February 7, 2014

Mr. Terrance J. Sullivan
Administrator of Community Utilities
City of Fall River
One Government Center
Fall River, MA 02722

Subject: Final Water System Master Plan

Dear Mr. Sullivan:

FAY, SPOFFORD & THORNDIKE, INC. is pleased to submit the Final Water System Master Plan. We have evaluated previous reports and studies, completed hydrant flow tests, calibrated the computer model of the distribution system, evaluated the adequacy of water supply, treatment and distribution facilities, and developed a master plan prioritizing needed water system improvements.

We are available at your convenience to discuss the findings and recommendations. If you have any questions, please do not hesitate to call me at (781) 221-1266.

Sincerely,

Fay, Spofford & Thorndike, Inc.
By,

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Principal Engineer

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FALL RIVER WATER DEPARTMENT

FALL RIVER, MA

WATER SYSTEM MASTER PLAN

FEBRUARY 2014

FAY, SPOFFORD & THORNDIKE

BURLINGTON, MA

ACKNOWLEDGEMENTS

FAY, SPOFFORD & THORNDIKE, INC greatly appreciates those who assisted with the gathering of operational and historical water use and quality data, field testing that was instrumental to the development of this report, as well as those individuals that provided technical assistance. In particular we would like to thank the following individuals:

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1.1 INTRODUCTION

Water suppliers face ever increasing challenges in terms of quantity and quality of water required to meet customer expectations. Unknown to the majority of the customer base, water supply is a limited commodity and is becoming more difficult to find as a result of competition with environmental, recreational, industrial, private development and other stakeholders. Water suppliers must constantly maintain and upgrade aged infrastructure and treatment processes to meet the ever changing and increasingly stringent Safe Drinking Water Act regulation as well as aesthetic water quality requirements, while at the same time providing water that is reasonable from an economic standpoint.

By considering present and future demands, the adequacy of the water supply, treatment and distribution system, and priority of needs and costs of improvements, a capital improvement plan (CIP) can be developed. The CIP will provide a valuable planning tool with which the City can establish a phased program of improvements which will provide for adequate water supply, treatment and distribution facilities well into the future. By establishing a phased improvements program over several years, the CIP can provide a tool for financial planning so the City can plan to complete a portion of the recommended improvements each year, as funds permit and priority demand.

This report presents the results of our field tests and investigations, computer model hydraulic analyses, calculations and studies of the City's water supply, treatment and distribution facilities. The results were obtained by collecting and reviewing previous reports as well as various data provided by the City and local agencies including population, zoning, water production, water usage and water quality. The steps involved in developing this report included the following:

- Reviewing previous studies and reports
- Reviewing and analyzing existing data
- Updating demand projections through the year 2035
- Updating and calibrating a hydraulic model using WaterCAD software
- Analyzing the water supply and distribution system
- Identifying system deficiencies
- Evaluating alternatives
- Estimating costs
- Developing a prioritized capital improvement plan

A summary of our detailed investigations are outlined in the following three major sections:

- Water Supply
- Treatment
- Distribution

Findings and recommendations for each major section along with a recommended capital improvement program with estimated costs are presented below.

1.2 HISTORY

In the early 1870's the textile industry in the United States was on the rise and the City of Fall River experienced a dramatic industrial expansion as mills were moving in and growing and new corporations were being founded. Population increased exponentially. This was when the City of Fall River began providing drinking water from the Watuppa Pond Complex to the public in 1874. Over 140 years ago, the 1873 Pump Station, the 1875 Standpipe Tower and almost 50 miles of water mains were constructed to supply this rapidly growing community. These historical structures still stand to this day and are both on the National Register of Historic Places. In those early days, the water demand of the City was 1.5 million gallons of water per day (MGD).

At the turn of the century and through the First World War, the textile and cotton industries grew. The City's population and its need for safe and reliable drinking water and fire protection grew as well. Despite several devastating mill fires in the 1920's, the industry managed to survive and adjust to the changing demands of the city, including being a major contributor to the manufacturing effort to support World War II. The City continued to expand its water system to keep up with the growing needs of its community. From 1929 through 1945 the City constructed five water storage facilities, totaling over 7 million gallons (MG) of finished water storage, strategically located at the highest elevations around the City. In the 1960's the City added two more storage facilities increasing the total storage volume of the system to 22 MG.

In the early 1970's, the circa-1870 water mains on Bedford Street that carry water from the original water supply structures into the City were cleaned and cement lined to eliminate the 100 years of built-up tuberculation and to restore them to their original carrying capacity. In 1976, the City completed the construction of its 24 MGD Water Treatment Plant on the western shore of North Watuppa Pond, to safely treat the water coming from the City's public water supply.

In 1999 FST prepared a Waterworks Facilities Master Plan for the City. It recommended that an Infrastructure Rehabilitation Program (IRP) be developed for the replacement of the remaining 12-inch and smaller diameter mains and the rehabilitation of its storage facilities. The 1999 report also recommended further studies and reports on the water supply and treatment systems.

In 2002 FST prepared a Waterworks Facilities Master Plan update for the City which provided the City with a 20-year recommended improvement plan to address the "multi-barrier" approach of watershed protection, water treatment and distribution system maintenance necessary to meet current and upcoming regulations for the protection of public health.

With funding assistance from the Massachusetts Drinking Water State Revolving Fund and the City Street Improvement Program, the City has replaced over 55 miles of cast iron water mains and reduced the number of lead services in the distribution system from over 4,800 to less than 1,000. Since 2007, the City replaced two and rehabilitated four of its seven water storage facilities. The assessment and rehabilitation or replacement of these storage facilities was a focus of the Massachusetts Department of Environmental Protection (DEP) in its Annual Sanitary Survey. Currently in design is the replacement of the existing Airport Road tank in the Fall River Industrial Park with a new elevated tank at a higher elevation as part of new high service area to increase working pressures in the northern part of the City.

Today the City's Water Division supplies an average of 10 million gallons of purified water to the City and surrounding towns, with peak capacity exceeding 20 million gallons.

1.3 WATER SUPPLY

Population within the City has been decreasing since the 1920's. Similarly, water demand within the City has also been decreasing. In the early 2000s water demand was at or above 12 million gallons per day (MGD) while in 2012 the average day demand was below 10 MGD. Average day demand conditions are anticipated to remain between 10 and 11 MGD throughout the 20-year planning period.

In addition to the master meter at the source of supply measuring total flow supplied into the distribution system, the City has approximately 19,809 water use meters installed within the City. In order to understand whether these meters are accurately measuring water produced and used in the City, FST recommends a water audit be completed. Included as part of this water audit, water meter records should be accurately classified to match the reporting categories included in the DEP's annual statistical report. This will increase the accuracy of these reports and the calculated percent of unaccounted-for water or water lost due to leaking water mains (the City is slightly above the performance level of 10 % unaccounted-for water).

The estimated safe yield of North Watuppa Pond and Copicut Reservoir and the WMA Registration Statements adequately meet the water supply requirements for the system. Historically, over the last eleven years the average day demand has been about 11.4 MGD, about 3 MGD lower than the WMA registrations and also lower than the reported 14.5 to 15.5 MGD safe yield of both supplies. The projected 2035 average day demand of 10.7 MGD, which assumes a steady demand by the existing interconnected neighboring communities and no additional major water users in the planning period, can be adequately supplied with the existing sources of supply.

If the City plans to wholesale additional water to other municipalities, there is another 4 to 5 MGD on average of available water supply that could be provided within the City or to other municipalities, and stay below the safe yield and WMA limits. The use of South Watuppa Pond as an additional source of supply will most likely result in degradation of water quality in North Watuppa Pond. Based on projections, the supply from North Watuppa Pond and Copicut Reservoir are more than adequate and the use of South Watuppa Pond as an additional source is not recommended. The future use of South Watuppa Pond may be considered pending proper management of residential industrial areas, as well as additional treatment processes at the water treatment plant if water demands increase above the capabilities of North Watuppa Pond and Copicut Reservoir.

Each source of supply includes a dam to impound the water and the City owns extensive lands within the watershed. Dam inspections were completed in 2011 and the City is addressing these recommendations. Similarly, with the many buildings located within the watershed limits, upgrades to these facilities are also required. Table 1-1 summarizes the costs associated with the water supply recommendations.

TABLE 1-1 WATER SUPPLY RECOMMENDATIONS

Item No.	Improvement Recommendation	Estimated Cost
1	Conduct Water Audit including updates to meter records	$30,000
2	Conduct Safe Yield Study	$40,000
3	Conduct Feasibility and Pilot Study for treatment at South Watuppa Pond	$400,000
4	Upgrades to Copicut Transfer Station	$250,000
5	Watershed Facilities Improvements	$750,000
6	Dam Improvements	$1,725,000
7	Update Watershed Protection Plan to conform to MADEP Guidelines	$50,000
8	Evaluate Raw Water Intake	$30,000
	CAPITAL COST TOTAL	**$3,275,000**

1.4 TREATMENT

The entire water demand of the City of Fall River is supplied through the Water Treatment Plant (WTP) located on North Watuppa Pond at 1831 Bedford Street in Fall River, Massachusetts. The purpose of the water treatment plant is to be able to meet the maximum day demand with the largest pump out of service, protect the public health and maintain aesthetic water quality goals. The WTP was originally constructed in 1975-1976 with a reported capacity of approximately 26 MGD. In 2004 the WTP underwent a series of upgrades. These upgrades included:

- Upgrading of the automatic backwashing (ABW) filter system to include 16-inches of dual media (anthracite over sand), filter to waste capabilities, and an improved control system.
- Installation of new chain and flight solids removal system in each sedimentation basin.
- Installation of a carbon dioxide feed system for pH and alkalinity adjustment.
- Installation of an aqueous ammonia feed system for the future use of chloramines for secondary disinfection (not on-line).
- Installation of variable frequency drives on the raw water pumps.
- Installation of a new roofing system over the pump room, chemical feed room and administrative areas.

- Installation of a new Supervisory Control and Data Acquisition (SCADA) system for the WTP processes.
- In 2012, new switchgear and motor control center (MCC) for the finished water pumps were installed.

Raw water quality entering the WTP is typical of soft, low turbidity New England surface water supplies summarized in the following table:

TABLE 1-2 RAW WATER QUALITY

Parameter	Min.	Max.	Avg.
Color (CU)	0	78	19
Turbidity (NTU)	0.08	4.20	1.00
Total Organic Carbon, TOC (mg/L)	1.6	4.0	3.0
pH	5.68	7.61	6.72
Alkalinity (mg/L as $CaCO_3$)	1.0	9.5	3.3

The WTP utilizes the conventional water treatment process including: coagulation utilizing polyaluminum chloride, sodium hydroxide and carbonic acid for coagulation pH and alkalinity adjustment, flocculation, sedimentation, and shallow bed filtration through automatic backwash filters (ABW). Primary and secondary disinfection is performed utilizing chlorine gas. Sodium hydroxide is added prior to the clearwell for finished water pH adjustment. Sodium silicafluoride is added for dental health.

1.4.1 Safe Drinking Water Act

The quality of a potable water supply is subject to Safe Drinking Water Act (SDWA) regulations established by the Environmental Protection Agency (EPA). The SDWA was originally passed in 1974 and was revised in 1986 and again in 1996.

Total Coliform Rule (TCR)

Total coliform is an indicator parameter that measures for the potential presence of bacteria in a water distribution system. The City most recently violated the TCR in July of 2008. During July 2008 one sample tested positive for e.coli but all repeat and upstream samples tested negative for both E.coli and total coliform. While this was a violation of the TCR, this positive sample may have been inadvertently contaminated resulting in a positive. The City has maintained compliance with the TCR since this last violation.

Lead and Copper Rule (LCR)

Lead and copper sampling is conducted on residential houses to determine if treated water in the City's distribution system is corrosive to lead and copper present in customer service lines and residential plumbing systems. Since the City has demonstrated that the Action Levels have not been exceeded for three consecutive years of "standard monitoring", it now conducts "reduced monitoring," sampling once every 3 years.

Stage 1 and 2 Disinfectants & Disinfection By-products (D/DBP) Rule

Disinfection by-products are formed when the organics in the raw water combine with the disinfectant (chlorine) added to protect the water from waterborne pathogens and from potential bacteria growth in the distribution system. Disinfection by-products have potential to have adverse health effects if consumed over a long period of time and are therefore limited by EPA. The Stage 1 D/DBP Rule includes maximum contaminant levels (MCLs) for specific disinfection by-products including total trihalomethanes (TTHMs), haloacetic acids (HAA5), bromate, and chlorite.

Under the Stage 1 D/DBP Rule, the MCLs are based on a system-wide running annual average of four samples per quarter. The Stage 2 D/DBP Rule maintains the DBP MCLs outlined in the Stage 1 Rule however compliance is based on the Locational Running Annual Average at each individual sampling location identified in the Initial Distribution System Evaluation (IDSE) instead of averaging all of the sample locations together.

Historic TTHM and HAA5 concentrations in the Fall River distribution show that the City maintained compliance with the Stage 1 D/DBP Rule for both TTHMs and HAA5s. The first compliance calculation for locational running annual average was performed in August 2013 for the Stage 2 D/DBP Rule. The 691 Airport Road location had the highest concentration of TTHMs at 84.0 ug/L, exceeding the 80 ug/L MCL. These data indicate that the City has NOT maintained compliance with the Stage 2 D/DBP Rule. Specifically the City has exceeded the TTHM MCL of 80 ug/L at the 691 Airport Road sampling location.

In an effort to address the TTHM issues the City is beginning a pilot program in the Fall River Industrial Park, with the construction of a new high service area. The construction will consist of a new pumping station that will boost water to a new elevated storage tank through the existing distribution mains in the Industrial Park. The new water storage tank will include an aeration and mixing system to keep the water in the tank fully mixed and be capable of removing disinfection by-products (DBPs) from the water. With this aeration and mixing system, the water being supplied by the storage tank will have lower levels of DBPs and will help to provide high quality water to this section of the City and to the Freetown meter in Innovation Way.

Enhanced Coagulation

The Surface Water Treatment Rule requires all surface water treatment facilities utilizing conventional treatment like Fall River to provide a specified level of Total Organic Carbon (TOC) removal. Combining chlorine for disinfection with naturally occurring organic carbon in the water forms disinfection byproducts such as TTHMs and HAA5s, by reducing TOC, the potential for DBP formation is also reduced. Compliance with TOC removal is based on a running annual average (RAA), computed monthly, of raw water TOC concentration and raw water alkalinity. TOC removal data indicates the WTP has regularly been able to achieve greater than the required 35% removal of TOC.

In summary, the WTP has been able to meet all the primary state and federal drinking water quality regulations with exception of the Stage 2 D/DBP.

1.4.2 Performance Limiting Factors

Performance limiting factors in the areas of design, operation and maintenance were identified based on performance and design assessments and special studies conducted at the treatment plant as well as information obtained from plant operations staff. The factors identified were prioritized (A, B or C) as to their relative impact on performance and are summarized with estimated construction costs in the following Table 1-3.

TABLE 1-3 WATER TREATMENT RECOMMENDATIONS

1	Chemical Feed Relocation	$0
2	SCADA System and Process Control Equipment	Software $15,000 Hardware $35,000 TOTAL $50,000
3	Telemetry Upgrade	$250,000
4	Annual Budget to Replace Aging Equipment/Maintain Building	$200,000/yr
5	Filter Media Replacement (materials, delivery, and installation)	$125,000/10 yr
6	Turbidimeters and Monitoring Equipment	$5,000
7	Rehabilitation of Low Lift Pumps & new VFD's	$150,000
8	Refurbish or Replace Three High Lift Finish Water Pumps	$325,000
9	Training for Plant Personnel	$10,000/yr
10	Residual Disposal and Management	Residuals Study $50,000 Lift Station $750,000
11	Backwash Segregation and Recycling	$2,000,000
12	Emergency Reserve Fund	$350,000
13	Convert from Gas to Liquid Chlorine	$600,000
14	Electrical Systems Upgrade	$125,000
	CAPITAL COST TOTAL	$4,600,000

1.4.3 Additional Alternatives & Technologies

The relatively minor treatment plant modifications listed previously will help to optimize treatment plant performance by eliminating or reducing the severity of the performance limiting factors identified during the facility investigations. In addition to making the modifications to optimize plant performance, several additional alternatives were identified to:

- increase protection from pathogenic microorganisms such as C*ryptosporidium*,
- increase removal of natural organic matter to minimize formation of disinfection by-products following chlorination
- minimize color, taste and odor in the finished water
- allow plant staff more flexibility with the general operation of the facility

A summary of the costs as well as the advantages and disadvantages of each of the alternatives identified is summarized below.

TABLE 1-4 COST OF FILTRATION AND ENHANCED TREATMENT ALTERNATIVES

Alternative	Capital Cost (million $)	Additional Annual O&M Cost	Present Worth
Ozonation	$5.6	$200,000	$9,100,000
Membrane Filtration	$22.2	$500,000	$30,100,000
Nanofiltration	$30	$2,400,000	$72,000,000
Intermediate Pumping and Deep Bed Filtration (Conventional or GAC)	$18.0	$110,000 (intermediate pumping & blowers) $60,000 (multi-media replacement) $600,000 (GAC replacement)	Incl. below $21,000,000 $30,500,000
Chloramination	$0	$14,000	
Ultraviolet Disinfection	$2.1	$100,000	$3,800,000
Intermediate Pumping, Ozonation, GAC and UV	$25.7	$110,000 (intermediate pumping & blowers) $200,000 (ozonation) $400,000 (GAC replacement) $100,000 (UV disinfection) $810,000 TOTAL	$40,000,000
MIEX	$8	$450,000	$16,000,000
MIEX & Membranes	$28.2	$950,000	$45,000,000

The present worth costs are based on an inflation rate of 3% and a 4% interest rate over a 20 year period.

TABLE 1-5 ADVANTAGES/DISADVANTAGES OF ALTERNATIVES

Alternative	Advantages	Disadvantages
Ozonation	Reduction of organics Reduction of taste and odor Additional microbial reduction	Moderate capital cost Moderate O&M cost
Membrane Filtration	Excellent microbial reduction	High capital cost High O&M cost Requires additional flow for backpulsing Increased residuals
Nanofiltration	Reduction of organics Excellent microbial reduction	High capital cost High O&M cost Requires intermediate pumping Increased residuals
Intermediate Pumping and Deep Bed Filtration	Increased microbial and solids removal Lower effluent turbidities DBP reduction with GAC Taste and odor reduction with GAC	High capital cost High O&M cost Requires intermediate pumping Potential permitting issues
Chloramination	Reduction of DBPs Longer residence times Already has been installed at the WTP	Potential release in lead Nitrifying bacteria in distribution system Public education about chloramines
Ultraviolet Disinfection	Excellent microbial reduction Will reduce chlorine dose required in clearwell Low capital cost Low O&M cost	Primarily used for surface water treatment as a second disinfectant to ozonation for unfiltered supplies
Ozonation, GAC and UV	Excellent microbial reduction Lower effluent turbidities Maximum reduction of organics Maximum reduction of taste and odor	High capital cost High O&M cost UV not currently approved for surface water treatment Required intermediate pumping Potential permitting issues
MIEX	Reduction of organics and lower DBPs	Additional brine waste stream
MIEX & Membranes	Reduction of organics and lower DBPs Excellent microbial reduction	High capital cost High O&M cost Requires additional flow for backpulsing Increased residuals Additional brine waste stream

1.5 DISTRIBUTION

The City of Fall River's water distribution system includes approximately 216 miles of water main, one booster pump station and seven (7) storage tanks, totaling over 20 million gallons of finished water storage. These storage tanks provide adequate storage to meet daily demand fluctuations in addition to the required fire flow storage volumes.

Finished drinking water exits the water treatment plant through a single 36-inch diameter cement-lined ductile iron transmission line that connects to the "backbone" of the distribution system beginning at the bottom of the Bedford Street hill. Bedford Street, which runs from Watuppa Pond, east to west through the center of the City, has three (3) cement-lined cast iron water mains - 36-inch, a 24-inch, and a 20-inch diameter and forms the backbone of the distribution system. The City's overall water distribution system is relatively well-looped in its interior, with some dead ends towards the extremities of the system. In some of these areas, single mains run from the main distribution system to the extremities of the system without a redundant supply main. Additionally the City's major 36-inch transmission line lacks redundancy from the water treatment plant (WTP) to Bedford Street and there are no other major transmission lines feeding the distribution system outside of Bedford Street. The City should investigate an underground redundant line from the water treatment plant.

The City's water distribution system is divided into to two services areas a lower elevation area and a higher elevation area. The majority of the City is provided with water from the low service system at a hydraulic gradeline elevation (HGL) of 318 feet. Due to the City's varying topography, many areas at higher elevations experience low working pressures. To increase pressures for residents at some of the higher elevations, the City recently established a high service area along South Main Street. The Townsend Hill High Service Area provides water at an HGL of 365 feet to the southern part of the City. Another part of the City that experiences low pressures is the Fall River Industrial Park in the northern part of the City off North Main Street. Currently in design is the proposed Airport Road High Service Area that will provide water to the customers in and around the Industrial Park at an HGL of 368 feet.

There are other areas in the City that experience low water pressure due to high ground elevation, most notably, the Flat Iron, Charlton Hospital, and Chicago Street. These are relatively small areas, isolated from each other and the established high service areas. They are located in central City locations, rather than at the extremities of the system like Townsend Hill and the Industrial Park. Creating high service areas to serve these locations is not impossible but it is more complicated hydraulically and results in an unfavorable amount of dead end water mains. The City should consider further evaluations for better serving the customers in these locations including the installation of individual booster pumps, creating additional high service areas, or expanding the existing high service areas.

1.5.1 Storage Tank Operation and Maintenance

Maintaining fire protection and water quality have competing goals in terms of the recommended range in operating levels of storage tanks. To maximize fire protection the storage tank water level should be maintained as high as possible. However, to maintain chlorine residual for the protection of water quality, the entire volume of water in the storage tanks should “turn-over” every 5 days, or 20 percent every day. Based on a review of the City’s water storage tank data, most tanks fluctuate approximately 5 to 10 percent. The City should continue their efforts to maximize water quality in the distribution system.

Maintaining clean water in storage tanks is a key component to maintaining distribution system water quality. In recent years, six of the seven water storage facilities have either been replaced or rehabilitated, and the design of the replacement of the seventh is underway. While this work is recent, these water storage facilities should be inspected every three to five years to ensure continued, reliable service. Future rehabilitation projects should continue to be budgeted for. Recoating these tanks in 15-20 years will help the City to protect the capital investments recently completed.

1.5.2 Pipe Replacement and Maintenance

As a result of the Infrastructure Rehabilitation Plan (IRP) developed by FST in 1999, and assistance from the City Street Improvement Program, the City has replaced approximately 55 miles of cast iron water main, and has 45 more miles to go. To assist the City with the continuation of their infrastructure replacement program, FST has developed a prioritized list for the remaining cast iron and AC water mains.

FST also recommends that the City conduct a leak detection program at least once every two years. Not only can the leak detection program identify leaks in the distribution system located on mains, at fire hydrants or services, leaking valves or fittings, but it can also assist with prioritizing future pipeline replacements.

The City currently completes targeted distribution flushing when discolored water complaints are received. The City should continue annual water distribution system flushing. In addition to distribution system flushing, a valve exercising program should also be implemented.

1.5.3 Water Meters

An annual meter replacement will assist with minimizing unaccounted-for water by accurately measuring the water consumed at each service connection. While replacing or right-sizing commercial and industrial meters can capture additional water used at these facilities, replacing a portion of the smaller residential meters on an annual basis will contribute to having accurate and operable meters at all residential connections.

1.5.4 Water Distribution Maintenance Area

As part of the water system operations, there are a number of facilities and areas that require maintenance. Many of the buildings are located out of sight of the general public and general maintenance practices have been limited due to funding. Some buildings could be eligible for registration as historic structures and could be re-purposed for water department operations. The City has been working with historical architects to stabilize and rehabilitate the 1873 pump station, the 1875 tower and the 2929 Blossom Road Watuppa Reservation Headquarters. The

City should continue to pursue this specialized assistance, in the interest of renovating and preserving these historical facilities.

Both sides of Bedford Street, between the Water Department building at #1620 and the Route 24 overpass, had long been used by the City as disposal areas for cobblestones and other construction related debris. In the interest of further protecting and improving the watershed of Watuppa Pond, the plan for the removal and disposal of the cobblestones and other construction related debris and site restoration for the north side of Bedford Street should continue and a plan for the south side should be developed and implemented.

The Water Department currently has multiple antiquated or obsolete buildings and facilities on both sides of Bedford Street, as well as the access road to the Water Treatment Plant. The structures, buildings and garage space are in need of repair or replacement and there is inadequate indoor storage space for materials, equipment and vehicles. The City should consider an evaluation of the buildings and maintenance facilities.

TABLE 1-6 WATER DISTRIBUTION RECOMMENDATIONS

Item No.	Improvement Recommendation	Estimated Cost
1	Redundant Transmission Line from WTP	$3,000,000
2	Annual Pipeline Replacement Program	$1,200,000 to $5,000,000 per year
3	Paving allowance for annual pipe replacement program	$950,000 to $3,000,000 per year
4	Cleaning and Painting of Storage Tanks (every 15-20 yrs)	$8,500,000
5	Replace Airport Road tank, construct Commerce Drive Pump Station	$4,000,000
6	Future High Service Area Dedicated Water Main (if necessary)	$500,000
7	Investigation and Improvements for Low Pressure Areas	$4,100,000
8	Perform Leak Detection Survey of approximately 250 miles of water main (every 2 years)	$40,000/2 yrs
9	Allowance for leak repair	$50,000/yr
10	Comprehensive Hydrant Flushing & Valve Exercising Program	$60,000/yr
11	Annual Valve and Hydrant Replacement Allowance	$50,000/yr
12	Storage Facility Inspections (every 3 to 5 years)	$20,000/3yrs
13	Annual Meter Replacement Program (1,000 meters per year)	$300,000/yr
14	Debris Removal along Bedford Street	$1,400,000
15	Distribution Maintenance Area, Buildings, and Structures	$5,200,000
16	Evaluate / Remove 1950 Pump Station and Screen House	$1,000,000
17	Repair and Rehabilitate Structures of Historical Significance	$2,500,000
18	Distribution Vehicle Maintenance and Replacement	$100,000/yr
	CAPITAL COST TOTAL	**$170 million**

1.6 RECOMMENDED IMPROVEMENT PROGRAM

The following program of improvements, when completed, will provide the City of Fall River with adequate water supply, treatment and distribution through the year 2035. An engineer's opinion of probable cost has been presented in the following table for a capital improvement and maintenance program based upon those high priority recommendations noted in the report. These construction costs do not include legal fees, land and easement costs or City administrative costs, but they do include rock excavation, pavement replacement and other items generally involved in a water works construction projects. These estimates do include a 20 percent allowance for engineering and contingencies and all cost estimates are based on present year costs.

Also, estimated costs should be verified with the Engineer prior to allocation of funds for selected project(s) for each fiscal year. In some cases preliminary engineering studies will be required to identify the extent of the work and associated costs.

The tables summarizing the recommended improvements for each of the water system components presented previously have been scheduled to show when the recommended improvements should be implemented over the 20-year planning period. The proposed annual expenditures for the 20-year water system improvement program range from about $6,000,000 to $16,000,000. Table 1-7 summarizes the 20-year capital improvement plan.

TABLE 1-7 20-YEAR CAPITAL IMPROVEMENT PLAN

SECTION 2 - WATER SUPPLY

2.1 WATER REQUIREMENTS

Water requirements of a city are determined based on the population served; domestic, commercial, municipal and industrial usage; unaccounted for usage (leakage, main flushing, fire protection, etc.); and wholesale service. The projected water requirements in this report are calculated through the year 2035.

2.1.1 Population Projections

Evaluation of the City of Fall River's water distribution system must take into account historical, future and present populations. Any increase in population affects the water supply and distribution needs of the system. The following Table 2-1 summarizes the historic population data provided by the United States Census Bureau.

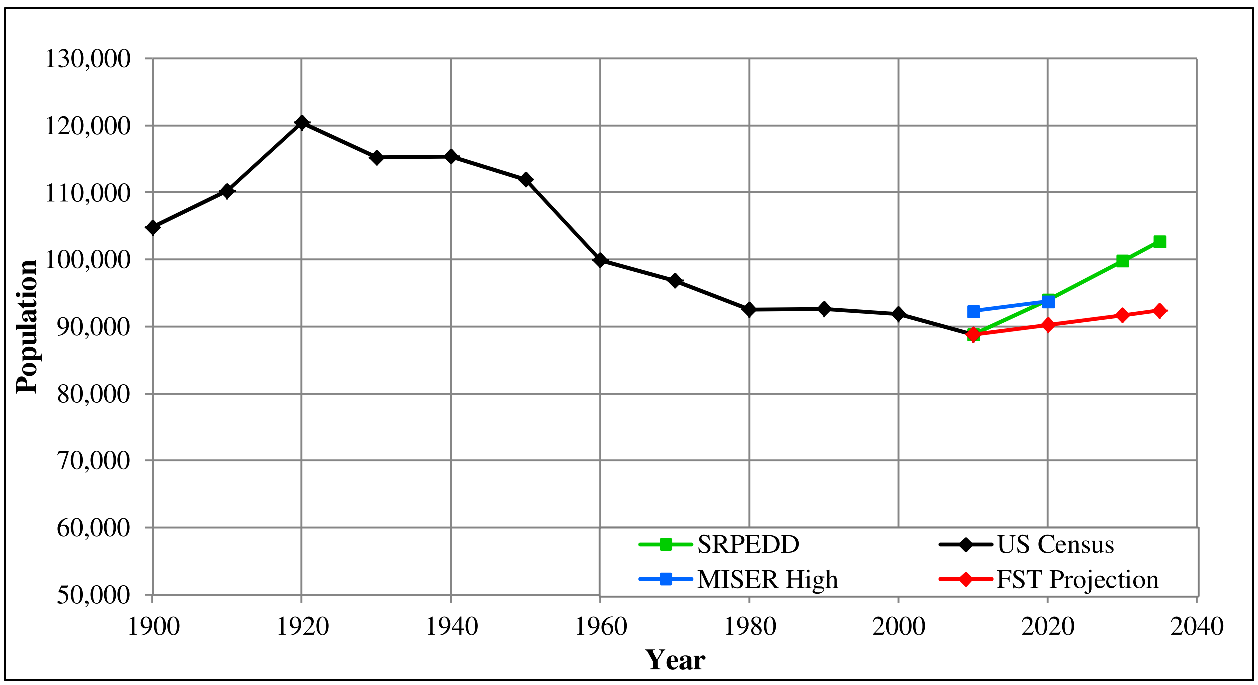
TABLE 2-1 HISTORIC POPULATION - US CENSUS

Year	US Census Population	% Change
1900	104,863	
1910	110,295	4.9%
1920	120,485	8.5%
1930	115,274	-4.5%
1940	115,428	0.1%
1950	111,963	-3.1%
1960	99,942	-12.0%
1970	96,898	-3.1%
1980	92,574	-4.7%
1990	92,703	0.1%
2000	91,938	-0.8%
2010	88,857	-3.5%

According to the United States Census Bureau, from 1960 to 2010 the City population decreased approximately 12.5% from 99,942 down to 88,857. From 1960 to 1980, the City experienced a population decrease of approximately 8.0%. However, in the period of 1980 to 2010 the City experienced a population decrease of approximately 4.2%. This corresponds to an average decrease rate of 0.40% per year and 0.14% per year respectively, indicating a relatively minor downward trend. In the early 1900s, the City was a major manufacturing center for the textile industry. As that industry declined, the population of the City declined as well.

Population projections were obtained from the Southeastern Regional Planning and Economic Development District (SRPEDD) and the Massachusetts Institute for Social and Economic Research (MISER). SRPEDD and MISER projections show an average yearly increase rate of 0.54% and 0.16% respectively. Although previous population trends illustrate a downward trend, both methods of projection predict a rise in overall population. This increase in projected population is largely due to two proposed MBTA Commuter Rail Stations located in downtown Fall River and the waterfront industrial area at Battleship Cove. Additional access to job opportunities in Boston and surrounding communities through the proposed commuter rail will likely boost population surrounding the proposed stations. The population projections presented by SRPEDD show a significant increase in population as a result of these improvements. The MISER projections seem more reasonable based on the previous downward trend for population. FST has chosen the MISER population projections to develop future based water consumption for the City over the planning period. Historical and projected population data are plotted in Figure 2-1.

FIGURE 2-1 POPULATION PROJECTIONS



2.1.2 Water Consumption

A city's projected water consumption is based upon potential for growth, in terms of population as well as industrial expansion. Water consumption may be separated into various classes, as follows:

Domestic:	water used in residences and apartments, for drinking, bathing, sanitation and lawn watering.
Commercial:	water used in restaurants, service stations and retail establishments.
Industrial:	water used in manufacturing and warehousing facilities.
Municipal:	water used by city owned facilities.
Unaccounted for:	water which includes all unmetered uses, such as system leakage, hydrant flow for fire protection and other uses and meter inaccuracy.
Wholesale:	water sold to other municipalities via metered interconnections.

2.1.2.1 Residential Water Consumption

Projection of population served by the City provides the basis for forecasting residential water consumption and assessing water system needs. Since the City provides public water to all its customers, the population of the City in the year 2035 will be equal to the population served by the water distribution system. It is estimated that the population will stay relatively constant with a small increase of about 0.16% per year for the duration of the 25 year planning period. Population data provided by the City is displayed in Table 2-2.

As well as being affected by population, residential water consumption is affected by land use restrictions and water consumption habits. Since it is primarily dependent on the population served, residential water consumption is often expressed in terms of gallons per capita per day (GPCD). Table 2-2 lists residential consumption records, population served and average water consumption per resident from 2002 to 2012. Average daily water consumption per resident is expressed in GPCD and was determined by dividing the total residential consumption in gallons per day (GPD) by the population served.

TABLE 2-2 RESIDENTIAL WATER CONSUMPTION

Year	Gallons*	GPD*	% of Total Production*	Population Served*	GPCD
2002	2,342,404,615	6,417,547	49.5%	91,938	69.8
2003	2,551,430,010	6,990,219	57.9%	91,938	76.0
2004	2,632,697,032	7,212,869	56.0%	91,938	78.5
2005	2,526,526,554	6,921,991	55.7%	91,938	75.3
2006	2,789,260,854	7,641,811	63.9%	91,938	83.1
2007	2,373,132,667	6,501,733	54.2%	91,938	70.7
2008	2,208,150,000	6,049,726	53.5%	91,938	65.8
2009	2,208,400,000	6,050,411	56.3%	91,938	65.8
2010	2,372,895,000	6,501,082	62.4%	88,857	73.2
2011	2,373,895,000	6,503,822	66.9%	88,857	73.2
2012	2,373,895,000	6,503,822	72.5%	88,857	73.2

*Data taken from City of Fall River's Annual Statistical Report

The average water consumption per resident is approximately 73 gallons per capita per day for the 11 years shown in Table 2-2. In projecting residential water requirements, FST calculated the average daily residential water consumption of 6,748,485 gallons per day for the year 2035 by multiplying 73 gallons per capita per day by the population projection of 92,445. Although population has been experiencing a downward trend FST has projected an increase in population based on MISER population projections.

