. When high infiltration was observed, based on the measured flow in these large reaches, individual sewer reaches (manhole to manhole) were then flow isolated to measure flow, and permit subsequent calculation of infiltration rates in each sewer reach. The results of the flow isolation work are shown on Table 5-3. Areas exhibiting high I/I (greater than 2,800 gpd) are shown on Figure 5-3. Complete results from the flow isolation work can be found in Appendix A-14. Some lengths may vary between Table 5-3 and Appendix A-14, because sewer reaches that were attempted, but not able to be flow isolated have been removed from the lengths shown in Table 5-3.

A total of 751,104 gpd of infiltration was identified in the system during the flow isolation work. Of this, 553,528 gpd was measured in sewer pipes, 42,696 gpd was identified in manholes and 164,880 gpd was observed from services that discharged directly into manholes.

22% of the flow identified during flow isolation was from service flow entering manholes, in only 15 services. These are individual services that enter a manhole rather than a public sewer, and were found to be running with clear water. This equates to 10,992 gpd per service. Three of these services were from known industrial/institutional users, PEA and Osram. Removing those three services, the remaining flow averages 4,380 gpd/service for the remaining 12 services. It should be noted that a substantial repair was made to water and sewer lines in the area of the PEA service, and the Town believes that the water entering from the PEA service has been mitigated.

Manholes with identified service leakage should be investigated further to identify the source of the service water and ascertain whether or not it is private infiltration or inflow that can be removed from the system.

Table 5-3
Flow Isolation Results
Exeter, NH
April - May 2009

Subbasin	Length Flow Isolated, lf	Percent of Total Sewers Flow Isolated	Pipeline I/I measured, gpd	MH leakage identified, gpd	Service Line leakage identified, gpd	Number of Services entering MHs	Total I/I identified, gpd	Avg Infiltration, gpdim
A	5,466	9%	13,680	0	0	NA	13,680	1,352
B	11,072	103%	30,240	7,920	13,248	2	51,408	1,260
C	19,727	99%	95,544	2,880	10,080	2	108,504	2,271
D	6,400	104%	22,320	0	69,120	2 (PEA)	91,440	2,041
F	59,178	105%	163,944	22,536	11,952	4	198,432	1,721
F1 (Stratham)	4,809		2,448	0	0	NA	2,448	351
G	11,231	99%	58,968	0	8,640	3	67,608	3,346
H	21,860	100%	125,424	9,360	43,200	1 (Osram)	177,984	3,666
I	4,190	101%	30,960	0	8,640	1	39,600	4,323
	143,933	74%	543,528 72%	42,696 6%	164,880 22%	15	751,104	

Total I/I Identified (in 75% of system) = 751,104

Notes: 543,528 gpd pipeline includes flow from pipes and services entering the pipe between manholes
Service line leakage is only for service lines entering directly into a MH. All other service flow is included in pipeline I/I

Table 5-4 below summarizes the manholes that were identified as leaking, or which has leaking services entering them. See also Figure 5-3, which illustrates manholes that were leaking during any phase of the investigation.

Table 5-4
Flow Isolation Manholes with Identified Leakage or Service Leakage
April-May 2009

Subbasin	Manholes identified with leakage	Manholes with identified service leakage
B	1165, 1161, 895, 1192	1072, 1122
C	662	846, 872
D		891, 891.2
F	16, 21, 216, 217, 1063, 46, 52, 68, 59, 220, 221, 222, 225, 227, 228, 179, 229,232, 1142, 1140	209, 117, 64, 173
G		334, 160, 148
H	247, 248, 1025	1024.1
I		831

Typically, sewer reaches with infiltration greater than 4,000 gpdim are considered to have 'excessive' infiltration and further evaluation with TV inspection is considered cost effective. Given the tendency for groundwater to 'migrate' from one defect or reach to another as defects are repaired, additional reaches with greater than 2,800 gpdim were identified and more sizeable areas were recommended for TV inspection, to provide more contiguous TV areas. These areas typically included several several reaches with infiltration greater than 4,000 gpdim, reaches with infiltration greater than 2,800 gpdim, and occasionally reaches with less than 2,800 gpdim that were located between or immediately adjacent to reaches with high infiltration.

Based on the results of the flow isolation work, approximately 38,500 linear feet of sewers in 22 different areas were recommended for TV inspection. Due to funding issues, the areas were prioritized at Tier 1 through Tier 3, based on their ranking from observed infiltration as shown in Table 5-5.

Approximately 24% of the sewers that were flow isolated were recommended for further investigation through TV inspection, in attempt to further identify about 58% of the I/I observed during flow isolation.

Table 5-5
Sewer Reaches Recommended for TV Inspection
Based on Flow Isolation Results April-May 2009

Inspection Area	Rank by gpd/idm	Streets	Length to be Inspected (LF)	Flow Isolation I/I (gpd)	Flow Isolation I/I (gpd/idm)
10	1	Elm/Spring Street	376	66,240	72,187
4	2	Bonnie Drive	3,634	66,240	12,110
11	3	Tan Lane	230	5,760	10,957
3	4	Hampton Road	230	3,600	10,330
7	5	Holly Court	596	8,640	9,568
8	6	Ridgewood Terrace	1,300	18,000	9,138
12	7	Pine Street	1,509	20,160	7,816
14	8	Rockingham Street	210	2,880	7,200
6	9	High Street	4,792	46,800	6,446
21	10	Ashbrook Road	1,208	11,520	6,294
15	11	Front Street	3,636	44,280	6,093
13	12	Main Street	3,141	28,800	5,376
18	13	Hampton Road	1,489	11,520	5,106
5	14	Towle Avenue	1,367	10,008	4,832
1	15	Hayes Park/Jady Hill	2,120	12,960	4,789
19	16	Ashbrook R.O.W.	3,549	34,200	4,120
17	17	Hampton Road	2,684	16,416	4,037
20	18	Roberts Drive	663	3,600	3,584
2	19	Allen Street	1,450	7,200	3,277
16	20	Westside Drive	1,098	6,480	3,166
22	21	Hampton Falls Road	2,407	11,232	3,003
9	22	Pleasant View Drive	788	2,880	1,286

TOTALS: **38,477** **439,416 gpd**

Within Pilot Area

Flow Isolation flows and infiltration rate based on sewer line flow only (not MH and service flow)

5.3.2 MH Inspection

A total of 651 Manholes were inspected in sub-basins A, B, C, D, F, G, H, and I from April 13 to July 14, 2009. Manhole inspections were conducted from the surface and entered only if needed. The majority were completed between April 13 and May 30, 2009. Approximately 47,440 gpd of infiltration was identified (note, although not all the same manholes were observed, the data is similar to manhole leakage found during flow isolation.)

The breakdown by sub-basin is as shown in Table 5-6. Figure 5-4 highlights manholes that were found to be leaking during any phase of the investigation.

Table 5-6
Summary of Manhole Inspection Observations
April-July 2009

Basin	Number of MHs inspected	Frame leakage, gpd	Corbel leakage, gpd	Wall leakage, gpd	Floor leakage, gpd	Invert Leakage, gpd	Pipe connection leakage identified, gpd	Total Infilatration found, gpd	Average Infiltration per MH inspected, gpd
A	29	0	2,376	1,728	0	0	288	4,392	151
B	70	0	0	936	0	0	720	1,656	24
C	91	0	0	0	0	1,440	360	1,800	20
D	32	0	0	0	0	0	0	0	0
F	255	0	720	3,960	0	0	12,520	17,200	67
G	50	0	0	0	0	0	2,160	2,160	43
H	105	144	144	5,328	1,008	1,440	12,024	20,088	191
I	19	0	0	0	0	0	144	144	8
Total	651	144	3,240	11,952	1,008	2,880	28,216	**47,440**	

In general, the period during the manhole inspection was not a period of high WWTF flow, nor high groundwater. Manholes in sub-basins A and H were found to exhibit the most leakage.

Complete manhole inspection reports are included in Appendix Volume II. Appendix A-15 includes summary tables, by sub-basin, where manholes are prioritized, first by infiltration observed, and then by the condition assessment, to aid the Town in targeting repairs.

5.3.3 *Internal CCTV*

Internal TV inspection was conducted in approximately 38,500 linear feet of sewers in sub-basins A, B, C, D, F, G, and I as a result of infiltration observed during the flow isolation phase. Figure 5-3 illustrates the 22 different 'areas' that were TV inspected.

Eastern Pipeline Services, Inc. conducted TV inspection, under subcontract to UEI, during the following periods during the spring of 2010 (Figure 5-1):

March 8-11 (Tier 1)
March 25-26 (Tier 2 & 3)
March 29- April 2 (Tier 2 & 3)
April 5-9 (Tier 2 & 3)
April 12 (Tier 2 & 3)
May 6 – 18 (Tier 2 & 3)

Based on flow metering at the WWTF influent and main PS, the late March and early April TV inspections appear to have occurred during a peak groundwater/infiltration period.

In general, there was a lack of snowpack this year to maintain high groundwater conditions through snowmelt. However, a very rainy spring raised groundwater levels sufficiently. Two very significant rainfall events occurred on February 25th and March 14-15 which raised the groundwater table for TV inspection in late March and early April. In reviewing groundwater levels in several USGS wells in New England, most exhibited peak groundwater levels this year that were higher than any other years in recent history. Because of the extensive quantity of TV inspection performed, the work spanned a large time period (3 months), over which groundwater conditions varied significantly.

The results of the TV inspections are summarized below in Table 5-7. I/I identified during flow isolation includes mainline sewer flow only, not manhole leakage. The TV inspection allows further division of the mainline sewer flow between mainline flow and service flow. Complete summary tables including leakage observed for each reach, and proposed repairs are included in Appendix A-16. Complete TV inspection reports can be found in Appendix Volume III.

Table 5-7
Results of TV Inspections
April-May 2009

Tier	Inspection Area	Rank by gpd/idm	Streets	Length to be Inspected (LF)	Flow Isolation I/I (gpd)	Observed I/I During TV Inspection: TV Inspection - Services (gpd)	TV Inspection - Mainline (gpd)	TV Inspection - Total Flow (gpd)
Tier 1 (5,066 ft)	10	1	Elm/Spring Street (PEA)	376	66,240	2,880	0	2,880
	4	2	Bonnie Drive	3,634	66,240	5,300	15,855	21,155
	11	3	Tan Lane	230	5,760	0	0	0
	3	4	Hampton Road	230	3,600	0	3,612	3,612
	7	5	Holly Court	596	8,640	4,680	1,810	6,490
Tier 2 (15,796 ft)	8	6	Ridgewood Terrace	1,300	18,000	4,820	5,320	10,140
	12	7	Pine Street	1,509	20,160	29,180	2,000	31,180
	14	8	Rockingham Street	210	2,880	50	0	50
	6	9	High Street	4,792	46,800	12,460	15,920	28,380
	21	10	Ashbrook Road	1,208	11,520	5,120	0	5,120
	15	11	Front Street	3,636	44,280	8,260	5,400	13,660
	13	12	Main Street	3,141	28,800	3,940	2,680	6,620
Tier 3 (17,615 ft)	18	13	Hampton Road	1,489	11,520	6,460	9,950	16,410
	5	14	Towle Avenue	1,367	10,008	7,580	1,380	8,960
	1	15	Hayes Park/Jady Hill	2,120	12,960	70	2,060	2,130
	19	16	Ashbrook R.O.W.	3,549	34,200	6,940	2,800	9,740
	17	17	Hampton Road	2,684	16,416	9,610	9,150	18,760
	20	18	Roberts Drive	663	3,600	800	2,440	3,240
	2	19	Allen Street	1,450	7,200	3,280	500	3,780
	16	20	Westside Drive	1,098	6,480	3,880	0	3,880
	22	21	Hampton Falls Road	2,407	11,232	1,640	1,100	2,740
	9	22	Pleasant View Drive	788	2,880	0	3,580	3,580
			TOTALS:	**38,477**	**439,416**	**116,950**	**85,557**	**202,507**
			Percent of Total (Based on TV Observations)			58%	42%	

(Shaded) Within Pilot Area

5.4 Inflow Investigations

5.4.1 Property Questionnaires

UEI developed an informational pamphlet and questionnaire which were mailed to all sewer users in Town. The questionnaires, which were distributed on September 8th and 11th, 2009 via USPS, asked residents to answer questions about floor drains, sump pumps, roof drains and yard drains on their property. A sample of the brochure and questionnaire is included in Appendix A-17. 3,200 questionnaires were mailed to customers. A total of 580 surveys were returned (18%). From the 580 surveys returned, 45 respondents indicated that they had a sump pump that was piped to the sewer (or the discharge of their sump pump was unknown). This equates to about 8% of the total number of respondents. 65 respondents indicated that their homes had drains that were piped to the sewer or had drains with unknown terminations. This equates to about 11% of the respondents. In addition to the fliers, a public information meeting was held on September 16, 2009. Drawings included in Appendix A-17 present a graphical representation of the questionnaire results.

5.4.2 Building Surveys/Inspections

UEI subcontracted with FAS to perform internal house inspections from October 6 – 28, 2009 in the three 'pilot' areas previously described. A total of 306 homes were surveyed, however entry was only obtained to 243 homes, which resulted in the following findings:

- 62 of the homes inspected were found to have sump pumps that discharged to the sewer, or the pump discharge was unknown (14 in the West Side Drive Pilot Area, 12 in the Downing Court Pilot Area, and 36 in the Jady Hill Pilot Area). This equated to 25% of the homes to which entry was gained
- 5 of the homes inspected were found to have exterior drains that discharged to the sewer or the termination of the drain was unknown. This equated to 2% of the homes inspected.

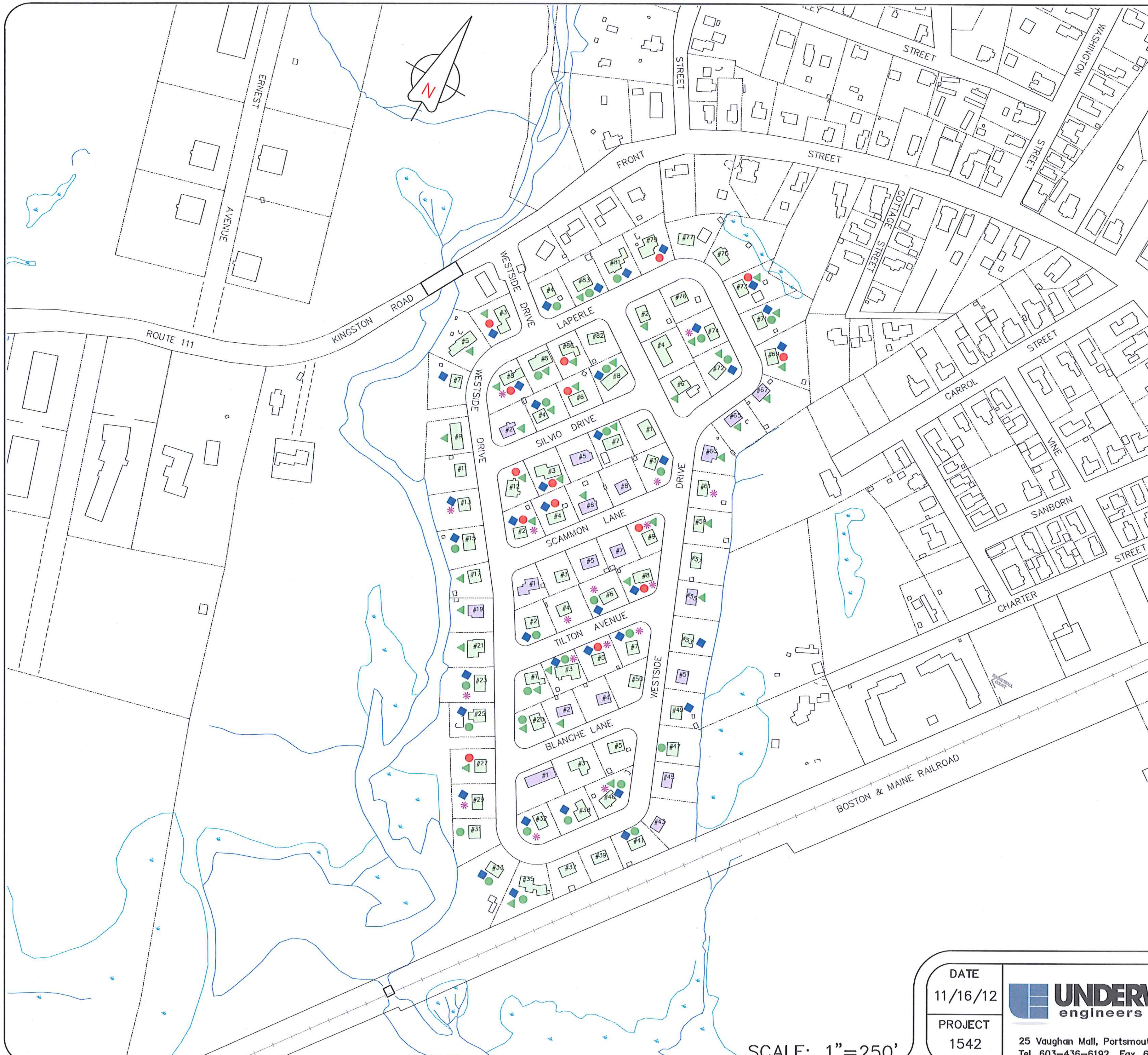
Although a small number of drains were identified, illicit drains are inherently difficult to identify. Foundation drains and other drains can be connected to sewer laterals below grade with no visible indication that they exist.

Figures 5-5 through 5-7 illustrate the results of the house-to-house survey. Completed house surveys are included in Appendix Volume V.

5.4.3 Town-Performed Smoke Testing

Subsequent to UEI's August 2010 draft of this report, the Town performed smoke testing of the area in the vicinity of the former Exeter High School and around Front St./Tan Lane. Roof leaders and a catch basin on the Phillips Exeter Academy (PEA) Campus were found to be connected to the sewers. In addition, roof leaders and catch basins from the U.S. Post office and a street curb drain in front of the Post Office were also observed to be connected to the sewer. It should be noted that the PEA roof leaders did not smoke during testing, but the connections were confirmed by PEA personnel who said that the roof drains are equipped with traps that prevents smoke to exist the roof. In the vicinity of PEA and the Post Office, it is estimated that a drainage encompassing approximately 3 acres was identified as directly connected to the sewer (Figure 5-8), approximately 2.4 acres of which is connected to the Water St. side of the Spring St. Diversion Structure (Basin C) and 0.6 acres connected to the Spring St. side of the Spring St. Diversion Structure (Basin D). This suspected inflow contribution will be discussed in later sections of this report.

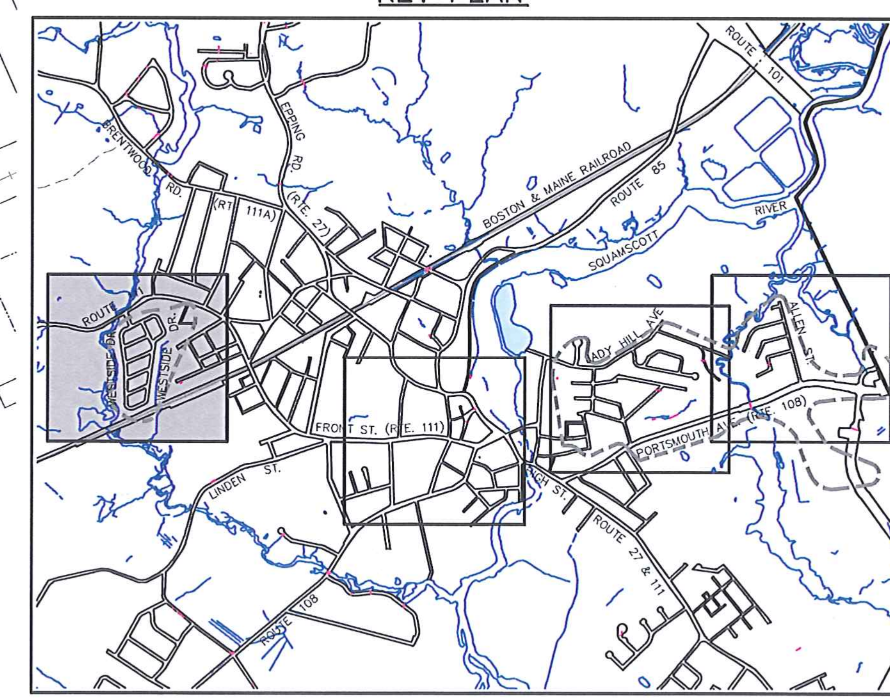
While drainage from the former Exeter High School (roof leaders, etc.) is still suspected to be connected to the wastewater collection system, it was not confirmed through smoke testing.



LEGEND:

- HOUSE SURVEY COMPLETE
- DENIED ACCESS
- SUMP PUMP DISCHARGE SEWER OR UNKNOWN
- SUMP PUMP DISCHARGE SURFACE
- EXTERIOR DRAIN TO SEWER
- EXTERIOR DRAIN TO SURFACE/STORM DRAIN
- BASEMENT FLOODING
- BASEMENT DRAINS

KEY PLAN



DATE	UNDERWOOD engineers	Pilot Area 1	FIGURE
11/16/12	25 Vaughan Mall, Portsmouth, N.H. 03801	Building Inspection Summary	5-5
PROJECT	Tel. 603-436-6192 Fax. 603-431-4733	I&I Evaluation Report	
1542		Exeter, New Hampshire	

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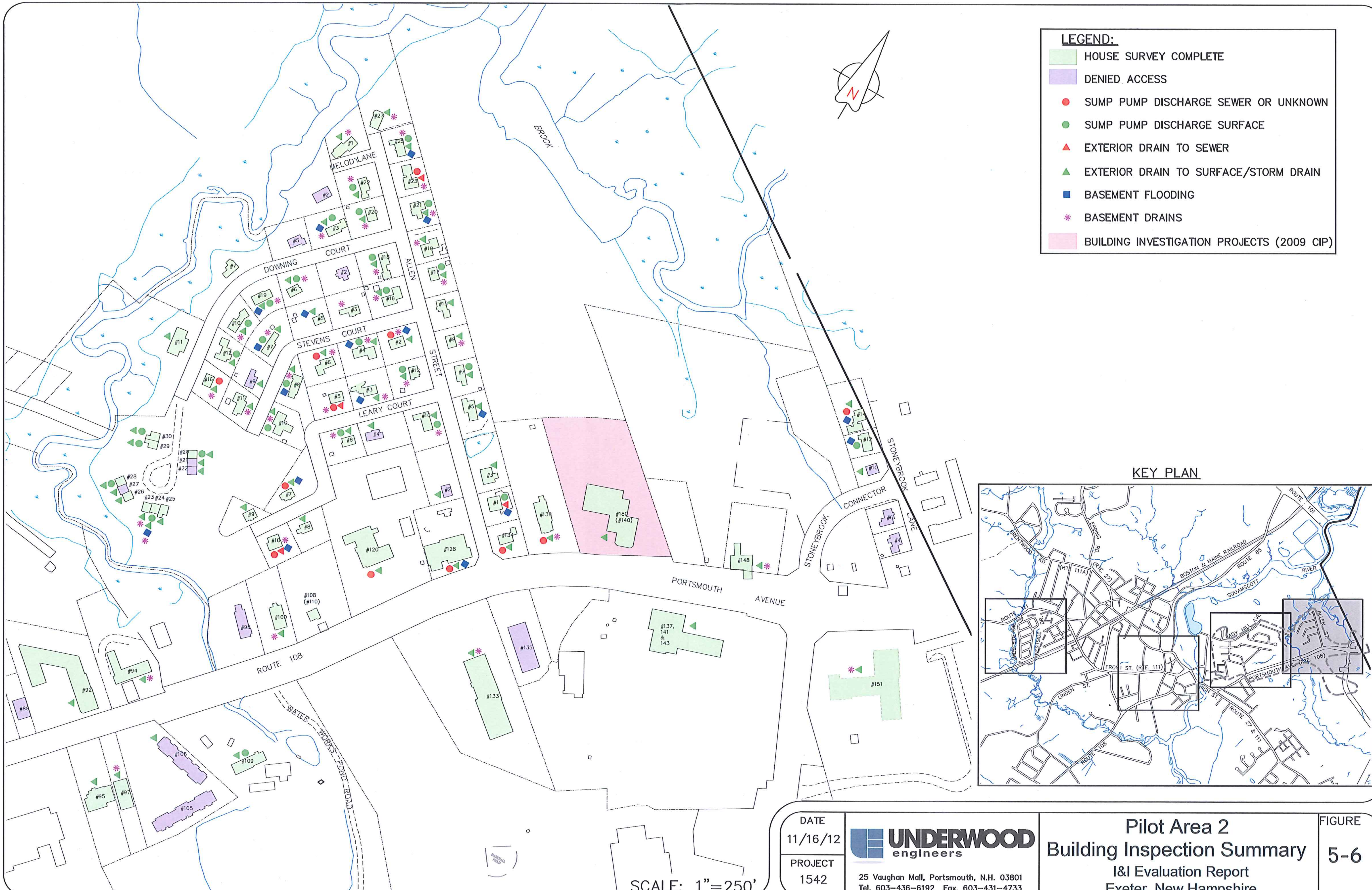

DATE		Pilot Area 2	FIGURE
11/16/12	UNDERWOOD engineers	Building Inspection Summary	5-6
PROJECT	25 Vaughan Mall, Portsmouth, N.H. 03801	I&I Evaluation Report	
1542	Tel. 603-436-6192 Fax. 603-431-4733	Exeter, New Hampshire	

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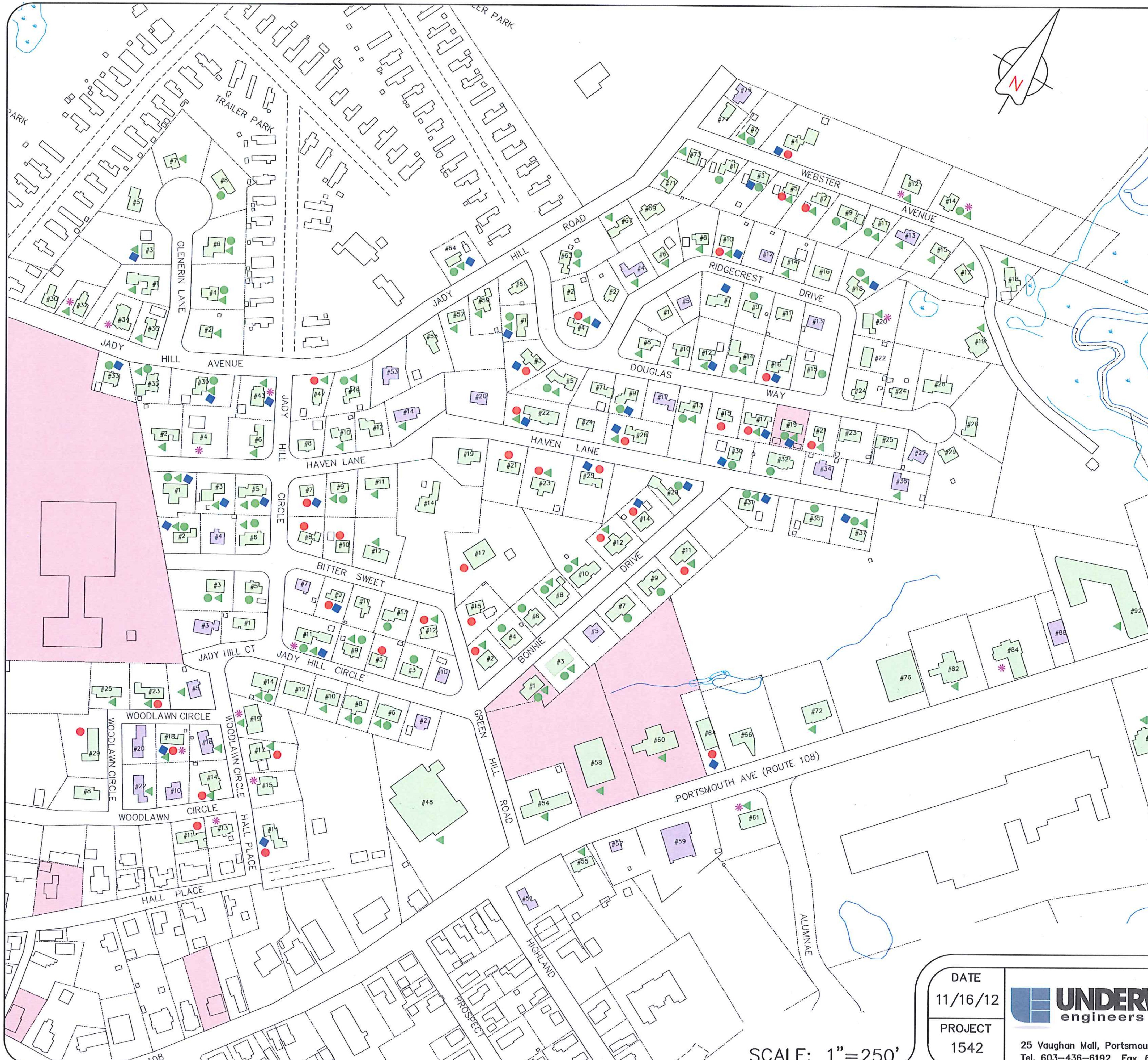
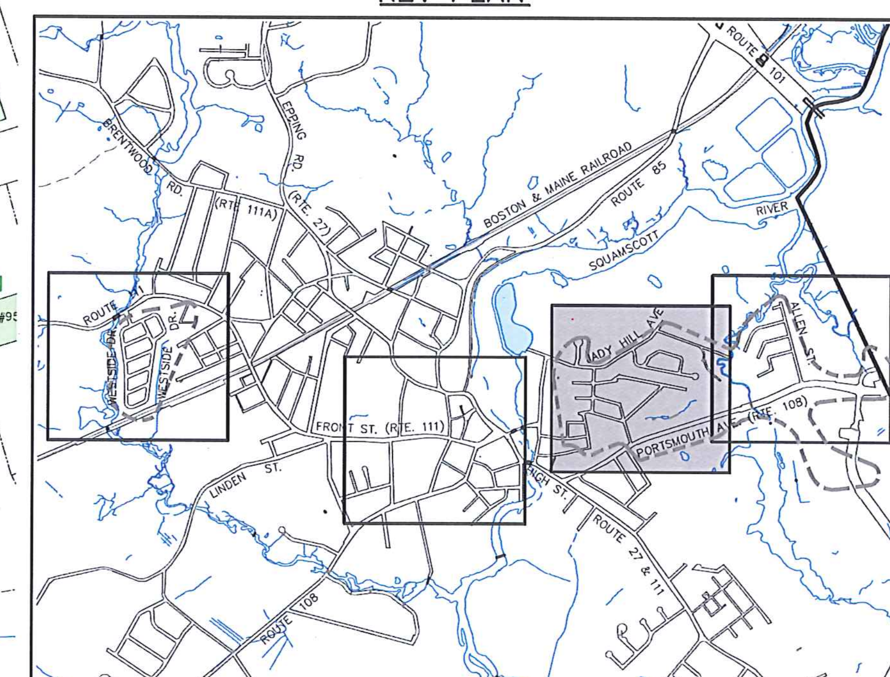


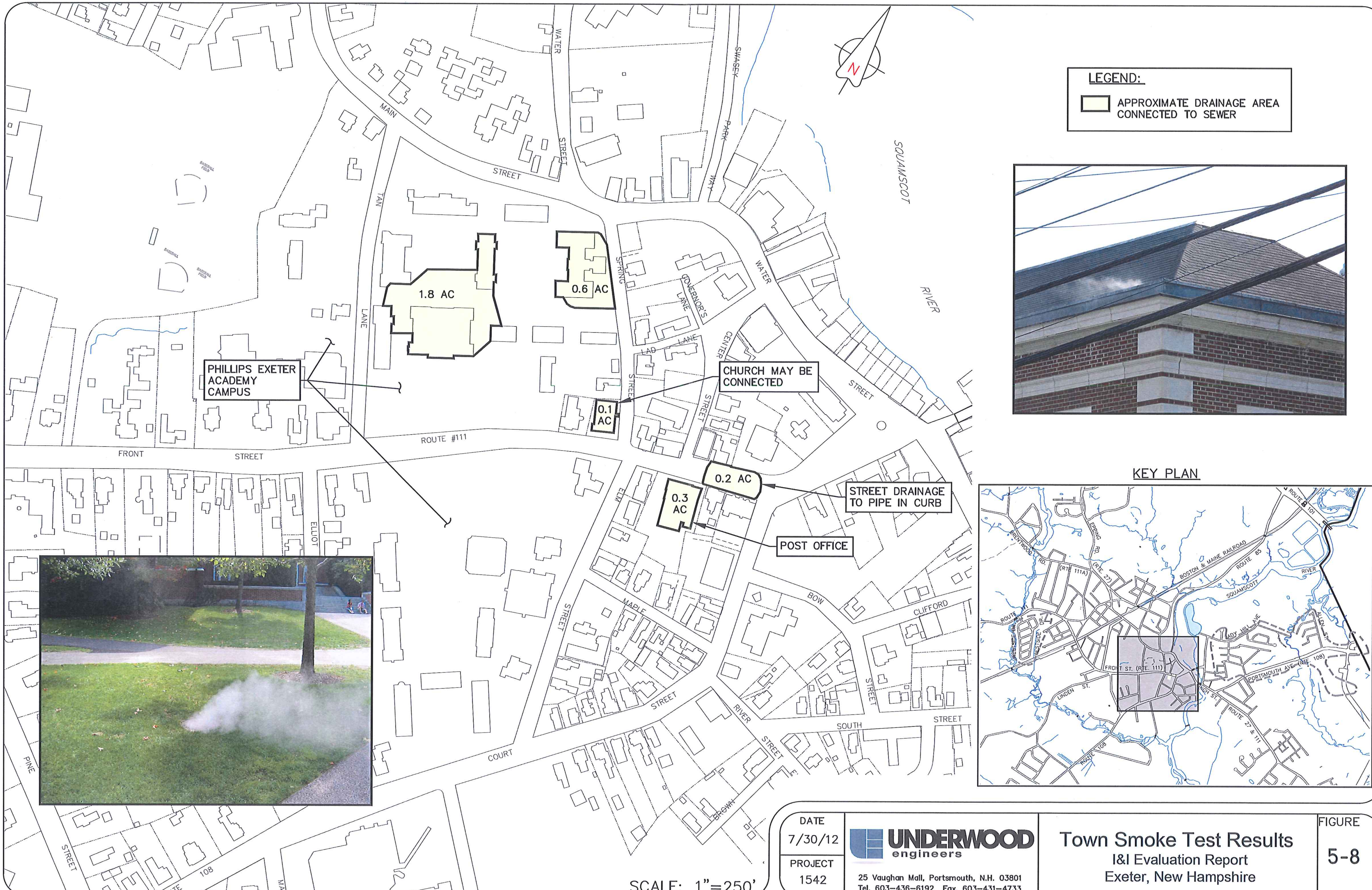
LEGEND:

- HOUSE SURVEY COMPLETE
- DENIED ACCESS
- SUMP PUMP DISCHARGE SEWER OR UNKNOWN
- SUMP PUMP DISCHARGE SURFACE
- EXTERIOR DRAIN TO SEWER
- EXTERIOR DRAIN TO SURFACE/STORM DRAIN
- BASEMENT FLOODING
- BASEMENT DRAINS
- BUILDING INVESTIGATION PROJECTS (2009 CIP)

KEY PLAN



DATE	UNDERWOOD engineers	Pilot Area 3	FIGURE
11/16/12	25 Vaughan Mall, Portsmouth, N.H. 03801	Building Inspection Summary	5-7
PROJECT	Tel. 603-436-6192 Fax. 603-431-4733	I&I Evaluation Report	
1542		Exeter, New Hampshire	



DATE			FIGURE
7/30/12	UNDERWOOD engineers 25 Vaughan Mall, Portsmouth, N.H. 03801 Tel. 603-436-6192 Fax. 603-431-4733	Town Smoke Test Results I&I Evaluation Report Exeter, New Hampshire	5-8
PROJECT 1542			

H:\SDSKPROJ\1542\1542LDD\dwg\1542_FIG_INVESTIGATION.dwg, smoke fig 5-8, 11/12/2012 2:55:23 PM, rla

5.5 Diversion Structure Investigations

5.5.1 Evaluation and Calibration

Flow Assessment Services (FAS), under subcontract to UEI visited both the Spring Street and Water Street diversion structures to document and evaluate their components on May 11, 2009. Copies of the inspection reports are included in Appendix A-18. A forced CSO event to calibrate the overflow weirs was not recommended due to regulatory concerns. However, UEI personnel observed an actual CSO event on March 30, 2010 (see Memo Appendix A-19) and performed rough field measurements to measure flow over the weir to verify calibration. FAS noted in their report that the sonic target for the ultrasonic level measuring device at the Spring Street diversion structure (which is set at the same elevation as the weir) was tilted downward 1-inches in a 6-inch run and needed to be leveled prior to calibration. However, the CSO metering/monitoring system was upgraded in December 2010, addressing deficiencies and adding additional instrumentation.

