Executive Summary

Introduction

This Water Master Plan is the third phase of an effort that began in 1999 with the initial development of the Water System Master Plan (Phase I) and was continued in 2003 with the development of the Water Supply Master Plan and Madbury WTP Evaluation (Phase II). The Phase I Master Plan consisted primarily of an evaluation of deficiencies in the distribution system, the condition of the water system infrastructure, and a water quality evaluation. The Phase II Plan included an in-depth investigation to determine the sustainable yield of the system's water sources.

The City has completed a number of projects recommended in the Phase I and Phase II plans, including the Spinney Road Tank, the Madbury Water Treatment Plant, and the Constitution to Congress Street water main.

The objectives of the current study include:

- Water storage tank condition assessment
- System growth and water demand projection update
- Distribution system hydraulic model update and hydraulic analysis of the system
- Capital improvements recommendations/capital improvements plan update

The Portsmouth water system is supplied by one surface water source and nine active wells. The 4.5 MGD-capacity Madbury Water Treatment Plant (WTF) treats water from the Bellamy Reservoir. A transmission main carries water from the Madbury WTF and three Madbury wells through Madbury, Durham, and Newington to the Newington Tank and Booster Pump Station, from where it is pumped into the Portsmouth distribution system.

The majority of the distribution system is served from two pressure zones: the Main Pressure Zone (nominal hydraulic grade 171 ft MSL), and the Pease Pressure Zone (nominal hydraulic grade 230 ft MSL). A few customers are served directly from the transmission main at a hydraulic grade of approximately 140 ft MSL. There are currently five active distribution storage tanks. The Hobbs Hill and NHANG tanks are located in the Pease Pressure Zone, and the Lafayette Road and Spinney Road Tanks are located in the Main Pressure Zone. The Newington Tank is located at the downstream end of the transmission main and operates at the transmission main hydraulic grade. The Newington Booster Pump Station pumps water from the transmission main and Newington Tank to the Main Pressure Zone. The Newington Booster Pumps Station can also pump to the Pease Pressure Zone. The Harrison, Haven, and Smith wells are located in and pump directly into the Pease Pressure Zone. The Collins Well, Greenland Well, and Portsmouth Well No. 1 are located in and pump directly into the Main Pressure Zone.

Population and Water Demand Projections

Population Projections

Population projections for areas served by the Portsmouth Water system through 2030 are presented in Table ES-1. This information is based on the most recent available population projections by the New Hampshire Office of Energy and Planning (NH OEP) which was completed in January 2007. The population projections presented in Table ES-1 represent only the population expected to be served by the Portsmouth Water system; the percentage of the total population in each Town expected to be served is indicated in the table. These projections reflect an average population increase of approximately 0.8% per year over the 20 year planning horizon.

TABLE ES-1
Population Projections for the Portsmouth Water System

Municipality	% of Population Served	Population Served by Portsmouth System				
		2010	2015	2020	2025	2030
Greenland	50	1,775	1,854	1,934	2,034	2,113
New Castle	100	968	1,013	1,049	1,086	1,113
Newington	95	715	750	776	802	827
Portsmouth	100	20,779	21,900	22,637	23,514	24,290
Rye	5	265	275	282	289	296
Madbury	8	142	148	153	159	164
Durham	1	5	5	5	5	5
Total		24,648	25,945	26,837	27,888	28,809

Historical Water Demands

Figure ES-1 shows historical water demands for the Portsmouth water system, including both the Pease and main Portsmouth pressure zones.

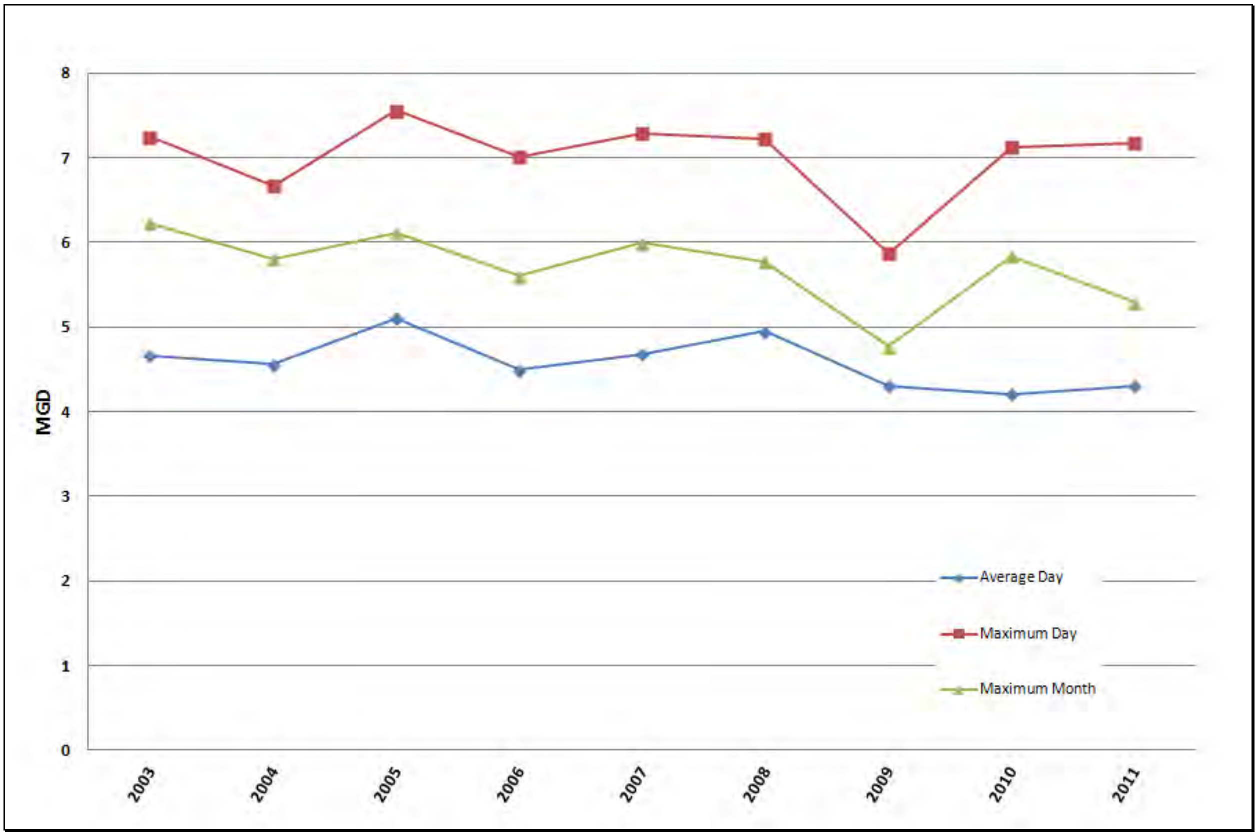

FIGURE ES-1 Historical Water Demands

As indicated in the figure, system-wide average day demand has leveled off at approximately 4.25 mgd. Maximum monthly demands have dropped from just over 6 MGD to approximately 5.5 MGD over the last two years, with a drop-off during 2009 likely due to the fact that the summer of 2009 was very cool and wet, leading to less use of water for irrigation purposes. With the exception of 2009, Maximum Day Demand has remained consistent at slightly above 7 MGD.

Water Demand Projections

Potential for growth varies in different parts of the Portsmouth Water System service area. System-wide water demand projections, as well as projections for portions of the system where growth is anticipated, are provided in the following paragraphs.

Pease Tradeport

The Pease Development Authority anticipates a 10% increase in building square footage and a 20% increase in employees at the Pease Tradeport over the next decade. We anticipate that the water demand at the Tradeport will increase by about 15% over the next ten years, and another 15% in the decade beginning in 2020. These projections are shown in Table ES-2.

TABLE ES-2

Pease Tradeport – Projected Water Demands through 2030

Year	Average Day Demand (mgd)	Maximum Month Demand (mgd)
2011	0.46	0.74
2015	0.49	0.79
2020	0.52	0.85
2025	0.56	0.91
2030	0.60	0.97

Greenland

Potential areas for the growth and expansion of the water system in Greenland include the Breakfast Hill Road area and the Post Road area south of Breakfast Hill Road. There is also the possibility of an interconnection with the Aquarion system by installing a 1.5 mile water main in Post Road. We are adopting a projection of 5% new water demand for Greenland every five years. Table ES-3 summarizes the Greenland demand projections:

TABLE ES-3
Greenland – Projected Water Demands through 2030

Year	Average Day Demand (mgd)	Maximum Month Demand (mgd)
2011	0.16	0.19
2015	0.17	0.20
2020	0.17	0.21
2025	0.18	0.22
2030	0.19	0.23

Madbury, Durham, Newington, New Castle, and Rye

No increase in demand large enough to represent a significant impact to the overall system demand is anticipated in Madbury, Durham, New Castle, or Rye.

City of Portsmouth

Water usage in the City of Portsmouth has gone down since 2003. Future increase in demand in Portsmouth is likely to be the result of redevelopment with lower demand uses replaced by higher demand uses. The following areas are expected to experience redevelopment:

- The Lafayette Road corridor is a prime location for future redevelopment; zoning rules allow for increased density in this area.
- The Community Campus area is zoned Industrial and has a number of lots that could be developed or combined for redevelopment
- The Brewery Lane area along Islington Street has the potential for redevelopment
- The Northern Tier area is slowly being redeveloped and there are pending projects under consideration there.

Projected System-Wide Demands

Based on projected demand increases in Greenland and Pease and some increase due to redevelopment in Portsmouth, we project that system-wide water demand will increase at approximately 1% per year. The following table summarizes the anticipated Average and Maximum Day through 2030:

TABLE ES-4
Projected Water Demand for the Portsmouth Water System through 2030

Year	Average Day Demand (mgd)	Maximum Day Demand (mgd)	Maximum Month Demand (mgd)
Average '04-'11	4.59	7.02	5.71
2015	4.78	7.31	5.94
2020	5.02	7.68	6.24
2025	5.28	8.07	6.56
2030	5.55	8.48	6.90

Available Water Supply

Sustainable Yields

We analyzed the withdrawals from the City's sources utilizing monthly data for 2003 to 2011. This data was analyzed for the months that the sources were actually in service (for example, the Collins Well has had periods where it has been offline for maintenance). The average and maximum monthly pumpage was assessed for each source. The 75th percentile of average pumpage is taken as the likely sustainable yield of the supply source. This data was compared with the 2003 Weston & Sampson Master Plan Update data. The Table ES-5 presents a summary. Appendix B includes all of the monthly pumpage data for reference.

TABLE ES-5
Pumpage Data and Likely Sustained Yield of Portsmouth's Water Supply Sources

2003 to 2011 Pumpage Data	WTF Finished Water	Madbury Wells	Greenland Well	Port #1 Well	Collins Well	Haven Well	Smith Well	Harrison Well[6]	TOTAL Sources	MGD
Total Operating Months [1]	108	108	108	108	85	105	91	67	108	
Total Pumpage (MG) [2]	7,612	2,481	1,670	1,261	485	699	447	331	15,010	
Average Monthly Pumpage (MG) [3]	70	23	15	12	6	7	5	5	139	
Max Month Pumpage (MG)	109	37	22	18	12	15	11	10	192	
75% Month Pumpage (Total MG)	84	27	20	13	7	8	6	6	171	
75% Month Pumpage (Average GPM)	1,909	735	454	301	159	180	142	132	4,012	5.78
W&S Safe Yield (GPM) [4]	1,736	559	460	227	153	534	163	134	3,966	5.71
T&B Likely Sust. Yield (GPM) [5]	1,736	647	457	264	156	534	153	133	4,080	5.87

Notes:

1. Total Operating Months includes all months the source of supply was in operation and pumping at a close to normal capacity. Some months show minimal pumpage and are likely due to well maintenance or low water demand. These months were dropped from the analysis.
2. Total Pumpage includes the total water pumped for all the months the source was considered to be fully operational.
3. Average Monthly Pumpage includes the Total Pumpage divided by the Total Operating Months
4. Water Supply Master Plan and Madbury WTP Evaluation Report, Weston & Sampson, June 2003 and Updated Assessment of Bellamy Reservoir Yield, 2008.
5. Average of the 75% Average Day Pumpage and the W&S Safe Yield GPM except:
   - Madbury WTF safe yield is assumed to be 2.5 MGD per the W&S Bellamy Reservoir Assessment
   - The Haven Well pumpage history includes some years where the well flow was restricted by an agreement with the Pease Air Base; therefore, the calculated yield of 534 GPM is the likely safe yield of this source
6. The Harrison Well was placed into service in May 2006 after rehabilitation of the well and pump facilities.

Margin of Safety Analysis

The Margin of Safety is defined as the available water supply divided by the system demand. Table ES-6 provides an overview of the historical margin of safety analysis for the Portsmouth Water System.

TABLE ES-6
Portsmouth Water System - Margin of Safety Analysis

		Average Day					Maximum Month			
Year	Incremental Demand	Demand	Available Water (MGD)		Margin of Safety		Demand	Available Water	Peaking Factor	Margin of Safety
	(MGD)	(MGD)	(1)	(2)	(1)	(2)	(3)	(1)		
Historical										
2003		4.66	5.68	2.91	1.22	0.62	6.23	5.68	1.34	0.91
2004	-8.3%	4.30	5.68	2.91	1.32	0.68	5.29	5.68	1.23	1.07
2005	15.5%	5.09	5.68	2.91	1.12	0.57	6.19	5.68	1.22	0.92
2006	-13.7%	4.48	5.87	3.10	1.31	0.69	5.56	5.87	1.24	1.06
2007	4.2%	4.68	5.87	3.10	1.25	0.66	5.99	5.87	1.28	0.98
2008	5.3%	4.94	5.87	3.10	1.19	0.63	5.77	5.87	1.17	1.02
2009	-14.8%	4.30	5.87	3.10	1.36	0.72	4.77	5.87	1.11	1.23
2010	-2.4%	4.21	5.87	3.10	1.40	0.74	5.85	5.87	1.39	1.00
2011	2.3%	4.30	5.87	3.10	1.36	0.72	5.29	5.87	1.23	1.11
Average		4.55	5.87	3.10	1.29	0.68	5.66	5.87	1.24	1.04

(1) Sustained yield based on 2012 analysis
(2) 24h/day pumping with largest source off line (Madbury WTF)
(3) Maximum month based on actual system data

It is common practice to assess the Margin of Safety for a water system with its largest source off line. In this instance the table above assesses this margin with the Madbury Water Treatment Facility off line during an average day. It does show that losing this source for an extended period of time would be difficult. However, a more likely scenario is for the WTF's output would be reduced during periods of drought. In such a scenario, the City, with proper management mechanisms and the implementation of an Emergency Action Plan, should be able to meet demands.

Additionally, as stated above, during dry periods the City should be able to rely more on its groundwater sources. The safe yield analysis of the Bellamy Reservoir Watershed Yield Update in 2008 shows that, though low flow periods can occur for up to four months, recovery is very quick. With this in mind, it is feasible to assume that the groundwater sources could be pumped for a period of time at their maximum capacity. Once precipitation occurs and the reservoir refills, the WTF could likely be returned to its 4 mgd flow and the groundwater sources could be reduced so that they are able to recharge and recover. Utilizing this operating strategy we analyzed the maximum monthly withdrawals from each source to assess the overall capability of the system under this scenario. The following table shows the max pumpage capability of each source per our analysis.

TABLE ES-7
Sustained Yield vs. Maximum Yield of Portsmouth's Water Supply Sources

Source	Sustained Yield (mgd)	Maximum Yield (mgd)	Max vs. Sustained (mgd)
Madbury WTF	2.50	4.00	+1.50
Madbury Wells	0.93	1.21	+0.28
Greenland Well	0.66	0.71	+0.05
Portsmouth Well	0.38	0.58	+0.20
Collins Well	0.22	0.40	+0.18
Haven Well	0.77	0.77	+0.00
Smith Well	0.22	0.35	+0.13
Harrison Well	0.19	0.33	+0.14
TOTAL	**5.87**	**8.35**	**+2.48**

The following figure shows the current sustained yield of the City of Portsmouth's water sources compared with the current average day demand and the projected 2030 average day demand. It also shows the Maximum Yield and the theoretical Maximum Yield with the largest source off line. Refer to Section 1.5 of the report for a discussion of safe yield.

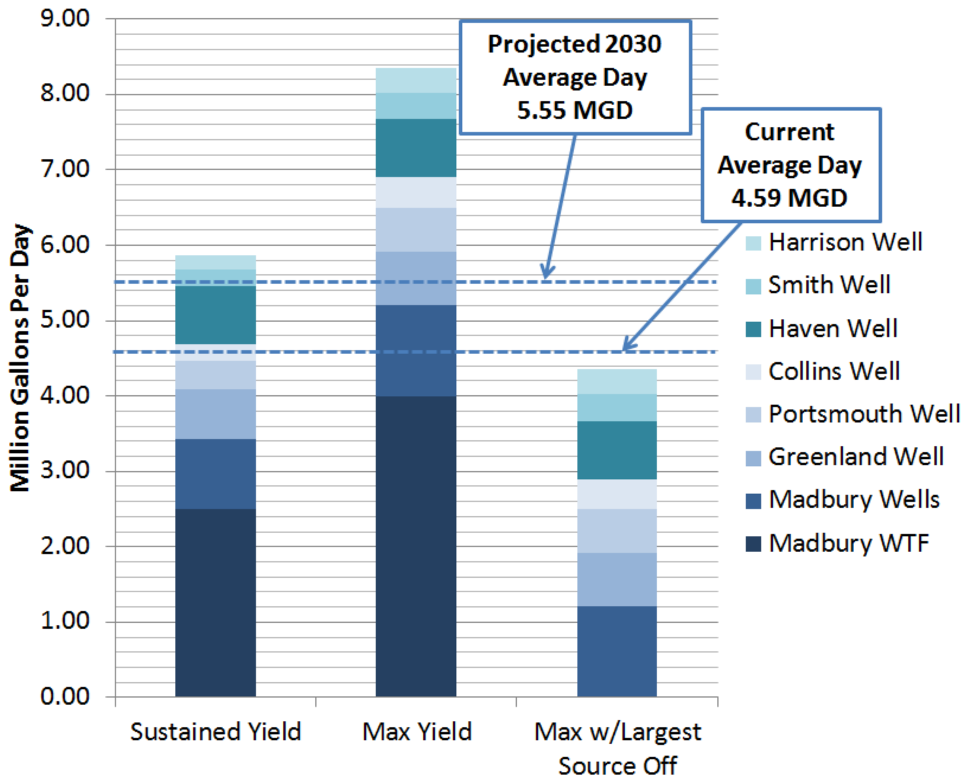

FIGURE ES-2

Water System Supply Yields vs. Average Day Demands

Storage Capacity Evaluation

There are three components to consider when evaluating needed storage in a system:

- Equalization storage
- Fire protection storage
- Emergency storage

Equalization storage represents the amount of water needed to satisfy peak demands during the course of the day and can be estimated based upon historical diurnal demand data. Fire protection storage represents the amount of water needed in the event of a major fire and is generally estimated based on Insurance Service Office (ISO) recommendations. Emergency storage represents the amount of water needed in the event of a short-term water system emergency, such as a supply source off-line or a major water main break.

Main Pressure Zone

Useable storage capacity in the Main Pressure Zone currently consists of the Lafayette Road Tank and the Spinney Road Tank. The Newington Tank storage is also available to the main pressure zone but requires pumping via the Newington Booster Station. The Osprey Landing Tank is not in service and was not considered in this evaluation.

The useable storage in the system above 20 psi, calculated by adding 46 feet to the highest point in the service system, is presented in Table ES-8.

TABLE ES-8
Useable Storage in the Main Pressure Zone

Tank	Total Storage (gal)	Useable Storage (gal)
Newington Tank	1,500,000	1,500,000
Lafayette Road Tank	7,500,000	2,266,000
Spinney Road Tank	1,000,000	1,000,000
Osprey Landing Tank	200,000	0
Total	8,700,000	4,766,000

For the Main Pressure Zone, equalization storage was calculated as the difference between Peak Hour Demand and Max Day Demand flow rates for a period of 8 hours. Fire Protection storage was calculated as 3,500 gpm fire flow for a period of 3 hours. Emergency storage was calculated as the difference between the Max Day Demand and the firm supply capacity for a period of eight hours. Firm capacity for the Main Pressure Zone is defined as the total supply capacity from sources with backup power, with the largest source (the Madbury WTF), out of service. Storage recommendations for the Main Pressure Zone are summarized in Table ES-9.

TABLE ES-9
Main Pressure Zone Storage Capacity

Item	2011	2020	2030
Recommended Storage			
Equalization	680,000	795,000	874,000
Fire Protection	630,000	630,000	630,000
Emergency	913,200	1,253,000	1,487,000
Total	2,223,200	2,678,000	2,991,000
Existing Storage Capacity			
Newington Tank	1,500,000	1,500,000	1,500,000
Lafayette Road Tank	2,266,000	2,266,000	2,266,000
Spinney Road Tank	1,000,000	1,000,000	1,000,000
Total	4,766,000	4,766,000	4,766,000
Surplus/(Deficit)	2,542,800	2,088,000	1,775,000

As indicated in the table, there is a surplus of storage in the Main Pressure Zone, and it is anticipated that the existing storage volume will provide a surplus through 2030.

Pease Pressure Zone

The Pease Pressure Zone storage capacity recommendations were based on the same methodology as the Main Pressure Zone evaluation discussed previously. Storage capacity recommendations for the Pease Pressure Zone are summarized in Table ES-10. Connecting parts of Newington and Greenland to the Pease Pressure Zone was recommended for consideration in this study. In calculating the Pease Pressure Zone storage capacity, it was assumed that parts of Newington and Greenland would be connected to the Pease Pressure Zone in the future.

TABLE ES-10
Useable Storage in the Pease Pressure Zone

Tank	Total Storage (gal)	Useable Storage (gal)
Hobbs Hill Tank	366,000	366,000
NHANG Tank	366,000	366,000
Total	732,000	732,000

TABLE ES-11
Pease Pressure Zone Storage Capacity – Newington and Greenland Connected to Pease Zone

Item	2011	2020	2030
Recommended Storage			
Equalization	290,000	330,000	370,000
Fire Protection	630,000	630,000	630,000
Emergency	0	0	0
Total	920,000	960,000	1,000,000
Existing Storage Capacity			
Hobbs Hill Tank	366,000	366,000	366,000
NHANG Tank	366,000	366,000	366,000
Total	732,000	732,000	732,000
Surplus/(Deficit) w/ Existing Hobbs Tank	(188,000)	(228,000)	(268,000)
Replacement Hobbs Tank Size to Provide Recommended Storage	554,000	594,000	634,000

As indicated in the Table, the recommended storage for the Pease Pressure Zone exceeds the existing capacity. Replacement of the existing Hobbs Hill Tank is under consideration. If the tank is replaced, a new tank with a minimum capacity of 634,000 gallons is recommended to meet the projected storage capacity needs through 2030.

Storage Tank Conditions Assessment

The Hobbs Hill, Spinney Road, Lafayette Road, and Newington Tanks were inspected by Utility Service Company in June 2012. The Hobbs Hill Tank was found to have deteriorated coatings, and complete rehabilitation of the tank as soon as possible is recommended if the tank is to remain in service. Minor items were identified on the Spinney Road, Lafayette Road, and Newington tanks for which immediate repairs are recommended. A summary of recommendations is provided in Table ES-12.

TABLE ES-12
Tank Rehabilitation Cost Summary

Tank	Description of Work	Rehabilitation Costs
Hobbs Hill Tank[1]	Complete rehabilitation recommended	$800,000 to $900,000
Spinney Road Tank	Repair antenna mount, replace entry hatch & safety climb system	$4,900
Lafayette Tank	Replace vent screen, re-grade rip-rap at overflow discharge, repair FAA lights, provide lockable gate on ladder. Re-inspect in 3 years	$4,700
Newington Booster Tank	Clear soil encroachment around foundation, replace vent screen, provide fall protection device on ladder, provide second hatch	$6,800
Total Cost for all Recommended Tank Improvements[2]		**$816,400 to $916,400**

Notes:
1. Estimated costs provided by Utility Services Company for the Hobbs Hill Tank totaled $654,000 and did not include engineering, specification and contract generation. Therefore, for the purposes of CIP planning it is estimated that the final cost of this rehabilitation would range from $800,000 to $900,000.
2. The estimated costs provided by Utility Services Company are based on current 2012 pricing.

Distribution System Assessment

A distribution system model was prepared based on the City's GIS database. The model was used to evaluate distribution system hydraulics and water quality.

Hydraulic Analysis

A system wide fire flow analysis was performed assuming maximum day demand conditions. Several areas of the system were identified as having deficient available fire flows. Areas that have a significant number of hydrants with model predicted available fireflow <1,000 gpm include:

- Northern portions of the Main Pressure Zone, including the majority of Newington
- Atlantic Heights
- North Mill Pond area
- Sherburne Road area
- Sections of New Castle
- The majority of Greenland

Water Quality Analysis

A system-wide water age evaluation was performed assuming average day demand conditions. The model predicts that relatively high water age (>100 hours) occurs in New Castle, the southern part of the main pressure zone, and at the periphery of the system in Greenland and Newington. The highest system water age is expected to occur in the southern part of the main pressure zone in areas that are influenced by the Lafayette Tank.

Disinfection byproducts are products of reactions of chlorine with natural organic matter (NOM) that is present in all natural waters. Surface waters such as the water produced by the Madbury WTF typically have higher concentrations of NOM compared to groundwater; consequently, the potential for forming DBPs is greater in the Madbury WTF water compared to the system's groundwater sources. The reactions that form DBPs including the regulated trihalomethanes (TTHM) and haloacetic acids (HAA5) are slow and may continue for several days after chlorination, until either the chlorine residual or the reactive natural organic matter is depleted. As the time after chlorination increases, the concentrations of TTHM and HAA5 increase and chlorine residual decreases. Thus, high water age is associated with high DBP concentrations and low chlorine residuals.

Compliance with the Stage 2 Disinfectant/Disinfection Byproduct Rule is currently based on the locational running annual averages (LRAA) of TTHM and HAA5 concentrations. LRAAs are the annual averages at each sampling location in the distribution system. This accounts for spatial variations in DBP exposure because the annual average at each sampling location cannot exceed the maximum contaminant levels (MCLs) for TTHM and HAA5. The MCLs for TTHM and HAA5 are 80 µg/L and 60 µg/L, respectively. Operational Evaluation Levels (OELs) must also be calculated. The OEL is determined as the sum of the two previous quarter's TTHM or HAA5 result plus twice the current quarter's TTHM or HAA5 result at that location, divided by 4. If an OEL exceeds the MCL for TTHM or HAA5 then the system must conduct an operational evaluation that includes an examination of the treatment and distribution systems' operational practices that may contribute to TTHM and HAA5 formation and steps to minimize future exceedances. A written report of the evaluation must be submitted to the state no later than 90 days after being notified of the analytical results that caused the exceedance(s) and a copy of the report must be made publically available upon request. Starting in the 3rd quarter of 2013 the City will have to assess each individual sample site for compliance rather than averaging the four sites in the system that they currently monitor. This will make compliance with the regulation more difficult.

Tighe & Bond reviewed TTHM and HAA5 monitoring data collected from 2006 through 2012 at seven sampling locations throughout the distribution system. The data showed that the LRAAs at each sampling location were below the MCLs for TTHM and HAA5. However, concentrations of TTHM and HAA5 in a limited number of individual samples were above the MCLs and the OEL calculations were close to the compliance limits for both parameters.

The historic results of the quarterly disinfection byproduct monitoring data are consistent with the water age modeling results, which show high water age in New Castle and the southern part of the main pressure zone. The surface water contribution modeling results showed that New Castle also has a relatively high contribution of surface water, whereas the southern part of the main pressure zone has a relatively low surface water contribution under typical current operating practices. Since the potential for disinfection byproduct formation in surface water is typically higher in surface water compared to groundwater, New Castle would be expected to have relatively high disinfection byproduct concentrations.

Conclusions

Updating the City of Portsmouth's Water Supply Master Plan and Hydraulic Model was an iterative and collaborative effort between Tighe & Bond and City of Portsmouth Water System staff. Numerous meetings and site visits took place over the course of this project, which began in April 2012 and concluded with the submittal of first draft report in November 2012. Subsequent reviews and updates have resulted in this final report submittal to the City by Tighe & Bond.

Throughout this process, the findings of the hydraulic modeling effort, infrastructure review of available sources of supply, pumpage capability at the Madbury Water Treatment Facility, the nine wells, and two booster stations, and an assessment of the type and age of distribution system water mains were reviewed. This, coupled with comprehensive inspections of the City's water storage tanks, resulted in the development of a list of recommended projects for the City's water system. This list was then assessed and prioritized to align with the water system's current and projected capital improvements program.

The following list of project recommendations was therefore developed to provide projects in a reasonable manner that would allow for the City to further study alternatives, where needed, and begin implementation where the project benefits are without question. The following is a brief summary of these projects:

Annual Water Line Replacement:

We recommend that the City continue to replace older and aging water lines in conjunction with their ongoing sewer separation project. Through the bundling of these projects with sewer, stormwater and roadway improvements, the City will realize construction cost savings.

Water Quality Improvements Related to Stage Two Disinfection By-Product Rule:

The City has been planning for the impact of the EPA's Stage Two Disinfection By-product Rule for many years. Compliance with the maximum contaminant levels for two groups of disinfection byproducts (TTHM and HAA5) will be calculated for each monitoring location in the distribution system. This approach, referred to as the locational running annual average (LRAA), differs from current requirements, which determine compliance by calculating the running annual average of samples from all monitoring locations across the system. Compliance monitoring for meeting the locational running average will begin in October 2013. Our analysis of water age throughout the distribution system, distribution tank storage turnover and water quality analysis shows that there is a potential that portions of the system have the potential to exceed the new compliance levels for these parameters. Therefore, we have recommended that the City pursue the design of upgrades of two of their storage tanks, the Newington Booster Station and the Lafayette Road water tanks. This design would include further study of the potential that mixing and aeration at these tanks will reduce disinfection by-products without detrimentally effecting other water quality parameters like chlorine residual. Additionally, further refinement of the City's Integrated Water Supply Management of their surface versus groundwater sources is recommended and discussed in more detail in the recommendations portion of this executive summary and the report.

Maplewood Avenue Waterline Replacement

This project is recommended for inclusion in the City's CIP, however, our hydraulic analysis showed that substitution of a 12-inch water main rather than the proposed 16-inch main would provide similar benefit at a reduced cost.

Osprey Landing Water Tank Demolition

The Osprey Landing Tank is a 200,000-gallon elevated storage tank with a base elevation of 100 feet and an overflow elevation of 170 feet. According to system operators, this tank has been offline for approximately two years. Distribution system improvements over the years in the area of this tank have improved the fire flows and pressure in the area. The tank is due to be painted; therefore, based on the model predictions and the fact that the tank has been off-line for an extended period without significant negative impacts, we conclude that the tank's impact on available fire flow outside the immediate vicinity of the tank is not significant and that the tank could be removed rather than re-painted. Recovery of the tanks metals during demolition may provide payback to the City as well. The City may also want to explore options to sell the tank to a buyer wanting it for another site, thus, repurposing the tank in a sustainable manner.

Hobbs Hill Landing Water Tank Replacement

The Hobbs Hill Tank was inspected on June 11, 2012 as part of this project. The exterior and interior coatings are no longer providing an effective corrosion barrier to the underlying steel surfaces. If the existing were to remain in service, it is recommended that tank be completely rehabilitated as soon as possible to prevent aggressive metal loss of the exposed steel substrate along the interior and exterior surface as a result of the degrading exterior coating. Due to the extensive cost to repair the existing tank, it is recommended that the City of Portsmouth consider replacing it with a new water storage tank. This, coupled with the fact that additional storage in the Pease zone of the water system might be necessary if Greenland and Newington portions of the system are converted to the Pease pressure zone, provides further need to replace this tank with one that has larger storage capacity.

Therefore, we recommend that the City pursue this option and determine the proper sizing of the replacement tank as part of this effort.

New Castle Waterline Improvements

There are numerous water line improvements recommended in this study. A number of these improvements are dependent on the New Castle and Rye Water District for implementation. Prior to proceeding with water line upgrades on the Island we recommend that the City meet with these two Districts and discuss the benefits of the City taking over portions of their infrastructure. The benefits to New Castle would be the elimination of sub-meters that currently restrict flow into New Castle for fire-fighting purposes. In addition, the New Castle Water District would no longer have to fund their own water line improvements, a costly endeavor for a small water system. Likewise, the Rye Water District would see improved fire-fighting capacity in the portion of their system served by the City through a water meter. Additionally, if a new water line were to be constructed into Rye from the existing New Castle water line near the Wentworth Hotel, the Rye portion of the system would have additional redundancy.

As the City proceeds with these projects we recommend that additional flow testing be performed to confirm the improved flows in New Castle. This information will provide the City with additional data so that they will be able to assess and track the progress of these improvements as a means of measurement. This will enable them to direct their future funding toward additional projects on the Island that will provide the most benefit to water system customers.

Water System Pressure and Storage Improvements

This study determined that the low pressure and flows experienced in portions of the south end of Portsmouth, Greenland and outlying areas of Newington could be improved by changing the hydraulic gradient in Greenland and Newington to the Pease zone. This, together with providing a PRV connection to the south end of the Portsmouth system, would provide the necessary improved flow and pressure. Additionally, improvements to the Newington Booster Station pumps are warranted due to their age and condition. Our recommended upgrades to this station include better matching the pumpage from the Madbury Water Treatment Facility and Madbury wells with the flows from the Booster Station to improve operating efficiency.

We recommend that the City proceed with design of these upgrades in FY14 so that their construction might commence in FY15. This design would be coupled with the Hobbs Hill storage tank replacement, as the tank sizing is integral to the overall implementation of these pressure and storage improvements.

Recommendations

This section describes recommended projects. Recommended projects are summarized in Tables ES-13 and ES-14.

TABLE ES-13
Recommended Water System Improvements (Pumping & Storage)

Location / Scenario	Project Description	Estimated Project Cost	Project Objective
Newington			
N-2	Modifications to Newington Tank Inlet/Outlet	$220,000	Water quality, stabilize pressures for customers on transmission main
N-3	Newington Tank Re-Painting & Aeration System	$1,310,000	Water Quality
N-4	Pump Station Modifications including new VFDs	$420,000	Improve reliability and operational flexibility
Portsmouth			
PO-7b	Lafayette Road Tank mixing, spray aeration, and chlorination system (additional evaluation required)	$360,000	Water quality
PO-8	Osprey Landing Tank removal	$100,000	Eliminate tank maintenance
Pease			
PE-2a	Hobbs Hill tank replacement	$2,760,000	Upgrade aged and deteriorated tank, provide adequate storage volume
PE-3	Portable generator for Smith and Harrison Wells	$100,000	Reliability
Sherburne Rd			
S-1	Set Sherburne PRV to allow flow from Pease to main pressure zone	$0	Fire Flow

TABLE ES-14

Recommended Water System Improvements (Water Mains)

Location / Scenario	Project Description	Existing Pipe Diameter (in)	Proposed Pipe Diameter (in)	Length (ft)	Estimated Project Cost	Project Objective
Newington						
N-1	Connect Newington to Pease	NA	8	1,400	$760,000	Fire flow, increased pressure
New Castle						
NC-1	Remove meter pits/check valves/replace small diameter main	4,6 + valves, meters	8	100	$100,000	Reliability, Fire Flow, replace aging pipe
NC-4	Replace water main on Wild Rose Lane	6	8	2,600	$810,000	Reliability, Fire Flow, replace aging pipe
NC-7	Wentworth Road water line	8	12	650	$340,000	Fire flow
NC-14	Connect to Rye Water District Line across bridge on Wentworth Road + Replacing Wentworth Road Main	8	12	1,500	$2,640,000	Reliability, Fire Flow, replace aging pipe
Greenland						
G-1	Connect Greenland to Pease + Upgrade Greenland Well + New PRV on Ocean Road	NA	12	700	$600,000	Improved pressure, fire flow, water quality
Portsmouth						
PO-1b	Maplewood and Woodbury Avenue	6, 8	12	7,100	$3,300,000	Fire flow, replace aging pipe
PO-5	Atlantic Heights loop	NA	12	700	$340,000	Fire flow

Newington

Scenario N-1 - Newington Distribution System Improvements

As discussed in Section 3, low pressures and low available fire flow exist in the Nimble Hill/Fox Point area of Newington. Connecting this portion of the Newington system to the Pease pressure zone is recommended as the most feasible and cost effective improvement to address these hydraulic issues. Connecting to the Pease pressure zone would raise the nominal hydraulic grade in the area from 171 ft MSL (the main zone grade) to 230 ft MSL (the Pease zone grade), increasing the static pressure by ~25 psi.

Connecting the Newington area with the Pease pressure zone would require installing approximately 1,400 ft of 8-inch water main from the terminus of the existing main in Nimble Hill Road to an existing 8-inch water main in the Pease zone. A PRV station would be installed in Nimble Hill Road near the intersection of Coleman Drive.

Scenario N-2- Newington Tank Retrofit

Under the existing configuration, approximately 30% of the Newington Tank volume is exchanged per day by allowing the tank's level to fluctuate. Limited volume exchange could result in high water age that could potentially cause water quality deterioration in the tank. Additionally, pressure fluctuations in the transmission main between Madbury and the Newington Tank have created problems for customers connected to the transmission main.

In order to address the pressure fluctuation and potential water age issues, reconfiguring the inlet/outlet piping of the tank is proposed. A new inlet line would direct all incoming water from the transmission main directly into the top of the tank. The existing line from the bottom of the tank would remain in service and would serve as the outlet. With the proposed improvement, all water would have to flow through the tank, significantly reducing the average water age in the tank.

An engineered spray aeration system for removing TTHMs is also recommended. Such a system, when designed correctly, will circulate the water in the tank and discharge it through special aeration nozzles. These nozzles are designed to optimize air to water interaction in order to volatize TTHMs and remove them from the water. The proposed system is expected to significantly reduce the total trihalomethanes (TTHMs) in the water by discharging it through the aeration system. TTHM sampling from the transmission main directly upstream of the tank is recommended to evaluate the potential of an aeration system to improve water quality.

Scenario N-3 - Newington Booster Pumps

A retrofit of the Newington Booster pumps is recommended, including replacing pumps and providing VFDs. We recommend providing new pumps sized to provide the following flow ranges:

- Pump #1: 1,875 GPM
- Pumps #2 and #4: 1,000 to 1,500 GPM
- Pumps #5 and #6: 2,000 to 2,500 GPM

New Castle

The New Castle portion of the water system has a history of pressure and flow deficiencies. This is primarily due to the fact that it is at the furthest end of the water system, has older and undersized water mains and is also sub-metered for the New Castle Water District service territory. Half of the island's water mains are also owned and controlled by the New Castle Water District. The City is currently in discussions with the Town regarding potentially taking over this system. If so, capital improvements of this portion of the system would then be the responsibility of the City. Ultimately, this would be in the best interest of both the City and New Castle as planning, funding and phasing these improvements could be allocated to projects that will result in the greatest benefit to the residents of New Castle.

Scenarios NC-1 and NC-14 – Remove Meter Pits & Rye-New Castle Connection

Parts of New Castle have deficient available fire flow. Several potential distribution system improvements were evaluated with respect to their effectiveness in improving available fire flows. The City's CIP has $3.0 million earmarked for replacement of the water line that runs from Odiorne Point in Rye to the Great Island. Our analysis of other alternatives shows that installing a new connection between the Rye Water District line on Route 1B, replacing the Wentworth Road water line near the Wentworth Hotel, and removing the existing meter pits on either side of the New Castle water system would improve available fire flows more than the replacement of the Odiorne Point water main, and would be less expensive. This alternative would allow additional improvements to the Wentworth Road water main and other water main improvements on the Island within the existing capital budget.

Chlorine residuals are generally low in New Castle as a result of long hydraulic retention time and aging water mains. In general, replacement of aging water mains may improve disinfectant residuals by eliminating corrosion byproducts and biofilms that can consume chlorine; however, the alternatives considered in this study are not expected to reduce hydraulic retention time. Therefore, some localized improvement in chlorine residual may result from replacement of aging water mains, but no significant overall improvement in the New Castle area is expected as a result of the scenarios considered.

We recommend that the City begin discussions with both the New Castle and Rye Water District with respect to these options as they will impact the ability for Portsmouth to implement in a timely manner.

Scenario NC-4 and NC-7 – Replacement of Water Main on Wild Rose Lane and Wentworth Road

This scenario consists of replacing the existing water main in Wild Rose Lane (approximately 2,600 ft) with new 8-inch water main and replacement of approximately 625 ft of 12-inch piping on Wentworth Road from North Gate Road to Spring Hill Road.

Greenland

Scenario G-1– Connect Greenland to Pease Pressure Zone

The Greenland portion of the water system has areas with low available fire flow and low pressure. Our analysis indicates that the most effective and economical alternative for improving available fire flow and pressure in Greenland is connecting this portion of the system to Pease pressure zone.

The proposed connection from Greenland to the Pease system would be provided near the Smith Well. A PRV would be installed on Ocean Road to allow flow from the new Greenland high pressure zone into the Portsmouth main pressure zone.

Portsmouth Main Pressure Zone

Scenario PO-1b Maplewood Avenue and Woodbury Avenue – New 12” Mains

The City’s CIP currently includes $3.0 million in the FY15 budget for a water main replacement project that consists of replacing approximately 7,500 feet of water main on Maplewood Avenue from Woodbury Avenue to Raynes Avenue with new 16-inch cement-lined ductile iron waterline. Our analysis indicates that the proposed 16-inch main provides a significant benefit in available fire flow in the immediate vicinity of the new main, but does not provide much benefit in the downtown area. Substitution of a 12-inch water main rather than the proposed 16-inch main would provide similar benefit at a reduced cost.

Scenario PO-5 - Atlantic Heights Loop

This scenario consists of installing a new 12-inch pipe to connect Atlantic Heights at Crescent Street to Dunlin Road. While not providing a significant benefit outside the Atlantic Heights area, the modeling results indicate that proposed water main would provide a significant benefit in the area, which currently has available fire flow of less than 1,000 gpm. Since the project is estimated to cost less than $500,000 we recommend that the City include this project as part of their ongoing water main replacement projects.

Scenario PO-8 - Osprey Landing Tank Removal

Based on the model predictions and the fact that the tank has been off-line for an extended period without significant negative impacts, we conclude that the tank’s impact on available fire flow outside the immediate vicinity of the tank is not significant and that the tank could be removed. We recommend that the City proceed with this project.

Scenario PO-7b Lafayette Road Tank Improvements

The Lafayette Road tank has a storage capacity of approximately 7 MG, of which only 2.3 MG is “useable.” As a result of the large volume, the Lafayette Road Tank and surrounding area in the southern portion of the City that is influenced by the tank experience high water age. High water age has the potential to cause water quality problems, including loss of disinfectant residual and excessive concentrations of disinfection byproducts including total trihalomethanes (TTHM) and haloacetic acids (HAA5), which are a concern to the City in light of the new Stage 2 Disinfectants/Disinfection Byproduct Rule.

Recent research demonstrated that spray aeration systems can significantly reduce TTHM concentrations in storage tanks. We recommend that the City perform pilot testing of a mixing, spray aeration and chlorination system for installation next year prior to the summer season. Information gathered during this pilot would be helpful to assess overall water quality mixing in the tank, the effect it has on TTHMs, HAA5s, chlorine residual and overall water quality.

Pease Pressure Zone

Scenario PE-2a - Hobbs Hill Tank Replacement

If the existing tank remains in service, complete rehabilitation is recommended as soon as possible to prevent metal loss as a result of the degrading coatings. The estimated cost to rehabilitate this tank is $800,000 to $900,000. A total storage capacity of 1 MG is recommended for the Pease pressure zone projected to the year 2030. If the Hobbs Hill tank is replaced with a new tank, a minimum useable volume of 634,000 gallons would be required to provide the recommended capacity. Due to the extensive cost to repair the existing tank, and the potential need for additional storage during the planning period covered in this report, it is recommended that the City of Portsmouth consider replacing the Hobbs Hill Tank with a new 0.65 MG water storage tank.

Scenario PE-3 Portable Generator with Quick Connect Hookups for Smith and Harrison Wells

The Smith and Harrison Wells are not equipped with standby power. We recommend that the City consider purchasing a portable generator set capable of running one of these wells during an extended power outage. If both of these sites are upgraded to have electrical quick-connections installed then utilizing a standby power system arrangement like this would provide additional flexibility and redundancy to the system.

Water Supply Management Recommendations

Rye Water District Emergency Interconnection

The New Hampshire Department of Environmental Services commissioned a study in 2006 to examine the potential for mutual aid between ten seacoast water systems. The City of Portsmouth was included in this study. The most feasible interconnection identified for Portsmouth is a connection between the Portsmouth main pressure zone and Rye.

A 4,000-foot length of new 16-inch water line on Lafayette Road between the two systems was modeled, and the proposed interconnection was determined to be feasible from a hydraulics standpoint, noting that flow from the Portsmouth main pressure zone would need to be pumped to Rye, and a PRV would be needed to supply water from Rye to Portsmouth. We recommend that the City of Portsmouth meet with the Rye Water District to explore opportunities to install this connection to provide emergency supply for both systems.

Madbury Well Replacements

The three active Madbury Wells (#2, #3 and #4) have been in service for over 60 years. Based on an evaluation performed in 2012, Well #2 is starting to show signs that the screen may need to be replaced. Though it is possible to install new screens inside existing screens of wells to extend their life, this practice often leads to declines in the well yield. Therefore, instead of installing a new screen, we recommend that the City plan to start a replacement program for these wells, beginning with Well #2. New Hampshire regulations for replacing wells are discussed in more detail in the full text of this report, however, it should be noted that replacing existing wells is a much simpler process than sighting, drilling, testing and permitting a new well.

Bedrock Well Potential for Additional Supply

The City commissioned the firm Emery-Garrett Groundwater, Inc. in 2009 to investigate potential sites for potential bedrock well development. A number of locations were identified. We recommend that the City continue to explore the potential to obtain either ownership or easement agreements at some of the sites identified by Emery-Garrett to continue exploration and identify final locations for potential drilling and permitting of a new large groundwater withdrawal for the water system.

Integrated System Supply and Management Plan

The City has been conjunctively managing their one surface water and nine groundwater sources of supply for many years. Their normal procedure calls for optimizing their surface water source when it has available quantity and good quality. By doing this they are able to rest their groundwater sources so that the aquifers are as recharged as possible and their yields will be maximized and available when either water customer demands go up or the surface water source quantity or quality necessitates reducing the yield on their supply. It is noted that the use of surface water sources, especially during the late summer, may increase the potential for disinfection by-products to form in the water system. As previously mentioned, we are recommending that the City update its water supply management and source protection program. Currently, the City's water system has real-time monitoring of all of its sources of supply. They also have the ability to eventually get real-time customer usage information via their new water meter reading system.