2.1.2.2 Commercial Water Consumption

The best approach for making commercial water consumption projections is the relationship between industrial activity and the population which sustains it. The method used to make these projections is based on the total commercial use as a percentage of the residential use. Based on the historic data provided by the Annual Statistical Reports from 2002-2012, commercial water consumption records show an average use of approximately 9.5% of the total water consumption and 15.0% of the residential consumption. FST has assumed that this percentage will remain constant throughout the planning period. The projected average daily commercial water consumption is 1,017,629 gallons per day for the year 2035.

2.1.2.3 Industrial and Agricultural Water Consumption

Based on Annual Statistical Report (ASR) data, FST has chosen to combine industrial and agricultural usage in the same category. Review of the 2011 and 2012 ASR, a large decrease in agricultural use indicates a relatively insignificant usage. Using the same methodology as described in the previous paragraph, FST has found that the industrial and agricultural consumption is an average of approximately 6.0% of the total consumption and 9.5% of the residential consumption over the historic period. With the expansion of the Industrial Park in the northwest region of Fall River an increase in industrial usage can be expected. However the increase in usage has been accounted for in the projected population increase. For the purposes of the 25 year planning period FST has assumed that these percentages will remain constant. The projected average daily industrial and agricultural water consumption for the year 2035 is 642,713 gallons.

2.1.2.4 Municipal Water Consumption

Municipal water consumption includes the facilities and offices that use water and are owned by the City. Municipal consumption generally accounts for only a small portion of the total demand. Using the same methodology as described in the previous paragraphs, FST has found that municipal consumption is an average of approximately 2.0% of the total consumption and 3.0% of the residential consumption over the historic period. For the purposes of the 25 year planning period it has been assumed that these percentages will remain constant. The projected average daily municipal water consumption is 214,238 gallons per day for the year 2035.

2.1.2.5 Confidently Estimated Municipal Use (CEMU)

Included in the ASRs is a category called Confidently Estimated Municipal Use (CEMU). Water supplies can provide calculations and documentation for unmetered water use such as water used for fire protection and training, water main flushing and flow testing, bleeders and blow offs and sewer and storm drain cleaning. Typically, this CEMU usage has ranged from 2 to 15 percent of the total volume supplied. The Annual Statistical Reports indicate that average CEMU in Fall River is approximately 7.0% of the total water produced or about 11.1% of the residential usage. The projection made for CEMU through the year 2035 is 749,832 gallons per day.

2.1.2.6 Unaccounted-for-Water

Unaccounted-for usage is determined by comparing the value of water passing through the customers' meters with the volume of water supplied to the system. Typically, unaccounted for water usage has ranged from 11 to 20 percent of the total volume supplied. The Annual Statistical Reports indicate that average unaccounted-for-water in Fall River is approximately 10.5% of the total water produced or about 17.0% of the residential usage. The projection made for unaccounted for water through the year 2035 is 1,125,748 gallons per day.

2.1.2.7 Other Public Water Systems

The City also sells water to other public water systems including the Town of Tiverton, Town of Westport, and Freetown Water Commission. The historical data indicates that the water consumed by other public water systems accounted for approximately 2.0% of the total water consumption. Assuming the value of 2.0% will be the same throughout the planning period, the projected water consumption of other public water systems is 214,238 gallons per day through the year 2035.

2.1.3 Total Water Requirements

One of the objectives of this plan is to estimate future water demands and use these estimates to determine if current water system supply, treatment and distribution facilities are adequate. If inadequacies are found, a plan for system improvements will be developed.

Estimated future demands include domestic, commercial, agricultural, industrial, municipal, wholesale, and unaccounted for water usage. Residential demand is dependent on changes in population. Commercial and industrial demand depends on changes in economic development. As population and commercial and industrial activities increase, the amount of water needed increases. By estimating the future residential demand and knowing the percentage of total demand represented by the commercial, industrial, municipal, and unaccounted for water, the future total demand can be calculated.

After determining the future total demand, average daily, maximum daily, and maximum hourly demands can be determined. Average daily demands are used to determine the adequacy of the source of supply. Maximum daily demands are used to determine the adequacy of pumping facilities. Maximum hourly demands and maximum daily demands plus fire flows are used to determine the adequacy of distribution storage facilities, transmission mains and distribution mains.

2.1.3.1 Average Day Demand

Average daily rates of consumption are used to determine the adequacy of the source of supply. The total average daily demand for the years 2002 to 2012 has declined about 3.5% per year. This is likely due to the decrease in population, customers conserving water and a reduction in industrial use. Table 2-3 presents the average daily demand by category of user through the years 2002 to 2012. The overall average daily demand during this period was about 11.4 million gallons per day (MGD).

TABLE 2-3 AVERAGE DAY DEMAND (MGD)

User Category	Average 2002-2012
Residential	6.663
Commercial	1.220
Industrial/Agricultural	1.095
Municipal/Institutional	0.372
Other	0.280
CEMU	0.687
Unaccounted	1.338
Total	11.407

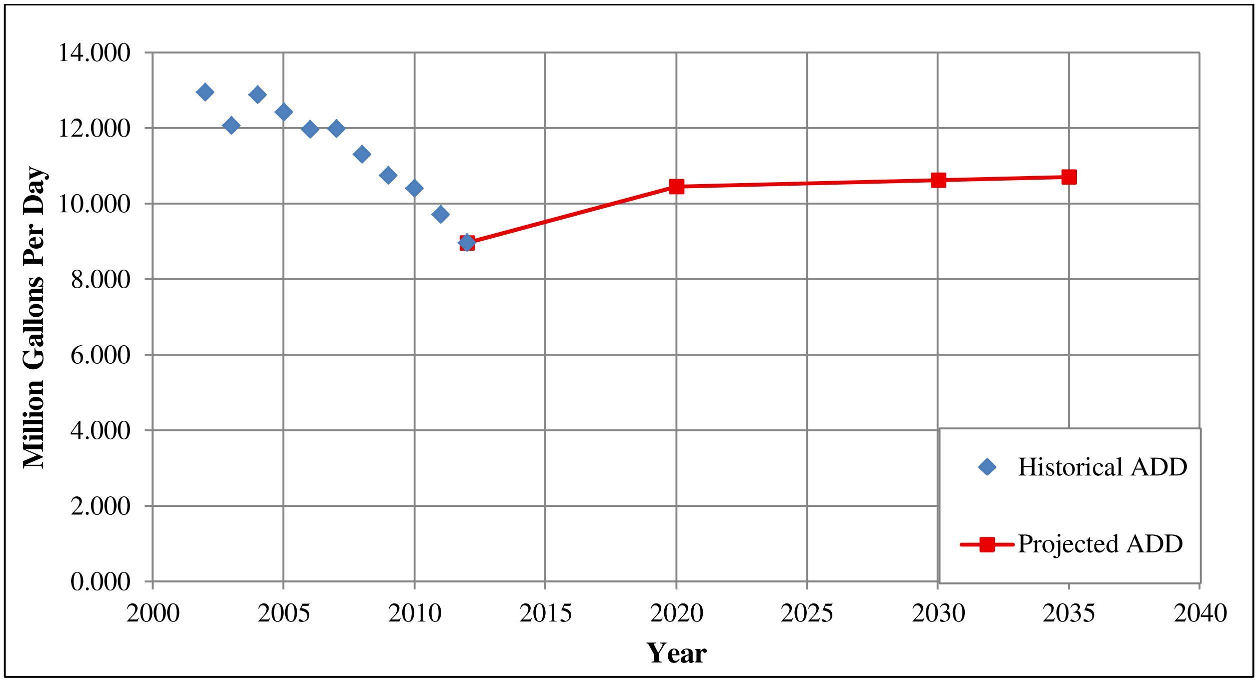
To project future water demands, the residential water use was estimated based on historical residential gallons per capita per day multiplied by the population projections developed for 2020, 2030 and 2035. The remaining water consumption categories were also estimated for the planning period. Projected Average Day Demands through the 2035 planning period are presented in Table 2-4. These water demand projections are based on historical water use and population projections and do not take into account a new large water user moving into the City.

TABLE 2-4 PROJECTED AVERAGE DAY DEMANDS (MGD)

Year	Residential	Commercial	Industrial/ Agricultural	Municipal/ Institutional	Other	Confidently Estimated Municipal Use	Unaccounted for water	Total
2020	6.591	0.994	0.628	0.209	0.209	0.732	1.099	10.462
2030	6.696	1.010	0.638	0.213	0.213	0.744	1.116	10.629
2035	6.748	1.017	0.642	0.214	0.214	0.749	1.124	10.711
% of Total	63.0%	9.5%	6.0%	2.0%	2.0%	7.0%	10.5%	100.0%

Figure 2-2 presents historic and projected demands graphically.

FIGURE 2-2 HISTORIC AND PROJECTED FUTURE AVERAGE DAY DEMAND



2.1.3.2 Maximum Daily Demand

Maximum daily demands are used to determine the adequacy of the supply sources and pumping facilities. Maximum daily demand is the largest volume of water used over a single 24 hour period during the year. It is determined from records and is expressed as a ratio of the average day, typically ranging from 1.4 to 2.5. In addition, this ratio is also a function of the relative importance of each component of the total demand: residential, commercial, municipal, and industrial. The ratio of maximum to average daily consumption is generally higher for residential than for industrial, commercial, and municipal users. Industry normally uses water at a relative constant rate each day. Residential consumers, however, can easily double or triple their average daily consumption by such activities as lawn watering, car washing and swimming pool filling. Water supply statistics on the maximum daily rate of demand are presented in Table 2-5.

TABLE 2-5 MAXIMUM DAY DEMAND

Year	Average Daily Demand (MGD)	Maximum Daily Demand (MGD)	Maximum Day Ratio
2002	12.96	18.13	1.40
2003	12.08	16.64	1.38
2004	12.88	18.03	1.40
2005	12.44	18.23	1.47
2006	11.97	14.95	1.25
2007	11.99	14.28	1.19
2008	11.32	14.77	1.31
2009	10.60	12.92	1.22
2010	10.42	11.56	1.11
2011	9.72	12.05	1.24
2012	8.97	15.33	1.71
Average	11.41	15.17	1.33

Analysis of the City's water consumption records for the last seven years indicates that the maximum daily consumption rates average 1.33 times (or 133% of) the average daily rate. This relatively low multiplier is representative of a community with a higher level of industrial/commercial use, large amount of multifamily homes, and relatively low lawn and recreational water use.

2.1.3.3 Maximum Hourly Demand

Maximum hourly rates of consumption and maximum daily rates of consumption plus fire flows are used to determine the adequacy of distribution storage facilities, transmission mains and distribution mains. The maximum hourly consumption is the maximum volume of water used over a single 60 minute period. Generally, the maximum hourly consumption occurs during the maximum day. Like maximum daily demand, peak hourly consumption is usually projected by calculating the ratio of peak hourly to average daily demand and multiplying it by the projected average daily consumption. Based on pumping records and tank charts for the last two years the peak hour demand is calculated to be 1.51 times (or 151% of) the average daily demand. Table 2-6 presents the projected average day, maximum day, and maximum (peak) hour water consumption rates that have been adopted.

TABLE 2-6 WATER CONSUMPTION RATES (MGD)

	Percent of Annual Daily Average	Year 2020 Demand	Year 2030 Demand	Year 2035 Demand
Average Day	100	10.462	10.629	10.712
Maximum Day	133	13.915	14.136	14.247
Maximum Hour	151	15.798	16.049	16.175

2.2 WATER RESOURCES COMMISSION PERFORMANCE STANDARDS

The Massachusetts Water Resources Commission (WRC) is responsible for developing, coordinating and overseeing the Commonwealth's water policy and planning activities. As part of the Massachusetts Water Conservation Standards, the WRC has identified performance standards to encourage the efficient use of water. These performance standards include maintaining unaccounted for water at 10% or lower and having residential water use at or below 65 gallons per capita per day (RGPCD)

Presented previously in Table 2-2, the City of Fall River's residential water demand has fluctuated from a high of 83 RGPCD in 2006 to a low of 65.8 RGPCD in 2008. While the City has exceeded the performance standard of 65 RGPCD in many years, recently the City is very nearly meeting this requirement. To consistently meet this performance standard the City should consider promoting efficient water use to its residential customers through education on minimizing outdoor water use and promoting the use of water-efficient household appliances. Also, the accuracy of the residential metered water use would be improved by more accurately classifying meter records using the categories in the ASR.

Reviewing historical unaccounted for water (UAW) presented previously, the City's UAW levels have averaged 10.5%. This is only marginally above the performance standard of 10% UAW. To meet this performance standard the City should ensure that it is metering 100% of its water users, implement a water meter repair/replacement program, continue to calibrate the master meters and complete leak detection and leak repairs throughout the distribution system.

2.3 WATER SUPPLY FACILITIES

The City of Fall River (City) has three water supply sources: North Watuppa Pond, South Watuppa Pond and Copicut Reservoir. Table 2-7 summarizes details on each source of supply.

TABLE 2-7 WATER SUPPLY RESERVOIRS

Source	Status	Watershed Area (sq mi)*	Total Volume (MG)	Surface Area (sq mi)	Spillway Elevation (feet)	Safe Yield (MGD)
North Watuppa	Active	11.6	8,000	2.8 +/-	136.5	8.5-9
South Watuppa	Emergency	15.0	7,250	2.3 +/-	136**	9
Copicut Reservoir	Active	7.0	3,000	1.0 +/-	149.5	6-6.5

*from MassGIS watershed limits
** Top of Quequechan Control Structure Intake

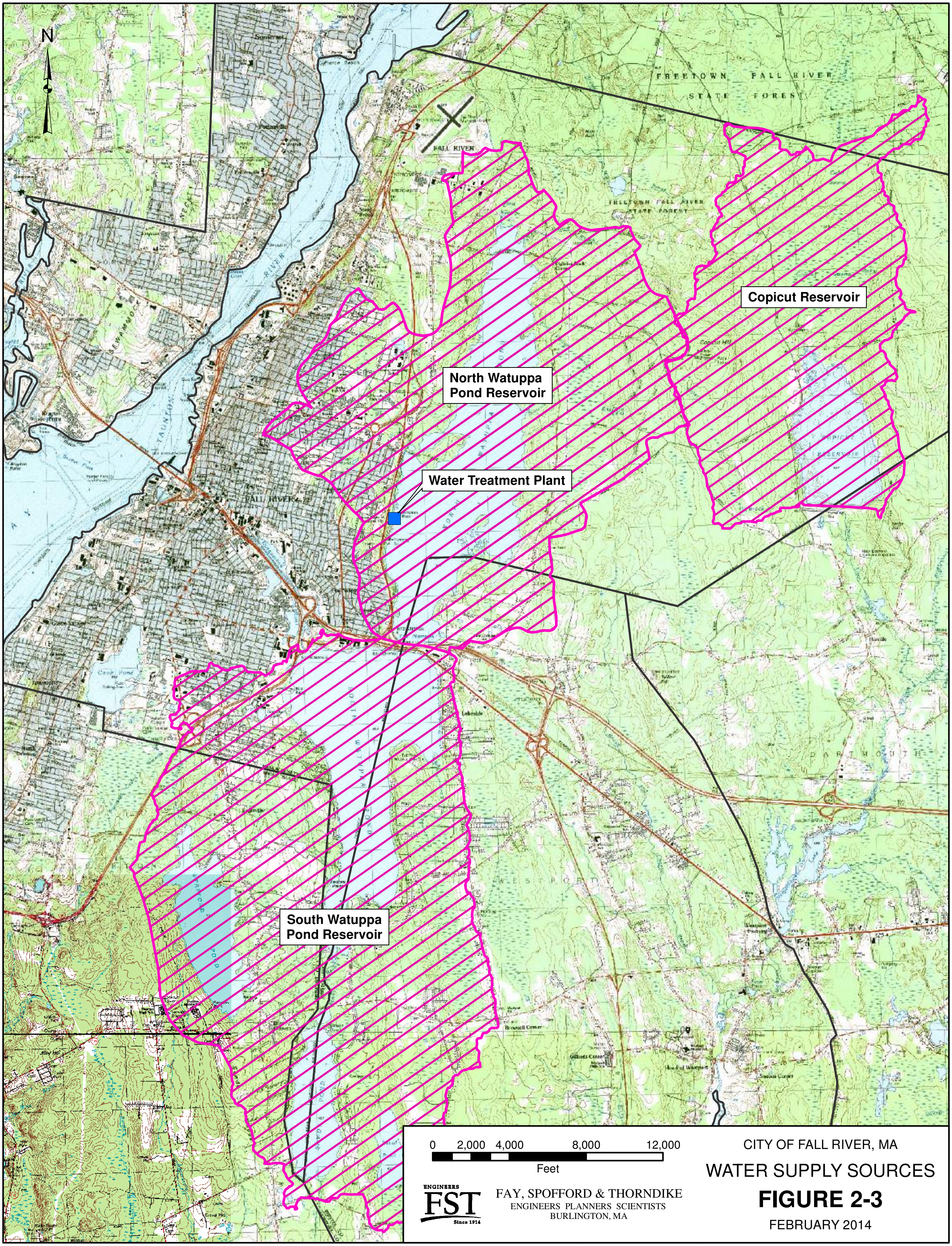
The City of Fall River obtains its water from North Watuppa Pond, located to the east of the City. North Watuppa Pond, with a volume of about 8 billion gallons, has a watershed area of approximately 11.6 square miles and a safe daily yield reported to be between 8.5 and 9 MGD according to the 1990 report, *Feasibility Study of Water Treatment Plant for South Watuppa Pond*; the 1981 Report, *Drought Management in New England*, by Water Purification Associates; and City personnel. North Watuppa Pond has been utilized as a potable water supply since the late 1870's.

The Copicut Reservoir, with a volume of about 3 billion gallons, was developed as an additional source of supply in 1975. Water is pumped from the Copicut Reservoir, which has a watershed area of about 7 square miles, on an as needed basis to maintain the level in North Watuppa Pond. The pumping station located at the southern end of Copicut Reservoir contains two 6 MGD pumps which have a combined pumping rate of approximately 10 MGD. The safe daily yield of Copicut Reservoir has been reported to be between 6 and 6.5 MGD on average for the year according to the 1990 feasibility study, the 1981 drought management report and discussions with City personnel. The perimeters of each water body are controlled by the City and development within each watershed is extremely limited. Furthermore, no recreational activities are allowed on either supply with the exception of shoreline fishing at Copicut Reservoir.

South Watuppa Pond, with a volume of about 7.25 billion gallons, is located directly to the south of North Watuppa Pond and has a watershed area of approximately 14.8 square miles and a safe yield of about 9 MGD according to previous reports. A dam and gatehouse separate North and South Watuppa Pond and allow water to flow by gravity from the pond with the higher water level to the pond with the lower water level. However, South Watuppa Pond has rarely been used as a source of supply in the past and is only available as a source of supply under emergency conditions. The last time that water was allowed to flow from South Watuppa Pond to North Watuppa Pond for treatment and consumption was around 1981. At that time, the water was treated with copper sulfate to help minimize taste and odor problems caused by algae present in South Watuppa Pond.

Development in the Pond's watershed has not been regulated. The areas surrounding South Watuppa Pond are heavily developed with both residential and industrial areas. Many of the adjacent homes have septic systems which may allow leaching of contaminants into the pond. Also, runoff and other waste from the industrial areas to the northwest of the pond can cause further contamination. Nitrate and other contaminants appear to cause seasonal algae blooms, creating significant variations in turbidity, color and pH, especially during the summer months when water consumption is at its highest levels. South Watuppa Pond is currently designated by the Massachusetts Department of Environmental Protection (DEP) as an emergency supply to augment the supply from North Watuppa Pond and Copicut Reservoir. Figure 2-3 shows the water supply sources for the City.

In addition to North Wattuppa, South Wattuppa and Copicut Reservoirs, the City of Fall River also has water rights to the follow sources: Sawdy Pond, Terry Brook, Lake Noquochoke, Forge Pond, Rattlesnake Brook and Stafford Pond.



FAY, SPOFFORD & THORNDIKE
ENGINEERS PLANNERS SCIENTISTS
BURLINGTON, MA

CITY OF FALL RIVER, MA

WATER SUPPLY SOURCES

FIGURE 2-3

FEBRUARY 2014

FIGURE 2-4 HISTORIC WATER PRODUCTION

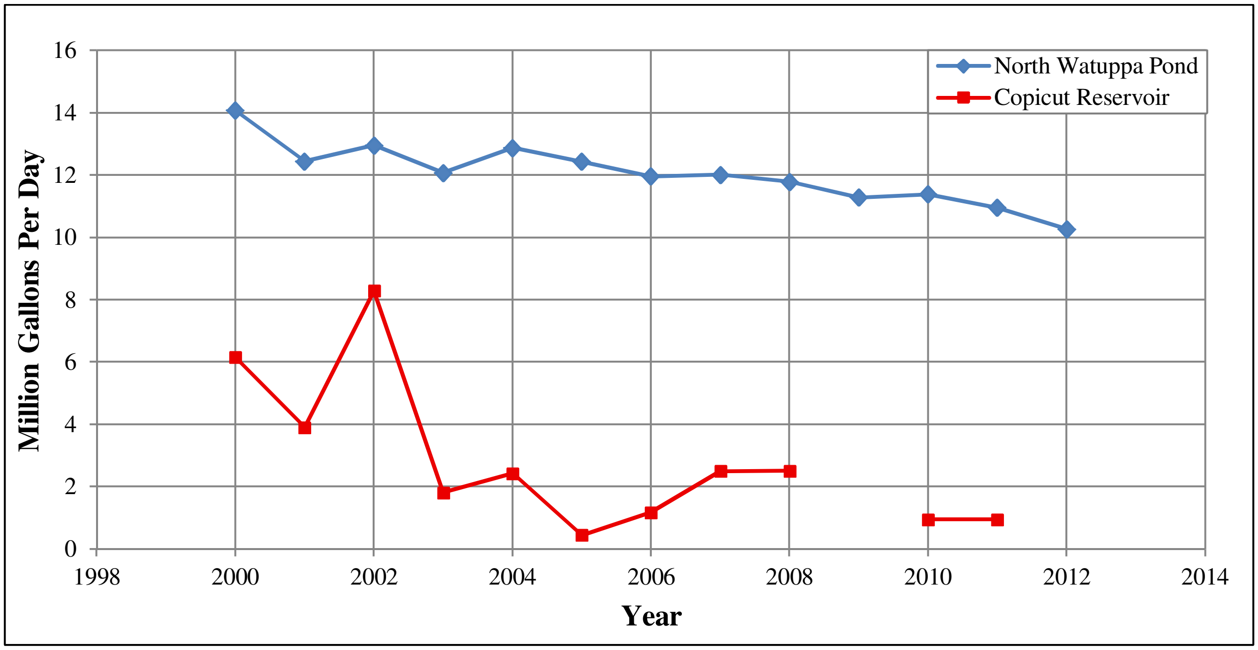


Figure 2-4 presents the historical yearly average water production from each source that the City utilizes based on MADEP Annual Statistical Reports. Since water from the Copicut Reservoir is initially pumped to the North Watuppa Pond, the water supplied by North Watuppa Pond represents the total raw water pumped to the treatment plant. The graph shows a stable downward trend in water production from North Watuppa Pond as a result of decreasing population over the last 10 years. This decrease in supply demand limits the amount of water pumped from the Copicut Reservoir to mainly annual maintenance operations to ensure pumping capabilities. In 2012, the City water supply facilities provided approximately 10.3 MGD of raw water to the treatment facilities.

At the City's Water Treatment Plant, water enters the facility through a single raw water intake. This intake is a critical component of the water system. FST recommends that the City inspect this intake to determine the integrity of the pipeline and evaluate whether improvements should be completed. This evaluation should also include the cost and feasibility of installing a second, redundant intake. The estimated cost for this evaluation is $30,000.

2.4 ADEQUACY OF WATER SUPPLY

The adequacy of a water supply source is determined by its ability to satisfy average daily demands including unaccounted for water and water required for WTP processes including backwash water.

The City's Water Management Act (WMA) Withdrawal Registration Statements are administered by the DEP and are attached as Appendix A. A summary of the WMA withdrawal registrations are included in Table 2-8.

TABLE 2-8 WMA WITHDRAWAL REGISTRATIONS

Source	River Basin	Average Volume per Day (MGD)	Total Annual Volume (MGY)
Copicut Reservoir	Buzzards Bay	6.37*	2,325.05*
North Watuppa and South Watuppa	Narraganset Bay	8.22*	3,000.3*
TOTAL		**14.59***	**5,325.35***

*Normal Variation: In accordance with 310 CMR 36.39(3), the MADEP has determined that with normal variation Fall River can withdraw up to 4,460.3 million gallons per year from the Buzzards Bay Basin and up to 5,325.35 MGY in the Narraganset Bay Basin. Fall River's combined withdrawal volumes from the Buzzards Bay Basin and the Narraganset Bay Basin cannot exceed the total registered withdrawal volume of 5,325.35 MGY, including the normal variation.

While the total safe daily yield of North Watuppa and Copicut without South Watuppa has been reported to be between 14.5 and 15.5 MGD, the DEP Water Management Act Registrations authorized a total withdrawal of 14.59 MGD. The water consumption trends and projections of the City have been analyzed and documented in previous sections of this report. The projected 2035 Average Day Demand is 10.71 MGD. With an authorized withdrawal of 14.59 MGD, the City has 3.88 MGD in available excess supply.

DEP is undertaking a broad initiative to revise Water Management Act permitting called the Surface Water Management Initiative (SWMI). Currently, the City has a permitted withdrawal rate of 14.59 MGD. Future permits may be based on the results of the SWMI. The SWMI framework states that for a river basin, the safe yield equals 55% of the flow in a statistical low flow simulated year. Reservoir systems may get a storage credit based on their size relative to the annual demand and stream flow into the reservoir during the low flow year. While these changes to SWMI are not finalized, the City should monitor the industry discussions on this methodology in order to maintain its existing safe yield.

2.6 WATERSHED IMPROVEMENTS

The Watuppa Watershed and Reservation consists of the Watuppa Reservation Headquarters campus as well as the watersheds of the North and South Watuppa Ponds and Copicut Reservoir.

The North Watuppa Pond is the primary source of water supply, which is treated at the Fall River Water Treatment Plant located at the end of Bedford Street on the west shore of North Watuppa Pond. Runoff from the west side of the watershed is intercepted by a concrete box channel that flows into South Watuppa Pond. This interceptor drain was installed to divert the suspected contaminated runoff of the Fall River industrial usage within the City. A second concrete channel exists crossing Frontage Street at Adirondack Lane in Westport which diverts run-off from that area to South Watuppa Pond. Therefore most of the watershed to the east and north of Watuppa Pond provides the water to the City's North Watuppa Pond.

Copicut Reservoir provides a supplemental source of water and is pumped to North Watuppa Pond when needed. Recently the water level in North Watuppa Pond has been adequate and pumping from Copicut Reservoir has been reduced to a monthly maintenance and equipment exercise.

FST performed an inspection of the watershed and reservation facilities on October 16, 2013, with Mr. Michael Labossiere, Watuppa Reservation Superintendent. The walk-thru inspection included a review of the nine structures at the Reservation Headquarters at located at 2928 and 2929 Blossom Road, Copicut Dam and Pump Station, and the access road and surrounding areas within the watershed. A general review and discussion was conducted and conditions were compared to that which had been previously presented in the Conditions Assessment and Preservation Plan for the Watuppa Reservation Headquarters prepared by Michael L. Keane in April 2011. A copy of the Conditions Assessment and Preservation Plan is included in Appendix B. A detailed summary of FST's walk through inspection is included as Appendix C.

In addition to watershed improvements, the dams that impound the Copicut Reservoir, North Watuppa Pond and South Watuppa Pond are in need of maintenance and repairs based on 2011 inspection reports completed by Pare Engineering. A copy of the Dam Assessment and Recommendations Sections of each inspection report is included in Appendix D. The City has been addressing these improvements over the last few years.

Further, the retaining walls surrounding portions of the North Wattuppa Pond are in poor condition and in need of rehabilitation. The City should further consider rehabilitation of the access roads in the reservation area including the causeway at Wilson Road.

A Watershed Protection and Management Plan was developed by the City in 2001. A draft version of the report was issued but the report was never finalized. A copy of this report is included as Appendix E. FST recommends this report be reviewed and updated to conform to the MADEP guideline *Developing a Local Surface Water Protection Plan*. Once complete, this report should be submitted to the MADEP. Having an approved plan can assist with obtaining funding available through the MADEP.

2.7 CONCLUSIONS AND RECOMMENDATIONS

Population within the City has been decreasing since the 1920s. Similarly, water demand within the City has also been decreasing. In the early 2000s water demand was at or above 12 MGD while in 2012 the average day demand was below 9 MGD. Average day demand conditions are anticipated to remain between 10 and 11 MGD throughout the planning period.

In addition to meters at the source of supply, the City has approximately 19,809 meters installed within the City. In order to accurately understand whether these meters are accurately measuring water produced and used in the City, FST recommends a water audit be completed. Included as part of this water audit, water meter records should be accurately classified to match the reporting categories included in the DEP's annual statistical report. This will increase the accuracy of these reports.

The estimated safe yield of North Watuppa Pond and Copicut Reservoir and the WMA Registration Statements adequately meet the water supply requirements for the system. Historically, over the last eleven years the average day demand has been about 11.4 MGD, about 3 MGD lower than the WMA registrations and also lower than the reported 14.5 to 15.5 MGD safe yield of both supplies. The projected 2035 average day demand of 10.7 MGD can be adequately supplied with the existing sources of supply.

If the City plans to wholesale additional water to other municipalities, there is sufficient water supply to stay below the safe yield and WMA limits. The use of South Watuppa Pond as an additional source of supply will most likely result in degradation of water quality in North Watuppa Pond. Based on projections, the supply from North Watuppa Pond and Copicut Reservoir are more than adequate and the use of South Watuppa Pond as an additional source is not recommended. The future use of South Watuppa Pond may be considered pending proper management of residential industrial areas, as well as additional treatment processes at the water treatment plant.

Each source of supply includes a dam to impound the water and the City owns extensive lands within the watershed. Based on dam inspections completed in 2011, maintenance and upgrades to these dams are required and have been on-going. Similarly, with the many buildings located with watershed limits, upgrades to these facilities are also required. Table 2-9 summarizes all of the costs associated with the water supply recommendations.

TABLE 2-9 WATER SUPPLY RECOMMENDATIONS

Item No.	Improvement Recommendation	Estimated Cost
1	Conduct Water Audit including updates to meter records	$30,000
2	Conduct Safe Yield Study	$40,000
3	Conduct Feasibility and Pilot Study for treatment at South Watuppa Pond	$400,000
4	Upgrades to Copicut Transfer Station	$250,000
5	Watershed Facilities Improvements	$750,000
6	Dam Improvements	$1,725,000
7	Update Watershed Protection Plan to conform to MADEP Guidelines	$50,000
8	Evaluate Raw Water Intake	$30,000
	CAPITAL COST TOTAL	**$3,275,000**

SECTION 3 - WATER TREATMENT

3.1 GENERAL

The entire water demand of the City of Fall River is supplied through the Water Treatment Plant (WTP) located on North Watuppa Pond at 1831 Bedford Street in Fall River, Massachusetts. The purpose of the water treatment plant is to be able to meet the maximum day demand with the largest pump out of service, protect the public health and maintain aesthetic water quality goals.

There are three avenues available to the City to protect the public health and maintain aesthetic water quality goals, namely, watershed protection, water treatment and distribution system maintenance. This is otherwise referred to as the "multi-barrier protection" concept. If one barrier fails then two other barriers are in place to meet water quality goals. In 2001, the City developed a draft Watershed Protection and Management Plan, the first barrier of protection. A copy of this report is included in Appendix E. Section 5.6 of this report focused on maintaining water quality in the distribution system, the third barrier of protection. The water treatment plant is the City's second barrier of protection. The purpose of this section is to evaluate the existing treatment processes and determine whether the treatment plant will be able to meet the increasingly stringent Safe Drinking Water Act requirements and future aesthetic water quality goals. Historical raw water quality was reviewed as well as the City's historical compliance with regulatory requirements.

This section also describes alternatives that will optimize the existing treatment plant's performance and recommends modifications and/or additional unit processes that may be utilized to help the City meet the treatment goals at the projected 2035 flow requirement of 14 MGD. Primary treatment goals are as follows:

- maintain filtered water turbidity less than 0.1 NTU,
- meet current and known future requirements of the Safe Drinking Water Act,
- maintain color levels less than 5 SCU,
- maintain iron and manganese levels below the SMCL's,
- minimize taste and odor
- maximize ease of operation and
- minimize capital and operation and maintenance costs.

3.1.1 Historical Raw Water Quality

The water treatment plant (WTP) receives water directly from the North Watuppa Pond via a screened intake. Water contained in the Copicut Reservoir is pumped on an as needed basis to maintain the level in the North Watuppa Pond. Typical operation is to run the Copicut Reservoir pump station one weekend per month over the entire course of the year.

The quality of the water entering the treatment plant for the past three year from North Watuppa Pond is summarized in Table 3-1.

TABLE 3-1 RAW WATER QUALITY

Parameter	Min.	Max.	Avg.
Color (CU)	0	78	19
Turbidity (NTU)	0.08	4.20	1.00
Total Organic Carbon, TOC (mg/L)	1.6	4.0	3.0
pH	5.68	7.61	6.72
Alkalinity (mg/L as $CaCO_3$)	1.0	9.5	3.3

Historical raw water color and turbidity are presented in Figure 3-1 and Figure 3-2, respectively. As shown in the figures, color and turbidity levels typically peak during spring and fall when reservoir turnover occurs. With the exception of alkalinity, the parameters in the above table were collected as part of daily monitoring by treatment personnel. Finished water quality characteristics for the past two years are presented in Table 3-2.

TABLE 3-2 FINISHED WATER QUALITY

Parameter	Average
Total Organic Carbon, TOC (mg/L)	1.69
Turbidity (NTU)	0.09
Total Dissolved Solids, TDS (mg/L)	88
Color (CU)	<5
Odor (TON)	0
pH	8.19
Alkalinity (as $CaCO_3$)	33
Hardness ($CaCO_3$)	9.40
Calcium (mg/L)	2.37
Magnesium (mg/L)	0.85
Aluminum (mg/L)	ND
Iron (mg/L)	ND
Manganese (mg/L)	ND

.