5.5.2 Level Survey

Doucette Survey, under contract to UEI, completed a level survey of the sanitary sewer system between the diversion structures and the main pumping and of the CSO infrastructure downstream of the Spring Street and Water Street diversion structures in May 2009. The Clemson Pond outlet structure was also included in the level survey. However, the two, 36" siphon outlets at Clemson Pond were not visible for measurement, and as-built information for the siphon was assumed/used for the evaluation. Profiles developed from this survey are included in Appendix A-20. Evaluation of system hydraulics using this information is included in Section 7.

6. INFILTRATION AND INFLOW ANALYSIS

6.1 Overview and Approach

Current State and Federal regulations were reviewed to establish criteria for determining if the I/I found in the Town of Exeter wastewater collection system is considered to be excessive or not. It is understood that CSOs are triggered due to high I/I in the system.

I/I is typically categorized into two components, infiltration and inflow. Infiltration is considered to include groundwater entering the system through defects in mainline sewers, manholes and services. Inflow is typically considered caused by precipitation events (or snowmelt) and includes extraneous water directly entering the system through catch basins, drains, roof leaders, and other stormwater piping that is directly connected to the system. Inflow sources were considered to be identified by the 1997 CDM phase II study, and public sources of inflow were subsequently removed from the system. Although the intent of this study was to evaluate infiltration in the system, the nature of the collection system and study methodology often provides us with results that include not only infiltration into the public system, but also infiltration and inflow from private sources. For example, flow isolation involves measuring night-time flow in a sewer reach, during dry-weather, using a weir. Flow results from this metering will include infiltration into the public sewer and private service laterals. It may also include flow from private foundation drains or sump pumps that are connected to a service lateral. For this reason, the term I/I has and will be used in this study where appropriate, as opposed to strictly infiltration.

6.1.1 State of New Hampshire

The New Hampshire Department of Environmental Services does not have an established threshold limit for excessive I/I. New sewers are required to be designed to provide and I/I allowance of 300 gallons per day per inch diameter mile (gpd/idm). However, exceeding this limit in old sewers is not necessarily considered excessive.

Since New Hampshire does not have an established criteria, the MA DEP *Guidelines for Performing Infiltration/Inflow Analyses and Sewer System Evaluation Survey*, Revised January 1993, are often used. This standard indicates that 4,000 gpd/in-dia/mile be used as the threshold value for determining whether or not I/I is excessive based on dry weather flows attributable to infiltration.

Given that the State of New Hampshire does not have a threshold value, the MADEP threshold (4,000 gpd/idm) was used in this study to determine whether or not I/I in a sewer reach was excessive and worthy of further evaluation through TV inspection.

Cost effectiveness of removal is then used as the criteria for improvements to the system.

6.1.2 Federal Criteria

The Environmental Protection Agency (EPA) indicates in 40 CFR Part 133 that 1,500 gpd/idm may be used as the threshold value for determining whether or not I/I is excessive based on dry weather flows attributable to infiltration.

Based on past experience and cost effective analyses, coupled with the expense of TV inspection and sewer rehabilitation, the MADEP value of 4,000 gpdim appears to be a more 'cost effective' value for further investigation and rehabilitation, and was used for this study in lieu of the EPA value.

6.2 Impacts of Infiltration/Inflow

The impacts of infiltration and inflow on the system include the following:

1. Discharges (permitted) from the Spring St and Water St CSO diversion structures to Clemson pond during certain rain events.
2. Sanitary Sewer Overflows (SSO) (unpermitted). The Town reported that during the period from April 1, 2004 to April 16, 2007, twenty-five (25) SSOs occurred during wet weather (Appendix A-12).
3. Backups in the Main Pump Station Wetwell
4. During periods of high flow, additional personnel resources may be needed.
5. Increased capital costs associated with capital improvements. This may be significant when the Town needs to replace/upgrade the WWTF to meet anticipated future nutrient limits.
6. Reduced wastewater treatment facility reserve capacity for additional connections to the sewer system.
7. Increased equipment use requiring increased electricity thereby increasing operational costs.
8. Increased equipment wear and maintenance.
9. Sewer line surcharges and back-ups.
10. Discharge permit violations, including effluent quality, CSOs and SSOs.

6.3 Infiltration Evaluation

6.3.1 Pilot Areas

The use of pilot areas will enable the Town to better determine the methods of mitigation, cost and effectiveness of various technologies approaches/policies that might be used for I/I reduction. This will provide a basis to extend these practices system wide, if successful and cost effective.

Qualitative observations of Flow Monitoring graphs (Appendix A-13):

Site 1 – West Side Drive: Exhibited a slight increase in instantaneous peak flow during some rain events. For example, the normal range of minimum to maximum flow is about 0.032 mgd during dry weather (0.01 mgd daily peak to 0.042 mgd daily peak). During rain events in June, instantaneous wet weather minimum to maximum flow ranges of about 0.072-0.075 mgd were observed (0.035 mgd daily min to 0.11 mgd daily peak and 0.018 mgd daily min to 0.09 mgd daily peak). This would indicate **rainfall induced inflow** of approximately 0.043 mgd in the pilot area (43,000 gpd). Considering the West Side Drive area has relatively new PVC sewers, sump pumps would be suspect. Minimum flows increase from 5,000 gpd during dry-weather to 35,000 gpd (about a 700% increase), indicating about 30,000 gpd of infiltration is due to wet weather effects. This may be due to leaks in the system or sump pumps.

Earlier rain storms in May produced little if any increased flow – in general the minimum to maximum flow range remained fairly constant and the flows increased uniformly, indicating infiltration rather than inflow, or delayed inflow due to sump pumps.

Site 2 – Downing Court: Exhibited a typical minimum to maximum range of about 0.026 MGD. Peaks were higher during rain event by about 0.005 MGD (inflow). Minimum flows were approximately 0.01 MGD (150%) higher during and after rain events, than during dry-weather, indicating approximately 10,000 gpd of infiltration may be entering the area. Flows appeared to take approximately 10 days or more after rain events to return to dry-weather levels, indicating infiltration may be due to a high groundwater table induced by rainfall.

Site 3 – Jady Hill: Exhibited the most pronounced response to rain events of the 3 metering sites. Flows increased quickly and dramatically with rainfall events. Minimum flows increased by up to 130,000 gpd (over 400% increase) during rain events, compared to dry weather flows (infiltration). The response to rain events decreased significantly during the metering period (over 400% increase during the first rain event, to about 75% increase during the last event), likely indicating that as groundwater levels decreased from April to June, I/I into the system decreased. This may be due to decreasing groundwater levels decreasing infiltration into pipe defects, or may be due to decreased flow from sump pumps into the system as groundwater levels subside.

Considering that all three pilot areas are about the same size (100 homes), Area #3 (Jady Hill) should be targeted for infiltration and private inflow removal, while Area #1 (Westside Dr) should be targeted for inflow removal. Proposed pilot projects might include sewer and service replacement in Area #3 (Jady Hill) and sump pump and other private inflow removal in Area #1 (West Side Dr.)

The increases in flows observed during precipitation events in each of these basins may be due to infiltration into the system, or private inflow sources (sump pumps, roof drains, etc.)

Considering each of the 3 pilot areas are approximately the same size (100 homes), infiltration removal should be targeted in Pilot area #3 (Jady Hill).

Flow isolation, manhole inspections, house-to-house inspections, and selected TV inspections were performed within the pilot areas in an attempt to characterize the sources of I/I within a focused study area since performing all these investigations system-wide is cost prohibitive.

6.3.2 Flow Isolation

A total of 566,568 gpd of I/I was identified in main-line sewers during flow isolation (75% of the system). In addition, 42,696 gpd of manhole leakage was identified (6%), and 146,167 gpd of flowing services entering manholes were identified (19%), for a total of 755,431 gpd of I/I in the flow isolated basins.

Nearly 20% of the total flow identified during flow isolation was due to only 15 services entering manholes. Considering that most services enter the mainline and not manholes, the total amount of I/I from services will be much greater than 20%.

Flow isolated I/I flows were compared with those estimated by continuous monitoring by CDM in 1997. Since groundwater conditions are different from year-to-year, direct comparisons cannot be made, but rather only benchmark comparisons. In addition, many modifications to the system have been made since the 1997 flow monitoring. Approximately 75% of the pipe length flow monitored by CDM in 1997 was flow isolated in 2009. Peak infiltration was estimated at 1,591,000 gpd in 1997. Flow isolation in 2009 identified 755,431 gpd, or about 47% of the peak infiltration estimated in 1997. When corrected for the different lengths in pipeline used in the two studies, approximately 63% of the peak infiltration estimated in 1997 was identified during flow isolation in 2009. This reduction in flow may be due to improvements made to the system since the 1997 study, as well as variation in groundwater conditions.

6.3.3 Manhole Inspections

Manhole inspections were not performed during high groundwater conditions, so infiltration identified during the inspections (47,440 gpd) is expected to be higher during high groundwater conditions. The inspections provide a method to prioritize manhole repairs, based on condition.

Additional inspections may be warranted during periods of high flow, to further identify surface water leakage that may be occurring at manholes.

We suspect that additional water may enter the system through defective manholes during periods of higher groundwater and heavy rain. For example, during TV inspections, during heavy rain, inspectors reported evidence of significant inflow entering the manholes along the interceptor in basin F from MH 210 to 201. DPW staff have previously identified this interceptor from MH 228 to MH 201 as 'suspect'.

Appendix A-15 contains lists of inspected manholes for each sub-basin, sorted first by infiltration observed, and then by condition from worst to best. Condition was ranked based on the cumulative condition rating of the components (frame/cover, corbel, walls, floor/invert).

6.3.4 TV Inspections

Table 5-7 summarizes the infiltration identified during TV inspection.

Key observations were made from the TV inspections:

1. More than half the I/I identified during TV inspection is from private sources.
2. Approximately 50% of the flow observed during flow isolation in 2009 was observed during TV inspection.
3. In general, where AC sewers were present, defects observed were primarily leaky services, break-in service connections, and minor spalling of the pipe interior.
4. In general, where PVC sewers were present, leakage was observed from services.
5. In general, where VC sewers were present, defects were primarily leaky services (often asphalt-impregnated fibre pipe services), break-in services, as well as miscellaneous pipe defects. VC sewers are in need of the most attention.

As has been found common with many systems, a large percentage of water entering the sewer system is believed to emanate from private sources. This water may be infiltration (groundwater) entering through the private service lateral, foundation (or other) drains connected to the lateral, sump pumps, or roof laterals that discharge to the private service lateral.

Although much less, main-line infiltration was observed than service line infiltration, much of the main-line infiltration that was found, was due to break-in services. These are services that were installed simply by breaking a hole in the mainline sewer, inserting the service pipe, and attempting to seal the connection. As expected, these types of connections are much more prone to leakage than a integral factory wye-type connection.

It is not surprising to obtain different infiltration rates during the TV inspections in 2010 when compared to the flow isolation work in 2009. While flow isolation provides a definitive instantaneous flow measurement using a weir, TV inspection relies on a visual estimation of flow from the technician. Flow isolation is conducted during night-time low flow periods, and all flow is assumed to be infiltration . Obviously this is not always the case, and some nighttime flows may be due to sewer usage. TV inspection is done during the day when sanitary flows are occurring. TV operators attempt to identify if flow from a service is sanitary or clear water and identify only those that appear to be clear water. In addition, groundwater conditions are never the same from year to year. As expected, higher infiltration quantities were observed during the TV inspections during heavy rain events, than during drier periods. Due to the large quantity of TV inspection performed, not all reaches could be inspected during the peak groundwater time period. Contractual issues, including a Town meeting vote for funding of TV inspection work caused a slight delay in beginning TV inspection of Tier 2 and 3 areas.

Some generalizations that can be made from the TV inspection results include:

- Developing a plan to deal with service laterals and private I/I sources should be a high priority for the Town. Over 50% of the flow observed was due to service flow. This flow may be from leaky/defective service laterals, or other connections to the lateral such as sump pumps, foundation drains, basement drains, etc.

- Vitrified Clay (VC) sewers in general are in the worst condition, often with leaking joints, roots, offset joints, cracks, sags, etc. In general, services in these areas are in poor condition also, often being constructed of asphalt-impregnated fibre pipe or VC. The 1997 CDM study indicated that approximately 96,700 linear feet of VC sewers existed in the Town, or about 42% of the sewers.

- Mainline infiltration was found to be negligible in many several areas (pilot areas 2 and 3) and therefore certain areas where high I/I was found do not necessarily require improvements to the public sewers.

6.3.5 Jady Hill Pilot Area

As part of their capital improvement plan, the Town voted to move forward with water, sewer and road improvements in the Jady Hill area. In an attempt to incorporate sewer work from this study, UEI performed a detailed analysis of the I/I issues, pipe and manhole condition evaluations in the Jady Hill area, recommended repairs and developed project cost opinions for the work recommended in the Jady Hill Area. Sewers in the Jady Hill area were evaluated for rehabilitation (test and seal), lining, and replacement. As part of the Jady Hill evaluation, a cost-effective analysis was conducted to evaluate the cost-benefit to removing I/I when compared with pump and treat costs for a new treatment facility. These results are included and discussed further in Section 9. Appendix A-27 contains detailed analysis of the Jady Hill area.

A summary of observations and conclusions for the Jady Hill Pilot area include:

- **Jady Hill Area Observations**:
    - Sewer pipe is 8-inch VCP with 2 and 3 ft joints.
    - Most manholes are block; several missing apron, poor inverts
    - Majority of services (42 of 64) appear to be coal tar impregnated fibre pipe. This pipe is also known by manufacturing names Orangeburg or Bermico.

This pipe was typically installed in the mid 1940's through the 1960's. It has a tendency to deform (easily out-of-round), has no joint gaskets or sealant (just compression fittings) and is therefore very susceptible to infiltration and root intrusion, it is easily broken by roots, is difficult to couple new pipe onto the existing pipe, it is easy to cut with shovel, and has a limited structural life expectancy. This type of pipe has been referred to as a "coal tar impregnated toilet paper tube".

  - Majority of services are 'break-ins'.
  - Groundwater was not at its 'peak' the week Jady Hill was televised. As a result less infiltration flow was observed during TV inspection than during Flow Isolation.

- **Jady Hill Area Conclusions**:
  - Given the existing system, there are many potential I/I pathways (joints, broken joints, broken-in service connections, fibre pipe services, block MHs, private sumps and drains, etc.). Because of this, the cost of standard trenchless rehabilitation is higher than it might be in another system.
  - Three options were evaluated –
    - Rehabilitation (trenchless test and seal),
    - Cured-in-place-pipe (CIPP) lining (trenchless)
    - Replacement (open cut excavation).
    - Each option (in order) offers a better product (i.e. replacement offers the best product). The initial cost of replacement is more, but results in a better product (new, modern pipe with longer life span, less potential for future infiltration)
    - We recommended cured-in-place lining of some defective sewers where drainage pipes already exist, and replacement of sewers where new drainage would need to be installed.
    - We recommended replacement of private services to the house.
  - "Private I/I" (i.e.services) must be dealt with now. If only the mainline is dealt with, there will be issues trying to connect a new service stub at the curb to an existing fibre pipe service. In addition, a large portion of the I/I observed during flow isolation and continuous flow metering would remain if services are ignored. On a system wide basis, almost 60% of the I/I observed during TV inspection was private I/I.
  - New drainage piping with drain services is proposed on several streets where there is currently no drainage and sump pumps were observed during house-to-house inspections. There are other options to deal with sump pumps and private inflow sources, but a new storm sewer is the most costly, and therefore the most conservative assumption at this early stage of planning.
  - Our flow isolation results did not show high I/I in some areas of Jady Hill where the Town had proposed sewer replacement (i.e. Jady Hill Ave, Bittersweet). We recommend that higher priority be placed on sewers that were included in TV area No. 4.

- The proposed Jady Hill sewer project would include:
  - New Drainage piping with drain services on Bonnie Drive, Haven Lane, Bittersweet, and Woodlawn Circle.
  - Replace sewers on Bonnie Drive, Haven Lane, Bittersweet and Greenhill Road, where new drainage is proposed.
  - Install cured in place liners in sewers on Jady Hill Circle, Jady Hill Court, Hall Place and Woodlawn Circle, where there is existing drainage.
  - Replace Manholes #294, 305, 306, 307, 308, 312, 312A, 313, 314, 315, 316, 317, and 318.
  - Rehabilitate Manholes # 309, 310, 319, 324 and 323.
  - Replace sewer services (whole project area).
  - Separate private I/I sources. This would include replacement of service laterals, disconnections of drains and removal of sump pumps. Additional information and alternatives for accomplishing this are included in Section 9.

The total cost of the proposed Jady Hill Sewer project was about $3.1Million (see Appendix A-27).

6.3.6 Services

If infiltration reduction is to be successful in the Town of Exeter, private services must be dealt with. Large expenditures can be made to repair and replace main-line public sewers, but infiltration will remain unless it is removed at the source, which in most cases involves private services. Nearly 20% of the I/I identified during flow isolation was due to services, and this included only services directly entering manholes. Nearly 60% of the flow identified during TV inspection was due to private services. In addition, during house-to-house inspections of three small pilot areas, over 20% of homes were found to have sump pumps or other drains that discharged to the sewer system or had an unknown discharge point.

This is not an unusual situation, as many towns and cities across the country are finding out. It is however, a more difficult problem to tackle than repairing/replacing mainline sewers in the public right-of-way. Municipalities across the country have adopted a wide variety of programs in attempt to tackle private I/I removal. Some of these options are discussed later in Section 8.

6.4 Inflow Evaluation

To determine the affects of rainfall on overall I/I in the Town's sewer system, metered flows and rainfall data were reviewed. The impacts of rainfall vary depending upon the rain intensity and duration of the storm event.

Inflow sources could include any of the following:

- Roof leaders connected to the sewer system
- Basement, yard, patio and/or stairwell drains connected to the sewer system
- Sump pump connections to the sewer system
- Open sanitary cleanouts in basements
- Manhole cover holes
- Loose manhole covers
- Cracked and deteriorated manhole brickwork
- Cracked pavement around manholes
- Manholes located in the shoulder of the roadway or in depressions
- Cross connections from storm drains and catch basins

Total inflow into the sewer system consists of direct inflow and delayed inflow. Direct inflow results from direct connections between stormwater flow paths and the sewer system (i.e. holes in manhole covers, cross connections, roof leaders, basement drains, yard drains, etc.) and is indicated by a wastewater flow increase close to the start of the rain event and a drop off soon after the rain event is over. Delayed inflow results from connections such as sump pumps, manholes in depressions where puddles develop, etc., which discharge inflow into the sewer system slowly for a period of time following the storm event. Delayed inflow is indicated by a slow return to normal flow rates following a rain event.

The components of inflow are illustrated by the following figure taken from the page 68 of the MA DEP *Guidelines for Performing Infiltration/Inflow Analyses and Sewer System Evaluation Survey*, Revised January 1993:

Figure 6-1

Total Inflow Schematic

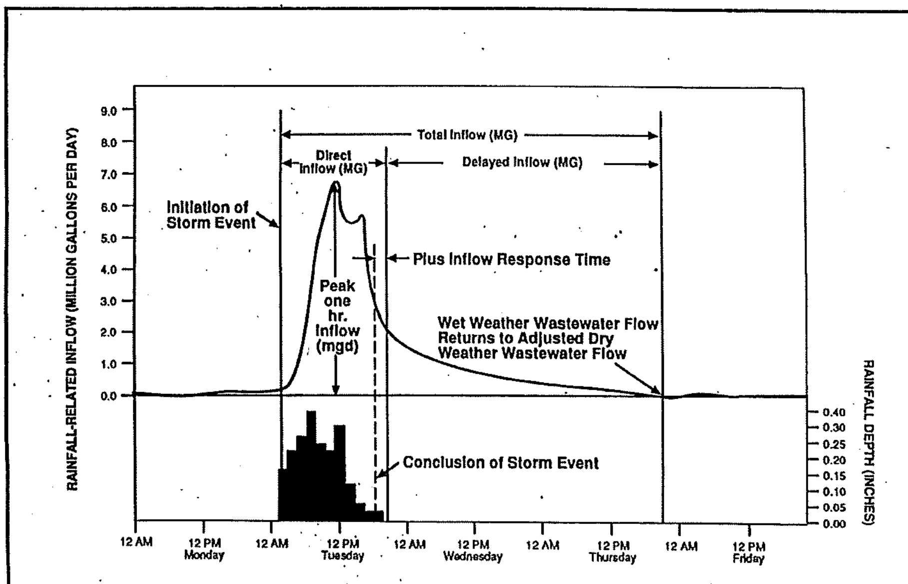


Figure 4
TOTAL INFLOW HYDROGRAPH CURVE
WITH RELATED RAINFALL HYETOGRAPH

In order to quantify inflow, the metered flow rate (shown as "Wet Weather Wastewater Flow" on the figure) during a storm period, rainfall, and pre-storm flow rate (shown as "Adjusted Dry Weather Wastewater Flow" on the figure) were reviewed from continuous flow monitoring graphs. The area between the pre-storm and actual flow rates is calculated to be the total inflow. A period of time immediately preceding the storm event is used to represent the pre-storm flow rate. This period was repeated continuously for the period of delayed inflow. Rainfall is plotted to determine the inflow response time, or the time for an increase in flow rate to be observed following the beginning of the storm event. Direct and indirect inflow were quantified, where direct inflow was calculated as the volume of wastewater between the actual flow rate and pre-storm flow rate curves from the beginning of the storm event to the end of the storm event plus inflow response time and delayed inflow was calculated as the volume of wastewater between the flow rate and pre-storm flow rate curves from the end of direct inflow to the point at which the pre-storm and actual flow rate were approximately equivalent.

Inflow investigation was a priority of the 1997 CDM work. Smoke testing and dyed water testing were conducted across the system, and many inflow sources were identified. Since that study, the Town has undertaken many projects to remove inflow from the system. Most 'public' inflow sources have been removed from the system, but many "private" sources remain. Many suspect inflow sources identified by the CDM study, including roof leaders, still exist, and require further investigation by the Town. In addition to these sources, this study included house-to-house inspections in three small 'pilot' areas, and a town-wide questionnaire to users, to aid in identifying additional sources of private inflow to the system.

6.4.1 House Surveys

House surveys were conducted in only the three "pilot" areas of town (approximately 300 homes), in attempt to assess the prevalence of private inflow sources in a reasonably sized area, and to permit 'before and after' comparisons to evaluate the effectiveness of private inflow removal.

Of the homes inspected, 25% were found to have sump pumps that were connected the sewer, or the discharge was not identified. 2% were found to have other drains connected to the system.

If we assume a sump pump might contribute between 2 gpm (3,240 gpd) and 5 gpm (8,100 gpd), and 25% of the 3,200 users in the town may have sump pumps discharging to the sewer, approximately 1,940,000 gpd – 4,860,000 of flow could be the result of sump pumps.

Tables in Appendix A-21 summarize the results of the House to House inspections. Addresses highlighted in blue are believed to have sump pumps, while red addresses are believed to have roof drainage discharging to the sewer system.

In addition, approximately 72 remaining potential private inflow sources (mainly roof drains) were identified by the Town in their 308 response to EPA (Tables 4-1 and 4-2). Assuming approximately 0.04 acres per roof for a standard home, a runoff coefficient of 0.9 and a rain storm of 0.9 inches/hr, a peak inflow rate of approximately 14.5 gpm (21,000 gpd) could be discharged to the system from each roof. If 50% of those potential sources is actually connected to the system (36 roof drains), over 750,000 gpd peak flow rate could be discharged to the system. If a 0.28-inch/hour (1.7" over 6 hours) rain storm is considered, the total peak flowrate could be over 230,000 gpd.

6.4.2 House Questionnaires

A town-wide questionnaire was distributed as part of a public education mailing, in attempt to gain information as to the prevalence of private I/I sources in the Town as a whole. Unfortunately, the response to the questionnaire was relatively low, and responses were scattered over a wide geographic area.

Because of the relatively low (<20%) and scattered response to the questionnaire, few conclusions have been drawn from the information. Approximately 10% of respondents indicated the presence of a sump pump or drain that discharged to the sewer, or to an unknown location. These individual locations can be further investigated by the Town.

6.4.3 CSO Flow Metering and Town-Performed Smoke Testing

Smoke testing performed by the Town in September 2010 revealed approximately 3 acres of private inflow area in the vicinity of the PEA campus and U.S. Post Office (Figure 5-8) contributes flow to the Spring St. Diversion Structure (Sewer Basins C & D). It is understood that PEA and the Post Office are working to remove these direct connections which is anticipated to be completed by the fall of 2012. However, analysis of the August 19, 2011 CSO storm event reveals evidence that additional areas contribute to direct inflow than the 3 acres identified during Town smoke testing.

6.4.3.1 CSO Events – August 2011

During the August 19, 2011 storm event, approximately 1.64" of rain fell in approximately 1 hour which resulted in a 12 mgd hourly wastewater flow spike during and immediately following the rain (Figure 6-2). The main pumping station pumped at a rate of approximately 7 mgd for approximately 2 hours (580,000 gallons) and the Spring St. CSO discharged an average of approximately 5 mgd for 1 hour (220,000 gallons). There was no CSO flow from the Water St. Diversion Structure during this event. Since baseline sewer flows were low at the time of this CSO event, and the rapid response of flow in the system to rainfall, this two hour spike in flow can generally be attributed to direct inflow with some in-line storage. Subtracting baseline wastewater flows, **it is estimated that over 15 acres of impervious area is still connected to the system to generate the spike in wastewater flows observed during the August 19, 2011 1.64" rain event.** In actuality, this area might be much larger depending on whether connected drainage areas contain pervious surfaces. While there is not sufficient metering in the system to determine the location of the sources of inflow, the Spring St. CSO flow alone (neglecting the wastewater flow that entered the Main Pumping Station from Spring St.) indicates that over 5 acres of impervious area is connected in Sewer Basins C, D & I, the sewer basins that contribute flow to the Spring St. Structure (Figure 7-1).

Furthermore, review of a similar "summer spike" storm event on August 10, 2011, when approximately 1.9" of rain fell over 4.7 hours resulted in a spike in Main Pumping Station discharge to 4.7 mgd, but no CSO event (Figure 6-3). However, the water level rise behind the Spring St. Diversion Structure weir indicates that, of the Spring St. Diversion Structure contributory flow, there appears to be more significant direct inflow contribution from sewer basins C & I (Water St. side) than Sewer Basin D (Spring St. side). This is consistent with the Town smoke testing findings where approximately 0.6 acres were identified in Sewer Basin D (near the U.S. Post Office) and approximately 2.4 acres were identified in Sewer Basin C (PEA Campus). However, as discussed above there appears to be several additional acres, most likely in Sewer Basins C & I that have not yet been identified. Unfortunately, there are gaps in the Water St. side water level data as shown on Figure 6-3 during the peak of the event that prevents surety in this assessment and the data provides indicators only.

