WATER SUPPLY SYSTEM ASSET MANAGEMENT PLAN

Northampton, Massachusetts

April 2013

TATA & HOWARD

Water and Wastewater Consultants

Water Supply System Asset Management Plan
Northampton, Massachusetts

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TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

SECTION 1 - Executive Summary

1.1 General

Tata & Howard, Inc. was retained by the City of Northampton Department of Public Works (City) to complete a Water Infrastructure Evaluation and Capital Improvement Study for the City's water system. The purpose of the study is to identify areas of the system in need of rehabilitation, repair, or replacement and prioritize improvements to make the most efficient use of the City's capital budget. The study evaluates the existing water infrastructure including water transmission and distribution piping and appurtenances. In addition, water storage needs are evaluated and prioritized.

In accordance with the January 2008 Sustainable Northampton Comprehensive Plan, the Water Supply System Asset Management Plan accomplishes several listed goals. As discussed in goal IC-1, this study ensures the capital improvement program is coordinated with Sustainable Northampton plan goals and objectives by monitoring the status of infrastructure and creating a prioritized schedule for replacements and upgrades based on development patterns and density. In accordance with Goal ED-2 it also provides long-term economic sustainability and security by creating an efficient plan of action for the City's water infrastructure rehabilitation and development and provides fire protection security throughout the City. The Asset Management Plan also provides a positive business environment by creating a plan for infrastructure investment to provide adequate supply to the existing and projected businesses throughout the City, as stated in Goal ED-4.

Tata & Howard evaluated the water distribution system using the Three Circle Approach. The Three Circle Approach includes the following evaluation criteria:

- System hydraulic evaluation,
- Critical component assessment,
- Asset management considerations.

Each circle represents a unique set of evaluation criteria for each water system component. From each set of criteria, system deficiencies are identified. System deficiencies from each circle are then compared. Any deficiency that falls into more than one circle is given higher priority than one that does not. Using the Three Circle Approach, recommended improvements will result in the most benefit to the system. In addition, the Three Circle Approach allows us to identify any situations that mitigate a deficiency in one circle while eliminating a deficiency in another circle. By integrating all three sets of criteria, the infrastructure improvement decision making process and overall capital efficiency is optimized.

Priority 1 hydraulic improvements are intended to mitigate system pressure deficiencies and strengthen the transmission capabilities from the WTP. Priority 2 hydraulic improvements are intended to strengthen the transmission capabilities in the system or provide the recommended ISO fire flows. Priority 3 hydraulic improvements were identified as part of a system wide evaluation to improve fire flows.

A critical component assessment was performed for the water distribution system to evaluate the impact of potential water main failures on the system. The critical component assessment includes identification of critical areas served, critical water mains, and the need for redundant mains. Critical areas served were identified by the City and include water department facilities, hospitals, schools, and business districts. Critical water mains include primary transmission lines as well as water mains that cross over major highways, rivers, and railroad tracks.

An asset management assessment was completed for the system. The asset management assessment included rating each water main in the system with a score of 1 through 100. A number of factors are considered in the ratings including: age, material, diameter, break history, soil conditions, water quality, and pressure. These factors affect the decision to replace or rehabilitate a water main. Using our asset management rating approach, each water main in the system was assigned a rating based on these factors. Water mains with a total rating less than 35 are considered to be in good to excellent condition. Mains with a total rating ranging from 35 through 55 are considered to be in fair to good condition, and mains with a total rating of 56 or greater are considered to be in poor to fair condition.

Utilizing the Three Circles Approach, improvements were recommended and prioritized based on the aforementioned criteria. Phase I improvements include any recommended improvements that fall into all three circles and are hydraulically deficient, critical, and have a high asset management score. The total estimated probable construction cost of the Phase I recommended improvements is $12,658,000. Phase II improvements include any recommended improvements that fall into two of the circles. The total estimated probable construction cost of Phase II recommended improvements is $14,144,000. Phase III recommendations include any recommended improvements that are needed hydraulically or any recommended improvements that have a high asset management score indicating poor condition. The total estimated probable construction cost of Phase III improvements is approximately $7,510,000. These recommendations are discussed further in Section 7.