FIGURE 3-1 RAW WATER COLOR

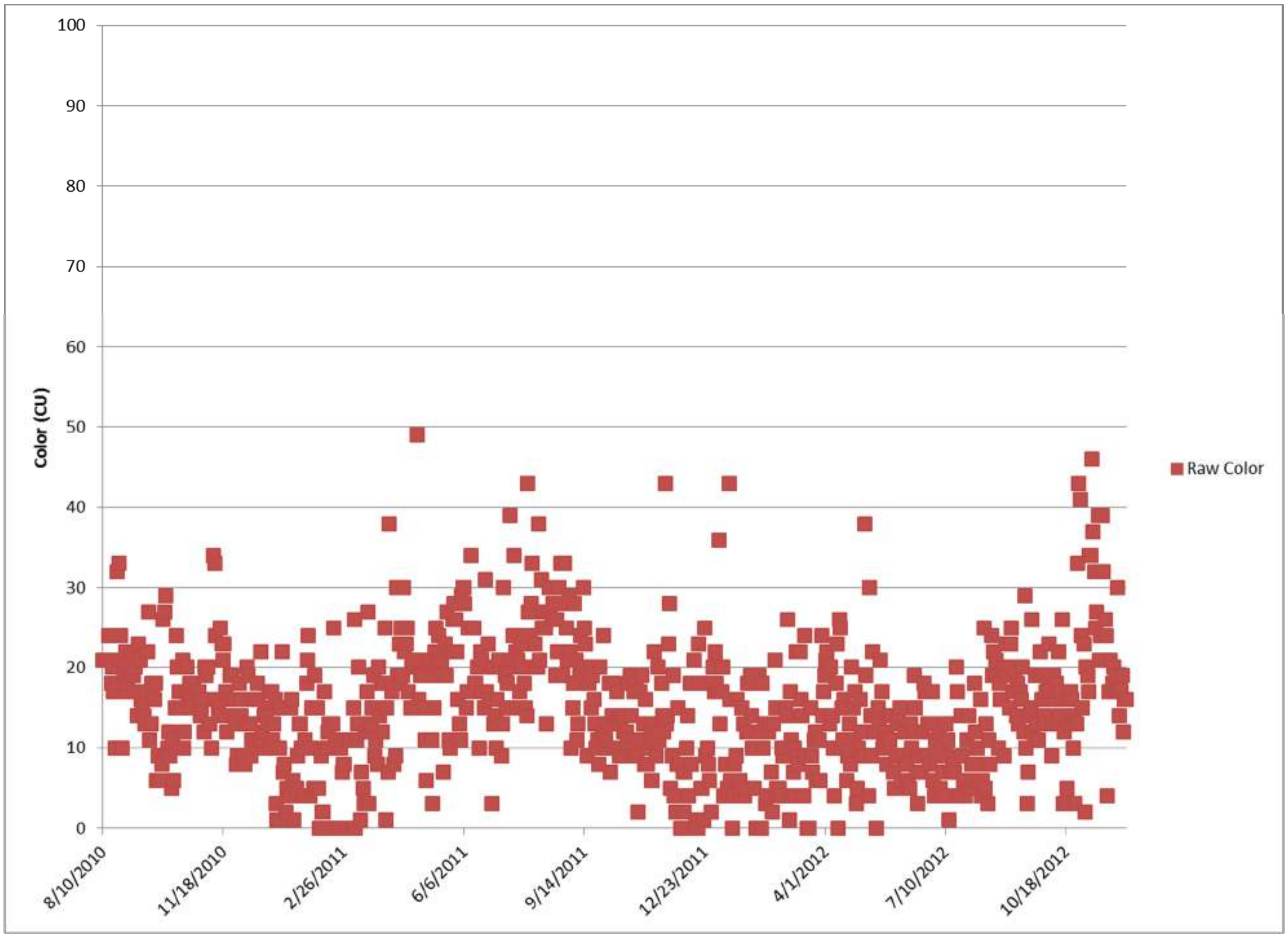
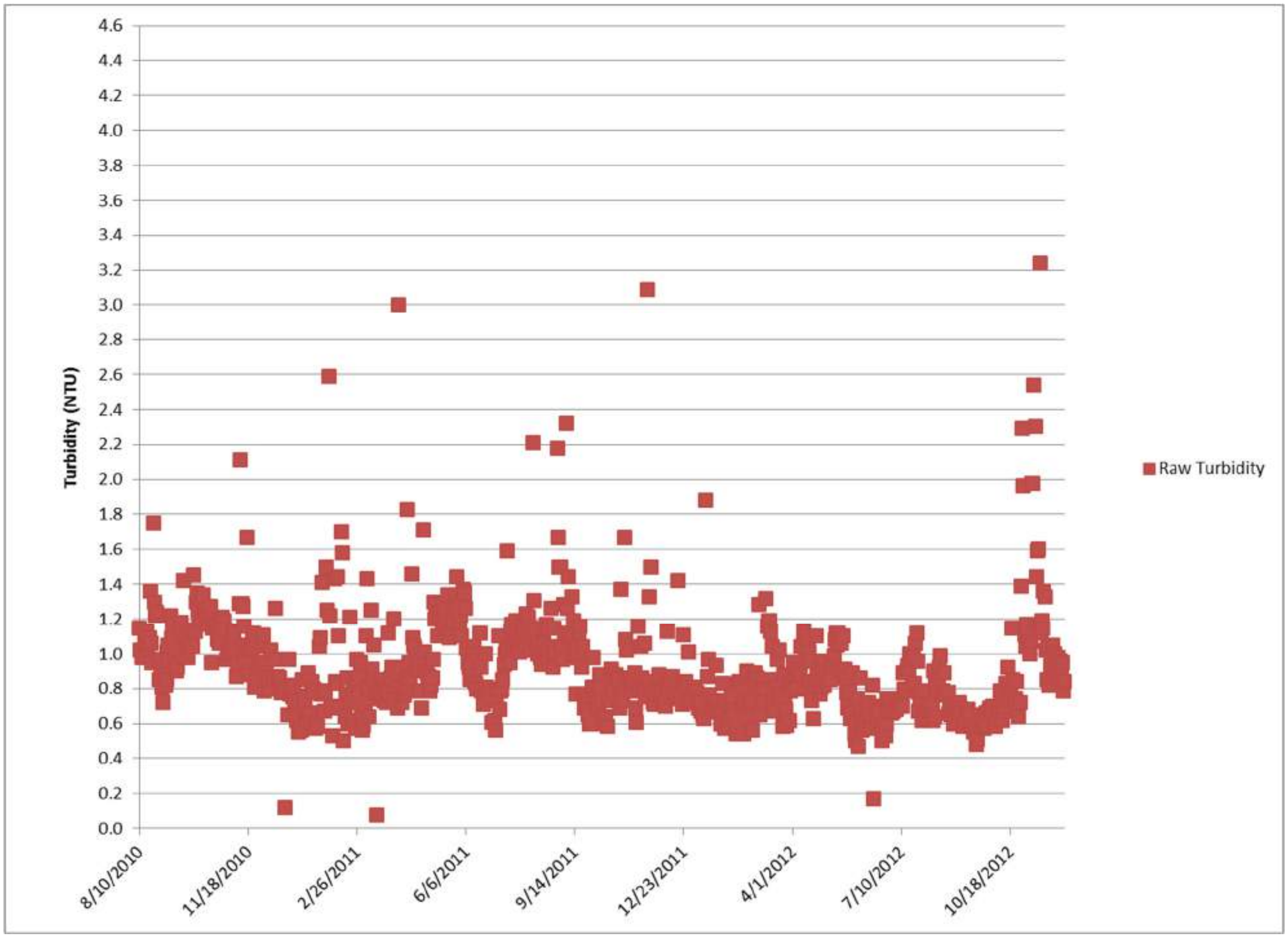


FIGURE 3-2 RAW WATER TURBIDITY



3.1.2 Treatment Facilities

Fall River's Water Treatment Plant (WTP), located on the western shore of North Watuppa Pond, provides treatment for the City's public water supply reservoirs. The WTP was originally constructed in 1975-1976 with a reported capacity of approximately 26 MGD. In 2004 the WTP underwent a series of upgrades. These upgrades included:

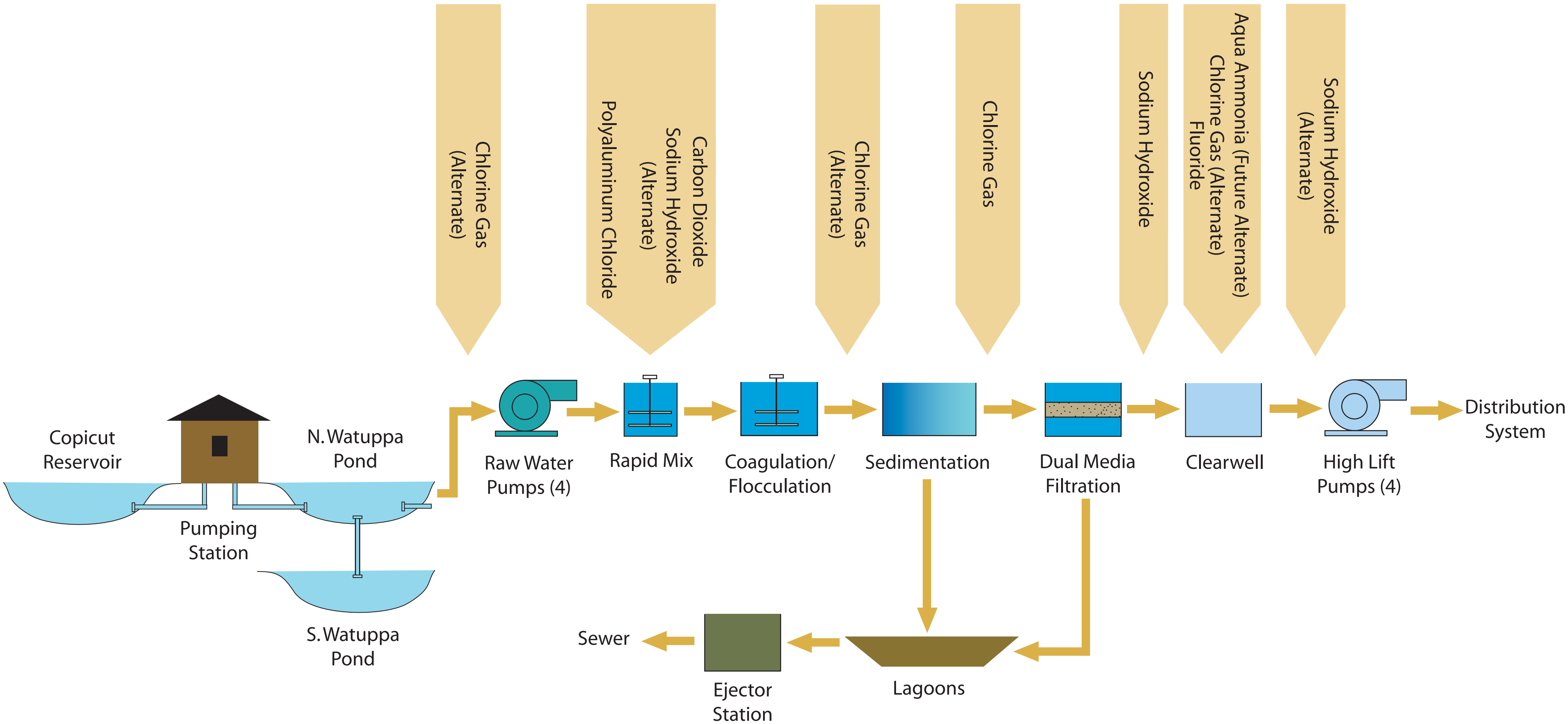
- Upgrading of the automatic backwashing (ABW) filter system to include 15-inches of dual media (anthracite over sand), filter to waste capabilities, and an improved control system.
- Installation of new chain and flight solids removal system in each sedimentation basin.
- Installation of a carbon dioxide feed system for pH and alkalinity adjustment.
- Installation of an aqueous ammonia feed system for the future use of chloramines for secondary disinfection. This was initially installed for bacteria control and is not currently on-line.
- Installation of variable frequency drives on the raw water pumps.
- Installation of a new roofing system over the pump room, chemical feed room and administrative areas.
- Installation of a new Supervisory Control and Data Acquisition (SCADA) system for the WTP processes.
- In 2012, new switchgear and motor control center (MCC) for the finished water pumps were installed.

A summary of the conventional treatment process currently utilized at the WTP is as follows:

- Pre-filtration chlorination at a dose of approximately 0.75 mg/L for taste and odor control is added after the sedimentation basins prior to filtration. Every 8 to 10 weeks the point of pre-chlorination is moved to the raw water clearwell prior to the rapid mix basin.
- Sodium hydroxide (caustic soda) at a dose of approximately 5 mg/L in conjunction with carbon dioxide at a dose of approximately 6 mg/L for coagulation pH adjustment to 6.5 to 6.8 and alkalinity to approximately 20 mg/L depending on raw water quality.
- Polyaluminum chloride at an average dose of 22-23 mg/L for coagulation.
- Rapid mix, (mixer is currently not used), flocculation and sedimentation to flocculate and settle particles.
- Shallow bed (16-inch) dual media filtration consisting of sand with an anthracite cap to remove fine suspended particles not previously removed by settling.

- Sodium hydroxide (caustic soda) addition just prior to entering the clearwell for post-pH adjustment to 8.2.
- Chlorination at a dose of approximately 2 mg/L through a 500,000 gallon capacity clearwell.
- Fluoridation (sodium fluorosilicate) for dental caries at 1 mg/L.

Figure 3-3 presents a schematic of the existing treatment plant processes. An aerial photo of the treatment plant and surrounding area is shown in Figure 3-4.



Fall River Water Supply and Treatment Plant Schematic
Existing Treatment Process

FST FAY, SPOFFORD & THORNDIKE 100 YEARS

Figure 3-3

City of Fall River, MA
Figure 3-4
Water Treatment Plant Location Plan
Fay, Spofford & Thorndike

3.2 SAFE DRINKING WATER ACT

The quality of a potable water supply is subject to Safe Drinking Water Act (SDWA) regulations established by the Environmental Protection Agency (EPA). The SDWA was originally passed in 1974 and was revised in 1986 and again in 1996. The Massachusetts Department of Environmental Protection (DEP) is the responsible State Agency for enforcing these regulations. Water utilities are required to sample raw water and water representative of that which consumers drink (finished water) to assure conformance with these regulations.

Table 3-3 summarizes the specific rules of the SDWA, the dates on which the rules were promulgated, dates on which they became (or are expected to become) effective, and whether the City is in compliance. The rules set maximum contaminant levels (MCLs) and/or establish guidelines for various contaminants and water quality parameters determined by the EPA to cause a risk to human health.

TABLE 3-3 SPECIFIC SDWA RULES

SDWA Rule	Promulgated	Effective	Fall River Compliance
Surface Water Treatment Rule	June 29, 1989	December 31, 1990	Yes
Total Coliform Rule	June 29, 1989	December 31, 1990	Yes
Lead & Copper Rule	June 7, 1991	December 7, 1992	Yes
Interim Enhanced Surface Water Treatment Rule	December 16, 1998	January, 2002	Yes
Stage 1 Disinfectant and Disinfection By-Products Rule	November 1998	January, 2002 for population >10,000	Yes
Long Term I - Enhanced Surface Water Treatment Rule	January 2002	February 13, 2002	Not Required Systems <10,000
Stage 2 Disinfectant and Disinfection By-Products Rule	January 2006	April, 2007*	NO
Long Term II - Enhanced Surface Water Treatment Rule	January 2006	January, 2007	Yes
Groundwater Rule	October 2006	December, 2009	Not Applicable
Revised Coliform Rule	February 2013	April 1, 2016	Not Required

* For initial compliance the City prepared an Initial Distribution System Evaluation (IDSE) in April 2007. The City began compliance sampling in October 2012 and compliance reporting in October 2013

3.2.1 Total Coliform Rule (TCR)

Total coliform is an indicator parameter that measures for the potential presence of bacteria in a water distribution system. Coliform indicates the potential presence of other bacteria such as fecal coliform and E. coli. Sample sites located throughout a distribution system are used to measure for the potential presence of bacteria system.

- The City is required to collect a minimum of 90 samples per month based on the population of 94,000 residents.
- If total coliform is present (coliform positive) in a sample then three repeat samples must be taken. One at the original location and one within five service connections upstream and downstream.
- If more than 5% of the samples analyzed per month test positive for total coliform the MCL is violated and public notification must be carried out.
- If these samples are positive for fecal coliform or E.coli the DEP must be notified within 24 hours of this violation because it poses an acute risk to human health.

The City most recently violated the TCR in July of 2008. During July 2008 one sample tested positive for e.coli but all repeat and upstream samples tested negative for both e.coli and total coliform. While this was a violation of the TCR, this positive sample may have been inadvertently contaminated resulting in a positive. The City has maintained compliance with the TCR since this last violation.

3.2.2 Revised Total Coliform Rule (RTCR)

In December of 2012 the US EPA issued revisions to the 1989 TCR. One of the main revisions to the TCR is that the revised rule establishes a health goal (Maximum Contaminant Level Goal or MCLG) and Maximum Contaminant Level (MCL) for *E. coli* and eliminates the MCLG and MCL for total coliform replacing it with a treatment technique for coliform that requires assessment and corrective action.

Under the new treatment technique for coliform, a water supplier such as Fall River that exceeds a specified frequency of total coliform occurrences must conduct an assessment to determine if any sanitary defects exist, and if found take corrective action. If there is an *E. coli* violation the PWS must conduct an assessment and correct any sanitary defects found.

As a result of the elimination of MCL and MCLG for total coliform the RTCR eliminates the monthly reporting requirement based on the presence of total coliform. Instead the RTCR only applies when an *E. coli* MCL violation occurs.

The City will need to comply with the requirements of the RTCR beginning April 1, 2016.

3.2.3 Lead and Copper Rule (LCR)

Lead and copper sampling is conducted on residential houses to determine if treated water in the City's distribution system is corrosive to lead and copper present at customer service lines and residential plumbing systems. The LCR sets at-the-tap 90th percentile Action Levels (AL) for lead and copper of 0.015 mg/L and 1.3 mg/L, respectively. At-the-tap samples are collected by the residents at homes where there are known lead or copper service lines and/or interior plumbing. The "standard sampling" schedule required the City to collect samples at 60 sites annually throughout the distribution system, and two additional school sites: John E. Boyd Center and Spencer Borden School. Since the City has demonstrated that the Action Levels have not been exceeded for three consecutive years of "standard monitoring", they can now conduct "reduced monitoring", sampling once every 3 years. The next sampling round is scheduled for 2014. Table 3-4 presents a summary of recent at-the-tap lead and copper levels.

TABLE 3-4 AT THE TAP LEAD AND COPPER LEVELS

Sampling Date	Lead 90th % (mg/L)	Copper 90th % (mg/L)	Number over Lead AL	Number over Copper AL
August 2012	0.005	0.06	0	0

These data indicate the City is in compliance with the LCR.

3.2.4 Stage 1 and 2 Disinfectants & Disinfection By-products Rule

The Stage 1 D/DBP Rule includes MCLs for specific disinfection by-products. Table 3-5 shows the MCLs below.

TABLE 3-5 MCLS FOR SPECIFIC DISINFECTION BY-PRODUCTS

Disinfection By-Product	MCL
Total Trihalomethanes (TTHMs)	80 µg/L
Haloacetic Acids (HAA5)	60 µg/L
Bromate	10 µg/L(1)
Chlorite	1.0 mg/L(2)

(1) Typically a by-product of ozonation which is not used at the WTP.
(2) Typically a by-product of chlorine dioxide which is not used at the WTP.

Under the Stage 1 D/DBP Rule, the MCLs are based on a system-wide running annual average of four samples per quarter. Twenty five percent of the samples must be taken at the maximum residence time within the distribution system and 75% at average residence time from the treatment plant. The Stage 2 D/DBP Rule maintains the DBP MCL's outlined in the Stage 1 Rule however compliance is based on the Running Annual Average at each individual sampling location identified in the Initial Distribution System Evaluation (IDSE) instead of averaging all of the sample locations.

The D/DBP Rule also outlines Maximum Residual Disinfectant Level Goals (MRDLG). Table 3-6 shows the MRDLG.

TABLE 3-6 MRDLG FOR SPECIFIC DISINFECTION METHODS

Disinfection Chemical	MRDLG
Chlorine	4.0 mg/L as Cl_2
Chloramine	4.0 mg/L as Cl2
Chlorine dioxide	0.8 mg/l has as ClO_2

Historic TTHM and HAA5 concentrations in the Fall River distribution system are shown in

Figure 3-5 and Figure 3-6.

FIGURE 3-5 HISTORICAL DISTRIBUTION SYSTEM TTHM CONCENTRATIONS

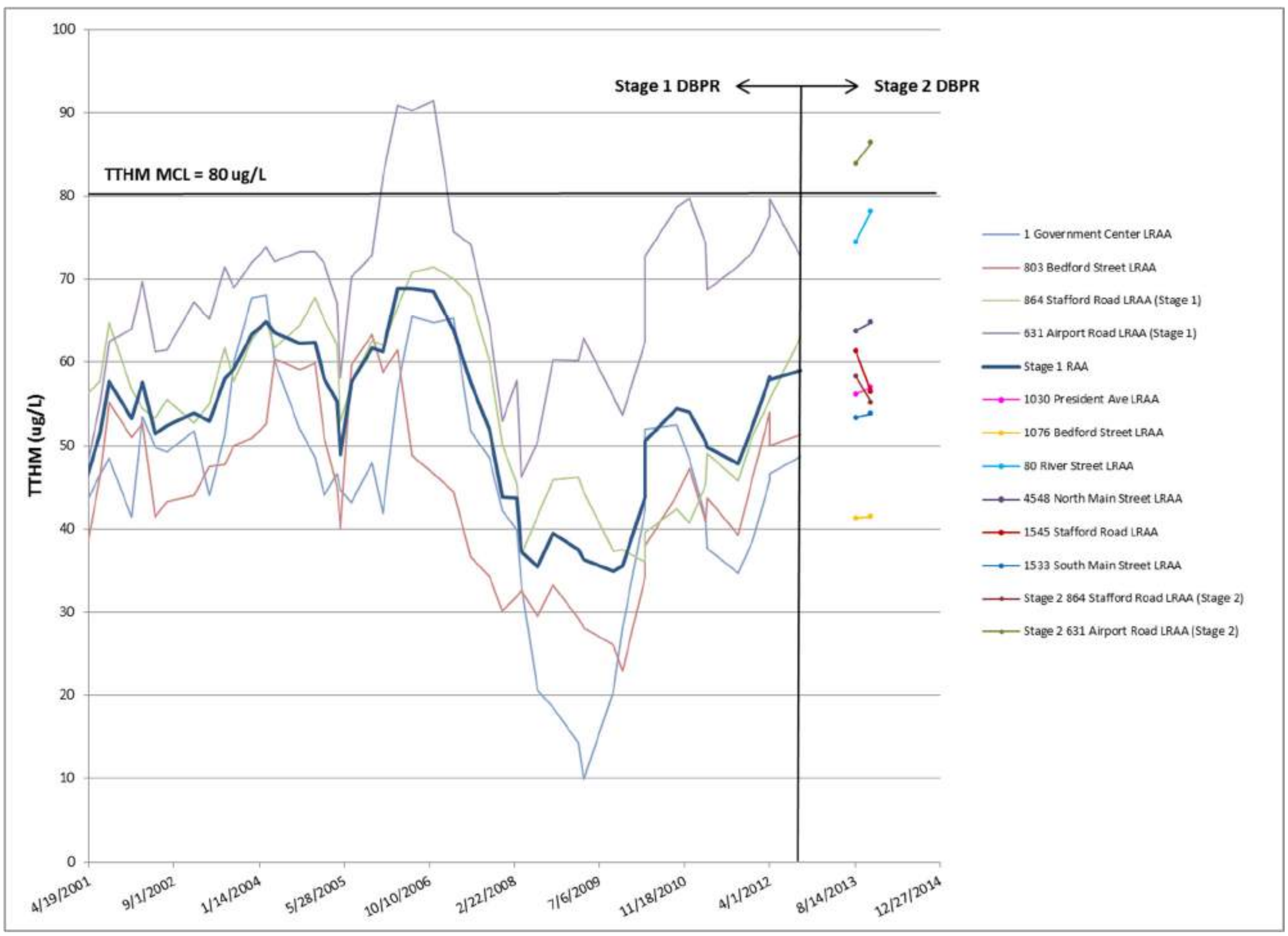
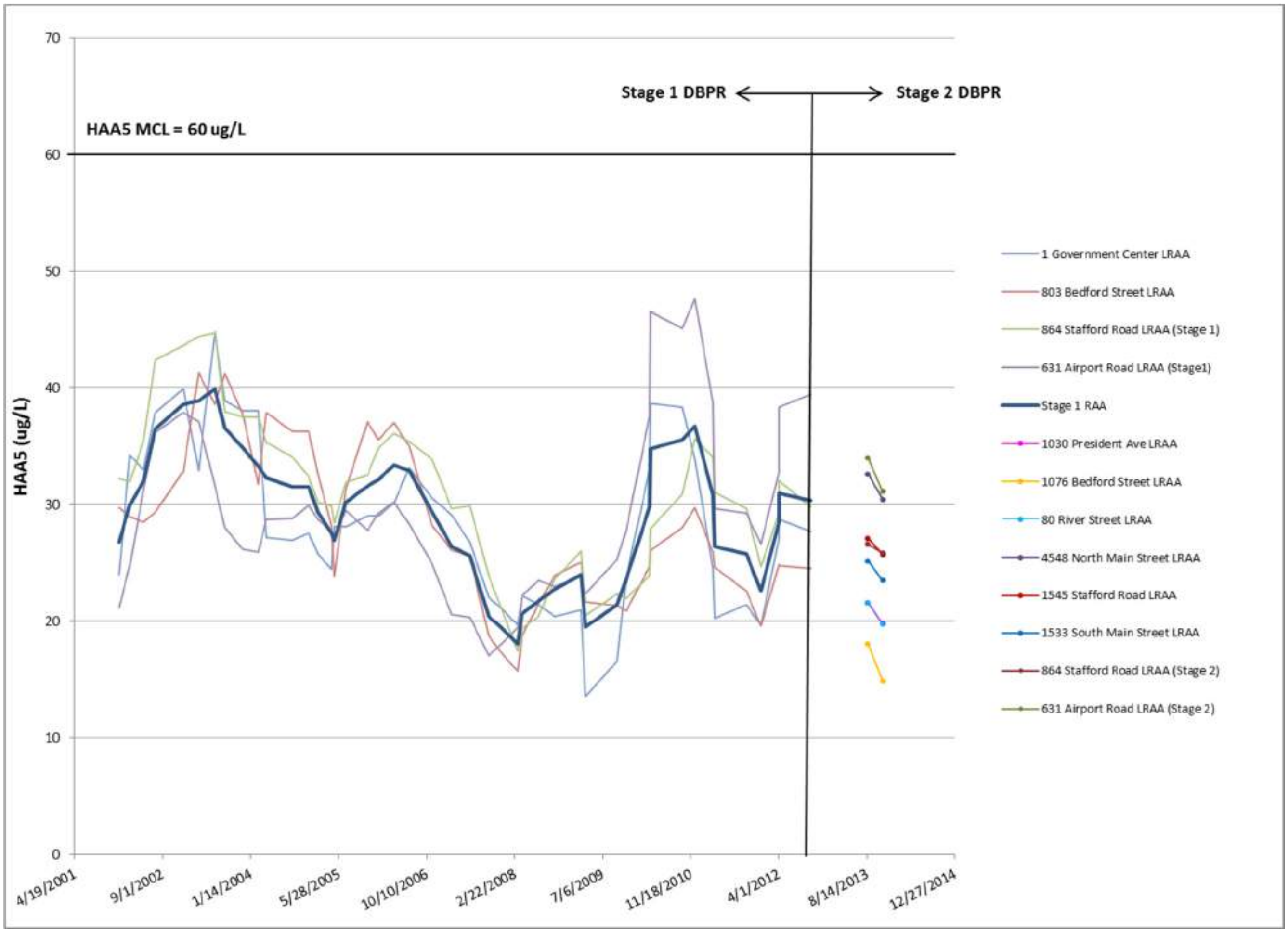


FIGURE 3-6 HISTORICAL DISTRIBUTION SYSTEM HAA5 CONCENTRATIONS



These data show that the City maintained compliance with the Stage 1 D/DBP Rule for both TTHMs and HAA5s. The first compliance calculation for locational running annual average was performed in August 2013 for the Stage 2 D/DBP Rule. The 691 Airport Road location had the highest concentration of TTHMs at 84.0 ug/L, exceeding 80 ug/L MCL and 4548 North Main Street had the highest concentrations of HAA5s at 32.6 ug/L, below the MCL. These data indicate that the City has not maintained compliance with the Stage 2 D/DBP Rule. Specifically the City has exceeded the TTHM MCL of 80 ug/L at the 691 Airport Road sampling location.

In an effort to address the TTHM issues the City is beginning a pilot program in the Fall River Industrial Park, with the construction of a new high service area. The construction will consist of a new pumping station that will boost water to a new elevated storage tank through the existing distribution mains in the Industrial Park. The new water storage tank will include an aeration and mixing system to keep the water in the tank fully mixed and be capable of removing DBPs from the water. With this aeration and mixing system, the water being supplied by the storage tank will have lower levels of DBPs and will help to provide high quality water to this section of the City and to the Freetown meter in Innovation Way.

3.2.5 Enhanced Coagulation

The Surface Water Treatment Rule requires all surface water treatment facilities utilizing conventional treatment like Fall River to provide a specified level of Total Organic Carbon (TOC) removal. Combining chlorine for disinfection with naturally occurring organic carbon in the water forms disinfection byproducts such as TTHMs and HAA5s, by reducing TOC, the potential for DBP formation is also reduced. Compliance with TOC removal is based on a running annual average (RAA), computed monthly, of raw water TOC concentration and raw water alkalinity. Table 3-7 summarizes TOC removal requirement based on raw water quality.

TOC percent removal is determined from paired samples of raw and filtered water taken at a time representative of normal operating conditions. The filtered water sample must be taken near the point of the turbidity reading.

TABLE 3-7 REQUIRED TOC REMOVAL REQUIREMENTS

Required TOC Removal by Enhanced Coagulation			
Raw Water TOC	**Source-Water Alkalinity, mg/L as $CaCO_3$**		
	0 -60	**> 60 - 120**	**> 120**
> 2.0 – 4.0	35%	25%	15%
> 4.0 – 8.0	45%	35%	25%
> 8.0	50%	40%	30%

Table 3-8 presents TOC removal at the WTP. Raw water TOC at the WTP is typically in the 2 to 4 mg/L range. Given a typical raw water alkalinity of 10 mg/L 35% TOC removal is required.

The data presented in Table 3-8, indicate that the WTP has regularly been able to achieve >35% removal of TOC based on the low raw water TOC. December 2012 had a raw water alkalinity of less than 2.0 mg/L, this allowed the City to follow the alternative compliance criteria and to not have to comply with a minimum TOC removal requirement of 35%.

TABLE 3-8 TOC REMOVAL PRFORMANCE

Date	Required % Removal	% Removal Provided
2012		
January	35%	42%
February	35%	40%
March	35%	42%
April	35%	43%
May	35%	49%
June	35%	43%
July	35%	41%
August	35%	45%
September	35%	41%
October	35%	40%
November	35%	35%
December	NA(1)	8%

(1) An alternative compliance criterion was used because the raw and/or finished water TOC was less than 2.0 mg/L.

3.3 PARTNERSHIP FOR SAFE WATER

Although keeping in compliance with regulations is important, it should not be the goal of the water treatment plant to simply meet the criteria set forth by the SDWA. The goal of the water treatment plant and operations personnel should be to provide the highest possible quality of finished water to consumers regardless of the MCLs set by the SDWA. If this philosophy is followed, it will not only provide the highest degree of public health protection, but will also maintain the highest level of consumer confidence in their drinking water supply.

When considering the optimization of a specific water treatment facility, it first appears that the most important variables in maintaining high quality finished water are tangible variables such as raw water quality, coagulant type, filtration rate and plant age. However, intangible variables such as operator skill and awareness and a commitment to achieve the best quality water are equally, if not more, important. In fact, some studies found that there is no significant correlation between the tangible variables studied and finished water turbidity. Many plants were able to produce a low turbidity effluent despite apparent physical limitations because of a highly dedicated operations staff.

This is not to say that the City's WTP operations staff is not committed to achieving a high quality finished water. However, it is extremely important for the City to support the WTP in terms of adequate staffing and funding to allow operations staff to maintain a strong commitment towards maintaining high effluent water quality standards and continuously seek to optimize the treatment plant's performance.

The Partnership for Safe Water, established in 1995, was formed to help public water utilities apply a standardized procedure for assessment of their surface water treatment plants. The goals of the program, which includes participation by many of the major national drinking water organizations, include:

- Improving public health protection beyond the existing SWTR
- Cooperation between water suppliers, regulatory agencies and the public
- Recognition for supplying a high quality drinking water with tenacity toward improved public health protection

The program is carried out in four phases:

Phase I - Commitment to the Program
Phase II - Data Collection and Reporting
Phase III - Self-Assessment
Phase IV - Third Party Assessment

Phase III is the most challenging and arguably the most important of the four phases. It includes a self-assessment of the entire utility organization following the existing Composite Correction Program (CCP). The first component of the CCP is the Comprehensive Performance Evaluation (CPE) phase. The CPE includes a thorough review and analysis of the water treatment plant's design capabilities and associated administrative, operational and maintenance practices. Based upon the findings of this evaluation, the utility may move forward with improvements to the existing facility that will help the plant achieve optimal performance without the high cost of major plant modifications or expansions.

The following section presents the findings of the evaluation of Fall River's water treatment facility.