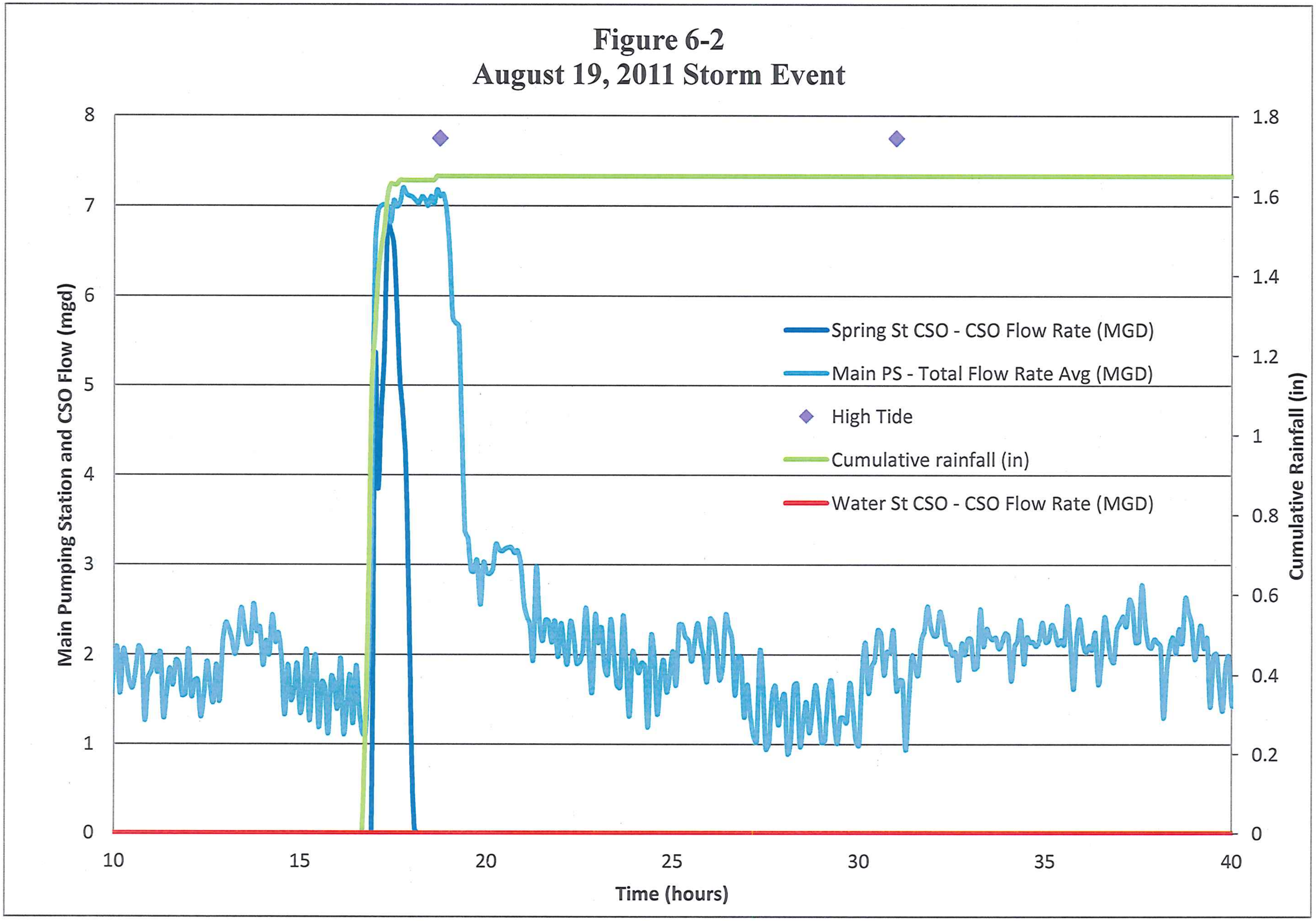
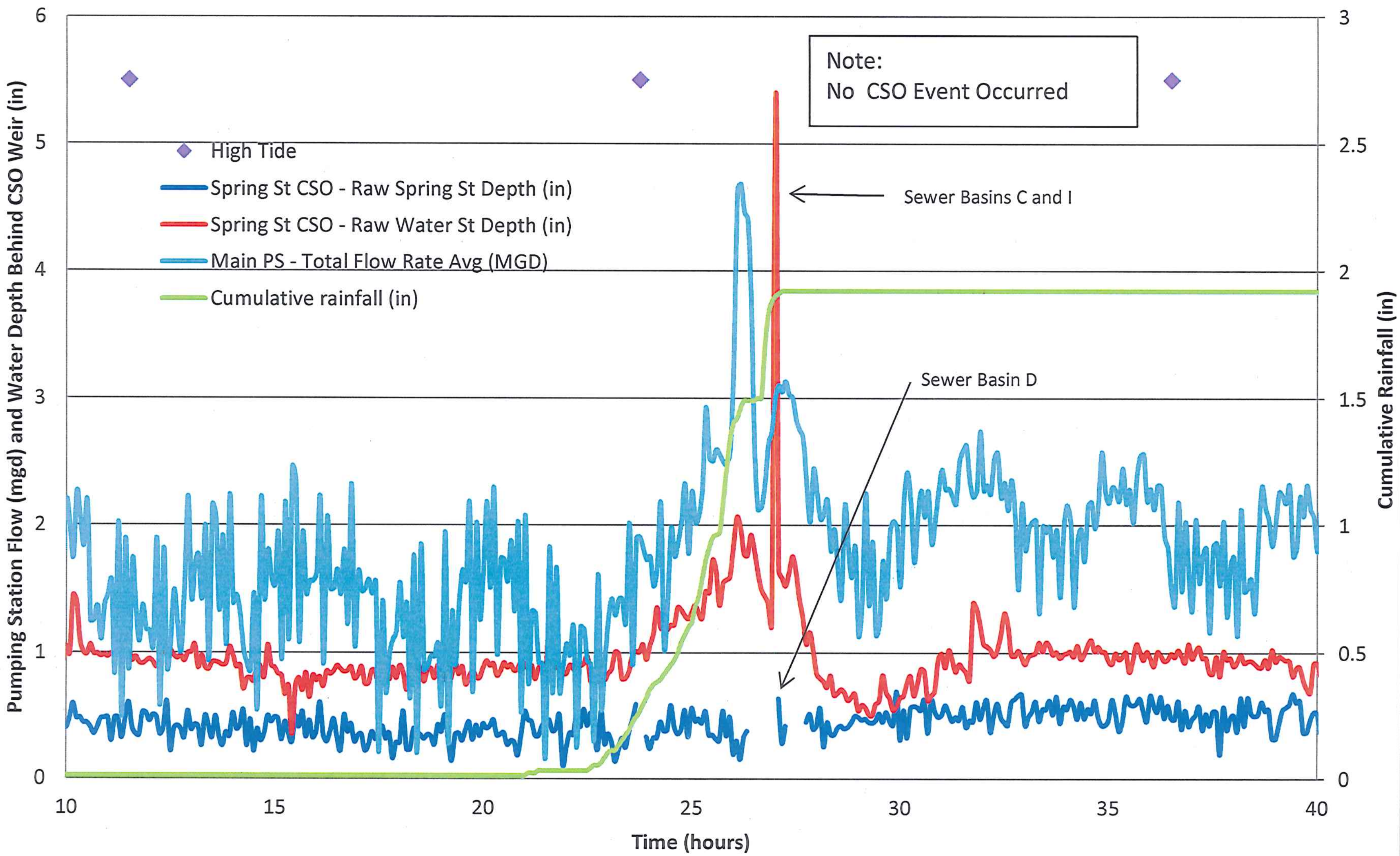


Figure 6-2
August 19, 2011 Storm Event
Main Pumping Station and CSO Flow (mgd)
Cumulative Rainfall (in)
Time (hours)
Spring St CSO - CSO Flow Rate (MGD)
Main PS - Total Flow Rate Avg (MGD)
High Tide
Cumulative rainfall (in)
Water St CSO - CSO Flow Rate (MGD)
0
1
2
3
4
5
6
7
8
0
0.2
0.4
0.6
0.8
1
1.2
1.4
1.6
1.8
10
15
20
25
30
35
40

Figure 6-3
August 10, 2011 Storm Event



6.4.3.2 CSO Events – March 2011

During the March 7, 2011 storm event, approximately 1.38" of rain fell over approximately 18 hours (plus snowmelt) which resulted in a total 14 mgd peak-hourly wastewater flow spike during the rain. Figure 6-4 shows hourly averages with an approximate 13 mgd peak-hour flow (7+5+1). The main pumping station pumped at a rate around 7 mgd for approximately 15 hours (4,400,000 gallons), the Spring St. CSO discharged a peak 5-minute flow of 5.5 mgd (2,200,000 gallons over 14 hours), and the Water St. CSO discharged a peak 5-minute flow of 1.5 mgd (400,000 gallons over 12 hours). In addition, main pumping station baseline flows continued at an elevated rate of 3 to 5 mgd for several days following the CSO event (assumed 12,000,000 million gallons) leading into the March 11, 2011 CSO event discussed below. A summary of the approximate flows and volumes during the March 7, 2011 storm/CSO event is as follows:

- Total rain = 1.38" (plus snowmelt)
- Peak 5-minute flow = 14 mgd
- Peak hourly flow = 13 mgd
- CSO volume = 2.6 million gallons (0.6 million gallons based on 15 ac. direct inflow)
- Total 3-day I/I volume = 19 million gallons (less approx. 1 mgd sanitary flow)

During the March 11, 2011 storm event, approximately 0.99" of rain fell over approximately 19 hours (plus snowmelt) which resulted in a total 12 mgd peak 5-minute wastewater flow spike during the rain. Figure 6-5 shows hourly averages with an approximate 11.5 mgd peak-hour flow. The main pumping station pumped at a rate around 7 mgd for approximately 13 hours (3,800,000 gallons), the Spring St. CSO discharged a peak 5-minute flow around 4 mgd (550,000 gallons over 10 hours), and the Water St. CSO discharged a peak 5-minute flow of around 1 mgd (70,000 gallons over 8 hours). In addition, main pumping station baseline flows continued at an elevated rate of 3 to 5 mgd for about a week following the CSO event (assumed 28,000,000 gallons). A summary of the approximate flows and volumes during the March 11, 2011 storm/CSO event is as follows:

- Total rain = 0.99" (plus snowmelt)
- Peak 5-minute flow = 12 mgd
- Peak hourly flow = 11.5 mgd
- CSO volume = 0.62 million gallons (0.4 million gallons based on 15 ac. direct inflow)
- Total 7-day I/I volume = 32 million gallons (less approx. 1 mgd sanitary flow)

In addition, Figure 6-5 illustrates that main pumping station flow and cumulative rainfall curves are almost mirror images of eachother in the from t=20 to t=35 immediately preceding the CSO event, providing further evidence that direct inflow is still a major contributor to the system resulting in CSO events. From these events we can conclude:

- Direct inflow sources exist due to the response of the CSO to rain
- Peak flows have been observed to be approximately 14 MGD three times in the last 2 years

Figure 6-4
March 7, 2011 Storm Event

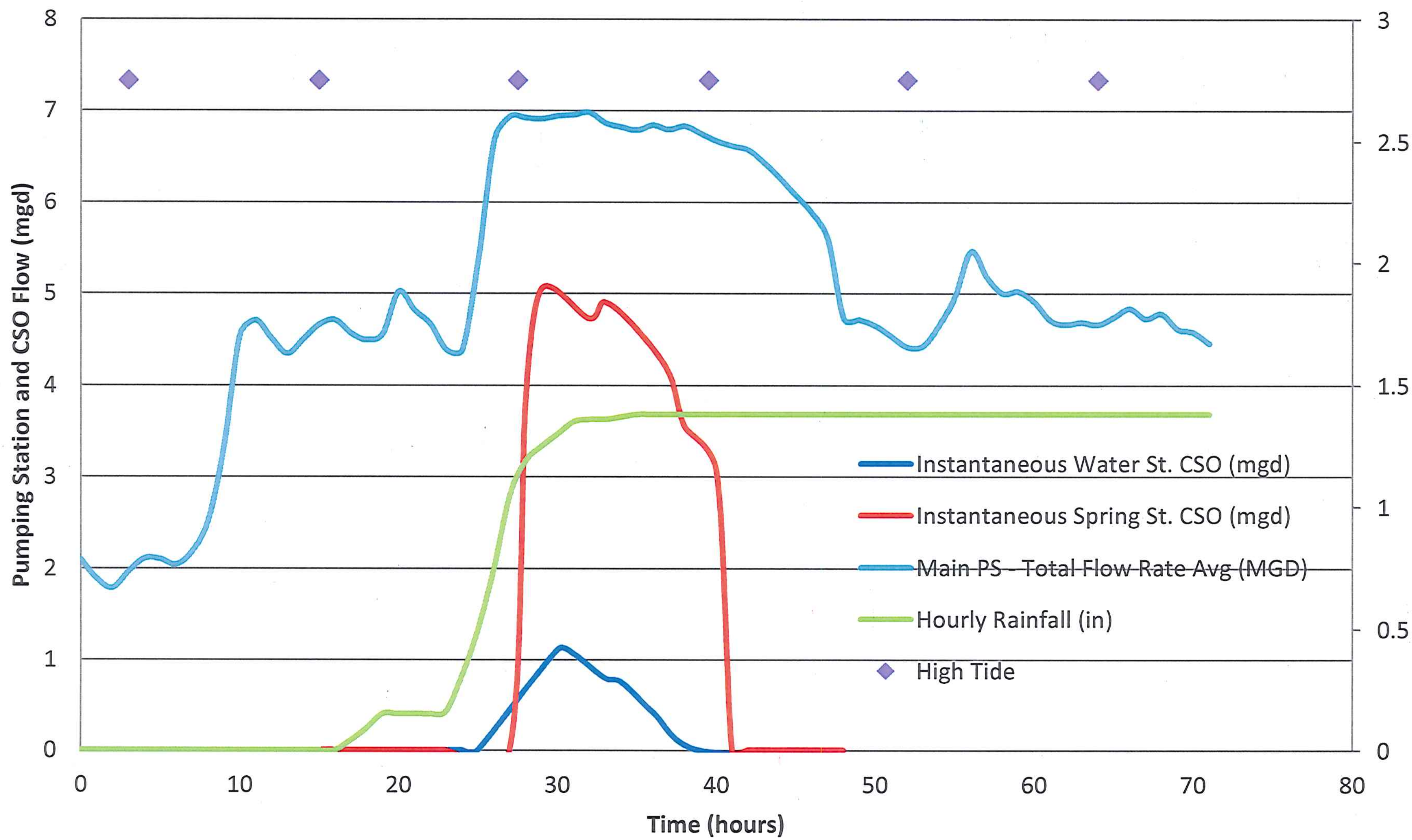
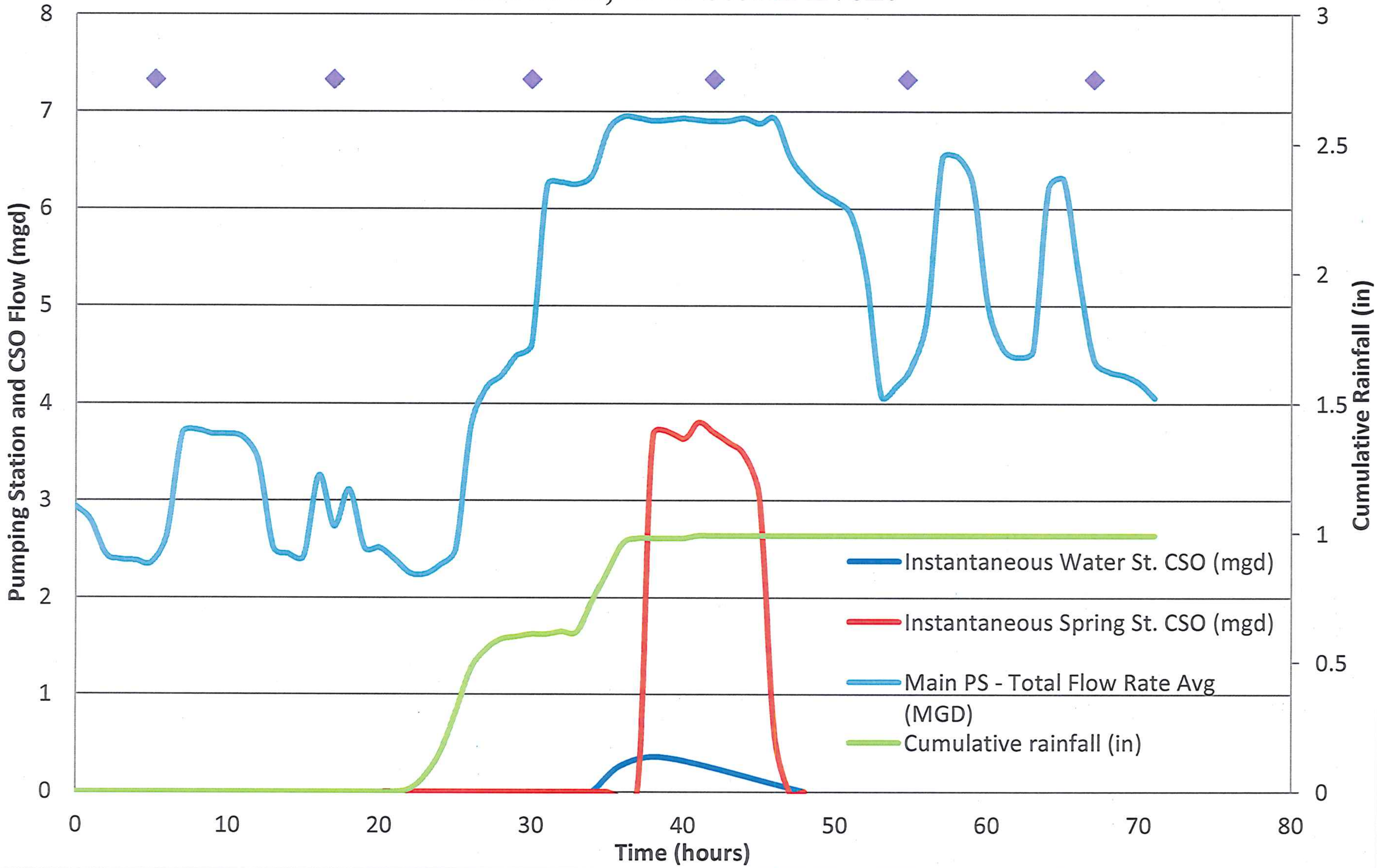


Figure 6-5
March 11, 2011 Storm Event



6.5 Estimated Flows and Loads

6.5.1 Infiltration and Inflow - Current

For the purposes of estimating a total I/I value to the system, we have summarized the infiltration (mainline, MH and service flow) identified during flow isolation, and potential peak inflow from identified or suspected sources. For the purposes of tabulating inflow, average values were used, based on ranges previously discussed.

Potential peak inflow from known/suspected direct inflow • 15 acres @ approx 3.71 inches/day	1,500,000 gpd
Potential peak inflow from sump pumps (25% of users) • 750 users @ 4 gpm	4,300,000 gpd
Infiltration identified during flow isolation (75% of system)	750,000 gpd
Total estimated peak daily I/I	6,550,000 gpd

The calculations shown above simply present an example of where high flows during rain events may be generated. In addition, as illustrated in previous sections of this report, high intensity rainfall events contribute significant peak flows to the system due to direct and delayed inflow. For example, during the August 19, 2011 CSO event 14 mgd peak 5-minute flows and 12 mgd peak hour flows were measured at a time when groundwater and baseline I/I levels were low. If a similar high intensity storm occurred during high groundwater conditions when sump pumps are running, the example above shows that peak hour flows in the system could exceed 16 mgd (12 mgd+4.3 mgd) similar to the lower intensity storm of March 7, 2011 that resulted in 14 mgd peak discharges.

Total flows from the system are well above the 6.5 MGD accounted for here, indicating that there are more sources of I/I in the system than have been identified to date. The Main Pump Station has been recorded pumping 7 mgd, in addition to CSO flows.

As evident from these approximations, private inflow sources such as sump pumps and roof drains can have a significant impact on sewer flows. With 48" of average annual precipitation, the estimated that 15 acres of impervious area contributing to inflow can contribute to 20 million gallons of extraneous flow to the system alone. In addition, the private inflow sources contribute to significant peak flows, especially during high intensity rain events leading to CSO events. As such, these types of sources should be a priority for the town to locate and remove.

7. COLLECTION SYSTEM HYDRAULIC EVALUATION

7.1 Collection System

Hydraulic evaluation of the collection system was performed by CDM and outside the scope of this report.

7.2 Hydraulic Capacity of Interceptors

UEI reviewed the hydraulic capacity of the sewer interceptors downstream of the Water St. and Spring St. diversion structures and compared that to the capacity and future dry weather flows reported in Appendix D of CDM's *Phase I Infiltration/Inflow Study* (Appendix Volume VII). The invert elevations of the sewer interceptors, diversion structure weir elevation and Clemson Pond outlet structure (CSO 003) was surveyed and pipe roughness coefficients (Manning's "n") were assumed to evaluate the interceptors as part of this study.

The hydraulics of the sewer interceptors that convey wastewater to the Main Pumping Station is dynamic depending on the flows from the various sewer basins and the wastewater level in the Main Pumping Station wetwell. There are three main interceptors that contribute flow to the Main Pumping Station (Figure 7-1):

- Downstream of the Water Street Diversion Structure (Basin A, Small portion of Basin B)
- Downstream of the Spring Street Diversion Structure (Basins C, D, E, F G, I)
- Jady Hill Inverted Siphon (Basin H, Most of Basin B)

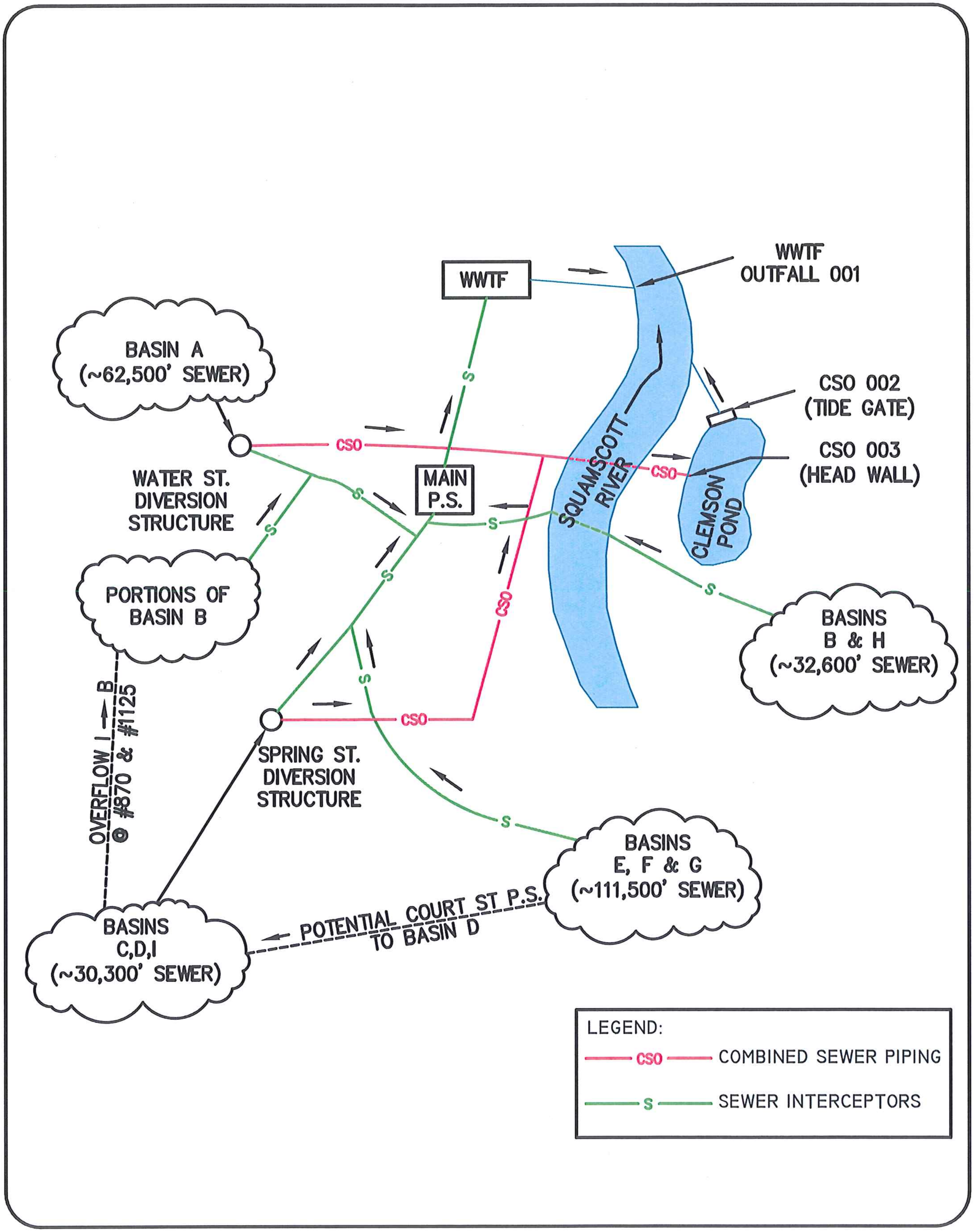
7.2.1 Sewer Interceptor Full Pipe Capacity

Generally the full pipe capacities evaluated based on the survey performed by UEI were consistent with the full pipe capacity and future dry weather flows identified in Appendix D of *Phase I Infiltration/Inflow Study* (Appendix Volume VII) with the following exceptions:

- SMH 937-938 (Beneath Exeter Housing Authority Building): UEI found that this reach was actually 2-18" pipes (instead of 1-18" pipe) which would have sufficient capacity to convey the CDM identified 4.4 MGD Future Dry Weather Flow. However, the Town plans to re-route this interceptor from beneath the Exeter Housing Authority building to improve maintenance issues.
- SMH 899-900 (Immediately Downstream of Spring Street Diversion Structure): UEI found that this reach was actually a 12" pipe (instead of 18" pipe) with a 3 MGD capacity. It should be noted that, although the actual pipe capacity is much less than

assumed by CDM, but it has sufficient capacity to convey the CDM identified 1.1 MGD Future Dry Weather flow.

It should be noted that the full pipe capacity alone is not sufficient to completely evaluate the sewer interceptor capacity because the sewer interceptor hydraulics is affected by the tailwater effects of the downstream interceptors and wetwell level as discussed in later sections of this report.



Exeter Sewer System Schematic

I&I Evaluation Report
Exeter, New Hampshire

UNDERWOOD engineers
25 Vaughan Mall, Portsmouth, N.H. 03801
Tel. 603-436-6192 Fax. 603-431-4733

DATE 7/30/10

FIGURE 7-1

7.2.2 Sewer Interceptor Hydraulic Grade Lines

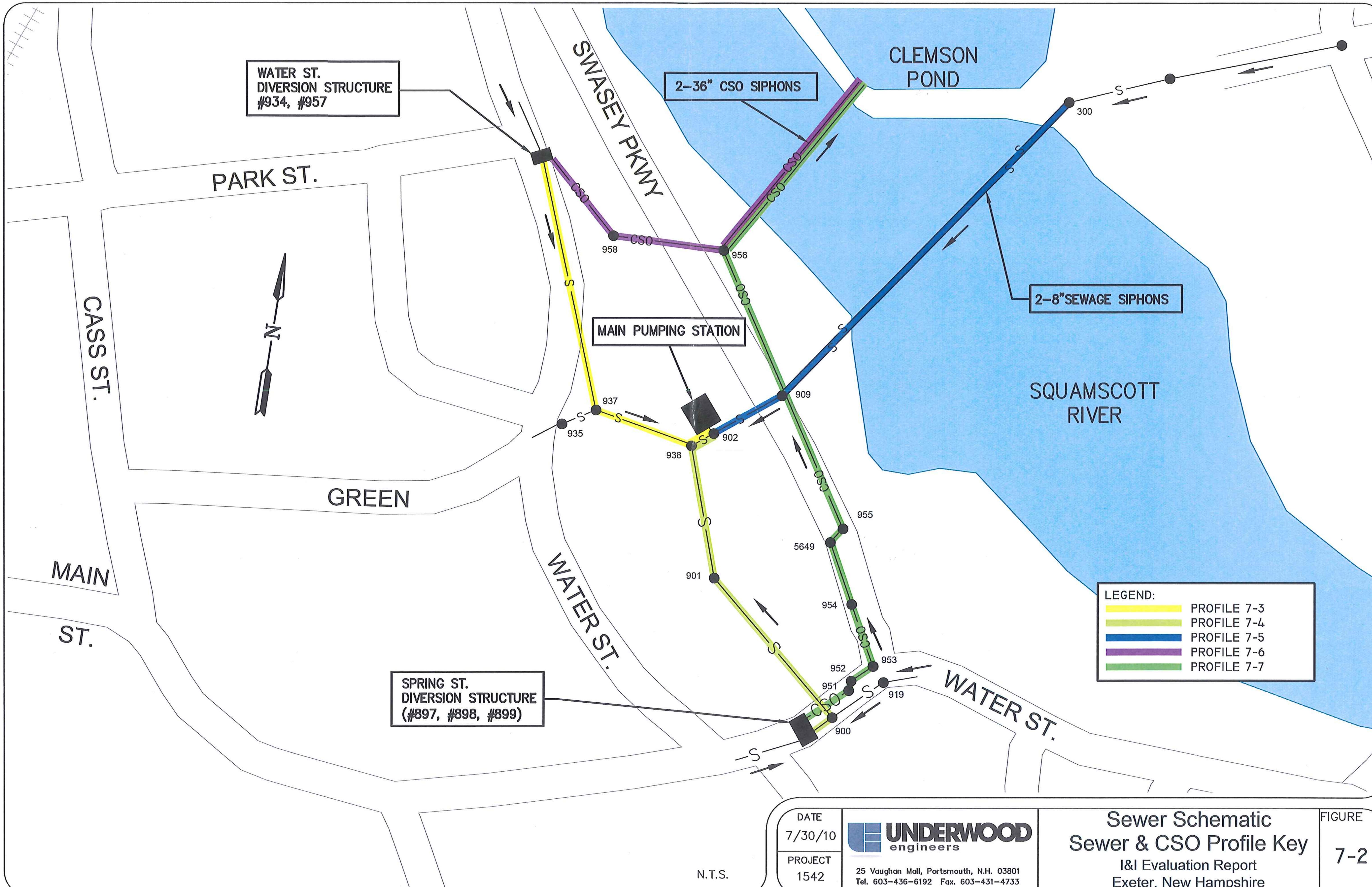
Figure 7-1 illustrates an overview schematic of the wastewater system. It should be noted that sewer flow from Basins B, F, G, and H are not conveyed through either the Spring Street or Water Street Diversion Structure. Wastewater from sewer Basins F and G are conveyed into the interceptor downstream of the Spring Street Diversion Structure and Basin B is conveyed downstream of the Water Street Diversion Structure (as well as through the Jady Hill inverted siphon). Basin H is conveyed through the inverted sewer siphon beneath the Squamscott River from Jady Hill and enters downstream of both diversion structures in the last manhole before the Main Pumping Station. Although a large portion of the wastewater system is not conveyed directly through the diversion structures, the hydraulic "tailwater" effects of the confluence of sewer interceptor flows downstream of the diversion structure affects sewer capacity and impacts CSO occurrences.

Figures 7-2 through 7-5 show the estimated hydraulic grade line based on UEI field observations during a March 30, 2010 CSO event. During that event the Main Pumping Station was observed to be pumping at a rate of approximately 6.6 MGD. In addition, the Main Pumping Station wetwell level was observed to be several feet above the wetwell grate floor and was estimated to be at elevation 4.5'+/- and the wastewater elevation over the Water Street and Spring Street Diversion weirs was measured at approximately 5.6 feet and 6.4 feet, respectively and UEI observed free flow CSO flow over the Water St. and Spring St. diversion weirs at that time. It should be noted that the wastewater elevation on the upstream side of the Jady Hill inverted siphon was not measured and was estimated from visual observations. Town personnel were observed pumping from the upstream side of the inverted siphon into Clemson Pond to lower water levels to prevent sewer service back-ups upstream of the inverted siphon.