Tighe & Bond recently completed work, together with Comprehensive Environmental, Inc., on the Massachusetts Sustainable Water Management Initiative Pilot Program. As part of that project we identified ways that water systems could utilize available data to assess potential withdrawal limits from multiple sources of supply. These assessments were also used in four Massachusetts water systems, all with surface and groundwater withdrawal capability, or alternative sources, like the City of Portsmouth's water system. The report, which is currently in draft form, recognizes the ability of these water systems to reduce withdrawal impacts by tracking and managing their sources of supply, especially during drought conditions. The report recommendations also included the following guidelines that water systems should implement:

1. Optimization of existing resources;
2. Use of alternative sources;
3. Interconnections with other communities or suppliers;
4. Outdoor water restrictions tied to streamflow [and/or groundwater availability] triggers (e.g., greater restrictions on outdoor watering than is currently applied);
5. Implementation of reasonable conservation measures;
6. Utilization of the New England Water Works Association's Best Management Practice (BMP) toolbox

We recommend that the City develop an Excel-based spreadsheet tool that the water system managers and operators can utilize to track and assess sources of supply over time. This spreadsheet would combine information that is already being gathered by the operators, the SCADA system and regional climate and hydrological data sources into one data set. From this data, past trends can be analyzed and compared to current operations data. A supply versus demand assessment can also be made from this analysis that would enable the City to determine if water restrictions are necessary or other measures needed to augment supply or declare an emergency. The following are a few of the parameters we recommend including:

1. Regional Precipitation data, stream flows, groundwater levels and drought conditions assessment from the New Hampshire Department of Environmental Services: http://des.nh.gov/organization/divisions/water/dam/drought/drought-conditions.htm
2. Bellamy Reservoir Information
    a. Oyster River streamgage data from USGS website to use for Bellamy inflow: http://waterdata.usgs.gov/usa/nwis/uv?01073000
    b. Water level at the dam
    c. Estimate of spillway water going over the dam
    d. Estimate of water flow through the dam's outlet pipe
    e. Daily raw water from Reservoir processed through the Madbury Water Treatment Facility
3. Madbury Water Treatment Facility
    a. Daily raw water
    b. Daily process water used at facility
    c. Daily treated water pumped into the system
4. Well Data
    a. Pump hour run times and rates
    b. Pumping and static water levels
    c. Daily water pumped into system
5. Booster Stations and PRVs
    a. Run times and/or pumpage data for each to assess flows
    b. Portsmouth into Pease or Pease into Portsmouth system flow
6. Storage Tank Data
    a. 8:00 am tank level
    b. Calculation of previous day's level to determine amount of increase or decrease of water storage
7. Water Usage and Demand Data
    a. Monthly billing data from Finance Department
    b. Daily data (if available) from individual customer water meters
    c. Flushing, Fire Use, Known leaks, etc. to determine unaccounted-for water in the system on a rolling 12-month basis

Once this tool has been developed we recommend that the City include this in their Standard Operating Procedures for all water systems staff as a guideline for system operations. We also recommend that water quality parameters be considered as part of these procedures. By tracking TOC, chlorine residual, water temperature and other indicators in the system with additional monitoring equipment it may be possible for the City to also manage its sources of supply such that surface water is utilized more from October through May and groundwater more during the warmer summer months to lower the potential for disinfection byproducts. By adding and tracking all of this information through the use of the Integrated Management Tool, water quality trends will also be tracked and managed better by operational staff.

Executive Summary

Section 1 Population and Water Demand Projections

Section 2 Storage Tank Conditions Assessment

Section 3 Distribution System Assessment

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Appendices

Figures

Tables

Section 1
Population and Water Demand Projections

1.1 Background

The Portsmouth Water System provides drinking water to the City of Portsmouth and portions or all of the Towns of New Castle, Rye, Greenland, Newington, Madbury and Durham. The Pease International Tradeport, located in Portsmouth and Newington, is home to commercial and industrial developments that also have a significant daytime water demand. The purpose of this Section is to evaluate population projections, estimate water demands, and determine the water availability margin of safety through the year 2030. Additionally, a review of past Water System Master Plans was performed to compare those projections with actual water supply availability and demand trends.

1.2 Population Trends and Projections for Communities Served by Portsmouth Water System

Population trends for each community served by the Portsmouth Water System are presented in Table 1-1, which includes the latest 2010 U.S. census data.

TABLE 1-1
Population Trends for Communities Served by the Portsmouth Water System

Town	1960	1970	1980	1990	2000	2010
Greenland	1,196	1,784	2,129	2,768	3,208	3,549
New Castle	823	975	936	840	1,010	968
Newington	1,045	798	716	990	775	753
Portsmouth	26,900	25,717	26,254	25,925	20,784	20,779
Rye	3,244	4,083	4,508	4,612	5,182	5,298
Madbury	556	704	987	1,404	1,509	1,771
Durham	5,504	8,869	10,652	11,818	12,664	14,638
Total	39,268	42,930	46,182	48,357	45,132	47,756

Tighe & Bond contacted the New Hampshire Office of Energy and Planning (NH OEP) since they are the primary planning agency responsible for developing population estimates for New Hampshire communities. NH OEP staff indicated that they are not currently developing new population projections using the latest 2010 Census data, although they may be available in the future. Therefore, we reviewed the last available update to the population projection by NH OEP which was completed in January 2007. Estimated population changes for 5-year periods through 2030 are presented in Table 1-2.

TABLE 1-2
Estimated Population Changes for 5-Year Periods Through 2030[1]

Town	2010 - 2015	2015-2020	2020-2025	2025-2030
Greenland	4.5%	4.3%	5.2%	3.9%
New Castle	4.7%	3.6%	3.4%	2.5%
Newington	4.8%	3.4%	3.3%	3.2%
Portsmouth	3.1%	3.4%	3.9%	3.3%
Rye	3.7%	2.7%	2.6%	2.4%
Madbury	4.4%	3.7%	3.6%	3.0%
Durham	4.6%	4.1%	3.7%	3.0%
Total	3.8%	3.6%	3.7%	3.1%

(1) Source: New Hampshire Office of Energy and Planning, January 2007

By applying the estimated population changes for each five-year period from 2010 to 2030 listed in Table 1-2, we developed a population growth forecast for the communities served by the Portsmouth Water System as shown in Figure 1-1. However, the Portsmouth Water System does not provide water to all areas of the seven communities. Therefore, an additional adjustment to the population forecast is warranted to reflect the water service area.

For the purposes of making adjustments to the population of the seven communities, we referred to the Phase I study by EarthTech (September 2000) which made the following assumptions, which we found to be appropriate for the purposes of our study:

- 100% of the expected future population growth and future residential housing units in Portsmouth, Greenland, New Castle, and Newington will be served by the Portsmouth system.
- Growth in the communities of Rye, Madbury, and Durham is expected to occur outside areas currently served by the Portsmouth system. In Rye, it was assumed that 25% of the projected population will be served by the system in 2030; in Madbury and Durham, it was assumed that 10% of the projected population will be served by the system in 2030.
- No residential growth is expected in the Pease International Tradeport system.

The total population of each community and the adjusted population served by the Portsmouth water system is presented in Table 1-3. The population percentage served was derived utilizing the census data, combined with the known residential customer service breakdown as presented in Table 1-4 of this report.

TABLE 1-3
Estimated Population Served by the Portsmouth Water System through 2030

Municipality	2010	2015	2020	2025	2030
Greenland	1,775	1,854	1,934	2,034	2,113
New Castle	968	1,013	1,049	1,086	1,113
Newington	715	750	776	802	827
Portsmouth	20,779	21,900	22,637	23,514	24,290
Rye	265	275	282	289	296
Madbury	142	148	153	159	164
Durham	5	5	5	5	5
Total	**24,648**	**25,945**	**26,837**	**27,888**	**28,809**

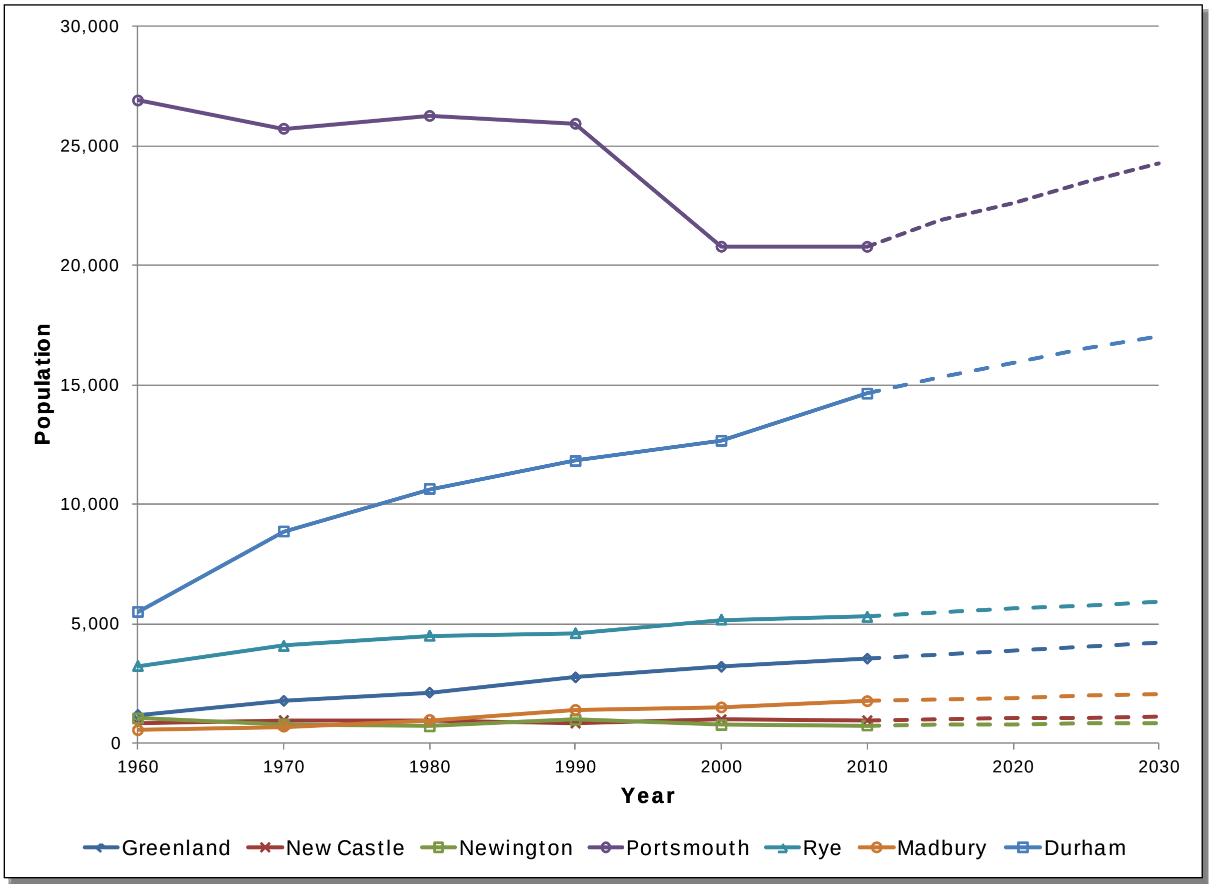

FIGURE 1-1
Historical and Projected Population Growth Trends, 1960 – 2030

1.3 Water Demand Trends and Projections

Historical daily water production records for 2003 through 2011 were reviewed to determine average day demand (ADD), maximum day demand (MDD), and maximum month demand (MMD). Historical demands are presented in Table 1-4. Peaking factors for ratios of MDD:ADD and MMD:ADD are presented in Table 1-5.

1.3.1 Historical Production Water Trends

The following table summarizes the annual water production data from 2003 to 2011 for the combined Portsmouth and Pease water systems:

TABLE 1-4
Historical Water Production for the Portsmouth Water System

Year	Average Day Demand (mgd)	Maximum Day Demand (mgd)	Maximum Month Demand (mgd)
2004	4.56	6.67	5.80
2005	5.10	7.56	6.12
2006	4.49	7.01	5.60
2007	4.68	7.29	5.99
2008	4.95	7.22	5.77
2009	4.30	5.87	4.77
2010	4.21	7.12	5.84
2011	4.31	7.17	5.29
Average	**4.59**	**7.02**	**5.71**

1.3.2 Historical Maximum Month and Maximum Day Peaking Factors

The following table summarizes the maximum day and maximum month average day peaking factors for the combined Portsmouth and Pease water systems.

TABLE 1-5
Maximum Day and Maximum Month Peaking Factors

Year	Ratio MDD:ADD	Ratio MMD:ADD
2003	1.55	1.33
2004	1.46	1.27
2005	1.48	1.20
2006	1.56	1.25
2007	1.56	1.28
2008	1.46	1.17
2009	1.36	1.11
2010	1.69	1.39
2011	1.65	1.22
Average	**1.53**	**1.24**

When compared with the ratios as analyzed by the previous water supply studies, which looked at use from 1997 to 1999, the ratios for MDD:ADD are nearly the same as they were during that period. However, the MMD:ADD ratio has dropped from 1.49 to 1.24, or approximately 20%.

Figure 1-2 shows the combined Portsmouth and Pease water system's pumpage trend from 2003 to 2011. As the graph shows, actual average use for the combined systems has leveled off at approximately 4.25 mgd. Maximum monthly demands have dropped from just over 6 mgd to approximately 5.5 mgd over the last two years, with a drop-off in pumpage during 2009. This trend is likely due to the fact that the summer of 2009 was very cool and wet, leading to less use of water for irrigation purposes.

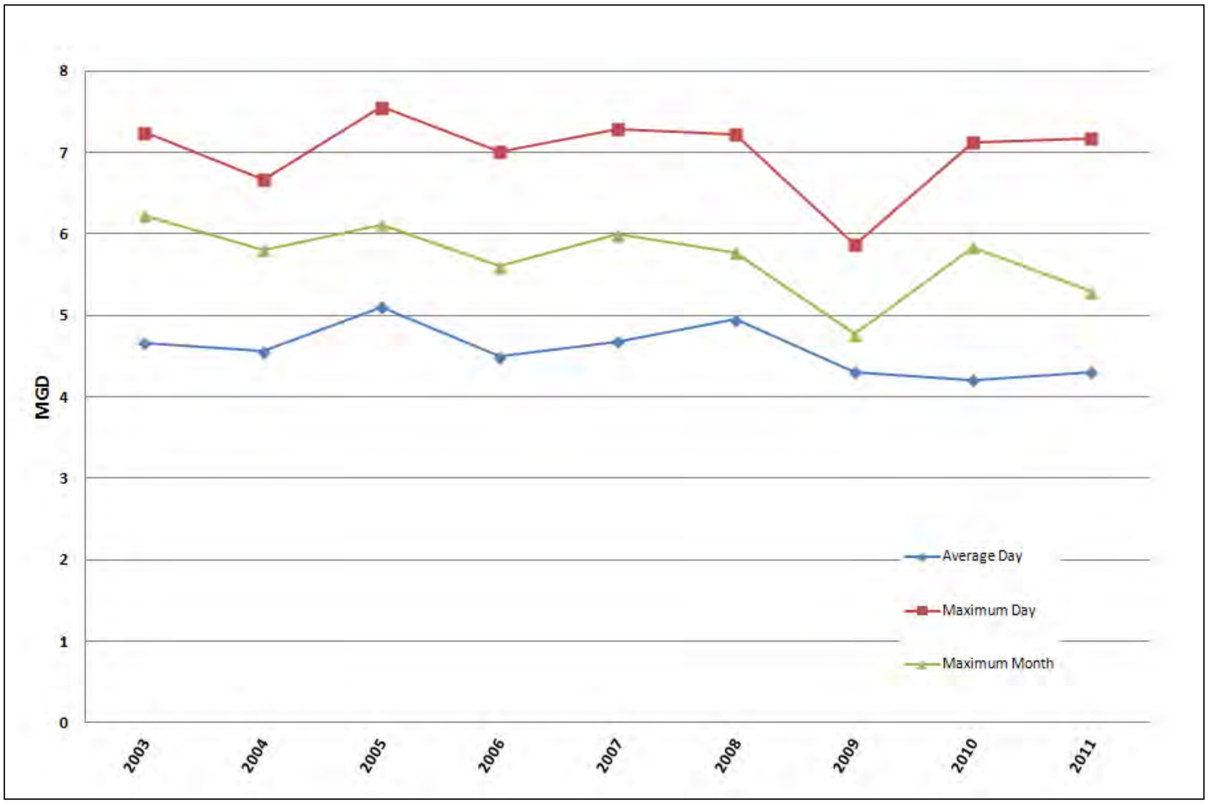

FIGURE 1-2
Historical Water System Pumpage Trend (Combined System), 2003-2011

Figure 1-3 shows the water system pumpage trend for Portsmouth's Main Pressure Zone from 2003 to 2011. As the graph shows, actual average use for this system has leveled off at approximately 4 mgd. Maximum monthly demands are approximately 4.5 to 5.0 mgd, with a drop-off in pumpage during 2009. The Portsmouth demand ratio of average day to maximum day peaks at about 1.36.

FIGURE 1-3
Historical Water System Pumpage Trend (Main Pressure Zone), 2003-2011

The Pease Pressure Zone's pumpage trend from 2003 to 2011 is displayed in Figure 1-4. As the graph shows, actual average use for the combined systems has leveled off just over 0.45 mgd. Maximum monthly demands are approximately 0.75 mgd, with a drop-off in pumpage during 2008 and 2009. The Pease summer demand ratio of average day to maximum day peaks at about 1.67, likely a reflection of the businesses at Pease that have automatic irrigation systems on at their facilities.

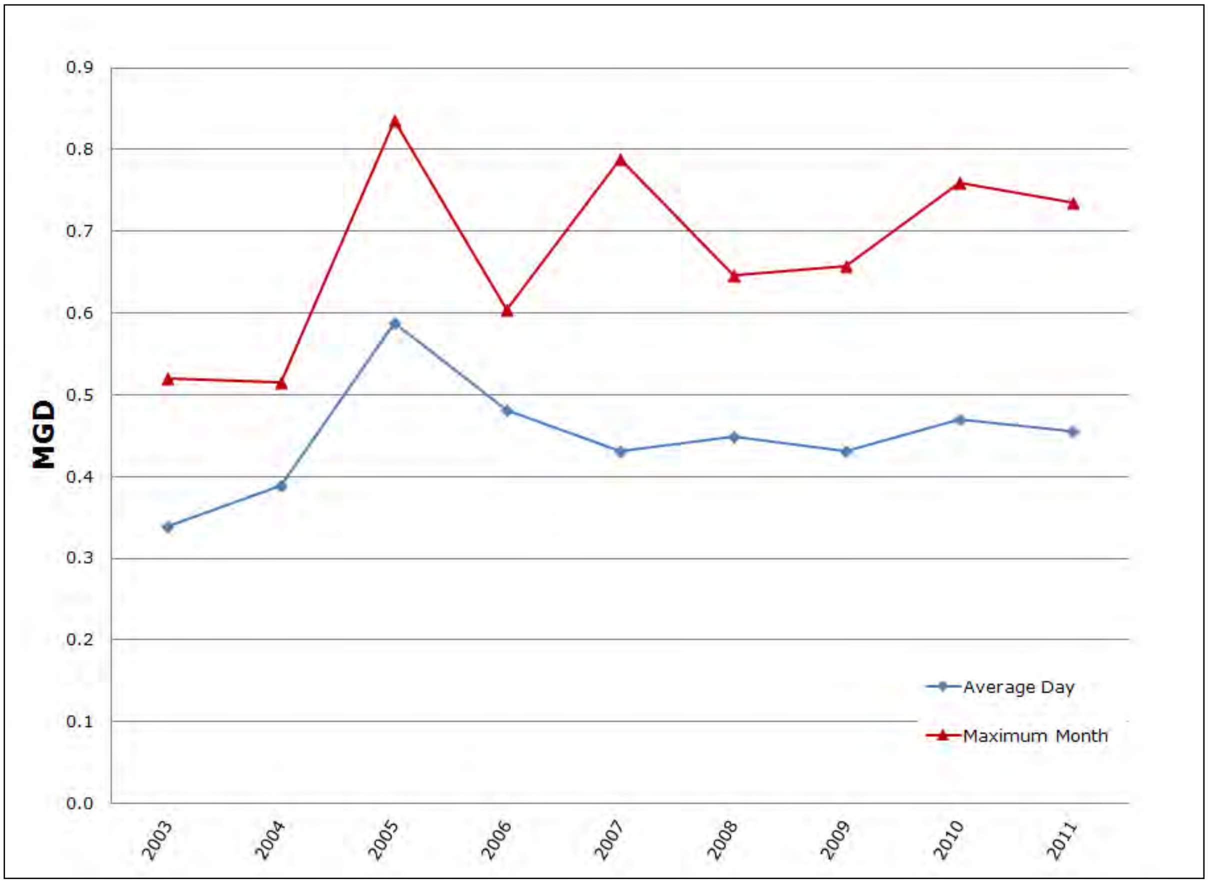

FIGURE 1-4
Historical Water System Pumpage Trend (Pease Pressure Zone), 2003-2011

1.3.3 Seasonal Demand Trends

The following tables summarize the seasonal demand trends utilizing pumpage data from 2003 to 2011. The tables also summarize the non-irrigation season versus irrigation trends for the combined systems and the Portsmouth core system and Pease system. As the data shows, the Pease system data shows it has a greater demand attributed to irrigation needs than the Portsmouth core system.

TABLE 1-6
Portsmouth Water System – Combined System Average Daily Demand (Gallons Per Day)

Month	2003	2004	2005	2006	2007	2008	2009	2010	2011	Average
Jan	4,008,671	3,505,742	4,697,439	4,183,999	4,184,595	3,883,459	4,359,355	4,110,806	3,505,742	4,048,868
Feb	4,132,545	3,720,750	4,852,309	4,089,199	4,344,968	3,836,200	4,347,000	4,163,643	3,720,750	4,134,151
Mar	4,026,451	3,800,710	4,964,056	4,219,625	4,341,528	4,182,705	4,277,516	3,995,548	3,800,710	4,178,761
Apr	3,904,877	3,953,133	4,884,486	4,068,909	4,119,590	5,229,933	3,908,333	3,251,800	3,953,133	4,141,577
May	4,864,764	4,222,613	5,088,801	4,261,522	4,736,682	5,573,456	4,597,161	3,698,677	4,222,613	4,585,143
Jun	5,845,355	4,560,367	4,697,731	4,942,778	5,775,116	5,509,942	4,774,367	4,471,467	4,560,367	5,015,277
Jul	6,192,333	5,294,774	5,827,379	5,598,167	5,702,562	5,663,290	4,713,129	5,844,839	5,294,774	5,570,139
Aug	6,227,660	5,191,506	6,192,972	5,506,635	5,986,996	5,750,581	4,655,710	5,562,387	5,191,506	5,585,106
Sep	4,578,878	4,742,170	5,441,087	4,781,306	5,479,248	5,766,033	4,438,533	4,774,167	4,742,170	4,971,510
Oct	4,176,368	4,582,568	4,521,117	4,209,373	4,156,922	5,077,968	3,947,355	3,694,387	4,582,568	4,327,625
Nov	3,847,816	4,048,408	4,537,158	3,838,713	3,644,981	4,841,633	3,799,633	3,367,233	4,048,408	3,997,109
Dec	4,191,160	4,034,381	5,420,192	4,074,137	3,671,858	3,981,710	3,834,697	3,530,826	4,034,381	4,085,927
Average Month	4,666,407	4,304,760	5,093,727	4,481,197	4,678,754	4,941,409	4,304,399	4,205,482	4,304,760	4,553,433
Maximum Month	6,227,660	5,294,774	6,192,972	5,598,167	5,986,996	5,766,033	4,774,367	5,844,839	5,294,774	5,664,509
Avg. Non-Irrigation Months	4,041,127	3,949,385	4,839,537	4,097,708	4,066,349	4,433,373	4,067,698	3,730,606	3,949,385	4,130,574
Avg. Irrigation Months	5,541,798	4,802,286	5,449,594	5,018,082	5,536,121	5,652,661	4,635,780	4,870,307	4,802,286	5,145,435
Est. Irrig Demand[2]	1,500,671	852,901	610,057	920,374	1,469,772	1,219,288	568,082	1,139,701	852,901	1,014,861
Summer Ratio[3]	1.37	1.22	1.13	1.22	1.36	1.28	1.14	1.31	1.22	1.25
Peak Ratio[4]	1.33	1.23	1.22	1.25	1.28	1.17	1.11	1.39	1.23	1.24

Notes:

1. Highlighted cells indicate summer months with irrigation demand.
2. Estimated Irrigation Demand = Avg. Irrigation Months – Avg. Non-Irrigation Months
3. Summer Ratio = Avg. Irrigation Months / Avg. Non-Irrigation Months
4. Peak Ratio = Maximum Month / Average Month

TABLE 1-7
Portsmouth Water System – Portsmouth Main Pressure Zone Average Daily Demands (Gallons Per Day)

Month	2003	2004	2005	2006	2007	2008	2009	2010	2011	Average
Jan	3,730,084	3,255,777	4,311,614	3,595,557	3,851,433	3,522,717	3,974,323	3,712,516	3,131,032	3,676,117
Feb	3,825,748	3,375,964	4,433,535	3,484,788	3,987,110	3,400,022	3,946,821	3,721,500	3,330,643	3,722,904
Mar	3,753,135	3,496,000	4,520,512	3,671,894	3,976,141	3,760,189	3,904,516	3,580,258	3,450,710	3,790,373
Apr	3,660,087	3,647,233	4,370,650	3,654,456	3,751,590	4,877,400	3,520,133	2,942,067	3,581,567	3,778,354
May	4,574,131	3,805,935	4,558,527	3,843,609	4,351,521	5,169,360	4,107,097	3,229,968	3,854,129	4,166,031
Jun	5,495,355	4,093,400	4,047,482	4,430,657	5,240,249	5,033,106	4,370,133	3,952,167	3,899,233	4,506,865
Jul	5,728,390	4,814,323	5,180,105	5,056,125	5,079,820	5,124,968	4,250,161	5,085,129	4,559,903	4,986,547
Aug	5,708,187	4,675,270	5,458,965	4,926,375	5,199,412	5,119,032	3,998,290	4,944,581	4,519,441	4,949,950
Sep	4,109,644	4,264,253	4,745,228	4,248,106	4,933,382	5,119,867	3,876,133	4,185,033	4,185,282	4,407,436
Oct	3,835,362	4,207,677	3,918,023	3,817,889	3,774,987	4,711,387	3,575,710	3,255,613	4,148,805	3,916,161
Nov	3,522,216	3,673,105	3,944,984	3,526,646	3,454,784	4,466,467	3,542,667	3,044,500	3,769,753	3,660,569
Dec	3,903,455	3,673,787	4,583,910	3,749,234	3,367,857	3,602,935	3,417,374	3,177,632	3,764,811	3,693,444
Average Month	4,320,483	3,915,227	4,506,128	4,000,445	4,247,357	4,492,288	3,873,613	3,735,914	3,849,609	4,104,563
Maximum Month	5,728,390	4,814,323	5,458,965	5,056,125	5,240,249	5,169,360	4,370,133	5,085,129	4,559,903	5,053,620
Avg. Non-Irrigation Months	3,937,036	3,655,322	4,416,892	3,704,439	3,876,623	4,252,666	3,758,889	3,347,405	3,657,317	3,845,177
Avg. Irrigation Months	4,975,388	4,410,984	4,669,960	4,495,831	4,845,570	5,021,672	4,014,086	4,284,505	4,262,533	4,553,392
Est. Irrig Demand[2]	1,038,351	755,663	253,068	791,392	968,947	769,006	255,197	937,099	605,216	708,215
Summer Ratio[3]	1.26	1.21	1.06	1.21	1.25	1.18	1.07	1.28	1.17	1.19
Peak Ratio[4]	1.33	1.23	1.21	1.26	1.23	1.15	1.13	1.36	1.18	1.23

Notes:

1. Highlighted cells indicate summer months with irrigation demand.
2. Estimated Irrigation Demand = Avg. Irrigation Months – Avg. Non-Irrigation Months
3. Summer Ratio = Avg. Irrigation Months / Avg. Non-Irrigation Months
4. Peak Ratio = Maximum Month / Average Month

TABLE 1-8

Portsmouth Water System – Pease Pressure Zone Average Daily Demands (Gallons Per Day)

Month	2003	2004	2005	2006	2007	2008	2009	2010	2011	Average
Jan	278,587	249,965	385,825	588,442	333,161	360,742	385,032	398,290	374,710	372,751
Feb	306,797	344,786	418,773	604,411	357,857	436,179	400,179	442,143	390,107	411,248
Mar	273,316	304,710	443,544	547,731	365,387	422,516	373,000	415,290	350,000	388,388
Apr	244,789	305,900	513,836	414,453	368,000	352,533	388,200	309,733	371,567	363,224
May	290,633	416,677	530,273	417,913	385,161	404,097	490,065	468,710	368,484	419,113
Jun	350,000	466,967	650,250	512,121	534,867	476,836	404,233	519,300	661,133	508,412
Jul	463,943	480,452	647,274	542,042	622,742	538,323	462,968	759,710	734,871	583,592
Aug	519,474	516,235	734,007	580,260	787,584	631,548	657,419	617,806	672,065	635,155
Sep	469,233	477,917	695,859	533,200	545,867	646,167	562,400	589,133	556,888	564,074
Oct	341,006	374,892	603,094	391,484	381,935	366,581	371,645	438,774	433,763	411,464
Nov	325,599	375,303	592,174	312,067	190,197	375,167	256,967	322,733	278,654	336,540
Dec	206,903	360,594	836,281	324,903	304,001	378,774	417,323	353,194	269,570	383,505
Average Month	339,190	389,533	587,599	480,752	431,397	449,122	430,786	469,568	455,151	448,122
Maximum Month	519,474	516,235	836,281	604,411	787,584	646,167	657,419	759,710	734,871	673,572
Avg. Non-Irrigation Months	282,428	330,879	541,933	454,784	328,648	384,642	370,335	382,880	352,624	381,017
Avg. Irrigation Months	418,657	471,650	651,533	517,107	575,244	539,394	515,417	590,932	598,688	542,069
Est. Irrig Demand[2]	136,228	140,771	109,600	62,323	246,596	154,752	145,082	208,052	246,064	161,052
Summer Ratio[3]	1.48	1.43	1.20	1.14	1.75	1.40	1.39	1.54	1.70	1.45
Peak Ratio[4]	1.53	1.33	1.42	1.26	1.83	1.44	1.53	1.62	1.61	1.50

Notes:

1. Highlighted cells indicate summer months with irrigation demand.
2. Estimated Irrigation Demand = Avg. Irrigation Months – Avg. Non-Irrigation Months
3. Summer Ratio = Avg. Irrigation Months / Avg. Non-Irrigation Months
4. Peak Ratio = Maximum Month / Average Month

1.3.4 Customer Demographics and Consumption Trends

The City of Portsmouth recently upgraded their water meter reading system to a radio-based Datamatic Firefly system that transmits water meter data to their Mosaic computer system database. This data is accessed by the City's Public Works and Finance Department staff. The Mosaic data is then transmitted to the City's Pentamation billing system through an electronic file transfer. This is generally done once a month. The Mosaic system data is essentially "real-time," as meter reading data comes into the system either once a day for most meters or hourly for meters that have higher use or leak codes. The water system staff utilizes this data to determine users that might have leaks or meters that are not working correctly. They contact the high consumption users about possible leaks and create service orders for staff to fix any meters that have other error codes like low batteries. This data is not yet tied to real-time monitoring of the system's overall water consumption.

The City replaced many of their water meters when they converted to the Datamatic system from 2008 to 2010, however, there were still a number of meters in the system that were simply upgraded to the Datamatic reading system. During this period of time the City also transitioned from sending out water/sewer bills every four months, or three times a year, to monthly. The general procedure was to have an account converted to monthly billing after the account had a Firefly radio read system installed. This was phased in and completed for all customers by July 2011. This change to monthly reading only impacted the City's residential customers. Commercial and Industrial accounts on the system have always been read and billed on a monthly basis. According to Finance Department staff, there was a period of time during this transition where they had to estimate water meter readings for a number of customers. Some of this was simply due to the reading system changeover and some due to the fact that they started to estimate bills for some customers that would not respond to their request to schedule the meter service so that the Firefly system could be installed on their service. This work was completed by the end of 2010. Therefore, we utilized data provided by the City for 2011 to develop a breakdown of customer usage averages for the water system.

The City of Portsmouth's water system currently serves approximately 8,000 customers. Of these, 238 also have separate irrigation meters. Therefore, for purposes of identifying water use demographics for the system we analyzed 2011 water billing data for Portsmouth's customers based on the following tables which summarize them by the Town the service is located and the category of use:

TABLE 1-9
Portsmouth Water System – Customer Accounts by Town Served (July 2012 data)

Town	Wholesale	Municipal	Commercial	Industrial	Residential	Irrigation	Total
Durham	0	0	0	0	1	0	1
Greenland	0	4	33	2	454	0	493
Madbury	0	0	0	0	2	0	2
New Castle	1	3	8	0	207	44	263
Newington	0	3	76	17	254	0	350
Portsmouth	0	56	852	62	5,862	194	7,026
Rye	1	0	7	0	63	0	71
TOTAL	**2**	**66**	**976**	**81**	**6,843**	**238**	**8,206**

TABLE 1-10
Portsmouth Water System – Annual Customer Water Usage by Town Served (Gallons x 1000)

Town	Wholesale	Municipal	Commercial	Industrial	Residential	Irrigation	Total	% of Use
Durham	0	0	0	0	73	0	73	0.01%
Greenland	0	2,422	17,393	5,040	32,990	0	57,845	4.39%
Madbury	0	0	0	0	5,153	0	5,153	0.39%
New Castle	18,533	1,816	4,217	0	15,042	2,226	41,834	3.17%
Newington	0	1,816	40,057	42,839	18,457	0	103,170	7.83%
Portsmouth	0	33,903	449,064	156,236	425,968	9,815	1,074,985	81.56%
Rye	26,690	0	3,689	0	4,578	0	34,957	2.65%

TABLE 1-11
Portsmouth Water System – Customer Water Demand by Town Served (Gallons per Day)

Town	Wholesale	Municipal	Commercial	Industrial	Residential	Irrigation	Total	% of Use
Durham	0	0	0	0	199	0	199	0.01%
Greenland	0	6,635	47,653	13,808	90,385	0	158,480	4.39%
Madbury	0	0	0	0	14,119	0	14,119	0.39%
New Castle	50,776	4,976	11,552	0	41,211	6,099	114,613	3.17%
Newington	0	4,976	109,746	117,367	50,568	0	282,656	7.83%
Portsmouth	0	92,884	1,230,311	428,043	1,167,036	26,891	2,945,165	81.56%
Rye	73,122	0	10,108	0	12,542	0	95,772	2.65%

The following table provides a summary of the total annual water use by customer category. As the table shows, a majority of water usage in the water system is by businesses. The combined usage of Commercial and Industrial customers totals 46% of the overall system usage.

TABLE 1-12
Portsmouth Water System – Total Annual Water Use by Account Category (July 2012 data)

	Accounts	Total Gallons	MGD	% of Use
Commercial	976	**514,420,544**	1.41	32.7%
Residential	6,843	**497,254,692**	1.36	31.6%
Industrial	81	**204,114,484**	0.56	13.0%
Other Districts	2	**45,374,428**	0.12	2.9%
Municipal	66	**39,956,641**	0.11	2.5%
Irrigation	238	**12,041,028**	0.03	0.8%
TOTAL	8,206	**1,313,161,817**	3.60	**N/A**
Pease Golf Use (unbilled)		10,600,656	5.79	0.7%
Hydrant Flushing		1,500,000	10.21	0.1%
Other known leaks		122,256	19.86	0.0%
Meter Adjustments		18,555,000	39.60	1.2%
Total Billed and other use		**1,343,939,729**	3.68	**85.4%**
Total System Water Produced		**1,572,904,437**	4.31	**N/A**
Unaccounted For Water		**228,964,708**	0.63	**14.6%**

The following graphic provides a visual breakdown of the water usage by category in the Portsmouth water system:

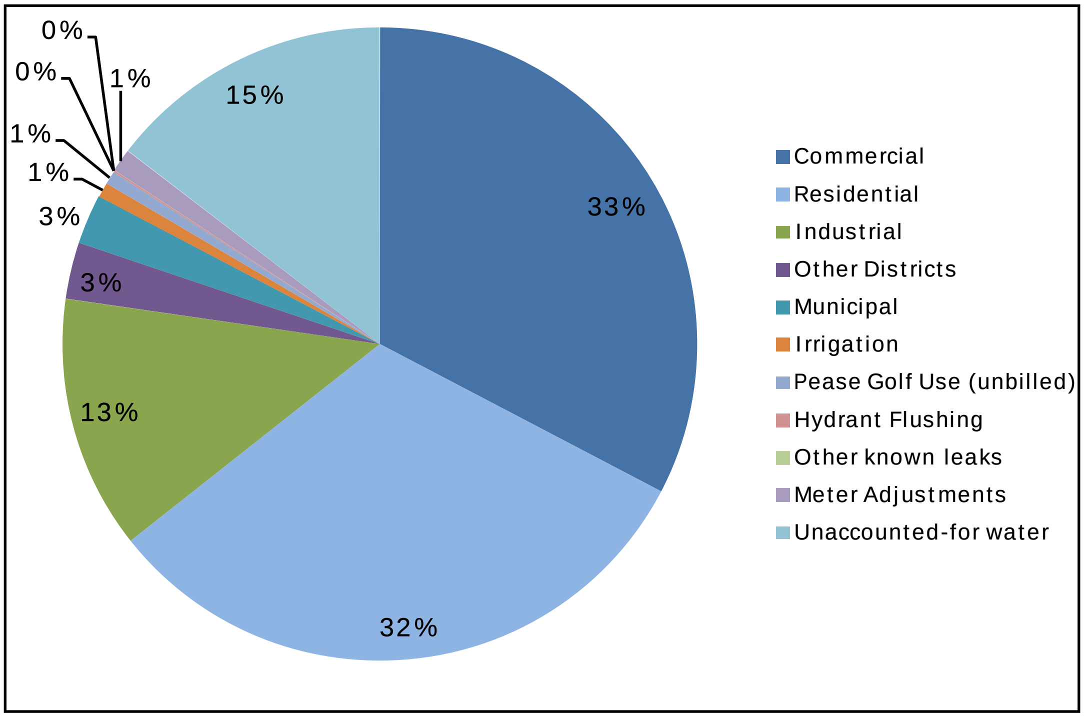

FIGURE 1-5
Customer Water Usage Breakdown (2011)

It should be noted that this is a general breakdown of customer usage by the City's current accounting of customer water use they utilize for billing. The primary reason for this breakdown is that these customer categories are billed based on different types of user rates. Therefore, residential customers include apartments and condominiums, many with multiple tenants after the meter. Additionally, in the past, the City had a separate category for Pease customers. These customers are now simply part of the City's overall customer base. The average day demand is defined as the total water use divided by 365 days. Finally, some users have changed categories and have been re-coded in the system. This applies mostly to the Commercial and Industrial customers in the system.

The following table provides a comparison of the customer classifications as reported in 1999 versus the current 2012 customer base:

TABLE 1-13
Portsmouth Water System – Service Connections by Customer Class

User Category	1999	2012	Difference
Residential	6,648	6,843	195
Municipal	50	66	16
Commercial	1,055	976	(79)
Industrial	57	81	24
Other Utilities	2	2	-0-
Pease	138	0	(138)
Irrigation	0	238	238
Total	7,950	8,206	256

Notes:
1. Other Utilities – New Castle Water District and Rye Water District
2. Pease Customers are now either Municipal, Commercial or Industrial customers
3. Irrigation Customers cannot be considered "new" customers because they are actually existing residential customers that have installed a second meter for their irrigation system per the City's sewer ordinance, allowing them to not have to pay for sewer charges related to irrigation water use.

1.3.5 Unaccounted for Water

According to the American Water Works Association's (AWWA) "Water Resources Planning" Manual M50, "demand projections must include expected system losses that also reflect the changes and remedies that will be implemented to reduce leaks and other losses." Assessing water produced by system sources and comparing that data with customer water usage data over the same time period allows a water system the ability to compare production verses consumption. The difference between those two numbers is what is generally termed as "Unaccounted-for Water." The AWWA manual states that normal unaccounted-for water "would range from 7 to 15 percent, except where very old systems are in place that have not had substantial upgrading." Currently, the Portsmouth water system's percentage, as shown on Table 1-12, is 14.6%, which is within the industry standard and is expected for a system of this size and age. However, our detailed analysis of specific water use of some larger customers revealed that there are likely a number of larger and compound metered services that would benefit from meter downsizing. We recommend that the City review these services and plan for eventual replacement of these meters. We also recommend that the City consider performing a more detailed water audit utilizing guidance in the American Water Works Association's Manual of Water Supply Practices, M36, "Water Audits and Loss Control Programs."

1.3.6 Water Demand Projections

As much of the data in this report shows, the Portsmouth water system is not experiencing major growth, especially with respect to water demands. However, the likely growth of the system water demands over the next twenty year horizon is in the following areas:

1. Redevelopment of existing properties. For example, a site that previously had an automobile dealership with minimal water and sewer demand might be redeveloped for a hotel or condominiums.
2. Build-out at the Pease Tradeport.

3. New developed properties in surrounding communities, like Greenland and Newington.
4. Additional wholesale service to outlying systems such as the Aquarion Water Company of New Hampshire.

In order to develop a level of understanding of what potential developments could impact the water system, we worked with City staff to investigate this potential by the areas served. The following information provides those findings.

City of Portsmouth

We met with Rick Tainter, the City of Portsmouth's Planning Director, on September 7, 2012 to get his input regarding what impact the City's zoning and potential redevelopment might have in the future. He pulled out the City's zoning map and made the following general comments with respect to recent projects that are pending before his department as well as what future development might occur and what it would look like with respect to type of use and density.

The following is a summary of that discussion:

- The Lafayette Road corridor is a prime location for future redevelopment and also allows for more density. The new Service Credit Union building on the south end of Lafayette Road is a good example of the type of facility that is going in along this road. The new 100,000 square foot office building is going to be the corporate headquarters and will combine staff who are currently working in three offices, one in Portsmouth and two others in surrounding communities. The Southgate Plaza redevelopment, which includes a new cinema complex, will likely draw more redevelopment to that area, like new restaurants.
- The Community Campus area is zoned Industrial and has a number of lots that could be developed or combined for redevelopment.
- The Brewery Lane area along Islington Street has the potential for redevelopment.
- The Northern Tier area is slowly being redeveloped and there are pending projects under consideration there.

As the data in Table 1-7 and graphic presented in Figure 1-3 show; water usage in the core Portsmouth system has gone down since 2003. Therefore, to project an increase in water demand for this Master Plan Update over the next 20 years horizon would not be justified. External factors, such as redevelopment that occurs with higher water demand, will play a greater role with respect to the City of Portsmouth's long-term water needs.

Pease Tradeport

According to the City's 2009 Comprehensive Annual Financial Report the top ten employers in the city are listed below. The five highlighted employers are primarily located in the Pease Tradeport:

- Hospital Corporation of America: 1,150
- **National Passport Center: 900**
- **Liberty Mutual: 837**
- City of Portsmouth: 729
- **Lonza: 650**
- **National Visa Center: 550**
- Thermo Fisher Scientific: 350
- Direct Capital: 326
- LabCorp: 225
- **Newmarket International:175**

According to the Pease Development Authority's (PDA) 2012 Update, "as of the Fall of 2012, the Tradeport was home to approximately 250 companies occupying more than 4.4 million square feet of office space and directly employing an estimated 7,000 people." The report noted that construction activity in 2011 continued with:

- Great Bay Community College completing a $10 million renovation.
- Northeast Rehabilitation Health Network completed construction of a 46,000 square foot 33 bed rehabilitation facility.
- The United States Passport Center completed construction of a 25,000 square foot expansion.
- BayRing Communications completed a 15,000 square foot addition.

Additional information from the PDA website reveals that, "Adding the approved future construction of another 665,000+ square feet the current projected total is 4.7 million square feet with a total of 8,400 employees at Pease over the next decade." Utilizing this data, Pease anticipates a 10% increase in building square footage and a 20% increase in employees.

Section 4 of the City's Draft Wastewater Master Plan and LTCP Update performed recently for Portsmouth (Weston & Sampson, Brown and Caldwell) projected growth for Pease through the year 2030. The analysis projected an overall 1% per year growth in employment.

Therefore, utilizing this information, we assumed that the projected future demand for the Tradeport in ten years would likely increase by about 15%. We then used this data to project water demands for the next 20 years. These demands are shown on the following table:

TABLE 1-14
Pease Tradeport – Projected Water Demands through 2030

Year	Average Day Demand (mgd)	Maximum Month Demand (mgd)
2011	0.46	0.74
2015	0.49	0.79
2020	0.52	0.85
2025	0.56	0.91
2030	0.60	0.97

Greenland

Tighe & Bond recently completed the draft of a Sewer Extension Study for the Town of Greenland that encompassed the same approximate area that is served by the Portsmouth Water Division. Current development in Greenland is limited due to the lack of a comprehensive sewer system network. The anticipated build-out within the Study Area was based on current zoning in the Town. For new development and redevelopment of the commercial and industrial zoned areas an amount of total developable acres was calculated to determine the overall potential development footprint within these zones. Sewer demand projections were then calculated based on currently developed properties for the current demand and for all the potential developed properties for the build-out projections. Based on this assessment, the build-out of the area, especially as it relates to potential commercial development and/or redevelopment if a sewer system were constructed, would increase the overall water demand by up to 90%. Again, this assumes total build-out with development that would need substantial water, such as restaurants. This scenario is very similar to the potential for additional water use along the Lafayette Road/Route 1 corridor in Portsmouth.

If a sewer system were to be constructed in Greenland, the system would be owned and operated by the Portsmouth Sewer Division. Therefore, any increase in sewered customer base beyond those that are already served by the water system would have to be reviewed by the City. Anticipated water demands for these projects would be calculated as part of this process and could then be assessed by the Water Division regarding their ability to serve these new customers.

Potential areas for the growth and expansion of the water system in Greenland may include the Breakfast Hill area with construction of a water line down Breakfast Hill Road. Additionally, the Post Road area south of Breakfast Hill Road, though not densely developed at this time, could also offer the potential for new growth in Greenland. In addition, the Aquarion Water Company of New Hampshire's water service territory extends nearly to the Greenland town line. If a line down Post Road were constructed, then providing an interconnection with the Aquarion system has potential for additional wholesale growth due to the fact that Aquarion has deficient supply capacity during peak, dry years. The distance down Post Road for this water main extension is approximately 1.5 miles. Hydraulically, utilizing the general elevation data from Google Earth, the Post Road/Breakfast Hill line starting point elevation is at approximately 100 feet, while the Greenland Town line elevation near the Aquarion system is at approximately 80 feet.

Due to the current unknowns with respect to these future developments, for planning purposes, we are using a conservative projection of 5% new water demand for Greenland every five years. The following table summarizes this demand:

TABLE 1-15
Greenland – Projected Water Demands through 2030

Year	Average Day Demand (mgd)	Maximum Month Demand (mgd)
2011	0.16	0.19
2015	0.17	0.20
2020	0.17	0.21
2025	0.18	0.22
2030	0.19	0.23

Madbury and Durham

It is not likely that there will be any new customers along the Madbury transmission line from Madbury through Durham. Per meetings with Dave Cedarholm, Durham Town Engineer, he does not anticipate any new development that would desire water service. The current transmission main runs through Wagon Hill Farm, a town-owned park.