3.4 WATER TREATMENT FACILITY EVALUATION

3.4.1 Performance Assessment

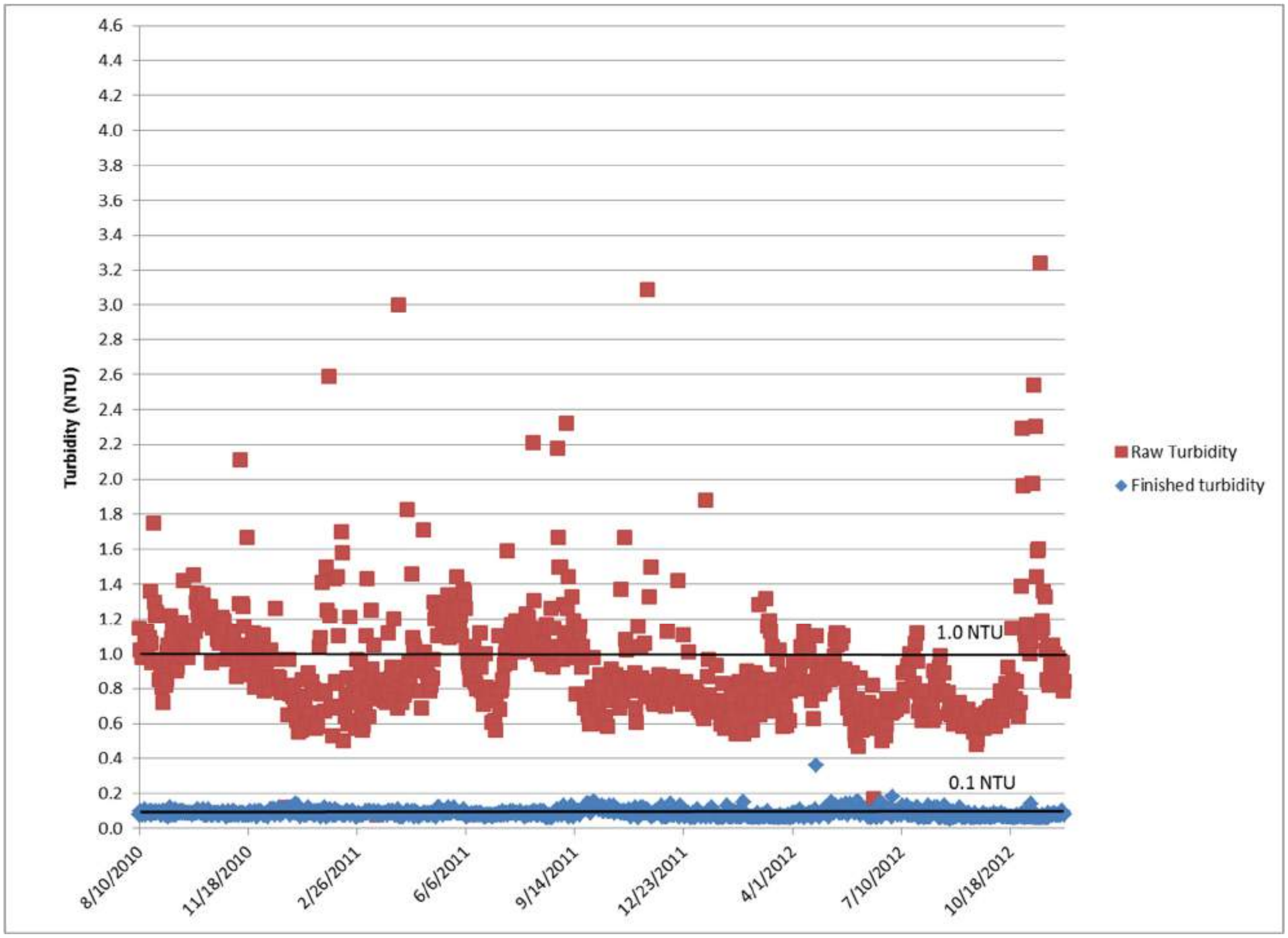
The major focus of this comprehensive performance evaluation (CPE) was to assess if, under existing conditions, the water treatment facility is able to comply with the turbidity and disinfection requirements of the IESWTR. To meet the IESWTR the plant must take a raw water source (North Watuppa Pond) of variable quality and produce consistent high quality finished water. Multiple treatment processes including coagulation, flocculation, sedimentation, filtration, and post disinfection, are provided in series to remove turbidity and cysts by settling and filtration and then inactivate cysts and other microorganisms utilizing chlorine. Each process represents a barrier to prevent the passage of cysts and other microorganisms through the plant. By providing multiple barriers, any microorganisms passing one process will be removed in the next, minimizing the likelihood of microorganisms passing through the treatment system and surviving in water supplied to the public. All treatment processes in the plant must be capable of providing this barrier at all times because even an instantaneous loss of a barrier could result in the passage of cysts and microorganisms into the distribution system. Any passage of cysts and microorganisms, even for short periods of time, represents a potential health risk to the community.

The CPE includes an assessment of the past and present performance of the City's plant. This performance assessment is intended to identify whether or not specific unit treatment processes are providing multiple barrier protection through optimal performance and if the plant is capable of complying with current regulations. The performance assessment is based on data from plant records and on data collected during special studies performed during the CPE.

The WTP has been able to meet all the primary state and federal drinking water quality regulations with exception of the Stage 2 D/DBP.

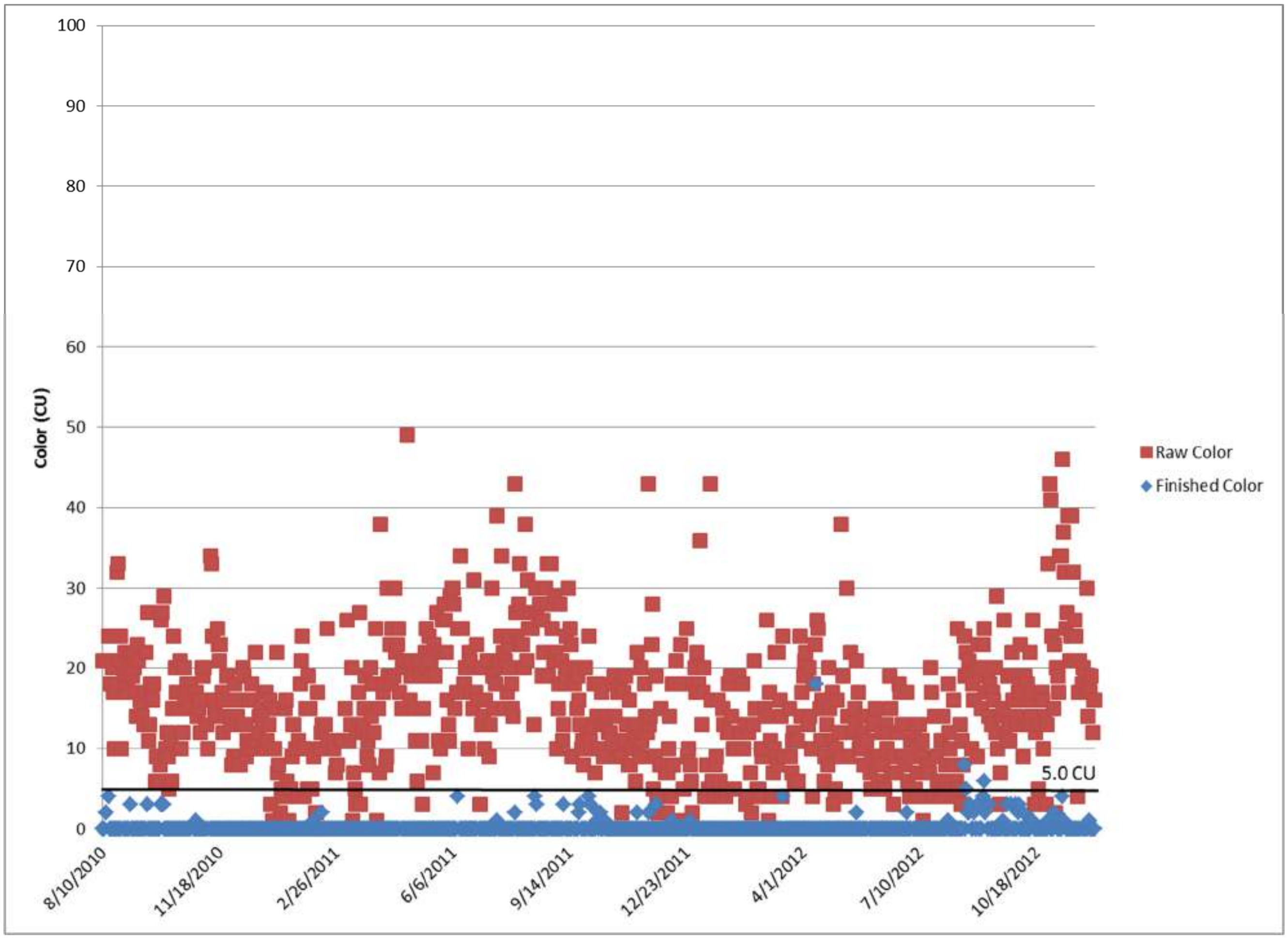
Figure 3-7 presents raw and finished water turbidity. The data presented indicate that finished water turbidity is consistently below 0.2 NTU. The Long Term 1 Enhanced Surface Water Treatment Rule requires that turbidity be less that 0.3 NTU 95% of the time.

FIGURE 3-7 FALL RIVER WTP HISTORIC TURBIDITY REMOVAL



Raw and treated water color is presented in Figure 3-8. These data indicate that much of the time finished water color is at or near zero color units. At times however, finished water color has exceeded the secondary standard of 5 color units. This secondary standard was developed for the aesthetics of the water and is not based on health impacts. Higher finished water color is likely related to seasonal pond turnover and which resulted in higher raw water color and therefore the coagulated demand increased.

FIGURE 3-8 FALL RIVER WTP HISTORIC COLOR REMOVAL



Finished water pH and alkalinity data are presented in Figure 3-9 and Figure 3-10.

FIGURE 3-9 FALL RIVER WTP HISTORIC FINISHED WATER PH

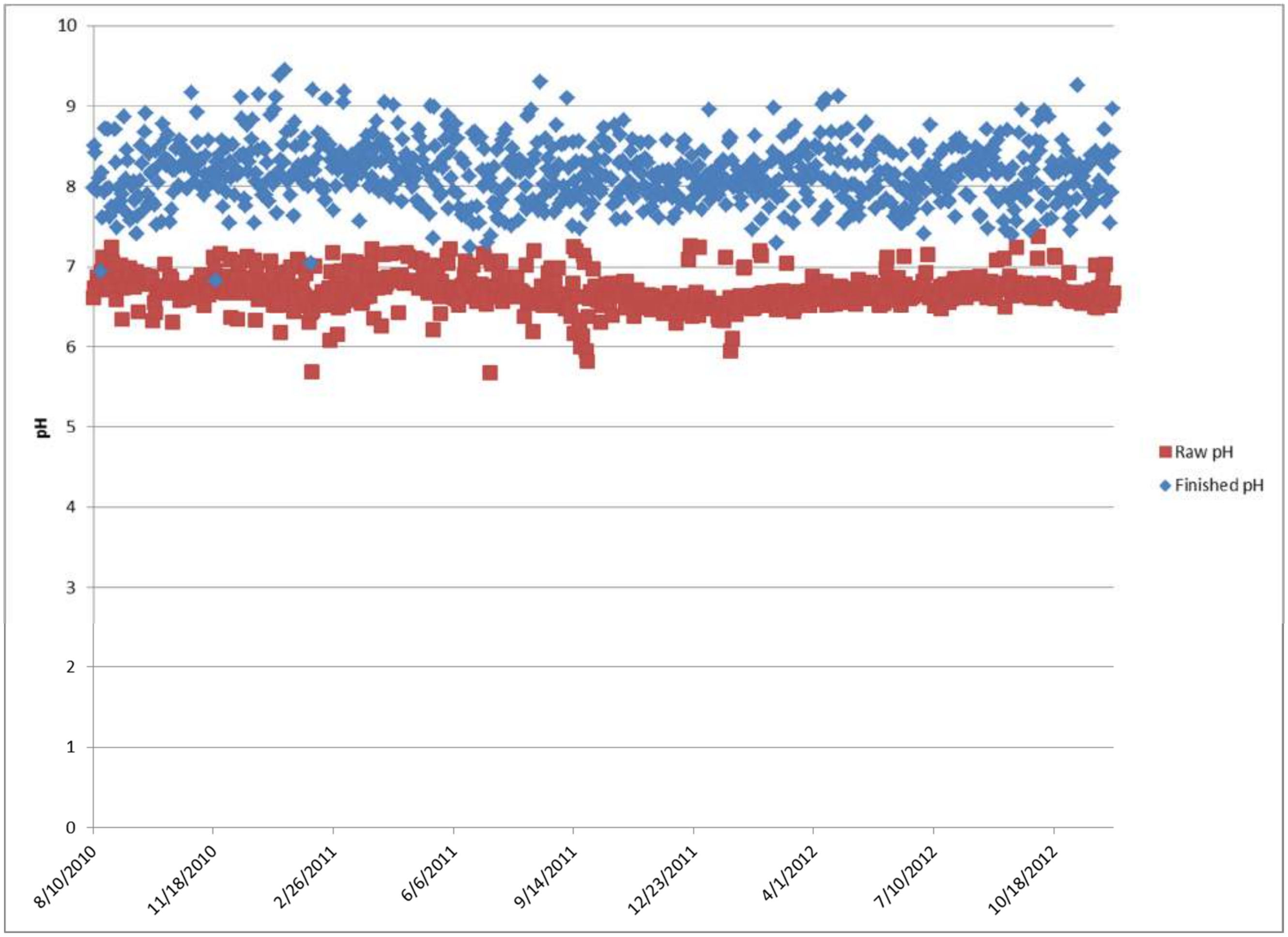
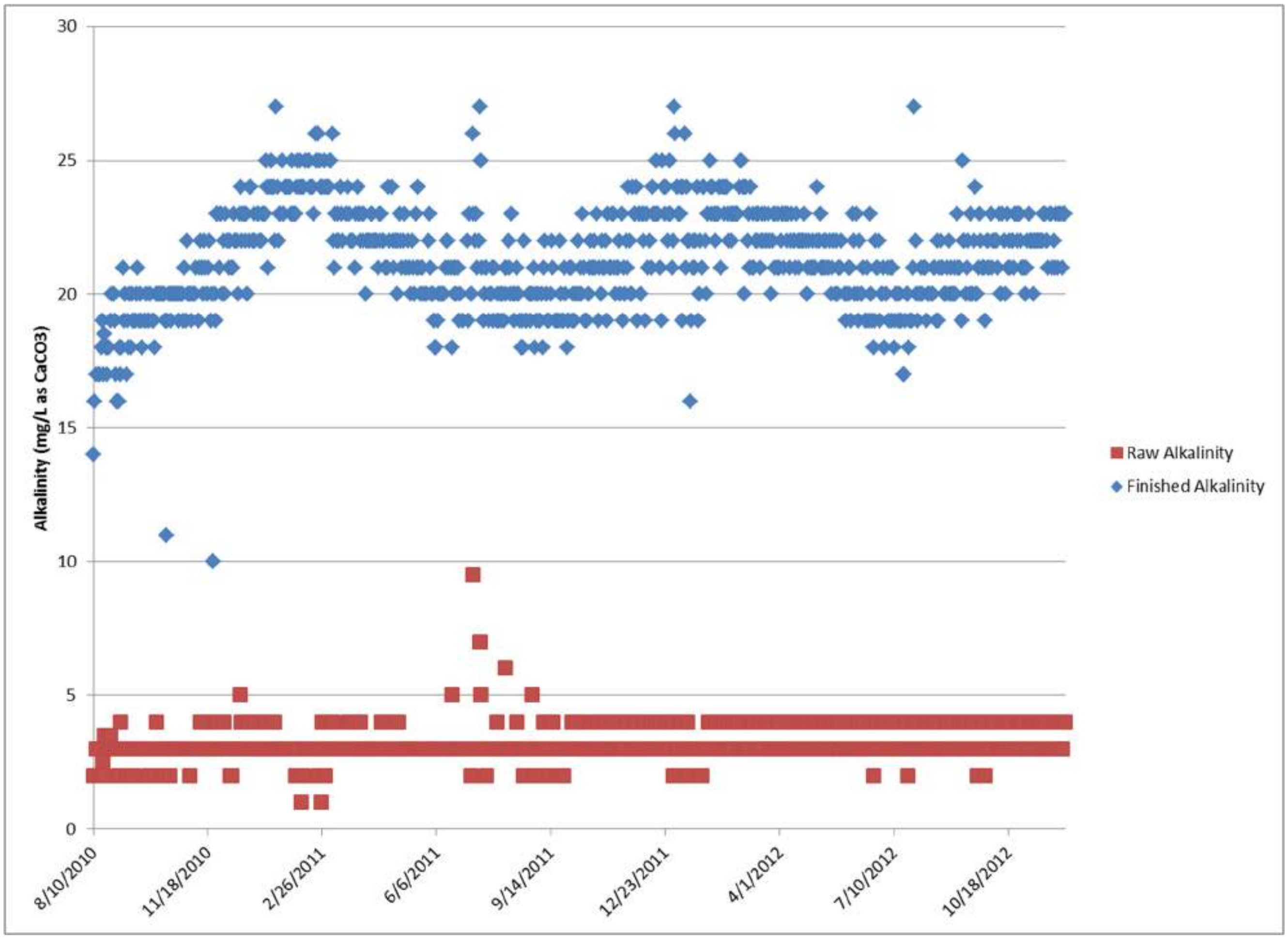


FIGURE 3-10 FALL RIVER WTP HISTORIC FINISHED TOTAL ALKALINITY



These data indicate that the Fall River WTP is able to consistently produce water with the target pH of 8.9 and a total alkalinity of 23 mg/L. Is should be noted that the finished water alkalinity varies over time. The alkalinity should remain consistently above 20 mg/L to minimize corrosion and conditions that could support microbiological growth in the distribution system. The City should consider operating the carbon dioxide system in the automatic mode instead of manual mode of operation to more consistently achieve alkalinity levels above 20 mg/L.

3.5 WATER TREATMENT PLANT CAPACITIES

The Fall River WTP has a reported design capacity of 26 MGD. The rated (or physical) capacity of a WTP unit processes is the amount of water that can be effectively treated given typical design standards used in the industry. A WTP's firm capacity is the amount of water that can be treated by each of the unit processes with one unit out of service. Table 3-9 presents the firm and rated capacities for the Fall River WTP.

TABLE 3-9 FALL RIVER WTP FIRM & RATED CAPACITIES

Unit Process	Fall River WTP	
	Firm Capacity	Rated Capacity
Rapid Mix	13.8	21.3
Flocculation (at 20 min detention time)	13	>24
Sedimentation (at 0.5 gpm/sf)	13.3	20
Filtration (at 2 gpm/sf)	12.8	17
Disinfection		
Summer[1]	> 24	> 24
Winter[2]	> 17	> 17
High Lift Pumping	14	20

Notes:
Firm Capacity – Capacity of a WTP unit process that can be effectively treated give typical design standards used in the industry.
Rated Capacity – Amount of water that can be treated by each unit process with one unity out of service.

(1) Temp. = 15°C, pH=8.0, Cl_2 residual = 2.0, Baffling Factor = 0.7
(2) Temp. = 0.5°C, pH=8.0, Cl_2 residual = 2.0, Baffling Factor = 0.7

The estimated rated capacity of the Fall River WTP is 17 MGD. The firm yield assuming one unit process is out of service is 12.8 MGD. These data indicate that the WTP has the rated capacity to meet the City's estimated 2035 maximum day demand (MDD) of 14 MGD.

3.5.1 Pumping Capacity

Typically, during periods of high demand, the 12 MGD, the 6 MGD and one of the two 4 MGD pumps are running. Due to headloss in the distribution piping and changing distribution tank levels, the total pump discharge with three pumps operating (and one of the 4 MGD pumps out of service) is between 17 and 20 MGD. The DEP's Guidelines and Policies for Public Water Systems recommends that a system should be able to provide the maximum day pumping demand with the largest pump out of service. If the 12 MGD high lift is out of service, then the plant can still produce approximately 14 MGD which is the 2035 maximum day demand.

3.5.2 Major Unit Process Evaluation

As previously discussed, the City of Fall River's Water Treatment Facility, constructed in 1975-1976 was rehabilitated in 2004, utilizes conventional treatment technology to protect consumers from waterborne pathogens as well as for the purpose of color, suspended solids and contaminant removal from North Watuppa Pond and Copicut Reservoir. The primary unit processes that make up the treatment train are as follows:

- Rapid mixing of chemicals
- Flocculation
- Sedimentation
- Filtration
- Disinfection
- Corrosion Control
- Residuals management

The capability of each individual unit processes to provide consistent performance was evaluated to insure that they consistently provide an effective barrier to passage of microorganisms. As discussed previously, specific turbidity performance goals for sedimentation and filtration were evaluated. It was determined that the sedimentation process produced water with a turbidity much lower than the 1 to 2 NTU goal. However, while the finished water turbidity meets the requirements of the Long Term 1 Enhanced Surface Water Treatment Rule requirement of turbidity levels of less than 0.3 NTU 95% of the time, it is slightly higher than the recommended performance goal of 0.1 NTU. Also, the capability of the disinfection process to inactivate microorganisms is evaluated based on CT values outlined in the U.S. EPA Guidance Manual for meeting filtration and disinfection requirements. Since the plant's treatment processes must provide an effective barrier at all times, the maximum flow through the treatment plant was used to evaluate the effectiveness of treatment. The maximum flow represents those conditions where the treatment processes are the most vulnerable to the passage of cysts-and microorganisms. If the treatment processes are adequate at the maximum flow, then the plant is likely assured of providing the necessary effective barrier.

The maximum flow through the plant from 2002 to 2012 ranged from 11.56 to 18.23 MGD well below the WTP reported design capacity of 26 MGD. Over this period the MDD of 18.23 MGD occurred in 2005. The 2035 required maximum day plant flow is projected to be 14.2 MGD. Assuming that the backwash requirements are equal to 3% of the water production, the maximum day plant flow based on a 2035 system wide maximum day demand of 14.2 MGD plus 3% for backwash and sedimentation basin cleaning requirements is 14.6 MGD (14.2 MGD + 3% = 14.6 MGD).

The overall goal of the treatment plant and its staff should be to optimize the available treatment processes to achieve the best possible, effluent water quality, not simply to meet the MCLs and

Secondary Standards set by the regulations. Each unit process's capability was assessed using a performance potential graph where the projected treatment capacity of each major unit process was compared against the current and future maximum day demands.

3.5.2.1 Rapid Mix

There are three rapid mix basins. Each of the three basins is approximately 9 feet by 8 feet and has an approximate water depth of 10 feet. The volume of each rapid basin is approximately 5,400 gallons and therefore provides approximately 1 minute of detention time at the maximum design flow of 24 MGD.

3.5.2.2 Flocculation

There are three flocculation basins. Each basin is 55 feet long by 42 feet wide and operates at an average depth of about 10.5 feet. Flocculation capacity was rated at approximately 19.6 MGD based on the volume of the three parallel flocculation units, approximately 544,300 gallons, and a minimum hydraulic detention time of 40 minutes for cold water conditions. The rated capacity of the flocculation basins is adequate to meet estimated 2035 maximum day flow of 19.3 MGD. The warm weather flocculation capacity is 39.2 MGD, well above the future flow requirements, based on a hydraulic detention time of 20 minutes.

3.5.2.3 Sedimentation

The sedimentation capacity is based on surface overflow rate (SOR) with consideration given to water depth and sludge removal characteristics. Typical overflow rates for conventional rectangular settling basins range from 0.5 to 0.7 gpm/sf for basin depths between 10 and 12 feet. The sedimentation process was rated at a flow of approximately 19.9 MGD, based on a surface overflow rate of 0.5 gpm/sf. Each of the three basins is 230 feet long by 40 feet wide with an approximate water depth of about 10.5 feet. Flocculated water flows through a baffle wall into the settling basin. The water flows through the sedimentation basin where flocculated particles settle to the bottom of the tank. A chain and flight sludge collection system keeps the settled sludge from becoming packed on the bottom of the basins. The sludge waste is periodically discharged to a small lift station where it is pumped to a sewer line.

At the end of each basin, clarified water flows over a weir into a collection trough located around the perimeter of the basin. The trough directs the settled water from the three basins to a common channel which flows to the filter influent trough. The rating of the sedimentation process at 20 MGD is adequate to meet the projected 2035 maximum day flow of 14.6 MGD.

3.5.2.4 Filtration

The filters remove the remaining particle matter contained in the water prior to entering the clearwell. The four (4) automatic backwash (ABW) filters are dual-media (sand with anthracite cap), shallow bed, filters divided into multiple separate cells. Each of the filters measures 92 feet by 16 feet and was originally designed to contain 11 inches of a single media type and was upgraded in 2005 to a total of 16 inches with dual media, 8 inches of sand with an 8 inch anthracite cap. The typical loading rate for mono-media shallow bed filtration is 1 to 3 gallons per minute per square foot (gpm/sf). At a maximum flow of 24 MGD, the original filter design resulted in a loading rate of 2.8 gpm/sf. According to the manufacturer, ABW filters that have been upgraded to include 16 inches of dual media typical have loading rates of 2 to 3 gpm/sf.

The DEP's recommended guideline is that treatment facilities have the ability to meet the maximum day demand with the largest filter out of service. While it is listed in the DEP's guidelines, many treatment facilities do not have the ability to treat the maximum day flow with one filter off line and treatment facilities with only two filters rarely have the ability. Fall River's WTP can treat approximately 15.9 MGD with one filter out of service at a conservative loading rate of 2.5 gpm/sf. This is greater than the 2035 MDD estimate of 14.6 MGD. At a peak loading rate of 3.0 gpm/sf, the Fall River WTP can treat approximately 19 MGD with 1 filter out of service or 25.5 MGD with all 4 filters on line.

3.5.2.5 Disinfection

Disinfection requirements set forth by the Surface Water Treatment Rule are based on disinfection contact time (CT) values needed for inactivation of Giardia cysts and viruses. CT is the disinfectant concentration (C), in mg/L, multiplied by the actual time (T), in minutes that the water is in contact with the disinfectant. To establish the required CT, a treatment plant is required to provide a total of 3 logs (99.9 percent) of cyst inactivation. A well operated conventional treatment plant provides 2.5 logs of inactivation, therefore requiring 0.5 logs of inactivation through disinfection. Chlorine is typically added as the disinfectant at the clearwell following filtration. The plant also has the capability to add chlorine gas prior to water entering the rapid mix stage as a pre-oxidant, in the sedimentation basin and prior to filtration.

The 0.5 log inactivation requirement is achieved through the clearwell. The typical operating depth in the clearwell is approximately 7.0 feet, corresponding to a volume of approximately 500,000 gallons. A baffling factor is used to quantify the amount of short-circuiting that occurs in a given clearwell. A baffling factor of 1.0 represents perfect baffling (plug flow) whereas a factor of 0.1 represents un-baffled or mixed flow conditions. A Tracer Study, located in Appendix F, was conducted in 2006 to verify the baffling factor with the installation of the baffles in the clearwell during the 2005 upgrades. A baffling factor of 0.70 was used in the DEP's evaluation of the plant's disinfection capabilities. Therefore, a clearwell volume of 500,000 gallons multiplied by the baffling factor of 0.70 yields an actual effective volume of 350,000 gallons.

Disinfection through the clearwell is rated at a maximum capacity of 17 MGD during the worst case winter conditions. This flow is greater than the current and projected maximum day flow rates. Worst case conditions (i.e. the conditions during which the CT requirement is the highest) occur when the temperature is the lowest and the pH is the highest. Over the past three years, the minimum temperature of water in the clearwell was 0.5 degrees C (32.9 degrees F), and the maximum pH was 8.9.

During the summer months when the temperature is greater and the disinfection achieved is better due to the increased temperature. During these months, the formation of DBPs is also greater due to the warmer water temperature. To help reduce THM formation, the City has reduced the chlorine residual leaving the clearwell to achieve the required disinfection requirement but minimize THM formation. With the same pH of 8.9 and flow rate of 12 MGD a chlorine residual of less than 2.0 mg/L is required to achieve the required 0.5 log disinfection. Disinfection residuals should be monitored in the distribution system to ensure there is a residual in the outer extremities of the distribution system with this lower disinfectant concentration. In addition, if the pH adjustment chemical is moved to the end of the clearwell, a lower pH will provide better disinfection through the clearwell during all times of the year, reducing the amount of chlorine required to meet the disinfection requirements thus potentially reducing DBPs in the distribution system.

3.5.2.6 Residuals Management

Waste streams produced by plant operations such as the backwashing of filters and the sludge discharge from the sedimentation basins compose the majority of the treatment plant's residuals. Settled sludge is pumped from each sedimentation basin twice per week. Each sludge removal cycle lasts eight hours and the sludge from all three basins is typically removed over a consecutive 24 hour period. Sludge is pumped from the settling basin collection trough to a lift station located between the two waste settling lagoons. The lift station pumps the sedimentation basin waste from its small wetwell to the City’s sewer collection system. The waste stream produced by filter backwashing flows to the two clay lined lagoons, which are operated in series. Lagoon #1, closest to the plant, is approximately 110 feet long and 190 feet wide and has a water depth of approximately 5.5 feet. Lagoon #2 is approximately 120 feet long and 180 feet wide and has a water depth of about 4.5 feet. The sides are sloped at 3:1. The capacity of lagoons 1 and 2 are approximately 850,000 and 725,000 gallons, respectively. After the sludge settles out in each of the lagoons, the supernatant water is returned to North Watuppa Pond.

3.5.2.7 Summary

The Fall River WTP has consistently produced a high quality finished water. As a result of the recent implementation of the Stage 2 D/DBR the City has come out of compliance with the locational running annual average THM concentration at one sampling location.

The projected 2035 maximum day flow is estimated at 14 MGD. The plant will therefore be required to treat about 14.6 MGD including water required for filter backwashing and sedimentation basin cleaning. Table 3-9 presents firm and rated capacities for the Fall River WTP. The data presented in this table shows that the flocculation and sedimentation processes are both capable of meeting the projected maximum day flow. The capacity of the filtration system is limited to approximately 17 MGD with all filters in service assuming the design loading rate of 2.0 gpm/sf and 12.8 MGD with one filter off-line. These flow rates can be increased if the loading rate of the filters is increased. The ability to meet the disinfection requirements of the SWTR is limited to approximately 17 MGD under worse case winter conditions. The ability of the plant to meet the disinfection requirements at higher flow rates is possible following the relocation of the caustic soda feed location to the clearwell discharge.

3.6 PERFORMANCE LIMITING FACTORS

Performance limiting factors in the areas of design, operation and maintenance were identified based on performance and design assessments and special studies conducted at the treatment plant as well as information obtained from plant operations staff. Each of the factors were classified as A, B, or C according to the following guidelines:

A Major effect on a long term repetitive basis.
B Minimal effect on a routine or major effect on a periodic basis.
C Minor effect.

The factors identified were prioritized (A, B or C) as to their relative impact on performance and are summarized with estimated construction costs as follows. The cost estimates are preliminary and are subject to change based upon more detailed investigations of hydraulic constraints, space restrictions, impact on existing utilities, maintaining treatment plant operations during construction and access into the facility. The cost estimates include 30% for engineering and contingencies.

3.6.1 Chemical Feed Relocation (A)

According to discussions with plant personnel, the original plant design provided for the injection of chlorine gas was directly into the high lift discharge main. Since original construction of the plant in 1975-1976, the application point for post-chlorination was relocated to the clearwell to provide sufficient detention time to meet the disinfection contact time (CT) requirements of the SWTR. Following the water treatment plant upgrades in 2005, the chlorine was injected at the entrance of the clearwell following the filters. In addition, the clearwell has baffle walls to encourage a serpentine flow pattern to increase detention time.

Currently, sodium hydroxide is used to raise the effluent pH at the end of filter effluent channels prior to the entrance of the clearwell. Disinfectant is not as effective at a higher pH. There are existing injection pipes allowing for the injection of sodium hydroxide into the effluent of the clearwell prior to the high lift pumps. These pipes may be connected at the finished floor to provide better disinfection at a lower pH instead of addition at the entrance of the clearwell. This will also assist to reduce the DBPs formed in the distribution system due to less chlorine addition.

This alternative would provide better disinfection in addition to reducing the amount of chemical required for disinfection. If the plant personnel conducted the work to move the chemical injection the cost would be minimal. Stub pipes located in the pump room will require a small amount of piping to connect and relocate the sodium hydroxide addition.

3.6.2 SCADA System and Process Control Equipment (B)

The Supervisory Control and Data Acquisition (SCADA) system was installed in 2005 after the last round of upgrades to the water treatment plant. These facilities include but are not limited to: on/off status of pumps and plant processes; plant clearwell and distribution tank levels;

distribution system flow and pressure; and treatment plant water quality parameters such as pH, turbidity, chlorine residual, particle counts, streaming current etc.

The existing SCADA system is in need of upgrades including data logging and telemetry. The existing system's telemetry has been unreliable and therefore the SCADA system is not receiving consistent data from the storage tanks. Also, telemetry was never implemented for the Copicut Pumping Station. The telemetry would allow the operators to control the height and turnover of the water storage tanks reducing water age in the process. Without reliable telemetry they are not able to see if the water storage tank is turning over at all, which may lead to high water age, chlorine residual reduction and DBP formation.

Also, SCADA systems provide operators and managers with computer generated water quality reports required by the State and the ability to monitor and/or control the system remotely via any networked computer terminal. The existing software is out of date and the more recent software has additional reporting capabilities that would assist in automating the State required reports. The cost to upgrade the reporting software is approximately $15,000. The cost to update the SCADA system is approximately $35,000. The total cost for upgrading the system is $50,000.

In addition to a SCADA system upgrade, the City's telemetry systems need to be completed or replaced with a new more reliable system. Recently the City has attempted to implement a cellular radio system to transmit the Chicago Street Tank elevation and several signals form the Hood Street Tank site. The cellular system has not been reliable and final connections to the SCADA system are incomplete. One recommendation is to install a radio spread spectrum system throughout the water system at a budgetary cost of $250,000. The City's waste water collection and treatment system utilizes a remote site via Comcast cable, which would be an optional recommendation for the water system at the same budgetary cost.

3.6.3 Annual Budget to Replace Aging Equipment/Maintain Building (B)

FST performed an inspection of the water treatment facility on December 19, 2013, with Mr. Michael Griffin, Plant Operator at the Water Treatment Plant. The walk-thru inspection included a review of the interior and exterior of the treatment building and associated structures. A general review and discussion was conducted and the interior and exterior conditions were assessed. A detailed summary of FST's walk through inspection is included as Appendix G.

Funds should be appropriated annually for the replacement of aging equipment such as valve actuators, flocculator drives, pumps, roofing, windows and doors, heating systems and furniture and technology, and other plant equipment. Replacement or repair of the noted items before the end of their useful life and failure would be in the best interest of the City to limit the possibility of unanticipated problems. The City currently has an annual maintenance budget of approximately $25,000 for the plant and its grounds. To allow more flexibility with the scheduling of these types of repairs and equipment replacement, the City should consider increasing the annual budget to $200,000.

The four items listed below are examples of projects that the City may wish to consider as part of their annual maintenance budget expenditures.

3.6.3.1 Filter Media Replacement (B)

The media was replaced in the filters as part of the 2005 plant upgrades. The overall depth of the filter media was increased from 11-inches to 16 inches with 8 inches of sand covered by 8 inches of anthracite. The anthracite cap was installed in 2005 into the filters and as a result of normal backwash practice some anthracite has been lost. The filter media has an estimated 10 year life span and has not been replaced since 2005. The media is reaching the end of its useful life and should be replaced to prevent any issues in the future in meeting effluent water quality goals. The media should be replaced approximately every 10 years in addition to periodic topping off of the anthracite media cap. Prior to replacement, the filters should be cored to determine the amount of media remaining as well as the uniformity coefficient, and filterability of the remaining media. The cost of media replacement with 8 inches of sand and 7 inches of anthracite is $125,000 including delivery, installations and removal of old media.

The underdrains and traveling bridge backwash system were replaced in the 2005 upgrades. This infrastructure is within its useful life. The City should continue to monitor operating in automatic mode instead of manual which initiates a backwash based on time instead of turbidity levels. Backwashing based on turbidity levels will reduce the amount of excess water used for unnecessary backwashing.