The wetwell level has a significant impact on the hydraulic grade line of the sewer interceptors. This is compounded by the presence of the CSO diversion weirs which regulate or essentially "fix" the upstream head to convey wastewater to the Main Pumping Station from the diversion structures. This means that as the wetwell level rises, presumably during high flow events, there is less head to convey wastewater to the main pumping station through the interceptors without surcharging the weirs. Based on assumed Manning's coefficient for the interceptor the following is an example of how the wetwell level can affect interceptor capacity:

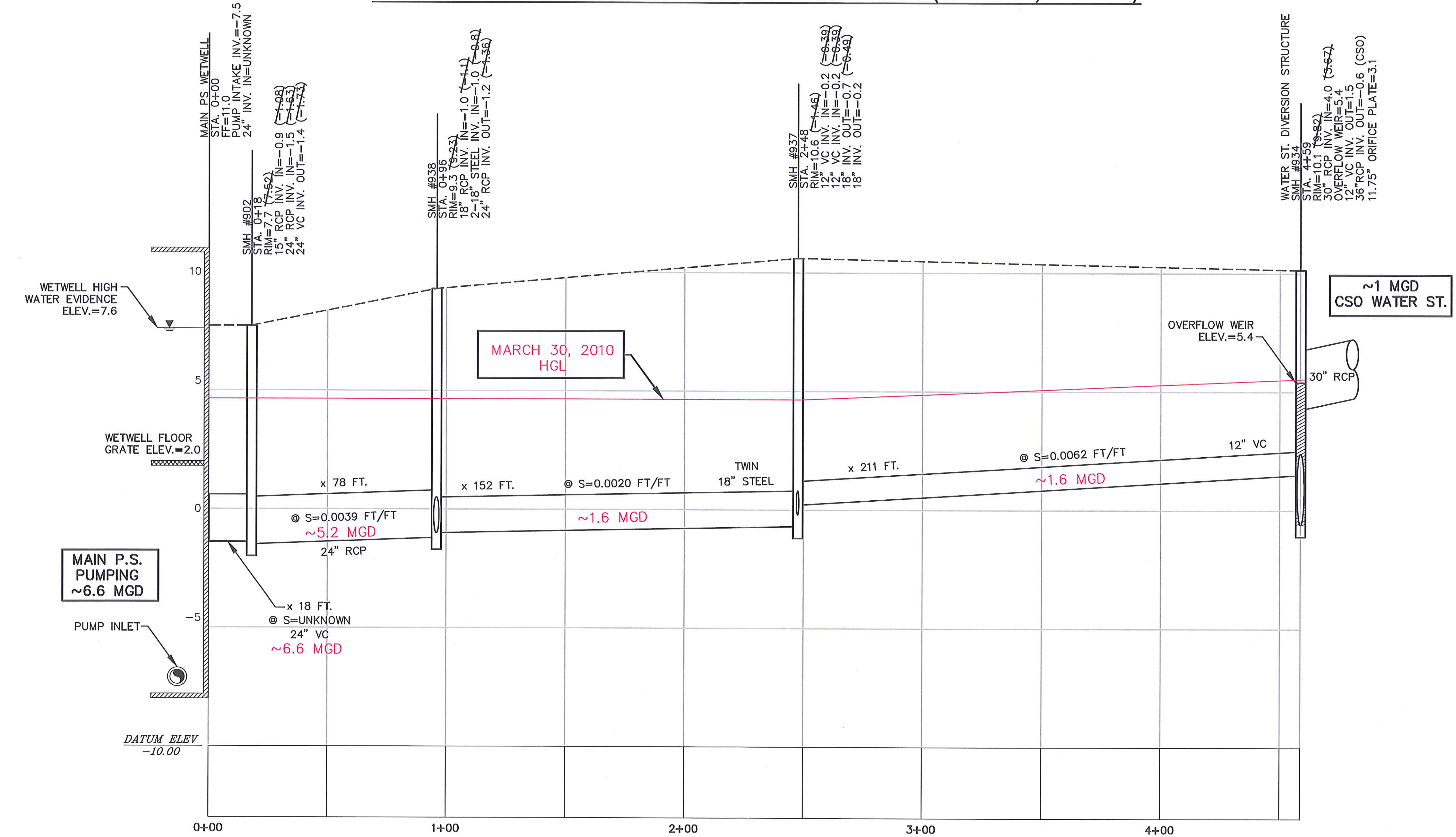
<u>Jady Hill Inverted Siphon Example</u>

- Free Flowing/No Wetwell or Manhole Surcharging (Assumed Elevation @ SMH #300 = 5.9') — Capacity = ~1.7 mgd
- Wetwell @ 2' (el), no Upstream Manhole Surcharging (Assumed Elevation @ SMH #300 = 5.9') — Capacity = ~1.3 mgd
- Wetwell @ 5' (el), no Upstream Manhole Surcharging (Assumed Elevation @ SMH #300 = 5.9') — Capacity = ~0.7 mgd



H:\SDSKPROJ\1542\1542LDD\dwg\1542_map.dwg, fig 7-2, 11/12/2012 2:59:10 PM, rla

MAIN PUMP STATION TO WATER ST. DIVERSION STRUCTURE (MARCH 30, 2010 HGL)



NOTES:

1. ELEVATIONS SHOWN BASED ON SURVEY PERFORMED BY DOUCET SURVEY IN MAY 2009.
2. ELEVATIONS IN PARENTHESES () FROM EXETER GIS SYSTEM.
3. MANHOLE DIAMETERS AND DIMENSIONS ARE NOT KNOWN. PIPE SLOPES SHOWN ARE BASED ON MEASUREMENTS TO THE CENTER OF THE MANHOLE.
4. WETWELL HIGH WATER ELEVATION ESTIMATED FROM EVIDENCE OBSERVED BY UEI IN THE MAIN PUMP STATION WETWELL IN APRIL 2009

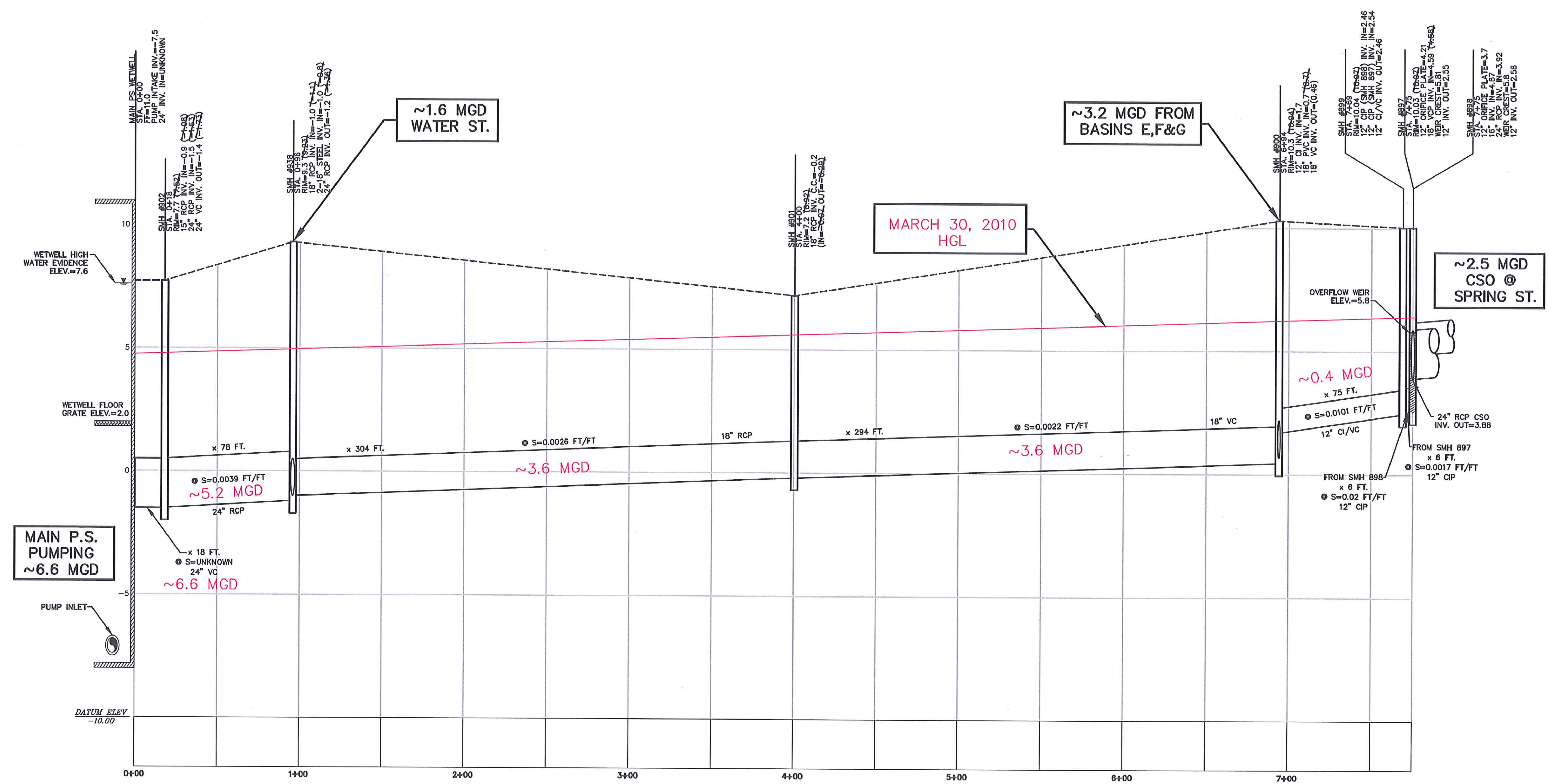
40 0 40 80
4 0 4 8

HORIZ: 1 INCH = 40 FT.
VERT: 1 INCH = 4 FT.

DATE	UNDERWOOD engineers	Sewer Profile	FIGURE
7/30/10	25 Vaughan Mall, Portsmouth, N.H. 03801	Water St. March 30, 2010 HGL	7-3
PROJECT	Tel. 603-436-6192 Fax. 603-431-4733	I&I Evaluation Report	
1542		Exeter, New Hampshire	

H:\SDSKPROJ\1542\1542LDD\dwg\1542base.dwg, fig 7-3, 11/12/2012 3:02:48 PM, rla
11x17fig.DWG

MAIN PUMP STATION TO SPRING ST. DIVERSION STRUCTURE (MARCH 30, 2010)



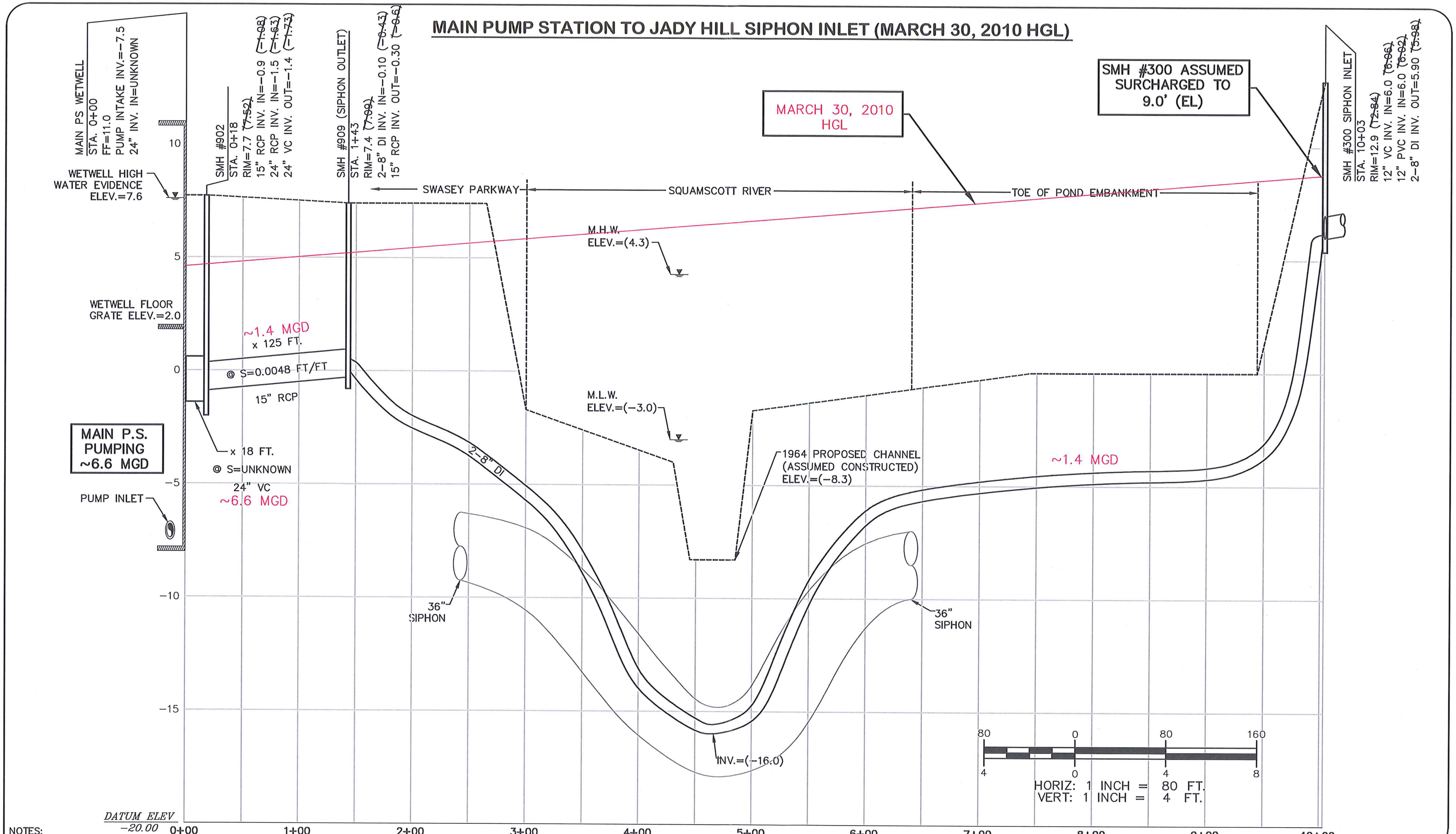
NOTES:
1. ELEVATIONS SHOWN BASED ON SURVEY PERFORMED BY DOUCET SURVEY IN MAY 2009.
2. ELEVATIONS IN PARENTHESES () FROM EXETER GIS SYSTEM.
3. MANHOLE DIAMETERS AND DIMENSIONS ARE NOT KNOWN. PIPE SLOPES SHOWN ARE BASED ON MEASUREMENTS TO THE CENTER OF THE MANHOLE.
4. WETWELL HIGH WATER ELEVATION ESTIMATED FROM EVIDENCE OBSERVED BY UEI IN THE MAIN PUMP STATION WETWELL IN APRIL 2009.
5. ELEVATIONS SHOWN FOR SMH #900 FROM UEI'S "EPPING ROAD WATER MAIN AND ROAD IMPROVEMENTS" 2007 PLANS.

60 0 60 120
4 0 4 8
HORIZ: 1 INCH = 60 FT.
VERT: 1 INCH = 4 FT.

DATE		Sewer Profile	FIGURE
7/30/10	UNDERWOOD engineers	Spring St. March 30, 2010 HGL	7-4
PROJECT	25 Vaughan Mall, Portsmouth, N.H. 03801	I&I Evaluation Report	
1542	Tel. 603-436-6192 Fax. 603-431-4733	Exeter, New Hampshire	

H:\SDSKPROJ\1542\1542LDD\dwg\1542base.dwg, fig 7-4, 11/12/2012 3:03:21 PM, rla

MAIN PUMP STATION TO JADY HILL SIPHON INLET (MARCH 30, 2010 HGL)



NOTES:

1. ELEVATIONS SHOWN BASED ON SURVEY PERFORMED BY DOUCET SURVEY.
2. ELEVATIONS IN PARENTHESES () FOR SIPHON STRUCTURES FROM WHITMAN AND HOWARD "INVERTED SEWER SIPHONS" RECORD PLAN DATED SEPTEMBER 1964. OTHER ELEVATIONS IN PARENTHESES () FROM EXETER GIS.
3. MANHOLE DIAMETERS AND DIMENSIONS ARE NOT KNOWN. PIPE SLOPES SHOWN ARE BASED ON MEASUREMENTS TO THE CENTER OF THE MANHOLE.
4. WETWELL HIGH WATER ELEVATION ESTIMATED FROM EVIDENCE OBSERVED BY UEI IN THE MAIN PUMP STATION WETWELL IN APRIL 2009.
5. MEAN HIGH AND LOW TIDE WATER ELEVATION FROM WHITMAN AND HOWARD, INC. "INVERTED SEWER SIPHON" RECORD PLANS SEPT. 1964 REPORTED AS 3.3' AND −3.3' RESPECTIVELY.
6. MEAN HIGH AND LOW TIDE WATER LEVELS SHOWN ARE FROM UEI, "WWTP OUTFALL IMPROVEMENTS" RECORD DRAWINGS 2/2002 AND ARE FROM A LOCATION APPROXIMATELY 1 MILE DOWNSTREAM.

DATE			FIGURE
7/30/10	UNDERWOOD engineers	Sewer Profile Jady Hill March 30, 2010 HGL I&I Evaluation Report Exeter, New Hampshire	7-5
PROJECT 1542	25 Vaughan Mall, Portsmouth, N.H. 03801 Tel. 603-436-6192 Fax. 603-431-4733		

H:\SDSKPROJ\1542\1542LDD\dwg\1542base.dwg, FIG 7-5, 11/12/2012 3:03:54 PM, rla

As shown above, Main Pumping Station wetwell surcharging contributes to the Town's historical overflow issues in the vicinity of the inlet to the Jady Hill Inverted Siphon.

Spring Street Sewer Interceptor

The hydraulics of the sewer interceptor downstream of the Spring Street diversion structure is also sensitive to the wetwell level. Observations during the March 30, 2010 CSO event and inferred HGL shows that high wetwell levels and tailwater effects from the interceptor entering downstream of the Spring Street diversion structure at SMH 900, increases the HGL causing more flow over the Spring Street CSO weir.

As shown on Figure 7-4 it is estimated that only 0.4 MGD sewer flow was conveyed through the sewer side of the Spring Street diversion structure during the March 30, 2010 event. The rest of the flow entering the Spring Street diversion structure (2.5 MGD estimated CSO flow rate) was passed over the CSO weir to Clemson Pond. It is estimated that 3.2 MGD flow was entering the Spring Street Interceptor downstream of the diversion structure. It should be noted that the "sewer side" flows and the HGL illustrated on Figures 7-3 through 7-5 were based on limited water level observation locations and no direct flow measurements were made. The HGL at SMH #900 was not observed, but if tailwater effects raised the HGL at SMH #900 above the elevation of the Spring St. diversion weir (el. 5.8') there is the potential that sewage actually flowed backwards into the Spring St. diversion structure from the downstream side and over the Spring St. diversion weir.

The 18" PVC sewer (Basin E, F&G Interceptor) entering the Spring Street interceptor downstream of the diversion structure has higher capacity than the interceptor downstream of SMH 900. This means that increased flow from the Basin E, F&G Interceptor results in only a slight rise in HGL upstream of SMH 900 (up the Basin F&G Interceptor), but causes a larger increase in the Spring Street Interceptor HGL from SMH 900 to 938.

In summary, based on UEI's observations during the March 30, 2010 CSO event it appears that the Main Pumping Station could not keep up with the sewer flows entering the system. The wetwell level increased raising the sewer interceptor HGL until a CSO event occurred.

In addition, UEI observed 7.6' el. high water evidence in the Main Pumping Station wetwell during a April 2009 site visit as shown on Figures 7-3 through 7-5. It is not known when this high level occurred. However, it must have occurred during a CSO event since this elevation is approximately 2 feet higher than the elevation of the diversion structure weirs. Also, this wetwell elevation is higher than the elevation of the rim of SMH #901 located on the Spring St. interceptor, so it is likely that a SSO also occurred at SMH #901 at that time.

7.3 CSO and Diversion Structure Hydraulic Profile

The hydraulic profile of the CSO piping downstream of the Spring Street and Water Street diversion structures is also a dynamic system due to the direct hydraulic connection to Clemson

Pond. This means that the hydraulic capacity of the CSO piping depends on the water level in Clemson Pond. CSO flow that passes over the Water Street and Spring Street overflow weirs are conveyed through separate piping until they combine at sewer manhole SMH#956 and conveyed through 2-36" inverted siphons to Clemson Pond.

Figures 7-6 & 7-7 show the hydraulic grade line for the Spring Street and Water Street CSO piping using an assumed CSO flow of 10 mgd over each of the diversion structure and assumed industry standard Manning's roughness coefficients for the CSO piping. 10 MGD CSO flow was assumed based on CSO chart records provided by the Town for a March 15, 2010 CSO event.

It was found that the Spring Street CSO hydraulic grade line is more influenced by the stage of Clemson Pond than the Water Street CSO. For the Spring Street CSO piping and 10 MGD CSO flows, Clemson Pond stages higher than El. 3.0' can create tailwater effects at the Spring Street CSO diversion weir. For the Water Street CSO piping and 10 MGD CSO flows, Clemson Pond stages higher than El. 4.9' can create tailwater effects at the Water Street CSO diversion weir. The stage of Clemson Pond is not monitored, so it is unknown whether Clemson Pond stage has impacted CSO capacity in the past.

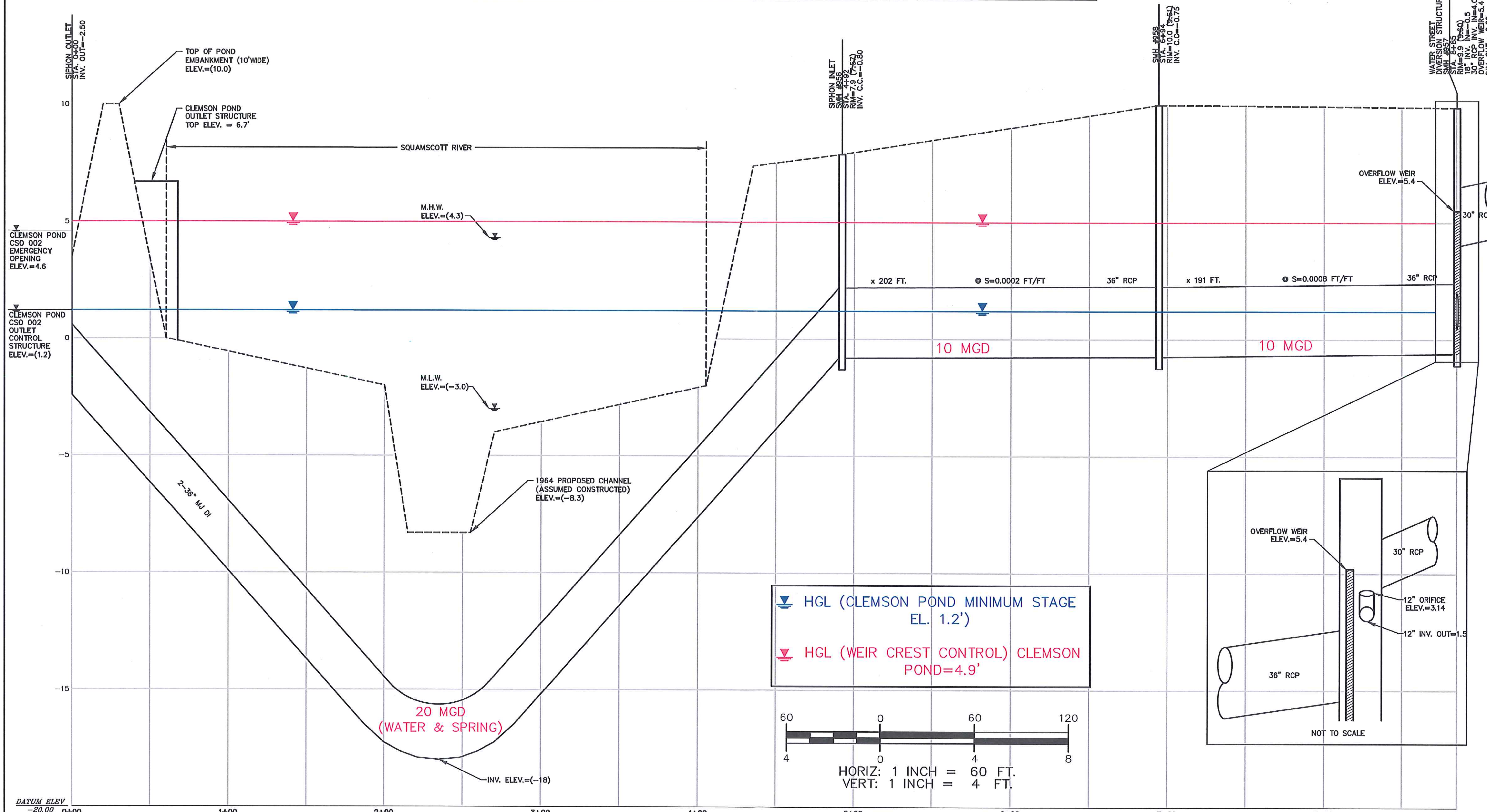
7.3.1 Clemson Pond High Water Evidence

Since CSO capacity is linked to the water level in Clemson Pond, UEI personnel looked for evidence of previous high water levels in Clemson Pond. It was found that Clemson Pond water levels were likely at or above the elevation of the emergency overflow invert on the Clemson Pond outlet structure (Invert Elevation ~4.6') based on debris observed in vegetation. It was found that CSO piping downstream of the diversion structures have the following approximate capacity with Clemson Pond 4.6 stage elevation without backing up water levels above the diversion structure weirs:

- Spring Street CSO piping capacity (Clemson Pond @ 4.6' el.) = 6 MGD
- Water Street CSO piping capacity (Clemson Pond @ 4.6' el.) = 16 MGD

Therefore there is significant CSO piping capacity downstream of the diversion structures even when Clemson Pond is at emergency overflow stage assuming CSO piping has been maintained and is clear of debris.

CLEMSON POND TO WATER ST. DIVERSION STRUCTURE (EXISTING CSO)



NOTES:

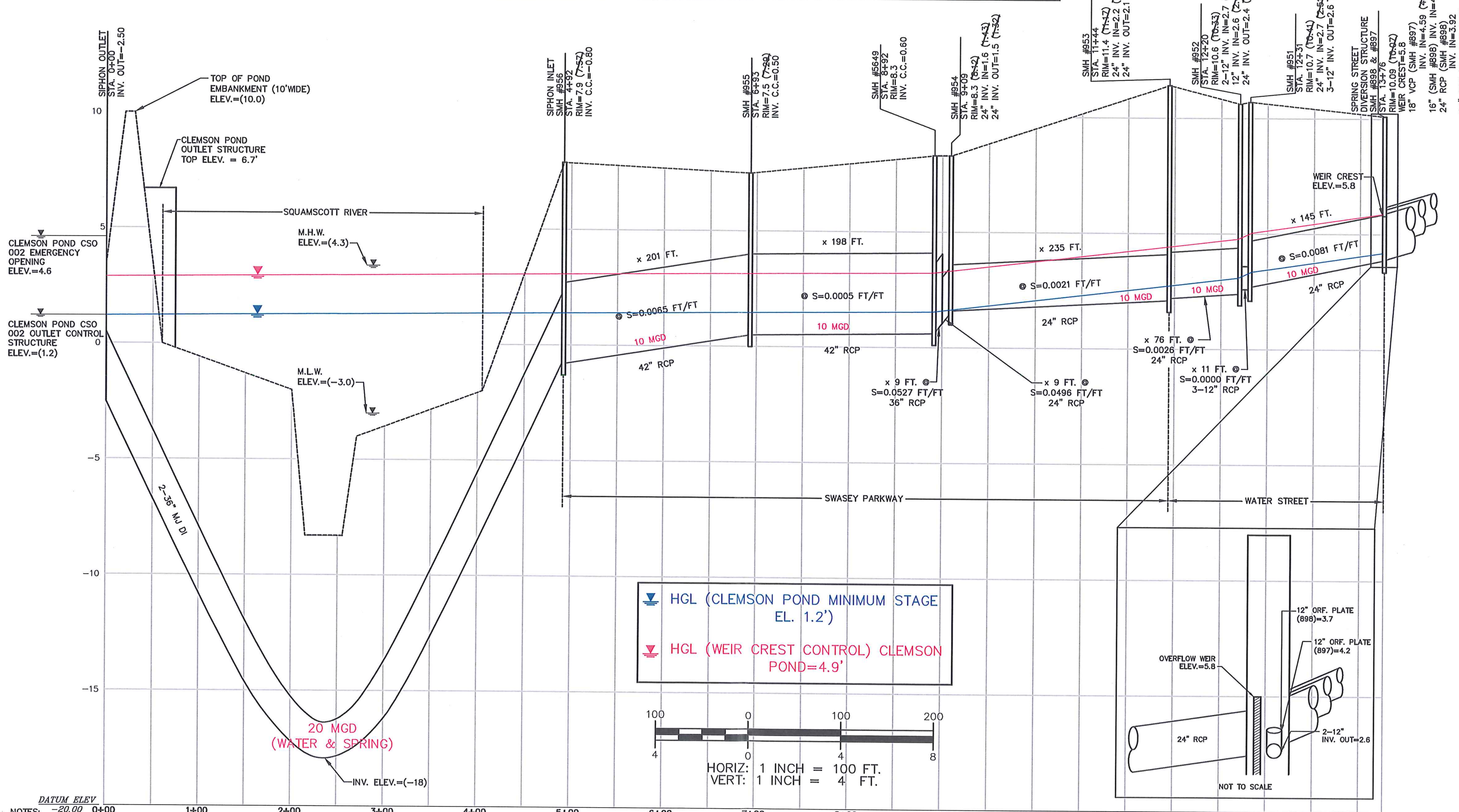
1. ELEVATIONS SHOWN BASED ON SURVEY PERFORMED BY DOUCET SURVEY.
2. ELEVATIONS IN PARENTHESES () FOR SIPHON STRUCTURES FROM WHITMAN AND HOWARD "INVERTED SEWER SIPHONS" RECORD PLAN DATED SEPTEMBER 1964. OTHER ELEVATIONS IN PARENTHESES () FROM EXETER GIS.
3. MANHOLE DIAMETERS AND DIMENSIONS ARE NOT KNOWN. PIPE SLOPES SHOWN ARE BASED ON MEASUREMENTS TO THE CENTER OF THE MANHOLE.
4. MEAN HIGH AND LOW TIDE WATER ELEVATION FROM WHITMAN AND HOWARD, INC. "INVERTED SEWER SIPHON" RECORD PLANS SEPT. 1964 REPORTED AS 3.3' AND -3.3 RESPECTIVELY.
5. 10 MGD FLOW ASSUMED BASED ON 3/15/10 CSO CHART RECORDS PROVIDED BY TOWN.
6. MEAN HIGH AND LOW TIDE WATER LEVELS SHOWN ARE FROM UEI, "WWTP OUTFALL IMPROVEMENTS" RECORD DRAWINGS 2/2002 AND ARE FROM A LOCATION APPROXIMATELY 1 MILE DOWNSTREAM.

DATE	UNDERWOOD engineers	Water St. CSO HGL	FIGURE
7/30/10	25 Vaughan Mall, Portsmouth, N.H. 03801	Assumed 10 MGD CSO Flow	7-6
PROJECT	Tel. 603-436-6192 Fax. 603-431-4733	I&I Evaluation Report	
1542		Exeter, New Hampshire	

H:\SDSKPROJ\1542\1542LDD\dwg\1542base.dwg, FIG 7-6, 11/12/2012 3:05:12 PM, rla

11x17fig.DWG

CLEMSON POND TO SPRING ST. DIVERSION STRUCTURE (EXISTING CSO)



NOTES:
1. ELEVATIONS SHOWN BASED ON SURVEY PERFORMED BY DOUCET SURVEY
2. ELEVATIONS SHOWN FOR SMH #953 TO SMH #951 FROM UEI'S "EPPING ROAD WATER MAIN AND ROAD IMPROVEMENTS" 2007 PLANS.
3. ELEVATIONS IN PARENTHESES () FOR SIPHON STRUCTURES FROM WHITMAN AND HOWARD "INVERTED SEWER SIPHONS" RECORD PLAN DATED SEPTEMBER 1964. OTHER ELEVATIONS IN PARENTHESES () FROM EXETER GIS.
4. MANHOLE DIAMETERS AND DIMENSIONS ARE NOT KNOWN. PIPE SLOPES SHOWN ARE BASED ON MEASUREMENTS TO THE CENTER OF THE MANHOLE.
5. MEAN HIGH AND LOW WATER ELEVATION FROM WHITMAN AND HOWARD, INC. "INVERTED SEWER SIPHON" RECORD PLANS SEPT. 1964 REPORTED AS 3.3' AND – 3.3' RESPECTIVELY.
6. 10 MGD FLOW ASSUMED BASED ON 3/15/10 CSO CHART RECORDS PROVIDED BY TOWN.
7. MEAN HIGH AND LOW TIDE WATER LEVELS SHOWN ARE FROM UEI, "WWTP OUTFALL IMPROVEMENTS" RECORD DRAWINGS 2/2002 AND ARE FROM A LOCATION APPROXIMATELY 1 MILE DOWNSTREAM.