Likewise, there is currently no development on the horizon for Madbury, thus we are not projecting any increased water use to the Town at this time.

Newington

The Town of Newington is currently almost entirely served by the Portsmouth water system. Expansion into areas not served is not likely due to the fact that the largest tract of undeveloped land includes the Great Bay National Wildlife Refuge. The large commercial customers served along the Piscataqua River are likely to remain. The reconstruction of the Spaulding Turnpike in Newington creates a new configuration for many of the exits and entries for the portions of highway in Newington. One area that might benefit from this new configuration is around the new Exit 3 ramp. A new entry into the Pease Tradeport will be created by this exit and a flyover will also be constructed that allows Travel from Pease back onto Woodbury Avenue. Most of this area is already developed, however, redevelopment with higher consumptive users is possible. Conversely, there are high water users in the area by the river, such as GP Gypsum and PSNH that use a lot of water. These systems could start curtailing water use. Therefore, we are conservatively projecting a 1% increase in annual water consumption over this planning period for Newington.

New Castle

The Town of New Castle is another community that is almost completely built out. With the exception of the Wentworth Hotel complex, the Portsmouth Harbor area, it is predominantly residential. The City of Portsmouth's portion of the water system serves approximately half of the Town of New Castle. The New Castle Water District serves approximately 320 customers and this base is anticipated to remain steady for years to come since their portion of the system is built out.

It is appropriate to note that during the last Water Master Plan update, the Wentworth Hotel complex was being renovated and redeveloped. This has since been built-out and on-line for a number of years so that demand has been absorbed in the overall demand of the Portsmouth system in this analysis.

Rye

The Rye Water District (RWD) serves most of the Town of Rye. The City of Portsmouth's system serves 63 residential and 7 commercial customers in areas that the RWD does not have water mains. According to our correspondence with Ken Aspen, Superintendent of the Rye Water District, they obtain wholesaled service from the City of Portsmouth's water system to serve portion of Wentworth Road, toward New Castle as well as the Elizabeth Lane area. A total of 84 individual customers are served by the District through these wholesaled connections. It is not anticipated that there will be any significant new growth in this portion of the system in subsequent years. This is based on the fact that the Rye Water District has increased its groundwater supply capacity in recent years and the likelihood of high density residential growth is limited by zoning and wastewater disposal and treatment issues.

New Castle and Rye Water District Demands

The following graphics present a summary of the average daily demand for the New Castle and Rye Water Districts. They show the contrast in the type of customer base that is served in these areas. The New Castle demand shows a steady water use with little seasonal or irrigation demand, while the Rye demand has a significant peak during the summer. It is assumed that this is due to lawn watering by the Harborview Drive, Elizabeth Lane and Frontier Road buildings. However, it is likely that that primary user of water in this area is the Wentworth by the Sea Country Club to irrigate its golf course.

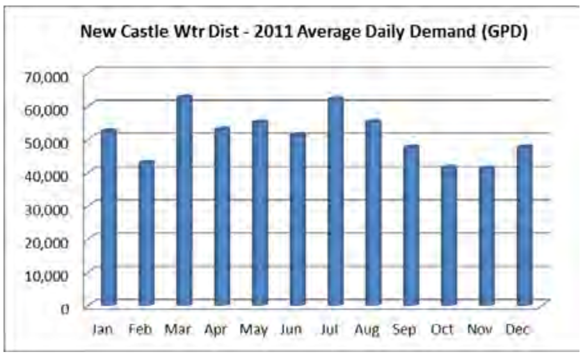

FIGURE 1-6
New Castle Average Daily Demand

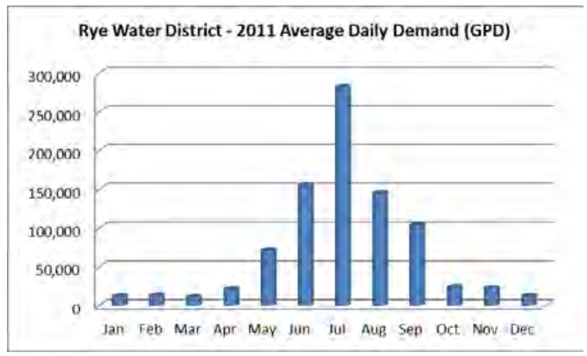

FIGURE 1-7
Rye Water District Average Daily Demand

Projected Demands

Based on recent customer demands and our analysis of the 20-year trend in overall consumption, demand in the City of Portmouth's water system is not anticipated to grow disproportionately in the next 20 years. Utilizing all of this information, coupled with the projections, we are utilizing a conservative estimate for planning purposes that water use demand will increase at a modest 1% per year. The following table summarizes the anticipated Average and Maximum Day demands through 2030:

TABLE 1-16
Projected Water Demand for the Portsmouth Water System through 2030

Year	Average Day Demand (mgd)	Maximum Day Demand (mgd)	Maximum Month Average Day Demand (mgd)
Average '04-'11	4.59	7.02	5.71
2015	4.78	7.31	5.94
2020	5.02	7.68	6.24
2025	5.28	8.07	6.56
2030	5.55	8.48	6.90

1.4 Comparison of Actual Demands Versus 2000 Master Plan Projections

The Phase I Water System Master Plan performed for the City of Portsmouth by EarthTech projected that the average day demand for the Portsmouth water system in the year 2010 would be 5.19 mgd. The following table provides a breakdown of the actual 1997 through 1999 system data, the Phase I projected 2010 usage and actual demands utilizing our recent analysis of 2011 water usage data:

TABLE 1-17
Portsmouth Water System – Comparison of Projected vs. Actual Water Usage

Customer Classification	Actual '97-'99 Usage	Projected 2010 Usage	Actual 2011 Usage	Difference	% Difference Actual vs. Projected
Residential Sales	1.44	2.00	1.36	(0.64)	-32%
Industrial Sales	0.92	1.15	0.56	(0.59)	-51%
Commercial Sales	0.88	0.98	1.41	0.43	44%
Municipal Sales	0.12	0.14	0.11	(0.03)	-21%
Other Utility Sales	0.13	0.15	0.12	(0.03)	-20%
Total Metered Sales	3.50	4.42	3.60	(0.82)	-19%
Unaccounted-For Water	0.61	0.77	0.63	(0.14)	-18%
Total Average-Day Demand (mgd)	**4.10**	**5.19**	**4.31**	**(0.88)**	**-17%**

Table 1-18 shows the current average water use per account in gpd. This data is the combined Portsmouth/Pease system customers and reflects the average usage based on 2011 data.

TABLE 1-18
Portsmouth Water System – Combined System Average Daily Demand (gpd)

Customer Classification	Accounts	Current Average Per Account GPD	1997-1999 Average Per Account GPD
Commercial	976	1,444	
Residential	**6,843**	**199**	**217**
Industrial	81	6,904	
Other Districts	2	61,932	65,000
Municipal	66	1,659	
Irrigation	238	275	N/A

The Phase I Master Plan noted that the total Portsmouth residential water use demands for the period 1997-1999 were between 33-37%. As Figure 1-5 shows, the current percentage of water use has dropped slightly, to 32% of the overall system demand. The Phase I Master Plan also noted that the average residential service in the Portsmouth system consumed an average of 217 gpd. As Table 1-18 shows, this average has dropped 8.3%, or 18 gpd. Using the same calculation for an average household size as the Phase I Plan utilized, this equates that an average of 76.5

gpd/person based on 2011 usage, compared with an average of 83 gpd/person based on 1999 usage. Again, it must be noted that this is simply an average of usage divided by total metered services. As previously noted, some services that are considered to be residential are actually multi-family and serve more than 2.61 residents per service. For example, the Bunker Hill service in Madbury is listed as one residential service, but the development itself has 51 units. Additionally, the commercial category of accounts includes condominium associations. These associations are likely to include a number of residential units. Therefore, a true accounting of customer water usage on a per capita basis would break these accounts down further so that a more detailed analysis could be arrived at. However, for the purposes of this study and for projecting future water use, the current breakdown is adequate.

One category that is likely to affect future water demands is the hotel and motels on the system as the City already has a number of new hotels, including a conference center, currently in the planning stages. Therefore, we queried the current hotels in the area to get a breakdown of the number of rooms and utilized their 2011 water use data to arrive at their water use breakdown. As Table 1-19 shows, water usage from hotels on the Portsmouth Water System is considerable. In fact, it accounts for 4% of the overall usage on the system. Hotels also have a higher peaking factor than the overall water system demographic.

TABLE 1-19
Hotel Usage Demographics

Hotel	Rooms	Avg Month Usage (gals)	Avg Daily Usage (gpd)	Avg Usage / Room (gpd)	Max Month Usage (gals)	Max Month Daily Usage (gpd)	Max Month Usage / Room (gpd)
Americas Best Inns	61	105,717	3,466	57	157,828	5,175	85
Anchorage Inn	92	121,785	3,993	43	263,296	8,633	94
Best Western Wynwood	169	285,237	9,352	55	510,136	16,726	99
Comfort Inn	121	225,335	7,388	61	375,496	12,311	102
Courtyard by Marriott	133	382,664	12,546	94	756,976	24,819	187
Economy Lodge	108	165,682	5,432	50	302,192	9,908	92
Fairfield Inn	105	173,848	5,700	54	385,220	12,630	120
Hampton Inn	125	350,472	11,491	92	492,932	16,162	129
Hilton Garden Inn	131	351,248	11,516	88	458,524	15,034	115
Holiday Inn	130	318,512	10,443	80	936,496	30,705	236
Homewood Suites	116	332,361	10,897	94	463,012	15,181	131
Residence Inn	128	324,694	10,646	83	749,496	24,574	192
Sheraton Hotel	200	565,176	18,530	93	869,176	28,498	142
Sise Inn	34	56,536	1,854	55	94,996	3,115	92
The Port Motor Inn	57	140,063	4,592	81	206,448	6,769	119
Wentworth by the Sea	161	656,058	21,510	134	904,332	29,650	184
Wrens Nest	32	63,473	2,081	65	74,052	2,428	76
TOTAL	**1903**		**151,438**	**80**		**262,315**	**138**
					Peaking Factor		**1.7**

Weston & Sampson performed a follow-up Phase II Master Plan assessment for the City in June of 2003 that also reviewed water demands and provided updated projections. The following tables provide a summary of the EarthTech and Weston & Sampson projections as they relate to the actual water demands over the same period of time:

TABLE 1-20
Comparison of Average Day Demand Projections from Previous Reports and Actual Usage During the Same Time Period

Year	EarthTech Average Day Projection (mgd)	Weston & Sampson Average Day Projection (mgd)	Actual Combined System Average Day (mgd)
2000	4.46	4.25	4.25
2005	5.17	5.17	5.09
2010	5.91	5.91	4.21

TABLE 1-21
Comparison of Maximum Day Demand Projections from Previous Reports and Actual Usage During the Same Time Period

Year	EarthTech Maximum Day Projection (mgd)	Weston & Sampson Maximum Day Projection (mgd)	Actual Combined System Maximum Day (mgd)
2000	7.73	6.10	6.10
2005	9.28	7.28	7.56
2010	10.61	8.61	7.12

As the tables show, both average day and maximum day demands did not increase as much as the previous studies projected. In review of other master plan reports, Whitman & Howard, Inc. noted in their 1979 report that average-day and maximum day demands in 1979 were approximately 4.4 and 7.0 mgd respectively. Whitman & Howard again updated the flows for their 1994 report and found the 1994 average day and maximum day demands to be 5.3 and 8.5 mgd respectively.

There are a number of factors that can be attributed to the leveling off of water consumption, including the recent downturn in the economy. However, it is evident that the customers on the water system are becoming more efficient with their water usage. This is not unique to Portsmouth; other New England systems are experiencing a similar decrease in their water demands. The Boston Water and Sewer Commission reported recently that their retail water sales have decreased approximately 32.5% from 81.3 mgd in 1985 to 54.9 mgd in 2011. According to their report with respect to their current rate setting methodology, "this long-run decline in sales is mainly attributable to increased conservation efforts on the part of all customers as a result of previous rate increases and the rise in the general level of conservation."

The City of Portsmouth has also been aggressive in promoting water efficiency over the last ten years. They routinely provide water conservation tips to all residential services and a few years ago they provided free water conservation retrofit kits to all residential customers that wanted them. The switch to automatic meter reading and monthly billing also provides the ability to better inform customers about their water use patterns and contact them when high use or leaks are suspected. Finally, the City was awarded $55,000 in federal stimulus funding distributed by the NH Department of Environmental Services, to offer Portsmouth water system customers up to two recycled plastic rain barrels for $30 each–half the market price–online and the cost is added to their water and sewer bill. This program has proven to be very successful.

Past studies have shown an increasing demand and need for water supply due to the documented increase of water use and the expansion of the water service territory. For example, Greenland was added to the system in the 1940's. Additionally, it wasn't until the United States Congress passed the Energy Policy Act of 1992 (which mandated that beginning in 1994, common flush toilets use only 1.6 gallons per flush) that indoor water use efficiency became a standard practice. Finally, rate increases, especially as they relate to water users being billed for sewer based on their water consumption, have increased the likelihood that the City's water customers have focused efforts on being more efficient with their water use. Figure 1-8 highlights this fact. As the graphic shows, overall average day pumpage in the City's combined water system has decreased slightly over the past 20 years. In fact, the City's 1961 financial report provided data from the City's water system that showed that their three-year average water demand (1959 to 1961) was 4.1 mgd.

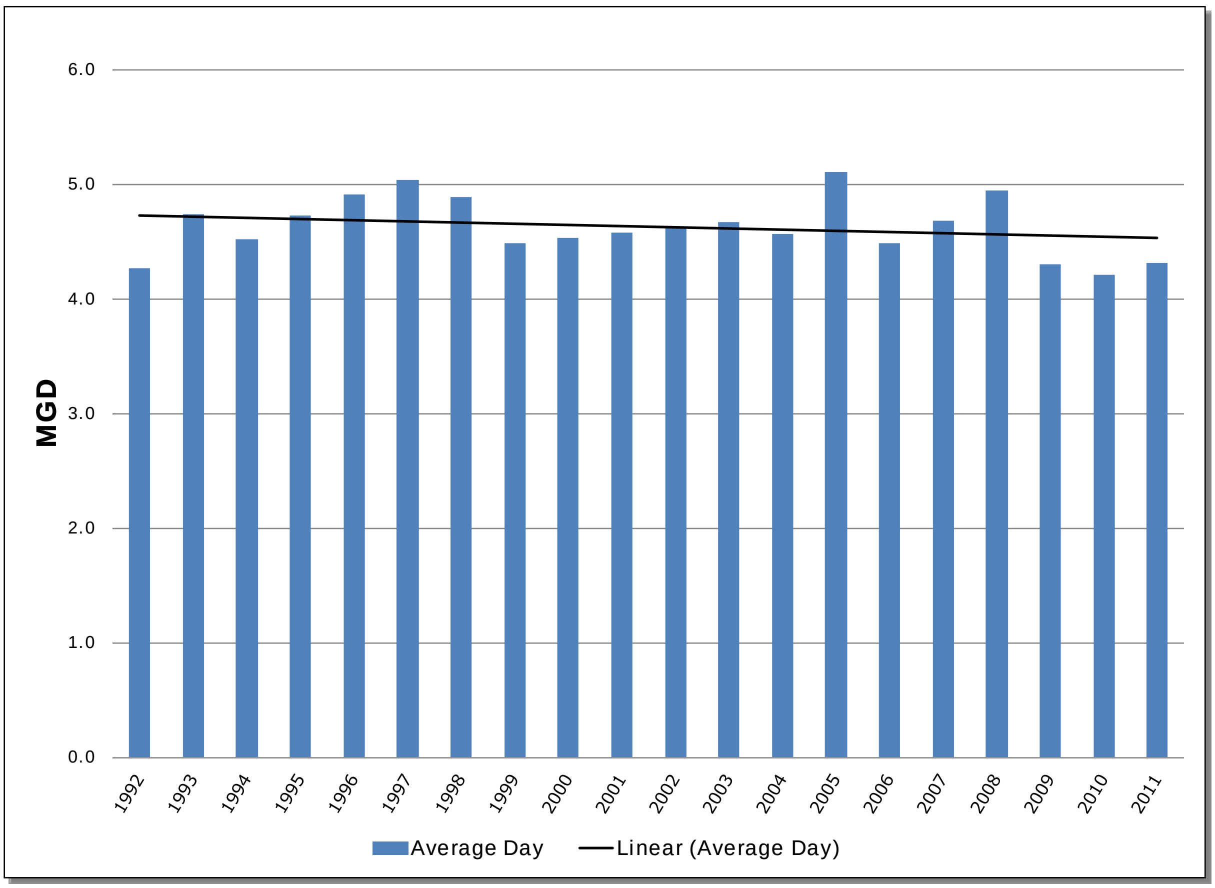

FIGURE 1-8
20-Year Historical Pumpage Trend – Combined Systems

1.5 Available Water Supply

Overview of Available Water

The City's water system is comprised of both surface and groundwater sources. Various upgrades to these sources of supply have taken place since the last master plan, including the reactivation of the Harrison well on the Pease portion of the system and the construction of an entirely new 4.0 mgd surface water treatment facility which replaced the 3.5 mgd facility in Madbury. In addition to these sources, the available volume of water from the Haven well in the Pease system has been increased due to water quality monitoring results in the area surrounding the well, combined with the construction of a new air stripping water treatment system that is capable of treating for volatile organics should they be detected in the surrounding monitoring wells.

The Bellamy Reservoir supplies water to the Madbury Water Treatment Facility. An updated yield study was performed in 2008 to assess the sustainable yield of the reservoir, especially as it relates to water availability during dry periods. The City continues to maintain and redevelop their nine groundwater wells such that they are able to efficiently withdraw water from those sources. All of these sources are operated in an integrated manner that utilizes the diversity of the sources in order to optimize their long-term yield and maintain the best quality possible. It is recognized by the water supply industry that utilizing surface water supplies during normal and wetter than normal periods allows groundwater sources to recharge and maintain optimal supply storage. Therefore, when dry periods occur the groundwater sources can be called upon to meet the majority of the water demand.

Safe Yield of Groundwater Wells

The safe yield of a well can be defined in numerous ways. AWWA states that "in the purest sense, it refers to the annual amount of water that can be withdrawn from an aquifer without producing an undesirable result, such as; withdrawal in excess of natural recharge; lowering of the water table below certain limits, interference with the groundwater rights of others, saltwater intrusion from the sea or other low water quality areas, reduction of baseflow to streams, degradation of groundwater quality, etc. Most commonly, safe yield is considered as the average annual natural inflow to a basin on the basis of data from 30 to 40 years of hydrologic record. Long-term changes in land and water use in a groundwater basin will change the safe yield of the basin." For the purpose of this study we utilized past reports and hydrogeological studies, operational data, withdrawal trends and precipitation data to arrive at the safe yield of the City's groundwater wells.

Safe Yield of the Bellamy Reservoir

There are many ways to determine the safe yield of a water resource. AWWA's M50 manual describes safe yield for surface reservoirs to be, "the maximum quantity of water that can be guaranteed to be available from the reservoir during a critical dry period." Recognizing the nature of the resource (precipitation) is very cyclical (especially in New England), identifying a single-low flow number that may only occur for a brief period of time is fairly impractical for the day-to-day operations of a surface water treatment facility, especially when that facility has groundwater sources available to supplement their surface water supply. In such a system withdrawals from the reservoir can be increased during wet periods while the water system's groundwater supplies can be rested, allowing wells to recharge so that they can be relied upon to meet system demands during dry periods when surface water may not be as available. Therefore, it is more practical to determine the confidence intervals for various flow scenarios.

According to Weston & Sampson's 2008 Bellamy Reservoir Watershed Update, "In 2002, Weston & Sampson presented the City of Portsmouth (the City) with a Phase 2 Master Plan, which included an analysis of water system demands and sustainable yield of the Bellamy Reservoir. The analysis employed the Army Corps of Engineers FORTRAN-based HEC-5 model to determine sustainable yield and drought response of the reservoir. A bathymetric survey of the Bellamy Reservoir was conducted to determine the capacity of the reservoir at various water level stages. In 2008, discussions with the City led to a reassessment of the Bellamy Reservoir's sustainable yield and likely drought response." That report provides detailed background and a description of the parameters used to assess the Safe Yield of the Reservoir during simulated drought conditions. Factors used included:

- Streamflow into the reservoir
- Precipitation trends
- Evaporation rates
- Required instream flow through the dam to the Bellamy River
- Withdrawal rates to the Madbury Water Treatment Facility

Weston & Sampson concluded, that based on their confidence interval assessment, that "reviewing these calculations from a historical perspective (including the drought of record) predicts that the Water Treatment Facility could withdraw 2 MGD, year-round, 99% of the time. Withdrawing 5 MGD can be accomplished 90% of the time. It is important to note that this assumes that water can be taken down to the 126-foot intake level. This data also assumes a year-round instream flow to through the dam to the Bellamy River of 1.7 MGD."

Therefore, utilizing this data, together with other past reports and current operating parameters we utilized the assumption that a withdrawal of 2.5 MGD as the safe yield of the reservoir for the purposes of this study.

Sustainable Water Supply Assessment

According to the U.S. Geological Survey, "The term 'safe yield' has historically been used to describe the amount of water available from a groundwater or surface-water source. Typically, the concept of safe yield implies that a single value represents the water available for withdrawal in a basin given some singular constraint, such as an engineering limitation or climate condition."

Massachusetts recently updated their definition of safe yield through the adoption of their Water Management Act, where they stated that Safe Yield was, "The maximum dependable withdrawals that can be made continuously from a water source, including ground or surface water, during a period of years in which the probable driest period or period of greatest water deficiency is likely to occur; provided however, that such dependability is relative and is a function of storage and drought probability."

The USGS has also recently expanded the view of water supply availability via shift from looking at the "safe yield" of water supply sources to their "sustainable yield," which will vary over time of year and depend on long-term climate conditions. Recent USGS documentation further describes this concept:

> *To address the limitations of the safe-yield definition, recent literature has proposed that water availability is better expressed as a "sustainable yield" rather than a safe yield (Sophocleous, 2000; Alley and Leake, 2004; and Maimone, 2004). Sustainable yield is a measure of water availability that simultaneously considers the spatial and temporal availability of water (Maimone, 2004), as well as the complex interplay between the time varying and competing demands for water, such as human and ecological water needs (Alley and Leake, 2004). The concept of sustainable yield signifies the complexity and interdependence of some variables that affect water availability. To understand and quantify the sustainable yield of a basin, water managers and planners require flexible tools that address as many of these variables as possible and at the appropriate time scales.*

The following graphic shows the precipitation trend over the last 14½ years, based on Portsmouth precipitation data. As the graph shows, the amount and time of precipitation events varies greatly. There have been periods of extended dry conditions such as the Seacoast of New Hampshire experienced in 2001-2002. There have also been very wet periods such as the flooding that occurred due to high rainfall events in May 2006 and March 2010.

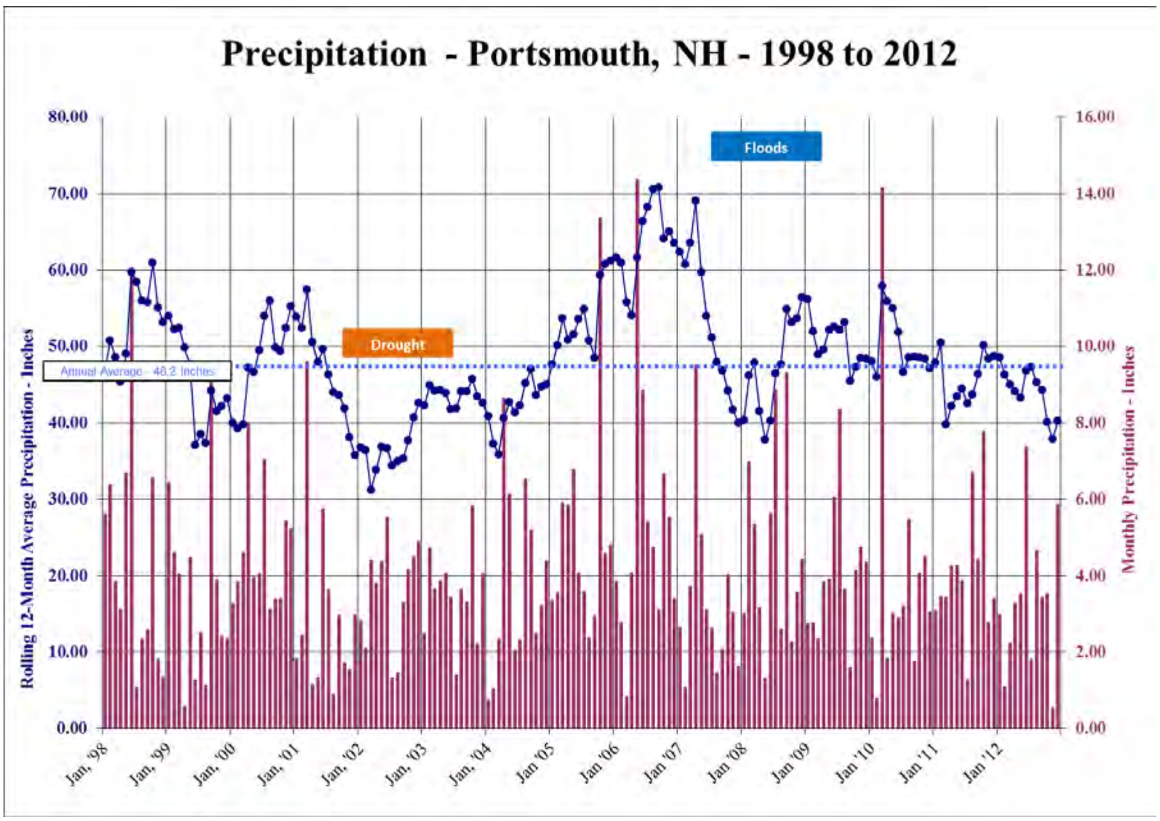

FIGURE 1-9
Precipitation Trends for Portsmouth – January 1998 to December 2012

The recently released report, "Climate Change in the Piscataqua / Great Bay Region: Past, Present, and Future," describes how the climate of the Piscataqua/Great Bay region has changed over the past century. The study found that "overall, the region has been getting warmer and wetter over the last century, and the rate of change has increased over the last four decades." The report also observed that, "seasonal precipitation is increasing in spring, summer, and fall but decreasing during winter."

The sustainable yield of water supplies is greatly influenced by these precipitation trends. Extended droughts will reduce both surface and groundwater supplies, though at varying rates. Floods will restore water levels in the surface water supplies quickly, such as the Bellamy Reservoir, but will not recharge groundwater sources as quickly. Therefore, developing adequate sustainable yields for the City of Portsmouth's water sources requires that numerous factors be taken into account.

The Weston & Sampson study in 2003 explored the sustainable yield of the City of Portsmouth's water supply sources by developing a hydrological model of the Bellamy Reservoir. They also conducted an extensive geologic and hydrogeologic investigation of the City's wells and aquifers. Geologic data was compiled, climatic records were reviewed, aquifer pumping tests were performed, hydraulic analyses were completed, recharge areas were delineated, and the sustainable yield of each aquifer was computed. Because the wells had to remain in operation during their study, they noted that extensive constant-rate pumping tests and recovery periods were not possible. An alternate mass-balance approach was then utilized to determine the aquifer capacity based on available recharge and the capture zones of the wells. They also collected and reviewed historical operational data to arrive at and predict sustainable yields. Subsequently, in 2008, Weston & Sampson provided an update to the Bellamy Reservoir's sustained yield that utilized a model simulation approach to revise the water availability projections for the new Madbury Surface Water Treatment Facility.

Our assessment of the current supply capabilities for the City of Portsmouth's water supply sources utilized data from the Weston & Sampson report and other historical information. Most importantly, though, we analyzed the actual withdrawals of these sources utilizing monthly data as reported by the water supply staff over a period of eight years, from 2003 to 2011. This data was then analyzed for the months that the sources were actually in service (for example, the Collins Well has had periods where it has been offline for maintenance). The average and maximum monthly pumpage was then assessed for each source. Per discussions with City staff, we then utilized the 75th percentile pumpage value as the likely sustained yield of the supply source. We then compared that with the Weston & Sampson data and provided an updated assessment of all the sources as to their likely sustained yields. The following table provides a summary of that data:

TABLE 1-22
Pumpage Data and Likely Sustained Yield of Portsmouth's Water Supply Sources

2003 to 2011 Pumpage Data	WTF Finished Water	Madbury Wells	Greenland Well	Port #1 Well	Collins Well	Haven Well	Smith Well	Harrison Well[6]	TOTAL Sources	MGD
Total Operating Months [1]	108	108	108	108	85	105	91	67	108	
Total Pumpage (MG) [2]	7,612	2,481	1,670	1,261	485	699	447	331	15,010	
Average Monthly Pumpage (MG) [3]	70	23	15	12	6	7	5	5	139	
Max Month Pumpage (MG)	109	37	22	18	12	15	11	10	192	
75% Month Pumpage (Total MG)	84	27	20	13	7	8	6	6	171	
75% Month Pumpage (Average GPM)	1,909	735	454	301	159	180	142	132	4,012	5.78
W&S Safe Yield (GPM) [4]	1,736	559	460	227	153	534	163	134	3,966	5.71
T&B Likely Sust. Yield (GPM) [5]	1,736	647	457	264	156	534	153	133	4,080	5.87

Notes:
1. Total Operating Months includes all months the source of supply was in operation and pumping at a close to normal capacity. Some months show minimal pumpage and are likely due to well maintenance or low water demand. These months were dropped from the analysis.
2. Total Pumpage includes the total water pumped for all the months the source was considered to be fully operational.

3. Average Monthly Pumpage includes the Total Pumpage divided by the Total Operating Months
4. Water Supply Master Plan and Madbury WTP Evaluation Report, Weston & Sampson, June 2003 and Updated Assessment of Bellamy Reservoir Yield, 2008.
5. Average of the 75% Average Day Pumpage and the W&S Safe Yield GPM for all the wells. The new Madbury WTF is rated at 4.0 MGD, however, the Bellamy Reservoir sustained yield during drought conditions is 2.5 MGD per the W&S Bellamy Reservoir Assessment. The Haven Well pumpage history utilizes some years where the well flow was restricted by an agreement with the Pease Air Base, therefore, the calculated yield of 534 GPM is the likely safe yield of this source
6. The Harrison Well was placed into service in May 2006 after rehabilitation of the well and pump facilities.

Other Notes:

- The Haven Well pumpage history includes some years where the well flow was restricted by an agreement with the Pease Air Base, therefore, the calculated yield of 534 GPM is the likely safe yield of this source.

1.6 Margin of Safety

The American Water Works Association's manual of Water Resources Planning, M50, describes Integrated Resource Planning as, "A continuous process that results in the development of a comprehensive water resource management plan. It identifies and gives balanced consideration to supply and demand management planning alternatives."

The City of Portsmouth's Water Division has adopted their own Integrated Resource Management Plan with respect to the operations of their sources of supply and demand. The system has implemented capital improvements and maintenance programs that have focused on that goal. They include the replacement of the Harrison well, the redevelopment of other wells, and the construction on the new Madbury Water Treatment Facility. All of these efforts will enable them to manage these supplies in an integrated manner. With a nearly 50/50 split of surface versus groundwater supply they will be able to continue their efforts to optimize surface water withdrawals when the reservoir's quality and quantity are acceptable. By optimizing this supply they will have the opportunity to "rest" groundwater sources to maximize their yields during dry periods. The Phase II Master Plan Report noted that "when surface water flows are high there should be less reliance on groundwater. These periods generally correspond to periods of groundwater recharge. By minimizing the use of groundwater during this time, water will be stored in aquifers for increased availability when surface waters are less plentiful."

This Integrated Supply Management approach, together with the further refinement of the City's demand management and source water protection programs, should allow the existing sources of supply to serve the demand needs into the foreseeable future. Should a new, high demand customer, desire to locate their business within the water system then it is most likely that Portsmouth would negotiate with this new user for the development of additional sources of supply as identified by the City during their 2009 study of potential new supplies. To adequately supply the incremental needs of other new customers on the system, such as residential or smaller commercial customers, we recommend that the City develop a reserve fund that new customers have to contribute to for future supply. This fund would then be utilized to develop new sources as needed.

The following table provides an overview of the historical margin of safety analysis for the Portsmouth Water System:

TABLE 1-23
Existing Portsmouth Water System - Margin of Safety Analysis

		Average Day					Maximum Month			
Year	Incremental Demand (MGD)	Demand (MGD)	Available Water (MGD) (1)	Available Water (MGD) (2)	Margin of Safety (1)	Margin of Safety (2)	Demand (3)	Available Water (1)	Peaking Factor	Margin of Safety
Historical										
2003		4.66	5.68	2.91	1.22	0.62	6.23	5.68	1.34	0.91
2004	-8.3%	4.30	5.68	2.91	1.32	0.68	5.29	5.68	1.23	1.07
2005	15.5%	5.09	5.68	2.91	1.12	0.57	6.19	5.68	1.22	0.92
2006	-13.7%	4.48	5.87	3.10	1.31	0.69	5.56	5.87	1.24	1.06
2007	4.2%	4.68	5.87	3.10	1.25	0.66	5.99	5.87	1.28	0.98
2008	5.3%	4.94	5.87	3.10	1.19	0.63	5.77	5.87	1.17	1.02
2009	-14.8%	4.30	5.87	3.10	1.36	0.72	4.77	5.87	1.11	1.23
2010	-2.4%	4.21	5.87	3.10	1.40	0.74	5.85	5.87	1.39	1.00
2011	2.3%	4.30	5.87	3.10	1.36	0.72	5.29	5.87	1.23	1.11
Average		4.55	5.87	3.10	1.29	0.68	5.66	5.87	1.24	1.04

(1) Safe yield based on 2012 analysis
(2) 24h/day pumping with largest source off line (Madbury WTF)
(3) Peak month based on actual system data

It is common practice to assess the Margin of Safety for a water system with its largest source off line. In this instance the table above assesses this margin with the Madbury Water Treatment Facility off line during an average day. It does show that losing this source for an extended period of time would be difficult. However, a more likely scenario is for the WTF's output would be reduced during periods of drought. In such a scenario, the City, with proper management mechanisms and the implementation of an Emergency Action Plan, should be able to meet demands.

Additionally, as stated above, during dry periods the City should be able to rely more on its groundwater sources. The safe yield analysis of the Bellamy Reservoir Watershed Yield Update in 2008 shows that, though low flow periods can occur for up to four months, recovery is very quick. With this in mind, it is feasible to assume that the groundwater sources could be pumped for a period of time at their maximum capacity. Once precipitation occurs and the reservoir refills, the WTF could likely be returned to its 4 mgd flow and the groundwater sources could be reduced so that they are able to recharge and recover. Utilizing this operating strategy we analyzed the maximum monthly withdrawals from each source to assess the overall capability of the system under this scenario. The following table shows the max pumpage capability of each source per our analysis.

TABLE 1-24
Sustained Yield vs. Maximum Yield of Portsmouth's Water Supply Sources

Source	Sustained Yield (mgd)	Maximum Yield (mgd)	Max vs. Sustained (mgd)
Madbury WTF	2.50	4.00	+1.50
Madbury Wells	0.93	1.21	+0.28
Greenland Well	0.66	0.71	+0.05
Portsmouth Well	0.38	0.58	+0.20
Collins Well	0.22	0.40	+0.18
Haven Well	0.77	0.77	+0.00
Smith Well	0.22	0.35	+0.13
Harrison Well	0.19	0.33	+0.14
TOTAL	**5.87**	**8.35**	**+2.48**

The following figure shows the current sustained yield of the City of Portsmouth's water sources compared with the current average day demand and the projected 2030 average day demand. It also shows the Maximum Yield and the theoretical Maximum Yield with the largest source off line:

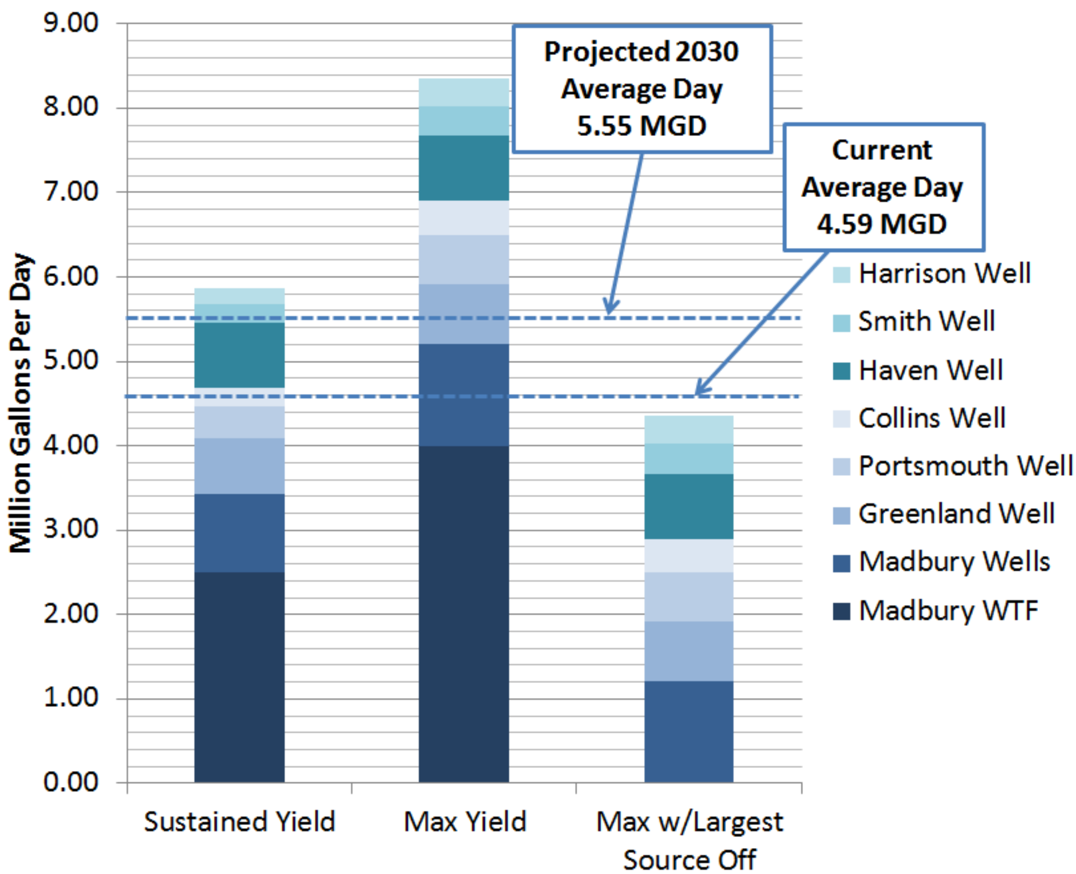

FIGURE 1-10
Water System Supply Yields vs. Average Day Demands

Note: The Weston & Sampson report noted that "sustainable yield appears to be limited by well and aquifer hydraulics," in a number of wells. They noted that with proper management, some redevelopment and possibly, replacement of a few of the wells, the output of the Portsmouth groundwater sources could likely be increased. Theoretically, they calculated that the groundwater source in the system were capable of approximately 3,335 gpm, or 4.8 mgd. This would bring the total maximum available supply of the system, together with the WTF, to nearly 10 mgd. Additionally, the Weston & Sampson report noted that the Haven Well could deliver up to 2.1 mgd based on well hydraulics. With this in mind, the City should consider the potential to install some satellite wells for backup capability. There will be more discussion of this alternative in our recommendations section of this report.

1.7 Water Supply Management

1.7.1 Rye Water District Emergency Interconnection

The New Hampshire Department of Environmental Services commissioned a study in 2006 to examine the potential for mutual aid between Ten Seacoast water systems. The City of Portsmouth was included in this study. Interconnections between the Portsmouth system and the City of Dover, the Town of Durham and the Rye Water District were considered. The study noted that the most feasible interconnection was between Portsmouth and Rye.

The Rye Water District's Washington Road Booster Station was modeled in this study with the City of Portsmouth's Lafayette Road water tank. A 4,000-foot length of new 16-inch water line between the two systems was modeled, and the proposed interconnection was determined to be feasible from a hydraulics standpoint, noting that flow from the Portsmouth main pressure zone would need to be pumped to Rye, and a PRV station would be needed to supply water from Rye to Portsmouth.

According to discussions with Rye Water District Superintendent, Ken Aspen, the current interconnection between the Portsmouth water system and the Rye Water District at the District's office on Sagamore Road has swing valves that only allow one-way flow, from the Portsmouth system to the Water District's system. The last time this connection was used was in 1993, however, it is flushed occasionally for routine maintenance purposes. With modifications, such as the installation of the pressure reducing valve to reduce the pressure coming from the District's system, this connection could be utilized to deliver water into the Portsmouth system in an emergency.

We recommend that the City of Portsmouth meet with the Rye Water District to explore opportunities to install this connection to provide emergency backup supply for both systems. At this time it is not known how much capacity the Water District might be able to supply the Portsmouth system in an emergency, but it is our understanding that they have been successful in expanding their groundwater supply capabilities in the recent years.

1.7.2 Madbury Well Replacements

The three currently utilized Madbury Wells (#2, #3 and #4) have been in service for over 60 years. Well # 1 has been off-line for a number of years and is no longer an approved source of supply for the system. These wells were originally installed as the replacement supply for the Pease wells that supplied water to the City of Portsmouth's system prior to construction of the Pease Air Force base in the late 1950s. Once the wells were placed in service, it was determined that they did not have adequate capacity to provide all the water necessary for the City's water system and therefore the Bellamy Reservoir and Madbury Water Treatment Facility were constructed.

Weston & Sampson conducted a detailed assessment of the history of these wells and their long-term yields. The Phase II Master Plan update assessed that the annual recharge to these wells was 544 million gallons. Though there are currently three wells on line (#2, #3 and #4) these wells are all considered to be drawing water from the same aquifer.

The City has a maintenance program for these wells that includes redevelopment and rehabilitation as necessary. This work is generally done when the well yields and/or specific capacity of the wells starts to decline. Work on well #2 was performed during the summer of 2012. According to the water system staff and Layne Christensen, the firm doing the work on the well, this well is starting to show signs that the screen may need to be replaced. Though it is possible to install new screens inside existing screens of wells to extend their life it often leads to further declines in the well yeild. Therefore, instead of installing a new screen we recommend that the City plan to start a replacement program for these wells, beginning with well #2.

Tighe & Bond contacted the NHDES Drinking Water and Groundwater Bureau staff to determine the regulatory requirements for pursuing the option of replacing this well. According to NHDES, the rules that apply to replacement wells are included in Env-Dw 302.30 – Replacing an Existing Large Production Well. According to these rules a water system can replace an existing well as long as it derives water from the same zone of contribution as the well that is being replaced. An assessment of the long-term sustainable yield must be performed to show that the new well will withdraw water at the approved capacity of the well being replaced or the long-term sustainable yield as tested, whichever is less. Once the new well is approved and in service, the well that it replaced must be abandoned.

1.7.3 Bedrock Well Potential for Additional Supply

The City commissioned the firm Emery-Garrett Groundwater, Inc. in 2009 to investigate potential sites for developing new groundwater sources. Since all of the existing surficial sand and gravel locations in the Portsmouth service territory have already been explored and/or developed, Emery-Garrett performed a study to locate feasible sites for potential bedrock well development. A number of locations were identified. This work complimented the detailed assessment performed during the Phase II Master Plan update. This update also recommended that the City explore new supplies through bedrock sources. Additionally, it recommended that adding satellite wells within existing aquifers would increase pumping capacity during peak periods. The NHDES regulation Env-Dw 302.29 allows construction of Back-up Large Production wells. This rule would allow the City to install additional wells for back-up supply without having to go through the extensive permitting process of a Large Groundwater Withdrawal Permit.

We recommend that the City continue to explore the potential to obtain either ownership or easement agreements at some of the sites identified by Emery-Garrett to continue exploration and identify final locations for potential drilling and permitting of a new large groundwater withdrawal for the water system. We also recommend that the City explore the feasibility of installing back-up wells at some of their existing well sites.

1.7.4 Integrated System Supply and Management Plan

The City has been conjunctively managing their one surface water and nine groundwater sources of supply for many years. Their normal procedure calls for optimizing their surface water source when it has available quantity and good quality. By doing this they are able to "rest" their groundwater sources so that the aquifers are as recharged as possible and their yields will be maximized and available when either water customer demands go up or the surface water source quantity or quality necessitates reducing the yield on their supply. Figure 1-11 shows the quarterly percentage of water supply provided by surface and groundwater supplies for the last ten years (November and December 2012 are estimated). It also shows the trend over this time.

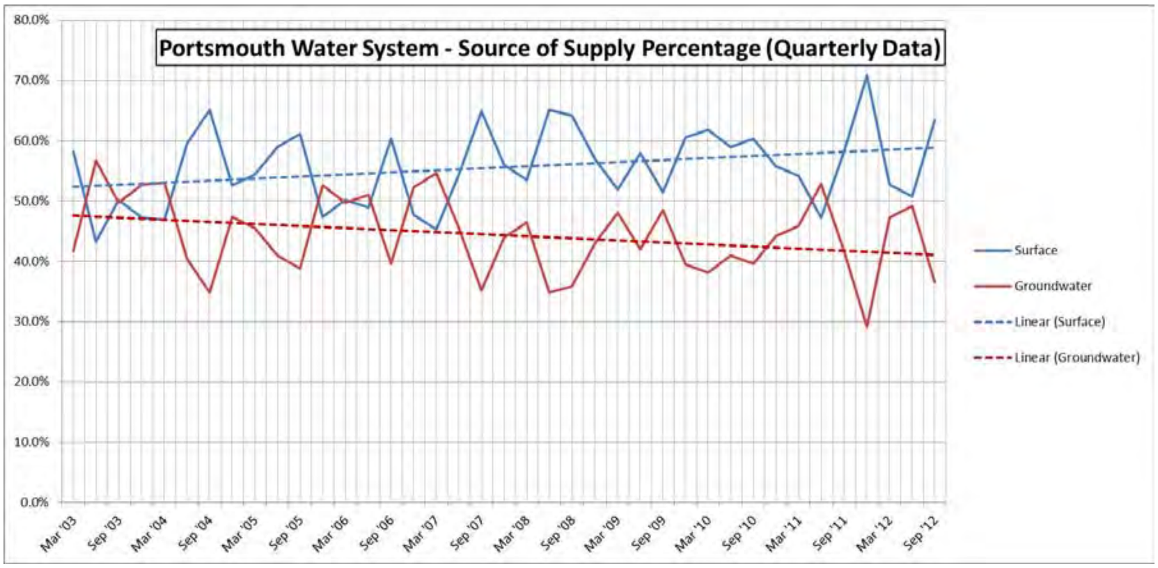

FIGURE 1-11
Source of Supply Percentage – Groundwater vs. Surface Water

The source of supply trend reveals that the water system's operational staff has been successful in optimizing the use of surface water sources over this period of time. It is important to note that the Harrison Well was not on line until mid-2006. Therefore, it is reasonable to assume that the existing groundwater sources and their aquifers are substantially recharged. This, coupled with recent wet weather years should enable the City with good options regarding their ability to manage their sources of supply in future years. However, the use of surface water sources, especially during the late summer, may increase the potential for disinfection byproducts to form in the water system. Therefore, we recommend that the City work on developing source of supply tracking tools, together with updated Standard Operating Procedures, to assist their operations staff with not only managing supply based on availability, but with water quality parameters included in the analysis. We recommend that the development of these procedures be included as part of a comprehensive update to the City's Integrated Water System Management Program. Further detail of this program is included in the recommendations section of this report.