3.6.3.2 Turbidimeters and Monitoring Equipment (A)

The monitoring equipment including residual chlorine analyzers (1), pH meters (4), turbidimeters (4), streaming current (1) and particle counters (2) were installed during the 2005 plant upgrades. Some parts of the monitoring equipment will need to be replaced and calibrated on a regular basis to maintain regulatory compliance. These items include pH probes (replace when they no longer calibrate, anticipate 1 per year), turbidimeter bulbs (annually), and chlorine analyzer 1 (approximately every 10 years). Maintenance for the analytical equipment should be conducted weekly, monthly or quarterly, based on the manufacturer recommendations to ensure proper operation. Reagents and calibration standards are required for the turbidimeters and chlorine analyzers (replace on a monthly basis) for calibration and replacement. Based on continuous use, the annual reagent and calibration costs are approximately $5,000 each year.

3.6.3.3 Rehabilitation of Low Lift Pumps (B)

The motors and VFDs were replaced and installed on the low lift raw water pumps during the 2005 plant upgrades, the pump impellers, shafts, screens and bearings have not been replaced on all of these pumps. The plant personnel conduct maintenance on these pumps originally from 1975 on a regular basis. Based on the expected maintenance requirements of this type of pump, it is likely that these pumps should be replaced. The City recently obtained a budgetary cost estimate from a pump supplier for the materials required to replace the bowl assemblies, shafting and stuffing boxes on the four pumps. The labor required to install the new equipment will be performed by City personnel. The cost of materials alone was quoted at approximately $100,000.

Two of the four raw water low lift pumpvariable frequency drives are not operable and are being operated with the soft-start bypass. This limits the automatic operation of these pumps and restricts the automatic control of raw water into the treatment facility. The VFD's for these two pumps should be replaced in the near future. A budget of approximately $50,000 should be allocated for this purpose.

3.6.3.4 Finished Water High Lift Pumps (A)

There are four (4) high lift pumps that deliver finished water from the treatment plant out into the distribution system. In 2013 FW#2 was refurbished but the other three pumps have not been addressed since 1991. A budgetary cost of $325,000 should be utilized for the refurbishment or replacement of FW #1, #3, and #4.

3.6.3.5 Training for Plant Personnel (B)

The City currently has approximately $3,000 annually budgeted for educational assistance for the operators, watchmen and laboratory personnel. The City may wish to consider implementing a training program for employees who seek to further their education and training. Allowing, encouraging and paying for staff members to participate in seminars, trade shows and other educational programs will help affirm the City's commitment to its employees. In turn, the employees will benefit by continuing their education and keeping up to date in the industry's latest technologies and practices for providing consumers with the highest possible water quality. An training program may require an annual budget of approximately $10,000.

3.6.4 Residuals Disposal and Management (A)

The majority of waste produced by the WTP is from sedimentation basin cleaning and filter backwashing operations. Sludge from the sedimentation process is directed to a small pump station where it is pumped to the City's wastewater collection system. Filter backwash waste flows to two clay lined lagoons located immediately north of the plant. From these lagoons, the supernatant water flows to North Watuppa Pond while the settled sludge accumulates in each of the lagoons.

The existing pump station was originally installed prior to 1980 and new pumps were installed in 1983 to increase capacity. The pump station has reached the end of its useful life. There are two 30 HP Smith & Loveless vertical turbine pumps in the station with, according to WTP personnel; they frequently have to remove one or both of the pumps for maintenance.

The City's existing pump station has an 8-inch pipeline that connects to the collection system. Portions of the pipeline are above ground and experiences freezing in the winter months. The pipeline also travels through an active culvert that travels under Route 24. This portion of pipeline has experienced significant leaks when the pump station kicks on in addition to freezing. The WTP personnel have discontinued use of the pump station during the winter months due to these freezing issues.

The Plant now sprays down the sludge in the lagoons with finished water and using portable mud pumps to send the sludge to the pump station for ultimate disposal into the collection system. This method uses additional potable water and is very labor intensive for the plant personnel.

According to plant personnel, the sludge has a solids content of approximately 2% to 4%. If the solids content of the sludge were increased prior to disposal, the cost of removal and hauling would be substantially reduced. A Residuals Management Study would indicate methods the City could utilize to decrease the costs associated with their residuals handling and disposal. These methods may include items such as changing the current series operation of the two lagoons to alternation of lagoons on an annual basis to provide batch dewatering of the sludge. Supernatant water could be directed to the pond as is currently done but no additional water would be added other than natural precipitation, therefore allowing the sludge to dry further before removal. Supernatant water could also be pumped to the head of the plant. Another option available to the City would be to install mechanical sludge dewatering equipment to provide a much drier cake product prior to shipment, thus greatly reducing the water weight of the waste and associated hauling cost. The cost of conducting a residuals management study is estimated at $50,000. In addition the City should consider replacing the current residuals pump station to increase system reliability. The estimated cost to replace this station is approximately $750,000.

The City will also have to maintain a National Pollutant Discharge Elimination System (NPDES) permit for discharge of lagoon supernatant water into the reservoir. The permit will increase monthly certified laboratory samples for compliance sampling.

3.6.5 Backwash Segregation and Recycling (B)

A large amount of water is wasted through backwashing the filters. A large portion of this water could be returned to the front end of the plant through a backwash recycle system. The system would allow any solid material in the backwash water to settle out while the clean water would be returned to the treatment system. The settled material would then be sent to the lagoons or sewer. The amount of water wasted through backwashing is almost 68,000 gallons per day. In addition to wasting water this amount of flow is overflowing the lagoons to prevent them from operating properly. The cost of installing a backwash recycle system is estimated to be $2.0 million.

3.6.6 Emergency Reserve Fund (B)

An emergency reserve fund of $350,000 should be established for repair or replacement of critical plant operating equipment in the event of unanticipated failure. These parts may include but are not limited to finished and raw water pumps, chemical feed control stations and pumps, Variable Frequency Drives (VFDs) and electrical equipment, sedimentation basin chains and flights, filter media and backwash equipment, on-line analytical equipment and SCADA issues.

3.6.7 Convert from Gas to Liquid Chlorine (C)

The existing chlorine gas system was upgraded from pressure to vacuum in 2005. The chlorine residual and injection have maintained proper operation since the upgrade.

The City may wish to consider switching from gas chlorine to liquid sodium hypochlorite. Safety concerns associated with the liquid product are much less than those for gas systems and do not require the regular filing of a Risk Management Plan. Based on an initial review of the area available, the bulk liquid tanks could be installed in the existing chlorine storage room and therefore no addition to the WTP would be required. The cost of switching from gas to liquid chlorine is estimated at approximately $600,000. Considerations would need to be made to provide a temporary chlorine feed system during the conversion.

3.6.8 Electrical System Upgrade

In 2013 the City completed electrical upgrades to the Water Treatment Plant including new electrical switchgear for the finished water pumps, new main service breaker and new manual transfer switch. There are additional electrical upgrades required for the facility. All lighting that was not replaced as part of the 2005 upgrades should be replaced along with new wiring and receptacles. The cost for these upgrades is estimated at $125,000.

3.6.9 Conclusions and Recommendations

Based on the performance limiting factors, Table 3-10 summarizes the recommendations for the Treatment section. The recommendations include minor improvements to the water treatment plant to optimize the existing unit processes as well as additional evaluations to help reduce annual operation and maintenance costs.

TABLE 3-10 TREATMENT PLANT IMPROVEMENT RECOMMENDATIONS

Item No.	Improvement Recommendation	Estimated Cost
1	Chemical Feed Relocation	$0
2	SCADA System and Process Control Equipment	Software $15,000 Hardware $35,000 TOTAL $50,000
3	Telemetry Upgrade	$250,000
4	Annual Budget to Replace Aging Equipment/Maintain Building	$200,000/yr
5	Filter Media Replacement (materials, delivery, and installation)	$125,000/10 yr
6	Turbidimeters and Monitoring Equipment	$5,000
7	Rehabilitation of Low Lift Pumps & new VFD's	$150,000
8	Refurbish or Replace Three High Lift Finish Water Pumps	$325,000
9	Training for Plant Personnel	$10,000/yr
10	Residual Disposal and Management	Residuals Study $50,000 Lift Station $750,000
11	Backwash Segregation and Recycling	$2,000,000
12	Emergency Reserve Fund	$350,000
13	Convert from Gas to Liquid Chlorine	$600,000
14	Electrical System Upgrades	$125,000
	CAPITAL COST TOTAL	$4,600,000
	ANNUAL COST TOTAL	$227,500

3.7 ADDITIONAL ALTERNATIVES & TECHNOLOGIES

The relatively minor treatment plant modifications listed in the previous section will help to optimize treatment plant performance by eliminating or reducing the severity of the performance limiting factors identified during the facility investigations. In addition to making the modifications named previously to optimize plant performance, several additional alternatives have been identified to:

- increase protection from pathogenic microorganisms such as C*ryptosporidium*,
- increase removal of natural organic matter to minimize formation of disinfection by-products following chlorination
- minimize color, taste and odor in the finished water
- allow plant staff more flexibility with the general operation of the facility
- increase disinfection residual in the distribution system to reduce the potential for microbial contamination
- increase corrosion control measures to reduce lead and copper in the distribution system

The following alternatives will address one or more of these issues:

- Ozonation
- Membrane Filtration
- Multi-Media or GAC Filtration
- Ultraviolet (UV) Disinfection
- Ozonation, GAC Filtration and UV Disinfection
- Ion Exchange Resin, MIEX® addition
- Ion Exchange Resin followed by Membrane Filtration

A description of each of these alternatives follows with estimated costs. The cost estimates are preliminary and are subject to change based on more detailed investigation of hydraulic constraints, space restrictions, impact on existing facilities, maintaining treatment plant operations during construction, access into the facility and residuals handling requirements. The capital cost estimates include 30% for engineering and contingencies. Piloting of selected process(es) is required to determine if water quality goals will be satisfied, to verify equipment sizing, and to verify capital, operation and maintenance costs.

3.7.1 Ozonation

Chemical oxidation is a destructive technique where a strong oxidant is applied to chemically modify objectionable compounds into less objectionable byproducts. Among the common oxidants are chlorine, potassium permanganate and ozone. Ozone, an unstable form of oxygen, is a stronger oxidant than chlorine and is also the most powerful disinfectant. It can be used to effectively remove taste, odor and color from the raw water supply as well as for purposes of disinfection where the CT values required for inactivation of Giardia and viruses is the lowest of the disinfectants, significantly less than chlorine. Addition of ozone prior to the rapid mix process would provide the benefits of oxidation of color, and taste and odor causing compounds and act as a microflocculant, possibly reducing the current coagulant dose. Another benefit would be replacing chlorine as the pre-oxidant thereby reducing disinfection by products. Another option is intermediate ozonation. This involves adding ozone prior to the filters and may be considered if new deep bed filters are added to the treatment train.

The difficulty with using ozone is related to the generation and handling of the chemical. Ozone is a gas and must be generated on-site. It is very unstable at ambient temperatures and pressures and reverts back to oxygen from which it was formed at temperatures above 35 degrees Celsius. For this reason, it cannot be manufactured at a central manufacturing plant and transported to the site. Its characteristic odor can be detected by most humans at 0.02 ppm, far below levels of acute toxicity.

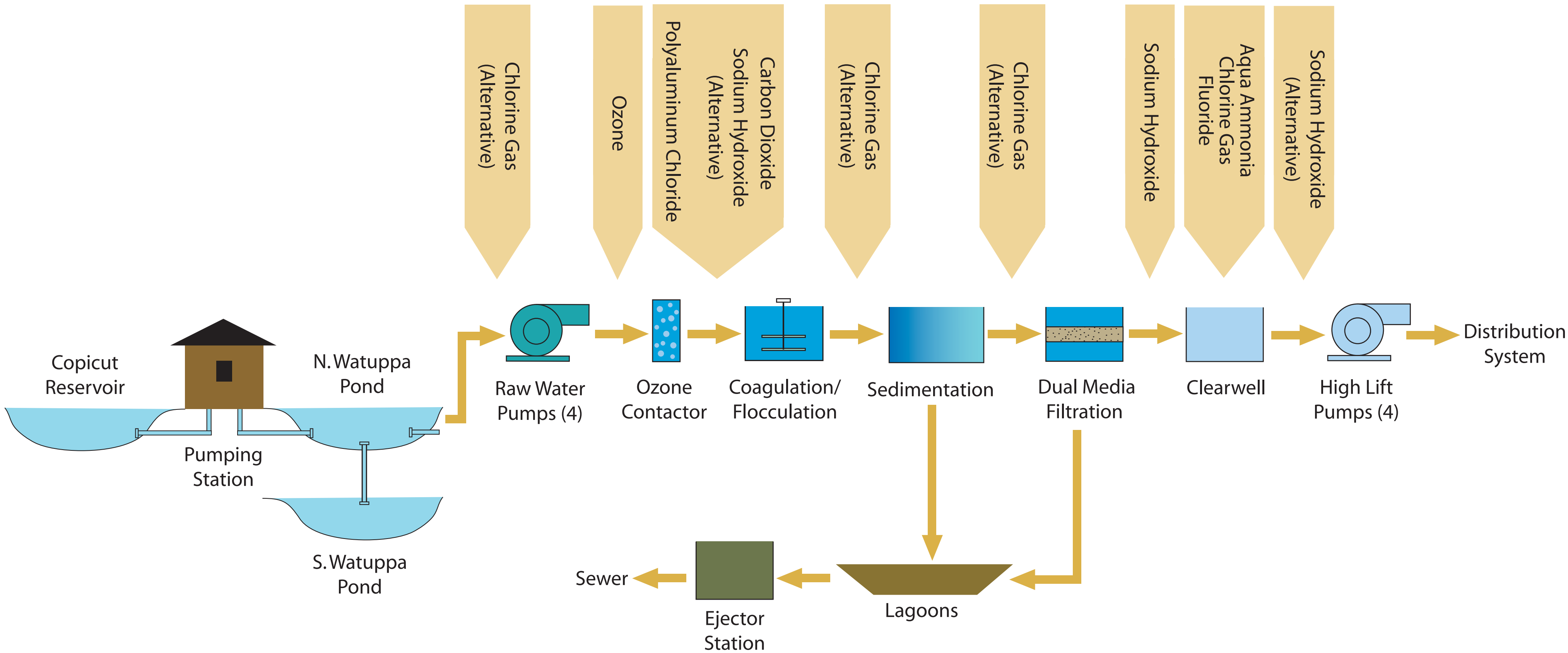
The major components of the ozonation process include gas preparation, power supply, ozone generation, ozone contacting and exhaust gas destruction. Ozone is generated by drying and cooling ambient air, oxygen enriched air or pure oxygen; and passing it through a generator where conversion to ozone occurs by exposing it to high voltage potential from a power supply. The ozone is then transferred to the water by injecting the gas at the base of a contact tank or injecting the gas into a motive water line, which is then injected into a contact tank. Excess ozone that did not react with the water must be destroyed before release to the atmosphere.

The effectiveness of ozone will vary depending upon characteristics of the source water quality.

The advantages of ozone are its power as an oxidant and its ability to achieve other treatment objective such as primary disinfection, oxidation of iron and manganese, and enhanced coagulation. Disadvantages of ozone include high capital and energy costs and the formation of assimilable organic carbon (AOC) which promotes bacteria regrowth in the distribution system. Other compounds, such as organic halides, heptachlorepoxide and aromatic polymers can be formed when certain compounds present in the raw water react with ozone.

As stated previously, contact tanks are required to provide an efficient medium for the transfer of ozone gas to the water supply to meet disinfection requirements. Ideal contact tank configurations for high ozone transfer efficiency are multiple stage countercurrent concrete structures with water depths from 16 to 20 feet.

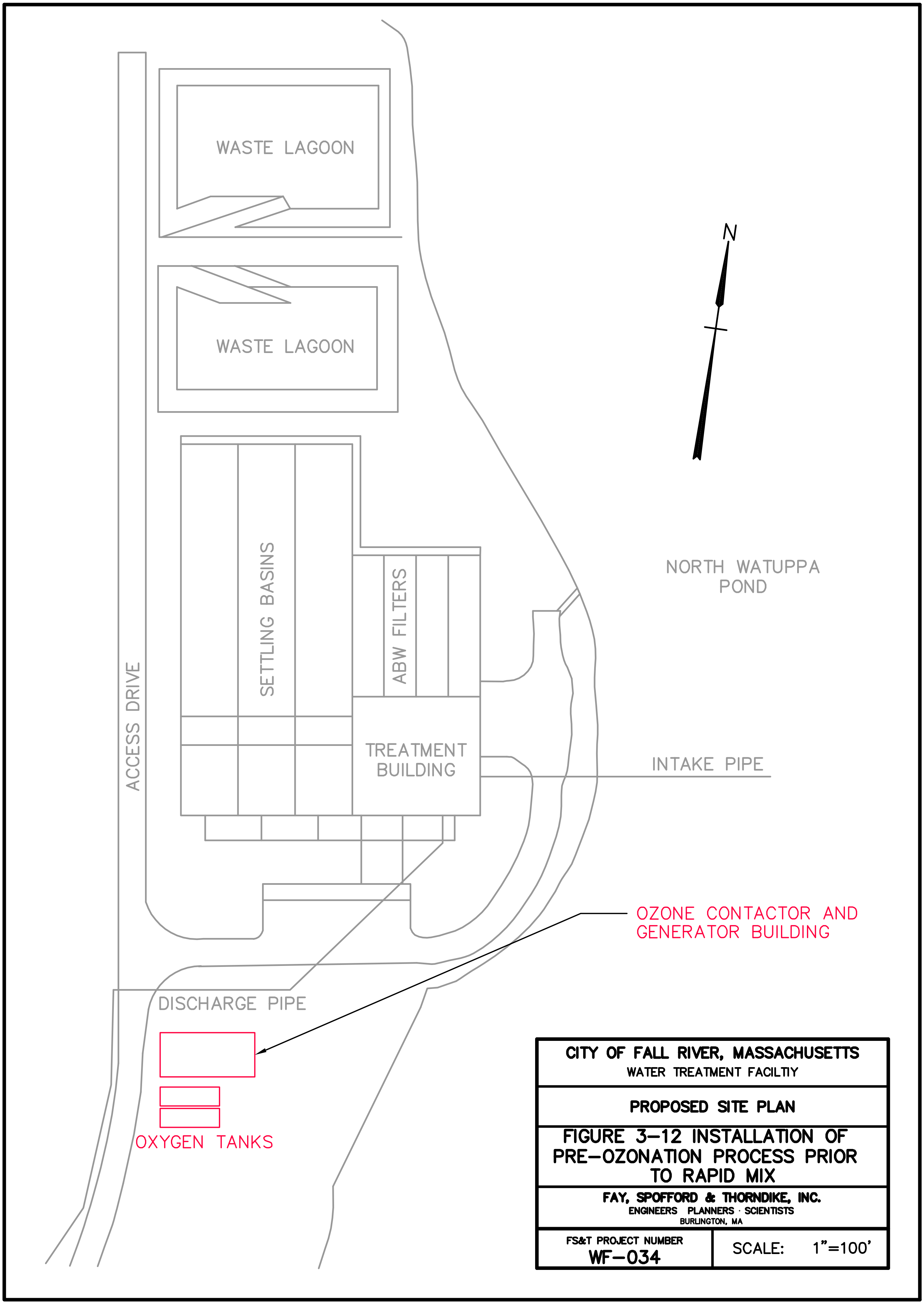
In order to provide 10 minutes of detention time at a flow of 24 MGD, a 165,000 gallon tank will be needed. At a water depth of 20 feet, and assuming two identical tanks would be constructed, the dimensions of each tank would be approximately 11 feet by 50 feet. The estimated cost of an ozone system, including generators, ozone destruct units, contact basins and an enclosure for the equipment is $4.0 million. Based on an average flow rate of 11.4 MGD and an ozone dose of 3 mg/L, the annual O&M cost of the system is estimated to be approximately $240,000. Figure 3-11 presents a treatment process schematic indicating the ozone injection location in the overall treatment train. Figure 3-12 presents a potential site layout for oxygen storage and ozone generation and contact equipment.



Fall River Water Supply and Treatment Plant Schematic
Ozone Addition

FST Fay, Spofford & Thorndike 100 YEARS

Figure 3-11


WASTE LAGOON
WASTE LAGOON
N
SETTLING BASINS
ABW FILTERS
NORTH WATUPPA
POND
ACCESS DRIVE
TREATMENT
BUILDING
INTAKE PIPE
OZONE CONTACTOR AND
GENERATOR BUILDING
DISCHARGE PIPE
OXYGEN TANKS
CITY OF FALL RIVER, MASSACHUSETTS
WATER TREATMENT FACILTIY
PROPOSED SITE PLAN
FIGURE 3–12 INSTALLATION OF
PRE–OZONATION PROCESS PRIOR
TO RAPID MIX
FAY, SPOFFORD & THORNDIKE, INC.
ENGINEERS PLANNERS · SCIENTISTS
BURLINGTON, MA
FS&T PROJECT NUMBER
WF–034
SCALE: 1"=100'

3.7.2 Membrane Filtration

The use of membrane filtration is becoming much more prevalent in the water industry as the quality of existing water supplies diminishes and the need for potable drinking water increases.

The application of membrane filtration at Fall River's WTP would reduce organics that act as precursors to disinfection by-products, provide a physical barrier to Giardia and Cryptosporidium, and possibly reduce concentrations of other contaminants that may be present or appear in the future in the source water. The effectiveness of membrane filtration for removal of various contaminants is dependent upon the type of membrane used. Table 3-11 presents the various types of membrane filters available and the pore sizes of each.

TABLE 3-11 MEMBRANE FILTER PORE SIZE RANGES

Type of Filtration	Size Range of Removal (µm)
Micro-filtration	0.07 - 8.00
Ultra-filtration	0.001 - 0.09
Nano-filtration	0.0002 - 0.003
Reverse Osmosis	0.00003 - 0.0006

Crypotosporidium cysts typically range from 4 to 8 µm and can therefore be removed utilizing any of these filtration processes. However, if the membranes will be required to remove organic precursors, pesticides or other larger contaminants, then nano-filtration or reverse osmosis membranes are recommended.

The estimated capital cost of a Nanofiltration system is estimated to be $18.0 million. The annual operation and maintenance are estimated to be $1.4 million.

Immersed membrane ultra-filtration filters could be installed in the latter portion of the existing sedimentation basins, with chemical addition, rapid mix and flocculation and sedimentation as the pretreatment. The existing ABW filters would no longer be required. The suspended solids content and size distribution found in the sedimentation basin would be acceptable for an immersed membrane system, where water is drawn, via a vacuum system, into the membrane fibers. Concentrate (waste) flow would be directed to the existing sludge lagoons and/or sewer while the permeate (filtered water) would be piped, possibly through the existing filtration building, to the existing clearwell. A blower system would be required to keep the fibers moving so as to prevent suspended particles from building up on the outside of the membrane fibers.

Also, as with conventional media filters, the membranes will periodically require backpulsing. Based on the water quality that would be entering the membranes, a recovery of over 95% is expected. If required, the waste stream could be filtered through another membrane where an additional recovery of about 75% could be achieved. Therefore, the total waste stream from the membrane system will be approximately 1.5% of the total throughput. The exact percent recovery would be determined by piloting.

There are three (3) potential installation options for membrane systems at the WTP:

1. Direct Membrane Filtration with a new ZeeWeed®500 system
2. Retrofit of the existing Sedimentation Basin with a ZeeWeed®1000 system
3. Retrofit of the existing Filters with a ZeeWeed®1000 system

Each of the membrane systems requires 8 trains with 5 cassettes per train. Each system has spaces for 6 cassettes, allowing for almost 20% additional spare space. The manufacturer also recommends a Clean In Place (CIP) system which includes tanks, analytical equipment, chemical injection and valves for automatic cleaning. The different styles of membranes proposed are for the application. The 1000 series is able to handle more solids that would be present in the sedimentation basin and potentially in the filtration area. The 500 series would only be installed if the upstream systems were replaced so as not to damage the 500 series membranes.

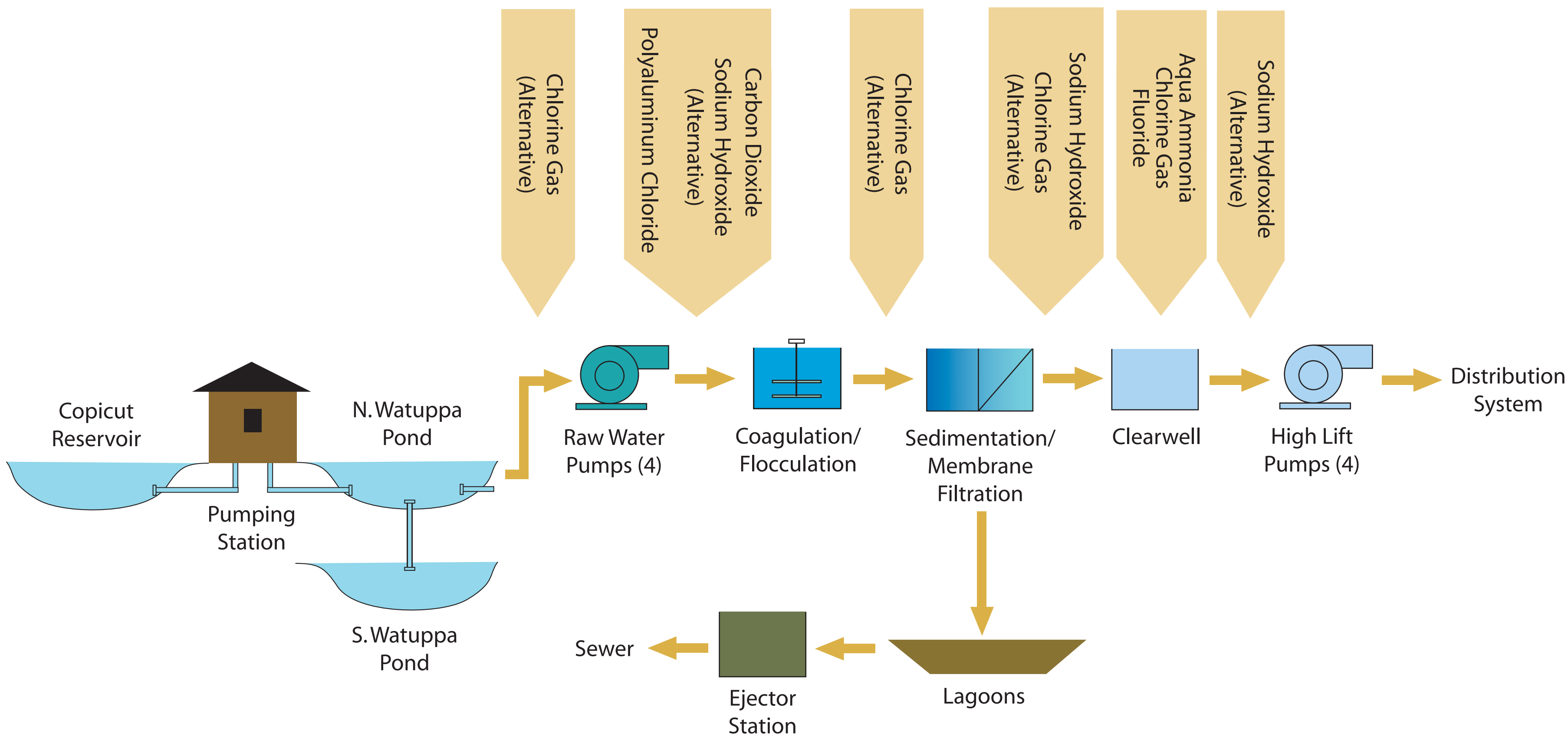
Each of the equipment costs are listed Table 3-12 below including the delivery costs to Fall River, MA.

TABLE 3-12 MEMBRANE EQUIPMENT COST

	Membrane Equipment Cost	CIP System Equipment Cost	Total Cost
Single ZeeWeed®500	$11,000,000	$9,800,000	**$20,800,000**
Retrofit Sed Basin ZeeWeed®1000	$8,200,000	$9,800,000	**$18,000,000**
Retrofit Filter ZeeWeed®1000	$8,500,000	$9,800,000	**$18,300,000**

If membranes were installed in the existing sedimentation basins, a building approximately 130’ x 110’ would be required over the basins to house the equipment. Two (2) 12’ x 50’ membrane cartridges are required in each of the three settling basins. The remainder of the space would be required for the pumping equipment, blowers, membrane cleaning chemicals and other ancillary equipment. In addition to the capital cost, an estimated cost of $460,000 will be required to operate the system on an annual basis.

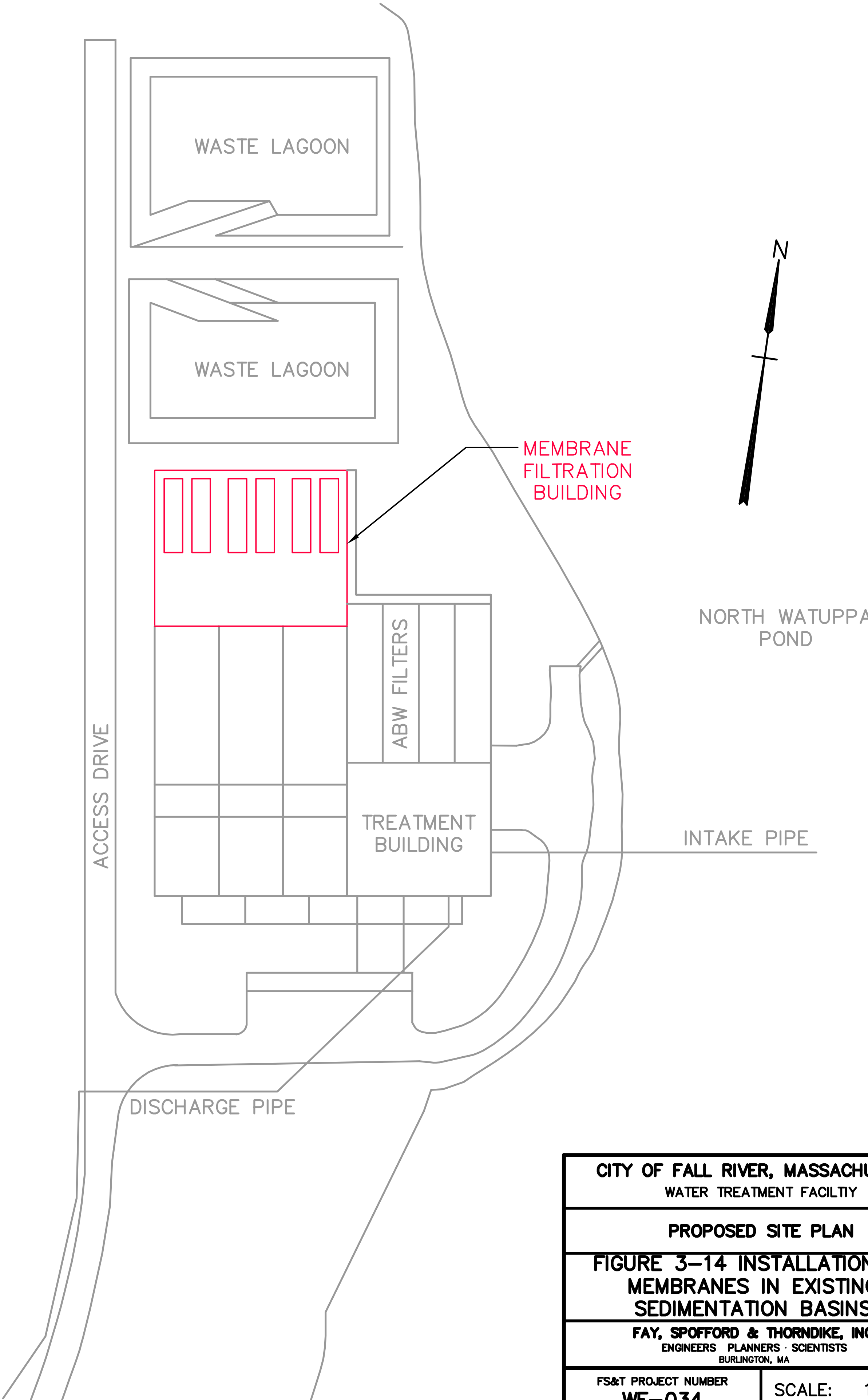
Figure 3-13 presents a process flow schematic indicating the location of membrane filtration in the overall treatment train. Figure 3-14 depicts a potential site plan including the potential location of a new membrane filtration facility.



Fall River Water Supply and Treatment Plant Schematic
Membrane Filtration Retrofit in Sedimentation Basins

FST FAY, SPOFFORD & THORNDIKE 100 YEARS

Figure 3-13



CITY OF FALL RIVER, MASSACHUSETTS
WATER TREATMENT FACILTIY

PROPOSED SITE PLAN

FIGURE 3–14 INSTALLATION OF MEMBRANES IN EXISTING SEDIMENTATION BASINS

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FS&T PROJECT NUMBER WF–034	SCALE: 1"=100'

3.7.3 Deep Bed Filtration

The ABW filters were upgraded in 2005 with new dual media and backwash systems. The filter media is reaching the end of its useful life and all media should be replaced in 2015. The cost for the new media is $60,000. The filter system is in good working condition, however, the existing shallow filters (16 inch of dual media) does not provide the level of protection against contaminant breakthrough as a modern deep bed filter and would benefit from additional filter depths as discussed below.

3.7.3.1 Conventional Deep Bed Filtration

Conventional deep bed filtration could consist of either a multi-media design or a deep single media design. Multi-media filters could be constructed with two or three layers made up of anthracite, sand and garnet. The total media depth for multi-media filters is typically between 3 and 4 feet, providing more effective solids removal and increased solids loading capacity over shallow bed mono-media filters. Also, the loading rate of multi-media filters is expected to be 5 gpm/sf, therefore requiring much less total filter area than shallow bed sand filters. Another option is to install a 6-foot deep bed of higher effective size anthracite media. The deeper single media bed provides a much more reliable and effective barrier for microbial contamination.

3.7.3.2 Granular Activated Carbon Deep Bed Filtration

Another alternative filtration process is granular activated carbon (GAC) filtration. In addition to particle removal, GAC also removes contaminants from the water using adsorption which is based on the transfer of contaminants from a solvent, in this case water, to the surface of the adsorbent, GAC. Due to the significantly larger surface area of GAC media as compared to conventional sand and anthracite coal, the biological activity is increased which also serves as a mechanism for organic removal. The efficiency of the process is affected by the water quality characteristics, whether the GAC is used as the primary filter or as a post filter adsorber (following existing filters), and the loading rate.

In addition to contaminant characteristics such as molecular weight and shape, polarity, and solubility, the physical and chemical qualities of the water also affect GAC adsorption efficiency. If the contaminant targeted for removal is in competition for adsorption sites with other adsorbates and dissolved solids, removal efficiency is reduced. The efficiency of the GAC is also dependent upon the loading rate or empty bed contact time (EBCT) which is defined as the amount of time that the water is in contact with the GAC. The EBCT required for taste and odor control is between 5 and 10 minutes while the optimum EBCT for the removal of organics is considered to be between 10 and 20 minutes. Based on a minimum EBCT of 10 minutes, a total volume of about 22,300 cf would be required to treat a flow of 24 MGD. A media depth of approximately 6 feet with a surface area of about 3,700 sf would be required which is equivalent to a loading rate of about 4.5 gpm/sf.