DATE	UNDERWOOD engineers	Spring St. CSO HGL Assumed 10 MGD CSO Flow	FIGURE
7/30/10			7-7
PROJECT	25 Vaughan Mall, Portsmouth, N.H. 03801 Tel. 603-436-6192 Fax. 603-431-4733	I&I Evaluation Report Exeter, New Hampshire	
1542			

H:\SDSKPROJ\1542\1542LDD\dwg\1542base.dwg, FIG 7-7, 11/12/2012 3:05:42 PM, rla

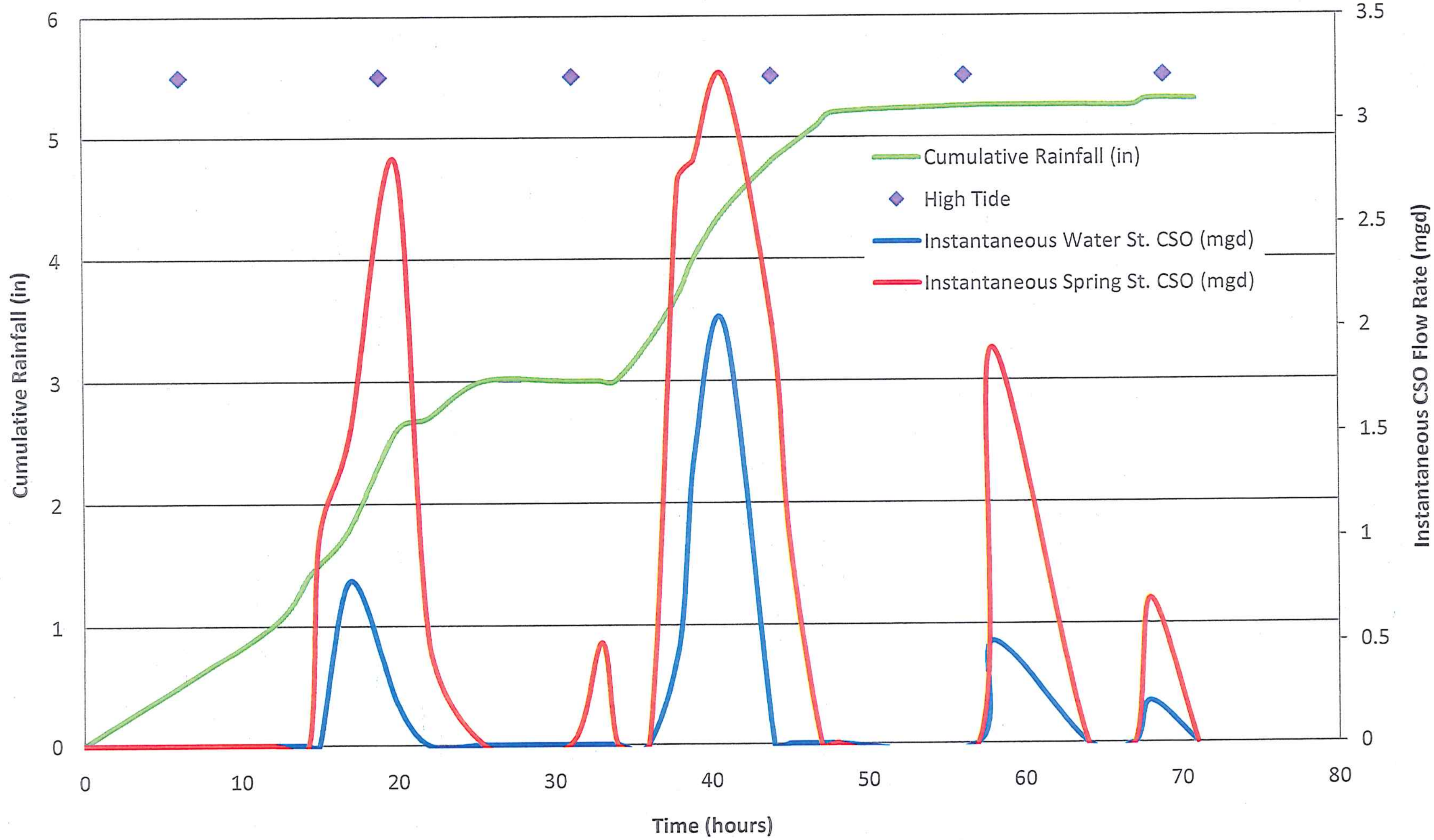
7.3.2 Potentially "False" Recorded CSO Events

While it does not appear to be a regular occurrence, it appears that the high water levels in Clemson Pond may have caused "false" CSO events during Exeter/Squamscott River flooding that occurred at the end of February 2010. If Clemson Pond levels rise above that of the diversion structure weir elevations, "false" CSO events can be recorded by the CSO ultrasonic meter due to high static water level in CSO piping. If this condition occurs during a CSO event, then overstated CSO flows may be recorded. To remedy this uncertainity, the Town installed additional metering in the diversion structures in December 2010 to monitor whether "false" CSO events occur in the future, but no significant flooding events have occurred since installation of the new metering.

Figure 7-8 shows cumulative rainfall, recorded instantaneous CSO flows, and Squamscott River high tides for the February 24-27, 2010 storm event. Four distinct CSO peaks were recorded around 20 hours, 40 hours, 60 hours, and 70 hours after rainfall began. The largest CSO peaks which occurred around t=20 hours and t=40 hours occurred during periods of the heaviest rainfall and is generally consistent with other observed CSO events. However, the peaks at t=60 and t=70 occurred at least 10 hours after the rainfall had ceased and approximately corresponded to Squamscott River high tide. It should be noted that the high tides at approximate t=6 hours and t=32 hours were the higher of the two tides in the 24-hour tide cycle and do not correspond with the largest CSO Peaks. However, based on the limited available information, it could be argued that all four CSO peaks observed during this storm event may be linked to high tides.

Unusually high tides and high Exeter River flows around t=60 hours and t=70 hours apparently caused that Squamscott River levels to rise to 8 feet (el.) based on Town personnel reports and observed flood debris in Swasey Parkway. While not directly observed, it is believed that Squamscott River floodwaters entered Clemson Pond at this time because flood waters can enter Clemson Pond when the Squamscott River stage rise above 6.7 feet (el.) due to the configuration of the Clemson Pond outlet structure. It is possible that approximately 5-10 million gallons of floodwaters may have entered Clemson Pond at this time which would correspond to an approximate 2 to 4 foot rise in the Clemson Pond stage and may have resulted in the "false" recorded CSO events at t=60 (15:00) and t=70 (01:00).

Figure 7-8
February 24-27, 2010 Storm Event
Rainfall , CSO and High Tide



7.4 Main Pumping Station Hydraulic Capacity

Except during CSO events, all sewage collected by the Town's wastewater collection system is conveyed to the WWTF by the Main Pumping Station. CDM's Phase I I/I Study indicated that the Main Pumping Station has a capacity of 5,500 gallons per minute (gpm) or 7.92 million gallons per day (mgd) in table 3-2.

Historical Main Pumping Station effluent flow, as measured at the WWTF influent weir, is not reliable due to system hydraulics, so confirmation of historical Main Pumping Station flows/pumping capacity is not possible. However, since strap-on Doppler flow meters were installed in February 2010, the highest daily Main Pumping Station flow as reported on the WWTF operations report for March 17, 2010 was 7.1 mgd. The March 17, 2010 highest daily pumping rate occurred during an extended CSO overflow event.

It should be noted that several other large CSO events occurred in the spring of 2010 after the Main Pumping Station flow meters were installed. In addition, it is understood that the Town installed an additional Main Pumping Station influent flow meter in the summer of 2011. However, it is understood that the recorded daily Main Pumping Station flow has not exceed 7.1 mgd since new metering was installed. It is recommended that the Town perform a pump test to evaluate the capacity of each pump the Main Pumping Station to evaluate why the station does not appear to be operating at its 7.9 MGD reported capacity. However, it should be noted that the existing 7 mgd capacity is consistent with industry standard peaking factors when compared to the existing approximate 2 mgd average daily WWTF flow.

Increased pumping capacity of the Main Pumping Station is one method to eliminate CSO events. If the Main Pumping Station pumping rate is increased to maintain a system HGL below the elevation of the CSO diversion weirs, then CSO events should not occur.

8. I/I REDUCTION STRATEGIES – GENERAL

8.1 Infiltration Reduction Strategies

Infiltration reduction strategies may include the following:

- Sewer main replacement
- Sewer main rehabilitation (sealing, lining)
- Manhole replacement
- Manhole rehabilitation (grouting, lining, frame re-setting)
- Service lateral replacement
- Service lateral rehabilitation (lining)

There are many different techniques and proprietary technologies for accomplishing the various types of replacement and rehabilitation. Some of the most common techniques are described in the following sections.

8.2 Sewer Collection System

8.2.1 Sewer Mains

This section covers options for reducing infiltration only in mainline sewers. Although infiltration can also be a result of service laterals, they will be discussed later in Section 8.3.

Although many techniques are available to install sewer mains, replacement is commonly accomplished by the following techniques:

- Open cut excavation (of entire reaches, or isolated sections)
- Pipebursting
- Directional Drilling or Jacking

Several options are available for rehabilitating leaking and deteriorated sewer mains. The most common methods include the following:

- Chemical grouting of mainline and/or service laterals (Test & Seal)
- Sliplining
- Fold and formed lining
- Cured-in-Place lining (complete line, "point repairs," or service laterals)

8.2.1.1 Open Cut Excavation and Replacement

Excavation and replacement of leaking and/or deteriorated pipe is typically undertaken only when more cost-effective trenchless technologies for replacement cannot be employed. Replacement should be considered for any of the following reasons:

- Severely deteriorated structural integrity
- Severely misaligned pipe
- Additional pipe capacity is needed
- Reduced capacity from trenchless technologies cannot be tolerated
- Reaches of pipeline are too severely damaged to be rehabilitated
- Severe sags
- Roads are being excavated for other reasons

Excavation and replacement using open cut techniques requires disturbance of pavement or other surface ground features and is often very disruptive in urban or densely developed areas. The existing pipe may be replaced with a new pipeline in the same alignment, or with a new pipeline in a parallel alignment.

Advantages of pipe replacement include the following:

- New structurally sound pipe with full life expectancy
- Ability to increase the size/carrying capacity of pipe
- New structurally sound and sealed service taps.

Disadvantages of pipe replacement include the following:

- Expense
- Traffic and urban disruption
- Possible interruption of other utilities
- Need to maintain service flows during construction
- Trench stabilization requirements
- Trench dewatering
- Surface disruption of nearby features such as landscaping, stone walls, etc.

8.2.1.2 Pipebursting

Pipebursting is a technique that was originally developed in the gas industry and has become more cost effective in recent years as the application of the technology has developed in the municipal market, and equipment has become available locally. Pipebursting consists of pulling a reaming head, attached to the new pipe, through the existing pipe to burst the pipe apart and compress the surrounding soil, while simultaneously placing the new pipe. Pipebursting replaces the existing pipe in the existing location, and may allow replacement with a larger size pipe than the existing pipe.

Advantages of pipebursting include the following:

- Limited risk of damage or interference with other underground utilities
- Requires minimal excavation or surface disturbance
- Less costly than excavation and replacement in congested areas
- New pipeline can slightly larger size.
- Provides a new structurally sound, sealed pipe with full service life
- Although larger diameter pipes may be installed, the magnitude of the size increase is dependent upon the specific conditions of the existing pipe and surrounding soil

Disadvantages of pipebursting include the following:

- Service flow needs to be bypassed
- Requires excavation of access pits at both ends of the pipe
- Straight sewer reaches are required
- Access pits need to be dug for each service connection.

8.2.1.3 Horizontal Directional Drilling/Jacking

Although directional drilling and pipe jacking or microtunneling are very different technologies, they both result in the installation of a new pipe, in a new location, by trenchless technologies.

Directional drilling has become a standard construction practice in recent years and may be used for the installation of pipes where surface disturbance is not acceptable. This includes crossings of wetlands or watercourses, highways or roads where excavations are not permitted, or other surface features such as stone walls, landscaping, walkways, where disturbance is not acceptable.

Directional drilling consists of drilling a pilot hole, from the drill rig to the termination point. The pipe is then pulled back through the hole. Directional drilling may be used to replace an existing pipe in a new location/alignment, or to replace a service. Due to difficulty in maintaining constant slope while drilling, this technology is better suited to shorter length gravity sewers. Two pits are often necessary for the installation of a gravity pipe.

Advantages of directional drilling include the following:

- Requires minimal excavation or surface disturbance
- Can be less costly than excavation and replacement in congested areas or where extensive surface features exist.
- Provides new structurally sound, sealed pipe with full service life
- Service flow does not need to be bypassed (new pipe in new location)
- Pipe diameter can be maintained, or enlarged if necessary

Disadvantages of directional drilling include the following:

- Proper soil conditions are required (no ledge or boulders)
- Requires excavation of access pits for a gravity pipe
- Difficult to maintain consistent pitch with no sags
- Excavations are required for the installation of service connections.

Where larger pipes are required, longer lengths are necessary, and/or consistent pitch is very important, pipe jacking may be used to install a sewer pipe inside a carrier pipe. Two pits are necessary for jacking, but greater grade control may be possible than with directional drilling.

8.2.1.4 Mainline Chemical Grouting (Test & Seal)

Before the development of other trenchless rehabilitation techniques, chemical grouting was the most common method to rehabilitate existing sewers without structural defects. Chemical grouting of sewer lines is primarily used to seal leaking joints, circumferential cracks, leaking service lateral connections and other small defects. Grouts are applied by pulling an inflatable packer through the sewer line, centering it on the crack, joint or defect, inflating the ends of the packer to seal it to the pipe, and injecting grout, under pressure, into the defect. The grout is forced into the defect and the surrounding soil outside the pipe. When a grouting project is undertaken, each joint in the sewer reach is air tested to identify potential leakage. Joints which fail the leakage test are subsequently grouted.

Grouting of service connections to the main line may be accomplished through the use of a special packer. Grouting of the first 3-5 ft of service laterals from the main may also be conducted.

Chemical grouts may be gel (acrylamide, acrylic, acrylate or urethane) or foam (polyurethane). Gels may be susceptible to dehydration and shrinkage cracking. Foam grouts are usually more difficult to apply and are, therefore, more expensive than gel grouts. Acrylamide gel grout has been successfully used since the 1950s and appears to have a long service life. A root growth inhibitor may be added to the grout in areas where roots present a problem.

Prior to performing any rehabilitative measures on a sewer reach, protruding service connections must be trimmed and, where necessary, roots chemically treated and removed. Both may be accomplished using trenchless technologies.

Advantages of grouting include the following:

- Does not damage or interfere with other underground utilities
- Does not require excavation or surface disturbance
- Usually one of the lowest cost alternatives
- Local contractors that perform grouting are readily available
- Sewer line may remain in service at low flow conditions when a hollow packer is used
- Services do not need to be reinstated

Disadvantages of grouting include the following:

- Does not improve structural strength or life of the pipeline
- May be susceptible to dehydration and shrinkage cracking
- Not reasonable for large cracks or offset joints/misaligned pipe which may not permit proper sealing of the packer
- Although longevity is claimed by grout manufacturers, grouting is not considered a 'permanent' long term rehabilitation by some.

8.2.1.5 Slip lining

Sliplining is accomplished by pulling or pushing a flexible liner of slightly smaller diameter into the existing pipeline. Services are reconnected by cutting the liner at the service location and grouting the space between the liner and the existing sewer. Liners may be made of fiberglass reinforced polyesters (FRP), reinforced thermosetting resins (RTR), polyvinyl chloride (PVC) or polyolefin such as polybutylene (PB) or polyethylene (PE), which is the most common. Sliplining is typically used to rehabilitate sewers with severe cracking/deterioration and structural problems. Access pits must be excavated to complete sliplining.

Prior to performing sliplining, protruding service connections must be trimmed and, where necessary, roots chemically treated and removed.

Advantages of sliplining include the following:

- Limited risk of damage or interference with other underground utilities
- Requires minimal excavation or surface disturbance
- Typically less costly than excavation and replacement
- Liners are corrosion resistant
- Service flow typically does not need to be bypassed
- Structurally capable of carrying loads

Disadvantages of sliplining include the following:

- Reduction in pipe capacity
- Requires excavation of an access pit
- Straight sewer reaches required
- Internal obstructions must be removed prior to sliplining

8.2.1.6 Cured-In-Place Lining

Although cured-in-place (CIP) lining has been in use for over 30 years, the cost of lining was cost prohibitive for most applications during the early use of the technology. Patents for the technology expired approximately 20 years ago, after which more manufacturers entered the market and as a result, costs for using this technology have decreased. Manufacturers have also developed technology to perform 'point repairs' or 'short liners' where small portions of a pipeline can be lined.

Cured-in-place lining is accomplished by inserting a resin-impregnated felt tube into the pipe, inverting it against the interior wall of the pipe and allowing it to cure using heated water or steam. After curing, service connections are re-opened using a cutting tool. This type of lining decreases the pipe diameter slightly, but results in a very smooth interior surface. Polyester and epoxy resins are widely used in this method of pipe rehabilitation. Cured-in-place lining may be designed to provide structural reinforcement of the pipe.

In addition to lining the complete reach, "point repairs" may be installed to line smaller sections of pipe from 1-10 feet long. Lining of mainline sewers may correct defects and reduce infiltration, but will not reduce infiltration at break-in service connections unless the lateral is also lined.

Prior to performing CIP lining, protruding service connections must be trimmed and, where necessary, roots chemically treated and removed.

More recent technological advances have also made it possible to line service laterals. Some systems permit lateral lining from the mainline sewer, while others require access from the basement clean-out.

Advantages of CIP lining include the following:

- Applicable to any sewer shape
- Does not damage or interfere with other underground utilities
- Does not require excavation or surface disturbance
- Small sections of pipe (point repairs) can be lined
- Service laterals can be lined
- Chemically resistant liners
- Often less costly than excavation and replacement
- Can be designed to provide structural reinforcement

Disadvantages of CIP lining include the following:

- Bypassing of flow is required
- Services must be not be used during the procedure
- Maximum effluent temperatures limited to 82° F.

8.2.1.7 Fold and Formed Lining

This process uses a thermoplastic pipe such as PE or PVC which is folded (typically a U shape), pulled into place, heated to make the plastic pliable, and forced back into a circular shape. Laterals are cut using a TV camera and cutter. This process may be considered a variation of sliplining, but does not require the excavation of pits if manhole access is available. Fold and formed lining is available for standard pipe sizes from 4-inches to 12-inches in diameter. This technique could be used for the same applications as sliplining, that is, severe cracking/ deterioration and structural problems. The length of pipe installed at any time is limited by the amount of steam generation and cooling of the liner. Typical maximum lengths are 700 feet for 6-inch, 400-500 feet for 8-inch and 300-400 feet for 10-inch diameter pipe.

Prior to performing any lining, protruding service connections must be trimmed and, where necessary, roots chemically treated and removed.

Advantages of fold and formed lining include:

- No risk of damage or interference with other underground utilities
- Requires no surface disturbance
- Can be less costly than excavation and replacement
- Liners are corrosion resistant and structurally capable of carrying loads

Disadvantages of fold and formed lining include:

- Reduction in pipe capacity
- Relatively straight sewer reaches required
- Must remove internal obstructions prior to lining (protruding service connections, roots)
- Heat/steam can cause pipe to re-fold
- Can develop water pockets between pipe and liner during installation if standing water remains in the pipe
- Annular space between pipe and liner can allow infiltration

8.2.2 Sewer Manholes

Several options are also available for rehabilitating leaking and deteriorated manholes. The most common methods include the following:

- Frame and Cover Rehabilitation
  - Resetting manhole covers
  - Sealing or replacement of the ring and corbel
- Sidewall and Base Rehabilitation
  - Construction of new inverts and floors
  - Sealing leaking pipe connections
  - Sealing leaking manhole riser joints
  - Root removal
- Excavation and replacement
- Lining (Cementicious and Epoxy)
- Drop-in fiberglass liners (inserts)

Rehabilitation of leakage into manholes is typically targeted at two areas, either the frame and cover, or the walls and base. Several rehabilitation techniques are discussed below.

8.2.2.1 Frame and Cover Rehabilitation

Rehabilitation of leaking manhole frames and covers may be accomplished by several methods as follows:

- Installation of stainless steel bolts with caulking compound and neoprene washers or corks in plug cover holes
- Prefabricated lid inserts installed between the frame and cover
- Joint sealing tape between the frame and cover
- Hydraulic cement and waterproofing epoxy to seal cracks and openings
- Raising of frame above grade
- Reconstructing the corbel and resetting the frame
- Installation of waterproof covers in areas prone to flooding

Plugs to block holes in the manhole cover are easy to install, but restrict natural venting. Prefabricated lid inserts prevent water and grit from entering the manhole around the cover, but they must be perfectly fit to the manhole to be successful. They are also subject to freezing. Joint sealing tape is simple to install, but has a short service life. Hydraulic cement can provide a strong waterproof seal, but may be labor intensive to install and may deteriorate with freeze-thaw cycles. Raising the frame above grade can minimize flow entering the system, but is limited to manholes which are not in the road. Where the existing corbel is in poor condition, reconstruction and resetting the frame may be the best solution. In areas that are prone to flooding, covers should be replaced with waterproof covers.

8.2.2.2 Sidewall and Base Rehabilitation

Rehabilitation of the sidewalls and base of leaking and/or deteriorated manholes is typically accomplished using one of the following approaches:

- Construction of new inverts and floors
- Sealing leaking pipe connections
- Sealing leaking manhole riser joints
- Root removal

Manholes with poorly constructed or damaged inverts may be rehabilitated by the construction of a new invert. However, this typically requires bypassing of flow unless a prefabricated fiberglass invert is used.

Chemical grouts like those used to seal leaking pipe joints are frequently used to help seal leaking manholes. Most manhole rehabilitations will begin by drilling holes through the structure in select locations to inject chemical grout into the soil behind the manhole wall. Grouting will typically stop any active leakage. Hydraulic cement is then used to seal the grouting holes, as well as the location of any other leaks or defects, such as at the riser joints ir pipe connections. Where roots have infiltrated between manhole joints or pipe connections, they must be removed prior to performing the aforementioned rehabilitation techniques. Grouting and plugging leaks with hydraulic cement is typically applied to precast concrete manholes.

8.2.2.3 Excavation and Replacement

Excavation and replacement of leaking and/or deteriorated manholes is typically undertaken only when necessary and when structural problems exist. Replacement should be considered for any of the following reasons:

- Severely deteriorated structural integrity
- Substandard manhole size
- When new sewers are installed in a different alignment

Excavation and replacement requires disturbance of pavement or other surface ground features and is often very disruptive in urban or densely developed areas. Depth of manholes is also a factor for evaluation when considering replacement, as deeper manholes will require larger excavations which may impact other nearby structures and/or utilities. The cost/benefit of replacement must be considered against the end product of replacement vs. rehabilitation.

Advantages of manhole replacement include the following:

- New structure with a full service life
- Watertight structure when constructed properly
- New structure may be tested to verify it is leak-tight

Disadvantages of manhole replacement include the following:

- Expense
- Traffic and urban disruption
- Possible interruption of other utilities

- Need to maintain service flows during construction
- Trench stabilization requirements
- Trench dewatering

8.2.2.4 Cementicious/Epoxy Lining

Lining manholes is a technology to stop infiltration in a manhole which is structurally sound. The process consists of cleaning (pressure washing) the manhole, removing any roots, sealing leaks with chemical grout or cement products, and applying a cementicious lining. The application of fully applying a cemeticious lining is usually applicable to block or brick manholes, rather than precast structures. The cemeticious lining can provide and extra infiltration barrier and seal the numerous joints in these types of structures.

In addition, an epoxy lining may be applied over the cementicious lining for added protection against hydrogen sulfide gas. Adding the epoxy coating increases the cost of the lining substantially, but can be beneficial if deterioration of the manhole due to hydrogen sulfide gas is evident. Epoxy lining may also be applied to precast manholes that may have corrosion due to hydrogen sulfide gas.

8.2.2.5 Drop-In Fiberglass Liners

Drop-In Fiberglass Liners are prefabricated liners that are inserted into existing manholes. The existing frame, cover and eccentric cone must be excavated and removed from the manhole. A prefabricated fiberglass manhole with a slightly smaller diameter is then lowered into the existing manhole. Holes are cut, the existing sewers connected, and the annular space between the existing and new manhole is grouted. This type of liner is well suited to deep manholes in urban areas where excavation is not possible.

8.2.3 Sewer Service Rehabilitation

Replacement of service laterals may be accomplished by the following techniques:

- Open cut excavation
- Pipebursting
- Directional Drilling

Service lateral rehabilitation may include the following:

- Testing and sealing (with grout) the first 3-5 ft of the lateral from the mainline
- CIP lining

8.2.3.1 Service Lateral Replacement

Complete replacement of leaking service laterals offers the most comprehensive method for addressing leaking services. Replacement of service laterals can have significant advantages, including reduced infiltration, reduction in backups from root intrusion, insufficient slope or breaks in the service. Service lateral replacement can mitigate infiltration entering through faulty piping, but will not necessarily address other infiltration sources like sump pumps. However, the presence of other sources of inflow, like sump pumps, foundation drains and roof leaders can be identified during replacement. A more detailed inspection of service laterals identified with clear running water may be beneficial to identify the source of the infiltration in the service line. TV inspection of the lateral from the cleanout can be conducted to identify defects in the lateral piping as opposed to the presence of any foundation drains, etc. Replacement provides a pipe with a full life span, with modern water-tight connections.

Service lateral replacement using excavation can be very disruptive as it requires disturbance of residential yards, landscaping, driveways, municipal streets and curbs. In cases where significant surface features exist, trenchless replacement options like pipebursting or directional drilling may be considered in lieu of open excavation construction.

Advantages of service line replacement include the following:

- New structurally sound pipe with full life expectancy
- The entire lateral can be replaced and issues addressed throughout the entire length
- New pipe with proper slope and no obstructions
- Addresses infiltration problem throughout the length of the lateral
- Provides a comprehensive and long-term repair
- A large portion of I/I can often emanate from the services, so replacement can result in significant infiltration reduction.
- Unknown sources of inflow such as area drains, foundation drains , basement drains, etc. can be identified and removed from the system.

Disadvantages of service line replacement include the following:

- Expense of restoration of surface features as landscaping, stone walls, etc.
- Possible interruption of other utilities
- Trench stabilization requirements
- Trench dewatering
- Involves cooperation with the homeowner

* Service laterals are typically privately owned, which raises issues with regard to who should bear the cost of replacement

8.2.3.2 Service Lateral Rehabilitation

Grouting of service connections is similar to the process for grouting mainline sewers, but a special packer is used that can extend approximately 5 feet up the service lateral. Service lateral leakage often occurs in the first 5-10 feet of the line where the lateral may be below the groundwater table and connects to the mainline sewer. Therefore, testing and sealing the lower portion of the lateral can be effective in removing infiltration. Using this trenchless technology prevents surface disturbance and is often the only area of a service lateral in which the utility can access. However, grouting is not considered a permanent repair, and the majority of the service lateral length is ignored because it is inaccessible to the grouting equipment.

Advantages of service line test & seal include the following:

* Less costly than lateral replacement
* Less disruptive than lateral replacement
* Can be performed from the Town-owned sewer and therefore does not require homeowner cooperation/involvement
* Can remove a some lateral leakage, particularly at the connection to the sewer (break-in connections)

Disadvantages of service line test & seal include the following:

* Addresses problems/leaks in only the first 5 feet of the lateral
* Can only be performed on structurally sound service lateral pipe (may be difficult or impossible to obtain a seal on fibre impregnated paper services
* May be susceptible to dehydration and shrinkage cracking
* Not reasonable for large cracks or offset joints/misaligned pipe which may not permit proper sealing of the packer
* Does not address infiltration entering the service from other sources like sump pumps or foundation drains

An alternative to grouting that has developed in recent years is cured-in-place liners for service laterals. Although this technology works the same way as mainline CIP lining, the technology for lining service laterals are still fairly new to the market (compared with chemical grouting for example), and remain relatively costly. Some systems can perform lining from an interior cleanout in the building, and from the mainline sewer, while others require the excavation of access pits. Typically, each individual lateral is different and the individual conditions of the installation often govern weather pits are necessary, how many, etc.

8.3 Inflow Reduction Strategies

Inflow may result from any stormwater connection to the sanitary sewer system. The focus of the CDM phase II study in 1997 was evaluation of inflow, and as such, many recommendations for removal of inflow from the system were included in the final report.

Inflow reduction strategies may include the following:

- Separation of catch basins
- Separation of storm drain piping
- Redirection of roof drains
- Redirection of foundation drains
- Redirection of Area/yard/driveway drains
- Redirection of Basement drains
- Redirection of sump pump discharges
- Waterproof manhole and cleanout covers in flood-prone areas

Although the Town has completed separation of nearly all of the major interconnects between the stormwater and sanitary sewer systems (catch basins, stormwater piping, etc), many less obvious connections remain. The Town has a list of 71 'potential' inflow sources, that include primarily roof drains, that require further investigation and perhaps separation. These sources were included in the Town's 308 response as Table 4-1. In addition, the Town's GIS includes approximately 8 "public" roof drains that require separation, which include various Town buildings and the High School.

A house-to-house survey in 3 small pilot areas of the town identified the presence of 62 sump pumps and 5 exterior drains that were either directed to the sanitary sewer, or the discharge point was unknown. TV inspection performed indicated that approximately 60% of all identified I/I in was from private sources (services).

Given the magnitude of flows that occur during significant rain events, there are significant inflow sources remaining that contribute stormwater to the sewer system. Many of these sources of inflow may emanate from private property. Although private I/I sources can be difficult to deal with, ignoring them will do little to reduce I/I in the Town.

8.3.1 Private Inflow Sources

Although private service laterals may contribute significant extraneous flows to the sewer system, inflow from private sources may greatly affect peak flows in the system. Sources of private inflow may include roof drains, foundation drains, sump pumps and other drain connections.

In recent years as more research and understanding of collection systems has occurred, the importance and magnitude of I/I contribution from private laterals has become better understood. Many municipalities are finding that over 50% of their I/I is resulting from private sources. Erie County, NY developed (and the NYSDEC approved) a table of prescribed flows contributed by specific defects, and then permits private developers pay for private I/I removal to "free-up" capacity for their developments. For example, while a broken mainline pipe was valued at 2 gpm of I/I contribution, defective service laterals (private) were valued at15-70 gpm of I/I, with an average value of 42 gpm used, and higher values allowed, if determined appropriate, based on the condition of the lateral. El Cerrito, CA replaced service laterals (private) and mainline in one sub-basin (110 residences) in 1987 and saw I/I reduced by 86%, after which they concluded that laterals were responsible for the majority of I/I. In 1997 Sarasota Florida undertook a pilot study and evaluated 65 service laterals. They found the majority of the laterals were of substandard materials and 62% required replacement. In 2002 they replaced approximately 300 service laterals using pipebursting techniques and found substantial I/I reduction in the area.

In recent years many municipalities have begun tackling the problem of private I/I and they have taken a wide variety of approaches to dealing with private sources. Some of these approaches have included the following:

- Town ownership of laterals @ least to ROW.
- Requirement for inspection and correction of clear water sources when properties are transferred; testing of service laterals that may be good for a certain time period.
- Enforcement of existing ordinances (sump pumps are illegal, so remove them).
- Surcharge for known dischargers. For example, over 10 years ago, in attempt to remove private inflow from the system, the Town of Jaffrey conducted a house-to-house survey to identify sump pumps in the system and instituted a fee of $200 per billing cycle ($600 per year) for each home with a sump pump.
- Mandatory surcharge for all users. Waterville, ME recently instituted a mandatory surcharge of $20 per quarter which is applied unless inspection proves the private system is in compliance. The town of Winslow, ME has a similar program with a surcharge of $50/quarter.
- Town sponsored construction projects for lateral replacement (owner paid construction, but Town administers/facilitates large contract)
- Town sponsored low interest loans for lateral replacements
- Town paying for work on private property. Sarasota, FL took this approach and justified the spending of public money on private property because all ratepayer benefit equally from I/I reduction.
- Permitting developers to pay for private I/I removal (to reduce flows and "free- up" capacity for their new developments) where expenditure of public money on private property is prohibited (Erie County, NY)

All case studies have stated a common theme, that being that the key to success was getting the public and stakeholders involved in the process.