1.8 Storage Capacity Evaluation

The purpose of distribution system storage is to compensate for the difference between peak system demands and the rate of supply from the water sources, while also providing an adequate standby volume of water for periods of equipment malfunctions at supply sources, critical supply main failure, and fire fighting purposes. Customer water usage varies substantially on an hourly, weekly, and seasonal basis. Supply production must operate under the constraints of supply source capacity, pumping rates, and withdrawal rate capabilities.

The goal of water system planning, construction, and operation is to provide uninterrupted and adequate flow, pressure, and water quality to all customers under system conditions which may be reasonably anticipated or expected. There are three components to consider when evaluating needed storage in a system:

- Equalization storage
- Fire protection storage
- Emergency storage

Equalization storage represents the amount of water needed to satisfy peak demands during the course of the day and can be estimated based upon historical diurnal demand data. Fire protection storage represents the amount of water needed in the event of a major fire and is generally estimated based on Insurance Service Office (ISO) recommendations. Emergency storage represents the amount of water needed in the event of a short-term water system emergency, such as a supply source off-line or a major water main break.

Most water systems occasionally experience water main breaks and/or system mechanical malfunctions that could potentially require up to several days to correct. It is not unreasonable to expect that a major reduction in water supply capacity could persist for a week or more. It is neither practical nor economical to provide on-line storage for long-lasting system disruptions, however adequate storage should be provided to allow for reaction time to implement appropriate contingency plans. A reasonable storage volume is necessary to address the level of risk and potential customer inconvenience acceptable to both the City officials responsible for the system and to the system customers who must both live with and pay for the system.

1.8.1 Summary of Previous Reports

The 2000 Master Plan found that the Portsmouth distribution system required an optimum storage capacity of 2,711,000 gallons and would have inadequate fire flow and peak demand storage by 2008. The Spinney Road Tank, a 1 MG elevated storage tank, was constructed to address the storage deficit. The Spinney Road Tank increased the storage capacity to a surplus of 155,000 gallons over the optimum storage requirement.

The Master Plan also found the storage capacity of the Pease Water system to be deficient to meet the anticipated typical demands by 2020. The Plan proposed replacing the existing 0.4 MG Hobbs Hill Tank with a 1 MG elevated composite tank. No measures have been taken to address the Pease System storage deficiencies.

The Master Plan contained the following findings regarding pumping capacity in the Portsmouth System:

- The pumping capacity of the Portsmouth and Pease Systems were inadequate to meet existing needs. The Portsmouth System was found to lack 1.14 MG of pumping capacity in 2000 and was projected to lack 4.20 mgd in the year 2020. The Pease system was found to lack 0.88 mgd and was projected to lack 3.29 mgd by the year 2020.

The Master Plan made the following additional recommendations regarding storage and pumping capacity in the Portsmouth System:

- Demolish the Islington Street Standpipe (this has been accomplished)
- Inspect the interior and exterior of each tank every five years
- Paint the tanks every 20 years
- Develop additional water sources

Phase II of the Master Plan included an investigation to determine the sustainable yield of the system. The sustainable yield of the system's aquifers was found to be 3.2 mgd. The sustainable yield of the Bellamy Reservoir was found to be 4.3 mgd. The total sustainable yield of the water system was 7.5 mgd. The average day demand for 2020 was projected to be 7.39 mgd, which would not leave a large margin of safety for the system. The Master Plan recommended developing new water sources in order to increase the margin of safety.

1.8.2 Main Pressure Zone Storage Capacity

Available storage capacity in the Main Pressure Zone currently consists of the Lafayette Road Tank and the Spinney Road Tank. The Newington Tank storage is also available to the main pressure zone but requires pumping via the Newington Booster Station. The Osprey Landing Tank is not in service and was not considered in this evaluation.

Due to the configuration of the tanks and the need to maintain certain minimum pressures in the distribution system, not all of the storage capacity is considered "useable". The useable storage capacity is the volume of water in the tank that can be used to maintain adequate system pressures.

The 2000 Master Plan calculated useable storage as the available storage that provided a minimum of 20 psi of pressure to all customers in the system. The 2000 Master Plan recommended a minimum of 30 psi and a maximum of 100 for regular customer usage. Pressure during fire flow should remain above 20 psi at all locations in the system. The available storage in the system above 20 psi, calculated by adding 46 feet to the highest point in the service system, is presented in Table 1-25. The total useable storage in the Main Pressure Zone is currently 4.766 MG with the Osprey Landing Tank offline.

TABLE 1-25
Available Storage in the Main Pressure Zone

Tank	Total Storage (gal)	Useable Storage (gal)
Newington Tank	1,500,000	1,500,000
Lafayette Road Tank	7,500,000	2,266,000
Spinney Road Tank	1,000,000	1,000,000
Osprey Landing Tank	200,000	0
Total	8,700,000	4,766,000

1.8.2.1 Equalization Storage

Under normal water supply system operations, supply source pumps are turned on throughout the day to supply the system and fill storage tanks, and turned off during low demand periods when tanks are full to allow the tanks to supply the system. Due to this daily variation in tank water levels, a volume of storage determined by diurnal demands is needed. Equalization storage is defined as the volume of water that is needed to meet the variations in water demand that exceed the constant pumping capacities of the sources in a particular system. This volume was determined by the following methods:

1. Equalization storage was computed based on a typical summertime diurnal demand curve
2. Equalization storage was computed as 20-25% of maximum day demand (Chin, 2006).
3. Equalization storage was computed as (peak hour demand - maximum day average demand) x 8 hr (Ulasir *et. al.,* 2005)

Method 1

Hourly supply and tank level data was provided by the City for July 21 through 23, 2011 and May 22 through 23, 2012. July 22 provided a data set with the maximum day demand (6.03 mgd for the Main Pressure Zone) for the data provided and was chosen for this method. The area under the diurnal curve and above the average demand was approximately 0.40 mgd, or 7% of the maximum daily demand. The diurnal curve and average demand are presented in Figure 1-12. This method was also employed using the data from the remaining available dates, and the percentage of the MDD required ranged from 6% to 8%, or 0.38 MG to 0.60 MG.

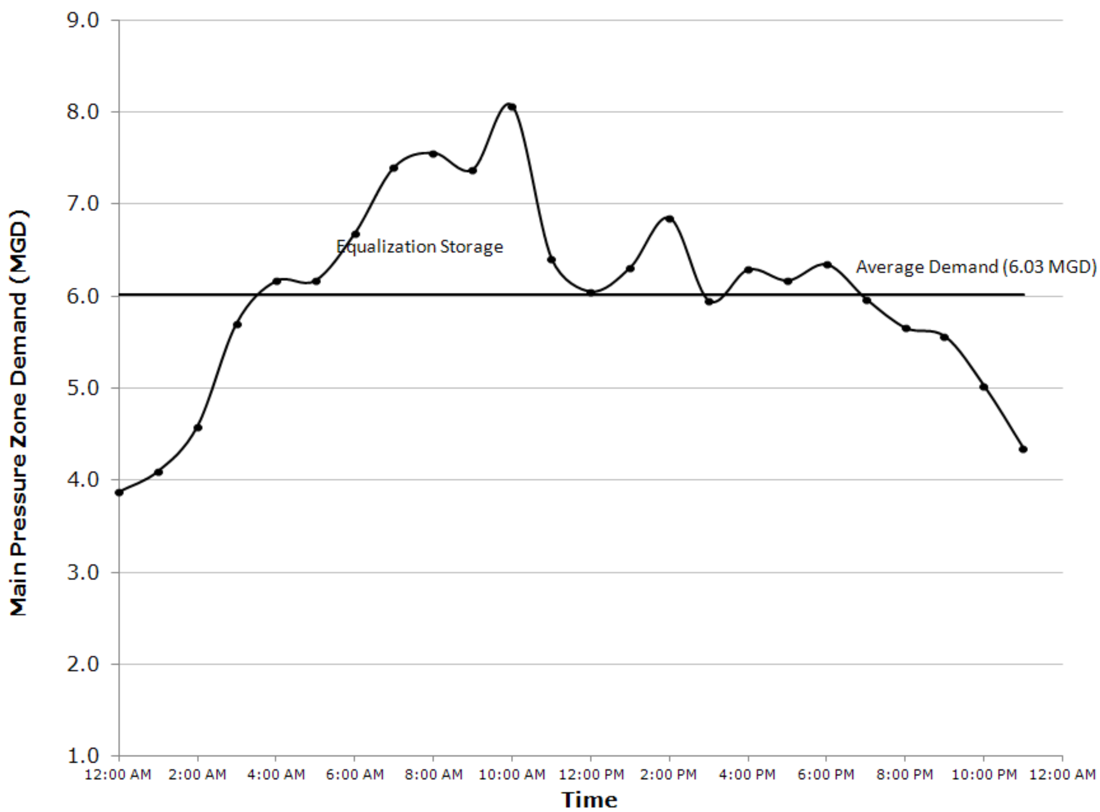

FIGURE 1-12
Diurnal Demand Curve for the Main Pressure Zone on July 22, 2011

Method 2

Under Method 2, the equalization storage is estimated to be 20 to 25% of the MDD. The maximum day demand, based on the July 22, 2011 demand, was 6.03 mgd for the Main Pressure Zone; thus, the equalization storage calculated by Method 2 is in the range 1.21 to 1.51 MG.

Method 3

Under Method 3, the equalization storage is calculated as a function of the peak hour demand and the MDD. Thus, the equalization storage estimated by Method 3 is (peak hour demand - maximum day demand) x 8 hr = (8.07 mgd – 6.03 mgd) x 8 hr = 0.68 MG.

Summary and Current Practices

Table 1-26 summarizes the equalization volume calculations and related parameters.

TABLE 1-26
Summary of Equalization Storage Calculations – Main Pressure Zone

Calculation Method	Volume (mgd)
Method 1: Area under diurnal demand curve	0.38 – 0.60
Method 2: 20% - 25% of MDD	1.21 – 1.51
Method 3: (Peak hour - MDD) x 8hr (see Table 1-27)	0.68
2000 Master Plan Method (15% of MDD)	0.89

As indicated in Table 1-26, Method 2 results in the most conservative estimate of equalization storage. However, Methods 1 and 3, which are based on actual conditions in the pressure zone, are lower (0.38 MG to 0.68 MG). Method 3 is approximately in the middle of the range estimated by Methods 1, 2, and 3 and is deemed to be a reasonable estimate. Therefore, equalization storage is estimated to be 0.68 MG per Method 3. Equalization storage calculation and projections for 2020 and 2030 are presented in Table 1-27.

TABLE 1-27
Method 3 Equalization Storage Calculation - Main Pressure Zone

Item	2011	2020	2030
Average Day Demand (mgd)	3.85	4.50	4.95
Maximum Day Demand (mgd)	6.03[(1)]	7.05[(2)]	7.75[(2)]
Peak Hour Demand (mgd)	8.07[(1)]	9.43[(2)]	10.38[(2)]
Equalization Storage = Peak hour-MDD x 8 hours (MG)	0.68	0.79	0.87

Note:
1. Maximum day demand (MDD) and peak hour demand were derived from actual operating data on July 22, 2011 for the Main Pressure Zone.
2. Projections of MDD and Peak Hour Demand for 2020 and 2030 above are based on peaking factors derived from the July 22, 2011 operational data.

1.8.2.2 Fire Protection Storage

Based on ISO recommendations, the maximum needed fire flow that a community water system is expected to provide is 3,500 gpm for 3 hours provided that there are commercial or industrial customers within the service area with needed fire flows that meet or exceed this value. A storage volume of 0.63 MG is recommended for fire protection storage. This number was also used in the 2000 Master Plan Report.

1.8.2.3 Emergency Storage

Firm Pumping Yield

There is no absolute formula for calculating the amount of emergency storage that a water system should have available. Engineering judgment must be used based on the vulnerability of an individual utility's water supply. The availability of auxiliary power at supply sources lessens the need for emergency storage. However, some emergency storage should be available to handle a catastrophic water main break that cannot be isolated easily. For the purpose of this study, firm pumping yield is defined as the average daily withdrawal from the water supply system that can be sustained through the available pumping sources without entirely depleting the system storage. Our study took into account all the City's pumping stations that have backup power supplies or standby engine powered pumps.

Since the Portsmouth system has multiple sources with backup power available, it is not expected that a power failure would eliminate the ability of all of the supply sources to provide water to the distribution system. However, the Portsmouth system is still vulnerable to emergencies such as a major water main break or pump failure. For purposes of this analysis, the "firm pumping capacity" is defined as the rate of supply to the pressure zone with the largest supply source out of service. We recommend estimating emergency storage as the difference between maximum day demand and the firm pumping capacity, projected over 8 hours. For the Portsmouth system, the largest supply source out of service was defined as the Madbury WTF, assuming its capacity is 4.0 mgd. Although it is possible that conditions could exist such that multiple facilities could be prevented from providing any water to the distribution system, this assumption is unreasonably conservative and would result in significantly larger recommended storage capacities. Engineering judgment must be applied in order to balance the amount of storage needed to supply the system during an emergency with the water quality impacts and cost ramifications of increased storage under normal system operation. Supply capacity and equalization storage calculations for the Portsmouth main pressure zone are summarized in Table 1-28.

TABLE 1-28
Emergency Storage Calculation – Main Pressure Zone

Main Pressure Zone Supply Capacity			
Sources	**Capacity (mgd)**	**Power Failure Backup**	**Included in Firm Pumping Capacity?**
Madbury Well No. 2	0.43	Engine drive	Yes
Madbury Well No. 3	0.50	Engine drive	Yes
Madbury Well No. 4	0.61	Engine drive	Yes
Portsmouth Well No. 1	0.65	Generator	Yes
Collins Well	0.46	Engine drive	Yes
Madbury WTF	4.00	Generator	No
Greenland Well	0.63	Engine drive	Yes
Total Pumping Capacity	7.29		
Firm Pumping Capacity	3.29		
Emergency Storage Calculation	**2011**	**2020**	**2030**
Firm Pumping Capacity (mgd)	3.29	3.29	3.29
Max Day Demand (mgd)	6.03	7.05	7.75
Emergency Storage = MDD-Firm Pumping Capacity x 8 hours	0.91	1.25	1.49

1.8.2.4 Total Recommended Storage for the Main Pressure Zone

The method for calculating total storage is to sum the equalization, fire, and emergency storage. Based on this method, the recommended total storage volume for 2011 conditions is 2.68 MG (0.68 MG equalization storage, 0.63 MG fire protection storage, and 0.91 MG emergency storage). The Main Pressure Zone currently includes 4.77 MG of storage. The total storage provided exceeds the recommended storage capacity. The current storage capacity is projected to be adequate through 2030 as shown in Table 1-29. As indicated in the table, the existing tanks provide a surplus of storage in the Main Pressure Zone.

TABLE 1-29
Main Pressure Zone Storage Capacity

Item	2011	2020	2030
Recommended Storage			
Equalization	680,000	795,000	874,000
Fire Protection	630,000	630,000	630,000
Emergency	913,200	1,253,000	1,487,000
Total	2,223,200	2,678,000	2,991,000
Existing Storage Capacity			
Newington Tank	1,500,000	1,500,000	1,500,000
Lafayette Road Tank	2,266,000	2,266,000	2,266,000
Spinney Road Tank	1,000,000	1,000,000	1,000,000
Total	4,766,000	4,766,000	4,766,000
Surplus/(Deficit)	2,542,800	2,088,000	1,775,000

1.8.3 Pease Pressure Zone Storage Capacity

Available storage capacity in the Pease Pressure Zone currently consists of the Hobbs Hill Tank and the New Hampshire Air National Guard (NHANG) Tank. Using the methodology described in Section 1.7.2, the total useable storage in the Pease Pressure Zone is currently 0.73 MG. The tank capacities are listed in Table 1-30.

TABLE 1-30
Available Storage in the Pease Pressure Zone

Tank	Total Storage (gal)	Useable Storage (gal)
Hobbs Hill Tank	400,000	366,000
NHANG Tank	400,000	366,000
Total	800,000	732,000

1.8.3.1 Equalization Storage

Equalization storage for the Pease Pressure Zone was calculated using the methodology described in Section 1.7.2. Three methods were utilized and compared with the results presented in the 2000 Master Plan.

Method 1

The hourly data used to create a diurnal curve was collected on July 22, 2011. The average demand in the Pease Pressure Zone on July 22 was 1.22 mgd and the peak hour demand was 1.87 mgd. The area under the diurnal curve and above the average demand was approximately 0.20 MG, or 16% of the daily demand. The diurnal curve and average demand are presented in Figure 1-13.

The equalization volume calculated based on July 22 data is 0.20 MG. The equalization storage required on the other available days was also checked and the July 22 storage volume was found to be the most conservative, with storage ranging from 0.14 MG on May 23, 2012 to 0.20 MG on July 22, 2011.

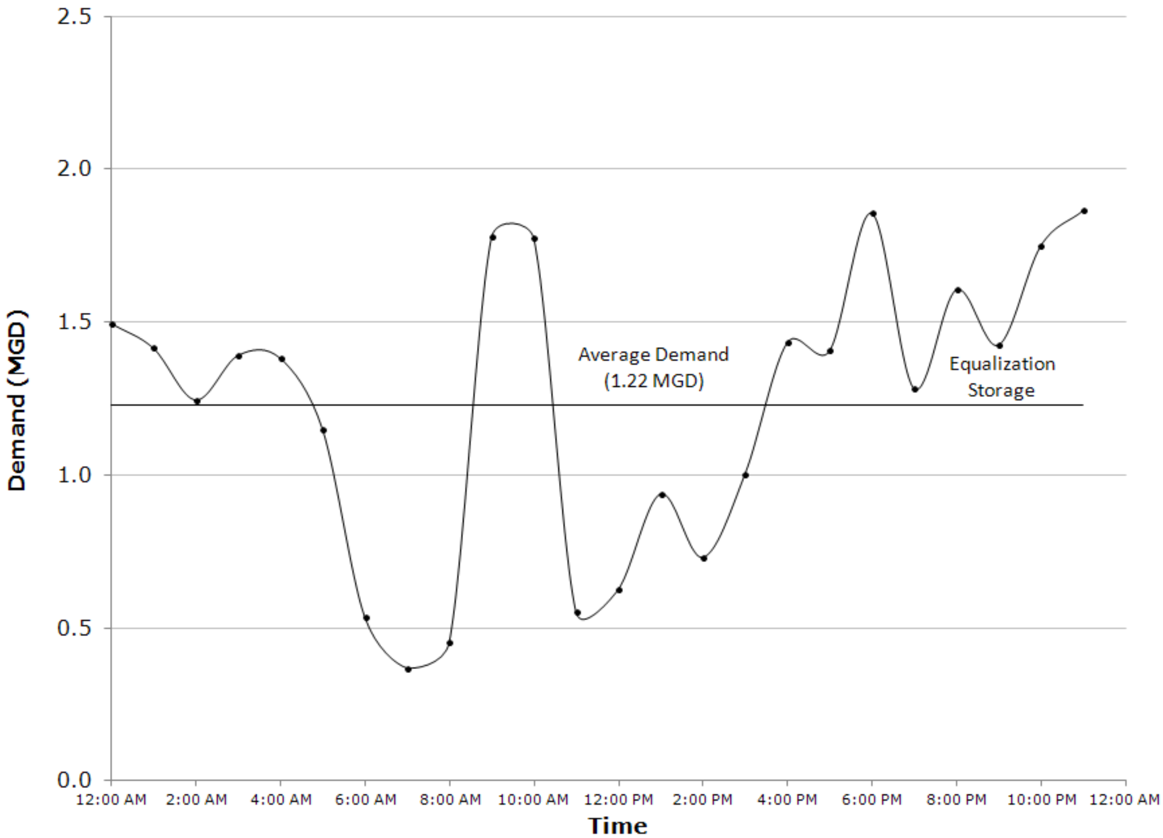

FIGURE 1-13
Diurnal Demand Curve for the Pease Pressure Zone on July 22, 2011

Method 2

Under Method 2, the equalization storage is estimated to be 20 to 25% of the MDD. The maximum day demand is 1.22 mgd; thus, the equalization storage calculated by Method 2 is in the range 0.24 to 0.31 MG.

Method 3

Under Method 3, the equalization storage is calculated as a function of the peak hour demand and the MDD, based on data from July 22, 2011. Thus, the equalization storage estimated by Method 3 is (peak hour demand - maximum day demand) x 8 hr = (1.87 mgd – 1.22 mgd) x 8 hr = 0.22 MG.

Summary and Current Practices

Table 1-31 summarizes the equalization volume calculations and related parameters.

TABLE 1-31
Summary of Equalization Storage Calculations – Pease Pressure Zone

Calculation Method	Volume (MG)
Method 1: Area under diurnal curve	0.20
Method 2: 20% - 25% of MDD	0.24 – 0.31
Method 3: (Peak hour - MDD) x 8hr	0.22
2000 Master Plan Method (15% of MDD)	0.18

As indicated in Table 1-31, Method 1 through 3 result in a range of 0.20 to 0.31 MG, and the 2000 Master Plan Method recommendation is 0.18 MG. We recommend the mid-range estimate of 0.22 MG provided by Method 3. Table 1-32 provides a summary and projection of equalization storage requirements projected through 2030, assuming the existing pressure zone configuration. For purposes of this analysis, future maximum day and peak hour demands are calculated based on the actual 2011 peaking factors.

TABLE 1-32
Method 3 Equalization Storage Calculation - Pease Pressure Zone

Item	2011	2020	2030
Average Day Demand (mgd)	0.46	0.52	0.60
Max Day Demand (mgd)	1.22	1.38	1.59
Peak Hour Demand (mgd)	1.87	2.11	2.44
Equalization Storage = Peak hour-MDD x 8 hours (MG)	0.22	0.24	0.28

Note:
1. Maximum day demand (MDD) and peak hour demand were derived from actual operating data on July 22, 2011 for the Pease Pressure Zone.
2. Projections of MDD for 2020 and 2030 above differ slightly from projected MDD as presented in Table 1-16 because peaking factors derived from the July 22, 2011 operational data were used in this analysis.

As discussed in Section 3, the possibility of combining sections of Greenland and Newington to the Pease Pressure Zone is considered. Table 1-33 provides equalization storage recommendations assuming that both Newington and Greenland are part of the Pease Pressure Zone.

TABLE 1-33
Method 3 Equalization Storage Calculation - Pease Pressure Zone, including Greenland and Newington

Item	2011	2020	2030
Average Day Demand (mgd)	0.62	0.69	0.79
Max Day Demand (mgd)	1.64	1.83	2.10
Peak Hour Demand (mgd)	2.52	2.81	3.21
Equalization Storage = Peak hour-MDD x 8 hours (MG)	0.29	0.33	0.37

1.8.3.2 Fire Protection Storage

Based on ISO recommendations, the maximum needed fire flow that a community water system is expected to provide is 3,500 gpm for 3 hours provided that there are commercial or industrial customers within the service area with needed fire flows that meet or exceed this value. A storage volume of 0.63 MG is recommended for fire protection storage. This number was also used in the 2000 Master Plan Report. It is noted that there is a total of 6.49 mgd pumping capacity available to augment fire protection storage in the Pease system, including the Smith, Haven, and Harrison Wells; Pease WTP booster station, abd Newington Booster pumps No. 1 and No. 3.

1.8.3.3 Emergency Storage

Emergency storage is designed to supply the system's average daily demand for the estimated duration of a possible emergency (e.g., water main break, supply source out of service). However, the availability of standby power and multiple water sources can lessen the need for emergency storage. The Pease system has multiple water sources that are capable of supplying more than the maximum day demand to the system. Sources available to provide water to the Pease system during power failures include the Haven Well, the Newington Booster Pumps No. 1 and 3 (currently valved off in the system), and the booster pumping system at the Pease WTP which can boost water from the Portsmouth system to Pease. The combined total capacity of all of these sources is 6.5 mgd, significantly higher than the maximum day demand (1.22 mgd). Supplies and demands for the Pease system are summarized in Table 1-34.

TABLE 1-34
Emergency Storage Calculation – Pease Pressure Zone

Pease Pressure Zone Supply Capacity			
Sources	**Capacity (mgd)**	**Power Failure Backup**	**Included in Firm Capacity?**
Smith Well	0.50	None	No
Haven Well	1.27	Engine drive	Yes
Harrison Well	0.32	None	No
Newington Booster #1	2.70	Generator	No
Newington Booster #3	0.97	Generator	No
Pease WTP Booster (one pump)	0.72	Generator	Yes
Total Capacity	6.49		
Firm Capacity	1.99		
Emergency Storage Calculation	**2011**	**2020**	**2030**
Firm Pumping Capacity (mgd)	1.99	1.99	1.99
Max Day Demand (mgd)	1.22	1.38	1.59
Emergency Storage = MDD-Firm Pumping Capacity x 8 hours	Firm Pumping Capacity > MDD - no additional storage required		

Table 1-35 provides supply and emergency storage projections for the Pease pressure zone assuming that the Greenland and Newington systems are part of the Pease pressure zone.

TABLE 1-35
Emergency Storage Calculation – Pease Pressure Zone, including Greenland and Newington

Pease, Greenland, and Newington Pressure Zone Supply Capacity			
Sources	**Capacity (mgd)**	**Power Failure Backup**	**Included in Firm Pumping Capacity?**
Smith Well	0.50	No	No
Haven Well	1.27	Engine drive	Yes
Harrison Well	0.32	No	No
Newington Booster #1	2.70	Generator	No
Newington Booster #3	0.97	Generator	No
Greenland Well	0.72	Engine drive	Yes
Pease WTP Booster (one pump)	0.72	Generator	Yes
Total Capacity	7.21		
Firm Capacity	2.71		
Emergency Storage Calculation	**2011**	**2020**	**2030**
Firm Pumping Capacity (mgd)	2.71	2.71	2.71
Max Day Demand (mgd)	1.64	1.83	2.10
Emergency Storage = MDD-Firm Pumping Capacity x 8 hours	Firm Pumping Capacity > MDD - no additional storage required		

As indicated in Tables 1-34 and 1-35, owing to firm supply capacity in excess of the maximum day demand, the recommended equalization and fire protection storage will be adequate for the Pease pressure zone with no additional emergency storage required.

1.8.3.4 Total Recommended Storage for the Pease Pressure Zone

The method for calculating total storage is to sum the equalization, fire, and emergency storage. The Pease Pressure Zone currently includes 0.73 MG of storage including the Hobbs Hill and NHANG tanks. As discussed in Section 2, the Hobbs Hill Tank is due for either rehabilitate or replacement, with the possibility of replacement with a larger tank under consideration. Table 1-36 summarizes storage capacity recommendations for the Pease pressure zone assuming the existing pressure zone configuration. Recommended active storage volume for the proposed replacement Hobbs Hill tank is provided in the table.

TABLE 1-36
Pease Pressure Zone Storage Capacity

Item	**2011**	**2020**	**2030**
Recommended Storage			
Equalization	220,000	240,000	280,000
Fire Protection	630,000	630,000	630,000
Emergency	0	0	0
Total	850,000	870,000	910,000
Existing Storage Capacity			
Hobbs Hill Tank	366,000	366,000	366,000
NHANG Tank	366,000	366,000	366,000
Total	732,000	732,000	732,000
Surplus/(Deficit) w/ Existing Hobbs Tank	(118,000)	(138,000)	(178,000)
Replacement Hobbs Tank Size to Provide Recommended Storage	484,000	494,000	524,000

As discussed in Section 3, combining parts of Greenland and Newington with the Pease Pressure zone is Considered. Table 1-37 summarizes storage capacity recommendations assuming that Greenland and Newington are connected to the Pease Zone.

TABLE 1-37
Pease Pressure Zone Storage Capacity – Newington and Greenland Connected to Pease Zone

Item	2011	2020	2030
Recommended Storage			
Equalization	290,000	330,000	370,000
Fire Protection	630,000	630,000	630,000
Emergency	0	0	0
Total	920,000	960,000	1,000,000
Existing Storage Capacity			
Hobbs Hill Tank	366,000	366,000	366,000
NHANG Tank	366,000	366,000	366,000
Total	732,000	732,000	732,000
Surplus/(Deficit) w/ Existing Hobbs Tank	(188,000)	(228,000)	(268,000)
Replacement Hobbs Tank Size to Provide Recommended Storage	554,000	594,000	634,000

As indicated in Table 1-37, a total storage capacity of 1 MG is recommended for the year 2030, assuming that Newington and Greenland are connected to the Pease zone. If the Hobbs Hill tank is replaced with a new tank, a minimum useable volume of 634,000 gallons would be required to provide the recommended capacity. Therefore, if the Hobbs Hill tank is replaced with a new tank, an elevated tank with a capacity of 634,000 gallons is recommended.

1.9 References

Water Resource Engineering (2nd Edition), David A. Chin, Prentice Hall, 2006

“Determining Distribution System Storage Needs”, Murat Ulasir, Robert Czachorski, and Vyto Kaunelis, Opflow, American Water Works Association, 2005

“Distribution System Requirements for Fire Protection”, AWWA Manual M31 (Fourth Edition), American Water Works Association, 2008

Water Resources Planning (Manual M50), American Water Works Association, 2001

“The Massachusetts Sustainable-Yield Estimator: A decision-support tool to assess water availability at ungagged sites in Massachusetts,” United States Geological Survey, Scientific Investigations Report 2009–5227

Section 2
Storage Tank Conditions Assessment

2.1 Introduction

The City of Portsmouth owns, operates and maintains five water distribution tanks storage facilities. In addition, the NH Air National Guard (NHANHG) Elevated Tank is connected to the Pease distribution system, although not maintained by the City. The following table identifies these six water distribution tanks:

TABLE 2-1
Water Storage Facility Information

Tank	Pressure Zone	Capacity (MG)	Type	Year Built	Elevation (ft MSL)		Tank Dimensions (ft)	
					Overflow	Base	Dia.	Ht.
Newington	Suction side of booster Station	1.5	Welded Steel	1957	142	102	80	40
Lafayette Road	Portsmouth	7.5	Welded Steel	1995	171	75	114	96
Spinney Road	Portsmouth	1.0	Composite	2002	171	51	70	40
Hobbs Hill	Pease	0.4	Elevated Welded Steel	1957	230	90	48	35
Osprey Landing	Portsmouth	0.2	Elevated Welded Steel	1941	170	100	38	~25
NHANG	Pease	0.4	Elevated Welded Steel	1957	230	90	48	35

All tanks listed above, with the exception of the Osprey Landing and NHANG Storage Tanks, were inspected as part of this evaluation.

This Section provides recommendations to improve the long-term condition of the storage tanks. The evaluation includes the technical documentation to support the recommendations and opinions of probable cost for implementation of the recommendations.

2.2 Tank Inspection Dates

Inspections were completed from June 11, 2012 to June 15, 2012. Tighe & Bond subcontracted the tank inspection services to Utility Service Company. A representative from Tighe & Bond and the City of Portsmouth Department of Public Works were also present to witness the four (4) tank inspections. Copies of the Tank Inspection Reports prepared by Utility Service Company are enclosed as appendices with this report. The tanks were inspected to assess the coating, structural, sanitary, safety and security conditions. In addition, exterior and interior paint samples were taken and analyzed at a certified laboratory to determine the total lead and chromium content of the existing coatings. The table below provides the dates and method of inspection for each tank.

2.3 Method of Interior Tank Inspections

The four water storage tank inspections were conducted while the tanks were "in service". For the purpose of the "in-service" water storage tank inspections, a remote operated vehicle (ROV) was used to inspect the interior of the tank. This specially designed underwater unit eliminates the need for divers to enter the tanks or removing tanks from service. The video from the ROV mounted underwater camera is viewed onsite by the inspector/operator and was documented on video in digital format. Prior to being placed in the tank, the ROV equipment was disinfected in accordance with all guidelines set forth in AWWA C652-02 Standards.

TABLE 2-2
Tank Inspection Summary

Tank Location	Date of Inspection
Hobbs Hill	June 11, 2012
Spinney Road	June 12, 2012
Lafayette Road	June 14, 2012
Newington	June 15, 2012

A brief description of the findings during each tank inspection is provided below. Complete inspection reports prepared by Utility Services Company have already been provided to the City, along with digital disks of the reports and inspection videos, under separate cover.

2.4 Hobbs Hill Tank

2.4.1 Tank Description

The Hobbs Hill Tank was inspected on June 11, 2012. The 400,000 gallon elevated tank was built in 1957 and is approximately 35 feet high with a 48-foot diameter. When full, this tank holds approximately 400,000 gallons of water.

2.4.2 Inspection Results

The exterior and interior coatings are no longer providing an effective corrosion barrier to the underlying steel surfaces. If the existing were to remain in service, it is recommended that tank be completely rehabilitated as soon as possible to prevent aggressive metal loss of the exposed steel substrate along the interior and exterior surface as a result of the degrading exterior coating. Due to the extensive cost to repair the existing tank, it is recommended that the City of Portsmouth consider replacing it with a new water storage tank.

2.4.3 Coating Conditions

If rehabilitated, the exterior coating of the tank should be abrasive blast cleaned to a SSPC-SP-6 commercial blast surface preparation and painted with a four-coat zinc/epoxy/urethane coating which includes two coats of an acrylic polyurethane finish. To complete this work, it will be necessary to completely encapsulate localized areas of the tank or even the entire tank structure with Class IA containment during the blast cleaning operations in order to not adversely impact the surrounding grounds.

A dense network of discarded cathodic rods and cables were observed in the base of the bowl which prevented inspection of these surfaces. The coatings of the shell and remaining bowl surfaces that could be observed, demonstrated extensive blistering of the interior coating. In addition, failure to the substrate and subsequent corrosive activity was also witnessed. Thus, the interior surfaces including the roof, shell, bowl and riser will need to be abrasive blast cleaned to an SSPC-SP-10 Near White Metal surface preparation and painted with a three-coat Zinc/Epoxy system.

2.4.4 Structural Conditions

If the existing tank were to remain in service, it is recommended that structural repairs be performed. Most of the following repairs would be completed prior to surface preparation and re-coating. Repairs that need to be completed after the blasting and re-coating should be re-cleaned and primed as originally specified.

- Clean and paint the roof hatch.
- Replace the existing hinges for the cover to the weir box and re-weld them to both the top of the weir box and the cover.
- Replace the overflow pipe support brackets and re-weld to the pipe and leg column.
- Cut additional drainage holes along the low lying areas of the walkway to prevent water retention.
- Repair the anchor bolt assemblies along the base of the riser as they are demonstrating severe metal loss.
- Remove and replace damaged concrete surfaces along the concrete footings. Also, clean all concrete surfaces above ground as needed and apply a sealer to preserve the integrity.

- Spot weld areas of pitting that represent a 35% or greater reduction in plate thickness to bring the pits to at least flush with the original plate surface.
- Re-weld all areas all areas of weld seams exhibiting severe undercut or sectional loss in order to restore the sectional loss and provide a weld crown at least level with the parent metal.
- Fill areas of pitting or metal lost representing 20% to 35% reduction in plate thickness with NSF approved 100% epoxy filler/surfacer to bring the surface flush with original plate surface.

2.4.5 Sanitary Conditions

The entire finial vent, inclusive of the center stub, should be completely replaced as soon as possible with a new center stub receiver flange and aluminum freeze/vacuum resistant vent. In addition, the overflow pipe should be fit with a one way check valve or a bolting flange in which a 24-mesh screen and assembly can be installed.

2.4.6 Safety and Security Conditions

The structural integrity of the existing ladder assembly and finial vent are questionable due to substantial corrosive activity and moderate metal loss. As noted above, the entire finial vent should be replaced. In addition, the existing roof/shell ladder assembly should be permanently mounted to the roof and shell surfaces by welding steel standoffs to the roof/shell surfaces and the rails of the ladder.

If the existing tank were to be rehabilitated, the height of the handrails should be increased to meet OSHA regulations and consideration should be given to a second roof hatch 180 degrees from the existing hatch with safety railings along both the hatches.

2.4.7 Cost Impacts

Utility Service Company's estimated cost for the scope of work described above is $654,000. This price includes abrasive blasting and re-coating and repairs for welding 500 pits, 50 linear feet of seams, 10 square feet of patching and 20 gallons of epoxy filler/surfacer. It does not include engineering specifications or any other costs of an on-site construction inspector. Therefore, for budgetary purposes, overall project costs would range from $800,000 to $900,000.

Due to the extensive rehabilitation cost for the existing Hobbs Hill tank, it is recommended that the City of Portsmouth consider replacing it with a new water distribution storage tank. Further discussion of this recommendation is included in the Recommendations section of this report.

2.5 Spinney Road Tank

2.5.1 Tank Description

The Spinney Road Tank was inspected on June 12, 2012. The 1,000,000 gallon composite elevated tank was built in 2002 and is approximately 40 feet high with a 70-foot diameter bowl.

2.5.2 Inspection Results

The interior and exterior surfaces of the existing tank do not require immediate rehabilitation. The tank should be re-inspected in five years to reassess the tank condition. There a few immediate concerns that should be addressed relating to the sanitary and structural condition of the tank that are summarized below.

2.5.3 Coating Conditions

As summarized below, the antenna mount pipe on the exterior of the roof has split open and should be repaired. Once this has been repaired, the coating failure and the rusting along the exterior of this pipe should be power tool cleaned in accordance with the SSPC-SP-3 surface preparation, cleaned and painted with two coats of a surface tolerant, modified urethane primer.

In addition, if the tank should be de-watered for any reason prior to the next inspection in five years, consideration should be given to performing spot cleaning and painting of two areas observed to have rust tubercle formations on the interior shell.

2.5.4 Structural Conditions

One of the antenna mounts on the exterior roof has split open due likely as a result of ice buildup within the pipe. The top of the pipe was not plugged allowing rain water to enter the pipe. It is recommended that the top of the pipe be sealed with a cap. In addition, the split in the pipe should be welded to prevent water from entering the pipe.

2.5.5 Sanitary Conditions

The handle and latching devices along the roof hatches are difficult to open. Also, the T-handle of the hatch into the water chamber is cracking. It is recommended that these devices be replaced to ensure they continue properly function.

Two 2-inch diameter couplings and two 1-inch diameter couplings that penetrate the access tube on the roof are partially sealed with a mastic material. Consideration should be given to apply additional elastomeric sealer to the exterior of the openings of these penetrations to ensure they remain sealed.

2.5.6 Safety and Security Conditions

Replacement of the T-rail safety climb system is recommended. The lower portion of the climb system has widespread corrosion deposits and is no longer operable.

2.5.7 Cost Impacts

Utility Service Company's estimated cost for rehabilitation to the antenna mounting poles is $4,900.

2.6 Lafayette Tank

2.6.1 Tank Description

The Lafayette Tank was inspected on June 14, 2012. The 7.5 MG ground storage tank was built in 1995 and is approximately 96 feet high with a 114-foot diameter.

2.6.2 Inspection Results

The interior and exterior surfaces of the existing tank do not require immediate rehabilitation. It is recommended that the tank be re-inspected in three years to reassess the tank's condition. Also, there a few immediate concerns that should be addressed relating to the sanitary, safety and security conditions of the tank that are summarized below.

2.6.3 Coating Conditions

The protective coatings along the interior and exterior surfaces of the tank continue to provide an acceptable level of protection. The coatings on the exterior of the tank are in very good condition. The presence of scattered rust tubercles along the interior shell surfaces were observed. Thus, it is recommended that the tank be re-inspected in 2015 to review whether there is further degradation or metal loss along the interior shell surface.

Over 1 inch of sediment was observed at the time of the inspection which obscured visual assessment of the floor of the tank. During the next inspection, consideration should be given to dewatering the tank to allow for removal of the existing sediment build-up in order thoroughly assess the floor surfaces and perform any spot maintenance that may be required.

2.6.4 Structural Conditions

Overall the tank appeared to be in sound structural condition at the time of inspection. No immediate repairs are required at this time.

2.6.5 Sanitary Conditions

The protective screen has been torn away from the finial vent assembly. A new stainless steel screen should be installed to prevent contamination to the water supply (it is our understanding that the City of Portsmouth has completed this work). Also, the overflow pipe discharge is less than 12 inches above ground level. It is recommended the rip-rap that the overflow pipe discharges to be excavated to increase separation between the end of the pipe and ground to provide at least 12 inches to be in accordance with AWWA recommendations.

2.6.6 Safety and Security Conditions

The FAA lights on the roof lacked any bulbs or globes. If functionality of the FAA lights is required, the bulbs and globes should be replaced as soon as possible. Consideration should also be given to improving the bottom two sections of the ladder cage in order to prevent unauthorized access. The bottom two sections could be improved by installing additional slats between the existing vertical slats with a lockable gate at the opening or by removing the bottom two sections of the cage and installing a hinged, lockable gate which completely encapsulates at least eight feet of the access ladder.

2.6.7 Cost Impacts

Utility Service Company's estimated cost to address the immediate sanitary, safety and security concerns outlined above is $4,700.

2.7 Newington Tank

2.7.1 Tank Description

The Newington Booster Station Tank was inspected on June 15, 2012. The 1.5 MG ground storage tank is approximately 40 feet high with an 80-foot diameter.

2.7.2 Inspection Results

The interior and exterior surfaces of the existing tank do not require immediate rehabilitation. It is recommended that the tank be re-inspected in three years to reassess the tank's condition. Also, there are a few immediate concerns that should be addressed relating to the sanitary, safety and security conditions of the tank that are summarized below.

2.7.3 Coating Conditions

The protective coatings of the exterior and interior are in generally in fair to good condition. The coating on the exterior of the roof is poor condition but no significant corrosion appears to be associated with the coating failure to date. Thus, the protective coatings along the interior and exterior surfaces of the tank are still providing an acceptable level of protection. It is recommended that the tank be re-inspected in 2015 to reassess the exterior and interior coating condition.

A uniform layer of silt was observed at the time of the inspection which obscured visual assessment of the floor of the tank. However, there was no visual evidence of coating failure or corrosion protruding through the silt layer. There was some evidence of rust tubercle formation or surface corrosion along the lower bottom ring surfaces and junction with the floor but the degree of deterioration could not be assessed due the silt layer. During the next inspection, consideration should be given to dewatering the tank to allow for removal of the existing sediment in order thoroughly assess the floor surfaces and perform any spot maintenance that may be required.

Piping inside the valve vault and booster station vault do not appear to be coated and are exhibiting uniform surface corrosion.

2.7.4 Structural Conditions

Overall the tank appeared to be in sound structural condition at the time of inspection. No immediate repairs are required at this time.

Soil encroachment along portions of the tank foundation perimeter should be cleared back to improve site drainage. Also, the grout at the floor plate extension to foundation junction is in poor condition and will require removal and replacement during the next tank maintenance.

2.7.5 Sanitary Conditions

In order to prevent large unsealed openings into the water chamber, the screening within the finial vent assembly should be replaced with new stainless screen or pallet assembly that is cut and fit to the inside edge of the finial cap.

2.7.6 Safety and Security Conditions

The shell and roof ladder assembly should be equipped a fall prevention device in order to ensure compliance with OSHA regulations. Consideration should also be given to improving the bottom two sections of the ladder cage in order to prevent unauthorized access. The bottom two sections could be improved by installing additional slats in between the existing vertical slats or by removing the bottom two sections of the cage and installing a hinged, lockable gate which completely encapsulates at least eight feet of the access ladder.

Installation of a second roof hatch that is 180 degrees is recommended during the next tank maintenance operation to aid in compliance with OSHA confined space guidelines. Also during the next maintenance, the roof ladder assembly should be secured. The roof ladder is currently secured to the shell ladder by a rope. A securing tab should be welded to the roof or knuckle plate to secure the roof ladder assembly in a fixed position within reach of the shell access ladder.

2.7.7 Cost Impacts

Utility Service Company's estimated cost to address the immediate sanitary, safety and security concerns outlined above is $6,800. This cost does not include the installation of a second hatch opening.

2.8 Tank Rehabilitation Cost Summary

The following table provides a summary of costs to implement the recommended repairs for each tank included in this report.

TABLE 2-3
Tank Rehabilitation Cost Summary

Tank	Rehabilitation Costs
Hobbs Hill Tank[1]	$800,000 to $900,000
Spinney Road Tank	$4,900
Lafayette Tank	$4,700
Newington Booster Tank	$6,800
Total Cost for all Recommended Tank Improvements[2]	**$816,400 to $916,400**

Notes:
1. Estimated costs provided by Utility Services Company for the Hobbs Hill Tank totaled $654,000 and did not include engineering, specification and contract generation. Therefore, for the purposes of CIP planning it is estimated that the final cost of this rehabilitation would range from $800,000 to $900,000.
2. The estimated costs provided by Utility Services Company are based on current 2012 pricing.

Section 3
Distribution System Assessment

3.1 GIS and Hydraulic Model Development and Calibration

A hydraulic model of the Portsmouth Water System was originally developed by EarthTech in 2000 as part of the development of a Water Master Plan. Their model was used to make recommendations for water main improvements and other distribution system improvements.

The focus of the Phase III Master Plan included developing an updated hydraulic and water quality model of the Portsmouth Water System using InfoWater (Innovyze, Arcadia, CA), to be used to identify distribution system improvements projects. Infowater Suite 8.5 was the current software package utilized for this effort.

The City provided Tighe & Bond with a geodatabase of the system piping and infrastructure. The geodatabase was developed by the City using ArcGIS (ESRI Inc., Redlands, CA). The geodatabase provided by the City contained several water distribution system layers, including the following mapping features:

- Water pipes and fittings
- Tanks
- Pump stations
- Wells
- Reservoirs
- Treatment facilities
- Hydrants
- PRVs
- Meters

The pipe layer includes 10,025 pipe segments, representing all the water mains in the system, including both active, abandoned, and private pipes. The area served by the system and represented in the data includes the City of Portsmouth and portions of the neighboring towns of Greenland, Rye, New Castle, and Newington.

To be useful for model development, GIS data must include the length and diameter of the pipes, as well as information that will provide a basis for estimating friction factors. Friction factors may be estimated based on material and age, hydrant flow test data, or data from previous models. Since the information available relative to private water mains is limited, it was decided to exclude them from the active hydraulic model. The active mains owned by the City comprise approximately 8,500 pipe segments as represented in the GIS data. Material is known for approximately 85% of these segments, and year of installation is known for approximately 52%.

Tables 3-1 and 3-2 summarize the pipes that are active in the hydraulic model by diameter and material. The majority of the pipes in the system range from 6 to 12 inches in diameter. The majority of the pipes in the system are cast iron or ductile iron with some asbestos cement. Seventeen percent of the pipe material is unknown.