In addition to the prioritized water main improvements, general recommendations were developed to improve operation of the water distribution system. These recommendations include a raw water main evaluation, a low lift booster pump station, and development of a unidirectional flushing program. The total estimated cost for the general recommendations is approximately $2,767,000**.**

SECTION 2 - Existing Water Distribution System

2.1 General System Overview

The City of Northampton provided a description of the City's water system for this report. The system currently includes three surface water reservoirs, two groundwater supplies, two water storage tanks, and approximately 154 miles of distribution piping ranging in size from 4-inch to 36-inch in diameter. The system includes 8,667 metered service connections. The 2011 average daily finished water consumption for the system was reported to be approximately 2.835 million gallons per day (mgd), with approximately 57 percent attributed to residential usage and 43 percent attributed to industrial, agricultural, commercial, and other non-residential usage. In 2011 the residential water usage was 53 gallons per person per day and the unaccounted for water was calculated to be 10.1 percent.

The distribution system includes two major service areas, the Main Service System and the Leeds Village High Pressure System. The Main Service System includes the majority of the City's water system, and is gravity-fed from the clearwell at the Mountain Street Water Treatment Plant (WTP) through two transmission water mains. The transmission mains include an older 20-inch diameter field lined cast iron water main and a more recently installed 36-inch diameter ductile iron main. Both supply mains extend into the City's Main Service System from the Town of Williamsburg through the Beaver Brook Pressure Reducing Valve (PRV) Station located near the Williamsburg/Northampton town line. This PRV station was installed in conjunction with the 36-inch diameter main to reduce the higher supply gradient being provided from the 36-inch diameter main. Both supply mains are fed through the PRV station to maintain a gradient between approximately 400 and 415 feet for the distribution system. All elevations in this report are based on the National Geodetic Vertical Datum (NGVD).

Service elevations in the Main Service System range from as low as 120 feet at the south end of Mount Tom Road to as high as 350 feet at the south end of Florence Road. Storage for the Main Service System is currently being provided by the Mountain Street WTP clearwell which maintains an overflow operating gradient of 550 feet. The Turkey Hill Tank is an inactive, off-line water storage tank located in the Main Service System downstream of the PRV station. The overflow elevation of the Turkey Hill Tank is approximately 410 feet. The Leeds Village High Pressure System is located in the north-western section of the City and includes a booster pumping station, and one water storage tank, which maintains an overflow operating gradient of approximately 533 feet for the system. The Reservoir Road Tank is a steel ground level storage tank constructed in the mid-1980's. This tank is currently offline and drained. The City also maintains a smaller booster system that serves the residents along the upper section of North Farms Road. This system includes a below-ground booster pump station which maintains an operating gradient of approximately 510 feet.

2.2 Water Supply Sources

The water supply system for the City of Northampton includes both groundwater and surface water sources. The surface water supply consists of three reservoirs, Francis P. Ryan (Ryan), West Whately, and Mountain Street. The City also maintains a fourth reservoir, the Middle Roberts Reservoir, as an emergency source. The groundwater supply consists of the Clark Street Well (Well No. 1) and the Spring Street Well (Well No. 2).

Surface Water Supplies

The West Whately Reservoir operates as a diversion dam on West Brook with a discharge to the Mountain Street Reservoir located outside of the West Brook Drainage Basin. The West Whately Reservoir was constructed between 1901 and 1904. The West Whately Reservoir has a spillway elevation of 596 feet and an intake elevation of 582 feet. Fed primarily by an upper tributary of West Brook, the source water originates from the West Whately Reservoir and flows cross country through a 20-inch diameter transmission main discharging to an open channel. The open channel stream is referred locally as Borowski Brook and runs for approximately 4,000 feet before discharging to the Mountain Street Reservoir. According to the Final Report to the City of Northampton Analysis of Reservoir Safe Yield prepared by Metcalf & Eddy in October 1995, the safe yield of the West Whately Reservoir is 0.272 mgd. Spillway overflow from the West Whately Reservoir enters West Brook and eventually the Mill River in Hatfield.