3.7.3.3 Summary

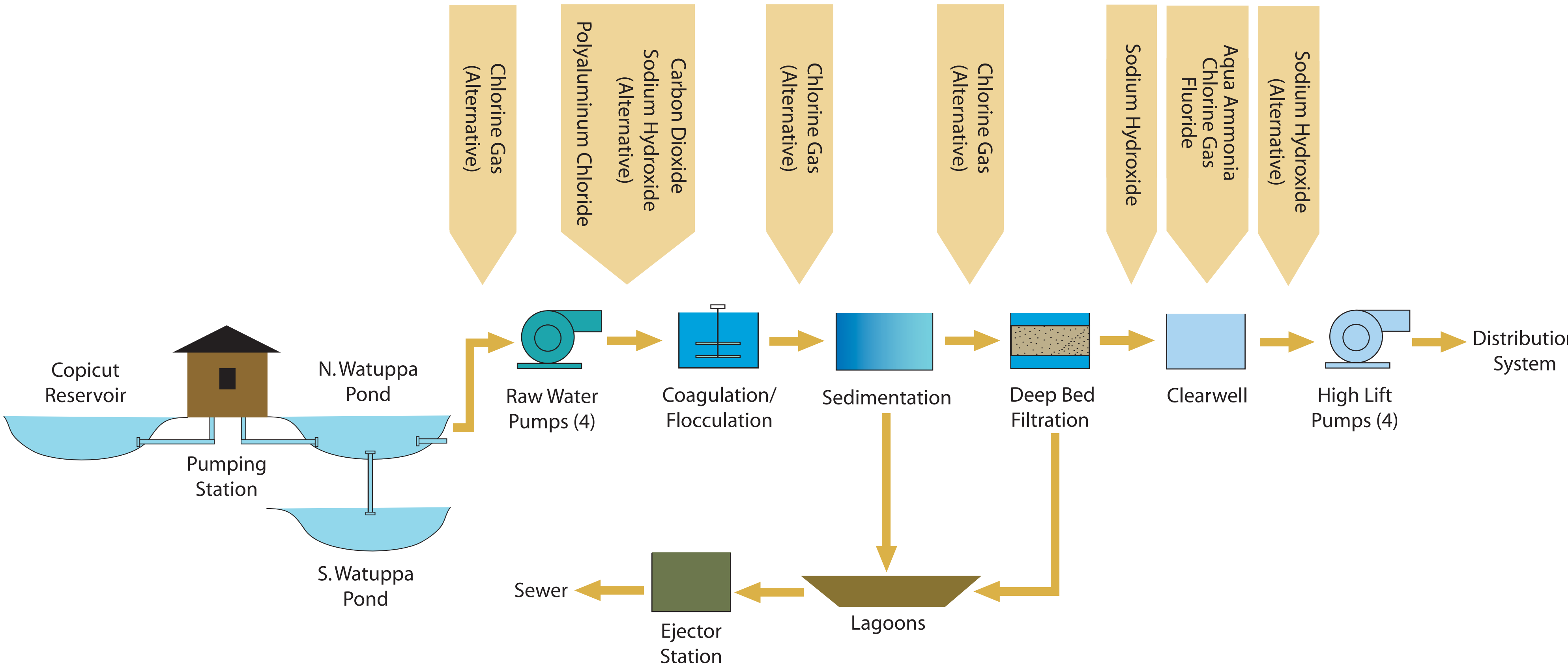
One disadvantage of deeper media, either multi-media, deep bed single media or GAC, would be increased headloss. With the current plant configuration and elevations of the clearwell and sedimentation processes, additional headloss through the filtration stage would require an intermediate pumping stage.

The new filters would be designed such that the existing water level in the sedimentation basins would remain constant. However, the effluent of the filters would need to be at a lower elevation than the existing ABW filters in order to provide enough driving head to push water through the media. As a result, a second clearwell and intermediate pumping equipment would be required to transfer water from the filters to the existing clearwell. One benefit of installing a new clearwell, pumping facility and filtered water transfer main is that the volume of water in this facility could be used to meet the disinfection contact time requirements of the SWTR. A disadvantage is the additional electrical costs associated with the pumping facility as well as dewatering costs during construction.

As an alternative, a pumping facility could be installed at the effluent of the sedimentation process to pump water up to the new filtration building. The filtration building would then be constructed at an elevation where the filter effluent could flow by gravity to the existing clearwell. This layout would not rely on a relatively long gravity main from the sedimentation process to the filter influent and would allow the new filtration building to be built at a higher elevation, thus possibly reducing construction costs.

The cost of constructing an intermediate pump station and separate filter building utilizing deep bed filtration is estimated at approximately $18 million. Multi-media filtration would require media replacement about every 12 years at an estimated cost of approximately $600,000. GAC media, when used as a primary filter may require replacement every 2 years at a cost of approximately $1,110,000. Additional power consumption will be required at an estimated cost of approximately $110,000 per year for the operation of the intermediate pumping equipment, backwash pumps and air scour blowers.

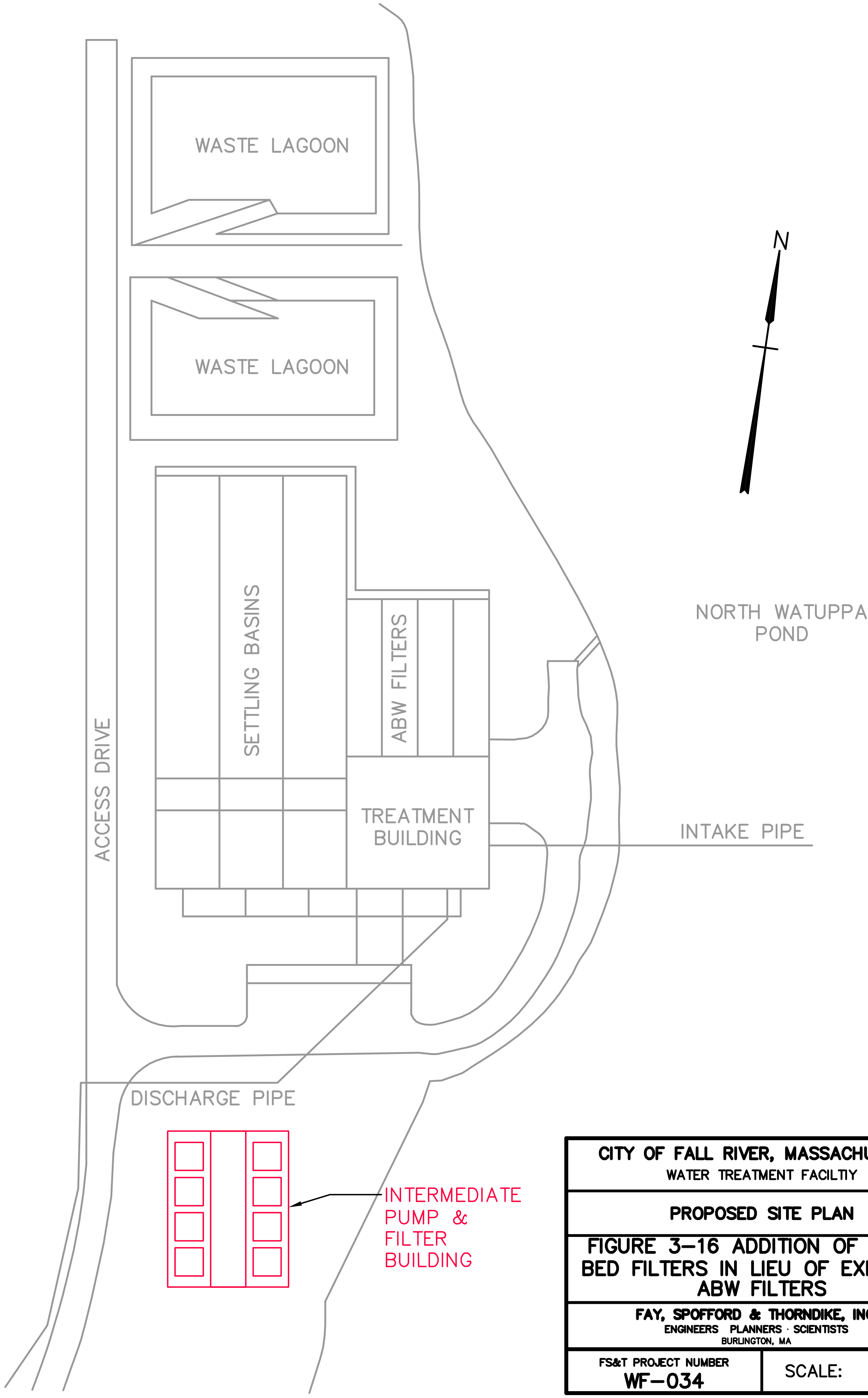
A schematic and site plan showing a potential location of an intermediate pump and deep bed filtration building are presented in Figure 3-15 and Figure 3-16, respectively.



Fall River Water Supply and Treatment Plant Schematic
Addition of New Deep Bed Filters

FAY, SPOFFORD & THORNDIKE

Figure 3-15



CITY OF FALL RIVER, MASSACHUSETTS
WATER TREATMENT FACILTIY

PROPOSED SITE PLAN

FIGURE 3–16 ADDITION OF DEEP BED FILTERS IN LIEU OF EXISTING ABW FILTERS

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FS&T PROJECT NUMBER WF–034	SCALE: 1"=100'

3.7.4 Chloramination

Chloramines, which are formed by the reaction of aqueous chlorine with ammonia, were used regularly as a disinfectant in the 1930's and 1940's. Due to the shortage of ammonia during World War II the use of chloramines for disinfection decreased. With increased public concern and awareness of chlorinated organics, and regulation or disinfection byproducts such as TTHMs and HAA5s, the usage of chloramines for disinfection has increased.

In addition to maintaining a more stable disinfection residual, chloramines reduce disinfection by-products such as TTHMs and HAA5s. Also, a chloramine residual in the distribution system has the ability to penetrate the tuberculation in the old unlined cast-iron pipe without being consumed by the iron (free chlorine is easily consumed by the iron tuberculation and therefore can not penetrate and disinfect). In some cases, this can result in a significant reduction of coliform and heterotrophic plate count (HPC) bacteria in distribution systems. Finally, disinfection with chloramines can potentially reduce the taste and odor of chlorine in consumer's water.

The bactericidal effects of free chlorine diminish considerably with time in the distribution system. When ammonia is added to chlorinated water, the bactericidal effects of the resulting chloramines continue well into the distribution system. Chlorine is a strong disinfectant, however, it is unstable. Chloramines are a weaker disinfectant than chlorine but are much more stable. The combination of these two would create a better disinfection scenario at the treatment plant. Chlorine, added prior to the clearwell, will provide the strong disinfectant ability and disinfection contact time (CT) required by regulations. Ammonia, added downstream of the clearwell, will create chloramines and maintain a higher residual concentration to the extremities of the distribution system. The optimum scenario would be to add UV disinfection for primary disinfection followed by the addition of chloramines to maintain a residual in the distribution system. UV disinfection is discussed in one of the subsequent sections.

The exact dose of ammonia needed is specific to the water supply and is primarily dependent on the chlorine to ammonia ratio and the pH. This is a weight ratio dependent on the effluent chlorine residual and not chlorine dose. Temperature and contact time also play a role in the formation of chloramines. The optimum ratio for creation of the monochloramine species of chloramines is typically 3 parts chlorine to 1 part ammonia.

Public notification of the switch from chlorination to chloramination is essential. This is of particular importance to kidney dialysis patients and any individuals or businesses that keep fish or shellfish tanks. Chloramines, if present in a dialysis solution, could possibly pass through the dialyzers semipermeable membrane and into the blood stream of a patient during treatment. If chloramines directly enter the blood stream it can cause hemolytic anemia. Accidental use of chloramine treated water for dialysis has been responsible for additional patient transfusions to treat the hemolytic anemia and was a possible factor for an increased mortality rate for the impacted patients. Treatment of water prior to dialysis treatment needs to be increased or implemented by the dialysis facility. Furthermore, chloramines damage the gill tissue of fish and may enter the bloodstream and damage the red blood cells. Fish hobbyists, pet shops as well as any establishments that keep live fish or shellfish in water need to be notified. A public notification process specific to the City must be developed and implemented while leaving enough time for public response.

Three forms of ammonia, gaseous (anhydrous) ammonia, liquid (aqueous) ammonia and dry (ammonium sulfate) are available for use at treatment facilities for purposes of chloramination.

An aqueous ammonia system was installed during the 2005 upgrades to the WTP, it is similar to other solution feed systems and consists of a bulk storage tank, a day tank, chemical feed pumps and a solution diffuser. Feed pumps were placed close to the storage tank to reduce the amount of ammonia vaporization in the piping.

Advantages of using chloramines include reduced formation of the DBPs that are currently regulated and residuals remain higher in the distribution system. A key disadvantage of chloramines are the release of high concentrations of lead at houses with lead services. Following the conversion to chloramination, Washington DC experienced elevated lead levels in the water distribution system. Based on an analysis of the lead service pipes in the Fall River distribution system conducted by the EPA, lead scales found in Fall River were similar to the ones found in the Washington DC. Washington DC converted to chloramines in 2000. As part of routine lead sampling in 2003 they discovered elevated lead levels in the distribution system. In 2004 they added a corrosion inhibitor which helped to reduce lead levels. Washington DC continues to use chloramination

Based on the EPA analysis of water quality and lead services conducted in 2007, soluble forms of lead that may be released into the drinking water are present on the interior of the lead services. Analysis was conducted on another set of lead services removed in 2013 and preliminary analysis indicated that the scales were still in a soluble form but were also transitioning into less soluble forms of lead. Although the less soluble forms of lead are an encouraging result, these forms may still release in the drinking water with the use of chloramines.

In addition to the potential for lead release, other disadvantages of chloramination include the potential for the DBPs formed from chloramines to potentially become regulated in the future, nitrifying bacteria formation in the distribution after continuous use and the need for public education and notification for dialysis patients and individuals with fish tanks. At this point in time there are no known DBPs formed from chloramines but future research could identify by-products. Nitrification in drinking water systems is caused when nitrifying bacteria form in the distribution system. This can be caused by warmer water temperatures, extended detention times and high concentrations of naturally occurring organic matter. When nitrification occurs there is an increase in nitrite and nitrate levels in the water along with a decrease in disinfection residual and often results in coliform positive samples. Distribution system maintenance can assist with preventing nitrification and many water systems conduct distribution water quality testing to identify early signs of nitrification. When the data indicates that nitrification is beginning water treatment plants temporary shut off the ammonia feed systems and switch to a free chlorine residual which removes the food source for the nitrifying bacteria.

The existing chemical system for aqueous ammonia has not been in use since its installation in 2005. Modifications and repair may be required for proper use of this equipment. The annual chemical cost of using aqueous ammonia is approximately $14,000.

3.7.5 Ultraviolet Disinfection

The new SDWA regulations through the IESWTR require 2-log inactivation of Cryptosporidium in addition to the current log removal requirement for Giardia. Cryptosporidium are much more difficult to inactivate than Giardia. The addition of ultraviolet (UV) disinfection can provide an additional barrier of protection against Cryptosporidium and other waterborne pathogens. UV light is a physical disinfection process not a chemical disinfection process and therefore does not form any known disinfection by-products. A secondary disinfectant such as chlorine or chloramines is still required since UV does not provide a disinfection residual that can extend into the distribution system.

Recent research has shown that UV is effective at inactivating Giardia and Cryptosporidium. UV systems are currently installed at surface water facilities across the United States simply to provide the additional barrier of protection against microbial contamination and not to meet regulatory requirements.

The City may wish to consider the installation of a 24 MGD UV disinfection system to provide the additional barrier of protection against waterborne pathogens. The cost for three 12 MGD capacity UV units, two active and one stand by, is estimated at about $2.1 million. This includes three 8-lamp, high intensity, medium pressure, UV units installed in a new concrete vault installed in the yard over the high lift pump discharge. The annual cost to operate the UV system is estimated at about $100,000 based on a power cost of 15 cents per kilowatt-hour and a lamp replacement cost of $300.

3.7.6 Ozonation, GAC Filtration and UV Disinfection

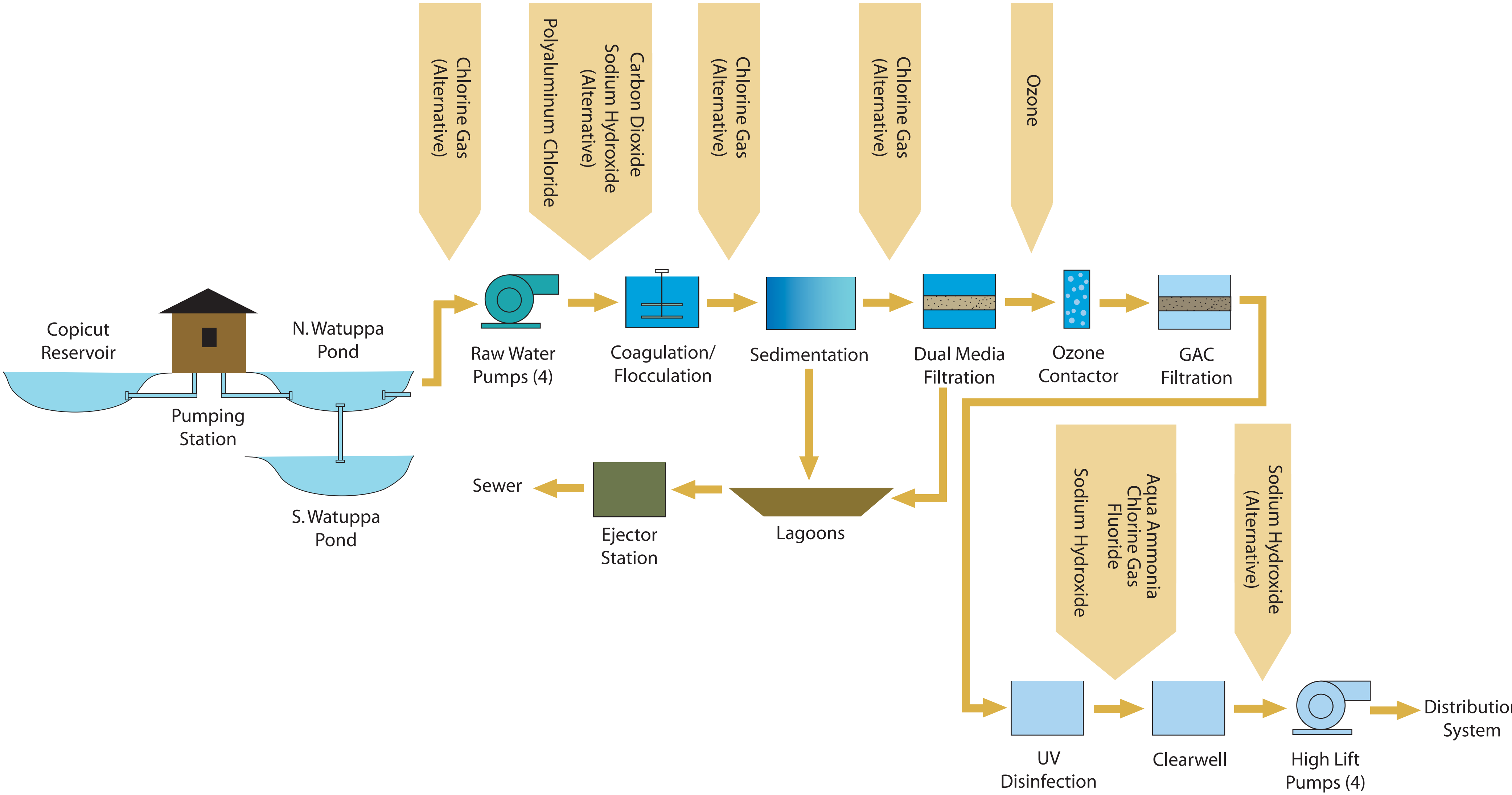
The combination of ozonation, GAC filtration and UV disinfection will provide the best possible water quality to consumers. While each individual unit process has significant benefits, the combination of the three will provide the highest level of protection against contamination.

The removal efficiency of a GAC bed is reduced when the filter is also utilized for suspended solids removal, as would be the case if the existing shallow bed filters were abandoned. The addition of GAC filtration following the existing ABW filters will extend the life of the GAC since the suspended solids will be removed in the shallow bed filters and the GAC will be used strictly to adsorb organics.

The addition of ozone to the water prior to a GAC filter bed further enhances the biological activity in the GAC media, hence the term "biologically activated carbon" or BAC. The ozone oxidizes the natural organic matter expressed at total organic carbon (TOC) and increases the assimilable organic carbon (AOC) fraction of the TOC. AOC is a food source for the many microorganisms that grow on the immense surface area within the BAC filter bed. The increase in microorganisms can result in the *biological* removal of organics that would otherwise have been removed by *adsorption*, thereby extending the life of the GAC. In addition to enhancing the removal of organic matter in GAC filters, ozone is a very strong disinfectant and may also oxidize micropollutants such as pesticides, if present in the raw water, some of which are then more easily removed by GAC. Finally, the GAC will also provide a second barrier to turbidity breakthrough in the event of a problem with the shallow bed ABW filters. Utilizing ozone along with a GAC media bed as a second filtration stage will greatly increase the life of the GAC media, thus reducing media replacement costs. It is estimated that the GAC media will require replacement every 3 years.

The addition of UV disinfection following GAC filtration and prior to the clearwell will greatly increase consumer protection from microbial contaminants that have not been previously removed by filtration.

Figure 3-17 presents a process schematic of the treatment process including the three new unit processes. The cost of providing all three of these additional processes is estimated at approximately $26 million. The additional O&M costs associated with these unit processes are estimated at $800,000 per year.



Fall River Water Supply and Treatment Plant Schematic
Ozone, GAC and UV Disinfection Addition

FST Fay, Spofford & Thorndike
100 YEARS

Figure 3-17

3.7.7 Ion Exchange Resin, MIEX®

MIEX® is a magnetic ion exchange resin for the removal of Dissolved Organic Compounds (DOC) from water. The resin forms agglomerates on the surface of the magnetically charged resin with the positively charged DOC and the negatively charged chloride that is attached to the surface of the resin called adsorption. Other positively charged compounds may also be removed during this process. After the resin is loaded with DOC it is soaked in a brine solution where the DOC attaches to the available chloride in the brine and the sodium chloride regenerates the resin. Alternative brines may be used instead of sodium chloride if sodium is not desired in the treated water.

The Fall River source water has concentrations of DOC present. DOC serves as a precursor to the formation of DBPs in the distribution system and it is very difficult to remove these dissolved suspended particles with the existing physical and chemical processes at the plant. The removal of the DOC with the MIEX® resin will help reduce the later formation of DBPs to maintain compliance with the Stage 2 D/DBPR.

The MIEX® resin is housed in an upflow clarifier with a separate fluidized section of resin and plate settlers or a pressurized vessel. The plate settlers contain the resin allowing treated water to leave the clarifier or pressure vessel without the resin. The clarifier and pressure vessel are continuously operational and do not require the system to be shut down for regeneration of resin. There is a continuous side steam of resin that is removed, regenerated and replaced in the fluidized bed of resin. These systems are capable of high loading rates up to 12 gpm/sf.

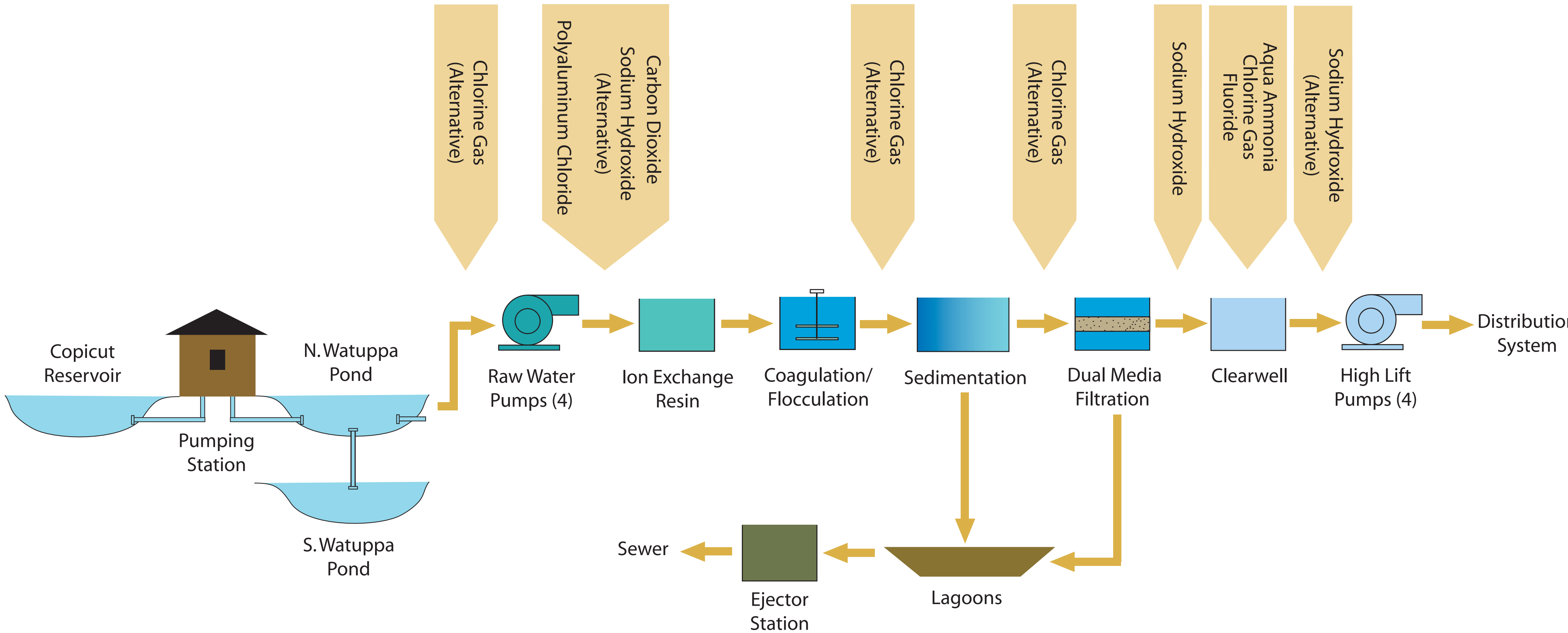
The system would be installed as pretreatment prior to settling or following another step in the treatment process.

The advantages of the MIEX® system is simple operation, low operator maintenance with no moving parts, multiple potable water installations in the United States and other areas of North America, and being a proven technology for reduction of DBPs. This technology may also reduce the amount of coagulant required for treatment because of the reduced DOC.

The disadvantages of the MIEX® system are brine waste stream produced at approximately 200-300 gallons per 1 million gallon treated per day and the need of the brine regeneration system and brine storage. In addition, this system would require additional pumping up stream of the MIEX® system.

Figure 3-18 presents a process flow schematic for the MIEX® ion exchange option and its location within the overall treatment train. Figure 3-19 depicts a potential site plan including the potential location of a new ion exchange facility.

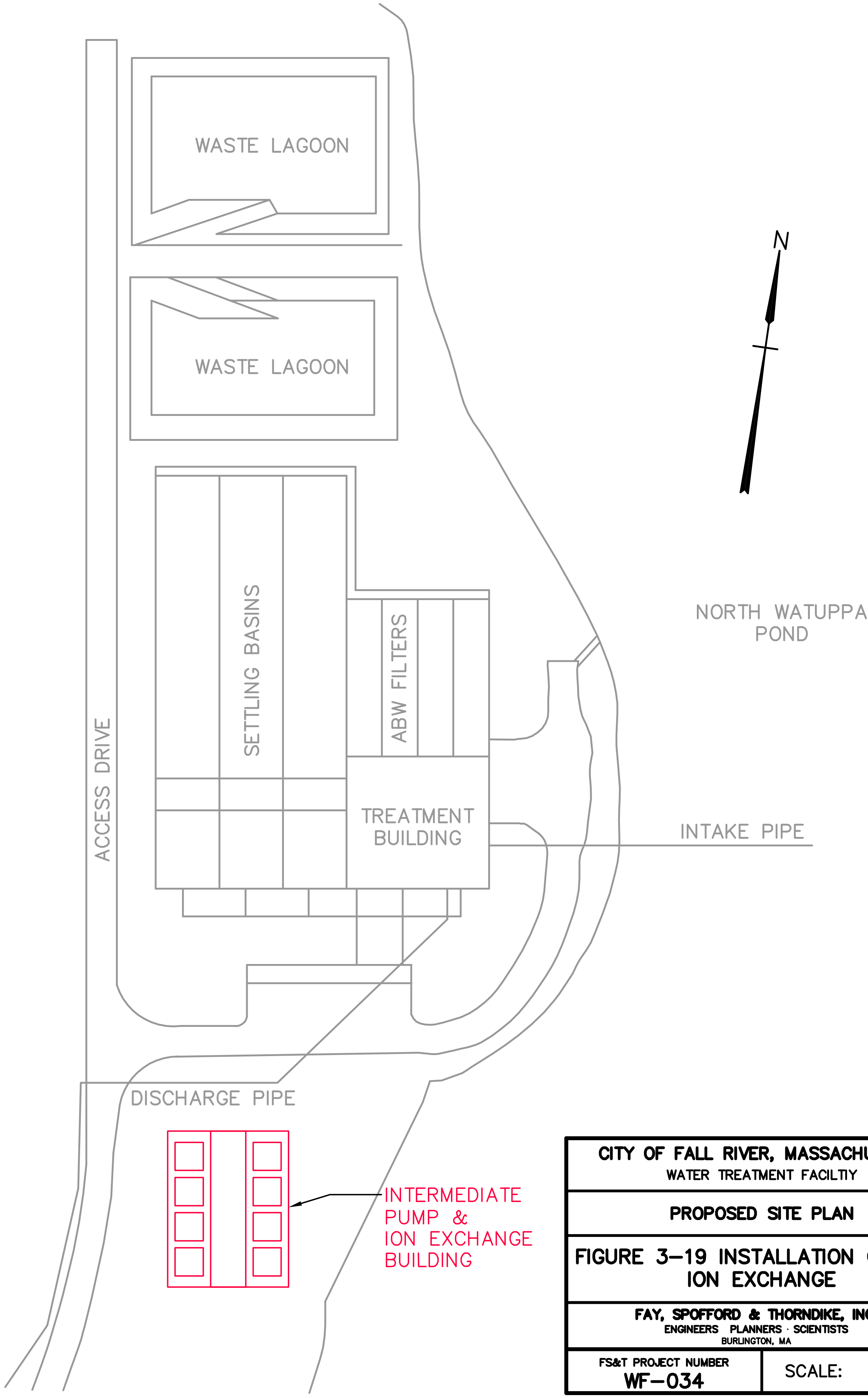
The cost of a MIEX system is estimated to be $8.0 million.



Fall River Water Supply and Treatment Plant Schematic
Ion Exchange Resin Treatment Process

FST Fay, Spofford & Thorndike 100 YEARS

Figure 3-18



CITY OF FALL RIVER, MASSACHUSETTS WATER TREATMENT FACILTIY	
PROPOSED SITE PLAN	
FIGURE 3-19 INSTALLATION OF MIEX ION EXCHANGE	
FAY, SPOFFORD & THORNDIKE, INC. ENGINEERS PLANNERS · SCIENTISTS BURLINGTON, MA	
FS&T PROJECT NUMBER **WF-034**	SCALE: 1"=100'

3.7.8 MIEX® Pretreatment Followed by Membranes

The combination of MIEX and membranes will provide high quality water with the combination of organic material reduction and removal of very small particles. The ability for the membranes to remove particles is reduced through reduction of solids and organic material.

The MIEX system would be installed prior to the rapid mix and the membranes would be in the sedimentation basins for increase in solids removal. This would also assist the filters with reduced solids loading in addition to less organics that may clog the filters. Reduced organics would also prevent the formation of DBPs in the distribution system.

The membranes would provide superior microbial reduction to reduce bacterial formation in the distribution system. The approximate cost of a MIEX and membrane system is approximately $25 million.

3.7.9 Comparison of Alternatives

A summary of the costs as well as the advantages and disadvantages of each of the alternatives described previously is summarized in Table 3-13 and Table 3-14. Each alternative was evaluated considering that capital costs would occur in the first year. First year O&M costs are different for each alternative; however, all O&M costs were inflated 3% per year to account for increases in labor and energy costs. O&M costs were calculated for each year over the 20 year planning period. The present value of the O&M costs was then determined by assuming a discount rate of 4%. The present value of the O&M costs was added to the respective capital cost to determine the present value of the alternative.

TABLE 3-13 COST OF FILTRATION AND ENHANCED TREATMENT ALTERNATIVES

Alternative	Capital Cost (million $)	Additional Annual O&M Cost	Present Worth
Ozonation	$5.6	$200,000	$9,100,000
Membrane Filtration	$22.2	$500,000	$30,100,000
Nanofiltration	$30.0	$2,400,000	$72,000,000
Intermediate Pumping and Deep Bed Filtration (Conventional or GAC)	$18.0	$110,000 (intermediate pumping & blowers) $60,000 (multi-media replacement) $600,000 (GAC replacement)	Incl. below $21,000,000 $30,500,000
Chloramination	$0	$14,000	
Ultraviolet Disinfection	$2.1	$100,000	$3,800,000
Intermediate Pumping, Ozonation, GAC and UV	$25.7	$110,000 (intermediate pumping & blowers) $200,000 (ozonation) $400,000 (GAC replacement) $100,000 (UV disinfection) $810,000 TOTAL	$40,000,000
MIEX	$8	$450,000	$16,000,000
MIEX & Membranes	$28.2	$950,000	$45,000,000

TABLE 3-14 ADVANTAGES AND DISADVANTAGES OF ALTERNATIVES

Alternative	Advantages	Disadvantages
Ozonation	Reduction of organics Reduction of taste and odor Additional microbial reduction	Moderate capital cost Moderate O&M cost
Membrane Filtration	Excellent microbial reduction	High capital cost High O&M cost Requires additional flow for backpulsing Increased residuals
Nanofiltration	Reduction of organics Excellent microbial reduction	High capital cost High O&M cost Requires intermediate pumping Increased residuals
Intermediate Pumping and Deep Bed Filtration (Conventional or GAC)	Increased microbial and solids removal Lower effluent turbidities DBP reduction with GAC Taste and odor reduction with GAC	High capital cost High O&M cost Requires intermediate pumping Potential permitting issues
Chloramination	Reduction of DBPs Longer residence times Already has been installed at the WTP	Potential release in lead Nitrifying bacteria in distribution system Public education about chloramines
Ultraviolet Disinfection	Excellent microbial reduction Will reduce chlorine dose required in clearwell Low capital cost Low O&M cost	Primarily used for surface water treatment as a second disinfectant to ozonation for unfiltered supplies
Ozonation, GAC and UV	Excellent microbial reduction Lower effluent turbidities Maximum reduction of organics Maximum reduction of taste and odor	High capital cost High O&M cost UV not currently approved for surface water treatment Required intermediate pumping Potential permitting issues
MIEX	Reduction of organics and lower DBPs	Additional brine waste stream
MIEX & Membranes	Reduction of organics and lower DBPs Excellent microbial reduction	High capital cost High O&M cost Requires additional flow for backpulsing Increased residuals Additional brine waste stream

SECTION 4 - WATER DISTRIBUTION SYSTEM

4.1 GENERAL

The purpose of the water distribution system analysis is to determine the adequacy of the City's water storage facilities, pumping and piping network under current and future projected demand conditions. The water distribution system was evaluated to determine its ability to meet minimum operating pressures during high demand periods, fire flows, and various operational scenarios.