TABLE 3-1
Pipes in Portsmouth System by Diameter

Diameter (in)	Length (ft)	Length (miles)	Percent of Total
<=4	70,953	13.4	7.6%
6	234,362	44.4	25.2%
8	256,323	48.5	27.6%
10	100,632	19.1	10.9%
12	167,454	31.7	18.0%
14	882	0.2	0.11%
16	20,170	3.8	2.2%
20	36,731	7.0	4.0%
24	41,639	7.9	4.5%
Total	929,146	176.0	100%

TABLE 3-2
Pipes in Portsmouth System by Material

Material	Length (ft)	Length (miles)	Percent of Total
Cast Iron	393,375	74.5	42.3%
Ductile Iron	259,135	49.1	27.9%
Asbestos Cement	63,141	12.0	6.8%
Copper	35,409	6.7	3.8%
Cement Lined Cast Iron	16,612	3.1	1.8%
Unknown/Other	161,474	30.6	17.4%
Total	929,146	176.0	100%

3.1.1 Hydraulic Model Development

3.1.1.1 Building the Hydraulic Model from GIS Layers

Water mains from the GIS data were imported into the model using the InfoWater software package. In addition to the pipes, the InfoWater software requires a set of junctions (also known as nodes) that represent intersections between pipes. Nodes carry essential information including how the pipes are connected to each other, as well as system demands and elevations. Nodes were created automatically by the InfoWater software at each pipe end as represented in the GIS data. The InfoWater node creation algorithm assumes that if the ends of two pipe segments as represented in the GIS data are within a specified distance of each other (1 foot used in this case), they are connected, and a node is created. There are 8,096 active model nodes. After the automatic node creation process was completed, the model was checked to verify proper connectivity. The connectivity check resulted in the addition of several pipe segments to correct connectivity issues:

- Segments were added at the Newington tank and booster station to accurately represent the flow paths to the tank and through the pump station

- Pipe segments were added and some existing segments were closed along the transmission main in Newington to correct cross connections between the transmission main and Portsmouth pressure zone
- A missing pipe segment was discovered and added at the intersection of Maplewood Ave. and Woodbury Ave.

In order to identify mains in the GIS data that are utilized in the hydraulic model during hydraulic calculations, a new field was created in the database to describe each pipe as "active" or "inactive". Inactive pipes were selected as pipes that either are privately owned, identified as abandoned in the GIS data, or having an unknown diameter. Both active and inactive pipes are carried in the model database; however, only active pipes are used for model calculations. There are 8,523 active pipe segments in the model.

3.1.1.2 Operational Information for Each Facility

Refer to Figure 3-1 for a distribution system schematic of the Portsmouth Water System. Operational information for each pump station, well, treatment facility, and reservoir was imported into InfoWater using data available in the existing Master Plan reports and the previously developed model. Operational information imported included the following parameters:

- Well water depth
- Pump parameters, including pump diameter, horsepower, shutoff head, and pump curves
- Tank minimum and maximum height, minimum volume, and diameter

The Madbury Water Treatment Plant, Newington Booster Station, wells, tanks, and control valves were entered into the model manually and checked for proper function.

3.1.1.3 Elevation Data

Two-foot contour elevation data obtained from NHGRANIT were utilized for assigning elevations to model nodes. The surface elevation of each node was calculated by linear interpolation based on the nearest 2-foot contours. Elevations of the nodes carried in the model data are approximate surface elevations.

3.1.1.4 System Demands

Customer consumption data for 2011 was used to create a base system demand. The customer data was provided by the City in spreadsheet format and included the total water billed during 2011 for every customer account number in the system, together with the addresses of the customers. The addresses were assigned geographic locations (in a process called "geocoding") and a spatial layer of points was created, each point representing a customer billing address. The ArcGIS software package was used to identify and assign the nearest model node to each customer address. Spreadsheets were used to calculate the average consumption rate in gpm for each customer during 2011 and total these consumption rates for each model node. These average demands were then imported into the model as "base demands" for each model node. Table 3-3 presents a summary of actual water demand versus the amount successfully geocoded and imported into the model as base demand. As indicated in the table, approximately 94% of the billed consumption was successfully imported into the model as base demand.

Insert Figure 3-1

Distribution System Schematic

TABLE 3-3
Summary of System Demands and Model Base Demand

Item	Total (MG)	Average daily demand (gpd)	Average flow rate (gpm)
Total water usage for 2011	1,571	4,304,760	2,989
Billed consumption for 2011	1,409	3,861,559	2,682
Successfully geocoded and imported into model	1,319	3,613,127	2,509

3.1.1.5 Base System Demand and Diurnal Daily and Monthly Patterns

As discussed above, the spatial allocation of demands was based on customer billing data for 2011 and thus represents the average spatial distribution of demands in the system over a year. In order to model high demand periods and to capture the fluctuation of demands during the day, the base demands are varied by applying adjusting factors.

To account for daily patterns and for the unallocated demands, patterns were developed for two 48-hour periods, representing an average demand condition and a high demand condition. Separate patterns were developed for the Pease pressure zone and the main zone. These patterns were based on operational data provided by the City for May 22-23, 2011 and July 21-22, 2011. The operational data includes water production data from all sources; Newington booster station flow data; and tank level data from the Spinney, Lafayette, Newington, and Hobbs tanks; all on hourly or shorter intervals. Overall system demands were determined by mass balance using this data. For each hour, the main system demand was calculated as the difference between the volume supplied by the Newington booster station and Greenland, Portsmouth, and Collins wells and the volume stored or released from storage in the Lafayette and Spinney tanks. The Pease system demand was calculated as the difference between the volume supplied by the Smith, Harrison, and Haven wells and the volume stored or released from storage in the Hobbs and NHANG tanks. It is noted that level data was not available for the NHANG tank; the NHANG tank level was assumed to correspond with the Hobbs tank level.

Figures 3-2 and 3-3 present system demands for the two base simulations as represented in the model.

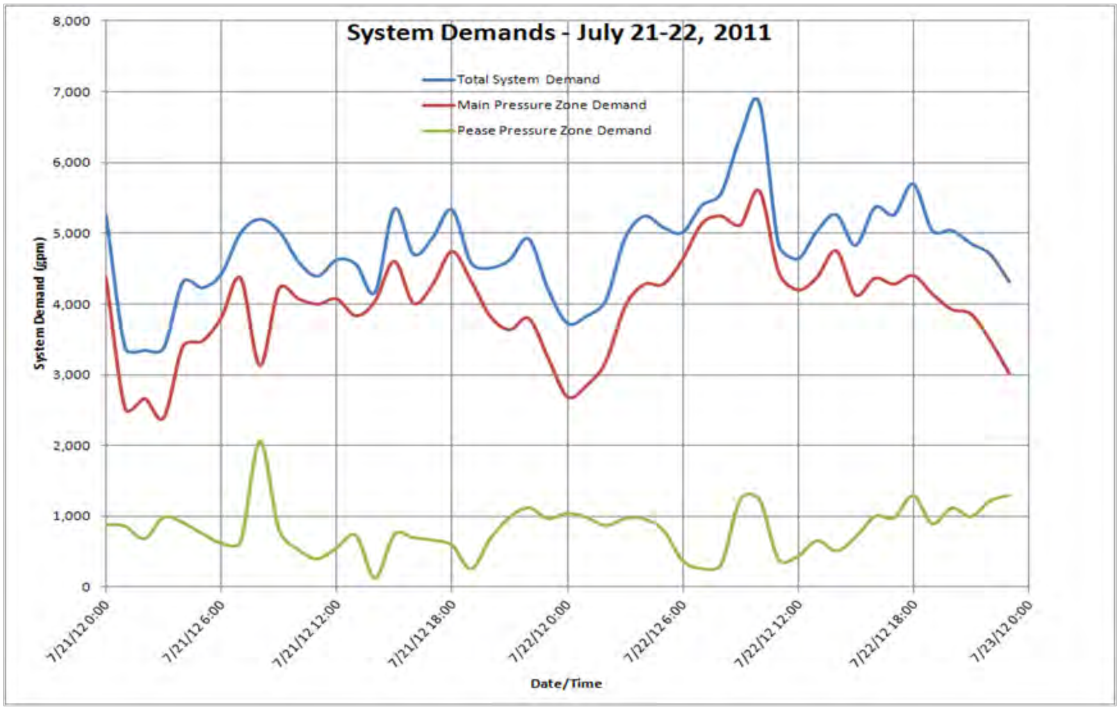

FIGURE 3-2
System Demands – July 21-22, 2011

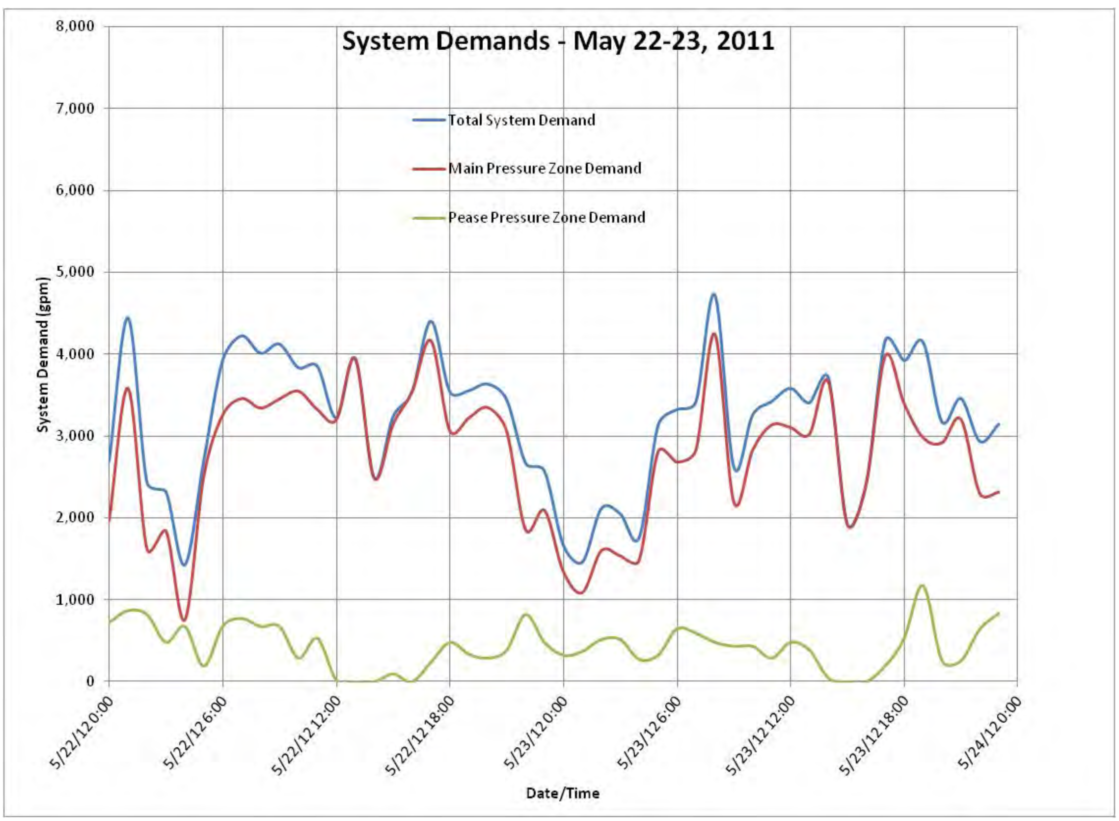

FIGURE 3-3
System Demands – May 22-23, 2012

Average system demands for the two representative 48-hour periods are presented in Table 3-4.

TABLE 3-4
Average System Demands for Representative 48-hour Periods

	May 22-23, 2011		July 21-22, 2011	
	(gpm)	(mgd)	(gpm)	(mgd)
Main Pressure Zone	2,691	3.9	3,843	5.5
Pease Pressure Zone	439	0.6	820	1.2
Total	3,130	4.5	4,663	6.7

3.1.1.6 Pipe Friction Factors

Initial assignment of friction factors to pipes was based on pipe material, size, and average flow velocity as predicted under average day conditions. A summary of initial friction factors is presented in Table 3-5.

TABLE 3-5
Pipe Friction Factor Summary

Material	Nominal diameter (in)	Model-predicted average flow velocity (ft/sec)	Assigned C-Factor
Ductile iron, cement lined cast iron	<=6	All	95
Ductile iron, cement lined cast iron	8-12	All	100
Ductile iron, cement lined cast iron	>12	All	105
Plastic, Copper, AC, concrete	All	All	105
Unknown material	<=6	<=0.05	60
Unknown material	<=6	0.05-1	80
Unknown material	<=6	>1	90
Unknown material	8-12	<=0.05	70
Unknown material	8-12	0.05-1	90
Unknown material	8-12	>1	100
Unknown material	>12	<=0.05	80
Unknown material	>12	0.05-1	100
Unlined cast iron, steel	<=6	<=0.05	35
Unlined cast iron, steel	<=6	0.05-1	60
Unlined cast iron, steel	<=6	>1	80
Unlined cast iron, steel	8-12	<=0.05	40
Unlined cast iron, steel	8-12	0.05-1	80
Unlined cast iron, steel	8-12	>1	90
Unlined cast iron, steel	>12	<=0.05	50
Unlined cast iron, steel	>12	0.05-1	90
Unlined cast iron, steel	>12	>1	100

3.1.2 Model Calibration

The model was checked and friction factors were refined by preparing simulations of recent flow test results. A comparison of flow test results and model predictions is presented in Table 3-6.

TABLE 3-6
Comparison of Flow Test Results with Model Predictions

Flow Test Location	Date of Test	Test Flow Rate (gpm)	Observed Static Pressure (psi)	Observed Residual Pressure (psi)	Model Predicted Static Pressure (psi)	Model Predicted Residual Pressure (psi)
Alden Ave Greenland	4/4/11	1,007	51.5	34	56	38
Ocean Rd Greenland	4/14/11	1,034	44	35	46	34
West Rd.	9/29/11	978	50	34	47	42
Lafayette Rd @ McKinley Rd	9/29/11	822	42	31	42	39
Portsmouth Ave Greenland	9/21/11	934	51	38	55	44
Newington Rd Newington	5/26/10	557	55	15	56	15
Corporate Drive/Oak Ave	5/12/11	1,300	80	74	85	75
International Dr/Oak Ave	5/12/10	978	68	67	71	70
Beane Lane Newington	5/28/10	787	50	25.5	47	0
Shattuck Way	7/22/11	1,198	42	36	46	36

As indicated in the table, model predicted static pressures are all within ±5 psi of observed pressures, as are the majority of residual pressures. For the tests at West Road, Lafayette Road at McKinley Road, and Beane Lane in Newington, the model predicted pressure drop during the test was more than 5 psi. For the West Road and Lafayette Road tests, the observed pressure drop during the test was significantly greater than the predicted drop, even with model C-factors set very low. In light of the proximity of these test locations to the Lafayette tank, these results suggest the likelihood of a hydraulic restriction in this area (e.g., a closed valve) that is not reflected in the model. For the Beane Lane test, the model predicts a much greater pressure drop during the test than the observed drop. Both these areas warrant additional evaluation to refine the model calibration.

3.2 Summary of Findings from Previous Hydraulic Analyses

The Phase I Water Master Plan, developed in 2000, included an analysis of the adequacy of the existing system to supply water for storage and fire protection. The following conclusions were included in the report:

- **Fire Flow Capacity:** Several areas of the Main Pressure Zone were deficient in fire flow capacity. They included:
    - Downtown Portsmouth,
    - New Castle,
    - Portions of Greenland,

  - Saratoga Way and Atlantic Heights, and the area near Sherburne Road.
  - The northern portion of the Pease system was deficient in fire flow capacity at higher elevations.

- **High Velocities:** Several areas of the distribution system had high pipe velocities (greater than 2 feet per second (fps)). During peak demands, areas of the Main Pressure Zone approach 4 fps. Velocities between 3 and 4 fps were observed between the Haven Well and the distribution system for present day conditions. The Phase 1 Water Master Plan model projected velocities of 5 to 9 fps under 2020 conditions from the Newington Booster Station to the Pease Pressure zone.

- **System Pressures:** Locations in the Main Pressure Zone experience pressures lower than 30 psi during certain periods.

- **Headloss**: The headloss in the distribution system between the Newington Booster Station and the Lafayette Road standpipe is significant, and was noted as restricting the filling of the tank during peak demand periods. This assessment was observed in the model and confirmed by observations made by operators.

- **Flow Capacity:** Pipes in the downtown area of Portsmouth have only 20 to 30 percent of their original flow capacity due to age and reduced diameters.

- **Looping:** Eliminating dead ends in the system would improve fire flow and water quality.

- **Pipe Conditions:** Unlined cast iron piping was prevalent in downtown Portsmouth, and some wooden piping still existed in the system. Replacement of the older unlined pipe and wooden piping was recommended.

- **Piping Reliability:** Over 60% of the potable water supply (Madbury WTF to Newington) is conveyed through a single pipeline. If the pipeline failed, the system would lose a significant amount of its supply.

The Master Plan included a list of recommended distribution system piping improvements and probable costs.

The Phase II Master Plan, developed in 2003, included a recommendation for improving the Greenland area. It was noted at that time that the Greenland Well was operated continuously in order to maintain system pressure in the Greenland area. A new Greenland Pressure Zone was suggested as a potential solution.

3.3 Hydraulic Analysis of Distribution System

The model was used to predict distribution system hydraulic response under maximum and average day demands and to simulate several scenarios to mitigate hydraulic deficiencies.

3.3.1 Standard Demand Scenarios

As described in Section 3.1, billing information from 2011 was used to allocate demands to individual nodes. The demands that were assigned to nodes were adjusted by factors to represent average and high demand conditions. Forty-eight hour scenarios were developed representing average and maximum demand conditions. The system-wide, 48-hour average demands for the average condition and maximum demand condition scenarios were 4.5 mgd and 6.7 mgd, respectively. The maximum demand scenario contains one 24-hour period with an average demand of 7.3 mgd and a peak-hour demand of approximately 10 mgd.

The system controls and operational patterns were modeled to mimic the typical recommended daily operating parameters for the Portsmouth water system (current operating staff do vary these parameters based on their own discretion). The Madbury Wells and WTF were operated on set patterns based on actual operations on July 21-22, 2011 and May 22-23, 2012. The Haven, Harrison, and Smith Wells were set to turn on and off based on Hobbs Hill Tank level set points. The Collins Well, Portsmouth #1 Well, and Greenland Well were set to turn on based on a Spinney tank level set point and turn off based on a Lafayette tank level set point. The Newington Booster Station pump VFDs were set to ramp up and down based on the Newington Tank level.

Plots of model-simulated tank levels versus time for average day and maximum day demand simulations are presented in Figures 3-4 and 3-5, respectively. Figures 3-6 and 3-7 show model-predicted source and pump station flows for the average day and maximum day demand condition simulations, respectively.

The average demand condition and maximum demand condition simulations were used to evaluate system hydraulics and water quality conditions, and as the baseline for modeling and evaluating proposed system improvements.

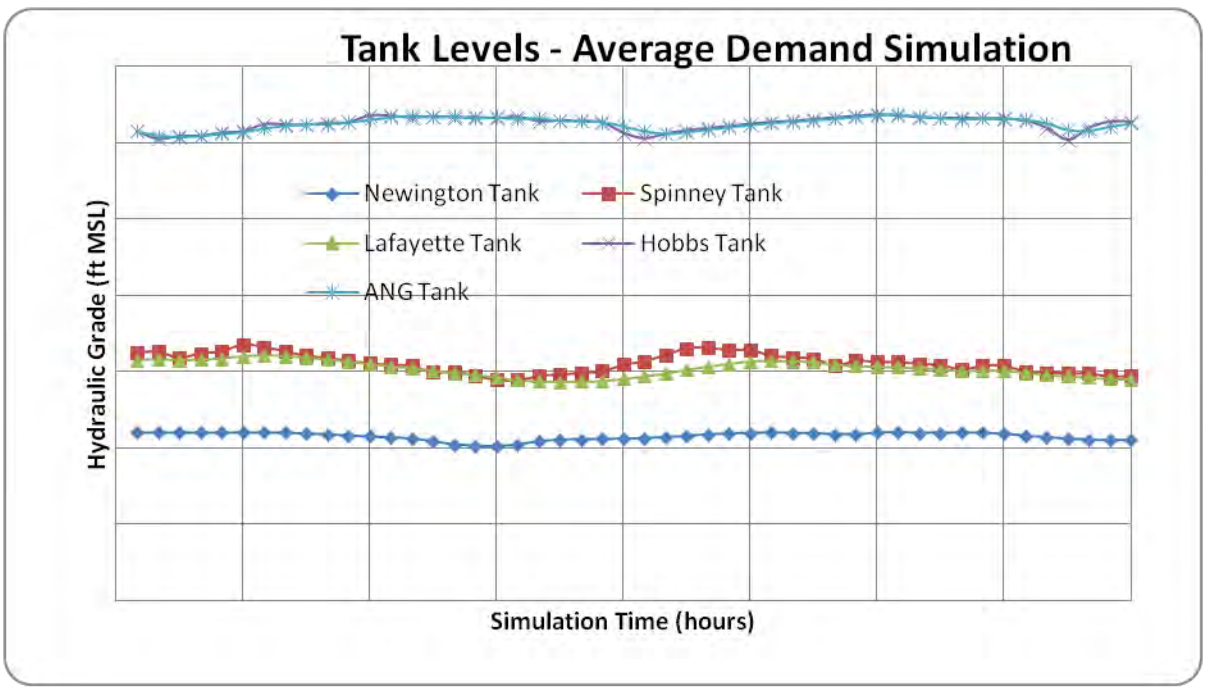

FIGURE 3-4
Model-Predicted Tank Levels – Average Day Demand Condition Simulation

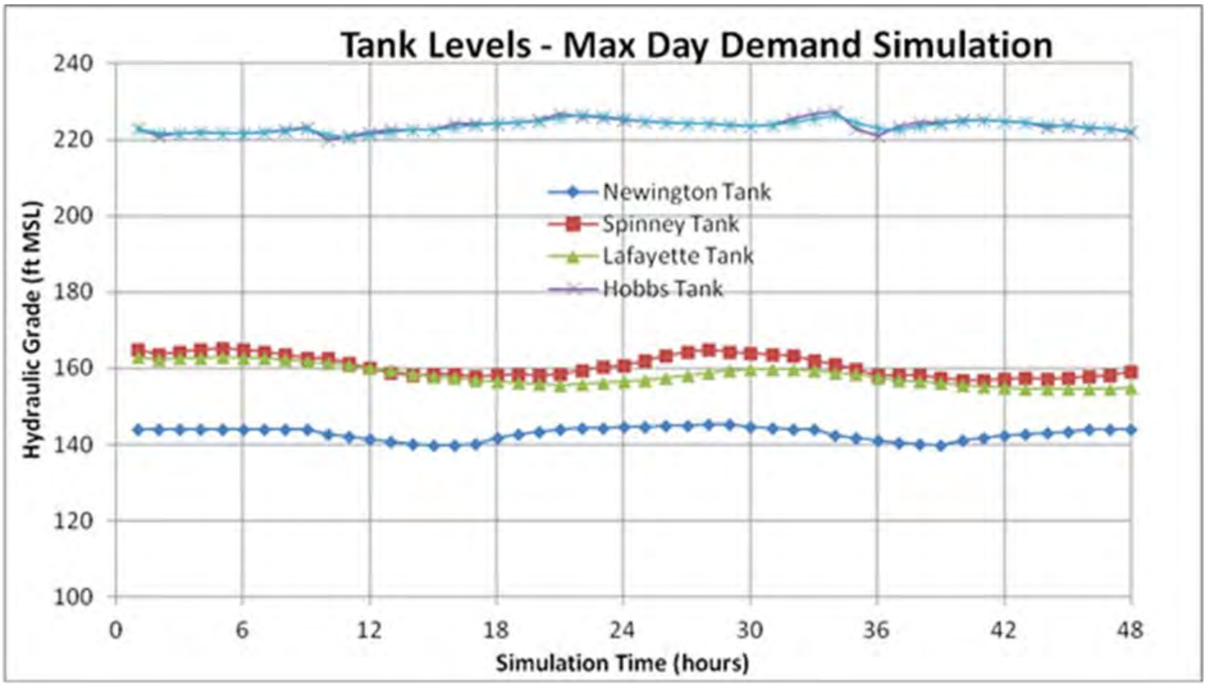

FIGURE 3-5
Model-Predicted Tank Levels – Maximum Day Demand Condition Simulation

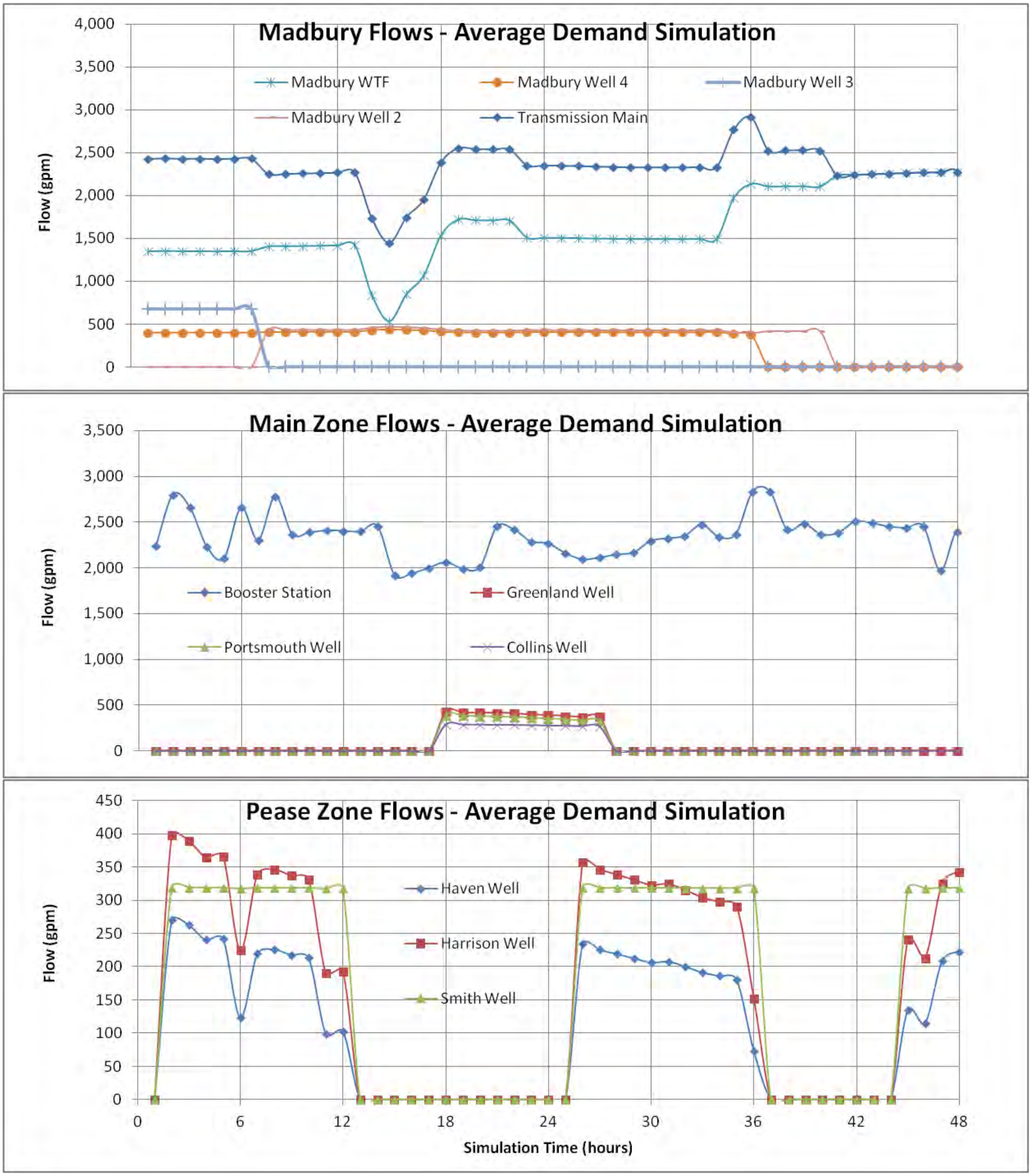

FIGURE 3-6
Model-Predicted Flows – Average Day Demand Condition Simulation

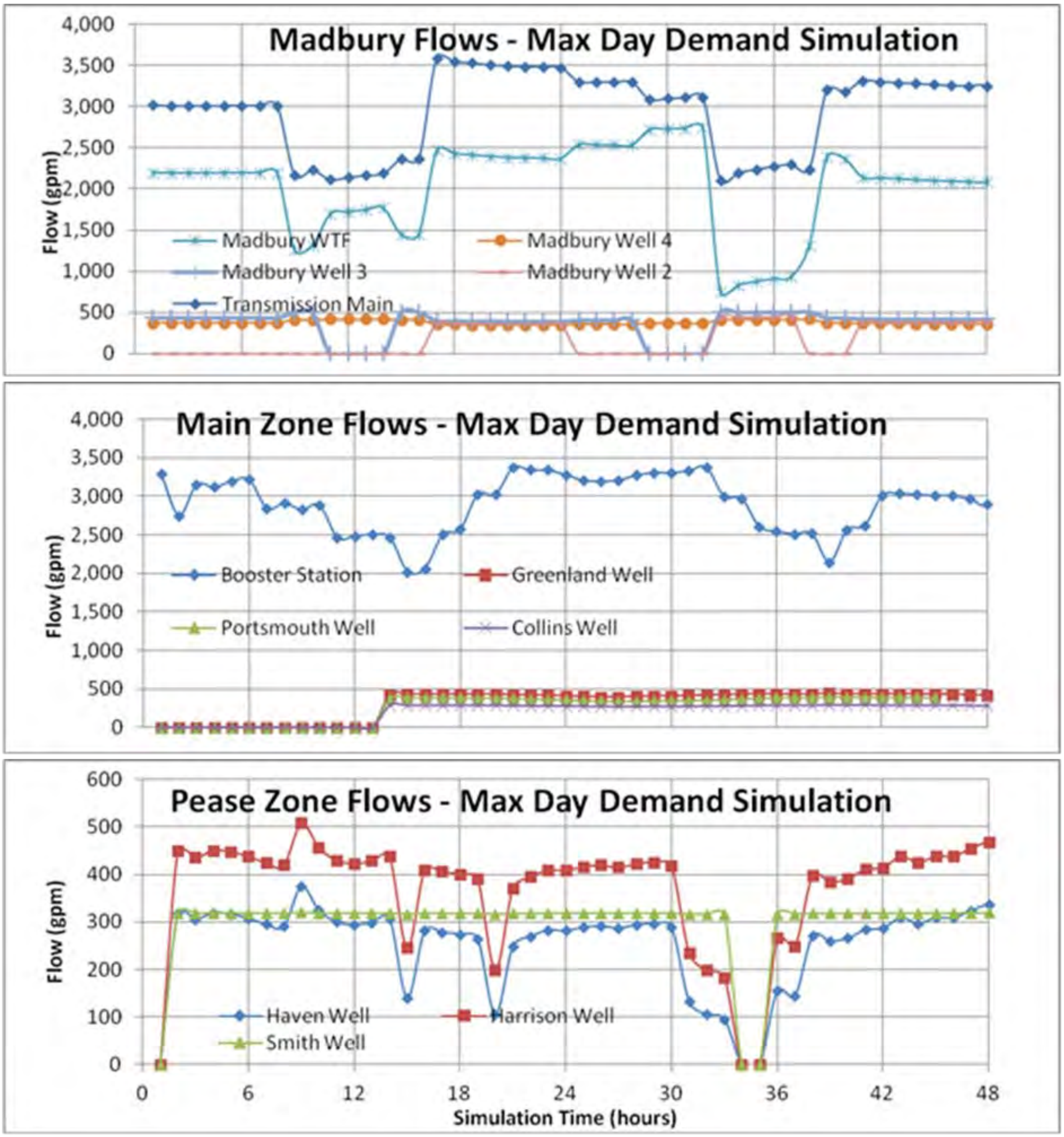

FIGURE 3-7
Model-Predicted Flows – Maximum Day Demand Condition Simulation

3.3.2 Fire Flow Analysis

The Phase I Master Plan found several areas in the Portsmouth distribution system where available fire flows were inadequate. The Plan recommended distribution piping improvements to increase the available fire flows in vulnerable areas.

A capability of the InfoWater software is the fire flow analysis feature. Using this feature, the model can calculate the available fire flow at every node in the system. The available fire flow at a given node is the maximum flow that can be withdrawn while maintaining a minimum residual pressure of 20 psi at every node in the pressure zone of interest. ISO recommends available fire flow for residential areas between 500 and 1000 gpm, depending on distance between houses.

A system wide fire flow analysis was performed using the maximum day demands as a baseline. This analysis was helpful both in determining the degree of deficiency in areas in the system, and in comparing the impacts of the mitigating scenarios that were tested.

Under current maximum day scenarios, several areas of the system appear to be fire flow deficient. Figure 3-8 presents the available fire flow at every hydrant in the system that is represented in the model. As indicated in the figure, hydrants that have a model predicted available fire flow (AFF) less than 500 gpm are depicted by red dots, and areas with model predicted AFF 500 to 1,000 gpm are depicted by orange dots. Areas that have a significant number of hydrants with model predicted AFF <1,000 gpm include:

- Northern portions of the Main Pressure Zone, including the majority of Newington.
- Atlantic Heights
- North Mill Pond area
- Sherburne Road area
- Sections of New Castle
- The majority of Greenland

Figure 3-9 shows available fire flow for the downtown area. As indicated in the figure, there are a few isolated areas with low AFF, but the downtown area has generally good AFF, due in part to recent water main improvements.

3.3.3 Pressure Analysis Results

Figure 3-10 presents the results of the system wide pressure analysis. The nodes are color-coded to represent the lowest pressure that occurred during the 48-hour maximum day demand condition simulation. As indicated in the figure, the areas that experienced pressures <35 psi in the maximum day model simulation include sections of Newington and Greenland, Seacrest Village, and a few other isolated spots.

Figure 3-11 provides a pressure analysis for the downtown area. As indicated in the figure, the downtown area has generally good pressure, due in part to recent water main improvements.

3.3.4 Pipe Velocity Analysis Results

Figure 3-12 presents the results of the system wide pipe velocity analysis. The pipes are color-coded to represent the highest flow velocity that occurred during the 48-hour maximum day demand condition simulation. As indicated in the figure, no pipes with excessive flow velocity were identified.

3.3.5 Pipe Headloss Analysis Results

Figure 3-13 presents the results of the system wide pipe headloss analysis. The pipes are color-coded to represent the highest headloss rate (expressed in headloss in ft per 1,000 ft of pipe) that occurred during the 48-hour maximum day demand condition simulation. As indicated in the figure, the following pipes with headloss/1,000 ft >5 ft were identified:

- 6-inch main in Woodbury Ave. between Granite St. and Echo Ave.
- 4-inch and 8-inch mains supplying Pease Golf course under assumed irrigation flows

The model indicated that several short small diameter dead end pipes and pipes at pump discharges also have high headloss rates, but these pipes do not have a significant effect on system hydraulics.

3.4 Water Quality Analysis

3.4.1 Water Age Analysis Results

Model-predicted water age at 4:00 p.m. from the average demand condition simulation is presented in Figure 3-14. Figure 3-14 shows the average model-predicted water age that occurs at each location during the simulation. As indicated in the figure, the model predicts that the average water age greater than 100 hours occurs in New Castle, the southern part of the main pressure zone, and at the periphery of the system in Greenland and Newington. High water age is also predicted in some of mains in along the airport runway in the Pease zone. These are fire protection mains with no service connections where very high water age is expected due to the lack of any demands. High water age has the potential to cause water quality problems, including loss of disinfectant residual and excessive concentrations of disinfection byproducts including total trihalomethanes (TTHM) and haloacetic acids (HAA5).

3.4.2 Effect of Surface Water Versus Groundwater on Disinfection Byproducts

Groundwater generally has lower concentrations of the natural organic precursors that react with chlorine to form disinfection byproducts. Therefore, there is generally less potential for TTHM and HAA5 formation in groundwater compared to surface water, and the potential for disinfection byproduct formation at any location is expected to correlate to the percent surface water contribution. Scenarios were prepared to evaluate the distribution of surface water and water age throughout the system, and how this distribution would be affected by reducing Madbury WTF production. The standard average demand scenario discussed in Section 3.1 was used as a baseline case. A second scenario was prepared with the same demands as the baseline case, but with the Madbury WTF production reduced from an average of approximately 1,600 gpm to 1,000 gpm. A comparison of the results of these scenarios is presented in Table 3-7. Figures 3-15 and 3-16 provide a graphic representation of surface water distribution in the baseline and reduced surface water contribution scenarios, respectively.

TABLE 3-7
Effect of Madbury WTF Production on Surface Water Distribution

Item	Average Day Baseline Scenario		Reduced Surface Water Contribution Scenario	
Groundwater Supply (mgd)	2.28		3.06	
Surface Water Supply (mgd)	2.30		1.44	
System-wide supply (mgd)	4.57		4.50	
	Percent Surface Water Contribution	**Water Age (hours)**	**Percent Surface Water Contribution**	**Water Age (hours)**
System-Wide Average	50%	220	32%	217
Downtown Area	65%	126	40%	139
Southern Main Pressure Zone	26%	548	5%	479
New Castle	61%	179	40%	149
Newington	55%	283	45%	302
Greenland	7%	158	0%	151

As indicated in the table and figures, the model predicts that reducing the production of the Madbury WTF would reduce the percent surface water contribution throughout the system, but most significantly would reduce both the surface water contribution and water age in the southern part of the main pressure zone (Lafayette tank area), which is the highest water-age area with the greatest potential for excessive TTHM and HAA5 formation. Thus, reducing the Madbury WTF production and relying more on the wells during the summer is expected to reduce average TTHM and HAA5 concentrations during the period when high concentrations of these substances are most likely.

3.4.3 Disinfection Byproduct Monitoring Data

Compliance with the Stage 2 Disinfectant/Disinfection Byproduct Rule is currently based on the locational running annual averages (LRAA) of TTHM and HAA5 concentrations. LRAAs are the annual averages at each sampling location in the distribution system. This accounts for spatial variations in DBP exposure because the annual average at each sampling location cannot exceed the maximum contaminant levels (MCLs) for TTHM and HAA5. The MCLs for TTHM and HAA5 are 80 µg/L and 60 µg/L, respectively. Operational Evaluation Levels (OELs) must also be calculated. The OEL is determined as the sum of the two previous quarter's TTHM or HAA5 result plus twice the current quarter's TTHM or HAA5 result at that location, divided by 4. If an OEL exceeds the MCL for TTHM or HAA5 then the system must conduct an operational evaluation that includes an examination of the treatment and distribution systems' operational practices that may contribute to TTHM and HAA5 formation and steps to minimize future exceedances. A written report of the evaluation must be submitted to the state no later than 90 days after being notified of the analytical results that caused the exceedance(s) and a copy of the report must be made publically available upon request. Starting in the 3rd quarter of 2013 the City will have to assess each individual sample site for compliance rather than averaging the four sites in the system that they currently monitor. This will make compliance with the regulation more difficult.

Tighe & Bond reviewed TTHM and HAA5 monitoring data collected from 2006 through 2012 at seven sampling locations throughout the distribution system. The data showed that the LRAAs at each sampling location were below the MCLs for TTHM and HAA5. However, concentrations of TTHM and HAA5 in a limited number of individual samples were above the MCLs and the OEL calculations were close to the compliance limits for both parameters.

The historic results of the quarterly disinfection byproduct monitoring data are consistent with the water age modeling results, which show high water age in New Castle and the southern part of the main pressure zone. The surface water contribution modeling results showed that New Castle also has a relatively high contribution of surface water, whereas the southern part of the main pressure zone has a relatively low surface water contribution under typical current operating practices. Since the potential for disinfection byproduct formation in surface water is typically higher in surface water compared to groundwater, New Castle would be expected to have relatively high disinfection byproduct concentrations.

insert Figure 3-8

Available fire flow at every hydrant in the system

insert Figure 3-9

Available fire flow for the downtown area

insert Figure 3-10

System wide pressure analysis

insert Figure 3-11

Pressure analysis for the downtown area

insert Figure 3-12

System wide pipe velocity analysis

insert Figure 3-13

System wide pipe headloss analysis

insert Figure 3-14

Average model-predicted water age

Insert Figure 3-15

Surface water distribution in the baseline surface water contribution scenario

Insert Figure 3-16

Surface water distribution in the reduced surface water contribution scenario

3.5 Mitigation Scenario Results

The model was used to evaluate the impacts of various proposed improvements to the distribution system. Baseline conditions reflect the existing distribution system piping configuration and represent available fire flow under maximum day demand conditions. A list of potential improvements was developed through meetings and conversations with the City of Portsmouth. Scenarios were developed to evaluate improvements in the following areas:

- Greenland, Newington, and Pease
    - Greenland
    - Transmission main and Newington Booster Station Tank
    - Newington
    - Pease – Hobbs Hill Tank
- Portsmouth
    - Sherburne Road area
    - Osprey Landing Tank
    - Downtown Portsmouth
    - Atlantic Heights
    - Vicinity of Sherburne Road
    - North Mill Pond area
    - Lafayette Tank
- New Castle
    - Various water main improvements
    - New storage tank

3.5.1 Greenland, Newington, and Pease

In this section, mitigation scenarios for Greenland, Newington, and the Pease Pressure Zone are discussed. Figure 3-17 shows the locations of mitigation scenarios discussed in this section.

3.5.1.1 Greenland Background

The Town of Greenland and the City of Portsmouth constructed the Greenland municipal water system under an agreement signed by the Greenland Board of Selectmen in 1944. Under this agreement the Town agreed to pay for all pipe necessary for such construction and the City of Portsmouth agreed to pay for and furnish all labor, and all joints, hydrants and other material necessary for such construction. After construction, the entire pipe line, including the pipe paid for by the Town, became the property of the City of Portsmouth. The City agreed to maintain the pipe and supply water service to the residents of Greenland who wished to have water service. This agreement stated that Portsmouth would not charge Greenland for the water used through the hydrants, but all residents of the Town would pay for water used at the regular rate of consumption. There is one water source, the Greenland Well, in this portion of the system. There are no water storage tanks connected to the water system in Greenland; the Town's water system is part of Portsmouth's Main Pressure Zone. The hydraulic grade is controlled by the water levels at the Spinney and Lafayette Road tanks.

The Greenland water system has a history of pressure and flow deficiencies. This is due location on the outer perimeter of the water system and the relatively high ground elevations. Some portions of the Greenland system (Post Road) have ground elevations that approach 100 feet, while most areas in Portsmouth are 30 to 75 feet in elevation.

insert Figure 3-17

Greenland, Newington, and Pease

The deficiencies with respect to system pressure and fire flows were analyzed in the previous hydraulic modeling and master plan study. The alternatives recommended at that time were not related to water line upgrades but to other ways to achieve pressure and flow improvements, such as installing a new tank or booster pumps. Three options were explored at that time:

- Create a separate pressure zone with either an elevated storage tank or a booster pump
- Provide a loop to Lafayette Road
- Create separate pressure zone by connecting the Greenland area to the Pease Pressure Zone

That study recommended further exploration of the Pease option, but did not provide cost estimates for capital planning purposes. It was noted that the City would address these issues in a further study and that, for the interim, individual customers would have to provide booster pumps for their own water service. The study also recommended providing a pipe connection between Moulton Road, cross country to Holly Lane, in order to improve flows and reliability.

The City recently met with the Greenland Town Administrator and Chair of the Board of Selectmen. They briefly discussed some of the flow and pressure issues in Town together with any needs that the Town felt were necessary to address. Aside from the flows and pressures, they had the following input:

- What is the City's intent with regard to replacing the water lines on Tuttle Lane?
- The Hillside Drive, McShane Avenue, Meadow Lane subdivision area experiences water main breaks and they would recommend replacing the water lines in this neighborhood.
- Increasing the pressure on Post Road is desired.
- The Town owns property on Breakfast Hill Road adjacent to I-95 that is envisioned as a potential location for a tank.

The scenarios discussed in this section were based on this information along with other items discussed during our meetings with City staff. A current day baseline conditions scenario was prepared representing the current modeled conditions of the system and the improvements scenarios were compared with this baseline. For each scenario modeled, available fire flow at the Greenland Central School, Portsmouth Avenue near Newington Road, and average available flow throughout the Greenland system are presented.

Greenland Scenario No. 1: Connect Greenland to Pease System

Under this scenario, a connection from Greenland to the Pease system would be provided near the Smith Well, at the location shown in Figure 3-17. An isolation valve would be provided near the connection point, which would put the entire Greenland area on the Pease pressure zone hydraulic gradient. A PRV would be installed on Ocean Road to allow flow from the new Greenland high pressure zone into the Portsmouth main pressure zone. Supply would be provided by the Haven, Smith, Harrison, and Greenland wells. The Newington Booster station and Pease WTF booster pumps are also available to provide supply. The Greenland well would have to be upgraded to operate against the increased hydraulic head.

The model predicted a significant increase in available fire flow throughout the Greenland area for this scenario. The recommended budget for the proposed improvements including 40% engineering and contingencies is $600,000. A conceptual level cost estimate is included in Appendix C.

Greenland Scenario No. 2: Stand-Alone Greenland Pressure Zone

Under this scenario, a new storage tank would be provided on the Town's property off Breakfast Hill Road. A new booster pump/PRV station would be provided on Ocean Road, and an isolation valve would be provided on the 12-inch Greenland line near where it connects to the 12-inch line in Sherburne Road near the intersection of Sherburne and Country Club Roads. This would create a new pressure zone comprising entire Greenland area. Supply would be provided by primarily by the Greenland Well. The new Greenland pressure zone could also be supplied by the proposed booster station on Ocean Road. The Greenland well would have to be upgraded to operate against the increased hydraulic head.

The model predicted that this scenario would result in significantly improved available fire flow throughout the Greenland area similar to the previously discussed scenario of connecting Greenland to the Pease pressure zone. The recommended budget for the proposed improvements including 40% engineering and contingencies is $4,290,000. A conceptual level cost estimate is included in Appendix C.

Greenland Scenario No. 3: Loop between Moulton Road and Holly Lane

This scenario consists of installing new water main between Moulton Road and Holly Lane, as shown on Figure 3-17. The proposed route is approximately 3,000 feet long, partly cross-country and partly on Moulton Avenue. The model indicated that this scenario would not provide a significant improvement in available fire flow or pressure.

3.5.1.2 Greenland Summary

Table 3-8 summarizes modeling results and estimated costs for the Greenland improvement scenarios.

TABLE 3-8
Greenland Improvement Scenarios

#	Improvement	Modeled AFF: Park Ave @ Greenland Central School (gpm)	% Inc or Dec	Modeled AFF: Post Road @ Breakfast Hill (gpm)	% Inc or Dec	Average Greenland AFF(gpm)	% Inc or Dec	Estimated Cost
G-0	Baseline existing conditions	420	N/A	132	N/A	661	N/A	N/A
G-1	Connect to Pease Near Smith Well, PRV to Main PZ	1,609	283	813	516	1,387	110	$600,000
G-2	Stand-Alone Greenland Pressure Zone	1,022	143	5,959	4414	1,009	53	$4,290,000
G-3	Install Moulton to Holly Lane Loop	453	7.9	162	23	677	2.4	$750,000

3.5.1.3 Newington High Pressure Zone

Low pressures and low available fire flow exist in the Nimble Hill/Fox Point area of Newington. Connecting the Newington area with the Pease pressure zone was explored to improve pressure and available fire flow in this area. This scenario consists of installing approximately 2,800 ft of 8-inch water main from the terminus of the existing main in Nimble Hill Road via Short Street to the existing 8-inch water main at the north west edge of the North Apron in the Pease zone. A PRV station would be installed in Nimble Hill Road near the intersection of Coleman Drive. Most of Newington would be connected to the Pease pressure zone. The PRV station would allow flow through Newington back to the main pressure zone. A bypass with check valve would be provided so that water could flow past the PRV from the main zone into Newington during low pressure events such as fires or water main breaks. The proposed improvements and Newington High Pressure Zone are shown in Figure 3-17.