The Mountain Street Reservoir in Williamsburg was constructed between 1901 and 1904. The dam has a spillway elevation of 459 feet and an intake elevation of 435 feet. According to the 1995 Metcalf & Eddy study, the Mountain Street Reservoir has a safe yield of 0.632 mgd. The Mountain Street Reservoir is fed by the discharge from the West Whately Reservoir and by the surrounding watershed. A portion of the watershed drainage area to the east of the reservoir is diverted outside the reservoir by two dikes and a drain pipe. The drain pipe has inlets by the dikes and then runs along the bottom of the reservoir. The drain discharges to the south of the dam. Discharge over the spillway enters Beaver Brook and eventually flows to the Mill River in Northampton. Water from this reservoir is pumped to the Water Treatment Plant by one or more of the three low lifts pumps at the low lift pump station through a 20-inch diameter transmission main. The pumps are operated by variable frequency drives, are equipped with 200 hp motors, and have a capacity of 3,400 gpm at 160 feet of total dynamic head (TDH).

The Ryan Reservoir on Avery Brook was constructed in 1971. This reservoir is the primary source of water for the City. The reservoir is fed by Avery Brook and other streams in the watershed. The spillway has an elevation of 675 feet and the overflow discharges to the West Whately Reservoir. There are three intake levels for the reservoir. According to the 1995 Metcalf & Eddy study, the safe yield of the Ryan Reservoir is 2.616 mgd. The combined watershed for the Ryan and West Whately Reservoirs extends from Whately into the Towns of Williamsburg and Conway. The 1995 Metcalf & Eddy study states that an additional 1.523 mgd yield results from the combined operations of the three reservoirs. The approved cumulative safe yield for the three reservoirs is 5.043 mgd.

The City has historically maintained three reservoirs in Roberts Meadow. Currently, the Upper Roberts Meadow Reservoir is scheduled for removal in the near future and the Lower Roberts Meadow Reservoir is used for swimming and recreation (Musante Beach). The Middle Roberts Reservoir is maintained as a potential back-up source of supply.

Groundwater Supplies

The City of Northampton currently operates two groundwater wells to supplement the surface water supplies. Both wells were installed in gravel deposits in the 1950s and are equipped with vertical turbine pumps housed in the pumping stations. The Clark Street Well (Well No. 1) is finished at a depth of 85 feet and is equipped with a 75 hp motor capable of producing approximately 1 mgd. The Spring Street Well (Well No. 2) is finished at a depth of 88 feet and is equipped with a 60 hp motor capable of producing approximately 0.8 mgd. Upgrades were completed between 2011 and 2012 at each well. These upgrades included well cleaning and redevelopment, installation of valve control and meter pit, complete electrical panel upgrades, new controls and alarms, and sodium hypochlorite injection equipment. Sodium hypochlorite is added at each well for disinfection. When operating, the wells will shut down automatically once the discharge pressure reaches 90 psi, which equates to about 120 psi in the downtown area.

Water Supply – Registered and Permitted Withdrawals

The City has two approvals from the MassDEP Water Management Act (WMA) program relative to water withdrawal. These approvals are a December 31, 2007 WMA Registration and an April 6, 2011 WMA Permit, both related to the ability of the City to withdraw water from the Connecticut River Watershed.

The City has a WMA Registration Statement (#10621401) for the period between 2008 and 2017 that was transmitted by MassDEP on December 31, 2007. The registration indicates that the City has four registered water withdrawal points, two groundwater points and two surface water points that are described as follows:

Well No. 1 – Source ID 1214000-01G
Well No. 2 – Source ID 1214000-02G
Roberts Meadow Brook – Source ID 1214000-02S
Mountain Street Reservoir – Source ID 1214-01S

From these sources the registered average volume per day that can be withdrawn is 3.96 mgd. It is important to note, that direct withdrawal of water from Ryan and West Whately Reservoirs is not allowed under this registration. Withdrawal from these two reservoirs is addressed in the City's WMA Permit. The WMA Permit was last issued on April 6, 2011. This permit expires on November 30, 2015. The permit authorizes water withdrawals from the following points:
Mountain Street Reservoir – Source ID 1214000-01S
Francis Ryan Reservoir – 1214000-02S
West Whately Reservoir – 1214000-03S

The permit authorizes an additional 0.84 mgd that can be withdrawn between 2008 and 2015, bringing the total withdrawal allowed to a maximum of 4.77 mgd.