9. ALTERNATIVE EVALUATION – I/I REDUCTION STRATEGIES

9.1 No Action

One alternative is to take no action to reduce I/I and simply construct the necessary infrastructure to adequately pump and treat all water that enters the collection system. This might include construction of a new main pumping station, expansion of the WWTF and/or a separate treatment facility for CSO flows.

9.2 Sewer Mains

I/I in sewer mains can be reduced by replacement or rehabilitation of existing mains. Although there are many rehabilitation techniques available to reduce I/I into an existing sewer, for the purposes of evaluating alternatives and preparing cost opinions, we considered replacement or rehabilitation of the pipe by lining to be permanent options to restore the life-expectancy of the pipe. These are typically two of the most common alternatives for replacing or rehabilitating sewers. Although sealing leaks using grout is reported to have a long life (over 10 years), we did not consider this an equal final product to replacement or lining. In special situations, if conditions warrant, more complex technologies such as pipe bursting might be necessary.

Each sewer line that was TV inspected was evaluated and repair strategies identified in tables included in Appendix A-16. Based on the 22 areas that were TV inspected, 'project areas' were developed, and costs opinions provided for rehabilitation necessary to reduce I/I in each project area. Each project area was then ranked based on cost effectiveness ($ spent compared to I/I removed).

9.2.1 Replacement

For the purposes of this evaluation, we assumed that open-cut excavation would be used, since this is the most basic construction method for which historic cost data is readily available. The cost for replacement of sewer lines (8"–15") was assumed to be about $200/lf for the purposes of this evaluation. This cost is assumed to include the cost of manholes for the new sewer alignment.

9.2.2 Rehabilitation

Although other rehabilitation strategies are available, for this evaluation, lining using a CIP or fold and formed liner was considered a permanent option to restore the life of the pipe. While there are other technologies available on the market, CIP and fold and formed are probably the most widely used in New England. However, only a formal construction bid for a designed lining project can determine the least costly type of lining for the specific conditions. The cost for lining of sewer lines (8"-15") was assumed to be about $100/lf for the purposes of this evaluation. This cost includes lining of the sewer reach and reinstating services, and assumed manholes with be rehabilitated separately.

9.3 Manholes

Various options for rehabilitation and replacement of manholes are available, as discussed below.

9.3.1 Replace

Replacement of manholes was considered for manholes that were in poor structural condition, constructed of brick, or when the sewer was proposed for replacement in a different alignment. Although many manholes require work, no manholes were identified as eminent for failure. Table 9-1 identifies the manholes with significant defects that are recommended to be addressed in the near future. We have assumed that these manholes will be replaced at a cost of $1,000 per vertical foot for a total cost of approximately $60,000 for the 10 manholes that are recommended for replacement.

9.3.2 Rehabilitation

Rehabilitation of manholes was recommended in most cases, and would consist of grouting and sealing the structure with hydraulic cement. In the case of brick or block manholes, cemeticous grout can be used to add structural stability to the manhole and impede leakage. Table 9-2 identifies the 86 manholes with observed leakage that are recommended to be addressed in the future. Based on the cost analysis performed for the Jady Hill area, a $2,800 manhole rehabilitation cost has been assumed for a total estimated rehabilitation cost of approximately $241,000. It should be noted that two private manholes in Hayes Park (Area B) shown on the table were not included in the leaking manhole count.

Table 9-1
Manholes with Significant Defects

Location	MHNumber	Inspection Date	SubSystem	Manhole Depth	Manhole Defects			Wall Material	Comments
					Walls Condition	Floor Condition	Invert Condition		
DEWEY STREET	834	4/30/2009	B	6.1	2	2	4	BRICK	
HAYES PARK	1158	4/24/2009	B	3.4	4	0	1	ROCK	
FRONT STREET AT LINCOLN	674	5/8/2009	C	6.4	2	4	4	BRICK	
LINDEN STREET AT TELEPHONE	735	5/18/2009	C	4.8	2	1	4	BRICK	
TREMONT STREET	851	5/7/2009	C	4.1	1	4	1	PRECAST	
PINE STREET AT GROVE STREET	685	5/7/2009	D	6.8	3	5	4	BRICK	
ELM STREET AT MAPLE STREET	700	4/30/2009	D	8.3	3	4	5	BRICK	
HILLARD CIRCLE	701	4/30/2009	D	4.7	3	4	2	PRECAST	
PINE MEADOWS DRIVE (NEXT	1064	5/7/2009	F	8.6	3	4	4	PRECAST	Frame like Storm Drain during TV
STONEY BROOK LANE	246	4/13/2009	H	5.7	4	1	2	PRECAST	
				58.9					

Table 9-2
Manholes with Observed Leakage

Location	MHNumber	Inspection Date	SubSystem	Manhole Depth	Manhole Infiltration (gpm) Corbel	Walls	Floor	Invert	MH Total	Manhole Defects Walls Condition	Floor Condition	Invert Condition	Wall Material	Comments
LAPERLE AVENUE	562	5/6/2009	A	7.8	1	1			2.0	3	1	1	PRECAST	
LAPERLE AVENUE	561	5/6/2009	A	7.7	0.25	0.2			0.5	3	1	1	PRECAST	
LAPERLE AVENUE	564	5/6/2009	A	6.7	0.3				0.3	2	1	1	PRECAST	
LAPERLE AVENUE	560	5/6/2009	A	7.4	0.1				0.1	1	1	1	PRECAST	
WATER STREET AT	895	5/1/2009	B	6.3					1.5	2	2	1	BRICK	
HAYES PARK	1161	4/23/2009	B	8.7					1.5	2	0	1	PRECAST	Manhole leakage observed during Flow
HAYES PARK	1165	4/23/2009	B	6.4		0.3			0.3	3	1	1	PRECAST	
WATER STREET (REAR OF	902	5/1/2009	B	9.2		0.25			0.3	2	0	0	PRECAST	
WATER STREET IN	901	5/1/2009	B	7.6		0.1			0.1	3	1	1	PRECAST	
FRONT STREET	662	5/18/2009	C	7.7					2.0	1	3	3	BRICK	Leaking Pipe Con. During TV & Flow Is
LINCOLN STREET	879	5/8/2009	C	8.3				1	1.0	1	1	3	PRECAST	
HAMPTON FALLS ROAD	207	5/4/2009	F	13.2		5			5.0	3	1	1	PRECAST	Leakage During TV
HAMPTON FALLS ROAD	59	5/27/2009	F	7.2					3.0	1	2	1	PRECAST	Leakage during Flow Iso.
CARRIAGE DRIVE	52	5/22/2009	F	7.5					2.5	1	1	1	PRECAST/PARG	observed during flow iso.
HIGH STREET AT	68	5/29/2009	F	13.2		1		1	2.0	1	1	1	PRECAST	Leaking Structure and Pipe Con. During
ROBERTS DRIVE	1140	6/10/2009	F	5.7					2.0	1	1	1	PRECAST/PARG	Pipe Con. Leakage During TV & Flow Is
HAMPTON FALLS ROAD	206	5/4/2009	F	10.8		1			1.0	3	1	1	PRECAST	Leakage During TV
HAMPTON FALLS ROAD	210	5/1/2009	F	16.8		1			1.0	3	1	1	PRECAST	Leakage During TV
HAMPTON ROAD	116	5/28/2009	F	9.1				1	1.0	2	1	1	PRECAST	Leakage During TV
HAMPTON ROAD	119	5/28/2009	F	11.8		1			1.0	2	1	1	PRECAST	Leakage During TV
HAMPTON FALLS ROAD	58	5/27/2009	F	8.7				1	1.0	1	1	1	PRECAST	Pipe Con. Leakage During TV
HAMPTON ROAD	115	5/28/2009	F	6.3				1	1.0	1	1	1	PRECAST	Leakage During TV
ROBERTS DRIVE AT	1142	6/10/2009	F	9.1				1	1.0	1	1	1	PRECAST	Leakage During TV, 2 gpm flow iso.
FOX CHAPEL COURT	232	5/15/2009	F	6.6					1.0	1	1	1	PRECAST	Leakage during Flow Iso.
HAMPTON ROAD	16	5/28/2009	F	6.1					1.0	1	1	2	PRECAST	Leakage during Flow Iso.
PINE MEADOWS DRIVE	1064	5/7/2009	F	8.6	0.5	0.3			0.8	3	4	4	PRECAST	Frame like Storm Drain during TV
HAMPTON FALLS ROAD	217	5/4/2009	F	15		0.5			0.5	3	1	1	PRECAST	additional 1 gpm during flow iso.
PINE MEADOWS DRIVE	1065	5/7/2009	F	7.8		0.5			0.5	2	1	1	PRECAST	
HAMPTON FALLS ROAD	216	5/4/2009	F	15.3		0.25			0.3	3	1	1	PRECAST	additional 0.75 gpm during flow iso.
PLEASANT VIEW DRIVE	220	5/4/2009	F	16.1					0.3	3	1	1	PRECAST	Leakage during Flow Iso.
PLEASANT VIEW DRIVE	221	5/4/2009	F	16.3					0.2	3	1	1	PRECAST	Leakage during Flow Iso.
PLEASANT VIEW DRIVE	222	5/4/2009	F	12.2					0.2	3	1	1	PRECAST	Leakage during Flow Iso.
PLEASANT VIEW DRIVE	227	5/5/2009	F	6.2					0.2	3	1	1	PRECAST	Leakage during Flow Iso.
PLEASANT VIEW DRIVE	228	5/5/2009	F	8.3					0.2	3	1	1	PRECAST	Leakage during Flow Iso.
FOLSOM STREET	229	5/15/2009	F	14.1					0.2	3	1	1	PRECAST	Leakage during Flow Iso.
PINE MEADOWS DRIVE	1063	5/7/2009	F	11.2					0.2	1	1	1	PRECAST	Leakage during Flow Iso.
FULLER LANE	111	5/15/2009	F	9.4		0.14			0.1	1	1	1	PRECAST	Leakage During TV
RIDGEWOOD TERRACE	138	5/15/2009	F	8.1		0.14			0.1	1	1	1	PRECAST	Leakage During TV
RIDGEWOOD TERRACE	137	5/15/2009	F	7.2		0.14			0.1	1	1	0	PRECAST	Leakage During TV
GRANITE STREET	141	5/15/2009	F	8.1		0.14			0.1	1	1	0	PRECAST	Leakage During TV
ASHBROOK ROAD AT	21	5/28/2009	F	11		0.1			0.1	2	1	2	PRECAST	additional 0.25 gpm during flow iso.
HAMPTON FALLS ROAD	46	5/27/2009	F	7.9		0.1			0.1	2	2	1	PRECAST	additional 0.5 gpm during flow iso.
PINE MEADOWS DRIVE	208	5/1/2009	F	14.9		0.1			0.1	3	1	1	PRECAST	
HAMPTON FALLS ROAD	209	5/1/2009	F	14.8		0.1			0.1	3	1	1	PRECAST	
ASHBROOK ROAD	22	5/28/2009	F	13.2		0.1			0.1	2	1	1	PRECAST	
ASHBROOK ROAD	23	5/28/2009	F	15.3		0.1			0.1	2	1	1	PRECAST	
HAMPTON FALLS ROAD	48	5/27/2009	F	7.7		0.1			0.1	2	1	1	PRECAST	
HAMPTON FALLS ROAD	49	5/27/2009	F	16		0.1			0.1	2	1	1	PRECAST	
HAMPTON FALLS ROAD	50	5/27/2009	F	17.2		0.1			0.1	3	0	1	PRECAST	
PINE GROVE ROAD	60	5/26/2009	F	8.4		0.1			0.1	2	1	1	PRECAST	
LITTLE PINE LANE	92	5/26/2009	F	6.1		0.1			0.1	2	1	1	PRECAST	
HAMPTON ROAD AT	1210	5/28/2009	F	11.2		0.1			0.1	2	1	1	PRECAST	
PLEASANT VIEW DRIVE	179	5/15/2009	F	11.7					0.1	1	1	1	PRECAST	Leakage during Flow Iso.
PLEASANT VIEW DRIVE	225	5/5/2009	F	7.4					0.1	3	1	1	PRECAST	Leakage during Flow Iso.
THORNTON STREET AT	148	5/11/2009	G	6.8				2	2.0	1	1	1	BLOCK	Pipe Con. Leak during TV
HIGH STREET	127	5/19/2009	G	8				0.5	0.5	2	0	1	BRICK	Pipe Con. Leak during TV
HIGH STREET	136	5/19/2009	G	4.8				0.2	0.2	2	2	2	BRICK	Invert Leak During TV
PORTSMOUTH AVENUE	248	4/15/2009	H	9.8					2.0	1	1	1	PRECAST	Leakage during flow iso.
WEBSTER AVENUE	279	4/20/2009	H	10.9				1	1.0	2	1	3	PRECAST	
DOWNING COURT	275	4/14/2009	H	16.3		1			1.0	3	1	1	PRECAST	
LEARY COURT	270	4/14/2009	H	10.2	0.1	0.2			0.3	3	1	1	PRECAST	
DOWNING COURT	265	4/13/2009	H	12.9		0.2			0.2	3	2	2	PRECAST	
BITTERSWEET LANE	303	4/23/2009	H	8.9		0.2			0.2	3	1	2	PRECAST	
LEARY COURT ATUTILITY	271	4/14/2009	H	9.8		0.2			0.2	2	1	2	PRECAST	
WEBSTER AVENUE	278	4/20/2009	H	13.2		0.2			0.2	3	1	1	PRECAST	TV insp -broken VCP for invert; no
STEVENS COURT	266	4/14/2009	H	15.7		0.2			0.2	2	1	1	PRECAST	
BONNIE DRIVE	313	4/23/2009	H	7.6			0.2		0.2	1	1	1	BLOCK	
LEARY COURT	269	4/14/2009	H	8.1		0.1			0.1	3	2	2	PRECAST	
PORTSMOUTH AVENUE	250	4/15/2009	H	9.8		0.1			0.1	2	2	1	PRECAST	
PORTSMOUTH AVENUE	251	4/15/2009	H	8.9		0.1			0.1	3	1	1	PRECAST	
ALLEN STREET	255	4/14/2009	H	12.1		0.1			0.1	3	1	1	PRECAST	
ALLEN STREET	257	4/14/2009	H	8.2			0.1		0.1	1	3	1	PRECAST	
ALLEN STREET	258	4/14/2009	H	7		0.1			0.1	2	1	2	PRECAST	
STEVENS COURT	267	4/14/2009	H	8.3		0.1			0.1	2	1	2	PRECAST	
PORTSMOUTH AVENUE	247	4/15/2009	H	9.9		0.1			0.1	2	1	1	PRECAST	
PORTSMOUTH AVENUE	249	4/15/2009	H	9.8		0.1			0.1	2	1	1	PRECAST	
PORTSMOUTH AVENUE	253	4/15/2009	H	9.3		0.1			0.1	2	1	1	PRECAST	Pipe Con. Leak During TV
DOWNING COURT	260	4/13/2009	H	11.8		0.1			0.1	2	1	1	PRECAST	
STEVENS COURT	268	4/14/2009	H	8.3		0.1			0.1	2	1	1	PRECAST	
DOUGLAS WAY	284	4/20/2009	H	9.5		0.1			0.1	2	1	1	PRECAST	
DOUGLAS WAY	287	4/20/2009	H	6.5		0.1			0.1	2	1	1	BLOCK	
RIDGECREST DRIVE	292	4/20/2009	H	7.3			0.1		0.1	1	2	1	BLOCK	
HAVEN LANE	316	4/23/2009	H	3.8		0.1			0.1	2	1	1	BLOCK	Walls Leak During TV
PORTSMOUTH AVENUE	1025	4/17/2009	H	9.3			0.1		0.1	1	2	1	PRECAST	
PORTSMOUTH AVENUE	1168	4/15/2009	H	12.2		0.1			0.1	2	1	1	PRECAST	
WEBSTER AVENUE	280	4/20/2009	H	10.8			0.1		0.1	1	1	1	PRECAST	
HAVEN LANE R.O.W.	301	4/23/2009	H	9.6		0.1			0.1	3	0	0	BLOCK	
JADY HILL COURT	324	4/23/2009	H	6.9			0.1		0.1	1	1	1	BLOCK	Requires cleaning and reevaluation
PARK STREET AT CASS	828	5/7/2009	I	4.5				0.7	0.7	2	2	3	BRICK	Invert Leak During TV
				869.4					49.5					

9.4 Private Inflow Removal

As evident from previous tabulations and calculations, inflow can have a significant impact on peak flows in the collection system. Removal of these inflow sources are typically the most cost effective way to reduce I/I in the system, but are sometimes the most difficult to remove because of the interaction required with private homeowners if it is from a private source.

9.4.1 Services

As previously discussed, sewer services are often a major contributor of I/I that is often overlooked. Services were found to contribute approximately 60% of the total I/I observed during TV inspection. Many services were found to be constructed of sub-standard materials, and not properly connected to the mainline sewer. Not only can infiltration enter the service through defects in the pipe, but other connections that contribute inflow, such as sump pumps, foundation drains and roof drains may be present. Flow from these inflow sources may be significant and should be a priority for removal.

9.4.2 Sump Pumps

Sump pumps were identified in the three pilot areas where house-to-house inspections were performed. In addition, some sump pumps were identified from the questionnaire that was mailed to residents. Thirdly, a few suspected sump pumps were identified during TV inspection.

Sump pumps that discharge to the sewer system can be removed by disconnecting the pump discharge from piping that flows to the sewer, and re-piping the pump discharge to another location. This may be to the yard of the home, to a dry-well if soils are suitable for infiltration, or to a public storm sewer, if one exists.

9.4.3 Leaders and Drains

Direction connections such as roof drains can have a dramatic impact on peak inflow rates. A single large (institutional/municipal) building with a ¼ acre roof could contribute over 200,000 gpd to the system during a severe rain storm. Residential homes can often be easily removed from the system by simply redirecting downspouts to the ground. Larger commercial/industrial/institutional buildings with flat roofs may have roof drains piped to the sewer system internally within the building, or hard piped underground. Because of the magnitude of flow from these buildings, they may be more difficult to disconnect and may require the presence of a storm sewer to receive the discharge.

Recent evaluations indicate that approximately 15 acres of impervious area may be connected to the system as of August 2011. While it is understood that approximately 3 acres from portions of PEA and sources around the U.S. Post Office are in the process of being removed, it is believed that significant inflow sources still remain (12+ acres). In addition, Town-performed smoke testing around PEA revealed that many roof leaders may be equipped with traps that prevent identification through smoke testing. It is therefore recommended that flood and dye testing be performed for buildings with drainage suspected to be connected to the sewer.

Other drains, such as foundation drains, or basement floor drains may contribute flow to the system continuously during periods of high groundwater.

9.5 Summary of Projects and Costs

9.5.1 Sewer Mainline with Private Services Projects

A total of 22 sewer mainline with services projects were identified which correspond to the TV inspection areas.

Appendix A-16 includes summary tables for each of the 22 areas that were TV inspected. Each reach inspected is identified, leakage indicated and recommended rehabilitation techniques indicated. In general, the following rehabilitation techniques were assumed:

9.5.2 Sewer Mainline with Private Services Project Methodology

The following methodology was used to evaluate various observed sewer defects for recommended repair solutions.

Point repair for break-in services: Break-in services are where a hole was broken in the main-line sewer to install a service pipe. These types of service connections are not water-tight and frequently leak. Most of the main-line leakage identified during TV inspection was due to these types of service connections. The proposed repair for this defect is to excavate (open cut) and install a new 'factory' service connection (wye) in the pipe. The service pipe would then be replaced to the curb. This repair was applied to any non-factory type service in the project area. In addition, many capped or abandoned services were noted in the inspection reports. If an abandoned factory service was identified with a factory cap, no action was assumed needed. If an abandoned service was identified and either a cap could not be seen, or it was capped without a 'factory' cap, or the factory cap was observed to be defective (i.e. leak) a point repair was assumed necessary to properly cap the abandoned service.

Point repair (short liner) for main-line defects: Isolated defects in the pipe can often be repaired with a short (less than 8 ft) cured-in-place liner. This would include repairs of slightly offset joints, circular cracks, small longitudinal cracks, holes in the pipe, etc. Short liner point repairs can be installed using trenchless technologies and do not require excavation.

Sewer Lining: Where multiple defects are present in a sewer reach, particularly where VCP is present, lining the sewer with a CIP or fold and formed liner is typically the most cost effective repair, so long as there are no defects like severely offset joints, broken pipes or sags that would require excavation.

Main-line replacement: Where defects are present that cannot be remedied by lining, such as sags, severely mis-aligned pipe, crushed pipe, etc., replacement may be necessary. Although replacement may be accomplished using open cut excavation or trenchless technologies like pipe-bursting, for the purposes of this report, we have assumed that open cut excavation would be permissible where replacement is necessary.

Service replacement/investigation: Where clear water was observed flowing from the service, it was assumed that replacement or further investigation was necessary. Services may be TV inspected to identify their condition, to identify defects, and any connections to inflow sources. For the purposes of preparing projects and cost opinions, it was assumed that defective services would be replaced using open cut excavation.

9.5.3 Ranking Sewer Rehabilitation Projects

For the purposes of preparing cost opinions and ranking projects, each was evaluated based on three alternatives. Option one was to construct repairs only in the public right of way. For this alternative we assumed that public sewer main rehabilitation/replacement included the following:

- Mainline sewer replacement or lining
- Sewer main point repairs
- Sewer service replacement within the ROW (wye at the mainline and service to the property line)

The second alternative (option 2) included private service repairs in addition to the mainline sewer described in Option 1. Private sewer rehabilitation/replacement was assumed to include the following (consistent with the Jady Hill Sewer Project estimate):

- Work as described in Option 1, and
- Service repair or replacement from the ROW to the building

The third alternative considered (Option 3) was to address public and private sewer issues, as well as provide storm drains and drain services where existing storm drains were not present, to provide a discharge point for the removal of private inflow. Each project area was reviewed for existing storm drainage. Where existing storm drains were not present to receive inflow from sump pumps or other inflow sources (basement drains, roof drains, foundation drains, etc), costs were included for installation of a storm sewer and drain services to the curb at each residence. Storm drains may not be necessary, or desirable in all instances, but this represents the worst-case situation where storm drains may be needed (for example to discharge roof drains from a large institutional building). Other less costly options may exist for some areas, such as discharging to the ground, installing dry-wells, or requiring homeowners to bear the cost of disposing of their stormwater. Option 3 was assumed to include the following (consistent with the Jady Hill Sewer Project estimate):

- Work described in Options 1 and 2, and
- Drain line installation where no existing drain line
- Drain services for each building with a sewer service

9.5.4 Project Cost Estimations

UEI performed a detailed budgetary cost estimate for a sewer rehabilitation project within the Jady Hill Pilot Area (Appendix A-25) which was used as a basis for developing unit costs which were used to evaluate the other recommended project areas.

Costs were estimated using the following unit costs:

Service repair to the curb (replace break-in services)	$3,000
Point repairs (short liners)	$3,000/ea
Service lateral replacement (curb to house)	$5,000/ea
Sewer replacement cost (includes MHs)	$200/lf
Sewer lining (generally includes MH rehab.)	$100/lf
Drain line installation	$150/lf
Drain service installation	$3,000/ea

Project cost rankings are based on $/gallon using flows measured during flow isolation. Flows identified during TV inspection were used for the purpose of proportioning the I/I contributed by services vs mainline. TV mainline flow to service flow (ratio) is valid because each reach was TV'd under the same conditions. However, flows measured during flow isolation are considered more accurate since they were measured with a weir rather than visual estimation. Also, flow isolation work was conducted during a shorter time-period, thereby giving more uniform results across the different areas. TV inspection spanned a 3 month time frame which gave varying groundwater conditions across the various TV areas.

Table 9-3
Sewer Main Projects Ranking and Cost Effective Analysis
PUBLIC SEWER MAINLINE WORK ONLY

Tier	Project Area	Streets	Approximate Length (LF)	Flow Isolation I/I (gpd)	Recommended Mainline Work	Storm Sewer Rqd. (LF)	Public (with 22.5% Engineering)	Public & Private Sewer Only (with 22.5% Engineering)	Subtotal (with 22.5% Engineering)
Tier 1 (5,066 ft)	10	Elm/Spring Street (PEA)	376	66,240	None		$4,594	$12,250	$12,250
	4	Bonnie Drive	3,634	66,240	**(See Jady Hill Est.)**	3,100	$1,345,969	$1,996,750	$3,099,250
	11	Tan Lane	230	5,760	None		$9,188	$24,500	$33,688
	3	Hampton Road	230	3,600	Pt. Repairs	300	$36,750	$59,719	$142,406
	7	Holly Court	596	8,640	Pt. Repairs, Lining		$114,844	$168,438	$200,594
Tier 2 (15,796 ft)	8	Ridgewood Terrace	1,300	18,000	Pt. Repairs	400	$142,406	$295,531	$479,281
	12	Pine Street	1,509	20,160	Pt. Repairs, Lining		$568,094	$904,969	$1,107,094
	14	Rockingham Street	210	2,880	Pt. Repairs, Lining		$84,219	$137,813	$169,969
	6	High Street	4,792	46,800	Pt. Repairs, Lining	3,000	$869,291	$1,313,353	$2,268,853
	21	Ashbrook Road	1,208	11,520	Lining	900	$97,081	$165,988	$414,050
	15	Front Street	3,636	44,280	Pt. Repairs, Lining, Replacement	1,500	$1,369,091	$2,096,434	$2,877,372
	13	Main Street	3,141	28,800	Pt. Repairs, Lining, Replacement	1,200	$1,154,103	$1,866,134	$2,568,978
Tier 3 (17,615 ft)	18	Hampton Road	1,489	11,520	Pt. Repairs, Lining	1,600	$168,744	$260,619	$683,244
	5	Towle Avenue	1,367	10,008	Pt. Repairs, Lining, Replacement	900	$333,659	$479,128	$773,128
	1	Hayes Park (Private)	2,120	12,960				$0	
	19	Ashbrook R.O.W.	3,549	34,200	Pt. Repairs, Lining	1,100	$261,844	$315,438	$600,250
	17	Hampton Road	2,684	16,416	Pt. Repairs, Lining, Replacement	2,500	$349,278	$548,341	$1,241,997
	20	Roberts Drive	663	3,600	Pt. Repairs, Lining	600	$152,206	$221,113	$400,269
	2	Allen Street	1,450	7,200	Pt. Repairs	800	$119,438	$264,906	$535,938
	16	Westside Drive	1,098	6,480	None	300	$27,563	$73,500	$169,969
	22	Hampton Falls Road	2,407	11,232	Pt. Repairs, Lining	1,400	$199,675	$299,206	$680,488
	9	Pleasant View Drive	788	2,880	Pt. Repairs, Lining	500	$178,084	$224,022	$366,428
				439,416			$7,586,119	$11,728,150	$18,825,494

Within Pilot Area

Assumptions:

1 I/I observed during flow isolation was used as the basis for I/I removal because areas were completed in relatively the same groundwater conditions (TV inspection spanned nearly 3 months)

2 Public sewer work includes rehabilitation of mainline sewer within the ROW, service connections to the sewer main, and services to the curb

3 Private sewer work includes services from the curb to the house

4 Assumed I/I removal @30% if only work in public ROW is completed and 70% of I/I removal if private and public work is completed

5 Project 10, 66,240 gpd I/I was observed during flow isolation that is believed to be related to a water main break in that area which has since been repaired. 2,880 gpd from a private service into a manhole in that area is believed to remain.

9.6 Predicted I/I Reductions in Flow

UEI identified approximately 426,000 gpd infiltration within the proposed project areas based on flow isolation. It should be noted that the 12,960 gpd I/I observed in the Hayes Park during flow isolation (Project Area 1) was not included in this because it is part of a private sewer system. Hayes Park is considered a single customer/user to the Town, and the Town would not be performing construction on the Hayes Park system to removal I/I. Approximately 60% of the I/I identified during the TV inspection was attributed to private sewer services. Given the uncertainty at this time as to whether or not the Town would be willing to perform work on private property, we developed cost opinions and estimated I/I reduction from each project for three conditions as follows:

1. Public Work Only: The first condition assumed that only mainline (public) work would be performed. This included replacement or rehabilitation of the mainline sewer, replacement of service wyes on the main, and replacement of service stubs to the property line.

2. Public and Private Work: The second option assumed that mainline and private work would be performed. This included work in the right-of-way described in the first condition, and additionally, replacement of services from the property line to the building.

3. Public, Private and Drain Work: The third option assumed all of the work described in the first two conditions, in addition to construction of new storm sewers where public storm sewers are currently not available. All new storm sewers would be installed with drain services to the property line. In addition, where existing storm sewers exist, new drain services would be installed to the property line.

When mainline sewer work alone (option 1) is performed, it is typical to quantitatively achieve minor (30-50%) I/I removal, because as leaks and defects in the mainline are repaired, this causes the groundwater table to rise in the area of the repair, and the infiltration migrates to the nearest defect that was not repaired (often in the services). As shown in Table 9-4, if I/I removal is undertaken only in the public sewers, approximately 85,000 gpd might be removed from the system.

It was assumed that if both public and private rehabilitation was undertaken (option 2), the percent of I/I removed would increase to approximately 70%, because as more comprehensive repairs are performed to the system, there are less defects to where infiltration may migrate. In addition, when services are repaired, other sources of private inflow can be identified and removed from the system. These may include sump pumps, foundation drains and other drains. If 70% of the I/I is removed from both public and private sources within the project areas, an I/I reduction of approximately 300,000 gpd is anticipated.

Table 9-4 below illustrates that if the Town is to ignore the private service I/I problem and only perform repairs on the public sewer, effectively only 28% of the total I/I identified in the project areas during flow isolation would be removed from the system. If both the mainline (public) sewer and the services are repaired, 70% of the I/I identified during flow isolation could be removed. This illustrates that 3.5 times the amount of I/I can be removed from the system by taking a comprehensive approach to repair both the mainline and services.

Table 9-4

Estimated I/I Removal for Mainline and Service Repairs

	Mainline (option 1)	**Services**	**Mainline & Services (option 2)**
Total I/I flow observed during flow isolation in the 22 project areas	170,000 gpd (40%)	256,000 gpd (60%)	426,000 gpd
Assumed total I/I removal at ~50% effectiveness	85,000 gpd	NA	NA
Assumed I/I removal at ~70% effectiveness	119,000 gpd	180,000 gpd	300,000 gpd
% of total I/I removal in project areas if all 22 projects are completed	28%	NA	70%

It should be noted that based on historical recorded flows only a very aggressive public and private I/I removal program might provide enough I/I reduction to eliminate the CSOs in Town. If the pilot results are successful and 30 mgd peak flows are found to be erroneous, CSOs could be eliminated with an aggressive I/I program, but identification and removal of inflow sources discussed in Section 6 is a critical step to reduce peaks.

9.7 Cost-Effective Evaluation – I/I Reduction

UEI evaluated the cost effectiveness of I/I projects based on the following three scenarios as described in the previous section:

- Public mainline repairs only (option 1, Table 9-5)
- Public mainline repairs and private sewer service replacement (option 2, Table 9-6)
- Public mainline repairs and private sewer service replacement plus drains (option 3, Table 9-7)

The cost effective evaluation is a process that should be adjusted after review of the success of the pilot areas is performed and the cost/schedule for the new WWTF is defined. However, as shown in Table 9-7, it is unlikely that the 22 identified sewer projects alone will not reduce peaks enough to eliminate the CSO. Identification and removal of the direct inflow sources evidenced by the Spring Street CSO monitoring discussed in Section 6, is critical to eliminating peak flows that contribute to CSOs. Additional discussion of CSO mitigation alternatives is discussed in Section 10.