“Before” and “after” modeling of this proposed improvement indicates that average available fire flow in the proposed Newington High Pressure Zone would increase from less than 300 gpm under existing conditions to more than 600 gpm. The recommended budget for the proposed improvements including 40% engineering and contingencies is $760,000. Modeling results from this scenario are presented in Table 3-9. A conceptual level cost estimate is included in Appendix C.

TABLE 3-9
Newington Improvement Scenario

#	Improvement	Modeled AFF: Nimble Hill Rd. @ School (gpm)	% Inc or Dec	Modeled AFF: Nimble Hill & Fox Point (gpm)	% Inc or Dec	Average Newington AFF(gpm)	% Inc or Dec	Estimated Cost
N-0	Baseline existing conditions	225	n/a	554	n/a	388	n/a	n/a
N-1	Connect to Pease PZ from Nimble Hill Rd, PRV to Main PZ	649	188	1,030	86	614	58	$760,000

3.5.1.4 Transmission Main and Newington Tank Inlet/Outlet

The transmission line from the Madbury Water Treatment Facility (WTF) was constructed in the late 1950's. The hydraulic capacity of the line was estimated to be 6.10 mgd. The recent upgrade of the WTF increased its typical daily flow volume from 3.5 to 4.0 mgd. Therefore, the current hydraulic capacity remains adequate through this study's planning horizon.

However, the upgrades of the WTF and its current operating scheme during backwash have led to the transmission line experiencing pressure fluctuations. During the course of this study we worked with the Portsmouth staff to investigate this issue further. Through this research it was discovered that the backwash cycle has an immediate impact on the water transmission line pressure. Normal pressures on the line prior to a backwash were about 50 PSI in Madbury and 45 PSI in Newington. When the WTF enters into a high rate backwash (2,500 GPM), the transmission main flow is reduced to avoid draining the clearwell, causing the pressure in the transmission main to drop. During backwash events, the pressure in Madbury drops and ranges between 39 and 45 PSI. Further detail of this research and recommendations is included in our August 1, 2012 memorandum to the City, which is included in Appendix H.

Another area of concern for the water operations staff is water age in the Newington Tank. This 1.5 MG ground storage tank's water level increases when the flow from the WTF is greater than the flow from the Newington Booster pumps that send water into the main Portsmouth system. According to system records, both of the water lines that enter and leave the tank are located at the bottom of the tank. The tank does not have any baffling system or other system to promote turnover other than tank level fluctuation. System records show that approximately 30 percent of the tank's volume is exchanged per day under normal conditions. System records also show that at times, especially during the warm summer months, the chlorine residual in this tank has to be boosted considerably to provide adequate residual throughout the rest of the Portsmouth system. The hydraulic model was used to evaluate alternatives for reducing water age in the tank.

In order to address the pressure fluctuation and water age issues, reconfiguring the inlet/outlet piping of the tank was evaluated. The existing common inlet/outlet would be changed by installing a new inlet line that would direct all incoming water from the transmission main directly into the top of the tank. The existing line from the bottom of the tank would remain in service and would serve as the outlet. Thus, all water entering the Newington Booster Station would have to go through the tank before being pumped out into the distribution system. This change would accomplish two goals:

1. It would allow for a consistent hydraulic gradient for the WTF transmission line because the grade line would be fixed at the elevation of the discharge to atmosphere at the top of the tank and would not depend on the tank water level.

2. It would significantly increase turnover and reduce water age. For example, assuming an average transmission main flow rate of 2,500 gpm and an average tank level of 40 ft, approximately 250% of the tank's volume would be exchanged per day, compared to 30% volume exchange that is currently typical. This improvement would also create more mixing of the water in the tank and allow for some cascading water out of the pipe at the top of the tank to provide aeration.

It would also be possible to provide an engineered spray aeration system, which is expected to significantly reduce the total trihalomethane (TTHM) concentration in the water transiting the aerator. TTHM formation starts at the treatment plant at the point of chlorination and continues over a period of days. Thus, the amount of reduction in TTHM concentration in the distribution system in general resulting from an aeration system at the Newington tank would depend on the extent of TTHM formation that occurs upstream of the aerator. If most TTHM formation occurs downstream of the tank, then this improvement would not be effective; however, if significant TTHM formation occurs upstream of the tank, this improvement could significantly improve water quality in the distribution system. TTHM sampling from the transmission main directly upstream of the tank is recommended to evaluate the potential of an aeration system to improve water quality.

The recommended budget for the proposed modifications to the tank inlet/outlet is $220,000. The recommended budget for the proposed TTHM stripping system including tank re-painting is $1,310,000. Both budgets include a 40% allowance for engineering and contingencies. Conceptual level cost estimates are included in Appendix C.

TABLE 3-10
Newington Tank Improvements

#	Improvement	Estimated Cost
N-2	Modifications to Newington Tank Inlet/Outlet	$220,000
N-3	Newington Tank Re-Painting & Aeration System	$1,310,000

3.5.1.5 Newington Booster Pumps

The recommended modifications to the Newington Booster Tank will include reconfiguring the tank to have a riser pipe installed on the transmission main that will fill the tank from the top and allow for better mixing of the water in the tank. With this modification in place raising and lowering the tank level to provide for turnover in the tank's water quality will not be critical. Therefore, we analyzed the typical flow scenarios anticipated for the Madbury WTF and the Newington Booster to match flows at both facilities. We utilized the maximum capacity analysis for our maximum flow for both facilities. We then used the average and maximum day pumping rates from actual August 2012 pumping data. Finally, we assumed a minimum day with the WTF running at 2.0 mgd (the current operating low flow setpoint per WTF operational staff) and one Madbury well running at 250 gpm. The following tables show the predicted pumping rates to match flows at both facilities:

TABLE 3-11
Current Madbury Pumping Rates

	Madbury WTF Pumping Rate (gpm)	Madbury Wells Pumping Rate (gpm)	Total Pumping Rate (gpm)	Total Pumping Rate (mgd)
Max Day based on Capacity	2,778	840	3,618	5.2
Ave Day - August 2012	2,250	320	2,570	3.7
Max Day - August 2012	2,480	485	2,965	4.3
Min Day - Plant @ 2.0 mgd	1,400	250	1,650	2.4

TABLE 3-12
Newington Booster – Recommended Pumping Rates

Pump #1 Rate (gpm)	Pump #2 Rate (gpm)	Pump #3 Rate (gpm)	Pump #4 Rate (gpm)	Total Booster Pump Rate (gpm)	Madbury vs. Booster Pump Rate (gpm)	Booster Pump Rate (mgd)	Tank Rise or Fall per Hour (Ft)
1,500	0	2,100	0	3,600	18	5.2	0.03
0	0	0	2,500	2,500	70	3.6	0.11
1,500	1,500	0	0	3,000	-35	4.3	-0.06
1,500	0	0	0	1,500	150	2.2	0.24

Based on this analysis we recommend the following pumping rates for the installation of five new pumps at the Newington Booster with VFDs:

- Pump #1 (Replace existing engine-driven pump serving the Pease pressure zone) 1,500 to 2,000 GPM
- Pumps #2 and #4: 1,000 to 1,500 GPM
- Pumps #5 and #6: 2,000 to 2,500 GPM

A conceptual level cost estimate for replacement of the pumps and new VFDs is presented in Table 3-13.

TABLE 3-13
Newington Pump Station Improvements

#	Improvement	Estimated Cost
N-4	Pump Station Modifications including new VFDs	$420,000

3.5.1.6 Eliminate the Newington Tank

The alternative of removing the Newington Booster tank and pumps entirely from the system was explored. Under this alternative, the tank and pumps would be abandoned and the well pumps and high lift pumps at the WTF would be modified to pump directly into the main pressure zone via the transmission main. Modeling indicated that this scenario would likely increase the pressure in the twin 20-inch transmission mains under Little Bay by approximately 25 psi to approximately 100 psi. Due to the increased risk of failure of these essential lines that would result from the increased pressure, this alternative is not recommended and is not considered further.

3.5.1.7 Replace the Newington Tank with an Elevated Tank

This alternative consists of replacing the ground storage tank with an elevated tank at the same site. This would eliminate the need for booster pumps at the station and require operation of the transmission line from the WTF and Madbury wells at a higher hydraulic grade. Modeling indicated that the hydraulic grade on the discharge side of the Newington booster station can be as high as approximately 190 ft during high demand conditions, so the proposed elevated tank overflow would need to be approximately 20 ft higher than the overflows on the other tanks in the main pressure zone. It would increase the pressure in the twin mains under Little Bay by ~25 psi to ~100 psi, increasing the risk of failure. This alternative is not recommended and is not considered further.

3.5.1.8 Replace or Rehabilitate the Hobbs Hill tank

The Hobbs Hill tank located in the Pease Pressure Zone is due for either replacement or rehabilitation. The inspection performed as part of this study confirmed that the tank is in need of repairs and recoating. As discussed in Section 1, increasing the total storage capacity in the Pease Pressure Zone is recommended. The existing NHANG and Hobbs Hill tanks have a combined capacity of 0.73 MG; 1 MG storage is recommended. Available fire flow and pressure were determined to be generally adequate. Therefore, no changes to the hydraulic grade of the tank is recommended.

In light of the need for rehabilitation of the existing tank and the storage deficit in the Pease pressure zone, replacement of the existing tank with a new, larger, elevated storage tank is recommended. A 650,000-gallon tank would meet the storage requirements for the Pease Pressure Zone through 2030. This tank size would also accommodate portions of Greenland and/or Newington should they be connected to the Pease zone in the future. Table 3-14 compares rehabilitation costs versus replacement cost for the Hobbs Hill tank. Two tank types are presented for the replacement option: 1) an elevated steel spheroid water tank, and 2) an elevated glass-fused-to-steel composite water tank. The glass-fused-to-steel tank option is less costly and will also result in reduced maintenance costs since these tanks do not require repainting.

TABLE 3-14
Pease Improvements – Hobbs Hill Tank

#	Improvement	Estimated Cost
PE-1	Rehabilitate Existing Hobbs Hill Tank	$900,000[1]
PE-2a	0.65-MG Elevated Spheroid Steel Water Tank	$2,760,000
PE-2b	0.65-MG Elevated Glass-Fused-to-Steel Composite Water Tank	$2,470,000

Note:
1. Cost based on estimate presented in Section 2.4.7.

3.5.1.9 New Generators or Portable Generators with Quick Connect Hookups for Smith and Harrison Wells

The Smith and Harrison Wells are not equipped with standby power. Installing emergency generators at these sites is recommended. If the expense of installing permanent generators at these sites is not feasible then we recommend that the City consider purchasing a portable generator set capable of running one of these wells during an extended power outage. If both of these sites are upgraded to have electrical quick-connections installed then utilizing a standby power system arrangement like this would provide additional flexibility and redundancy to the system. Table 3-15 presents conceptual cost estimates for providing either permanent generators or hookups for portable units.

TABLE 3-15
Pease Improvements – Standby Power

#	Improvement	Estimated Cost
PE-3	Install Portable Generator Receptacles for Smith and Harrison Wells and a Portable Generator	$100,000
PE-4	Install Permanent Standby Generators for Smith and Harrison Wells	$340,000

3.5.1.10 Sherburne Road Area

The existing Sherburne Road PRV is located on the 12-inch main in Sherburne Road at the south edge of the Pease pressure zone, as shown in Figure 3-17. If open, this valve allows flow from the Pease pressure zone into the main pressure zone. The valve is currently closed. The first scenario in this section models the effect of opening the Sherburne Road PRV. The PRV was set in the model to maintain a hydraulic grade of 165 ft on the downstream side. At this setting, the flow through the PRV varied between 0 and 500 gpm during the maximum day demand simulation. Table 3-16 presents the results of this evaluation. As indicated in the table, operating the valve is expected to result in a significant increase in available fire flow in the Sherburne Road – Borthwick Avenue area.

This area of the system also has older 6-inch pipes and a few dead-ends. We modeled the potential improvement that replacing the existing main in Greenside Avenue and creating a loop by installing a new main between Colonial Drive and Holly Lane. As indicated in Table 3-16, this improvement would not provide a significant available fire flow benefit outside the immediate vicinity of the new mains.

TABLE 3-16
Sherburne Road Area Improvement Scenarios

#	Improvement	Modeled AFF: Colonial Dr. & Victory Way (gpm)	% Inc or Dec	Modeled AFF: Portsmouth Regional Hospital (gpm)	% Inc or Dec	Average Sherburne Area AFF(gpm)	% Inc or Dec	Estimated Cost
S-0	Baseline existing conditions	1,169	N/A	2,882	N/A	1,886	N/A	N/A
S-1	Sherburne PRV Open	1,552	33	3,060	6	2,444	30	$0
S-2	Colonial-Holly-Greenside Improvements	1,242	6	2,882	0	1,922	2	$710,000
S-3	Colonial-Holly-Greenside Improvements with Sherburne PRV Open	1,762	51	3,067	6	2,560	35	$710,000

Utilizing the Sherburne Road PRV from the Pease system provides a significant benefit in increased available fire flow without capital expense. We recommend implementing a Standard Operating Procedure for setting this valve so that it opens when needed.

3.5.2 Portsmouth

3.5.3 Portsmouth Background

The City has been addressing the recommendations of the previous Master Plan studies through the inclusion of water system improvements in their ongoing Capital Improvements Plan (CIP). These projects have included:

- The replacement of the Spinney Road elevated storage tank with a composite 1.0-MG tank.
- The removal of the Islington Road standpipe. With the construction of the new Spinney Road tank it was determined that the hydraulic gradient in the area of the Islington tank was increased such that the Islington tank was not turning over adequately. This standpipe was over 100 years old and in need of rehabilitation, therefore, it was determined that removing it was the preferred alternative.
- The replacement of the water transmission main from the Lafayette Road standpipe to the downtown area of Portsmouth with a new 20-inch main. This new line has improved flows in this area. However, it was also intended that this new line would improve the filling and draining of the Lafayette Road tank but this goal may not have been as successful as desired.

- Other water line replacements in the downtown area, as part of the City's overall CIP to replace water lines and separate the sanitary sewer system from the stormwater system.

- Replacement of undersized and old water lines in the Atlantic Heights subdivision.

In the following sections, additional water main improvements and potential tank modifications in the Portsmouth main pressure zone are considered.

3.5.3.1 Downtown Water Main Improvements Scenarios

In this section, several water main upgrades in the main Portsmouth pressure zone are considered. The locations of the proposed improvements are shown on Figure 3-18. The hydraulic effects of the scenarios are compared based on the average model-predicted available fire flow in the downtown area. This area is bounded on the west by the North Mill Pond and U.S. Rte. 1 Bypass, on the south by Ledgewood Drive, the Jones Avenue Recreation Area, and Jones Avenue, and extends to the water to the north and east. The model-predicted existing average fire flow in the downtown area is 3,462 gpm.

Portsmouth Scenario 1: Maplewood Avenue and Woodbury Avenue (PO-1)

The City's CIP currently includes $3.0 million in the FY15 budget for a water main replacement project that consists of replacing approximately 7,500 feet of 6" and 8" 90 year old waterline on Maplewood Avenue from Woodbury Avenue to Raynes Avenue with new 16" cement-lined ductile iron waterline. We evaluated this improvement in conjunction with replacement of an additional water main in Woodbury Avenue from the intersection of Maplewood Avenue connecting to the existing 16-inch line in Arthur F. Brady Drive. This would provide a continuous 16-inch connection from the 20-inch main at the Spaulding Turnpike at Arthur F. Brady Drive to the downtown area via Maplewood Avenue. Part of this project was also included in the previous hydraulic modeling and master plan effort. The intent of evaluating this project in the current hydraulic modeling is to determine the current impact that this project would have on flow and water quality. The model results indicated that this project would not provide a significant increase in available fire flow in the downtown area, but would result in a significant increase in available fire flow in the vicinity of the new water main along Maplewood and Woodbury Avenue. For example, the model-predicted available fire flow at the hydrant at Cutts Street is >8,000 gpm with the proposed 16-inch main; however, the existing line provides over 4,000 gpm at this location. The City might consider replacing these mains with 12-inch rather than 16-inch pipe. This will still improve flows in the area and will have less impact to overall project costs. The conceptual cost estimate for the proposed Woodbury Avenue/Maplewood Avenue improvements is $3,300,000, including a 40% allowance for engineering and contingency.

insert Figure 3-18

Portsmouth Water System Alternatives

Portsmouth Scenario 2: Islington Street and Chapel Street (PO-2)

This Islington Street part of this scenario consists of installing approximately 3,600 ft of new 16-inch pipe along Islington Street, Congress Avenue, and Daniel Street between Woodbury Avenue and Chapel Street. The effect on available fire flow in the downtown area resulting from this improvement by itself is expected to be limited, with a predicted increase of about 3% in the average downtown area available fire flow. Installing an additional 300 ft of 12-inch water main on Chapel Street from Daniel Street to State Street provides a connection to recent improvements in State Street. If the Chapel Street improvements are included, this scenario results in a 6% average increase in the model-predicted available fire flow in the downtown area. The conceptual cost estimate for the proposed improvements is $2,140,000, including engineering and contingency.

Portsmouth Scenario 3: Miller Avenue (PO-3)

The Miller Avenue water main replacement consists of installing approximately 2,300 ft of 16-inch water main on Miller Avenue between Middle Street and South Street. The model-predicted available fire flow in the downtown area increases less than 3% under this scenario. The conceptual cost estimate is $1,210,000, including a 40% allowance for engineering and congtingency.

Portsmouth Scenario 4: Lafayette Road - Andrew Jarvis Road to Greenleaf Woods Drive (PO-4)

The new 12-inch water main on Lafayette Road terminates at its north end near the intersection of Greenleaf Woods Drive in Portsmouth. The section of pipe between the termination of the new line and Ledgewood Drive is currently older 6 and 8-inch pipe. The proposed improvements for this scenario include replacing approximately 1,600 of existing 6- and 8-inch pipe with new 12-inch pipe.

Model results indicated that this improvement would not provide a significant improvement except in the immediate vicinity of the new main, with an increase in the average available fire flow in the downtown area of less than 1% predicted.

The conceptual cost estimate for this scenario is $590,000, including engineering and contingency. (note: This project is included as part of the ongoing NHDOT project on Route 1, scheduled for 2013)

3.5.3.2 Summary of Downtown Portsmouth Scenarios

Portsmouth Scenarios 1 through 4 are intended to improve available fire flow in the downtown area. The model results and estimated cost of improvements for Portsmouth Scenarios 1 through 4 are summarized in Table 3-17.

TABLE 3-17
Downtown Portsmouth Improvement Scenarios

#	Description	Proposed Improvement	Model-Predicted Average Downtown AFF (gpm)	% Increase compared to Baseline	Estimated Cost
PO-0	Baseline existing conditions	N/A	3,462	N/A	N/A
PO-1a	Maplewood Avenue and Woodbury Avenue	7,800 LF 16-inch water main	3,482	0.6%	$4,110,000
PO-1b	Maplewood Avenue and Woodbury Avenue	7,100 LF 12-inch water main	3,462	0%	$3,300,000
PO-2	Islington Street and Chapel Street	3,600 LF 16-inch and 300 LF 12-inch water main	3,680	6.3%	$2,140,000
PO-3	Miller Avenue	2,300 feet 16-inch water main	3,560	2.8%	$1,210,000
PO-4	Lafayette Road - Andrew Jarvis Road to Greenleaf Woods Drive	1,600 feet 12-inch water main	3,520	1.7%	$590,000

3.5.3.3 Additional Main Pressure Zone Scenarios

In this section, two additional scenarios intended to improve available fire flows in the Atlantic Heights and North Mill Pond areas are presented. These scenarios are evaluated based on their effect on available fire flow and pressure in the vicinity of the respective improvements.

Portsmouth Scenario 5: Atlantic Heights Loop (PO-5)

This scenario consists of installing a new 12-inch pipe to connect Atlantic Heights at Crescent Street to Dunlin Road. While not providing a significant benefit outside the Atlantic Heights area, the modeling results indicate that proposed water main would provide a significant benefit in the area, which currently has available fire flow of less than 1,000 gpm. The conceptual cost estimate for the proposed improvements considered in this scenario is $340,000, including an allowance of 40% for engineering and contingency.

TABLE 3-18
Atlantic Heights Improvement Scenario

#	Description	Average Atlantic Heights AFF(gpm)	% Increase compared to Baseline	Estimated Cost
PO-0	Baseline existing conditions	849	N/A	N/A
PO-5	Atlantic Heights Loop	1,582	86%	$340,000

Portsmouth Scenario 6: North Mill Pond Area (PO-6)

The existing water mains in the North Mill Pond area (refer to Figure 3-17) are predominantly older 2, 4 and 6-inch pipes. This scenario consists of replacing the small diameter lines in the area with new 8-inch water lines. The model predicts that the proposed improvements would result in significantly improved available fire flow throughout the North Mill Pond area of the system, increasing the average available fire flow from approximately 1,700 gpm to approximately 3,000 gpm.

The proposed water main includes replacement of approximately 9,800 ft of older small diameter pipe with 8-inch pipe. The conceptual cost estimate for this improvement is $3,000,000, including engineering and contingency.

TABLE 3-19
North Mill Pond Area Improvements Scenario

#	Description	Average N. Mill Pond AFF(gpm)	% Increase compared to Baseline	Estimated Cost
PO-0	Baseline existing conditions	1,751	N/A	N/A
PO-6	N. Mill Pond Water Main Improvements	3,032	73%	$3,000,000

3.5.3.4 Lafayette Road Tank

The Lafayette Road tank has a storage capacity of approximately 7.5 MG, of which only 2.3 MG is considered to be useable. The majority of the storage volume of the tank is at an elevation that is too low to maintain satisfactory pressure in the distribution system. As a result of the large volume, the Lafayette Road Tank and surrounding area in the southern portion of the City that is influenced by the tank experience high water age, as discussed in Section 3.4.5. High water age has the potential to cause water quality problems, including loss of disinfectant residual and excessive concentrations of disinfection byproducts including total trihalomethanes (TTHM) and haloacetic acids (HAA5). TTHM and HAA5 concentrations are a concern to the City in light of the new Stage 2 Disinfectant/Disinfection Byproduct Rule.

The following potential improvements and operational changes that could mitigate loss of chlorine residual and excessive disinfectant byproduct formation were evaluated:

- Installation of a mixing system in the tank
- Installation of a spray aeration system in the tank to remove TTHM
- Increasing the contribution of the groundwater sources during the summer
- Replacement of 7.5 MG ground storage tank with a 1 MG Elevated Steel Spheroid Water Storage Tank

Lafayette Tank Mixing System

The proposed mixing system would not reduce water age in the tank; however, mixing would eliminate stagnant zones and lower the potential for loss of disinfection residual that could occur in these areas.

The proposed system consists of a Solar Bee model SB-5000 mixer and appurtenances. The conceptual cost estimate for installing this system in the Lafayette tank is $150,000 including a 40% allowance for engineering and contingency.

Lafayette Tank Mixing, Spray Aeration, and Chlorination System

Recent work by Dr. Robin Collins at the University of New Hampshire demonstrated that spray aeration systems can be effective in removing TTHMs. This type of system is well suited for installation in distribution storage tanks. The system proposed for the Lafayette tank would include a ventilation system in the top of the tank, pump system, piping, spray nozzles, and electrical equipment and controls. The system would continuously circulate water from the bottom of the tank through spray nozzles located at the top of the tank. TTHM would be stripped from the bubbles as they fall from the nozzles to the water surface in the tank. TTHM would be removed in the vapor phase from the top of the tank by the ventilation system. The proposed system would also include a sodium hypochlorite feed system to ensure that an adequate chlorine residual is maintained in the tank.

The conceptual cost estimate for the proposed mixing/spray aeration/chlorination system is $360,000, including a 40% allowance for engineering and contingency. Further discussion of our recommendation for piloting this system prior to final installation is discussed in the recommendations section of this report.

Lafayette Tank Replacement

The Lafayette tank located in the Main Pressure Zone has an abundance of unusable volume. It has an available capacity of 2,266,000 gallons. In accordance with Table 1-29, the Main Pressure Zone has a surplus capacity of 1,775,000 gallons through 2030. As a result, the existing Lafayette Road storage tank could be replaced with elevated water storage tank with a minimum capacity of 0.5 MG. For the purposes of this evaluation, it is assumed that the tank would be replaced with a 1.0 MG Elevated Steel Spheroid Water Storage Tank.

Similar to the Hobbs Tank replacement evaluation, two tank types are presented for the replacement option: 1) an elevated steel spheroid water tank (PO-07c), and 2) an elevated glass-fused-to-steel composite water tank (PO-07d). The glass-fused-to-steel tank option is less costly and will also result in reduced maintenance costs since these tanks do not require repainting.

The conceptual cost estimate for demolition of the existing Lafayette tank and replacement with a new elevated water storage tank including a 30% allowance for engineering and contingency is $4,550,000 for a 1 MG Elevated Painted Steel Spheroid Water Storage Tank and $3,700,000 for a 1 MG Elevated Glass-Fused-to-Steel Water Storage Tank.

Conceptual cost estimates for Lafayette tank improvements are summarized in Table 3-20.

TABLE 3-20
Lafayette Tank Improvements

#	Improvement	Estimated Cost
PO-7a	Lafayette Tank Mixing System	$150,000
PO-7b	Lafayette Tank Mixing, Aeration, and Rechlorination System	$360,000
PO-7c	Lafayette Tank Replacement – 1 MG Elevated Painted Steel Spheroid Water Storage Tank	$4,550,000
PO-7d	Lafayette Tank Replacement – 1 MG Elevated Glass-Fused-to-Steel Water Storage Tank	$3,700,000

3.5.3.5 Removal of the Osprey Landing Tank

The Osprey Landing Tank is a 200,000-gallon elevated storage tank with a base elevation of 100 feet and an overflow elevation of 170 feet. According to system operators, this tank has been offline for approximately two years. Distribution system improvements over the years in the area of this tank have improved the fire flows and pressure in the area. The tank is due to be painted; therefore, since it has been offline for this period of time without any noticeable impacts to the system, one of the goals of this study was to determine if it could be removed from the system or if it should be painted or replaced.

Available fire flow analysis was performed with and without the existing tank for the area roughly bounded by the Spaulding Turnpike, Gosling Road, Piscataqua River, and North Mill Pond. With the tank, the average model-predicted available fire flow in this area is approximately 2,300 gpm, as compared to approximately 1,900 gpm without the tank. The model predicted available fire flow on Woodbury Avenue at BJ's was not impacted by the presence or absence of the tank. At the Franklin School, the model predicted AFF is approximately 200 gpm higher with the tank; however, the AFF is >3,500 gpm for both cases at this location.

Based on the model predictions and the fact that the tank has been off-line for an extended period without significant negative impacts, we conclude that the tank's impact on available fire flow outside the immediate vicinity of the tank is not significant and that the tank could be removed.

TABLE 3-21
Effect of Osprey Landing Tank on AFF in vicininty

#	Improvement	Modeled AFF: Woodbury Ave/BJs (gpm)	Modeled AFF: Franklin School (gpm)	Modeled AFF: Average in vicinity (gpm)
PO-0	Baseline existing conditions (no tank)	693	3,544	1,938
PO-8	With Osprey Landing Tank	694	3,705	2,287

3.5.4 New Castle

3.5.4.1 New Castle Background

The New Castle portion of the water system has a history of pressure and flow deficiencies. This is primarily due to the fact that it is at the furthest end of the water system, has older and undersized water mains and is sub-metered for the New Castle Water District service territory. Half of the island's water mains are also owned and controlled by the New Castle Water District. The City is currently in discussions with the Town regarding potentially taking over this system. If so, capital improvements of this portion of the system would then be the responsibility of the City. Ultimately, this would be in the best interest of both the City and New Castle as planning, funding and phasing these improvements could be allocated to projects that will result in the greatest benefit to the residents of New Castle.

The deficiencies with respect to fire flows were analyzed and highlighted in the previous Phase 1 Water System Master Plan hydraulic modeling and master plan study. It was noted that several piping upgrades and distribution system modifications were modeled and evaluated for their improvement to fire flow and pressure. The results showed that piping upgrades provided little or no improvement. The recommendation from this study was that a new pressure zone should be created for the island. They also noted that a new small 300,000 gallon elevated storage tank, in combination with a booster pump station would increase the fire flows to acceptable values. This recommendation noted that the tank would be approximately 175-feet high with an overflow elevation of 186 feet.

The City recently met with the New Castle Firewards to discuss the deficiencies in their system and shared those discussion notes with us. The City also provided us with a copy of the Fireward's September 11, 2007 letter to the City of Portsmouth regarding water lines and fire flows. The letter expressed concerns the Firewards had regarding low fire flows in the downtown area of New Castle and the last third of Wild Rose Lane. Their specific recommendations for improvements to the system included:

- Replace the 1880's vintage 8-inch water line on Wentworth Road, from North Road to Main Street
- Reconfigure the maze of valves and water meters at the intersection of Main Street and Wentworth Road into a simple operation with the possibility of a two-way meter.
- Install a water line down the entire length of Pit Lane connecting both ends of the island.

According the New Castle Firewards letter and the ISO testing information the fire flows at the end of Wild Rose Lane are only 160 GPM. There are a number of potential options. They include:

- Loop Wild Rose Lane to Wentworth (Scenario NC-10)
- Replace Wild Rose Lane pipe with new 8-inch (Scenario NC-4)
- Replace the 1900's Odiorne Point line under the water feeding the Great Island.

Figure 3-19 is an excerpt from the City's 2013-2018 Capital Improvement Plan (CIP). It describes a budgeted project for the New Castle portion of the water system. Currently there is an 8-inch water main that runs from Odiorne Point to the New Castle island under the harbor. The proposed project in the CIP provides $3.0 million to replace this line with a new 8-inch water main. It would also re-route the line to Wild Rose Lane. Additional water main replacement would occur on Wild Rose Lane to Wentworth Road. This is intended to improve redundancy and fire flow on the island.

We assessed the baseline existing flow conditions in New Castle utilizing the hydraulic model. The modeled available fire flow was as follows:

- Wentworth Hotel – 1,537 gpm
- Elementary School – 893 gpm
- New Castle Average – 1,018 gpm

EF-WD-06: LITTLE HARBOR/WILD ROSE LANE WATER LINES

The project replaces the water line that crosses Little Harbor from Odiorne Point as well as that section of water line along Wild Rose Lane. These lines were installed in the 1940's and have been the source of significant maintenance in recent years. The project will allow a secure redundant feed into the New Castle portion of the Water System.

		FY 13	FY 14	FY 15	FY 16	FY 17	FY 18	Totals 13-18	6 PY's Funding	Totals
GF	0%							$0	$0	$0
Fed/State	0%							$0	$0	$0
Bond/SRF	100%		$3,000,000					$3,000,000	$500,000	$3,500,000
Other	0%							$0	$0	$0
Revenues	0%							$0	$0	$0
PPP	0%							$0	$0	$0
	Totals	$0	$3,000,000	$0	$0	$0	$0	$3,000,000	$500,000	$3,500,000
Commence FY:	2013	Quarter:	1st	Priority:	I	Impact On Operating Budget:		Negligible		

CIP FY 13-18 Planning Board January 19, 2012 153

FIGURE 3-19
Excerpt from Portsmouth's 2013-2018 CIP

3.5.4.2 New Castle Modeled Scenarios

The following scenarios were based on previous report information and our overall hydraulic model findings, along with other items discussed during our meetings with City staff. They were modeled and assessed for their potential to improve flow and pressure conditions in the New Castle portion of the water system. Baseline conditions assessed the current condition of the system in the hydraulic model and the improvement scenarios were compared with this baseline. Each model run assessed flow at the Wentworth Hotel and the Elementary School, and calculated the average flow in the New Castle area.

All of the modeling scenarios for New Castle, with the exception of the baseline existing conditions, were developed to assess the potential improvement if the New Castle Water District were to turn its system over to the City of Portsmouth to own and operate. It is our understanding that the City may at one point take over the operation of the New Castle system. If so, the system would no longer need to be metered at the two points that separate the systems and the meter pits, together with a number of existing valves, could be removed. They could all be replaced with new 8-inch lines. Therefore, we modeled the current existing conditions with respect to flow and pressure and the theoretical conditions if the meters and valves were removed.

The Portsmouth CIP currently has $3.5 million in their CIP to replace the line that runs from Odiorne Point in Rye to New Castle. This line provides redundancy for the New Castle island and also additional flow and redundancy to Odiorne Point. The City inquired as to other potential upgrades that could occur in and around the New Castle area that they might utilize the funds for upgrades rather than replacing this water line. Suggestions and modeled scenarios included:

- What effect does abandoning the Odiorne Point Main have on system flows, pressures and redundancy? (Scenario NC-2)
- What effect does replacing the Odiorne Point Main with a new 12-inch line have on system flows and pressures? (Scenario NC-7)
- What effect does removing the metering into the New Castle Water District and installing a new 12-inch line on Wentworth Road have on flows and pressures with and without the Odiorne Point Main? (Scenario NC-3)
- What effect would replacing the Wild Rose Lane waterline with a new 8-inch line have on the flows in that area? (Scenario NC-4)
- Could a new connection from the Rye Water District lines that end just across the bridge from the Wentworth Hotel improve flow and redundancy in this area enough to eliminate the need for the third line? (Scenario NC-11) This would require an arrangement with the Rye Water District and the installation of a new line under the bridge into New Castle.
- Could other water lines be installed anywhere between the Portsmouth system and New Castle to provide the flow and redundancy provided by the existing Odiorne Point Main?

Figure 3-20 shows the scenarios that were simulated by the hydraulic model. The following tables provide the results of these scenarios as well. They are listed in the order of the percentage of overall improvement to modeled AFF for New Castle. The first table shows the results with the Odiorne Point water main in place or being replaced. The second table shows the results of various system improvements if the Odiorne Point water main is abandoned.

3.5.4.3 Scenarios Assuming Abandoning the Odiorne Point Main

For the scenarios presented in this section, it is assumed that the main connecting New Castle with Odiorne Point in Rye would be abandoned, so that the funds set aside for replacing this main could be used for alternative projects. The Odiorne Point Main would be abandoned upon failure, but should be utilized until such time. For these scenarios, it is also assumed that the New Castle meter pits would be abandoned as discussed under Scenario 1.

New Castle Scenario 1: Eliminate Meter Pits (NC-1)

This scenario consists of eliminating the existing meter pits including removing the existing meters and valves and replacing small-diameter piping in the vicinity of the meters to eliminate the existing hydraulic restrictions and allow flow in both directions. The conceptual cost estimate for this alternative is $100,000, including a 40% allowance for engineering and contingency. This alternative results in a slight increase in average available fire flow and available fire flow at the Elementary School and Wentworth Hotel.

New Castle Scenario 2: Abandon Odiorne Point Main (NC-2)

This scenario consists of abandoning the Odiorne Point water main, and includes Scenario 1, eliminating the meter pits. This scenario by itself results in a 7% decrease in model-predicted available fire flow at the Wentworth Hotel, an 8% decrease in average New Castle available fire flow, and a 2% decrease in available fire flow at the Elementary School. This alternative is presented because it is included as a component in several other scenarios discussed below. The conceptual cost estimate for Scenario 2 (including the cost of Scenario 1) is $150,000, including a 40% allowance for engineering and contingency.

New Castle Scenario 3: New 12-inch Water Main in Wentworth Road (NC-3)

This scenario consists of replacing the existing water main in Wentworth Road from Little Harbor Road to Main Street (approximately 4,100 ft) with new 12-inch main, in conjunction with Scenario 1. The model simulation for Scenario 3 also includes abandoning the Odiorne Point main (Scenario 2), which by itself results in a reduction as discussed previously. The model-predicted effects of this scenario are an 11% increase in the average New Castle available fire flow, a 20% increase at the Elementary School, and a 6% decrease at the Wentworth Hotel. The conceptual cost estimate for Scenario 3 (including the cost of Scenario 1) is $1,670,000, including a 40% allowance for engineering and contingency.

New Castle Scenario 4: New 8-inch Water Main in Wild Rose Lane (NC-4)

This scenario consists of replacing the existing water main in Wild Rose Lane (approximately 2,600 ft) with new 8-inch water main. The model simulation for Scenario 4 also includes abandoning the Odiorne Point main (Scenario 2), which by itself results in a reduction as discussed previously. The model-predicted effects of this scenario for the New Castle system in general, at the Wentworth Hotel, and at the Elementary School are similar to Scenario 2; this alternative does not provide any improvement to available fire flow except along Wild Rose Lane. The model-predicted available fire flow at the end of Wild Rose lane is 875 gpm under Scenario 4, compared to <500 gpm for the baseline scenario. The conceptual cost estimate for Scenario 4 is

$810,000, including the cost of Scenario 1 and a 40% allowance for engineering and contingency.

insert Figure 3-20

New Castle Water System Alternatives

New Castle Scenario 5: New 8-inch Water Main in Pit Lane (NC-5)

This scenario consists of installing a new 8-inch water main in the full length of Pit Lane (approximately 1,700 ft). The model simulation for Scenario 5 also includes abandoning the Odiorne Point main (Scenario 2), which by itself results in a reduction as discussed previously. The model-predicted effects of this scenario are a 4% decrease in the average New Castle available fire flow, an 11% increase at the Elementary School, and a 6% decrease at the Wentworth Hotel. The conceptual cost estimate for Scenario 5 is $600,000, including the cost of Scenario 1 and a 40% allowance for engineering and contingency.

New Castle Scenario 6: New Water Main in Pit Lane and Wentworth Road (NC-6)

This scenario is a combination of Scenarios 1, 2, 3, and 5, including removing the meter pits (Scenario 1), replacement of 4,100 ft of water main in Wentworth Road with new 12-inch main (Scenario 3), and installing 1,700 ft of new 8-inch main in Pit Lane (Scenario 2), assuming that the Odiorne Point water main connecting to New Castle from Rye is abandoned (Scenario 2). The model-predicted effect of Scenario 6 as compared to existing conditions is a 14% increase in average New Castle available fire flow, a 49% increase in available fire flow at the Elementary School, and a 3% decrease in available fire flow at the Wentworth Hotel. The conceptual cost estimate for Scenario 6 is $2,120,000, including a 40% allowance for engineering and contingency.

New Castle Scenario 7: New Water Main in Wentworth Road – North Gate Road to Spring Hill Road (NC-7)

This scenario consists of replacing approximately 625 ft of 12-inch water main in Wentworth Road from North Gate Road to Spring Hill Road. The model simulation for Scenario 7 also includes abandoning the Odiorne Point main (Scenario 2), which by itself results in a reduction of available fire flow as discussed previously. The model predicts a decrease of 4 % in average New Castle available fire flow, a decrease of 8% in available fire flow at the Wentworth Hotel, and no significant change in available fire flow at the Elementary School, compared to the baseline scenario. The conceptual cost estimate for Scenario 7 is $340,000, including a 40% allowance for engineering and contingency.

New Castle Scenario 8: Loop Wild Rose Lane to Wentworth Road (NC-8)

This scenario consists of installing 1,100 feet of cross-country 8-inch main connecting Wild Rose Lane to the existing cross-country 8-inch main to the east of Little Harbor Road, and replacing the Wild Rose Lane main as discussed under Scenario 4. The model simulation for Scenario 8 also includes abandoning the Odiorne Point main (Scenario 2), which by itself results in a reduction of available fire flow as discussed previously. The model predicts an increase of 13 % in average New Castle available fire flow, an decrease in available fire flow at the Wentworth Hotel of 6%, and an increase in available fire flow at the Elementary School of 20%, compared to the baseline scenario. The conceptual cost estimate for Scenario 8 is $1,120,000, including a 40% allowance for engineering and contingency.

New Castle Scenario 9: Replace Mains in the Center of New Castle (NC-9)

This scenario consists of Replace piping on a loop in the center of town, including Main Street, Cranfield Street, Piscataqua Street, Walbach Street, and Wentworth Street with 12-inch DI, comprising approximately 4,500 ft of new water main. The model simulation for Scenario 9 also includes abandoning the Odiorne Point main (Scenario 2), which by itself results in a reduction of available fire flow as discussed previously. The model predicts an decrease of 5 % in average New Castle available fire flow, an decrease in available fire flow at the Wentworth Hotel of 7%, and an increase in available fire flow at the Elementary School of 3%, compared to the baseline scenario. The conceptual cost estimate for Scenario 9 is $1,930,000, including a 40% allowance for engineering and contingency.

New Castle Scenario 10: New Main Connecting Spring Hill Road and Walton Road (NC-10)

This scenario consists of installing a new 650-ft water main under the estuary connecting Spring Hill Road and Walton Road. Replacing approximately 1,000 ft of existing main in Spring Hill Road is also included. The model simulation for Scenario 10 also includes abandoning the Odiorne Point main (Scenario 2), which by itself results in a reduction of available fire flow as discussed previously. The model predicts an decrease of 1 % in average New Castle available fire flow, an decrease in available fire flow at the Wentworth Hotel of 5%, and an increase in available fire flow at the Elementary School of 23%, compared to the baseline scenario. The conceptual cost estimate for Scenario 10 is $950,000, including a 40% allowance for engineering and contingency.

Table 3-22 summarizes each improvement assuming abandoning the Odiorne Point main including average New Castle AFF and cost estimates. Detailed conceptual level cost estimates are included in Appendix C.

TABLE 3-22
New Castle Scenarios Assuming Abandoning the Odiorne Point Main

#	Improvement	Average New Castle AFF(gpm)	% Inc/Dec	Estimated Cost
NC-0	Base - existing conditions	1,018	N/A	N/A
NC-1	Remove meter pits/check valves/replace small diameter main	1,032	1.4%	$100,000
NC-2	Abandon Odiorne Point Main	963	-5.4%	$150,000
NC-3	Replace Main on Wentworth Road from Little Harbor Road to Main Street with 12" DI	1,142	12.2%	$1,670,000
NC-4	Replace Water Main on Wild Rose Lane with 8" DI	983	-3.44	$810,000
NC-5	Install 8" Main on Pit Lane	991	-2.7%	$600,000
NC-6	Install 8" Main on Pit Lane + Wentworth Road Main replacement	1,173	15.2%	$2,120,000
NC-7	Replace Main on Wentworth Road from North Gate Road to Spring Hill Road with 12" DI	990	-2.7%	$340,000
NC-8	Loop Wild Rose Lane to Wentworth Road with 8" DI	1,163	14.3%	$1,120,000

NC-9	Replace piping in center of Town - Main Street, Cranfield Street, Piscataqua Street, Walbach Street, and Wentworth Street with 12-inch DI	983	-3.4%	$1,930,000
NC-10	Loop New Castle with a 12-inch pipe drilled under estuary connecting Spring Hill Rd to Walton Rd	1,019	0.1%	$950,000

3.5.4.4 Scenarios Assuming Replacing the Odiorne Point Main

The following scenarios assume that the main connecting New Castle with Odiorne Point in Rye would be replaced.

New Castle Scenario 11: Replace Odiorne Point Water Main, Ocean Boulevard to Wentworth Road (NC-11)

This scenario includes replacing the existing main from Ocean Boulevard in Rye to Wentworth Road in New Castle with new 12-inch pipe, including approximately 2,800 ft of ductile iron and 2,600 ft of HDPE for the underwater section. The model-predicted hydraulic effect of this scenario is an 8% increase in average New Castle available fire flow, an 8% increase in available fire flow at the Wentworth Hotel, and a 2% increase in available fire flow at the Elementary School. The conceptual cost estimate for Scenario 11 is $3,100,000 including a 40% allowance for engineering and contingency.

New Castle Scenario 12: Replace Odiorne Point Water Main, Sagamore Road to Wentworth Road (NC-12)

This scenario includes Scenario 7 improvements plus replacing an additional 6,900 ft of 12-inch main in Ocean Boulevard and Pioneer Road continuing to the intersection with Sagamore Avenue. The model-predicted hydraulic effect of this scenario is a 13% increase in average New Castle available fire flow, an 15% increase in available fire flow at the Wentworth Hotel, and a 3% increase in available fire flow at the Elementary School. The conceptual cost estimate for Scenario 12 is $6,760,000 including a 40% allowance for engineering and contingency.

Table 3-23 summarizes each of the scenarios involving replacing the Odiorne Point, including average New Castle AFF and cost estimates.

TABLE 3-23
Replacing Odiorne Point to New Castle Water Main

#	Improvement	Average New Castle AFF(gpm)	% Inc/Dec	Estimated Cost
NC-0	Baseline existing conditions	1,018	N/A	N/A
NC-11	Replace Odiorne Point main with 12-inch DI from Wentworth Rd to Ocean Blvd	1,110	9.1%	$3,100,000
NC-12	Replace Odiorne Point main per Scenario 11 plus new 12-inch DI from State Park to Sagamore Road	1,170	15.0%	$6,760,000

3.5.4.5 Scenarios Including a New Connection from Rye on Wentworth Road

The scenarios included in this section explore providing a new connection to the existing Rye system water main in Wentworth Road (Route 1B). For the scenarios included in this section, it is assumed that the existing meter pits would be abandoned as described under Scenario 1, and that the Odiorne Point main would be abandoned as discussed under Scenario 2.

New Castle Scenario 13: New Connection from Rye on Wentworth Road (NC-13)

This scenario includes approximately 550 ft of new 12-inch water main on Wentworth Road in New Castle, and 1,000 feet of new under water main connecting to the existing 8-inch water main in Wentworth Road in Rye. The model-predicted hydraulic effect of this scenario is a 4% increase in average New Castle available fire flow, an 11% increase in available fire flow at the Wentworth Hotel, and no significant change in available fire flow at the Elementary School. The conceptual cost estimate for Scenario 13 is $1,070,000 including a 40% allowance for engineering and contingency.

New Castle Scenario 14: New Connection from Rye plus Scenario 3 (NC-14)

This scenario includes the new connection to Wentworth Road in Rye as described under Scenario 13, plus replacement of the existing Wentworth Road main in New Castle from Little Harbor Road to Main Street, as described under Scenario 3, providing continuous 12-inch main on Wentworth Road from the Wentworth Hotel to the intersection with Main Street. The model-predicted hydraulic effect of this scenario is a 33% increase in average New Castle available fire flow, an 21% increase in available fire flow at the Wentworth Hotel, and 29% increase in available fire flow at the Elementary School. The conceptual cost estimate for Scenario 14 is $2,640,000 including a 40% allowance for engineering and contingency.

Table 3-24 summarizes the proposed Rye Water District connection scenarios, assuming abandoning the Odiorne Point main.

TABLE 3-24
New Connection to Rye Water District on Wentworth Road

#	Improvement	Average New Castle AFF (gpm)	% Inc/Dec	Estimated Cost
NC-0	Base - existing conditions	1,018	N/A	N/A
NC-13	Connect to Rye Water District Line across bridge on Wentworth Road	1,115	9.5%	$1,070,000
NC-14	Connect to Rye Water District Line across bridge on Wentworth Road + Replacing Wentworth Road Main per Scenario 3	1,371	34.7%	$2,640,000

3.5.4.6 New Castle Tank

The previous hydraulic modeling study included a recommendation that creating a new pressure zone in New Castle would provide substantial improvements. This report stated that the zone would be isolated by valves at various locations in the distribution system and a new 300,000 gallon tank would be installed in combination with a booster pump station.