2.3 Water Storage Tanks

The clearwell at the WTP in Williamsburg serves as the major source of storage for the City of Northampton. The City also has one active storage tank located within the distribution system.

The Audubon Road Tank is an elevated tank located in the Leeds Village High Pressure System. The tank was constructed in 1935 to an overflow elevation of approximately 533 feet.

The Turkey Hill Tank was taken off-line in 1995 and drained completely. The tank is a ground level steel tank that was constructed in 1987 to an overflow elevation of approximately 410 feet. With the existing system configuration this tank cannot be used as the overflow elevation is too low and it remains full and stagnant all of the time.

The Reservoir Road Tank is a steel ground level storage tank was constructed in the mid-1980's adjacent to the Roberts Meadow reservoir in Leeds. This tank is connected to the main system and is currently offline and drained. With the existing system configuration this tank cannot be used as the overflow elevation is too low and it remains full and stagnant all of the time.

2.4 Water Distribution System

The majority of the distribution piping is over 70 years old and is primarily composed of unlined cast iron and asbestos cement water mains. Water mains installed during the 1970's are cement-lined ductile iron. The City maintains a current map of the water distribution system data including water main age, material and diameters in ArcGIS. The existing distribution system consists of two main pressure zones or service areas: the Main Service System, that operates at a hydraulic gradeline (HGL) between 400 and 415 feet, and the Leeds Village High Pressure System that operates at a HGL of approximately 533 feet. As previously noted, there is also a small boosted service system in the vicinity of North Farms Road which operates at a HGL of approximately 510 feet. Approximately 94 percent of the City's water demand is located within the Main Service System.

2.5 Water Treatment Facilities

The City of Northampton currently operates three treatment facilities. The following section describes the various water treatment facilities that are part of the water system.

Mountain Street Water Treatment Plant

The Mountain Street Water Treatment Plant (WTP) receives and treats water from the Mountain Street and Ryan Reservoirs. The treatment processes at the WTP include chemical addition, mixing, clarification, filtration, pH adjustment, and disinfection. After the raw water enters the WTP, alum and a coagulant aid polymer are added to enhance coagulation and flocculation in the adsorption clarifiers. The chemically dosed water is mixed by a static mixer. Once the water has passed the static mixer, it is equally divided between on-line adsorption clarifiers. The WTP uses a high rate up-flow clarification process that separates solids from the water by attachment of the flocculated solids onto the surface of a buoyant media that is captured in the clarifier by a retaining screen. The solids collected on the media are removed by a backwash cycle which introduces air into the tank to expand and agitate the media bed. The solids released from the media are flushed away with incoming raw water and transferred to the equalization tank and lagoons for settling. Overflow from the lagoons is discharged to the Mountain Street Reservoir. From the adsorption clarifiers, water flows by gravity to the filter header which supplies water to the on-line activated carbon filters. The filter media must be backwashed periodically to remove collected solids. Backwashing forces water to flow in the reverse direction, up through the filter underdrain and media, to dislodge the solids collected in the media. The spent washwater generated from a backwash cycle is collected in the filter wash water troughs, and then flows into the filter gullet and into a spent washwater pipe leading to the equalization tank prior to the lagoon. Filtered water flows by gravity through the air break box, then through the flow meter chamber, and finally into the clearwell. Sodium hypochlorite is added to the filtered water downstream of the air break box for disinfection. This is referred to as “finished water”. Finished water flows by gravity from the clear well to the transmission mains which deliver water to the City.

The WTP is designed for a peak delivery of 6.5 mgd that includes 0.6 mgd for filter backwash. The minimum design flow is 1.5 mgd and the average design flow is 4.2 mgd. For a more detailed process description see the Water Treatment Plant Operation and Maintenance Manual dated February 2008, prepared by Metcalf & Eddy/AECOM.