Table 9-5
Sewer Main Projects Ranking and Cost Effective Analysis
OPTION 1 - PUBLIC SEWER WORK ONLY (IN ROW)

Project Area	Streets	Total Project Cost Budget	$/gal I/I Removed (20% reduction)	Estimated I/I Removed (gpd)
1	Hayes Park (Private)			
10	Elm/Spring Street (PEA)	$5,000	**$0**	13,248
11	Tan Lane	$9,000	**$8**	1,152
16	Westside Drive	$28,000	**$21**	1,296
19	Ashbrook R.O.W.	$262,000	**$38**	6,840
8	Ridgewood Terrace	$142,000	**$40**	3,600
21	Ashbrook Road	$138,000	**$60**	2,304
3	Hampton Road	$37,000	**$51**	720
7	Holly Court	$115,000	**$66**	1,728
18	Hampton Road	$169,000	**$73**	2,304
2	Allen Street	$119,000	**$83**	1,440
22	Hampton Falls Road	$200,000	**$89**	2,246
6	High Street	$869,000	**$93**	9,360
4	Bonnie Drive	$1,346,000	**$102**	13,248
17	Hampton Road	$349,000	**$106**	3,283
12	Pine Street	$568,000	**$141**	4,032
14	Rockingham Street	$84,000	**$146**	576
15	Front Street	$1,369,000	**$155**	8,856
5	Towle Avenue	$334,000	**$167**	2,002
13	Main Street	$1,154,000	**$200**	5,760
20	Roberts Drive	$152,000	**$211**	720
9	Pleasant View Drive	$178,000	**$309**	576
				0
		$7,627,000		**85,291**

Within Pilot Area

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Table 9-6
Sewer Main Projects Ranking and Cost Effective Analysis
OPTION 2 - PUBLIC AND PRIVATE SEWER WORK ONLY

Project Area	Streets	Total Project Cost Budget	$/gal I/I Removed (70% reduction)	Estimated I/I Removed (gpd)	Estimated Peaks Removed (gpd)
1	Hayes Park (Private)				
10	Elm/Spring Street (PEA)	$12,000	**$0**	46,368	278,208
11	Tan Lane	$25,000	**$6**	4,032	24,192
19	Ashbrook R.O.W.	$315,000	**$13**	23,940	143,640
16	Westside Drive	$74,000	**$16**	4,536	27,216
21	Ashbrook Road	$207,000	**$26**	8,064	48,384
8	Ridgewood Terrace	$296,000	**$23**	12,600	75,600
3	Hampton Road	$60,000	**$24**	2,520	15,120
7	Holly Court	$168,000	**$28**	6,048	36,288
18	Hampton Road	$261,000	**$32**	8,064	48,384
22	Hampton Falls Road	$299,000	**$38**	7,862	47,174
6	High Street	$1,313,000	**$40**	32,760	196,560
4	Bonnie Drive	$1,997,000	**$43**	46,368	1,112,832
17	Hampton Road	$548,000	**$48**	11,491	68,947
2	Allen Street	$265,000	**$53**	5,040	30,240
12	Pine Street	$905,000	**$64**	14,112	84,672
15	Front Street	$2,096,000	**$68**	30,996	185,976
14	Rockingham Street	$138,000	**$68**	2,016	12,096
5	Towle Avenue	$479,000	**$68**	7,006	42,034
20	Roberts Drive	$221,000	**$88**	2,520	15,120
13	Main Street	$1,866,000	**$93**	20,160	120,960
9	Pleasant View Drive	$224,000	**$111**	2,016	12,096
	Subtotal I/I Area Project Cost	$11,769,493		**298,519**	2,625,739
	Additional Private Service Separation	$4,500,000			
	TOTAL	**$16,269,493**			

Within Pilot Area

Notes: A peaking factor of 6 was based on the April-June 2009 continuous flow monitoring data for the Westside Drive and Allen Street pilot areas. The 6 peaking factor was applied to all projects except Bonnie Drive. A peaking factor of 24 was used for Bonnie Drive based on April-June 2009 continuous flow monitoring information for the Jady Hill pilot area. No CSO events occurred during the April-June 2009 continuous flow monitoring, so peaking factors may be higher.

Additional private sewer separation includes estimated costs of $7,656 for 585 sewer services which represents 22% of all the sewer services in Town not included in the 22 project areas listed above. (($5000)25% cont and mob)22.5% eng. = $7656

Project costs generally include lining and point repairs if feasible. Project costs will be greater if the Town replaces sewers in lieu of lining and point repairs.

Table 9-7
Sewer Main Projects Ranking and Cost Effective Analysis
OPTION 3 - PUBLIC AND PRIVATE SEWER AND DRAIN WORK

Project Area	Streets	Total Project Cost Budget	$/gal I/I Removed (70% reduction)	Estimated I/I Removed (gpd)	Estimated Peaks Removed (gpd)
1	Hayes Park (Private)				
10	Elm/Spring Street (PEA)	$12,250	**$0**	46,368	278,208
11	Tan Lane	$33,688	**$8**	4,032	24,192
19	Ashbrook R.O.W.	$600,250	**$25**	23,940	143,640
7	Holly Court	$200,594	**$33**	6,048	36,288
16	Westside Drive	$169,969	**$37**	4,536	27,216
8	Ridgewood Terrace	$479,281	**$38**	12,600	75,600
21	Ashbrook Road	$455,394	**$56**	8,064	48,384
3	Hampton Road	$142,406	**$57**	2,520	15,120
4	Bonnie Drive	$3,099,250	**$67**	46,368	1,112,832
6	High Street	$2,268,853	**$69**	32,760	196,560
12	Pine Street	$1,107,094	**$78**	14,112	84,672
14	Rockingham Street	$169,969	**$84**	2,016	12,096
18	Hampton Road	$683,244	**$85**	8,064	48,384
22	Hampton Falls Road	$680,488	**$87**	7,862	47,174
15	Front Street	$2,877,372	**$93**	30,996	185,976
2	Allen Street	$535,938	**$106**	5,040	30,240
17	Hampton Road	$1,241,997	**$108**	11,491	68,947
5	Towle Avenue	$773,128	**$110**	7,006	42,034
13	Main Street	$2,568,978	**$127**	20,160	120,960
20	Roberts Drive	$400,269	**$159**	2,520	15,120
9	Pleasant View Drive	$366,428	**$182**	2,016	12,096
	Subtotal I/I Area Project Cost	$18,866,838		**298,519**	2,625,739
	Additional Private Service Separation	$7,200,000			
	TOTAL	**$26,066,838**			

Within Pilot Area

Notes: A peaking factor of 6 was based on the April-June 2009 continuous flow monitoring data for the Westside Drive and Allen Street pilot areas. The 6 peaking factor was applied to all projects except Bonnie Drive. A peaking factor of 24 was used for Bonnie Drive based on April-June 2009 continuous flow monitoring information for the Jady Hill pilot area. No CSO events occurred during the April-June 2009 continuous flow monitoring, so peaking factors may be higher.

Additional private sewer separation includes estimated costs of $12,250 for 585 sewer and drain services which represents 22% of all the sewer services in Town not included in the 22 project areas listed above. (($5000+$3000)25% cont and mob)22.5% engineereing = $12,250

Project costs generally include lining and point repairs if feasible. Project costs will be greater if the Town replaces sewers in lieu of lining and point repairs.

10. CSO MITIGATION STRATEGIES

10.1 Current LTCP Policy for CSO Control in Exeter

Exeter's current LTCP is based on EPA's *Presumptive Approach* stemming from CDM's recommendation of complete sewer separation to achieve CSO control to a 5-year rainfall event. To meet that goal, CDM had recommended several projects. As discussed in previous sections of this report, it is believed that the majority of CDM's recommended sewer separation and other recommended projects have been completed, but a 5-year level of CSO control does not seem to be achieved. It appears that the Town must also remove the significant suspected private direct inflow sources that were only recently detected with recent CSO metering, to achieve "complete sewer separation" scenario and 5-year level of control included in CDM's modeling.

In addition, during the course of this study, the Town has made significant strides identifying additional inflow sources, improving operations and maintenance of the wastewater collection system, and upgraded instrumentation to further mitigate CSO events including:

- Town smoke testing identified several acres of direct connections associated with catch basins and roof leaders in the vicinity of Phillips Exeter Academy and the US Post Office. These direct connections are anticipated to be removed by Fall 2012.

- Improved operations of the Town's water treatment plant to limit peak flows associated with filter backwash operations.

- Improved operation of a sewer siphon below the Squamscott River through more frequent cleaning.

- Additional instrumentation added to the diversion structures to monitor potential "back-flow" of Squamscott River floodwaters into the wastewater collection system that is suspected to have occurred in the past.

- Elimination of the Water St. diversion structure hydraulic restriction is anticipated to be completed by Fall 2012.

- Reduction public and private I/I identified in the Jady Hill area of town has commenced and is anticipated to be completed by 2013.

10.2 Long-Term CSO Policy Goal for Exeter

Exeter has a long-term CSO goal of ultimately eliminating CSO discharges except during extreme (i.e. 50-100 year) events. Because of the potential for those extreme events, Exeter intends to keep the diversion structures in place as a safeguard against private property damage associated with sewer backups during high flow events and to safely maximize existing in-line storage as required by EPA's nine minimum controls. Although the Town has the long-term goal to eliminate CSOs, the implementation schedule to eliminate the CSO must be considered within the context of the high cost of the anticipated WWTF upgrade.

10.3 Long-Term Alternatives Considered

The following is a summary of the CSO long-term alternatives evaluated as part of this study with the Town's ultimate goal to eliminate discharges from the CSO (except in extreme events). In addition to the direct capital and O&M costs for each alternative, the large indirect capital and increased O&M costs associated with the anticipated WWTF upgrade were also considered to compare the different alternatives.

To compare new WWTF capital and O&M costs between the different alternatives, it was assumed that a new WWTF would maintain the existing 3.0 mgd average day rated capacity (Alternative 2) at a minimum. Alternative 2 assumes 70% of the average I/I removal in 60% of the system (Table 3-1) or approximately 0.3 mgd, and this was added to the assumed average daily WWTF design flow under Alternatives 3 & 4 since no I/I projects were assumed under those alternatives.

Note, we have not evaluated a new WWTF due to more stringent permit limits. This will be driven by the NPDES permit so it will be necessary with any of the options considered.

10.3.1 Alternative 1 - No Action

The "No Action" alternative is not believed to be a viable long term solution for CSO control. Although the Town generally meets the requirements of the *Presumptive Approach,* the presence of sensitive areas downstream of the CSO discharge and recent NPDES permit CSO sampling and analysis requirements is not consistent with the presumptive approach. However Clemson Pond is listed as the CSO receiving water and is not currently listed included on EPA's 303(d) list, recently proposed nutrient limits for the Town's WWTF discharge to the Squamscott River, the 2002 NPDES permit modification indicating that CSO shall not contribute to violations of Water Quality Standards, the Squamscott River listing on EPA's list of threatened or impaired waters 303(d) list indicating that E. coli impairment as a result of CSO discharges; all indicate that the *presumptive approach* or "No Action" alternative is not a viable long-term CSO solution for the Town.

10.3.2 Alternative 2 - Complete I/I Removal

This alternative includes elimination of I/I to a point where CSO discharges no longer occur. This alternative is difficult to implement because as stated in previous sections of this report at least 60% of the observed existing I/I is due to private sources which are administratively difficult and expensive to permanently remove. However, new CSO metering installed in December 2010, and evaluation of the August 19, 2011 CSO storm event provided evidence of significant sources of direct inflow, which, if these sources can be identified and removed from the sewer, will significantly reduce peak flows that cause CSOs. The following capital projects were included in this alternative:

- All twenty-two TV Area I/I Projects with private separation ($19M, Table 9-7)
- Additional private service separation outside of project areas ($7.2M, Table 9-7)
- Additional suspected direct inflow investigation and removal ($5M)

Alternative 2 Capital Costs = $31,200,000

Advantages

- Reductions in CSOs will occur
- Is consistent with necessary collection system infrastructure improvement program to maintain the current level of service
- Reduces the environmental and aesthetic impacts of CSO discharges to Clemson Pond
- Is a viable long-term solution from a regulatory standpoint
- Reduces long-term O&M costs associated with pumping and treating extraneous flow in the system
- Has the potential to reduce the capital costs of the WWTF upgrade by reducing required WWTF capacity

Disadvantages

- High capital costs
- Will take time to implement
- Requires work and expenditures on private property
- Requires additional study to identify direct inflow sources that are suspected to remain connected to the system, but have not been identified through previous inflow investigations
- May still need CSO

10.3.3 Alternative 3 - Pump, Flow Equalization & Treat at WWTF

This alternative includes upgrade of the Main Pumping Station to convey wet-weather wastewater to the existing WWTF site (Figure 10-1). This alternative will require a parallel force main which would discharge to a flow equalization basin and metered into the main WWTF treatment train. The following capital projects were included in this alternative:

- Sewer collection system improvements = None
- Private I/I removal = None
- Upgrade of Main Pumping Station from 7 mgd to 16 mgd capacity (flow to be confirmed)
- New influent screen at main pumping station
- New parallel 20" force main from main pumping station to WWTF
- Modification of 1 WWTF lagoon for flow equalization including: sludge disposal, PVC liner, and new metering pumping station to bleed overflow storage water to the WWTF

Alternative 3 Capital Costs = $11,400,000 (Table 10-1)

Advantages

- Reduction in CSO events will occur
- Reduces the environmental and aesthetic impacts of CSO discharges to Clemson Pond
- Is a viable long-term solution from a regulatory standpoint

Disadvantages

- Requires long-term O&M costs associated with pumping and treating extraneous flow

10.3.4 Alternative 4 - CSO Treatment prior to Discharge to Clemson Pond

A separate CSO treatment facility to treat CSO events prior to discharge to Clemson Pond is not a viable alternative (Figure 10-2). NHDES indicated that discharges to Clemson Pond would have no dilution and EPA's position that all CSO discharges must meet water quality standards would require an independent treatment facility treating CSO discharges to the limit of technology, including phosphorus removal. However, there are existing permitted CSO treatment facilities in the state and they are allowed per EPA's CSO Policy. The following capital projects were included in this alternative:

- Upgrade of Main Pumping Station from 7 mgd to 16 mgd capacity (flow to be confirmed)
- New influent screen at main pumping station
- New 10 mgd ballasted flocculation CSO WWTF to be built near Clemson Pond
- New force main from main pumping station to new CSO WWTF

Alternative 4 Capital Costs = $12,200,000 (Table 10-2)

Advantages

- Mitigates the environmental and aesthetic impacts of CSO discharges to Clemson Pond because of improved water quality of discharges

Disadvantages

- Does not reduce volume of CSOs to Clemson Pond
- Requires long-term O&M costs associated with pumping and treating extraneous flow
- May not be a viable long-term solution pending future discharge limits
- Adds an additional WWTF that the Town must operate and manage
- There may not be a suitable site available for WWTF construction in the vicinity of Clemson Pond

10.4 Recommended Long-Term CSO Alternative

Alternative 2 – Complete I/I Removal has the lowest, 20-year, present worth, capital and O&M costs of the alternatives evaluated (Table 10-3). However, that does not necessarily mean that it is affordable for the Town. In addition, we recommend that this approach be reassessed every 2-years, after pilot projects have been implemented and when a new WWTF is required.

TABLE 10-1
ALTERNATIVE 3 - PUMP, FLOW EQUALIZATION, AND TREAT AT WWTF
OPINION OF PROBABLE CONSTRUCTION COST

ITEM	QUANTITY	UNIT	UNIT PRICE	PROBABLE COST
Main Pump Station Improvements				
General Requirements	1	LS	$ 498,080.00	$498,100
New 18" Gravity Sewer	75	LF	$ 200.00	$15,000
New 30" Gravity Sewer	600	LF	$ 350.00	$210,000
New 36" Gravity Sewer	80	LF	$ 500.00	$40,000
Spring St Diversion Structure Modification Allowance	1	LS	$ 20,000.00	$20,000
Dry Pit/Wetwell Expansion Allowance	1	LS	$ 300,000.00	$300,000
Demo Allowance	1	LS	$ 100,000.00	$100,000
New Mechanical Screen with Compaction	1	LS	$ 400,000.00	$400,000
New Slide Gates, Grating, Misc. Metals	1	LS	$ 20,000.00	$20,000
New 150 Hp Centrifugal Pumps (2)	1	LS	$ 350,000.00	$350,000
New Generator	1	EA	$ 150,000.00	$150,000
New Force Main	5800	LF	$ 250.00	$1,450,000
Process Piping and Valve Allowance	1	LS	$ 10,000.00	$10,000
Electrical Allowance	1	LS	$ 150,000.00	$150,000
Instrumentation Allowance	1	LS	$ 100,000.00	$100,000
Soil Disposal/Environmental Allowance	1	LS	$ 50,000.00	$50,000
WWTF Storage Lagoon Improvements				
New Gorman-Rupp Duplex Pump Station w/Enclosure	1	LS	$ 250,000.00	$250,000
New Wetwell	1	LS	$ 20,000.00	$20,000
New Generator	1	LS	$ 85,000.00	$85,000
New 8" Force Main	200	LF	$ 80.00	$16,000
Sludge Removal and Disposal Allowance	1500	Dry Ton	$ 1,000.00	$1,500,000
PVC Liner	400000	SF	$ 2.00	$800,000
Rip-rap	4000	CY	$ 20.00	$80,000
Electrical Allowance	1	LS	$ 100,000.00	$100,000
Instrumentation Allowance	1	LS	$ 25,000.00	$25,000
SUBTOTAL				***$6,739,000***
Contractor OH&P - 15%				$1,011,000
Contingency - 20%				$1,348,000
TOTAL PROBABLE CONSTRUCTION COST				***$9,098,000***
Admin, Engineering and Construction Services				$2,275,000
TOTAL PROJECT COSTS				***$11,373,000***
Notes:				

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JOB 1542 - EXETER I/I
SHEET NO. ______ OF ______
CALCULATED BY CSM DATE 11/12/12
CHECKED BY ______ DATE ______
SCALE ______

FIGURE 10-1

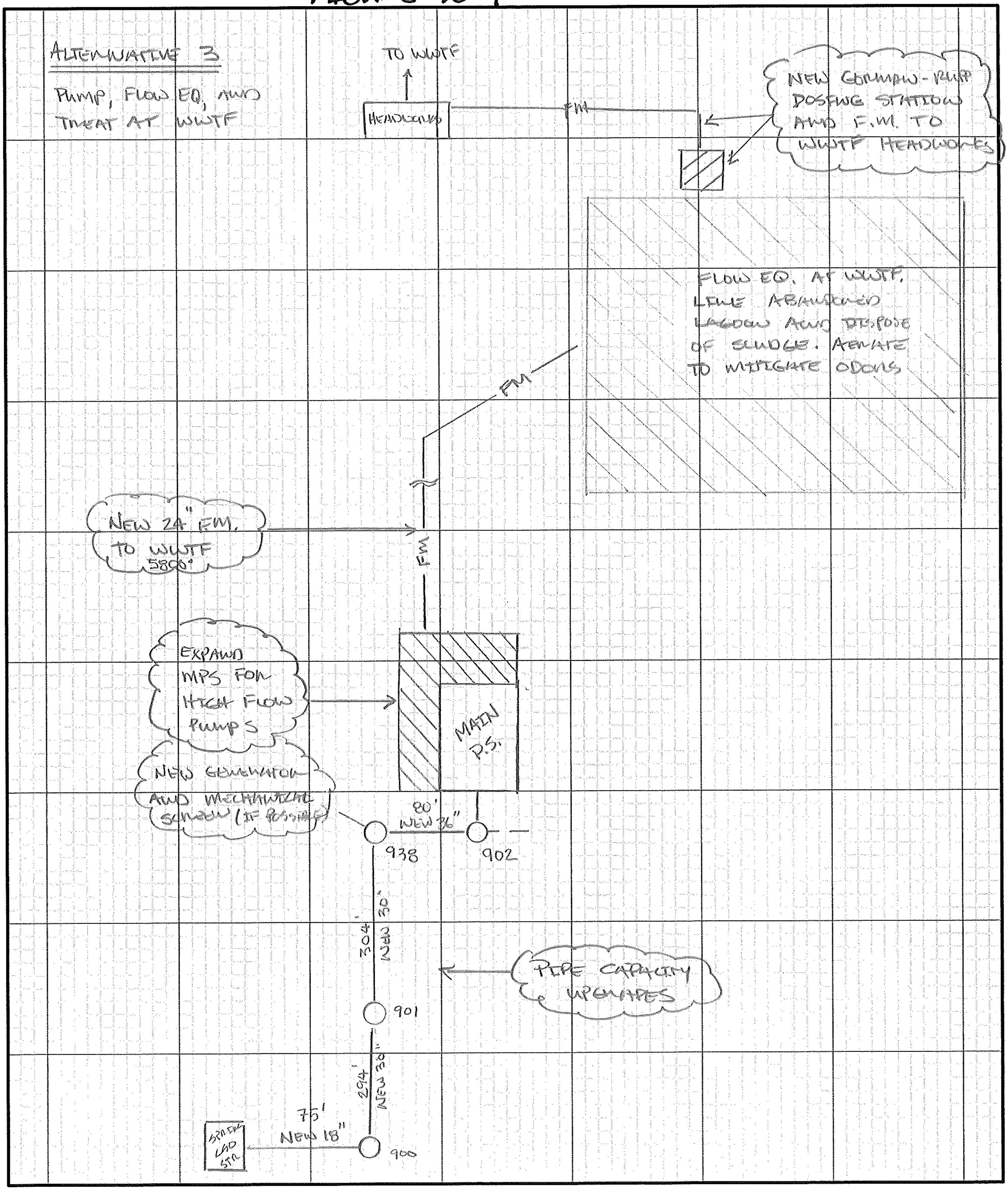

TABLE 10-2
ALTERNATIVE 4 - CSO TREATMENT PRIOR TO DISCHARGE TO CLEMSON POND
OPINION OF PROBABLE CONSTRUCTION COST

ITEM	QUANTITY	UNIT	UNIT PRICE	PROBABLE COST
General Requirements	1	LS	$ 518,800.00	$518,800
Land Acquisition	1	LS	$ 200,000.00	$200,000
Main Pumping Station Improvements				
New 18" Gravity Sewer	75	LF	$ 200.00	$15,000
New 30" Gravity Sewer	600	LF	$ 350.00	$210,000
New 36" Gravity Sewer	80	LF	$ 500.00	$40,000
Spring St Diversion Structure Modification Allowance	1	LS	$ 20,000.00	$20,000
Dry Pit/Wetwell Expansion Allowance	1	LS	$ 300,000.00	$300,000
Demo Allowance	1	LS	$ 100,000.00	$100,000
New Mechanical Screen with Compaction	1	LS	$ 400,000.00	$400,000
New Slide Gates, Grating, Misc. Metals	1	LS	$ 20,000.00	$20,000
New 150 Hp Centrifugal Pumps (2)	1	LS	$ 350,000.00	$350,000
New Generator	1	EA	$ 150,000.00	$150,000
New Force Main by Directional Drill	1000	LF	$ 600.00	$600,000
Process Piping and Valve Allowance	1	LS	$ 10,000.00	$10,000
Electrical Allowance	1	LS	$ 150,000.00	$150,000
Instrumentation Allowance	1	LS	$ 100,000.00	$100,000
Soil Disposal/Environmental Allowance	1	LS	$ 50,000.00	$50,000
CSO Treatmnent Facility				
Sludge Tank	1	LS	$ 300,000.00	$300,000
Ballasted Flocculation (10 mgd Actiflo)	1	LS	$ 2,000,000.00	$2,000,000
Control Building and Chemical Storage	1	LS	$ 400,000.00	$400,000
Chlorination/Dechlorination	1	LS	$ 900,000.00	$900,000
Piping Modifications	1	LS	$ 20,000.00	$20,000
New Generator	1	LS	$ 200,000.00	$200,000
Electrical Allowance	1	LS	$ 100,000.00	$100,000
Instrumentation Allowance	1	LS	$ 50,000.00	$50,000
SUBTOTAL				***$7,204,000***
Contractor OH&P - 15%				$1,081,000
Contingency - 20%				$1,441,000
TOTAL PROBABLE CONSTRUCTION COST				***$9,726,000***
Admin. Engineering and Construction Services				$2,432,000
TOTAL PROJECT COSTS				***$12,158,000***
Notes:				

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JOB 1542 - EXETER F/E
SHEET NO. ______ OF ______
CALCULATED BY CSM DATE 11/12/12
CHECKED BY ______ DATE ______
SCALE ______

FIGURE 10-2

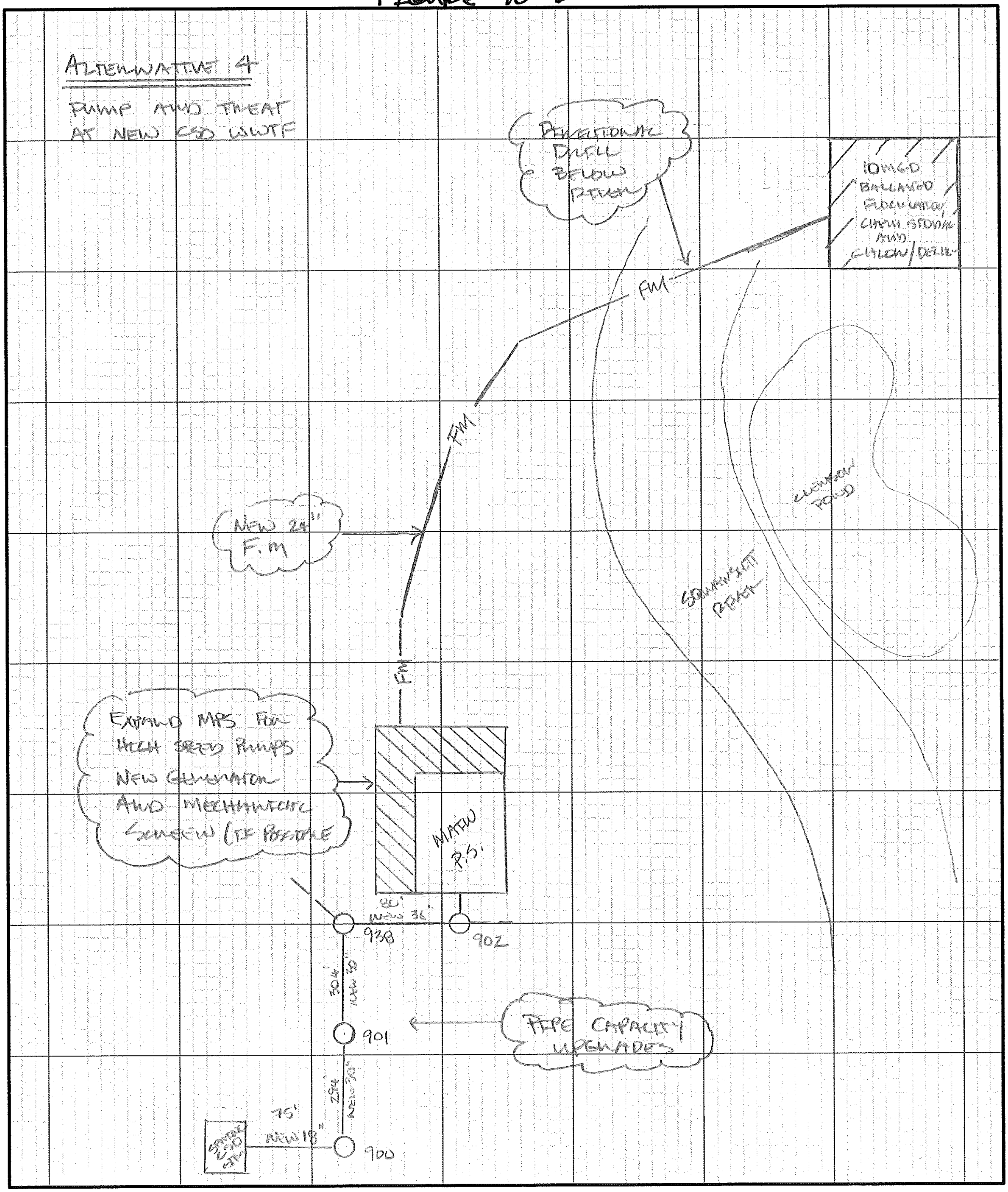

TABLE 10-3
Opinion of Probable Cost - 20 Year
Summary of Alternatives

	Sewer Separation I/I Removal Alternative 2 (3 mgd WWTF)	Pump & Blend Alternative 3 (3.3 mgd WWTF)	New CSO WWTF Alternative 4 (3.3 mgd WWTF)
WWTF Capital Costs			
WWTF Upgrade Cost ($15/gal design flow)	**$45,000,000**	**$49,500,000**	**$49,500,000**
Annual WWTF O&M Costs ($1/gpd ADF midpoint)	$2,550,000	$2,700,000	$2,700,000
13.01 Present Worth of Annual Costs (20-year @ 4.5%)	**$33,175,500**	**$35,127,000**	**$35,127,000**
I/I Project Capital Costs			
Identified Projects[3]	**$26,000,000**	**$0**	**$0**
Collection System Maintenance ($500,000/year)	**$0**	**$10,000,000**	**$10,000,000**
Additional Inflow Study and Removal Project Allowance (note 2)	**$5,000,000**	**$5,000,000**	**$5,000,000**
Main Pumping Station and WWTF Eq Capital Costs	**$0**	**$11,373,000**	**$0**
Clemson Pond CSO WWTF and Main PS Upgrade Costs			
New WWTF for CSO and Main PS Upgrade	**$0**	**$0**	**$12,158,000**
Annual CSO treatment O&M Costs	$0	$0	$248,200
13.01 Present Worth of Annual Costs (20-year @ 4.5%)	**$0**	**$0**	**$3,229,082**
Annual Main Pumping Station O&M Costs	$100,000	$110,000	$110,000
13.01 Present Worth of Annual Costs (20-year @ 4.5%)	**$1,301,000**	**$1,431,100**	**$1,431,100**
Total Capital Costs	**$76,000,000**	**$75,873,000**	**$76,658,000**
Total Present Worth (Capital and O&M Only)	**$110,476,500**	**$112,431,100**	**$116,445,182**

Notes:

1 Additional 300,000 gpd WWTF capacity for options 3 & 4 is to account for the costs of 440,000 gpd identified I/I that would not be targeded for removal under those options. Option 2 assumes that 70% of this I/I is removed and maintains the existing 3.0 mgd WWTF capacity baseline.
2 Portions of the $5,000,000 allowance assumed for inflow study and removal may be born by property owners pending findings of the study.
3 All identified project may not be needed to achieve targeted CSO mitigation pending periodic evaluation.

11. <u>COST IMPACT</u>

11.1 Financial Capacity Assessment

In addition to the I/I and CSO work, a new WWTF will likely be needed to meet new permit limits. Detailed cost evaluations for the WWTF have not been completed, but it has been reported that costs could be as high as $50M. Assuming $15/gpd capital costs, a new WWTF of 3 MGD capacity would cost the Town $45M. Total capital cost of $45M for the WWTF, plus $31M for identified I/I reduction projects = $76M. Note that this does not include any ongoing sewer work or O&M costs.

We performed an estimate of the increase in cost of service for the anticipated WWTF upgrade to frame a context for the affordability. The resultant cost of service increase assuming capital and O&M costs for the anticipated WWTF upgrade (based on an assumed $15/gal ADF WWTF upgrade capital cost, $1/gpd ADF treatment costs, and 150 gpd average residential usage) is greater than 2% of the median household income. This analysis deducted the existing approximate $500,000 WWTF O&M costs (Table 11-1).

Assuming the WWTF upgrade project takes priority over CSO mitigation projects, then no CSO mitigation projects would be financially feasible until the bond payments for the WWTF upgrade are complete.

TABLE 11-1
CSO LTCP and WWTF Upgrade Affordibility

	20-Year Costs (Table 10-3)
Capital Costs (WWTF and CSO Mitigation Alt. #2)	$76,000,000
Annual bond Payment (4.5%)	**$5,590,992**
Annual WWTF O&M Costs ($1/gpd ADF)	**$2,550,000**
Deducted Existing Annual WWTF O&M Cost (2010)	**($500,000)**
Total Annual Cost of Service Increase with New WWTF	**$7,640,992**
Number of sewer accounts (January 30, 2012)	3,567
Average Annual Metered Consumption (gallons)	400,000,000
Additional annual cost per gallon of metered consumption	$0.019
Annual increase in residential rate with new WWTF (based on 150 gpd metered consumption)	$1,045.86
Existing Residential Sewer Rate (based on 150 gpd)	$355.09
New Residential Sewer Rates to accommodate increased WWTF and CSO LTCP costs	$1,400.95
Exeter CDP MHI (2006-2010)	$56,073
New Sewer Rates as Percentage of Annual WWTF Upgrade Cost of MHI	**2.5%**

12. PUBLIC EDUCATION AND OUTREACH

12.1 Goals

Public participation is critical to inform sewer users about the challenges that the Town faces to provide wastewater management and treatment, and foster a relationship with the sewer users so that they feel ownership of the system. One of the challenges facing the Town is that a majority of the I/I entering the system comes from private sources and public participation and education is a critical first step to mitigate private I/I contributions.