The water distribution system in Fall River includes two water service areas (low service area at overflow elevation of approximately 318 feet and a high service area at overflow elevation 365 feet), seven water storage facilities, one water booster pumping station and approximately 216 miles of water main. The location of these facilities is illustrated on Figure 4-1 and on the water distribution system map attached as Appendix H.

In general, the distribution system should be capable of delivering the maximum rates of flow, including required fire flows, while maintaining suitable pressure within the system. The DEP's Guidelines and Policies for Public Water Systems recommends that the normal working pressure in the distribution system should be approximately 60 psi and not less than 35 pounds per square inch (psi). DEP also recommends that static pressures in a distribution system do not exceed 100 psi.

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CITY OF FALL RIVER, MA

FACILITIES LOCATION PLAN

FIGURE 4-1

FEBRUARY 2014

4.2 DISTRIBUTION STORAGE

In the evaluation of a municipal water system, present as well as future storage requirements must be considered. Some of the major advantages of providing storage within a distribution system are:

- Storage helps to meet hourly demand fluctuations minimizing changes in flow rates through pumping stations and treatment processes, thus reducing operational costs and increasing treatment efficiency.
- Storage helps to meet required fire flows, thus reducing pumping station capacity and cost.
- Storage provides a volume of water for emergencies in case of a pipeline break, mechanical equipment malfunction or power failure.
- Storage, if properly located, helps to equalize pressure throughout the system.

It is necessary to maintain storage levels as near to full as possible in order to maintain maximum available pressure in the distribution system and maximize fire flow availability. Currently, there are seven water storage tanks in the City's distribution system. Two (2) 1.6 MG tanks act as one 3.2 MG tank and are located adjacent to each other on Bedford Street across from the Water Department headquarters. There are three tanks; the Hood Street Tank (10.4 MG), the Haskell Street Tank (1.7 MG) and the Industrial Park Tank (1.6 MG), located to the north of Bedford Street. The Chicago Street Tank (1.1 MG) and the Townsend Hill Tank (2.3 MG) are located in the southern half of the City. In addition to the high lift pumps located above the clearwell in the treatment plant, there are three pumps, each with a capacity of 300 gpm, located in the booster pumping station at the intersection of Howe Street and South Main Street. The Howe Street Booster Pumping Station supplies water to fill the Townsend Hill Tank at its elevated overflow and remote location.

4.2.1 Recent Storage Facility Improvements

In April 2007, the DEP completed the City’s Annual Sanitary Survey recommending infrastructure improvements to bring the City into compliance with many of the DEP Regulations and Guidelines for safer operation of a public water supply system. Among the recommendations was a significant maintenance and improvement program for the City’s water storage facilities. With funding assistance from the Drinking Water State Revolving Fund, the City embarked on an aggressive program to replace or rehabilitate all seven of its water storage facilities.

All of the City’s storage facilities were found to have lead-bearing coatings which was contained and abated either during demolition or during abrasive-blasting. The six storage facilities that have been completed to date have been equipped with Tideflex Mixing Systems to facilitate better mixing and improved water quality. Many of the tanks are located on small lots with very close residential neighbors. The City worked closely with each contractor to ensure that the work was done in according to all City noise and work ordinances, resulting in very few resident complaints.

In 2007, the Townsend Hill tank was replaced with a 40-foot taller water storage tank and a new high service area was created to better serve the southern part of the City. In 2009 and 2010, the Bedford North, Bedford South and Haskell Tanks were cleaned and painted, and significant structural and sitework repairs were completed.

In 2009 the City had to repair multiple leaks on the Chicago Street Tank, so much so that it was not prudent to rehabilitate it. In 2010 it was replaced with a 10-foot taller tank to slightly increase working pressures in this area of the City. In 2012 the City's largest tank, the 10.4 MG Hood Street tank was cleaned and painted. In addition to the Tideflex Hydraulic Mixing System a chlorine injection pump station was built adjacent to the tank with a recirculation pump to maintain chlorine residual. The Hood Street tank was also outfitted with a new floor, due to the extensive pitting of the original floor.

In 2013, design is underway to replace the existing Airport Road tank in the Fall River Industrial Park with a new elevated tank at a higher elevation as part of new high service area to increase working pressures in the northern part of the City.

Updated information on each of the seven tanks including overflow elevation and height, ringwall elevation, diameter, gallons per foot of storage and construction information is presented in Table 4-1. The total volume of the system's distribution storage, calculated based on the height of each tank's overflow and the tank's diameter, is approximately 20.1 MG.

TABLE 4-1 STORAGE FACILITY DATA

	Bedford North	Bedford South	Hood	Haskell	Industrial Park (Airport Road) (Existing)	Industrial Park (Airport Road) (Proposed)	Chicago	Townsend
Overflow Elevation (feet)	318	318	325	319	321	368	320	365
Overflow Height (feet)	63	63	84	67	90	40-46	74	92
Ringwall Elevation (feet)	255	255	241	252	231	235.5	246	273
Diameter (feet)	65	65	145	65	55	25-35	50	65
Gallons/Foot	24,821	24,821	123,517	24,821	17,771	16,300 to-18,700	14,689	24,326
Total Tank Volume (gallons)	1,563,719	1,563,719	10,375,446	1,663,003	1,599,408	750,000	1,087,000	2,238,000
Year Constructed	1939	1945	1962	1959	1969	2014 (tbd)	2011	2007
Construction	Riveted	Welded	Welded	Welded	Welded	Elevated Welded	Welded	Welded
Contractor	Tippett and Wood	Bethlehem Steel	Chicago Bridge & Iron	Pittsburg-Des Moines	Chicago Bridge & Iron	TBD	Fisher Tank Company	Fisher Tank Company
Improvement Program	Repaired & Painted 2009 JPI Painting	Repaired & Painted 2009 JPI Painting	Repaired & Painted 2012 Rockwood Corporation	Repaired & Painted 2010 JPI Painting	Scheduled for replacement	To be built 2014-15	Replaced in 2011	Replaced in 2007

4.2.2 Storage Requirements

The volume of water required for storage is dependent upon the equalizing storage requirement and storage needed for fire protection. Equalizing storage is the volume of water necessary to satisfy hourly fluctuations in water consumption. It typically amounts to about 20 percent of the total water consumption of any given day for systems with typical residential and industrial demand percentages. In the case of Fall River, the high degree of industrial usage decreases the percentage of equalizing storage to approximately 10 percent of the maximum daily water consumption. The current equalizing storage required is about 1.5 MG or 10 percent of the average maximum day consumption from the years 2002-2012. In the year 2035, the equalizing storage requirement is projected to be 10 percent of 14.3 MG (2035 max day) or approximately 1.4 MG.

Fire flow requirements are based upon the various building types present. In this particular distribution system, residential and industrial areas are spread quite evenly throughout the City.

A fire flow volume requirement of 900,000 gallons was developed in accordance with the Insurance Services Office (ISO) requirements and is based on 5,000 gpm (the largest fire flow requirement in the City) for a three-hour duration. Addition of these water requirements yields the total storage requirements of the system.

Total useable storage is based upon the volume of water above the minimum level in the tanks required to maintain a minimum pressure of 20 psi (46 feet) for fire protection, at all locations in the distribution system. For calculating usable storage for the City's storage facilities, the maximum service elevation near each storage facility was identified. The maximum tank elevation is based on the highest tank level each tank could actually attain without causing any of the other tanks to overflow. The total usable storage per tank is presented below in Table 4-2. The total useable storage for the entire City is approximately 5.5MG, with 4.2 MG of it in the low service system that includes most of the City.

TABLE 4-2 USABLE STORAGE

Storage Tank	Maximum Service Elevation (ft)	Maximum Tank Elevation (ft)	Usable Storage (gal)
Bedford North	257	316	397,136
Bedford South	257	316	397,136
Hood	257	316	1,976,272
Haskell	252	316	521,241
Chicago	242	316	455,359
Townsend	265	363*	1,337,930
Industrial	248	316	444,275
Total			**5,529,349**

*High Service Area

The water storage requirements for the City are presented below in Table 4-3.

TABLE 4-3 WATER STORAGE REQUIREMENTS (GALLONS)

Equalizing Storage	Fire Flow Storage	Total Storage Required	Total Storage Usable
1,424,680	900,000	2,324,680	5,529,349

4.2.3 Adequacy of Distribution Storage

As stated previously, the total volume of distribution storage is approximately 20.1 MG. Notwithstanding minimum pressure requirements at the higher elevations throughout the City, this volume of water could supply the City for slightly less than two days at the average daily rate of consumption if no water were supplied to the system by the high lift pumps. However, as shown in Table 4-2 the total useable storage in the City amounts to approximately 5.5 MG. Therefore, assuming the tanks are as full as is practicable to start with, the storage tanks could supply the system for about half a day before the pressure in the higher elevations would drop below 20 psi. Table 4-3 also indicates that total useable storage exceeds the required storage of 2.3 MG by approximately 3.2MG. Typical system operation allows tank levels to fluctuate 20% or 10 to 20 feet each day, depending upon the height of the tank, to provide equalizing storage and to allow water to be replaced to avoid stagnant conditions. In addition, three of the distribution system tanks (Hood Street, Haskell Street and Industrial Park tanks) highest operating point is about 13 feet below the overflow on average. The system is operated this way to minimize excessive pressure near Mount Hope Bay and the Taunton River, and to allow the Bedford and Chicago Street tanks to contribute water to the system. This data indicates that the distribution storage tanks can provide an adequate volume of water for current and future fire flow and equalizing storage conditions.

4.3 DISTRIBUTION PIPING

The City's distribution piping system consists of approximately 216 miles of pipe up to 36-inch diameter, most of which was originally installed prior to 1930 and some as early as the late 1800's. The backbone of the distribution system lies under Bedford Street which runs east to west through the center of the City. A 36-inch pipe, a 24-inch pipe, and a 20-inch pipe all transmit water up the hill from the treatment plant to the Bedford Street tanks and continue into the system. As a result of the wide variation in topography, distribution system pressures range from 20 psi near the City's storage facilities and other high elevation areas, to 130 psi near Mount Hope Bay and the Taunton River. The pressures in these areas do not meet the DEP guideline range of 35 psi to 100 psi.

The approximate overall lengths of each diameter pipe are summarized in Table 4-4. The Fall River Water Department has 19,809 metered connections. The City's system includes 2,184 hydrants. The City completed a full survey of their hydrants in June of 2013, and is included as Appendix I.

TABLE 4-4 DISTRIBUTION PIPE DIAMETER

Diameter (Inches)	Length (Miles)
36	1
24	2.5
20	6
16	15.5
12	36
10	10.5
8	120
6	19.5
< 6	5
TOTAL	216

The installation year of all pipes are summarized in Table 4-5.

TABLE 4-5 DISTRIBUTION PIPE INSTALLATION YEAR

Year of Installation	Length (Miles)
Pre 1900’s	73
1900's	8
1910’s	9
1920’s	8
1930’s	1
1940’s	2
1950's	4
1960's	5
1970's	8
1980's	27
1990's	21
Post 2000	50
TOTAL	216

The approximate overall lengths of pipe material are summarized in Table 4-6.

TABLE 4-6 DISTRIBUTION PIPE MATERIAL

Material	Length (Miles)
Ductile Iron	124
Cast Iron	45
CLCI	46
Other/Unknown	1
TOTAL	216

4.3.1 Cast Iron Water Main Replacement Program

The 1999 Waterworks Facilities Master Plan prepared by FST, recommended the development of an Infrastructure Rehabilitation Program (IRP) for the replacement of the remaining 12-inch and smaller diameter mains. The focus of the IRP was to evaluate and rank individual pipes based upon the results of the evaluation in order of greatest community need.

In 2001, the City of Fall River Water Department utilized the IRP and embarked upon an aggressive cast iron water main replacement program, with the initial goal of replacing 42 miles of pipe over the course of seven years. Initially, the water main replacements were prioritized based primarily on water quality issues and break history. In 2006, the City began to focus on replacing lead services along with the cast iron water main replacements, using the amount of lead services on a particular street as part of the selection criteria. The City continued the cast iron water main replacement program beyond its original seven-year goal, opting to include the water storage maintenance program within the funding package each year as well. While the length of water main replacements may have decreased some after Year 7, the overall commitment to the program remained constant. Table 4-7 below summarizes the amount of water main replaced each year since the program started.

TABLE 4-7 CAST IRON REPLACEMENTS FUNDED BY DWSRF PROGRAM

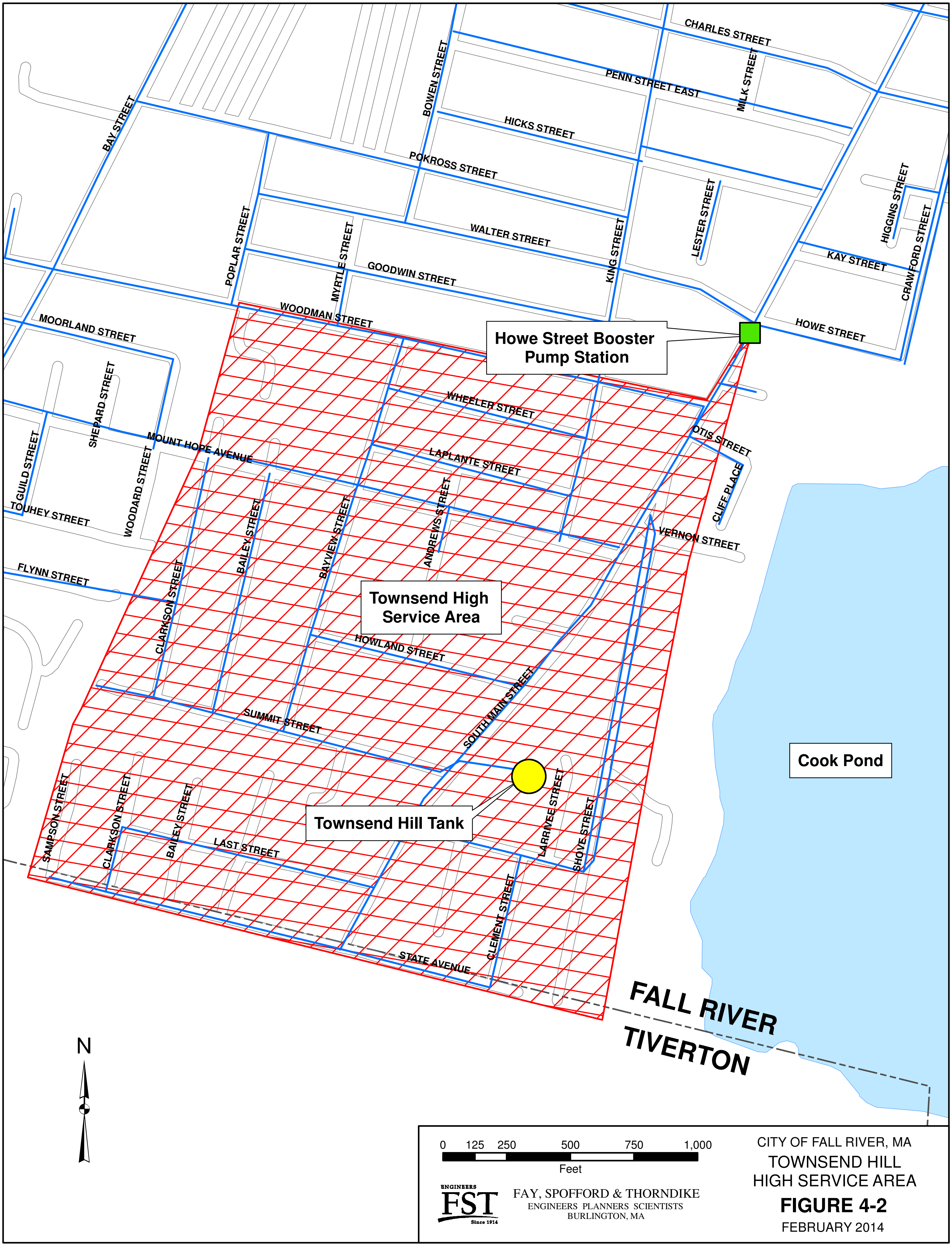
Phase	Total Length (ft)	Year Complete
1	32,775	2002
2	38,650	2003
3	28,000	2004
4	22,500	2006
5	6,775	2007
6	10,150	2008
7	8,450	2008
8	12,300	2009
9	12,080	2010
10	10,025	2011
11	6,350	2012
12	9,600	2013
*13	6,325	2014
*14	17,900	2015
TOTAL	221,880 ft (42.0 miles)	

*- currently in planning / design

With funding assistance from the Massachusetts Drinking Water State Revolving Fund and the City Street Improvement Program, the City has replaced over 55 miles of cast iron water mains and reduced the number of lead services in the distribution system from over 4,800 to less than 1,000. A map and a full list of the water mains replaced in each program is included as the Appendix J – Historical Cast Iron Water Main Replacement Program. There are still approximately 45 miles of old unlined cast iron pipe in the system. The replacement of these mains is addressed in Section 4.8

4.3.2 Townsend Hill High Service Area

In 2007 a new high service area was created within the Fall River water distribution system through the construction of the new Townsend Hill Tank and upgrades to the Howe Street Booster Pump Station. The previous Townsend Hill Water Storage Tank was 44 feet tall and had an overflow elevation of 305 feet. This tank could only provide approximately 20 psi of pressure to the nearby residents and could not supply any fire flow volume at the minimum pressure requirement of 20 psi. The new Townsend Hill Water Storage tank is a 2.3 MG standpipe with an overflow elevation of 365 feet. The upgraded Howe Street Booster Pump Station includes three (3) 300 gpm pumps which pump water to the storage tank. Valves are closed on a few streets in this area in order to isolate the high service area. Figure 4-2 shows the approximate high service area.



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TOWNSEND HILL
HIGH SERVICE AREA
FIGURE 4-2
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4.3.3 Adequacy of Distribution System

The distribution system piping can be considered adequate if it is capable of delivering the maximum demand, including fire flows, while maintaining suitable pressures throughout the service area. The Department of Environmental Protection's Guidelines for Public Water Systems recommend that a system maintain a minimum working pressure of 35 psi in the distribution system under normal demand conditions. When fire flows are considered, the DEP recommends that a system maintain a minimum pressure of 20 psi everywhere in the system.

Typically, a deficient fire flow at any given location is a result of poor pipe carrying capacity and, to a lesser extent, the hydraulic grade line in the tank. Since much of the City's water distribution system was installed early in the early to mid-1900's, much of the pipe was installed as unlined cast-iron pipe. In the 1970's the City cleaned and cement lined most of the larger diameter pipe (20-inch and greater), much of the smaller diameter unlined cast iron pipe has most likely lost carrying capacity due to tuberculation. The Infrastructure Rehabilitation Program (IRP) includes the prioritized replacement of these remaining water mains to restore the carrying capacity, meet fire protection requirements and improve water quality. "Looping" dead end mains where feasible should also be addressed to maintain water quality. As dead end mains are looped, bleeders should be eliminated therefore reducing the amount of lost water in the system.

4.3.3.1 Fire Flow Requirements

The Insurance Services Office (ISO) grades the overall firefighting capabilities of a community by evaluating a number of factors, including the fire flows available at a residual pressure of 20 psi. Several criteria go into determining the Needed Fire Flows, as defined by ISO. They include building size, type of occupancy, materials of construction, proximity to other buildings, and the existence of sprinklers. Needed fire flows are site specific; however, according to Section 340 of the ISO Fire Suppression Rating Schedule, the Needed Fire Flow shall not exceed 12,000 gpm or be less than 500 gpm.

Needed fire flows at specific locations are used to determine the adequacy of the distribution piping. Individually, fire flows generally result in higher and more localized flow rates in the piping leading to the fire flow location. The higher flow rates result in greater headloss and decreases in system pressures. Deficient pipes are easily identified and localized improvements can then be proposed. ISO's process of grading a water system's firefighting capabilities usually results in a number of site-specific flow tests of the system capacity and estimates of Needed Fire Flow at each test location. The following Table 4-8 summarizes the ISO fire flow tests conducted in Fall River on July 1, 2002. Since some of the ISO fire flow requirements contain two values, the lower value was the basis of the evaluation since the upper limit value is greater than what the distribution system is expected to meet.

TABLE 4-8 ISO FIRE FLOW TESTS

Test No.	Test Location	Static pressure (psi)	Residual Pressure (psi)	Needed Flow at 20 psi (gpm)	Available Flow at 20 psi (gpm)
ISO-1	Airport Road and Sykes Road	40	35	4,500/3,000	4,000
ISO-2	Meridian and Columbus Drive	29	25	750	1,300
ISO-3	Airport Road and N. Main Street	45	42	4,500	4,900
ISO-4	Weaver Street and N. Main Street	115	95	4,500	7,500
ISO-5	Ellsbree Street and Langley Street	65	60	4,500	6,600
ISO-6	President Ave and Garden Street	27	26	3,000	2,100
ISO-7	N. Main Street and Brightman Street	125	115	5,000/2,250	1,500
ISO-8	N. 7th and Bedford Street	70	67	5,000	8,500
ISO-9	London Street and Johnson Street	63	53	2,500	3,100
ISO-10	N. Eastern Ave and Locust Street	50	47	3,000	5,500
ISO-11	Pleasant Street and Mason Street	60	59	3,000	6,000
ISO-12	Eastern Ave and Martine	72	70	5,500	8,600
ISO-13	Pleasant Street and Keene Street Residential	62	61	1,500	12,000
ISO-14	Baird Street and Stevens Street	72	65	4,000	5,200
ISO-15	Jefferson Street and Stockton Street	65	55	5,000	4,100
ISO-16	Canning Blvd and Bishop Blvd	50	49	3,500	7,200
ISO-17	Stafford Road and Albert Street	42	40	4,500/2,250	3,400
ISO-18	E. Main Street and Dwelly Street	55	50	4,000	4,500
ISO-19	Bay Street and Byron Street	97	85	3,500	7,600
ISO-20	Bay Street and Chase Street	85	84	7,000	12,000
ISO-21	Almond Street and Division Street Residential	105	104	1,500	5,800
ISO-22	S. Main Street and Sullivan Drive	77	76	4,500	12,000
ISO-23	Rodman Street and Manchester Street	70	69	4,500	12,000
ISO-24	13th Street and Pleasant Street	72	71	5,500	9,400
ISO-25	Quequechan Street and Warren Street	72	71	8,000	12,000

NOTES: Shaded box indicates deficient fire flow based on lowest required fire flow. When two values are presented for Required Fire Flow, the larger flow represents the actual fire flow requirement for nearby building but the lower value represents what the water system is expected to meet at that location. The maximum fire flow a water system is expected to meet is 3,500 gpm.

4.3.3.2 Hydrant Flow Tests

The ISO flow tests conducted in 2002 identify three deficient locations throughout the distribution system when considering the lower of the two required flow values. Since it has been over ten years since these hydrant flow tests were completed by ISO, FST re-evaluated ten of the ISO tests locations, and used the results of the updated flow tests to further calibrate the computer model of the existing water system. The basis of the calibrated hydraulic model is included in Appendix K. The hydraulic model is used to assess the fire flow capabilities and pressures in all areas of the distribution system, and to evaluate pipeline improvements and possible areas of concern for the future demand conditions. Flow tests were conducted by measuring the rate of discharge from selected hydrants while observing the resulting drops in system pressure.

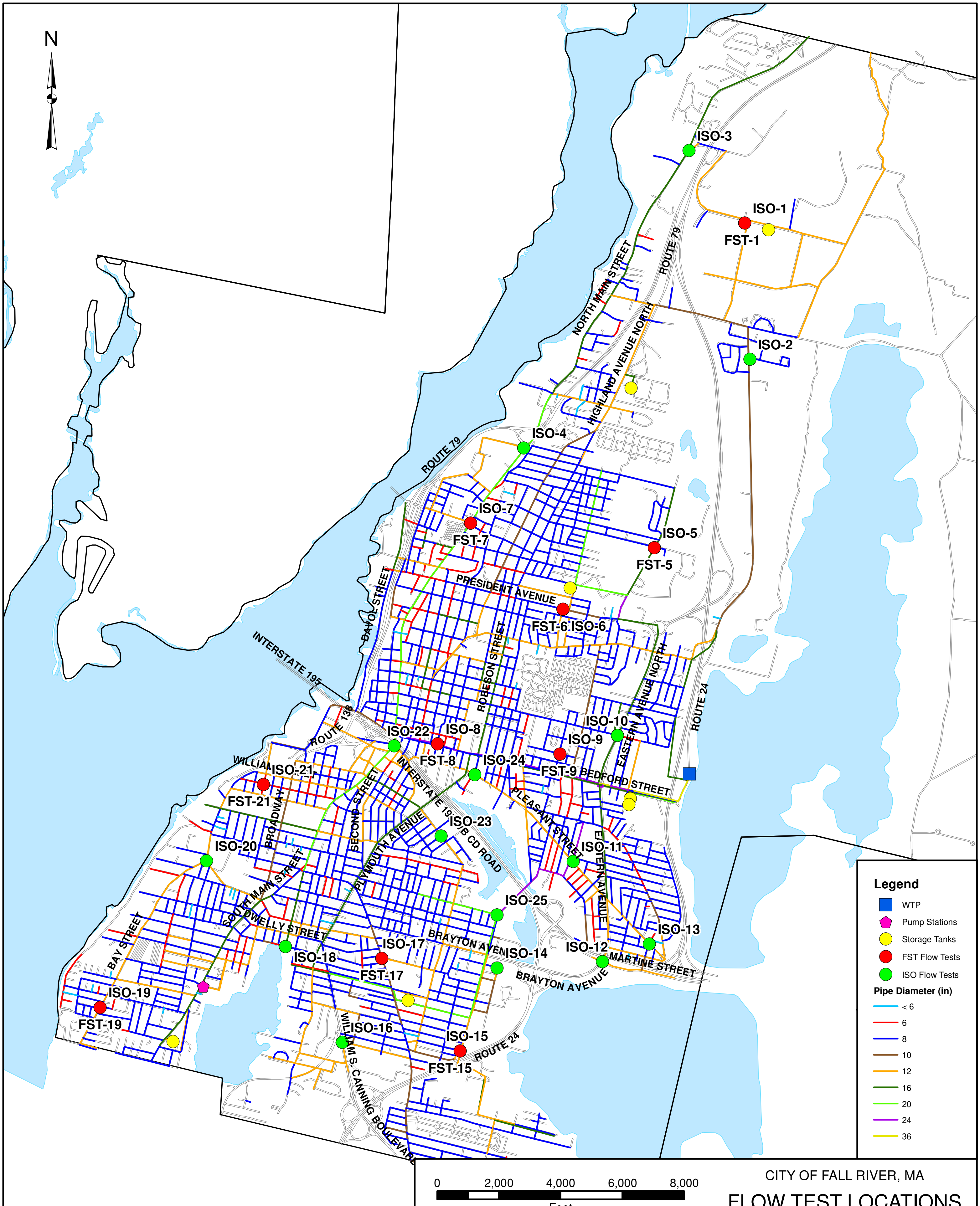
A summary of additional hydrant flow tests and pressure observations conducted by the City on September 24, 2013 and October 9, 2013 are shown in Table 4-9. Based on these tests, calculating the available fire flow at residual pressure of 20 psi yielded one test with a deficient fire flow. Hydrant flow test locations are shown on Figure 4-3 and on a plan included as Appendix L.

TABLE 4-9 FST HYDRANT FLOW TESTS

Test No.	Test Location	Static pressure (psi)	Residual Pressure (psi)	Hydrant Flow (gpm)	Needed Flow at 20 psi (gpm)	Available Flow at 20 psi (gpm)
FST-1	Airport Road and Sykes Road	42	40	1,000	4,500/3,000	3,630
FST-5	Ellsbree Street and Langley Street	66	65	1,190	4,500	9,380
FST-6	President Ave and Garden Street	32	28	540	3,000	960
FST-7	N. Main Street and Brightman Street	126	121	1,720	5,000/2,250	8,950
FST-8	N. 7th and Bedford Street	64	62	1,250	5,000	6,610
FST-9	London Street and Johnson Street	69	67	1,250	2,500	7,000
FST-15	Jefferson Street and Stockton Street	65	63	1,190	5,000	6,380
FST-17	Stafford Road and Albert Street	50	49	910	4,500/2,250	5,670
FST-19	Bay Street and Byron Street	101	95	1,440	3,500	5,850
FST-21	Almond Street and Division Street Residential	104	101	1,510	1,500	9,080

NOTES: Shaded box indicates deficient fire flow. based on lowest required fire flow. When two values are presented for Required Fire Flow, the larger flow represents the actual fire flow requirement for nearby building but the lower value represents what the water system is expected to meet at that location. The maximum fire flow a water system is expected to meet is 3,500 gpm.

The only deficient flow test is FST-6 at President Ave and Garden Street. Based on a review of the hydraulic model, there may be a closed valve restricting the flow to this location. The City is investigating the valves in this area to determine if any are in the closed position.



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FLOW TEST LOCATIONS

FIGURE 4-3

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4.3.3.3 Peak Hour Demand Condition

In addition to running the computer model using a maximum day plus fire flow scenario, a peak hour scenario was also used which typically represent the maximum day's highest hourly consumption rate. During the peak hour computer model run, the high elevation areas and areas of long-term pressure deficiencies in the City were analyzed to observe if residual pressures were maintained above 35 psi. It is recommended by the Massachusetts Department of Environmental Protection's Guidelines and Policies for Public Water Systems that water distribution systems be operated at pressures between 35 and 100 psi, and that normal working pressure should be approximately 60 psi. As stated earlier, in order for a distribution system to be considered adequate it must also maintain a system pressure of 20 psi during maximum day plus fire flow demand conditions. Fall River has expressed concern relative to low operating pressures in four areas in the distribution system. Table 4-10 presents the pressure results of these locations for peak hour demand condition and average day demand condition in the computer model runs.

TABLE 4-10 HIGH ELEVATION PRESSURES (PSI)

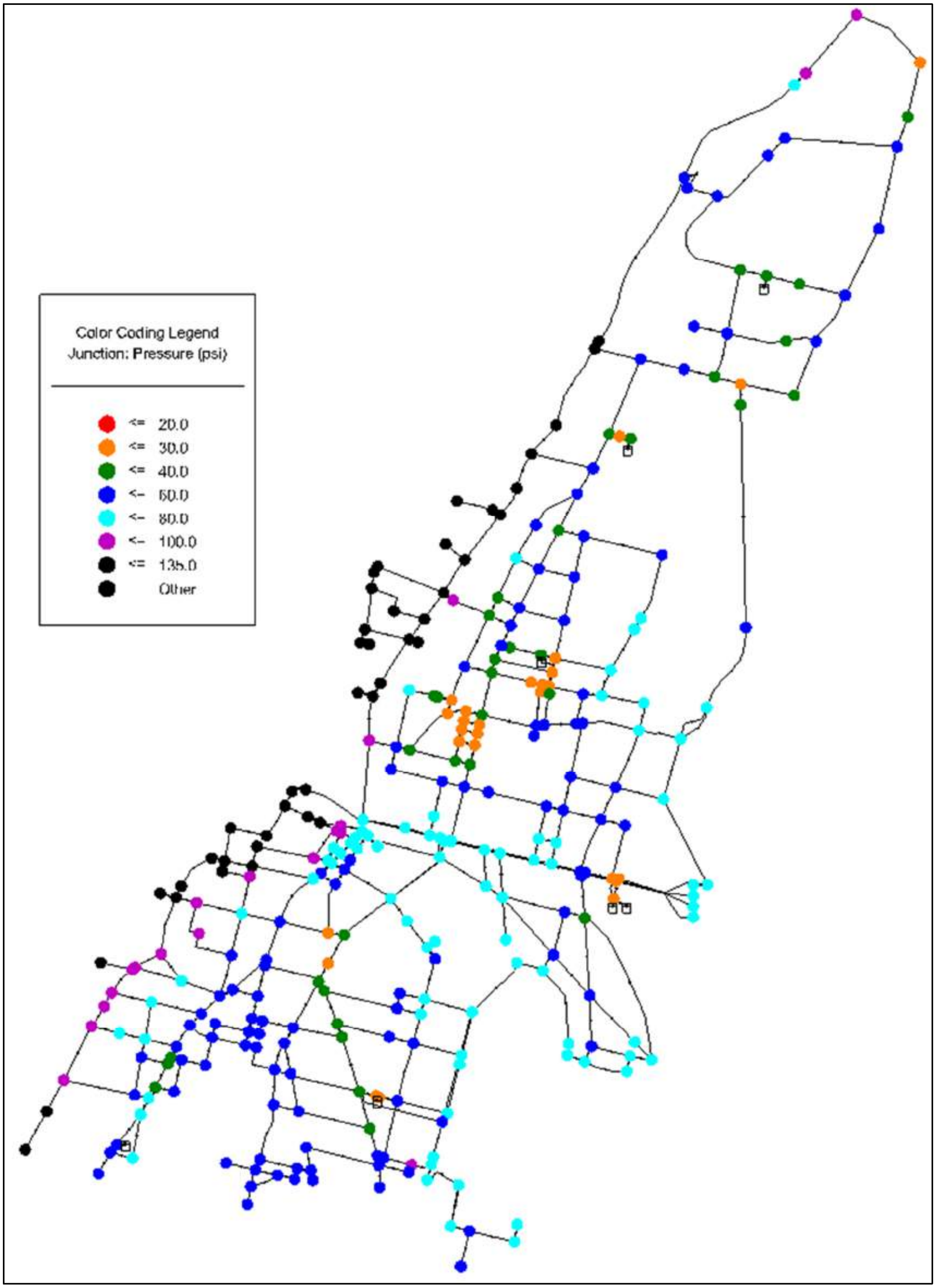
Node	Location	Elevation (feet)	Average Day	Peak Hour
J-65	Charlton Hospital	256	21.1	20.5
J-190	Flat Iron	245	25.4	24.5
J-251	Chicago Street	240	27.7	26.9
J-101	Industrial Park	236	28.8	28.1

*Sections 4.4 and 4.5 further address these low pressure areas

During peak hour conditions, as well as during all other demand conditions, system pressures drop below 35 psi. However, this only occurs in the higher areas of the system, typically near the distribution storage tanks. The topography of Fall River ranges from over 250 feet down to sea level (0 feet). In order to maintain 35 psi at the locations listed above, storage facilities would need to have an overflow elevation of 336 feet, or approximately 20 feet higher than the current tank overflows. This higher operating gradeline would result in pressures near the Taunton River and Mount Hope Bay reaching close to 145 psi. Guidelines and Policies for Public Water Systems also recommend that if pressures exceed 100 psi, then pressure reducing devices should be installed on the distribution mains. Currently, even during the peak hour demand conditions, a pressure of at least 20 psi is experienced at the highest elevations and the highest system pressures experienced are not more than 125 to 130 psi.