Corrosion Control Facility

The corrosion control facility (CCF) was constructed in 1999 and placed on-line in February 2000. The CCF is designed to feed a 24% solution of sodium hydroxide (caustic) for pH adjustment and liquid zinc orthophosphate as a corrosion inhibitor. The orthophosphate reacts to stabilize the water and reduce corrosion by forming a protective film over lead and other exposed metals in the water distribution system.

Since the completion of the WTP, the sodium carbonate (soda ash) addition is now introduced at that WTP and the CCF is only used for zinc orthophosphate addition.

Chlorinator Facility

Prior to the construction of the WTP, the City utilized three chlorinator facilities, Station No. 1, 2, and 3 that used gaseous chlorine for disinfection. When the WTP went on-line, the chlorinators were no longer used. Station No. 1 has been completely decommissioned and the building has been removed. Station No. 2 is being maintained for emergency use. If needed, a temporary connection from the Mountain Street Reservoir to the Mountain Street transmission main can be made upstream so chemicals can be added for disinfection or other purposes. Station No. 3 formerly provided booster chlorination to the Mountain Street transmission main. As with the other two facilities, it has not been used since the WTP was put on-line. This location continues to be used as a facility to monitor chlorine residual, pH, temperature, color and turbidity.

SECTION 3 – System Demands and Storage Evaluation

3.1 General

For the purposes of evaluating the water needs of a community, several parameters are typically reviewed to better understand the demands of a distribution system. These parameters are defined in the sections below and are presented with their existing and projected demand estimates.

3.2 Population Projections

Because population has a direct correlation to water consumption, population projections from various sources through the year 2032 were reviewed to reflect actual and planned growth within the City. The following section reviews historical population data and presents an estimated future population based on available information from the City.

According to the United States Census, the City of Northampton has experienced a decline in growth from 1980 to 2010. Based on the US Census data, the population in City was approximately 29,286 in 1980 and 28,549 in 2010.