12.1.1 Reduce Delayed Inflow

Private sump pumps connected to the sewer have been identified as a significant source of delayed inflow in the system. Many users may not be aware that their sump pump connections are illegal and/or of the impact that these illicit discharges have on the system. Through public education and outreach the Town has made the effort to educate users of the impact of sump pumps to garner voluntary compliance to minimize future enforcement actions.

12.1.2 Pilot Areas

The Town identified three pilot areas that were included in this study where there were suspicions of private sump pump activity. Individual home inspections were performed in these pilot areas and a significant number sump pump illicit connections were identified during inspection. The Town did not want to penalize homeowners that submitted to voluntary inspection, but instead used the home inspection information to educate users about illicit connections and urge users for voluntary compliance with the sewer ordinance.

12.2 Options

12.2.1 Education

Public education and voluntary Sewer Use Ordinance (SUO) compliance is the preferred method for and is the critical first step toward private I/I mitigation. An effective public education program can lead to voluntary compliance and provides a more defensible justification for enforcement actions if necessary.

12.2.2 Ordinance Modifications

Town recently updated the SUO including language to strengthen control of industrial discharges, establishment of local IDP limits, and general updates to revise ordinance to current practices and EPA and State standards.

The Town's SUO includes the following critical provisions:

- Building services are to be constructed, operated, repaired, maintained and reconstructed (if needed) by the property owner.
- Building services extend from the building to the main (includes area from main to the curb in private ownership).
- Old building sewers may be re-used if found, by the Town, to meet the requirements of the ordinance.
- Secure and watertight connections are required to connect the building sewer to the main.
- Prohibits connections of roof downspouts, foundation drains, areaway drains, or other surface runoff or groundwater to a building sewer or drain to the sanitary sewer.
- The Town shall inspect and approve new building sewers prior to backfilling.
- If wastewater discharged to the sewer is significantly greater than water consumed, the Owner can be required to install a recording flowmeter.
- Prohibits discharge of any stormwater to the sewer, including surface water, groundwater, roof runoff, subsurface drainage, uncontaminated cooling water or unpolluted industrial process waters.
- Stormwater shall be discharged to a storm sewer, if available, or to a natural outlet approved by the Town.
- Provides Town workers to enter all properties to conduct inspections, observations, measurements, sampling and testing pertinent to the discharge.
- When investigation reveals a violation, the Town shall give written notice of the violation either hand delivered or via certified mail with receipt acknowldeged.
- Any person guilty of violating the provisions of the ordinance can be fined between $1,000 and $10,000 per day per violation.

The following additions should be considered for future ordinance modifications:

- Ability for Town to spend public money on private laterals (private I/I separation)
- Requirement for lateral inspection when properties are transferred or when desired
- Require installation of cleanouts at the curb on service laterals.
- Provide authority to perform an annual inspection of private pump stations. Require a maintenance plan and flow data from private pump stations be submitted to the Town for review. Consider requiring a period of spring-time continuous flow monitoring upstream of private pump stations to provide baseline data for comparison to pumped flow.
- Applying a quarterly surcharge to users that deny entry for illicit connection inspection.

The following items are things the Town has authority to do, but should consider developing into a routine program:

- Inspect TV laterals from the house cleanout
- Inspect downspouts (dye test roof drains?)
- Require homeowner to fix laterals that are defective

12.2.3 Voluntary Improvements

Voluntary improvements to private services and sump pump removal are the most effective and preferred long-term solution to mitigate private I/I contributions. Voluntary compliance and improvements show that the users are committed to a common goal with wastewater operators, and under this scenario it is more likely that users will remain in compliance. Unfortunately, mandatory improvements and enforcement actions are sometimes the only way to obtain compliance with the SUO.

12.2.4 Mandatory Improvements

Mandatory improvements such as inspection and continued re-inspection for sump pump compliance, mandated sewer service replacement or rehabilitation, etc. can be effective for chronic and blatant violations of the SUO. However, this enforcement approach causes an adversarial relationship between the users and the Town and can lead to administrative difficulties for the Town, with continued and repeated inspections to confirm that compliance is maintained. A middle ground between voluntary and compulsory compliance is for the Town to create incentives for SUO compliance.

12.2.5 Incentives

Incentives for SUO compliance can include user fee abatements for users that demonstrate that they do not have illicit connections, cost sharing for work on private property, providing drain services for sump pump discharges, etc. We understand that the Town implemented the following incentives during the Jady Hill Pilot project currently under construction:

- The Town included much of the cost of new private sewer and drain laterals into the construction project if homeowners participate in the voluntary service lateral program.

- The voluntary service lateral program required that the individual homeowners be responsible for the first $1,000 of the cost of the new private sewer and drain laterals that were terminated 5-feet outside the foundation wall.

- The voluntary participation program allowed the $1,000 portion borne by the homeowner to be paid back to the Town over the next 10 years as a sewer bill surcharge at an interest rate of 0% ($25/quarter).

- The voluntary program requires the homeowners to hire a licensed plumber to make the final house sewer connection and certify that no illicit connections remain.

12.3 Efforts

12.3.1 Questionnaires, Pamphlets, and Public Informational Meetings

As part of this project the Town has implemented a number of public education and outreach avenues to help inform the users about some of the issues facing the Town Sewer Department. Questionnaires and informational pamphlets were mailed to all sewer users regarding illicit connections, and while the response rate to the questionnaires was relatively low, if nothing else the pamphlets and questionnaires helped inform users of some the concerns that DPW personnel have about sump pumps and other illicit connections.

The Town has also held several public informational meetings to present the findings of this report. In addition, many other public meetings were held to inform the public and receive public feedback regarding some of the ongoing projects.

13. CONCLUSIONS

- The Town has taken many steps to improve the operation and maintenance of the wastewater collection system to mitigate SSO and CSO discharges. In particular, operations modification/reduction of the Town water treatment plant waste discharges and more frequent cleaning of the dual 8-inch inverted siphons beneath the Squamscott River should help mitigate future SSO discharges in the vicinity of "Duck Point".

- The Town has significant I/I especially during storm events where 16 mgd peak flow (main pumping station plus CSO) has been observed since improved pumping station metering was installed in the spring of 2010 and improved CSO metering was installed in December 2010.

- Historical CSO metering, though not reliable, suggests peak flows of 30 mgd (main pumping station plus CSO) during severe flooding events. However, there is evidence that Squamscott River water "back-flowed" into the system during some of these flooding events and it is unclear whether the 30 mgd flows were real or a function of false flows measured in the system due to CSO tailwater effects. New CSO metering is now in place to evaluate future CSO tailwater effects.

- Approximately 60% of the I/I observed during I/I field investigations appeared to be from private sources. Private I/I sources include sump pumps, foundation drains, leaking services, roof leaders, etc. Future projects aimed at I/I reduction must include targeting private I/I mitigation to achieve any significant I/I removal.

- New CSO flow metering has revealed that significant direct inflow sources still appear to be connected to the wastewater collection system and that these direct inflow sources contribute to the CSO events because they generate high peak flows in response to rain. Many of these direct connections are believed to be private roof leaders with traps that prevent identification through smoke testing (as was the case for some buildings in PEA). Identification and removal of these suspected direct connections assist with the reduction of CSO events in the future.

- The Town has aging infrastructure that must be replaced over time to maintain the current level of service, and some of the private I/I mitigation approaches that the Town used for the Jady Hill project may be appropriate to implement in future infrastructure projects (pending the measured success of the Jady Hill Project).

- Certain collection system improvements are needed to maintain the current level of serviced regardless of the long-term CSO strategy that is selected. This is due to the age and condition of the existing sewers.

- Using flow measurements since 2010, the main pumping station peak discharge is approximately 7 mgd. This is slightly less than the WWTF permitted peak design flow of 7.5 mgd, so there is limited opportunity to increase main pumping station pumping rates without reevaluation of WWTF permitted design flows and/or improvements to the facility.

- Using flow measurements since late 2010, it may be possible to pump, equalize, and treat peak flows during storm events to reduce CSO discharges. However, significant capital improvements will be necessary. It may be appropriate to complete those improvements when long-term WWTF needs are identified. Since I/I improvements are necessary, it provides an opportunity for the Town to reduce flows prior to the major capital investments. Also, evaluation of pumping and CSO flow records over a longer time frame is required to refine long term design flows.

- A new WWTF is likely in Exeter due to more stringent permit limits. Reducing I/I prior to the new WWTF will reduce costs.

- Preliminary estimates of the anticipated WWTF upgrade and CSO mitigation efforts capital and O&M costs may result in annual sewer bills above 2% MHI, a common benchmark for affordability. Therefore, a well-managed approach is needed to balance the needs of the projects with affordability.

- The most cost effective approach is to complete I/I improvements to reduce peak flows until the needs for the new WWTF are determined. Confirmation of the success of I/I projects should be evaluated every 2-years and adjusted when necessary. Alternative CSO mitigation strategies (such as pump, equalize and blend) should be re-evaluated when the WWTF is needed.

14. RECOMMENDATIONS

14.1 Flow Monitoring and Measurements

- Provide improved metering at headworks and main pumping station so data can be easily compared with CSO and rainfall data (portions already implemented).
- Add wetwell level to the Flow Assessment Services web-based, pumping station and CSO flow monitoring system so more complete hydraulic evaluations can be performed in the future.
- Provide additional CSO flow monitoring (already implemented in 2010)
- Measure and evaluate the success of the Jady Hill Pilot Project to determine if adjustments in approach are needed.

14.2 Additional Evaluations

- Complete remaining items on 'to-do' list from CDM report (Table 2-2)
- Finish evaluating possible private inflow sources as identified in the CDM report (Appendix, Volume 1, A-7)
- Continue to monitor flows on a daily basis to assess the success of I/I projects and to provide design flows for future WWTF projects.
- Develop a policy for dealing with 'private' infiltration and inflow and update SUO appropriately to tackle removal of private I/I in the system. More proactive enforcement actions by the Town in the future may help eliminate some of the private I/I in the system.
- Perform additional inflow investigations starting in Sewer Basins C & I where the Spring St. Diversion Structure flows indicate significant inflow remains, but ultimately pursue private I/I removal system wide.
- Work with homeowners in pilot areas to remove identified and suspected sump pumps and other sources of inflow identified during the house-to-house study (Figures 5-5, 5-6, 5-7). This should be completed in conjunction with completion of the remaining 2 pilot area projects.
- Further evaluate 'suspect' cross country sewers that cross streams and low-lying areas for inflow during spring high groundwater and heavy rainfall. These are locations where ponded surface water or flooded streams/rivers could submerge manholes without being

easily visible, since they are not on a street. Examples include basin F from MH 228 to MH 201, basin E between Court St and Linden St.

- Evaluate private pump stations for direct connections to inflow sources. Update SUO to require that private pumping stations provide operation, maintenance, and flow records to the Town.

- Perform visual inspections of manholes during wet weather flow. For example, the cross-country interceptor in basin F has been identified as suspect by DPW personnel, and was reported to have significant inflow entering the manholes from MH 210 to 201 by the TV inspection crew. Visual inspections should be made during wet weather to investigate this situation.

- The Town should CCTV inspect 20% of the collection system annually and incorporate findings into a long-term sewer asset management plan.

- The Town should continue smoke and dye testing to identify direct inflow sources and evaluate CSO outfall modifications to mitigate the risk of Squamscott River "back-flow" into the system.

14.3 Capital Projects

- Complete the balance of the Pilot Projects. Two areas remain. The pilot areas will further refine the Town's approach with private I/I.

- Begin annual budgeting for sewer manhole rehabilitaiton. Manholes should be repaired as prioritized by the manhole inspections (Table 9-1 & 9-2). However, manhole rehabilitation should also be coordinated with routine sewer main evaluations so that rehabilitated manholes are not replaced as part of sewer infrastructure management projects. We have included a $300,000 allowance to address manhole deficiencies identified in this report.

- Complete I/I Improvements in a prioritized system to reduce I/I (and CSOs).

- Provide capital budgeting for ongoing sewer collection system improvements. We have included a $26,000,000 allowance to address I/I peak flows and sewer deficiencies identified in this report. Once I/I projects are no longer being pursued or needed, the Town should budget $500,000 to $650,000 per year to maintain the current level of service. This $500,000 to $650,000 per year budgetary estimate is based on the approximate 48.5 miles of Exeter wastewater gravity collection system and an assumed replacement metric of approximately $1,000,000 to $1,300,000 per mile of gravity sewer divided over 100-years. However, an asset management plan would refine these figures and help prioritize projects. Please note that this $500,000 to $650,000 per year budgetary figure only includes mainline upgrades to maintain the current level of service and does not include private sewer separation required to effectively remove the private I/I in the system. Projects that include comprehensive improvements and private sewer separation, such as the Jady Hill Project, can cost $3,000,000/mile.

- Reassess the recommendations of this report at a frequency of no less than every 2-years and when a new WWTF is needed. When the WWTF is needed consider designing the new WWTF with equalization storage to accommodate storm flows. It may be appropriate at that time to construct a 'high-flow' pumping station and new force main with flow equalization. Schedule to be defined by affordability and the schedule of the anticipated WWTF upgrade (Figure 14-1).

- See attached *Suggested Sewer Implementation Schedule and Cash Flow* (Table 14-1)

FIGURE 14-1

Exeter I/I and CSO

Decision Matrix
February 9, 2010

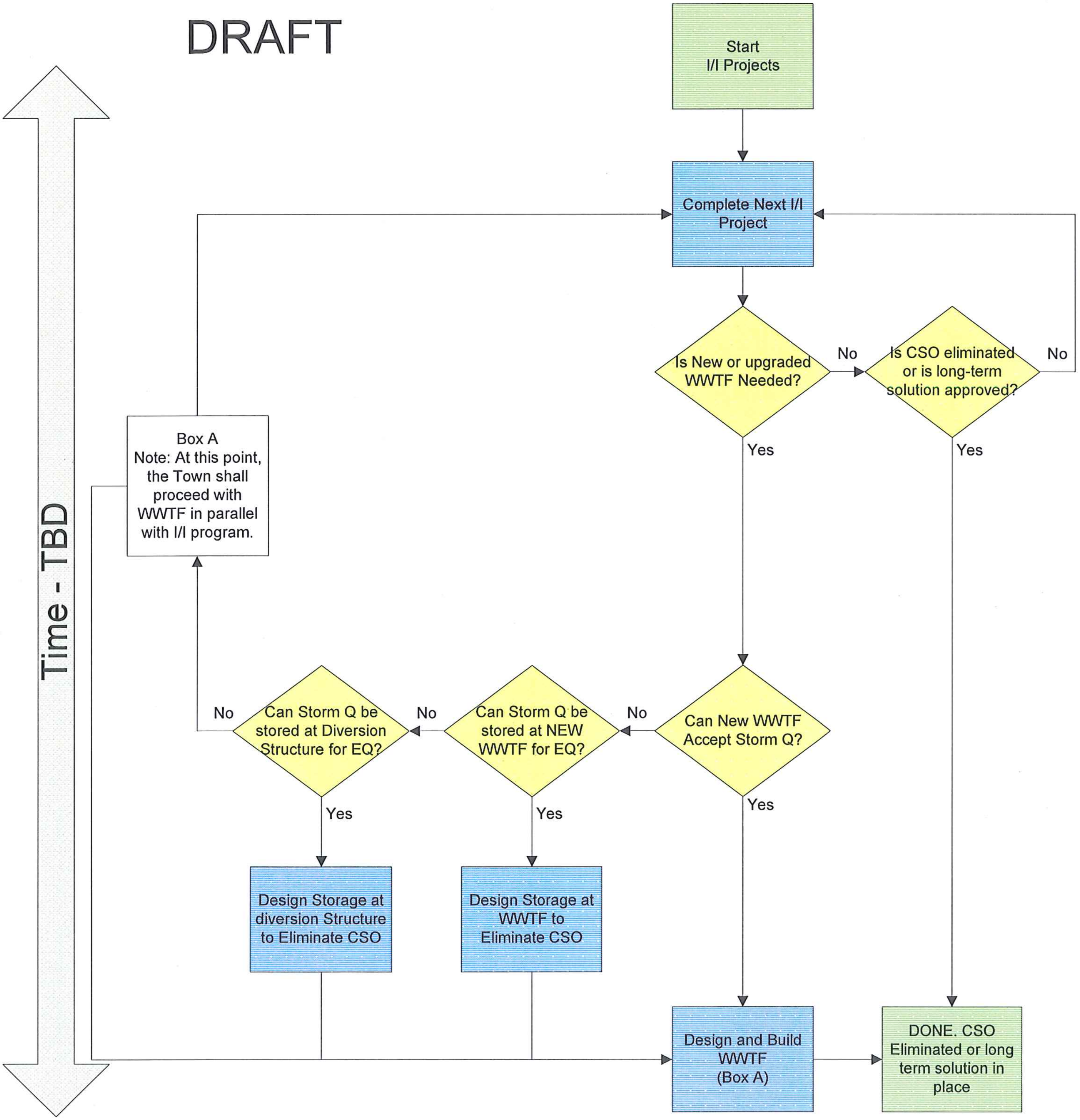

Table 14-1

Suggested CSO LTCP Sewer Implementation Schedule and Cash Flow - 5-Year Plan

Sewer Improvement Project/Program	Total Budgetary Cost [3,4,5]	2013	2014	2015		2016	2017		2018	2019	2020	2021	2022	2023	2024	2025	2026	2027
		Project Year																
WWTF Improvements [2]					Confirmation			Confirmation										
Facility Plan	$375,000	$375,000																
Design	TBD		TBD	TBD														
Construction	TBD					TBD	TBD											
Phase I On-Line (8 mg/L) [9]	TBD								*									
Non-point Nitrogen Evaluations and Controls [9]	TBD	TBD	TBD	TBD		TBD	TBD		TBD	TBD	TBD	TBD	TBD	TBD				
Phase II On-Line (3 mg/L)- If Necessary, TBD [9]	TBD																	
Long Term CSO Control Plan																		
Submit Report		*																
Jady Hill Project [1,6]																		
Construction	$3,436,000	$3,436,000																
Evaluation/Assessment	$20,000		$20,000															
Additional Evaluations/Monitoring/TV/Implementation	$515,000		$265,000			$250,000												
Manhole Rehabilitation			$60,000	$40,000		$11,000	$11,000											
Downing Ct./Westside Drive [1,8]																		
Design	$40,000			$40,000														
Construction/Implementation	$500,000			$500,000														
Evaluation/Assessment	$40,000					$40,000												
Subtotal Additional I/I Projects LTCP Driven		***$3,436,000***	***$345,000***	***$580,000***		***$301,000***	***$11,000***											
Sewer Collection CIP [7]																		
Portsmouth Avenue Sewer	$940,000	$940,000																
Lincoln Street Sewer	$196,000		$196,000															
Sewer Line Replacement	$1,700,000			$850,000			$850,000											
Subtotal Existing CIP Sewer Projects		***$940,000***	***$196,000***	***$850,000***		***$0***	***$850,000***											
ANNUAL TOTAL LTCP AND EXISTING SEWER CIP (WWTF COSTS NOT INCLUDED)		***$4,376,000***	***$541,000***	***$1,430,000***		***$301,000***	***$861,000***		***$TBD***	***$TBD***	***$TBD***	***$TBD***	***$TBD***	***$TBD***	***$TBD***	***$TBD***	***$TBD***	***$TBD***
		5-YEAR LTCP COMMITMENT (I/I) $3.34M Jady Hill + $1.24M Additional							***10-YEAR PHASE II LTCP Costs TBD if needed***									

Notes:

1. Pilot areas should be done initially to further refine private I/I approach.
2. A new WWTF may be needed due to revised permit limits. The schedule for this new facility is not known at this time. The above schedule should be reviewed/adjusted when the schedule and cost of the new WWTF is known.
3. All expenditures and projects indicated above are pending Town authorization through voting.
4. Reassessment of affordibility and approach of the program should be performed at a minimum of every 2-years and during critical milestones such as pilot area implementation, WWTF upgrade, and main pumping station improvements.
5. Budgetary project costs are present day and have not been escalated for the time value of money.
6. Jady Hill Project costs includes sewer related expenses only.
7. Sewer Collection CIP is a draft plan only.
8. Assumes enforcement only in Westside Drive.
9. Schedule is based on US Environmental Protection Agency (EPS) draft Administrative Compliance Order (ACO).

15. <u>CIP PLANNING</u>

15.1 Town Current CIP

The Town provided a work plan showing various water projects that were recommended by previous reports by others, replacement of existing 4"cast iron water mains based on DPW local knowledge, and Exeter Fire Department recommendations. However, the Town is currently facing a major WWTF upgrade project and the cost impacts of this WWTF upgrade may make future I/I removal or CSO mitigation projects unaffordable for the users. The Town must prioritize and reevaluate all sewer CIP projects for affordability as capital and O&M costs for the WWTF upgrade become better defined.

As previously noted it is recommended that the town budget a minimum of $500,000 to $650,000 annually for mainline replacement projects to maintain the current level of service. This budgetary allowance is calculated to replace the existing 48.5 miles of sewer over the next 100-years. The Town currently budgets approximately $400,000 to $500,000 per year for collection system improvements.

15.2 Pilot Projects

As discussed in previous sections of this report, sewer rehabilitation/replacement projects with private I/I separation is the most cost effective approach for long-term CSO mitigation. However, the Town must re-evaluate the I/I removal success of these projects through flow monitoring after completion (pilot areas) to confirm whether this approach is achieving effective flow reductions. We have included a budget of $19,000,000 to rehabilitate/replace the 22 project areas identified in this report with an additional $7,000,000 to separate other private services that may be outside the project areas for a total of $26,000,000. The additional costs for private sewer separation will include drainage improvements in many cases.

15.2.1 Pilot Area 1 – Westside Drive

This pilot area includes approximately 5,500 feet of sewer main including TV Area 16 (approximately 1,100 feet of sewer main) that was identified to have disproportionately high I/I during flow isolation. The sewer mainlines in this area were composed of PVC. However, during CCTV inspection no mainline defects were observed and I/I was observed only from private services. Four (4) leaking manholes were also observed in this pilot area. During house inspections UE also identified 14 sump pumps that discharge to the sewer or to an unknown location and 18 properties denied access for inspection.

For the purposes of budgetary cost estimating (Tables 9-7) UE included 300' of public storm sewers and private service separation (new private sewer services and storm drain services) for the 6 services within TV Area 16. However, UE recommends working with all homeowners to remove illicit connections, and, since I/I appeared to be from private sources in this area, the Town could consider an "enforcement-only" approach in this pilot area to reduce capital costs. Regardless of the approach used by the Town to mitigate identified this private I/I, it is recommended that the Town monitor springtime sewer flows after program implementation using continuous flow monitoring and compare the results to pre-implementation flow monitoring performed as part of this study.

15.2.2 Pilot Area 2 – Downing Court

This pilot area includes approximately 6,500 feet of sewer main including TV Area 2 (approximately 1,450 feet of sewer main) that was identified to have disproportionately high I/I during flow isolation. The sewer mainlines in this TV Area 2 were composed of AC. During CCTV inspection mainline defects included 2 cracks, break-in services, and defective capped services. Twenty one (21) leaking manholes were also observed in this pilot area. During house inspections UE also identified 12 sump pumps that discharge to the sewer or to an unknown location and 17 properties denied access for inspection.

For the purposes of budgetary cost estimating (Tables 9-7) UE included 7 point repairs, 800' of public storm sewers, and private service separation (new private sewer services and storm drain services) for the 19 services within TV Area 2. However, UE recommends working with all homeowners to remove illicit connections.

This pilot area appears to be good candidate for sewer rehabilitation. However, we recommend that the Town CCTV and evaluate the conditions other sewers (some are VC) from an infrastructure management perspective even though those sewers did not exhibit excessive I/I during flow isolation, so additional budgeting may be necessary. This evaluation/CCTV should be performed prior to rehabilitating the manholes. It is recommended that the Town monitor springtime sewer flows after program implementation using continuous flow monitoring and compare the results to pre-implementation flow monitoring performed as part of this study. Summary of recommended plan of action is as follows:

- CCTV remaining ~5,000' of gravity sewer within pilot area not included in TV Area 2
- Evaluate condition of the remaining ~5,000' of sewers (no action/rehabilitate/replace)
- Consider rehabilitation of manholes pending sewer main evaluation
- Work with homeowners to remove illicit connections
- Weight benefits of drain extensions to facilitate illicit connection removal pending sewer main evaluation

15.2.3 Pilot Area 3 - Jady Hill Pilot Area

UEI recommended this project prior to issuance of the 2010 draft of this report and the project is currently under construction. This pilot area includes approximately 5,900 feet of sewer main including TV Area 4 (approximately 3,600 feet of sewer main) that was identified to have disproportionately high I/I during flow isolation. UEI recommended that it be used as a pilot to evaluate the success of public and private I/I removal since significant private service I/I and deficient sewer services were observed. It is recommended that the Town monitor springtime sewer flows using continuous flow monitoring after construction is complete and compare the results to pre-construction flow monitoring performed as part of this study.

15.3 Gravity Sewer Collection System Projects

As discussed in previous sections of this report, the Town has significant VC sewers estimated to be over 100 years old in some cases and have reached the end of their useful life. Replacement (or rehabilitation if feasible) is recommended from an infrastructure management perspective in addition to I/I removal aimed for CSO control and to help mitigate WWTF treatment costs. In addition, the cost effective analysis in regard to I/I removal favors rehabilitating less deteriorated sewers due to the high cost of repairing sewers in very poor condition. Also, the $26,000,000 program budget includes significant private service investment and new drainage, the cost of which could be reduced depending on the Town's approach to enforcement.

We have flagged some of these recommended projects and separated them into two categories:

- Large scale comprehensive infrastructure improvement projects that encompass areas for planned water system improvements, and
- Smaller scale projects aimed to reduce I/I that have lower cost and are more cost effective from an I/I removal standpoint that the Town may want to tackle using Town forces

It should be noted that projects from TV areas 1, 10, and 11 were high ranking for cost-effective I/I removal because there were no mainline defects observed in these areas during TV inspection. Public costs indicated for these projects (Tables 9-5, 9-6, 9-7) are for portions of the sewer service in the ROW, but all the I/I observed during CCTV inspection was from private services. In addition, the projects listed below are suggestions only based on information provided by the Town. The Town could sequence sewer projects should they overlap better with other CIP projects and after reevaluation of Pilot Projects.

15.3.1 Large-Scale Comprehensive Infrastructure Improvements

The following projects (identified by TV area number) are recommended because they exhibited a disproportionally high level of I/I when compared to the rest of the system and the sewers are in need of attention from an infrastructure management perspective. The budgetary opinion of costs only includes sewer work within the areas that were TV inspected and does not include other infrastructure work that might be required within/adjacent to the project area.

TV Area 8 – Ridgewood Terrace

This project area consists of AC sewers with many observed break-in service connections that were observed leaking I/I both around the break-in connection and from the private service lines themselves. Although AC pipe is often a good candidate for lining rehabilitation, replacement may be necessary in this case due to the number of break-in connections requiring point repairs and apparent leaking private sewer service work that is required. Rehabilitation versus replacement and project costs should be refined during design. This project completely overlaps "Tier 1, Tier 2, and looping" water infrastructure projects, so is a good comprehensive infrastructure project. A budgetary project summary is as follows:

- Sewer Project Length = 1,300'
- Budgetary Sewer Project Cost (Including Private Service Separation) = $700,000

TV Area 12 – Pine Street/Front Street

This project includes VC sewers with significant defects, apparent leaking private sewer service work and alignment issues that would likely require mainline replacement. However, rehabilitation versus rehabilitation and project costs should be refined during design. This project completely overlaps with "Tier 1 and Fire Department" water infrastructure projects so is a good comprehensive infrastructure project. A budgetary project summary is as follows:

- Sewer Project Length = 1,500'
- Budgetary Sewer Project Cost (Including Private Service Separation) = $1,100,000

TV Area 15 – Front Street/Lincoln Street/Gill Street

This project includes primarily VC sewers with significant defects and alignment issues that would likely require mainline replacement in some areas. However, rehabilitation versus rehabilitation and project costs should be refined during design. This project generally overlaps with "Tier 2, CDM recommended, and Fire Department" water infrastructure projects, so it is a good comprehensive infrastructure project. A budgetary project summary is as follows:

- Sewer Project Length = 3,600'
- Budgetary Sewer Project Cost (Including Private Service Separation) = $2,900,000

It should be noted that there are additional water CIP projects that the Town identified on Union St., School St., Parker St., Arbor St., Garfield St. in the immediate vicinity of the recommended sewer project. It is recommended that the Town evaluate those additional sewers if the infrastructure project scope expands to include those water projects. Addition of the sewers in these water areas could approximately double the budgetary sewer project costs above.

TV Area 6 (Portions) – Appledore Avenue, Star Avenue, Langdon Street

This project area consists of AC sewers with many observed break-in service connections that were observed leaking I/I both around the break-in connection and from the private service lines themselves. Although AC pipe is often a good candidate for lining rehabilitation, replacement may be necessary in this case due to the number of break-in connections requiring point repairs and apparent leaking private sewer service work that is required. Rehabilitation versus replacement and project costs should be refined during design. This project completely overlaps "Tier 2" water infrastructure projects, so is a good comprehensive infrastructure project. A budgetary project summary is as follows:

- Sewer Project Length = 1,300'

- Budgetary Sewer Project Cost (Including Private Service Separation) = $700,000

TV Area 21 – Ashbrook Road
This project area consists of AC sewers and I/I from private service lines. The I/I portion of the work includes private service separation. However, the mainline pipes exhibited signs of spalling, which although not leaking at this time should be rehabilitated in the future. This project does not overlap with any known existing CIP projects. A budgetary project summary is as follows:

- Sewer Project Length = 1,200'
- Budgetary Sewer Project Cost (Including Private Service Separation) = $600,000

15.3.2 Small-Scale I/I Reduction Improvements Projects

The following projects (identified by TV area number) are recommended because they exhibited a disproportionally high level of I/I when compared to the rest of the system and the sewers are in need of attention. The budgetary opinion of costs included only includes sewer work within the areas that were TV inspected.

TV Area 3 – Hampton Road
This project area consists of defective AC sewers and I/I from private service lines. The I/I portion of the work includes private service separation. This project does not overlap with any known existing CIP projects. A budgetary project summary is as follows:

- Sewer Project Length = 240'
- Budgetary Sewer Project Cost (Including Private Service Separation) = $150,000

TV Area 7 – Holly Court
This project area consists of defective AC sewers and I/I was observed leaking I/I both around the break-in connection and from the private service lines themselves. The mainline pipes exhibited signs of spalling, which although not leaking at this time should be rehabilitated in the future. Although AC pipe is often a good candidate for lining rehabilitation, replacement may be necessary in this case due to the number of break-in connections requiring point repairs and apparent leaking private sewer service work that is required. Rehabilitation versus replacement and project costs should be refined during design. This project only overlaps with CIP water looping projects. A budgetary project summary is as follows:

- Sewer Project Length = 600'
- Budgetary Sewer Project Cost (Including Private Service Separation) = $300,000

15.3.3 Private Inflow Investigation and Removal Projects

Identification and removal of private I/I sources could lead to a significant reduction in peak flows in the system and CSO occurrences. However, it is believed that many of the remaining inflow connections are private, and that some private connections (as observed in PEA) have traps that prevent identification through smoke testing. Therefore, it is recommended that future private inflow investigations employ flood and dye testing techniques. We have included a $515,000 allowance for this task for the next 5 years. However, cost estimates should be refined as sources are identified and separation designs proceed.

15.4 Summary of Annual Cash Flow Requirements

As shown in previous sections of this report annual cash flow requirements for the Town Sewer Department are significantly impacted by the cost of the anticipated WWTF upgrade and CSO mitigation projects. The suggested 5-year CSO LTCP cash flow requirements were previously shown (Table 14-1).