Our analysis indicated that the hydraulic grade in the main pressure zone is high enough to provide New Castle with adequate pressure without the need for a booster station. We considered two scenarios that include a 0.5 MG elevated storage tank in New Castle, connected directly to the main pressure zone. Two alternative locations for the tank are considered: 1) behind the school, or 2) off Shaw Circle. The Table 3-25 summarizes those results:

TABLE 3-25
New Pressure Zone and Elevated Tank

#	Improvement	Average New Castle AFF(gpm)	% Inc/Dec	Estimated Cost
NC-0	Base - existing conditions	1,018	N/A	N/A
NC-15	New Tank off Shaw Circle (plus Booster Station)	1,588	56.0%	$3,790,000
NC-16	New Tank behind school (plus Booster Station)	1,268	24.6%	$3,820,000

3.5.4.7 Discussion of New Castle Scenarios and Preliminary Recommendations

In reviewing the results, it appears that utilizing the Rye Water District line together with replacing the Wentworth Road water main (Scenario NC-14) would potentially provide more benefit to the New Castle available fire flows than replacing the Odiorne Point water main (NC-7). The planning-level costs estimates show that, together, these two projects are anticipated to cost less than the $3.0 million currently in the City's CIP and would provide more AFF improvement (34.7% vs. 9.1% Average New Castle AFF). We recommend that the City begin discussions with the Rye Water District about the potential of partnering on this project. The Rye Water District will benefit from the loop back to the Portsmouth/New Castle system in this area as they are currently vulnerable to service disruptions if there is a main break or other issue with the water main along Route 1B in Rye. Additionally, a phased implementation of these recommended projects, together with field verification of improved flow conditions after each upgrade has been constructed, will allow for refinement of which of the currently recommended projects will produce the greatest benefit to New Castle.

3.5.5 Rye Improvements

There are currently no reported deficiencies in the Rye portion of the system. Certain portions of the New Castle assessment might have implications on Rye but are addressed in the New Castle analysis.

Section 4
Summary and Recommendations

4.1 Summary

Updating the City of Portsmouth's Water Supply Master Plan and hydraulic model was an iterative and collaborative effort between Tighe & Bond and City of Portsmouth Water System staff. Numerous meetings and site visits took place over the course of this project. Throughout this process, the findings of the hydraulic modeling effort, infrastructure review of available sources of supply, pumpage capability at the Madbury Water Treatment Facility, the nine wells, and two booster stations, and an assessment of the type and age of distribution system water mains were reviewed. This, coupled with comprehensive inspections of the City's water storage tanks, resulted in the development of a list of recommended projects for the City's water system. This list was then assessed and prioritized to align with the water system's current and projected capital improvements program. Recommended projects are discussed in Sections 4.2 and 4.3.

Our findings of the hydraulic modeling effort, infrastructure review, assessment of the type and age of distribution system water mains, and assessment of the City's water storage tanks are summarized below.

Based on the results of the available fire flow analysis performed using the hydraulic model, several areas of the system appear to be fire flow deficient. The hydraulic modeling analysis also identified several areas with low pressures. The following hydraulic deficiencies were identified:

- Low pressures and low available fire flow exist in the Nimble Hill/Fox Point area of Newington. Connecting this portion of the Newington system to the Pease pressure zone is recommended as the most feasible and cost effective improvement to address these hydraulic issues. Connecting to the Pease pressure zone would raise the nominal hydraulic grade in the area from 171 ft MSL (the main zone grade) to 230 ft MSL (the Pease zone grade), increasing the static pressure by ~25 psi.
- The New Castle portion of the water system has a history of pressure and flow deficiencies. This is primarily due to the fact that it is at the furthest end of the water system, has older and undersized water mains and is also sub-metered for the New Castle Water District service territory. Half of the island's water mains are also owned and controlled by the New Castle Water District. The City is currently in discussions with the Town regarding potentially taking over this system. This would enable Portsmouth greater control of system improvement projects
- The Greenland portion of the water system has areas with low available fire flow and low pressure. This is primarily due to the relatively high ground elevations with respect to the hydraulic grade line of the main pressure zone that currently serves the area. Like Newington, connecting Greenland to the Pease pressure zone would raise the hydraulic grade in the area and increase the pressure.

The results of the hydraulic modeling water age analysis indicated that average water age greater than 100 hours occurs in New Castle, the southern part of the main pressure zone, and at the periphery of the system in Greenland and Newington.

The Osprey Landing Tank has been off-line for an extended period without significant negative impacts. Distribution system improvements over the years in the area of this tank have improved the fire flows and pressure in the area. The results of available fire flow analyses indicate the tank's impact on available fire flow outside the immediate vicinity of the tank is not significant and that the tank could be removed.

Under the existing configuration, approximately 30% of the Newington Booster Tank volume is exchanged per day by allowing the tank's level to fluctuate. Limited volume exchange could result in high water age that could potentially cause water quality deterioration in the tank. Additionally, pressure fluctuations in the transmission main between Madbury and the Newington Tank have created problems for customers connected to the transmission main. Tank retrofits to improve both the water turnover and the transmission main pressure are recommended.

The Lafayette Road tank has a storage capacity of approximately 7 MG, of which only 2.3 is useable. As a result of the large volume, the Lafayette Road Tank and surrounding area in the southern portion of the City that is influenced by the tank experience high water age. High water age has the potential to cause water quality problems, including loss of disinfectant residual and excessive concentrations of disinfection byproducts including TTHM and HAA5, which are a concern to the City in light of the new Stage 2 Disinfectant/Disinfection Byproduct Rule. A mixing system with water quality monitoring equipment is recommended for this tank.

The exterior and interior coatings of the Hobbs Hill Tank are no longer providing an effective corrosion barrier to the underlying steel surfaces. If the existing tank remains in service, complete rehabilitation is recommended as soon as possible to prevent aggressive metal loss as a result of the degrading coatings. There is also the potential need for additional storage during the planning period covered in this report. Replacing this tank and determining its proper size during design is recommended. Sizing of the tank will be dependent on whether or not Greenland and portions of Newington are converted over to the Pease pressure zone.

The three currently utilized Madbury Wells (#2, #3 and #4) were constructed at the same time and have been in service for over 60 years. Well # 1 has been off-line for a number of years and is no longer an approved source of supply for the system. The three active wells are all considered to be drawing water from the same aquifer. Based on the evaluation performed in 2012, Well #2 is starting to show signs that the screen may need to be replaced. Though it is possible to install new screens inside existing screens of wells to extend their life, this practice it often leads to declines in the well yield.

The New Hampshire Department of Environmental Services commissioned a study in 2006 to examine the potential for mutual aid between ten seacoast water systems. The City of Portsmouth was included in this study. Interconnections between the Portsmouth system and the City of Dover, the Town of Durham and the Rye Water District were considered. The most feasible interconnection was between Portsmouth and Rye. The Rye Water District's Washington Road Booster Station was modeled in this study with the City of Portsmouth's Lafayette Road water tank. A 4,000-foot length of new 16-inch water line between the two systems was modeled, and the proposed interconnection was determined to be feasible from a hydraulics standpoint, noting that flow from the Portsmouth main pressure zone would need to be pumped to Rye, and a PRV station would be needed to supply water from Rye to Portsmouth.

Groundwater generally has lower concentrations of the natural organic precursors that react with chlorine to form disinfection byproducts. Therefore, there is generally less potential for TTHM and HAA5 formation in groundwater compared to surface water, and the potential for disinfection byproduct formation at any location is expected to correlate to the percent surface water contribution. Modeling scenarios were prepared to evaluate the distribution of surface water and water age throughout the system, and how this distribution would be affected by reducing Madbury WTF production. The model predicts that reducing the production of the Madbury WTF would reduce the percent surface water contribution throughout the system, but most significantly would reduce both the surface water contribution and water age in the southern part of the main pressure zone (Lafayette tank area), which is the highest water-age area with the greatest potential for excessive TTHM and HAA5 formation. Thus, reducing the Madbury WTF production and relying more on the wells during the end of the summer is expected to reduce average TTHM and HAA5 concentrations during the period when high concentrations of these substances are most likely.

The City has been conjunctively managing their one surface water and nine groundwater sources of supply for many years. Their normal procedure calls for optimizing their surface water source when it has available quantity and good quality. By doing this they are able to rest their groundwater sources so that the aquifers are as recharged as possible and their yields will be maximized and available when either water customer demands go up or the surface water source quantity or quality necessitates reducing the yield on that supply. It is noted that the use of surface water sources, especially during the late summer, may increase the potential for disinfection by-products to form in the water system.

4.2 General Recommendations

This section describes recommended projects to improve system efficiency, water quality, pressures and available fire flow. Some of these projects are already included in the City's Capital Improvement Program. If so, we have re-visited their scope and cost estimates to provide updated recommendations. Other projects are new and have evolved from our hydraulic model findings and other investigations we performed as part of this study as well as through our discussions with the City's water operations staff. Recommended projects are summarized in Tables 4-1 and 4-2. Recommended water distribution projects are presented in Figure 4-1.

TABLE 4-1

Recommended Water System Improvements (Pumping & Storage)

Location / Scenario	Project Description	Estimated Project Cost	Project Objective
Newington			
N-2	Modifications to Newington Tank Inlet/Outlet	$220,000	Water quality, stabilize pressures for customers on transmission main

N-3	Newington Tank Re-Painting & Aeration System	$1,310,000	Water Quality
N-4	Pump Station Modifications including new VFDs	$420,000	Improve reliability and operational flexibility
Portsmouth			
PO-7b	Lafayette Road Tank mixing, spray aeration, and chlorination system (additional evaluation required)	$360,000	Water quality
PO-8	Osprey Landing Tank removal	$100,000	Eliminate tank maintenance
Pease			
PE-2a	Hobbs Hill tank replacement	$2,760,000	Upgrade aged and deteriorated tank, provide adequate storage volume
PE-3	Portable generator for Smith and Harrison Wells	$100,000	Reliability
Sherburne Rd			
S-1	Set Sherburne PRV to allow flow from Pease to main pressure zone	$0	Fire Flow

TABLE 4-2

Recommended Water System Improvements (Water Mains)

Location / Scenario	Project Description	Existing Pipe Diameter (in)	Proposed Pipe Diameter (in)	Length (ft)	Estimated Project Cost	Project Objective
Newington						
N-1	Connect Newington to Pease	NA	8	1,400	$760,000	Fire flow, increased pressure
New Castle						
NC-1	Remove meter pits/check valves/replace small diameter main	4,6 + valves, meters	8	100	$100,000	Reliability, Fire Flow, replace aging pipe
NC-4	Replace water main on Wild Rose Lane	6	8	2,600	$810,000	Reliability, Fire Flow, replace aging pipe
NC-7	Wentworth Road water line	8	12	650	$340,000	Fire flow
NC-14	Connect to Rye Water District Line across bridge on Wentworth Road + Replacing Wentworth Road Main	8	12	1,500	$2,640,000	Reliability, Fire Flow, replace aging pipe
Greenland						
G-1	Connect Greenland to Pease + Upgrade Greenland Well + New PRV on Ocean Road	NA	12	700	$600,000	Improved pressure, fire flow, water quality
Portsmouth						
PO-1b	Maplewood and Woodbury Avenue	6, 8	12	7,100	$3,300,000	Fire flow, replace aging pipe
PO-5	Atlantic Heights loop	NA	12	700	$340,000	Fire flow

insert Figure 4-1

4.3 Newington Tank and Booster Pumps

4.3.1 Newington Tank Retrofit

Under the existing configuration, approximately 30% of the Newington Tank volume is exchanged per day by allowing the tank's level to fluctuate. Limited volume exchange could result in high water age that could potentially cause water quality deterioration in the tank. Additionally, pressure fluctuations in the transmission main between Madbury and the Newington Tank have created problems for customers connected to the transmission main.

In order to address the pressure fluctuation and potential water age issues, reconfiguring the inlet/outlet piping of the tank is proposed. The existing common inlet/outlet would be changed by installing a new inlet line that would direct all incoming water from the transmission main directly into the top of the tank. The existing line from the bottom of the tank would remain in service and would serve as the outlet. Thus, all water entering the Newington Booster Station would have to go through the tank before being pumped out into the distribution system. This change would accomplish two goals:

1. It would allow for a consistent hydraulic gradient for the WTF transmission line because the grade line would be fixed at the elevation of the discharge to atmosphere at the top of the tank and would not depend on the tank water level.
2. It would significantly increase turnover and reduce water age. For example, assuming an average transmission main flow rate of 2,500 gpm and an average tank level of 40 ft, approximately 250% of the tank's volume would be exchanged per day, compared to 30% volume exchange that is currently typical. This improvement would also create more mixing of the water in the tank and allow for some cascading water out of the pipe at the top of the tank to provide aeration.

An engineered spray aeration system for removing TTHM should also be considered. The proposed system is expected to significantly reduce the total trihalomethanes (TTHMs) in the water transiting the aerator. TTHM formation starts at the treatment plant at the point of chlorination and continues over a period of days. Thus, the amount of reduction in TTHM concentration in the distribution system in general resulting from an aeration system at the Newington tank would depend on the extent of TTHM formation that occurs upstream of the aerator. If most TTHM formation occurs downstream of the tank, then this improvement would not be effective; however, if significant TTHM formation occurs upstream of the tank, this improvement could significantly improve water quality in the distribution system. TTHM sampling from the transmission main directly upstream of the tank is recommended to evaluate the potential of an aeration system to improve water quality.

We recommend that the City proceed with further evaluation of this upgrade. The scope should include the following:

- TTHM and HAA5 sampling of the raw water transmission line water quality and tank inlet and outlet waters to profile the formation of these disinfection byproducts.
- A detailed structural assessment of the booster tank to assure that any retrofit of the tank is able to accommodate the modifications.

- Preliminary design of booster station pump and piping upgrades.

As discussed in Section 3, it is recommended that the new pumps be equipped with VFDs and sized to provide the following flow ranges:

- Pumps #1 and #2: 1,000 to 1,500 GPM
- Pumps #3 and #4: 2,000 to 2,500 GPM

4.3.2 Newington Distribution System Improvements

As discussed in Section 3, low pressures and low available fire flow exist in the Nimble Hill/Fox Point area of Newington. Connecting this portion of the Newington system to the Pease pressure zone is recommended as the most feasible and cost effective improvement to address these hydraulic issues. Connecting to the Pease pressure zone would raise the nominal hydraulic grade in the area from 171 ft MSL (the main zone grade) to 230 ft MSL (the Pease zone grade), increasing the static pressure by ~25 psi.

Connecting the Newington area with the Pease pressure zone would be accomplished by installing approximately 1,400 ft of 8-inch water main from the terminus of the existing main in Nimble Hill Road to the existing 8-inch water main at the north west edge of the North Apron in the Pease zone. A PRV station would be installed in Nimble Hill Road near the intersection of Coleman Drive.

4.4 New Castle Water Main Improvements

As discussed in Section 3, parts of New Castle have deficient available fire flow. Several potential distribution system improvements were evaluated with respect to their effectiveness in improving available fire flows. The City's CIP has $3.0 million earmarked for replacement of the water line that runs from Odiorne Point in Rye to the Great Island. Our analysis of other alternatives shows that installing a new connection between the Rye Water District line on Route 1B and the Wentworth Road water line near the Wentworth Hotel would improve available fire flows more than the replacement of the Odiorne Point water main, and would be less expensive. This alternative would allow additional improvements to the Wentworth Road water main and other water main improvements on the Island within the existing capital budget. Additionally, a new connection to the Rye Water District's line would provide additional available fire flow and redundancy to the Rye Water District. Currently, this portion of their water system is vulnerable since it is a dead end line. The new connection to New Castle would improve this situation.

We recommend that the City begin discussions with both the New Castle and Rye Water District with respect to these options.

4.5 Greenland Improvements

As discussed in Section 3, the Greenland portion of the water system has areas with low available fire flow and low pressure. This is primarily due to the relatively high ground elevations with respect to the hydraulic grade line of the main pressure zone that currently serves the area. Our analysis indicates that the most effective and economical alternative for improving available fire flow and pressure in Greenland is connecting this portion of the system to Pease pressure zone.

The proposed connection from Greenland to the Pease system would be provided near the Smith Well. An isolation valve would be provided near the connection point, which would put the entire Greenland area on the Pease pressure zone hydraulic gradient. A PRV would be installed on Ocean Road to allow flow from the new Greenland high pressure zone into the Portsmouth main pressure zone. Supply would be provided by the Haven, Smith, Harrison, and Greenland wells. The Newington Booster station and Pease WTF booster pumps are also available to provide supply.

Under this alternative, the Greenland well would have to be upgraded to operate against the increased hydraulic head. The following figure shows a general overview of the Greenland/Southern portion of Portsmouth part of the water system and how system fire flows could be improved by putting Greenland on the Pease pressure zone.

This alternative would provide the following additional benefits:

- Potential to allow better control and integration of blending the Pease wells with the Portsmouth system. The Greenland well would be added to this mix, providing the potential to increase the groundwater contribution in the south end of the main pressure zone.
- The proposed PRV at Ocean Road may also allow for increased turnover of the Lafayette Road Tank.
- The proposed PRV on Ocean Road would improve available fire flow in the south end of the main pressure zone by adding a higher pressure, higher flow source of supply to this area of the system.

4.6 Portsmouth Main Pressure Zone Improvements

4.6.1 Maplewood Avenue and Woodbury Avenue Upgrades

The City's CIP currently includes $3.0 million in the FY15 budget for a water main replacement project that consists of replacing approximately 7,500 feet of 6-inch and 8-inch 90-year old waterline on Maplewood Avenue from Woodbury Avenue to Raynes Avenue with new 16-inch cement-lined ductile iron waterline. Our analysis of this project as discussed in Section 3 indicates that the proposed 16-inch main provides a significant benefit in available fire flow in the immediate vicinity of the new main, but does not provide much benefit in the downtown area. Substitution of a 12-inch water mains rather than the proposed 16-inch main would provide similar benefit at a reduced cost.

4.6.2 North Mill Pond Area Improvements

The existing water mains in the North Mill Pond area are predominantly older 2, 4 and 6-inch pipes. Our analysis shows that replacing the small diameter lines in the area with new 8-inch water lines would result in significantly improved available fire flow throughout the North Mill Pond area of the system, increasing the average available fire flow from approximately 1,700 gpm to approximately 3,000 gpm. Therefore, we recommend that the City include this project as part of their ongoing CIP projects for waterline replacements.

4.6.3 Atlantic Heights Loop

This scenario consists of installing a new 12-inch pipe to connect Atlantic Heights at Crescent Street to Dunlin Road. While not providing a significant benefit outside the

Atlantic Heights area, the modeling results indicate that proposed water main would provide a significant benefit in the area, which currently has available fire flow of less than 1,000 gpm. Since the project is estimated to cost less than $500,000 we recommend that the City include this project as part of their ongoing water main replacement projects.

4.6.4 Osprey Landing Tank Removal

Based on the model predictions and the fact that the tank has been off-line for an extended period without significant negative impacts, we conclude that the tank's impact on available fire flow outside the immediate vicinity of the tank is not significant and that the tank could be removed. We recommend that the City proceed with this project.

4.6.5 Lafayette Road Tank Improvements

The Lafayette Road tank has a storage capacity of approximately 7 MG, of which only 2.3 is "useable." As a result of the large volume, the Lafayette Road Tank and surrounding area in the southern portion of the City that is influenced by the tank experience high water age. High water age has the potential to cause water quality problems, including loss of disinfectant residual and excessive concentrations of disinfection byproducts including total trihalomethanes (TTHM) and haloacetic acids (HAA5), which are a concern to the City in light of the new Stage 2 Disinfectant/Disinfection Byproduct Rule. Section 3 provides a discussion of potential retrofits to address water quality issues in the Lafayette Road Tank.

As discussed in Section 3, it has been demonstrated recently that spray aeration systems can significantly reduce TTHM concentrations in storage tanks. We recommend that the City perform pilot testing a mixing, spray aeration and chlorination system for installation next year prior to the summer season. Information gathered during this pilot would be helpful to assess overall water quality mixing in the tank, the effect it has on TTHMs, HAA5s, chlorine residual and overall water quality.

4.7 Pease Pressure Zone Improvements

4.7.1 Hobbs Hill Tank Replacement

The exterior and interior coatings are no longer providing an effective corrosion barrier to the underlying steel surfaces. If the existing tank remains in service, complete rehabilitation is recommended as soon as possible to prevent aggressive metal loss as a result of the degrading coatings. The estimated cost to rehabilitate this is $800,000 to $900,000.

As indicated in Section 1, a total storage capacity of 1 MG is recommended for the year 2030, assuming that Newington and Greenland are connected to the Pease zone. If the Hobbs Hill tank is replaced with a new tank, a minimum useable volume of 634,000 gallons would be required to provide the recommended capacity. Therefore, if the Hobbs Hill tank is replaced with a new tank, an elevated tank with a capacity of 634,000 gallons is recommended. Due to the extensive cost to repair the existing tank, and the potential need for additional storage during the planning period covered in this report, it is recommended that the City of Portsmouth consider replacing the Hobbs Hill Tank with a new water storage tank.

4.7.2 Portable Generator with Quick Connect Hookups for Smith and Harrison Wells

The Smith and Harrison Wells are not equipped with standby power. Installing emergency generators at these sites is recommended. We recommend that the City consider purchasing a portable generator set capable of running one of these wells during an extended power outage. If both of these sites are upgraded to have electrical quick-connections installed then utilizing a standby power system arrangement like this would provide additional flexibility and redundancy to the system.

4.8 Water Supply Management Recommendations

4.8.1 Rye Water District Emergency Interconnection

The New Hampshire Department of Environmental Services commissioned a study in 2006 to examine the potential for mutual aid between ten seacoast water systems. The City of Portsmouth was included in this study. Interconnections between the Portsmouth system and the City of Dover, the Town of Durham and the Rye Water District were considered. The most feasible interconnection was between Portsmouth and Rye.

The Rye Water District's Washington Road Booster Station was modeled in this study with the City of Portsmouth's Lafayette Road water tank. A 4,000-foot length of new 16-inch water line between the two systems was modeled, and the proposed interconnection was determined to be feasible from a hydraulics standpoint, noting that flow from the Portsmouth main pressure zone would need to be pumped to Rye, and a PRV station would be needed to supply water from Rye to Portsmouth. We recommend that the City of Portsmouth meet with the Rye Water District to explore opportunities to install this connection to provide emergency backup supply for both systems. At this time it is not known how much capacity the Water District might be able to supply the Portsmouth system but it is our understanding that they have been successful in expanding their groundwater supply capabilities in the recent years.

4.8.2 Madbury Well Replacements

As discussed in Section 1, the three currently utilized Madbury Wells (#2, #3 and #4) were constructed at the same time and have been in service for over 60 years. Well # 1 has been off-line for a number of years and is no longer an approved source of supply for the system. The three active wells are all considered to be drawing water from the same aquifer.

Based on evaluation performed in 2012, Well #2 is starting to show signs that the screen may need to be replaced. Though it is possible to install new screens inside existing screens of wells to extend their life, this practice it often leads to declines in the well yield. Therefore, instead of installing a new screen, we recommend that the City plan to start a replacement program for these wells, beginning with well #2.

According to NHDES regulation Env-Dw 302.30, a water system can replace an existing well as long as it derives water from the same zone of contribution as the well that is being replaced. An assessment of the long-term sustainable yield must be performed to show that the new well will withdraw water at the approved capacity of the well being replaced or the long-term sustainable yield as tested, whichever is less. Once the new well is approved and in service, the well that it replaced must be abandoned. An alternative would be to install back-up well(s) at the existing well sites, in accordance with NHDES regulation Env-Dw 302.29.

4.8.3 Bedrock Well Potential for Additional Supply

The City commissioned the firm Emery-Garrett Groundwater, Inc. in 2009 to investigate potential sites for potential bedrock well development. A number of locations were identified. We recommend that the City continue to explore the potential to obtain either ownership or easement agreements at some of the sites identified by Emery-Garrett to continue exploration and identify final locations for potential drilling and permitting of a new large groundwater withdrawal for the water system.

4.8.4 Integrated System Supply and Management Plan

The City has been conjunctively managing their one surface water and nine groundwater sources of supply for many years. Their normal procedure calls for optimizing their surface water source when it has available quantity and good quality. By doing this they are able to rest their groundwater sources so that the aquifers are as recharged as possible and their yields will be maximized and available when either water customer demands go up or the surface water source quantity or quality necessitates reducing the yield on their supply. It is noted that the use of surface water sources, especially during the late summer, may increase the potential for disinfection by-products to form in the water system. As previously mentioned, we are recommending that the City update its water supply management and source protection program. Currently, the City's water system has real-time monitoring of all of its sources of supply. They also have the ability to eventually get real-time customer usage information via their new water meter reading system.

Tighe & Bond recently completed work, together with Comprehensive Environmental, Inc., on the Massachusetts Sustainable Water Management Initiative Pilot Program. As part of that project we identified ways that water systems could utilize available data to assess potential withdrawal limits from multiple sources of supply. These assessments were also used in four Massachusetts water systems, all with surface and groundwater withdrawal capability, or alternative sources, like the City of Portsmouth's water system. The report, which is currently in draft form, recognizes the ability of these water systems to reduce withdrawal impacts by tracking and managing their sources of supply, especially during drought conditions. The report recommendations also included the following guidelines that water systems should implement:

1. Optimization of existing resources;
2. Use of alternative sources;
3. Interconnections with other communities or suppliers;
4. Outdoor water restrictions tied to streamflow [and/or groundwater availability] triggers (e.g., greater restrictions on outdoor watering than is currently applied);
5. Implementation of reasonable conservation measures;
6. Utilization of the New England Water Works Association's Best Management Practice (BMP) toolbox

We recommend that the City develop an Excel-based spreadsheet tool that the water system managers and operators can utilize to track and assess sources of supply over time. This spreadsheet would combine information that is already being gathered by the operators, the SCADA system and regional climate and hydrological data sources into one data set. From this data, past trends can be analyzed and compared to current operations data. A supply versus demand assessment can also be made from this analysis that would enable the City to determine if water restrictions are necessary or other measures needed to augment supply or declare an emergency. The following are a few of the parameters we recommend including:

1. Regional Precipitation data, stream flows, groundwater levels and drought conditions assessment from the New Hampshire Department of Environmental Services: http://des.nh.gov/organization/divisions/water/dam/drought/drought-conditions.htm
2. Bellamy Reservoir Information
    a. Oyster River streamgage data from USGS website to use for Bellamy inflow: http://waterdata.usgs.gov/usa/nwis/uv?01073000
    b. Water level at the dam
    c. Estimate of spillway water going over the dam
    d. Estimate of water flow through the dam's outlet pipe
    e. Daily raw water from Reservoir processed through the Madbury Water Treatment Facility
3. Madbury Water Treatment Facility
    a. Daily raw water
    b. Daily process water used at facility
    c. Daily treated water pumped into the system
4. Well Data
    a. Pump hour run times and rates
    b. Pumping and static water levels
    c. Daily water pumped into system
5. Booster Stations and PRVs
    a. Run times and/or pumpage data for each to assess flows
    b. Portsmouth into Pease or Pease into Portsmouth system flow
6. Storage Tank Data
    a. 8:00 am tank level
    b. Calculation of previous day's level to determine amount of increase or decrease of water storage
7. Water Usage and Demand Data
    a. Monthly billing data from Finance Department
    b. Daily data (if available) from individual customer water meters
    c. Flushing, Fire Use, Known leaks, etc. to determine unaccounted-for water in the system on a rolling 12-month basis

Once this tool has been developed we recommend that the City include this in their Standard Operating Procedures for all water systems staff as a guideline for system operations. We also recommend that water quality parameters be considered as part of these procedures. By tracking TOC, chlorine residual, water temperature and other indicators in the system with additional monitoring equipment it may be possible for the City to also manage its sources of supply such that surface water is utilized more from October through May and groundwater more during the warmer summer months to lower the potential for disinfection byproducts. By adding and tracking all of this information through the use of the Integrated Management Tool, water quality trends will also be tracked and managed better by operational staff.

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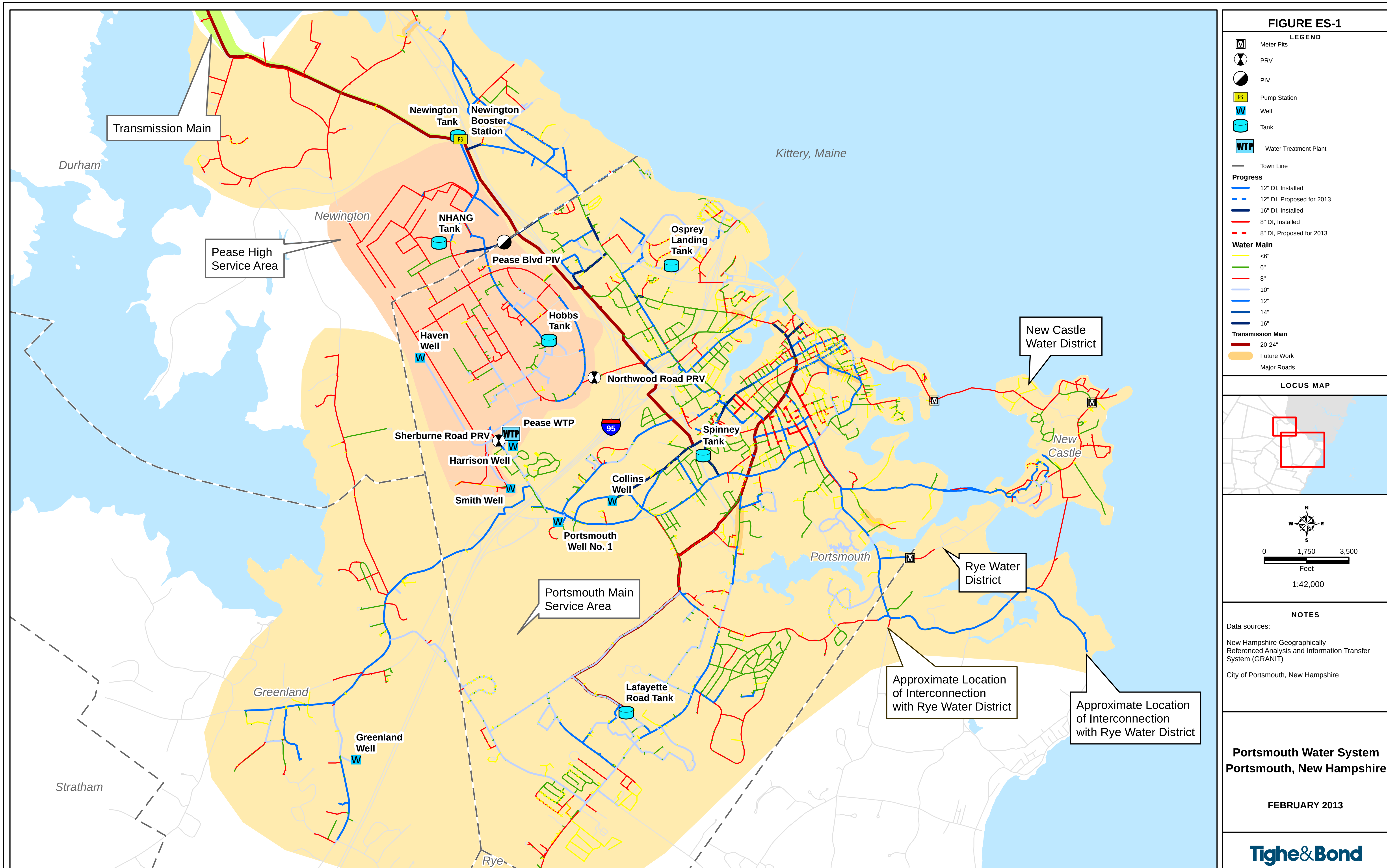

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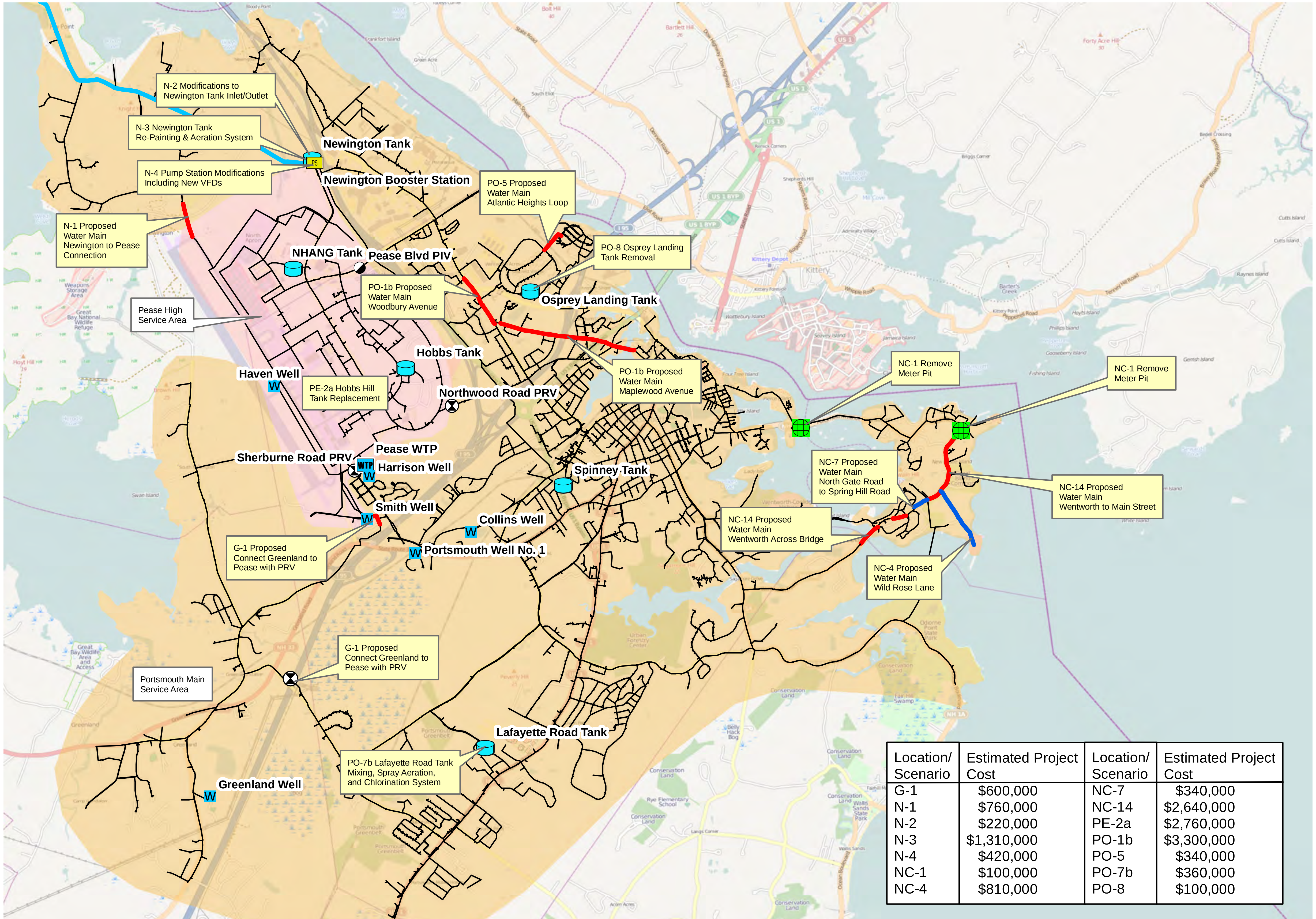

Location/ Scenario	Estimated Project Cost	Location/ Scenario	Estimated Project Cost
G-1	$600,000	NC-7	$340,000
N-1	$760,000	NC-14	$2,640,000
N-2	$220,000	PE-2a	$2,760,000
N-3	$1,310,000	PO-1b	$3,300,000
N-4	$420,000	PO-5	$340,000
NC-1	$100,000	PO-7b	$360,000
NC-4	$810,000	PO-8	$100,000

Figure ES-4 Recommended Water Distribution Projects

Proposed Water Main

Water Main

PS Pump Station

W Well

Tank

PRV

PIV

WTP WTP

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Portsmouth Water System
Portsmouth, New Hampshire
February 2013

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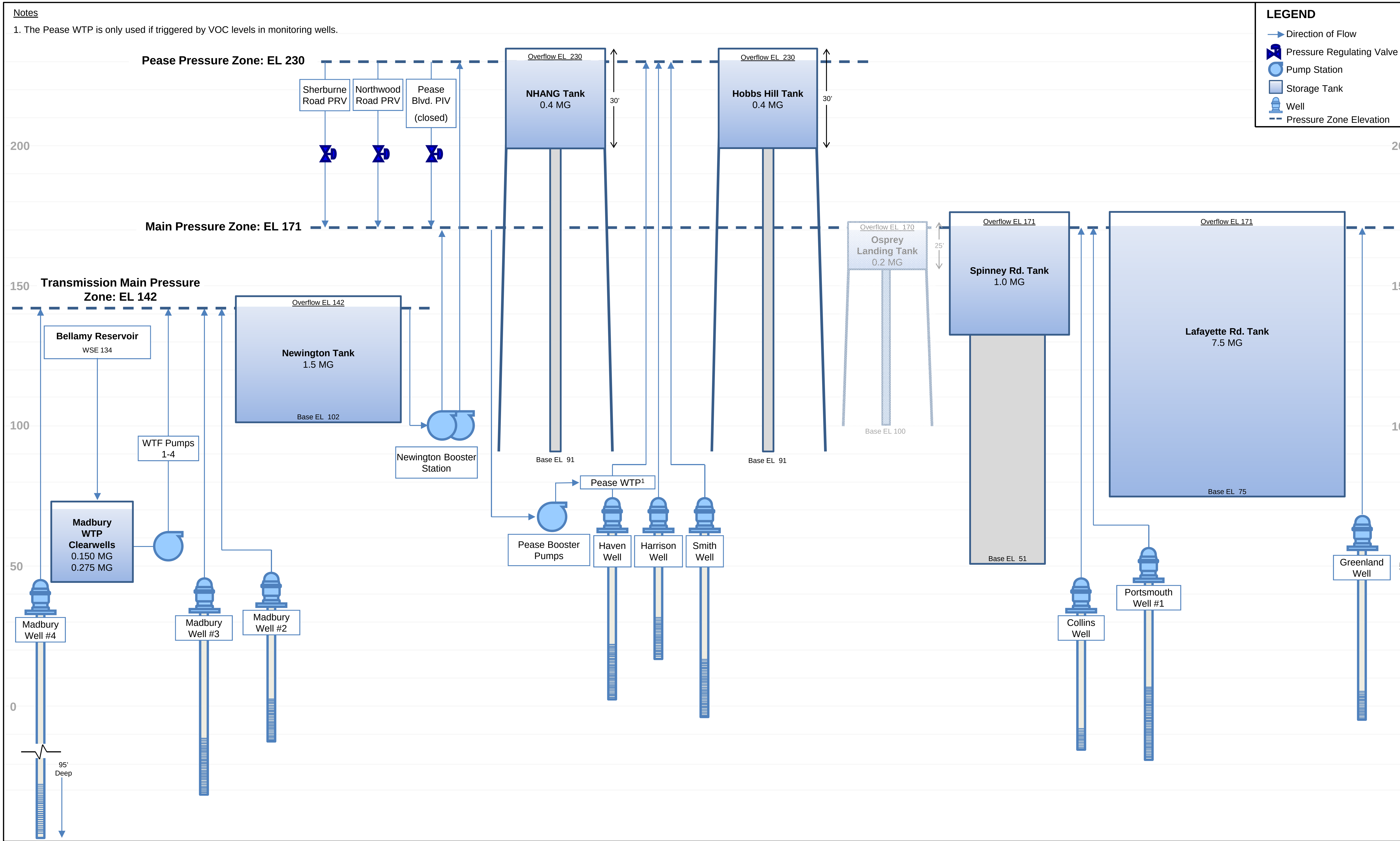

VERTICAL DATUM: USGS
HORIZONTAL SCALE: NOT TO SCALE

Figure 3-1 Distribution System Schematic - The Portsmouth Water System

The City of Portsmouth, NH

February 2013

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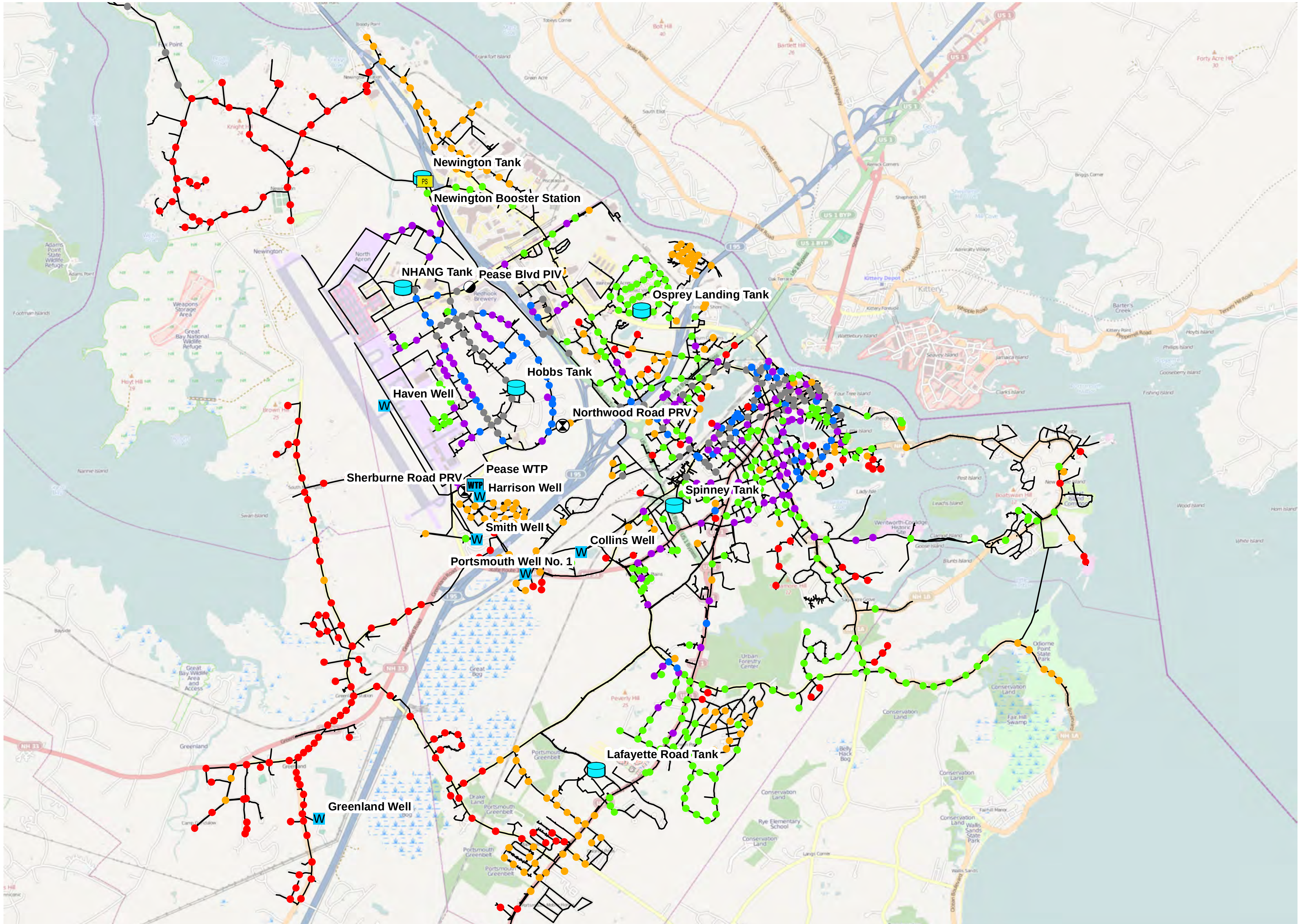

Figure 3-8
Model Predicted Available Fire Flow Maximum Day Demand (July 21-22, 2011 System Demand: 6.7 MG)

Model Predicted Available Fire Flow (gpm)

- **<500**
- **501 - 1000**
- **1001 - 2000**
- **2001 - 3000**
- **3001 - 3500**
- **>3500**

- Water Main
- Pump Station
- Well
- Tank
- PRV
- PIV
- WTP

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Portsmouth Water System
Portsmouth, New Hampshire
February 2013

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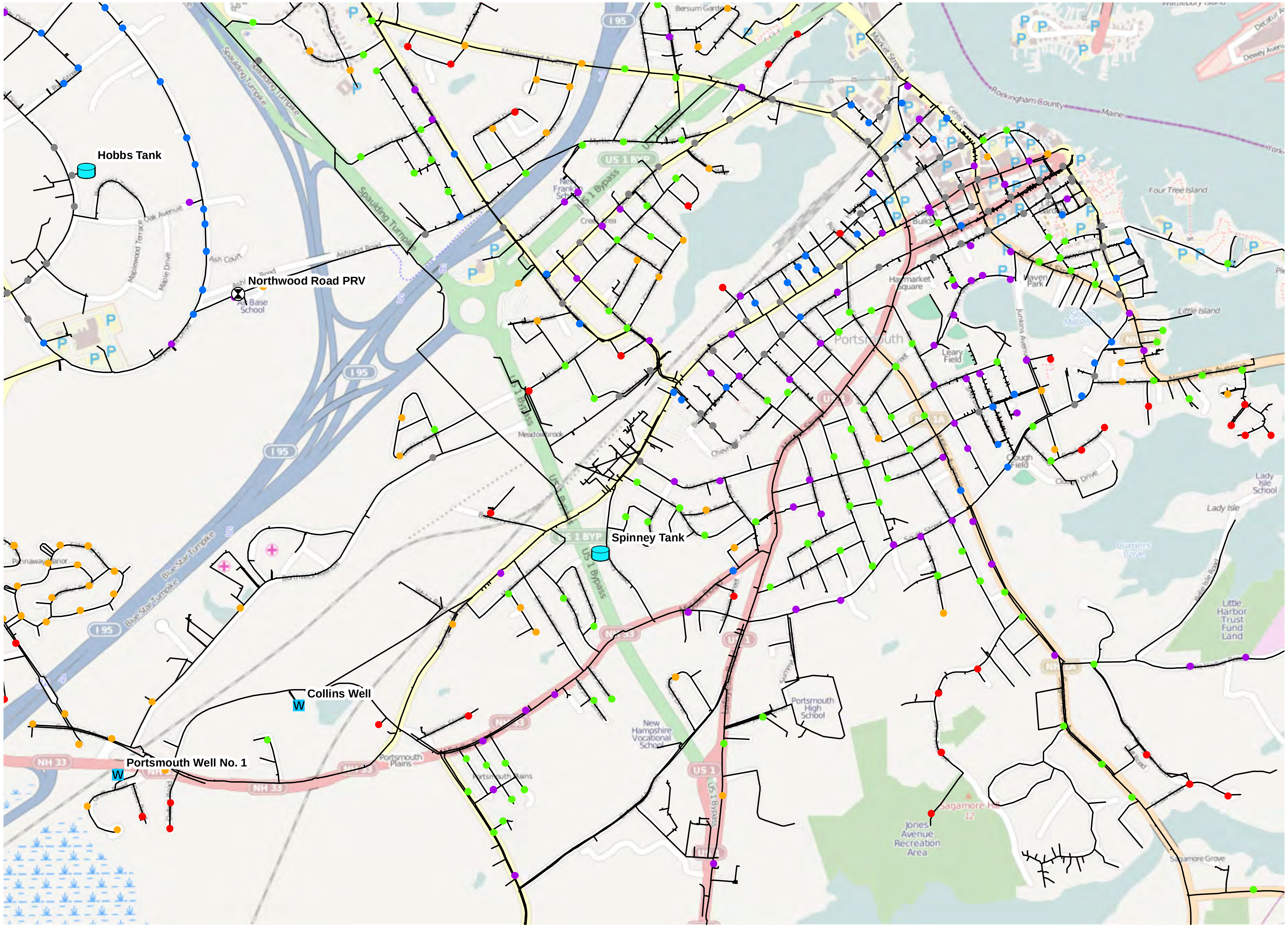