At this time the City is in the development of a high service area for the Industrial Park region of the City. A description of the high service area recommendations and plans can be found in the following Section 4.4. Figure 4-4 displays a map of the City, color coded according to system pressures during the peak hour demand condition. As shown in the figure, the areas of highest elevations, typically near storage facilities have the lowest operating pressures.

FIGURE 4-4 RESIDUAL PRESSURES



Note: Model presents schematic presentation of pipe network.

4.4 INDUSTRIAL PARK

4.4.1 Introduction and Previous Reports

Deficiencies in the Fall River Industrial Park, at the northern reaches of the system have been the focus of several reports, dating back to the 1980's. Conclusions early on were that the existing storage tank in the industrial park was not used to being filled to its full capacity because the water level in the Bedford Street and Chicago Street tanks reached their overflow elevations before the water level reached the overflows in the remainder of the tanks in the system. Typical system operating procedures included reducing water production from the treatment plant until the levels in the Bedford Street and Chicago Street tanks fell and pumping could again be increased. To eliminate the overflow problem and increase the hydraulic grade line (HGL) throughout the system, including the industrial park, altitude valves were installed at the Chicago and Bedford Street tanks in 1983. However the increased system operating HGL also increased the overall operating pressure, causing an increase in the frequency of water main breaks on older cast iron water mains. Following installation of the altitude valves, the industrial park experienced further expansion and additional water system improvements were made to provide a better looped system and increased capacity. However, the park continued to expand, placing greater demands on the water distribution system.

The focus of a 1995 report was to review the capacity and weaknesses of the water supply system in the industrial park region. The report concluded that although the distribution system was capable of supplying the industrial park region during normal day-to-day consumption, it determined that the system did not have the carrying capacity and pressure to supply all of the industry with adequate fire protection. The report recommended that in addition to general water system improvements such as cleaning and lining and water main replacement, altitude valves should be installed on all of the systems tanks. The report concluded that this would prevent tank overflows and allow the system to be operated at a higher grade line as well as increase the amount of "readily available stored water". At the same time it would eliminate any pressure relief from an overflow.

These recommendations could have improved fire protection due to due to the ability to fill the tank in the industrial park, but would most likely not have met the fire protection requirements of these facilities. These improvements could have had negative impacts to the distribution system such as causing excessive pressures in the lower elevation areas as well as not allowing distribution storage tanks to fluctuate.

In 2007, FST was tasked with evaluating the feasibility of the development of the Executive Office Park at the existing Industrial Park and whether the water system could provide the area with water at adequate pressure under the full build-out Maximum Daily Demand conditions, which was presented as 396 gpm. The report found that the City is not able to supply the necessary water to the proposed Executive Office Park at adequate pressure. The report recommended the creation of an Industrial Park High Service System as a possible alternative for supplying the Executive Office Park with water for domestic and fire protection purposes at adequate pressures. This new HSS would require the construction of a new tank at a higher HGL and the construction of a booster pumping station to fill the new tank.

In 2010, FST was tasked with evaluating the feasibility of the development of the South Coast BioPark at the existing Industrial Park and whether the water system could provide the area with water at adequate pressure under the full build-out Maximum Daily Demand conditions, which was estimated at 315 gpm. The report found that the City is not able to supply the necessary water to the proposed South Coast Bio Park at adequate pressure. The report recommended the creation of an Industrial Park High Service Area as a possible alternative for supplying the South Coast Bio Park with water for domestic and fire protection purposes at adequate pressures. This new HSS would require the construction of a new tank at a higher HGL, the construction of a booster pumping station, and replacement of the 8-inch water main in Airport Road.

4.4.2 Industrial Park Area Findings and Recommendations

In order to have the Industrial Park area be a viable location for growth with the City of Fall River, improvements to the water distribution system are required. While some previous reports recommended increasing the hydraulic gradeline elevation to the entire City in order to provide higher pressures in this area, these recommendations will adversely impact the rest of the water distribution system. Instead, FST recommends developing a new high service area to provide adequate pressure and fire protection to this section of the City.

The creation of a high service system will require the installation of a new, taller water storage tank along with a booster pumping station. For the construction of the new water storage tank, the adjacent property owner to the exiting tank site within the industrial park is willing to exchange parcels with the City and therefore a new water storage tank will be constructed adjacent to the existing City site. This allows the existing tank to remain on line during the construction of the new tank.

In order to provide adequate pressures in this area, a new water storage tank will need to have an overflow elevation of 368 feet. With ground elevations at the new storage tank parcel location of 236 feet this would result in an overall tank height of approximately 132 feet. Within the City, all water storage facilities are standpipe style tanks. For this site, to minimize stagnant water and maximize water quality, the new industrial park tank will be an elevated style tank.

The anticipated fire flow requirement to be supplied by the water distribution system for this area of the City is 3,500 gpm. In order be able to meet that flow rate for a duration of 3 hours, the tank must have a volume of 630,000 gallons of usable storage. Based on ground elevations within the area, this fire flow volume must be contained at elevation 239 feet and above. Since most storage tank manufacturers have standard economical sizes of storage tanks, a storage tank volume of 750,000 gallons was selected and will provide the anticipated fire flow requirement at an adequate pressure.

A new pumping station will be constructed on a City owned parcel at the intersection of Commerce Drive and Airport Road. This pumping station will have three pumps each with a capacity of 500 gpm. Two pumps will be available for normal operation with the third pump serving as a standby pump.

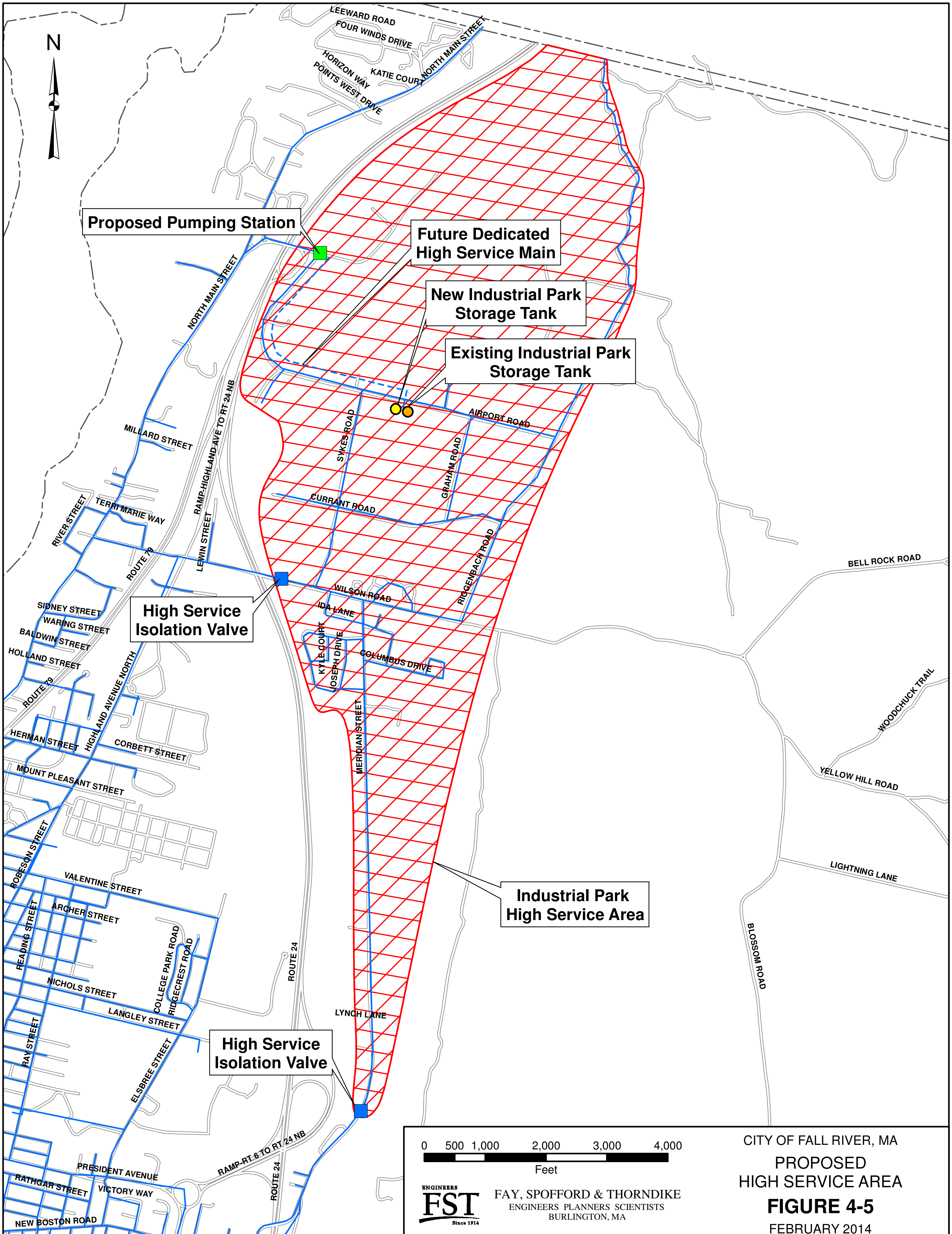
Once constructed, the new pumping station will receive suction from the existing 16-inch water main in North Main Street, the 8-inch and 12-inch water mains in Airport Road and Commerce Drive. The station will pump the water to the new elevated storage tank through the existing distribution mains in the Industrial Park. The new water storage tank will also include an aeration and mixing system to keep the water in the tank fully mixed. This aeration and mixing system will also be capable of removing disinfection by-products (DPBs) from the water. As presented in the Treatment Section of this report, the northern section of Fall River experiences elevated levels of TTHMs and HAAs which are disinfection by-products. With this aeration and mixing system, the water being supplied by the storage tank will have lower levels of disinfection by-products and will help to provide high quality water to this section of the City and to the Freetown meter in Innovation Way. Figure 4-5 shows the limits of the proposed high service area along with the locations of the new pumping station and elevated water storage tank.

4.4.3 Future Considerations for High Service Area Optimization

To fully realize the benefit of the DBP-removal of the mixing system being designed for the new Airport Road elevated water storage tank, the City would need to ensure that all the water leaving the tank and being provided to its customers is the fully-mixed and aerated water from the tank. The primary way to do this is to construct a dedicated water main between the Commerce Drive Pump Station and the Airport Road tank.

Under the currently designed scenario, low service water will be delivered to the Commerce Drive Pump Station via the suction line in Airport Road. The pump station will discharge the water through the existing distribution system to the new Airport Road Tank where the DBP's will be stripped. That water will then be delivered to the system through the tank outlet. Since some of the customers' connections are between the pump station and the tank, they may receive water prior to full DBP stripping, depending on the tank's draw and fill cycle.

Under a future scenario, low service water will still be delivered to the Commerce Drive Pump Station via the suction line in Airport Road. The pump station will discharge the water through a new distribution line directly to the Airport Road Tank where the DBP's will be stripped. That water will then be delivered to the system through the tank outlet and existing distribution piping. Customers' connections will remain on the existing distribution line and valves will be installed in key locations to fully isolate and direct the water so that all the DBP-stripped water is delivered directly to the customers. The distance between the pump station and the tank is approximately 4,500 feet, so the construction cost of this dedicated line should be budgeted as $500,000. To the extent possible and to minimize future shutdowns, the current design includes installation of the appropriate valves and water main stubs to facilitate the future construction of this dedicated line.



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PROPOSED
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FIGURE 4-5

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4.5 AREAS WITH LOW WATER PRESSURE

As noted prior, the inadequate water pressures in the Industrial Park are being addressed through the construction of a new high service area. The other areas noted in Table 4-10 also experience low water pressure due to high ground elevation. These areas, known as the Flat Iron, Charlton Hospital, and Chicago Street are relatively small areas, isolated from each other and isolated from the established high service areas. They are located in central City locations, rather than at the extremities of the system like Townsend Hill and the Industrial Park. Creating high service areas to serve these locations is not impossible but it is more complicated hydraulically and results in an unfavorable amount of dead end water mains.

4.5.1 Charlton Hospital

Charlton Hospital is located at 363 Highland Avenue, in the vicinity of Prospect Street and Hanover Street. The closest water storage facility to the Charlton Hospital is the 10.5 MG Hood Street tank which is about a mile away and has an overflow elevation of 325 feet. The Hospital frequently experiences pressures below the recommended 35 psi limit. Connecting to an existing high service area is not feasible at this location in the City since it would require extensive changes to the system hydraulically to encompass the Hospital in either of the existing high service areas. The creation of a new high service area is an option but is not recommended due to the relatively small amount of facilities experiencing low pressures and the difficulties of isolating the area around the Hospital. FST recommends the City invest in further examination of the Charlton Hospital high elevation area, and explore the possibilities of adding an individual booster pump system to improve localized pressure deficiencies at the Hospital.

4.5.2 Chicago Street

The Chicago Street area is in the southern part of the City, in the “triangle” created by Brayton Avenue, Stafford Avenue and Route 24. There is a new 1.0 MG water storage facility (2010) located between Chicago Street and Emmett Street with the overflow elevation of 318 feet. Typical operating pressures here range from 30-80 psi. Several dozen homes and businesses located in close proximity to the Chicago Street Tank regularly experience pressures below 35 psi. The existing Townsend Hill high service area is located approximately 1.8 miles to the west. The connect these high elevation homes to the existing Townsend Hill high service area would involve substantial changes to the operation of the distribution system and would create many dead ends. Additionally, these changes may increase in pressures in some of the lower lying areas resulting in operating pressures as high as 90 psi. Creating a high service area solely for the group of customers experiencing low pressures around the Chicago Tank would not be cost effective for the City and is not recommended. The creation of a combined high service area to include Chicago Street and Flat Iron high elevation areas is discussed in the following section.

4.5.3 Flat Iron

The Flat Iron area gets its name from the “triangle” created by Lyon Street, Second Street and Plymouth Avenue. The closest water storage facility to the Flat Iron area is the new Chicago Street tank, with an overflow elevation of 318 feet. The Townsend Hill (high service) tank is nearly 2 miles away, with an overflow elevation of 365 feet. Typical operating pressures in the Flat Iron area range from 30 to 36 psi. Approximately 200 water customers in and around the Flat Iron area regularly experience water pressures at or below 35 psi. Connecting to the existing Townsend Hill high service area two miles to the southwest is not feasible due to restrictions in system operation. FST recommends the City explore the possibility of combining the Flat Iron and Chicago Street high elevation areas into one high service area. This may require the high service area extend past the limits of the residents experiencing low pressures and encompass residents that fall into the 60 psi and lower operating range. The new high service area would contain approximately 1,500 buildings throughout the Chicago Street and Flat Iron high elevation areas. Although a number of customers in that area experience pressures that are sufficient, increasing the high service area limits will reduce future upgrade costs and eliminate two areas of low pressure concern for the City.

4.5.4 Summary

The City should consider further evaluations for better serving the customers in each of these areas including the installation of individual booster pumps, and creating an additional high service area. A budget of $100,000 should be set aside for a study and preliminary design to improve the water pressures in these areas. An additional $4,000,000 should be budgeted for future construction for water pressure improvement.

4.6 DISTRIBUTION SYSTEM RELIABILITY

The City's overall water distribution system is relatively well-looped in its interior, with some dead ends towards the extremities of the system. In many areas single mains run from the main distribution system to the extremities of the system without a redundant supply. Additionally the City's transmission system lacks redundancy from the water treatment facility.

4.6.1 Transmission Main Redundancy

Finished drinking water is supplied from the water treatment plant through a single 36-inch diameter cement-lined ductile iron water main that was installed at the time the plant was built in 1976. At the location of the old electric station along the access road, the ductile iron main is connected to a 36-inch diameter cast iron water main that was cement lined in the early 1970's. At this location, the 36-inch main is interconnected to a 16-inch water main that runs cross-country to Meridian Street and into the distribution system. However the carrying capacity of the 16-inch water main is not adequate to provide full redundancy in the event of a catastrophic failure of either section of the 36-inch main.

The 36-inch main from the plant connects to the backbone of the distribution system beginning at the bottom of the Bedford Street hill. Bedford Street, which runs from Watuppa Pond, east to west through the center of the City, has three cast iron water mains - 36-inch, a 24-inch, and a 20-inch diameter – all of which were cement lined in 1972 before the treatment plant was built. The City has recently completed a Transmission Main Redundancy construction project to interconnect these mains along Bedford Street. New valves and piping interconnections were installed, and the original 1873 Pump Station manifold piping was abandoned. A 16-inch water main, which had previously been part of the transmission system, was repurposed as a drain line as part of a drainage improvement program to better collect and direct the runoff coming down Bedford Street and protect Watuppa Pond from street runoff. Two 24-inch risers were constructed, and connected to the underground piping, to provide an opportunity for an emergency overland pipe between the water treatment plant and the water mains at the bottom of Bedford Street in the event of a catastrophic failure of the 36-inch water main in the area that lacks redundancy from the WTP to the Bedford Street water mains.

The emergency overland connection would be able to provide the City with redundancy from the plant in the event of a catastrophic failure of the 36-inch water main. However it may take several days to a week to organize, set up, fuse, test and disinfect, and get into service the half-mile of 24-inch HDPE line that would restore the water service. The City should address this within the Emergency Response plan and make arrangements with no less than two contractors/manufacturers that can provide the materials, equipment and expertise to respond and complete the work immediately upon notification by the City.

Additionally the City should continue to investigate an underground redundant line from the water treatment plant. The pipeline should be a minimum 24-inch in diameter, to handle a flow of 24 MGD at 11.8 fps which is the capacity of the WTP. This is a somewhat higher velocity than is typical designed for, however it should be adequate on a temporary basis. The construction of a truly redundant line from the water treatment plant would need to occur in the opposite direction from Bedford Street. In that direction (northwest from the plant) the City would need to be prepared to contend with permitting, design and construction in the vicinity of wetlands, areas of endangered species, MADOT Route 24, and potential private land-takings. There is an access route from the WTP to Meridian Street, which runs parallel to Route 24 that may be a possible redundant pipe route.

Another option is to construct an additional 24-inch or 36-inch water main parallel to the existing 36-inch transmission main. The benefits to this route are that it is all in City-owned streets, with minimal permitting required. The disadvantages of this route are that portions of the access road along Watuppa Pond have recently been paved, it frequently floods with high pond levels, and running alongside the existing main is not truly redundant. If the existing 36-inch main ruptures, it may cause rupture of utilities in its vicinity.

An estimated cost for the planning, design and construction of a redundant 36-inch water transmission main could range from $2 M to $3M, depending on its length and location.

4.6.2 Interconnections with Other Water Suppliers

The City provides water to surrounding communities on a regular basis. There are two 12-inch interconnections with the Town of Freetown, one located on North Main Street and the other on Innovation Way. A 16-inch water main from South Main Street provides an interconnection with Tiverton, Rhode Island. There is a 12-inch interconnection with Westport in the vicinity of Route 6, and a 12-inch interconnection with the Town of Somerset from Davol Street near Brightman Street, across Mount Hope Bay.

The City sells up to 600,000 gallons per day to these surrounding communities through these municipal interconnections. These communities have operating HGL's lower than that of Fall River's. Because Fall River's HGL is higher than those in the surrounding communities and none of them have their own supplies, the possibility of the City obtaining water from one of these interconnections is unlikely and could not be by gravity flow.

4.7 DISTRIBUTION SYSTEM OPERATION

4.7.1 Travel Time/Water Age

Travel time and water age are major factors in deterioration of water quality within a distribution system. Determining travel time throughout a water distribution system is helpful in identifying areas of concern for microbial contamination and discoloration due to stagnant water. The calibrated hydraulic model was utilized to determine travel time from the Fall River Treatment Plant through the City's distribution system. Several distribution water quality sampling site locations located throughout the City are presented in Table 4-11.

TABLE 4-11 AVERAGE TRAVEL TIME

Location	Travel Time (Days)
1030 President Avenue	0.5
1076 Bedford Street	0.4
80 River Street	1.2
4548 North Main Street	2.7
1545 Stafford Road	1.3
1533 South Main Street	0.4
864 Stafford Road	1.1
631 Airport Road	2.4
Westport Interconnection	0.4

Areas located along Bedford Street, in close proximity to the Treatment Plant, experience the shortest time spent in the system. As the water moves away from the plant, and towards areas of low demand, travel time increases. The areas in the distribution system that experience the longest travel times are located in the northern and southern extremities of the distribution system - along the Tiverton and Freetown borders. If any areas at the extremities of the water distribution system experience water quality complaints, FST recommends that localized flushing be implemented to improve water quality in the area.

Areas with the highest travel times, the North Main Street sample site and the Airport Road sample site, also experience the highest levels of disinfection by-products as presented in the Water Treatment Section of this report. To help address these high disinfection by-products levels, the City will be installing an aeration system within the new elevated Industrial Park water storage tank. This tank will be constructed to create a high service area to address pressure and fire flow concerns in this area and the storage tank will also include an aeration system. The water in this tank will be aerated which will release some of the disinfection by-products, resulting in higher quality of water.

4.7.2 Storage Tank Operation

Maintaining fire protection and water quality have competing goals in terms of the recommended range in operating levels of storage tanks. To maximize fire protection the storage tank water level should be maintained as high as possible. However, to maintain chlorine residual for the protection of water quality, the entire volume of water in the storage tanks should "turn-over" every 5 days, or 20 percent every day. Based on a review of the City's water storage tank data, most tanks fluctuate approximately 5 to 10 percent. The City should continue their efforts to maximize water quality in the distribution system.

4.7.3 Storage Tank Maintenance

Maintaining clean water in storage tanks is a key component to maintaining distribution system water quality. In recent years, all water storage tanks have either been replaced or rehabilitated. While this work is recent, these water storage facilities should be inspected every five years to ensure continued, reliable service. An inspection budget of $20,000 every three years will maintain a consistent inspection routine for all seven storage tanks.

4.7.4 Pipe Maintenance

There is no record of the City having performed a recent leak detection survey of the distribution system. A leak detection survey can identify leaks in the distribution system located on mains, at fire hydrants or services, leaking valves or fittings. Based on the leak detection results, further focused improvement recommendations can be made. FST recommends that the City conduct a leak detection program at least once every two years. In addition to leak detection, the City should implement an annual valve maintenance program to make sure that all water main valves are fully operational and a meter replacement program to replace water meters of 10 years of age and older.

4.8 DISTRIBUTION RECOMMENDATIONS

4.8.1 Water Main Rehabilitation Program

As noted in Section 4.3, over the past decade, the City has undertaken a major water infrastructure replacement program, with financial assistance from the MADEP State Revolving Loan Program and Federal Stimulus grants and the City Street Improvement Program. Since 2001, the City has replaced 55 miles of unlined cast iron water mains with new ductile iron water mains. The water distribution system still includes approximately 45 miles of unlined cast iron or transite (AC) pipe that needs to be addressed. Newer installations, post 1970's, have been cement lined ductile iron pipe. Almost all of the mains 16-inches in diameter and larger were cleaned and cement-lined in the mid to late 1970's, greatly increasing their carrying capacity. The distribution system is extremely well looped, minimizing many of the dead ends that are potential causes for water quality problems. In place of dead ends, which allow stagnant water and create problems such as microbial contamination and discoloration, the City has over 60 bleeders throughout its distribution system. These bleeders are helpful in terms of water quality but they result in lost water and should be eliminated when possible.

Moving forward, FST has developed a prioritized list to continue the replacement program for the remaining cast iron and AC water mains. Since most of the remaining cast iron water mains are small diameter (6-inch and 8-inch) with an average installation date of 1915, including the age and size as criteria with which to prioritize the replacement moving forward is superfluous.

More practical, yet less automatic criteria for prioritizing these replacements include the following:

- Fire protection. Areas in the City that the Fire Department or the hydraulic model results have indicated insufficient fire protection due to reduced carrying capacity of the water mains are a high priority for water main replacement due to public safety concerns.
- Water quality issues. Streets where residents complain of dirty or colored water and where targeted flushing is common is a priority for the City to replace.
- Water pressure issues. While low water pressure problems are often attributed to elevation, excessively tuberculated water mains can allow little to no flow through the mains, resulting in low pressure for residents.
- Water main break history. Replacing the mains that break often not only saves lost water, it saves time and money for the emergency repairs.
- Lead service replacements. The City continues to include lead service replacement as a priority of replacements, however more often it is a secondary priority to other criteria such as water mains with pressure and quality concerns.
- Proximity to the water treatment plant. Beginning near the water treatment plant and working out into the City brings the cleanest water through the cleanest pipes.
- Coordination with other utility and roadway improvement programs. If there are upcoming utility or roadway improvements being completed by others, the City's priority is to complete all the work at once to lessen the inconvenience to the residents in that area.
- Coordination with the City's on-going CSO program. The City is under mandate to separate their combined sewers in central areas of the City. In areas where the water main needs to be replaced as well, the water main replacement should be prioritized before or at the same time as the CSO work.
- Coordination with recent paving projects. Any pavement surfaces that have been reconstructed in the past 10 years have a water main replacement with a lower priority due to cost. Additionally a budgetary $150 per linear foot of water main replacement is included for pavement restoration (milling, overlay, sidewalks, curbing, etc.) following the water main replacements.
- Completing geographical areas of the City. To date the replacement programs have been all over the City, yet in each area there are a few streets that still need to be completed.
- Beginning in about 15-20 years, the water storage facilities may need re-coating again, and as such the later years may have less water main replacement to allow for funding of the storage improvements.

A summary of each year's program is presented in Table 4-12. A full list of the water mains proposed to be replaced in this 20-Year program is included as Appendix M along with a map of the replacement program.

TABLE 4-12 20-YEAR PIPELINE IMPROVEMENT PLAN

Phase	Length(ft)	Planned Construction Year	Construction Estimate (incl. Engineering, contingency & police)	Additional Paving Allowance
13(*)	7,000	2014	$1,200,000	$950,000
14(*)	19,000	2015	$4,533,570	$2,895,000
15	16,245	2016	$3,823,200	$2,436,750
16	13,290	2017	$3,223,800	$1,993,500
17	16,040	2018	$4,033,800	$2,406,000
18	9,395	2019	$2,430,000	$1,409,250
19	15,860	2020	$4,244,400	$2,379,000
20	16,035	2021	$4,422,600	$2,405,250
21	17,410	2022	$4,941,000	$2,611,500
22	11,005	2023	$3,207,600	$1,650,750
23	12,470	2024	$3,742,200	$1,870,500
24	7,605	2025	$2,284,200	$1,140,750
25	10,495	2026	$3,223,800	$1,574,250
26	12,850	2027	$4,260,600	$1,927,500
27	13,455	2028	$4,584,600	$2,018,250
28	15,230	2029	$4,941,000	$2,284,500
29	13,635	2030	$4,536,000	$2,045,250
30	12,255	2031	$4,163,400	$1,838,250
31	7,500	2032	$2,559,600	$1,125,000
32	11,555	2033	$3,936,600	$1,733,250
33	14,490	2034	$4,924,800	$2,173,500
34	14,290	2035	$4,860,000	$2,143,500
Totals	287,110		$84,076,770	$43,011,500

(*) Currently in Design or Planning Stage

4.8.2 Water Distribution Operation

Proper operation and maintenance of the City's water distribution system can maximize the protection of public health by maintaining reliable, high quality water for the customers. Proper disinfection procedures during the installation of new mains and when repairing leaks and breaks, a tenacious cross connection control program, and a regular hydrant flushing program, together with valve maintenance program and leak detection are all key components to maintaining distribution system water quality.

4.8.2.1 Pipe Maintenance

The City currently completes targeted distribution flushing when discolored water complaints are received. The City should continue an annual water distribution flushing program. In addition to distribution system flushing, a valve exercising program should also be implemented. A budget of $60,000 should be allocated for an annual flushing and valve exercising program. Additionally, an annual budget of $50,000 should be allocated to repair or replace any inoperable hydrants and valves encountered during the flushing and valve exercising programs.

The City should also implement a leak detection program, at least once every two years to identify and repair any leaks. While City's percent unaccounted for water has typically been approximately 11%, identifying and repairing leaks on an annual basis will allow the City to maintain an unaccounted for water level of less than 10%. An budget of $40,000 should be allocated for a leak detection survey every two years with an annual leak repair budget of $50,000.

4.8.2.2 Water Meters

In addition to leak detection, annual meter replacement will also assist with minimizing unaccounted for water by accurately measure the water consumed at each service connection. While replacing or right-sizing commercial and industrial meters can capture additional water used at these facilities, replacing a portion of the smaller residential meters on an annual basis will contribute to having accurate and operable meters at all residential connections. A budget of $300,000 should be allocated to replace 1,000 meters on an annual basis.

In addition to annually replacing water meters, the City should also update the database of meters. Currently the meter database does not include a field associated with the user class (residential, commercial, industrial, etc.). This makes it difficult to accurately report annual water use by classification to the DEP. By adding this information to the database, the reporting accuracy will increase.

4.8.3 Water Facility Maintenance

As part of the water system operations, there are a number of facilities and areas that require maintenance. Many of these buildings are located out of sight of the general public and as a result, general maintenance practices have been limited due to lack of funding. Some of these buildings date back to the 1800s and could be eligible for registration as historic structures and could be re-purposed for water department operations. The following summarizes the improvement needs in and around the Water Department Facilities.

4.8.3.1 Cobblestones

The north and south sides of Bedford Street, between the Water Department building at #1620 and the Route 24 overpass, had long been used by the City as disposal areas for cobblestones and other construction related debris. In the interest of further protecting and improving the watershed of Watuppa Pond, the plan for the removal and disposal of the cobblestones and other construction related debris and site restoration for the north side of Bedford Street should continue and a plan for the south side should be developed and implemented. The pile of debris includes cobblestones, as well as miscellaneous material such as railroad ties, concrete, bricks and metal. This material should also be removed and property disposed of offsite.

4.8.3.2 Distribution Maintenance Building & Area

The Water Department currently has multiple antiquated buildings and facilities on both sides of Bedford Street. The buildings and garage space are in need of repair and there is inadequate indoor storage space for materials, equipment and vehicles. The City should consider an evaluation of the buildings and maintenance facilities as detailed in the following tasks:

- Evaluate usefulness and condition of existing Distribution Maintenance Building.
- Evaluate historical significance of building(s).
- Develop plan for rehabilitation or replacement of Distribution Maintenance Building, to provide efficient use and adequate space for the following:
    - office space for
        - staff use
        - document storage
        - public access for meter sales
    - indoor garage space for adequate vehicle storage and maintenance
    - bulk pool water sales
    - fuel depot
    - spare parts and material inventory storage
    - outdoor parking for staff and customer vehicles
- Develop a plan to upgrade or relocate the materials work area on the south side of Bedford Street including the following:
    - Complete a soils testing and ground water monitoring program
    - Develop a plan for slope stabilization
    - Provide for an area of controlled material storage (i.e. concrete block bays)
- Consider a plan to utilize the City-owned space to the north of #1620 Bedford Street for the relocation of maintenance and storage facilities.
- Develop a plan for the rehabilitation and upgrade of the altitude valve / transmitter house in the vicinity of the Bedford Street tanks, including the piping, the altitude valves, and the building and vault that house the equipment.

4.8.3.3 1950 Pump Station / Screen House

The circa 1950 pump house and screen house, located along the Watuppa Pond access road, became obsolete with the construction of the WTP in the 1970's. The buildings are no longer operational or useful for their originally intended purpose. In the interest of protecting and improving the watershed of Watuppa Pond, the City should consider the following tasks:

- Evaluate usefulness of buildings, equipment, and materials.
- Evaluate historical significance of buildings.
- Develop plan for removal of pump house and screen house.

4.8.3.4 Historical Facilities

The City has been providing Watuppa Pond drinking water to its people since the 1870's. Several of the original facilities from that time are still standing, but are in a state of severe disrepair. The City has been working with historical architects, and should continue to pursue this specialized assistance, in the interest of renovating and preserving these historical facilities. The following summarizes the items that should be addressed by the City:

- Contract with Historical Architect sub-consultant to complete the evaluation of the buildings and site at the following locations:
  - 1873 pump house / screen house (Stabilization improvement program nearing completion)
  - 1873 storage tank / tower
  - 2929 Blossom Road Headquarters
- Investigate Historic Preservation Grant Funding
- Develop plan for protection against further deterioration of facilities.
- Determine potential future usefulness of each building.
- Develop plan for rehabilitation of buildings.
- Identify any necessary permitting required.

4.8.3.5 Distribution Vehicle Maintenance and Replacement Program

The Fall River Water Department owns and maintains over 40 vehicles and large equipment consisting of pickup trucks, work and utility trucks, dump trucks and trailers, compressors and message boards. A full list of the vehicles and equipment including make, model and year, is included as Appendix N. With a few noted exceptions, most of the vehicles and equipment are less than 15 years old. Several of the City's trailers are 30 to 40 years old. The City should continue to maintain the vehicles and equipment to keep them operable and safe and extend their useful life as long as possible. The City should budget $100,000 per year for vehicle maintenance and replacement as the vehicles begin to reach the end of their useful lives.

4.8.4 Recommendations

The following Table 4-13 summarizes the estimated costs for the water distribution system recommendations.

TABLE 4-13 WATER DISTRIBUTION RECOMMENDATIONS

Item No.	Improvement Recommendation	Estimated Cost
1	Redundant Transmission Line from WTP	$3,000,000
2	Annual Pipeline Replacement Program	$1,200,000 to $5,000,000 per year
3	Paving allowance for annual pipe replacement program	$950,000 to $3,000,000 per year
4	Cleaning and Painting of Storage Tanks (every 15-20 yrs)	$8,500,000
5	Replace Airport Road tank, construct Commerce Drive Pump Station	$4,000,000
6	Future High Service Area Dedicated Water Main	$500,000
7	Investigation and Improvements for Low Pressure Areas	$4,100,000
8	Perform Leak Detection Survey of approximately 250 miles of water main (every 2 years)	$40,000/2 yrs
9	Allowance for leak repair	$50,000/yr
10	Comprehensive Hydrant Flushing & Valve Exercising Program	$60,000/yr
11	Annual Valve and Hydrant Replacement Allowance	$50,000/yr
12	Storage Facility Inspections (every 3 to 5 years)	$20,000/3yrs
13	Annual Meter Replacement Program (1,000 meters per year)	$300,000/yr
14	Debris Removal along Bedford Street	$1,400,000
15	Distribution Maintenance Area, Buildings, and Structures	$5,200,000
16	Evaluate / Remove 1950 Pump Station and Screen House	$1,000,000
17	Repair and Rehabilitate Structures of Historical Significance	$2,500,000
18	Distribution Vehicle Maintenance and Replacement	$100,000/yr
	CAPITAL COST TOTAL	**$170 million**