Figure 3-9
Model Predicted Available Fire Flow Downtown Area Maximum Day Demand (July 21-22, 2011 System Demand: 6.7 MG)

Model Predicted Available Fire Flow (gpm)

- <500
- 501 - 1000
- 1001 - 2000
- 2001 - 3000
- 3001 - 3500
- >3500

- Water Main
- Pump Station
- Well
- Tank
- PRV
- PIV
- WTP

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Portsmouth Water System
Portsmouth, New Hampshire
February 2013

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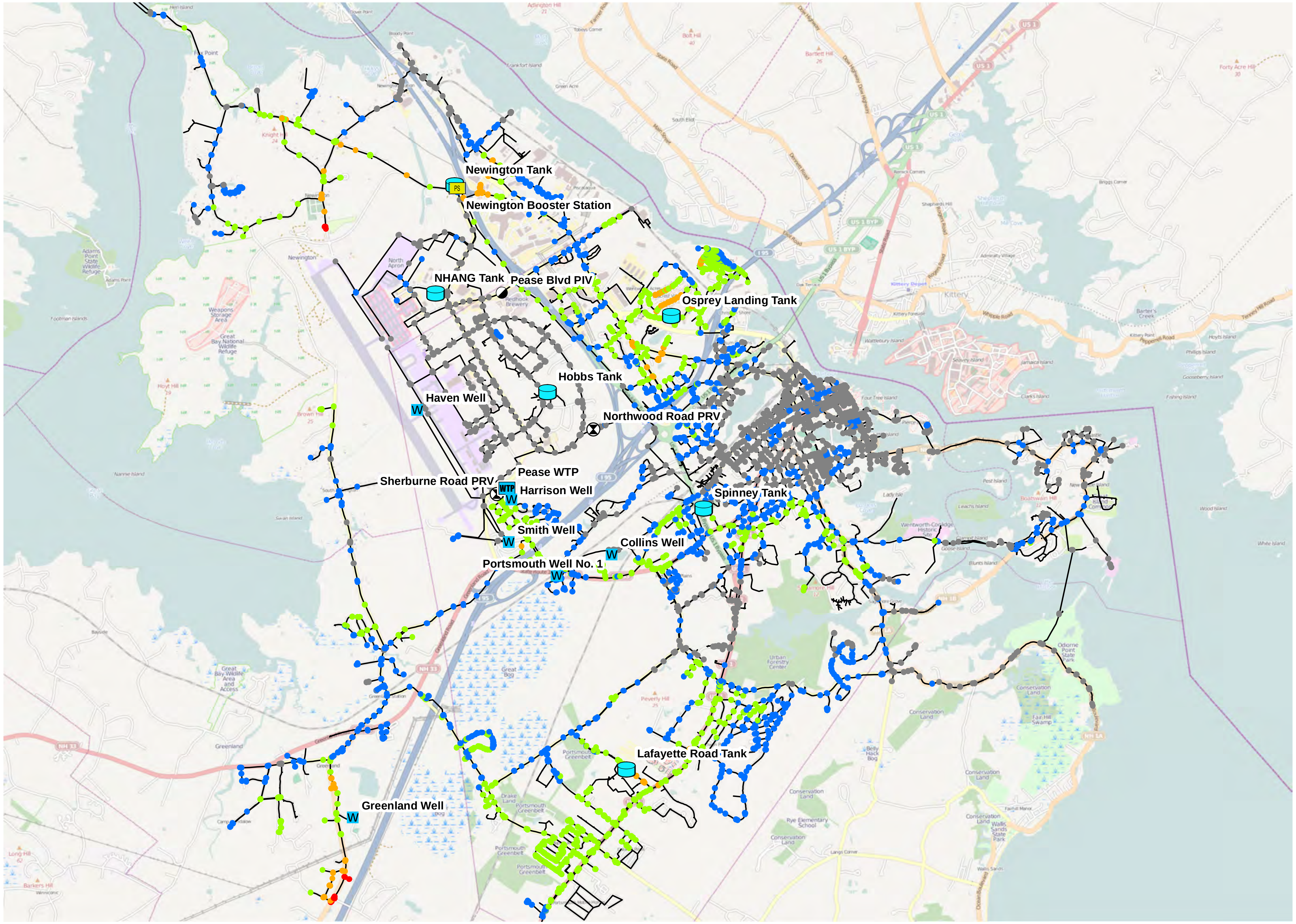

Figure 3-10
Model Predicted Minimum Pressure Maximum Day Demand (July 21-22, 2011 System Demand: 6.7 MG)

Model Predicted Minimum Pressure (psi)

- <20
- 21 - 35
- 36 - 45
- 46 - 55
- >55

- Water Main
- Pump Station
- Well
- Tank
- PRV
- PIV
- WTP

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Portsmouth Water System
Portsmouth, New Hampshire
February 2013

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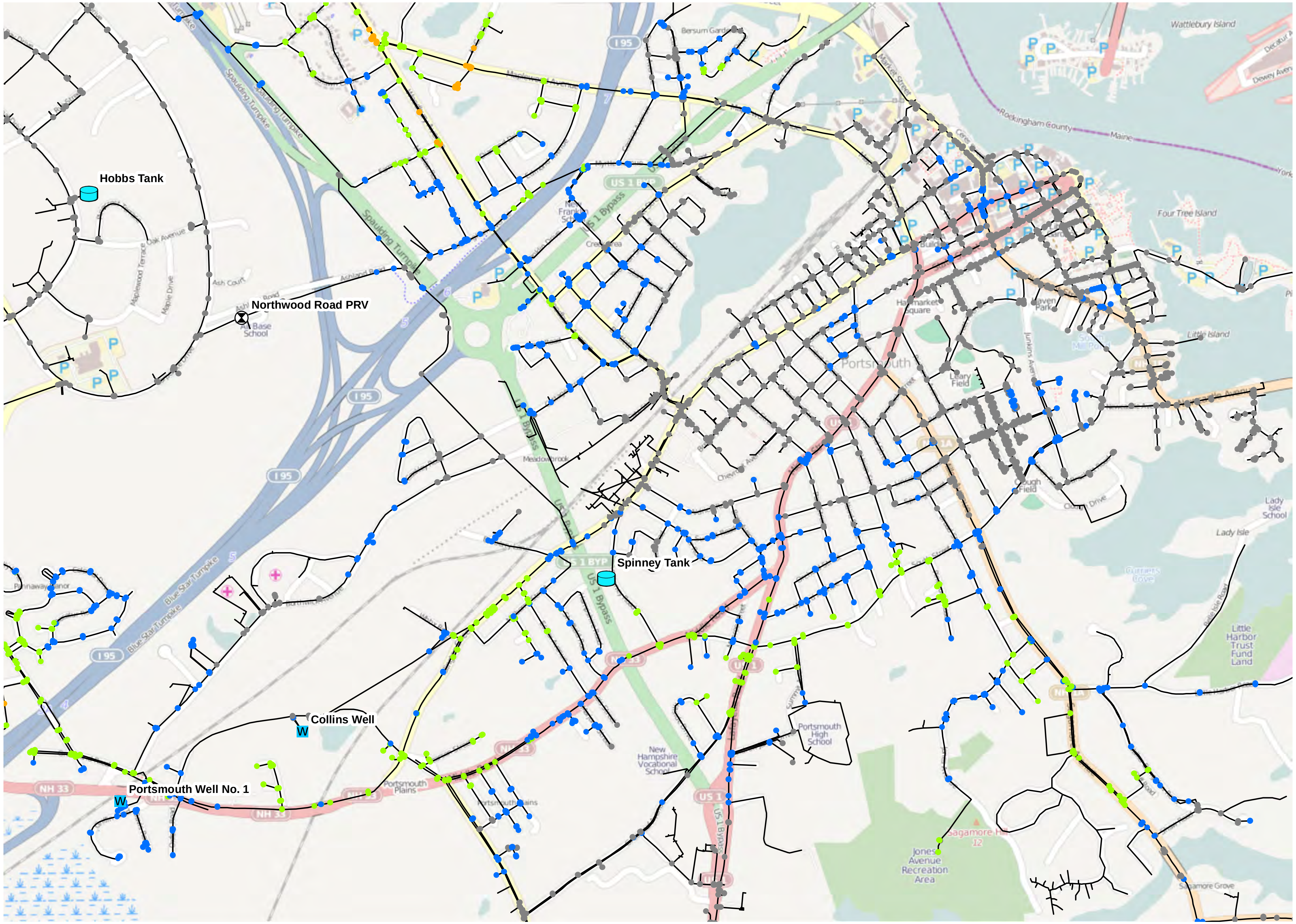
Figure 3-11
Model Predicted Minimum Pressure Downtown Area Maximum Day Demand (July 21-22, 2011 System Demand: 6.7 MG)



Model Predicted Minimum Pressure (psi)

- <20
- 21 - 35
- 36 - 45
- 46 - 55
- >55

- Water Main
- Pump Station
- Well
- Tank
- PRV
- PIV
- WTP

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Portsmouth, New Hampshire
February 2013

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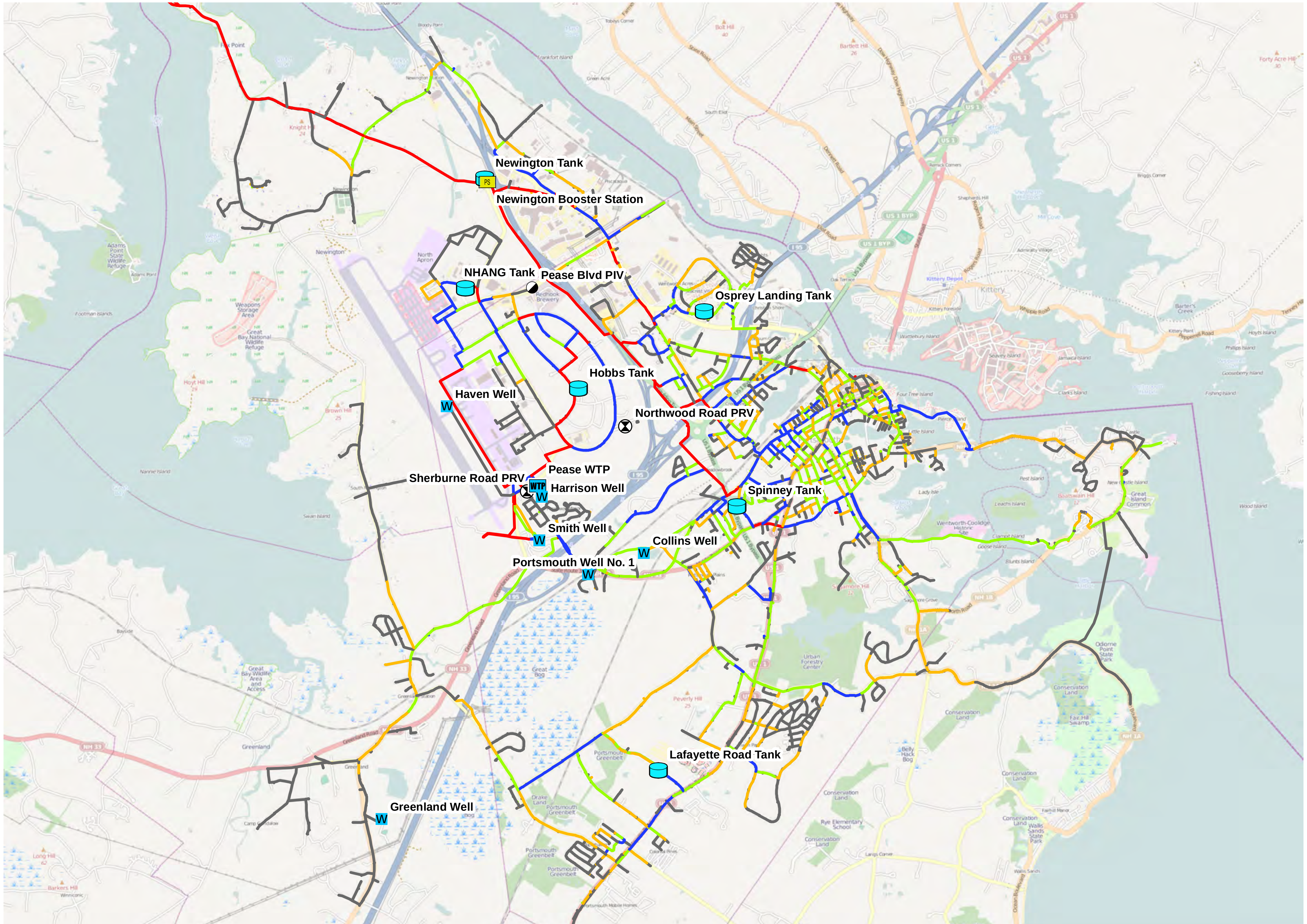
Figure 3-12
Maximum Model Predicted Velocity Maximum Day Demand (July 21-22, 2011 System Demand: 6.7 MG)



Model Predicted Velocity (ft/s)

- <0.10
- 0.11 - 0.20
- 0.21 - 0.40
- 0.41 - 1.00
- >1.00

- Pump Station
- Well
- Tank
- PRV
- PIV
- WTP

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Portsmouth Water System
Portsmouth, New Hampshire
February 2013

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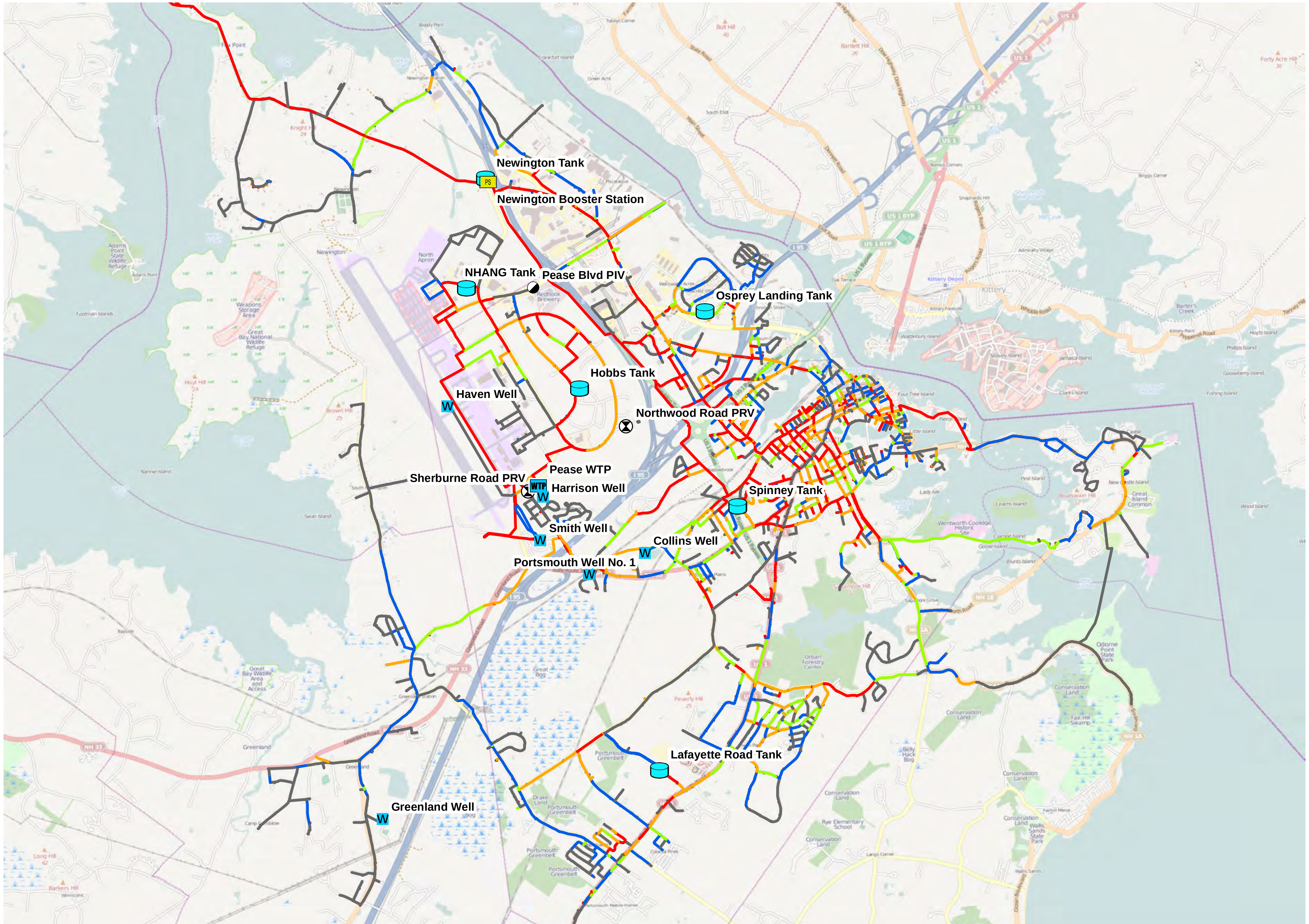

Figure 3-13
Maximum Model Predicted Head Loss Maximum Day Demand (July 21-22, 2011 System Demand: 6.7 MG)

Model Predicted Head Loss (ft) per Thousand Feet

- <0.01
- 0.02 - 0.05
- 0.06 - 0.10
- 0.11 - 0.25
- >0.25

- Pump Station
- Well
- Tank
- PRV
- PIV
- WTP

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Portsmouth Water System
Portsmouth, New Hampshire
February 2013

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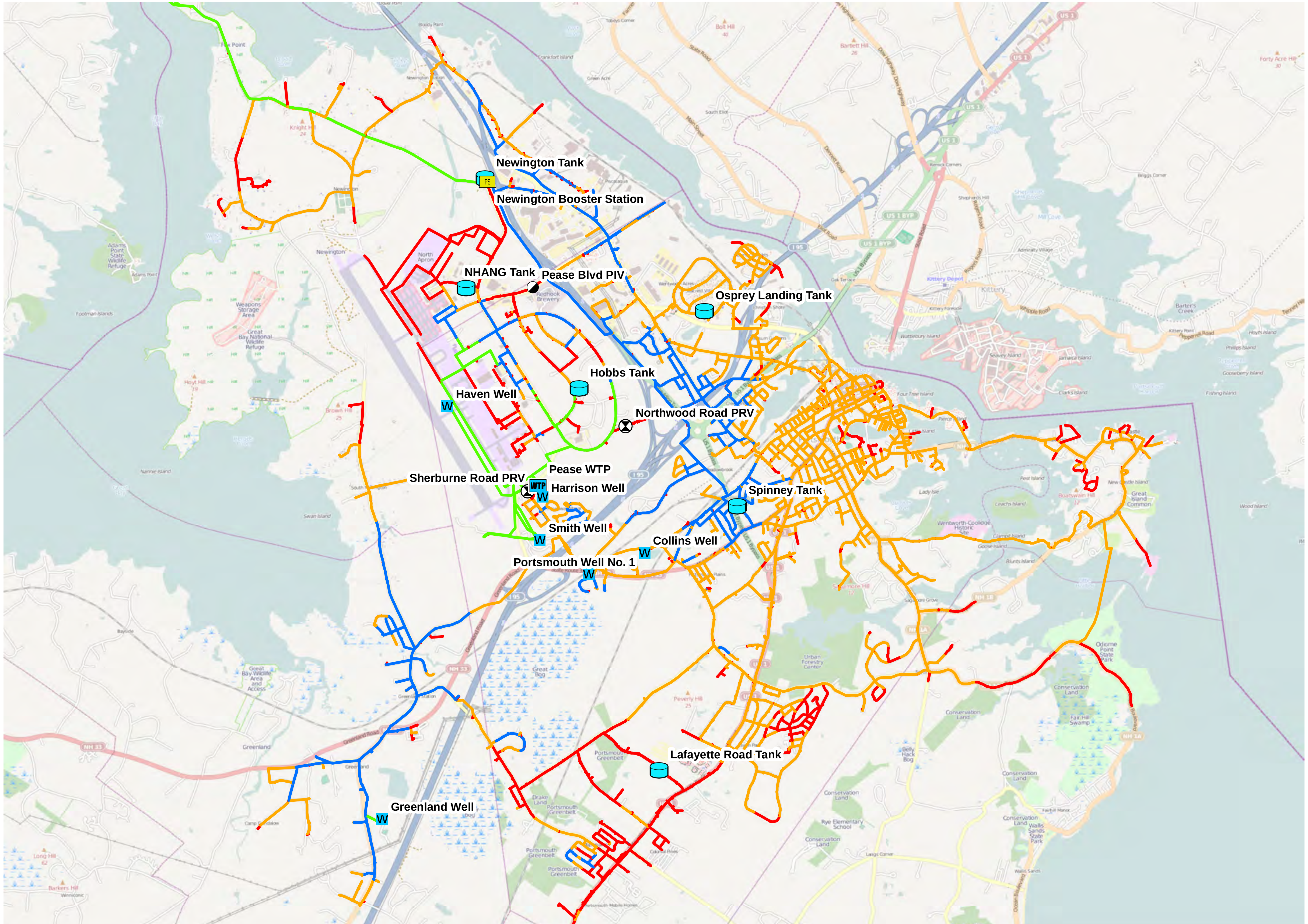

Figure 3-14
Model Predicted Water Age
Average Day Demand
4:00 PM
(May 22-23, 2011
System Demand: 4.5 MG)

Model Predicted Water Age (hrs)

- <15
- 16 - 30
- 31 - 100
- >100
- Pump Station
- Well
- Tank
- PRV
- PIV
- WTP

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Portsmouth, New Hampshire
February 2013

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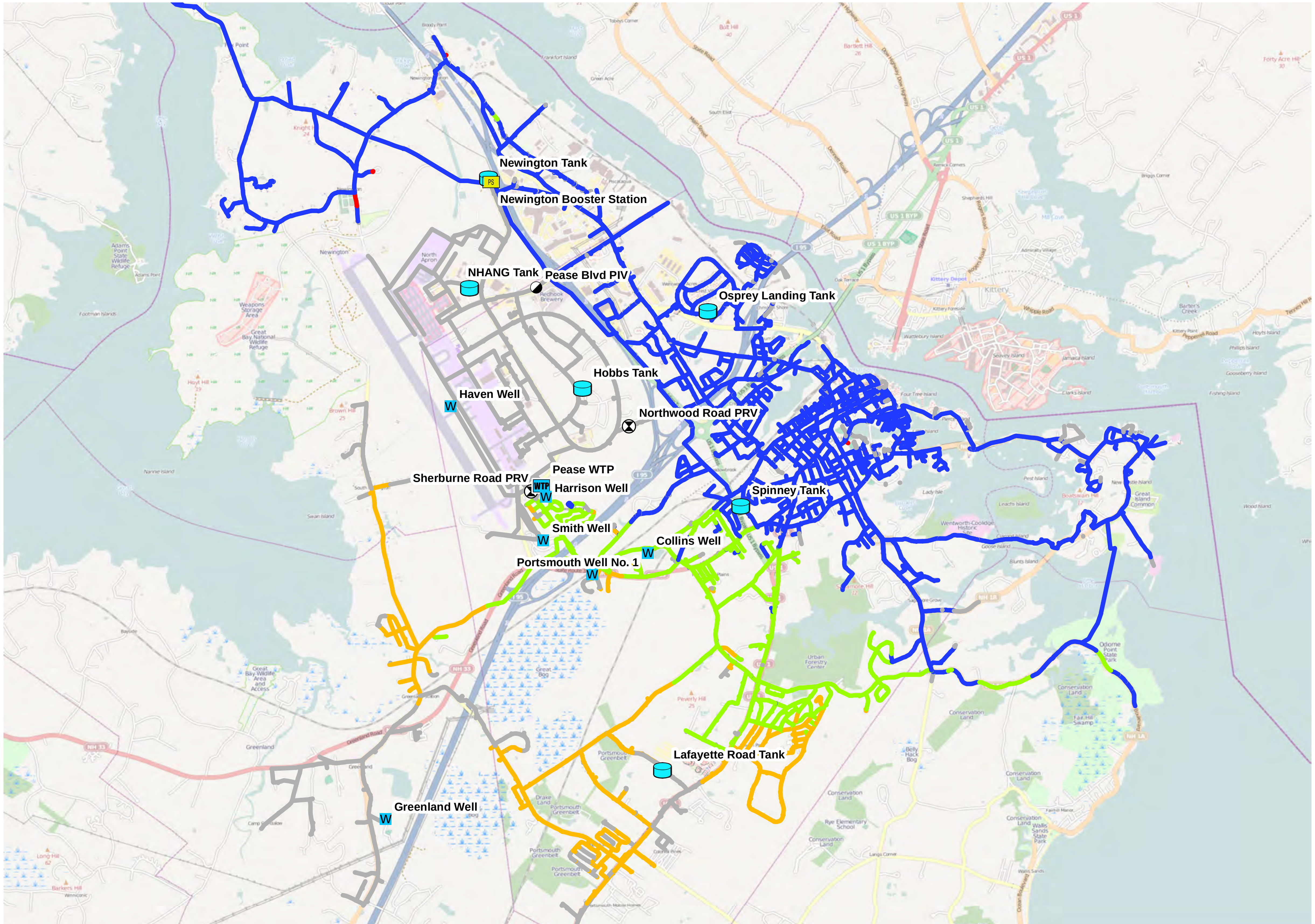

Figure 3-15
Model Predicted Surface Water Distribution Baseline Scenario Average Day Demand (May 22-23, 2011 System Demand: 4.5 MG)

Model Predicted Surface Water Contribution (%)

- <20%
- 20 - 40%
- 40 - 60%
- 60 - 80%
- >80%
- Pump Station
- Well
- Tank
- PRV
- PIV
- WTP

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February 2013

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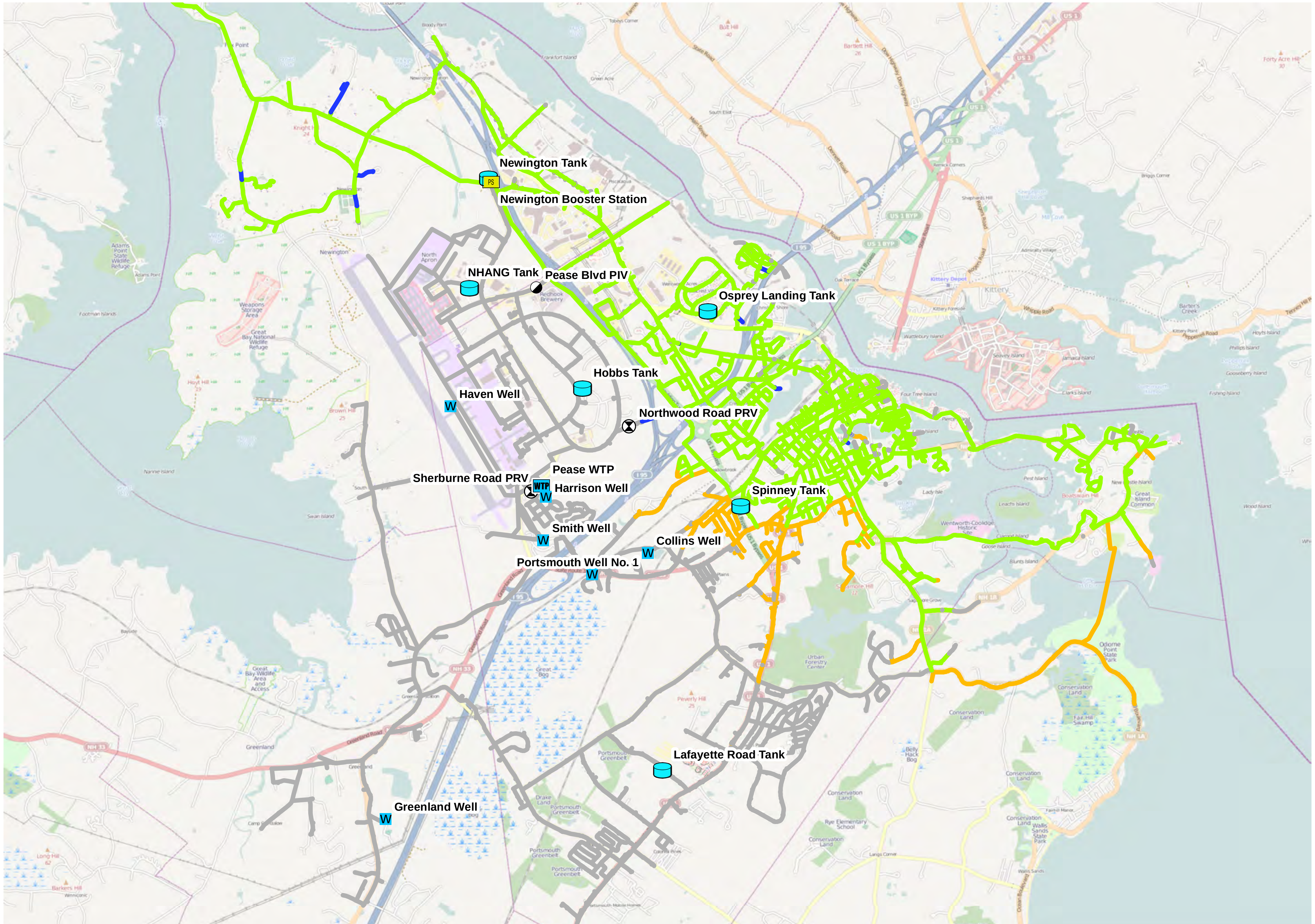

Figure 3-16
Model Predicted Surface Water Distribution Reduced Surface Water Contribution Average Day Demand (May 22-23, 2011 System Demand: 4.5 MG)

Model Predicted Surface Water Contribution (%)

- <20%
- 20 - 40%
- 40 - 60%
- 60 - 80%
- >80%
- Pump Station
- Well
- Tank
- PRV
- PIV
- WTP

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Portsmouth Water System
Portsmouth, New Hampshire
February 2013

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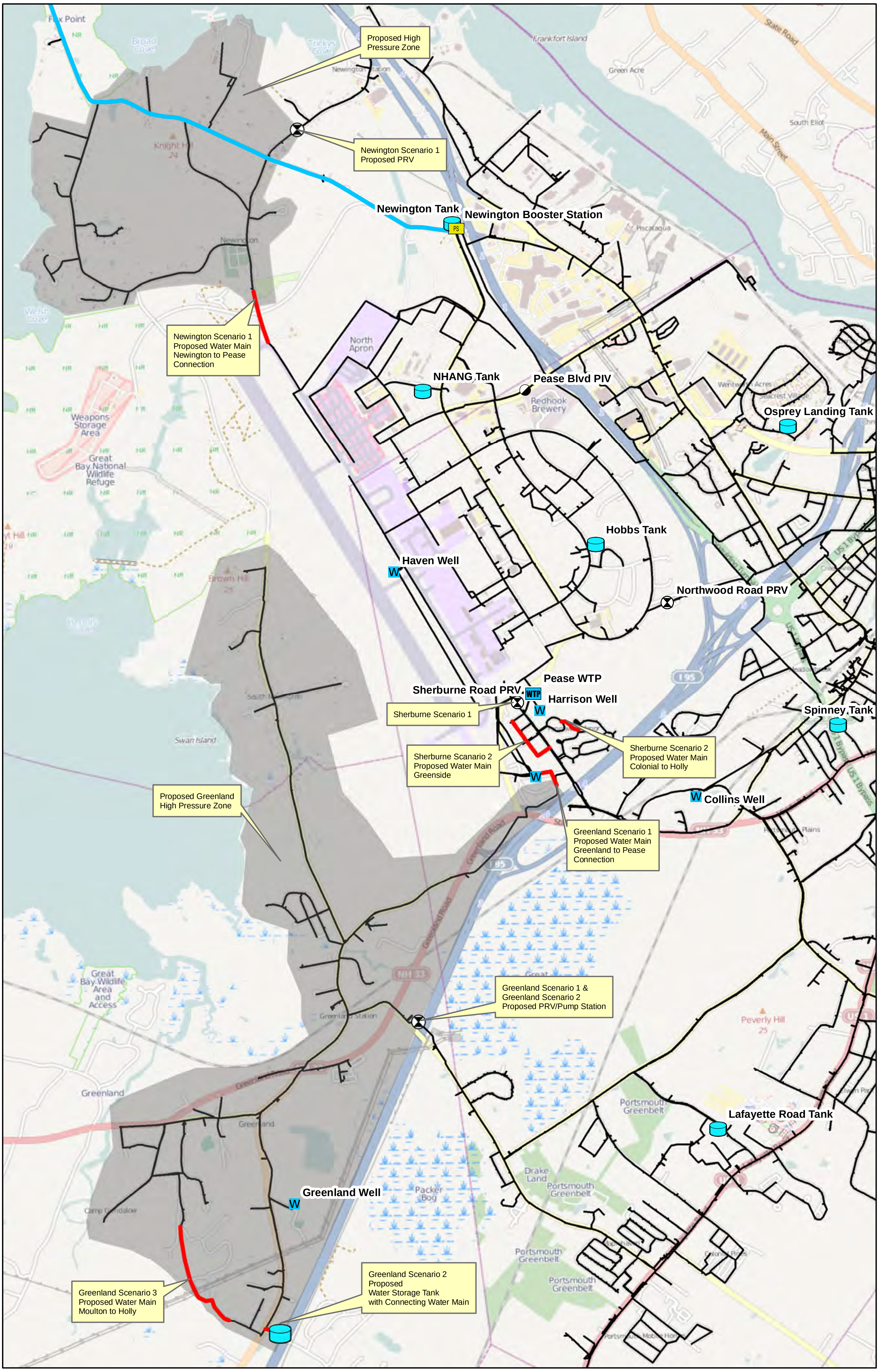

Proposed Pressure Area
Proposed Water Main
Existing Water Main
PS Pump Station
W Well
Tank
WTP WTP
PRV
PIV

Figure 3-17
Greenland, Pease, and Newington Water System Alternatives

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Portsmouth, New Hampshire
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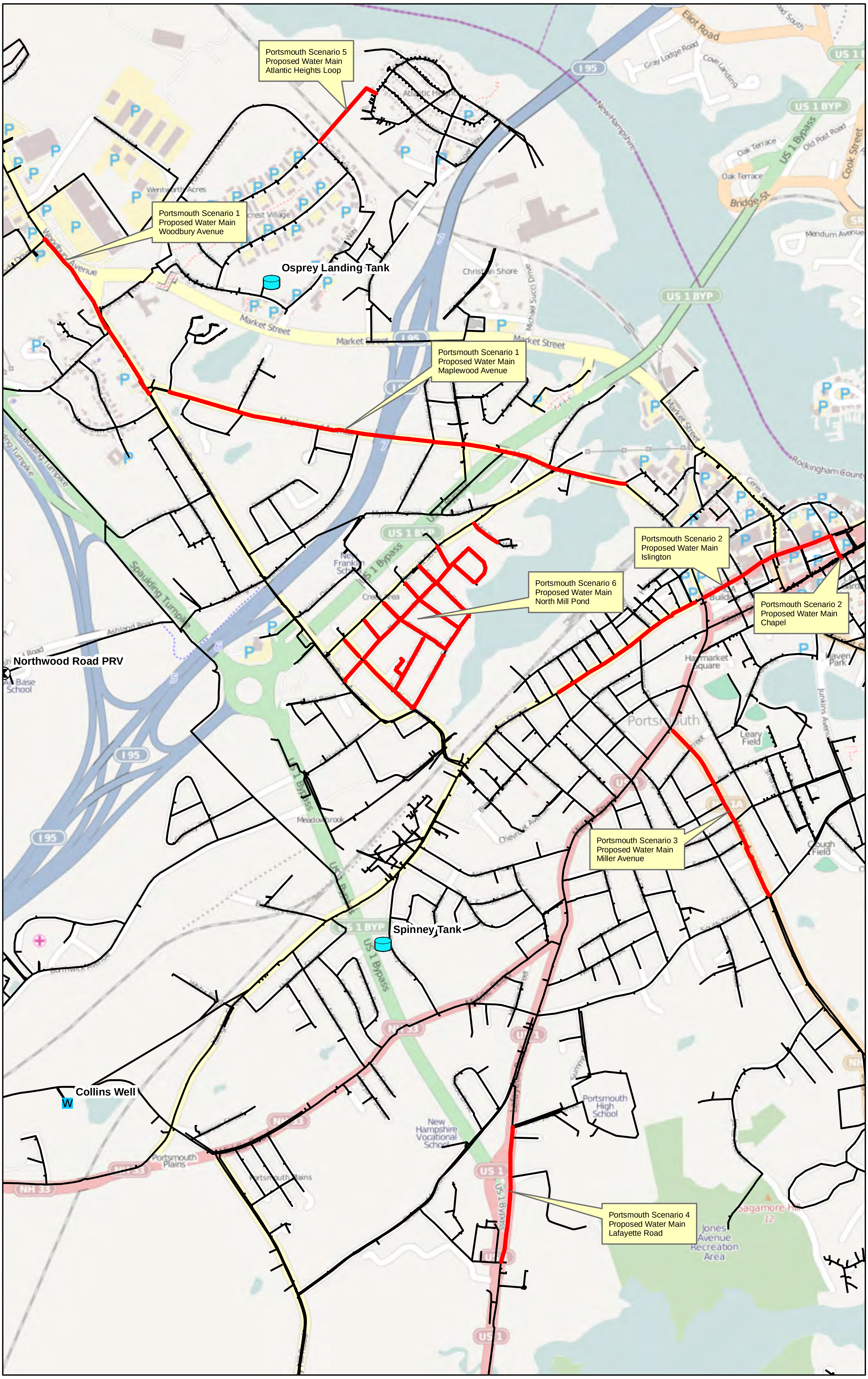

Existing Water Main

Proposed Water Main

PS Pump Station

W Well

Tank

PRV

Figure 3-18
Portsmouth Water System Alternatives

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Portsmouth Water System
Portsmouth, New Hampshire
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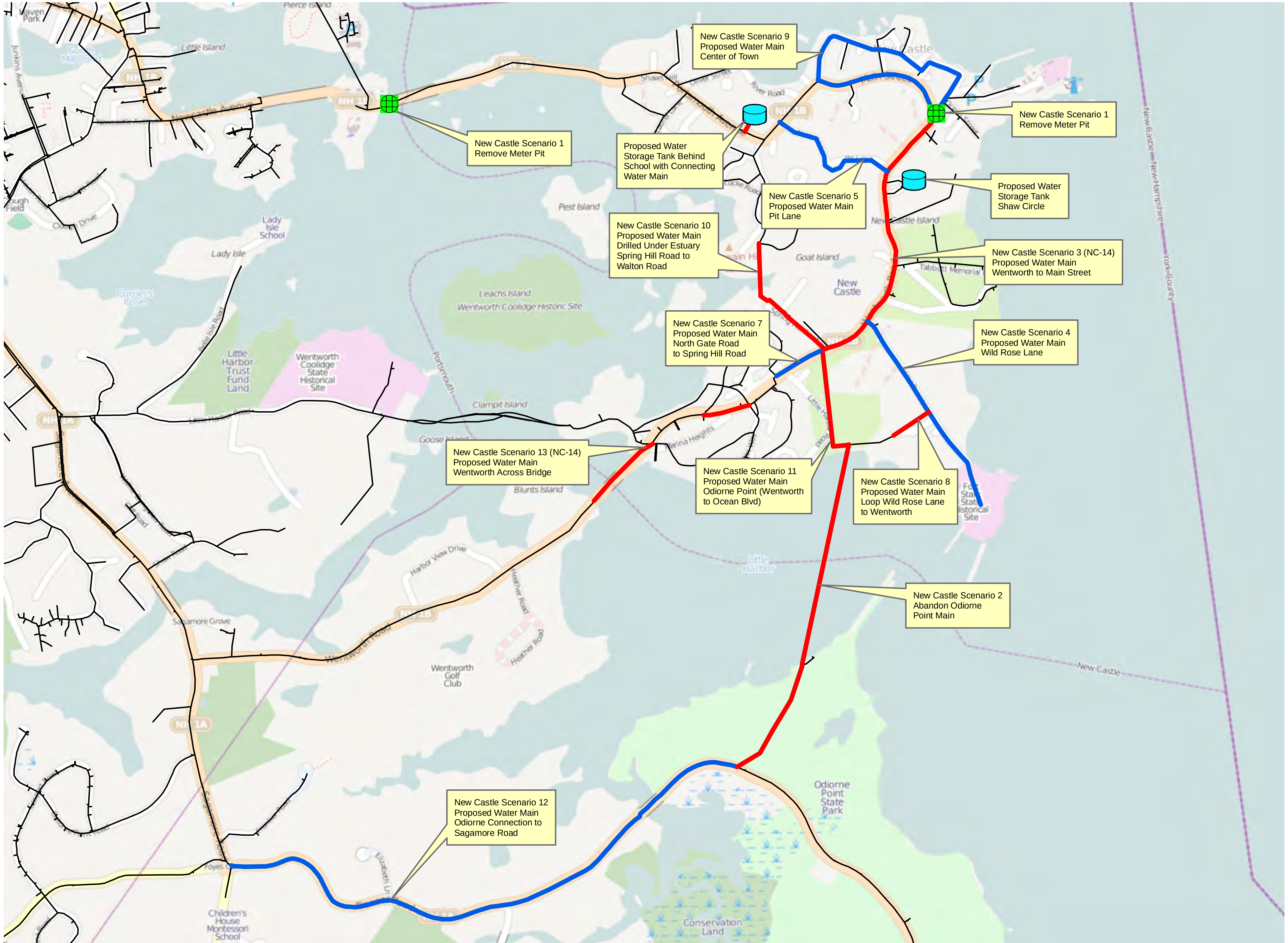
Figure 3-20
New Castle Water System Alternatives



Water Main

Water Main Improvement

Pump Station

Well

Tank

PRV

Remove Meter Pit

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Portsmouth, New Hampshire
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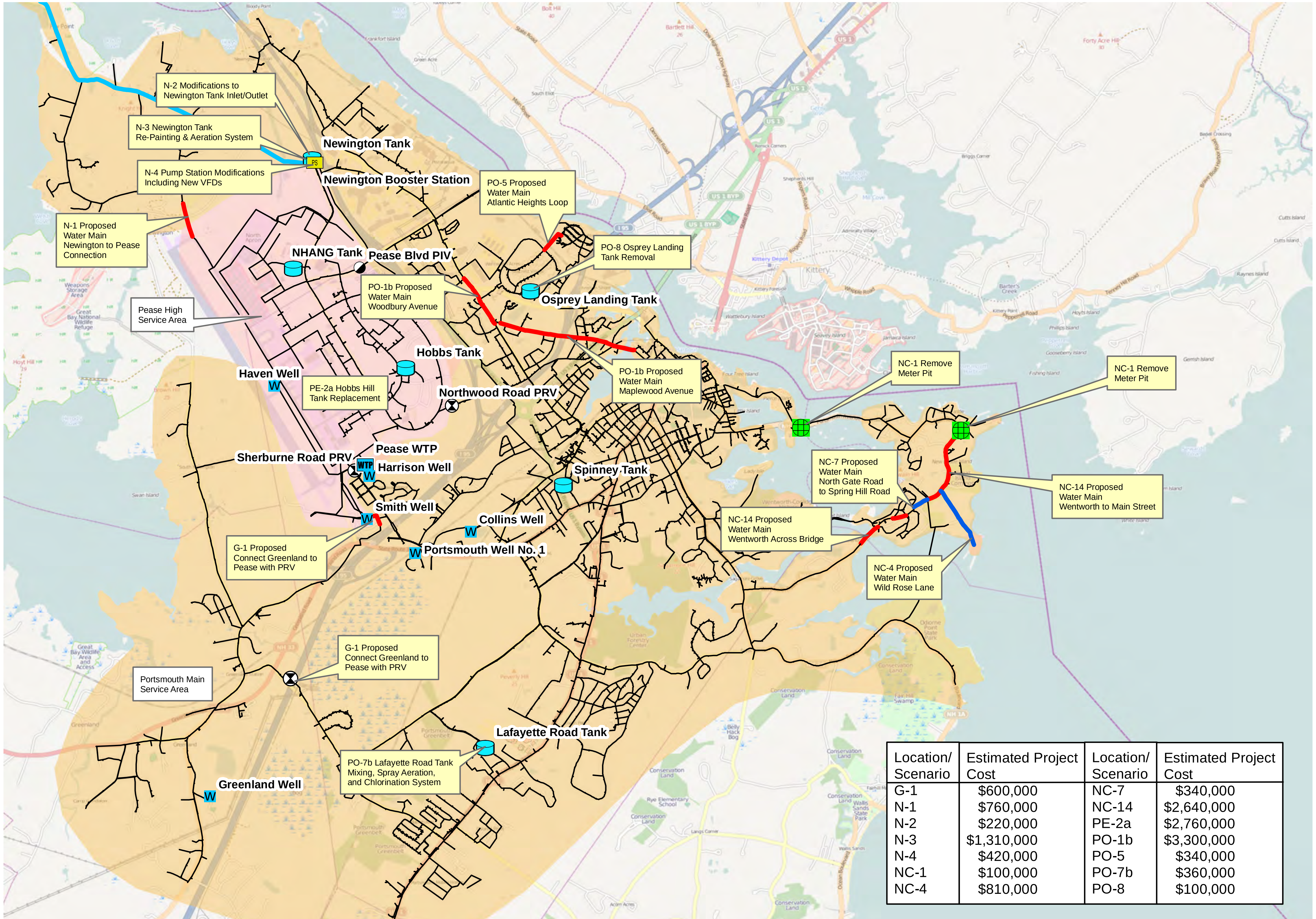
Figure 4-1
Recommended Water Distribution Projects



Location/ Scenario	Estimated Project Cost	Location/ Scenario	Estimated Project Cost
G-1	$600,000	NC-7	$340,000
N-1	$760,000	NC-14	$2,640,000
N-2	$220,000	PE-2a	$2,760,000
N-3	$1,310,000	PO-1b	$3,300,000
N-4	$420,000	PO-5	$340,000
NC-1	$100,000	PO-7b	$360,000
NC-4	$810,000	PO-8	$100,000

Proposed Water Main
Water Main
Pump Station
Well
Tank
PRV
PIV
WTP

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Portsmouth, New Hampshire
February 2013

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