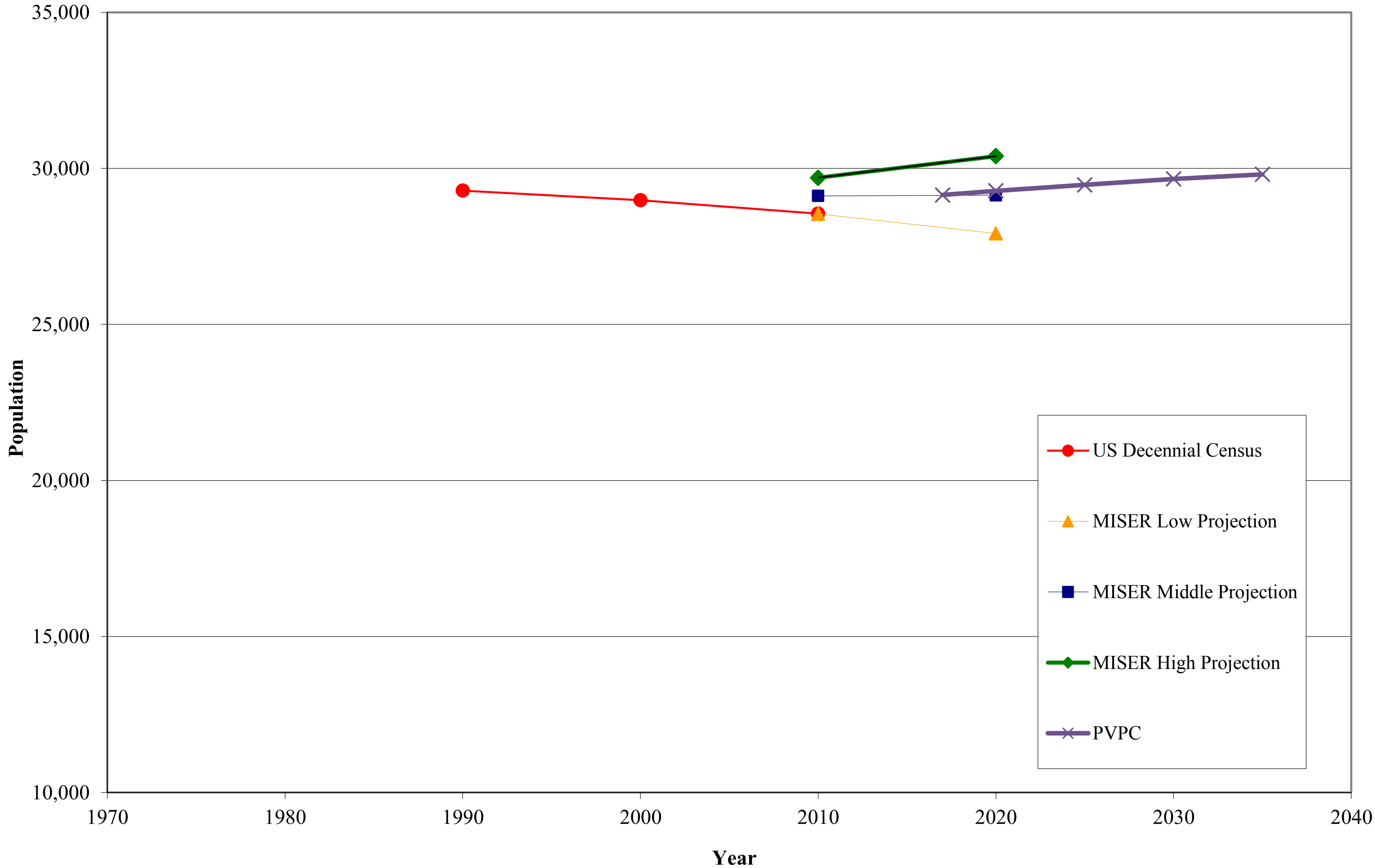
Population projections for the City of Northampton were obtained from Pioneer Valley Planning Commission (PVPC) and the Massachusetts Institute for Social and Economic Research (MISER). The PVPC projections and trend lines are similar to the MISER middle series projections. These projections indicate an estimated population of 29,807 people in 2035. Tata & Howard used the PVPC projections to estimate a 2032 population of approximately 29,720 people.

In addition to the population projections, known and planned residential developments listed in the Comprehensive Wastewater Management Plan completed by Kleinfelder in September 2012 were considered. These include Village Hill and The Oaks. The Village Hill and The Oaks development have approximately 254 proposed households. Based on data in the Kleinfelder report, it was assumed the average population per household in Northampton is 2.1. These developments will add approximately 530 people to the City's population. The projected 2032 population was adjusted to be approximately 30,250. The historic and projected population of the City is included on Figure No. 3-1.

3.3 Water System Demands

The Massachusetts Department of Conservation and Recreation (DCR) follows specific guidelines when projecting the water usage for communities in conjunction with the MassDEP Water Management Act program. These guidelines incorporate trends in the use of water conservation devices in homes and industry, and emphasize the importance of monitoring the distribution system through water audits and leak detection surveys to reduce unaccounted-for

Figure No. 3-1
Historic and Projected Population



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water. It is important to note that the DCR has a key role in the water management approval process. The DCR has not recently completed demand projections for the City of Northampton. Demand projections must be approved by the DCR before MassDEP will approve development of a new water supply source or authorize the withdrawal of additional volume from existing sources.

Based on recent developments, the Massachusetts Water Resource Commission (MWRC) has adopted new Water Management Standards for all registered and permitted withdrawals. The policy includes performance standards and conditions for all registered and permitted public water suppliers in the following areas:

- Maximum residential consumption of 65 gallons per capita per day (gpcd).
- Maximum of 10 percent unaccounted-for water.

Residential Consumption

Residential consumption is calculated by dividing water supplied to residential connections by the reported population. MassDEP has developed standards for all Public Water Suppliers to meet 65 gpcd. Public Water Suppliers currently meeting 65 gpcd will be required to develop a Seasonal Demand Management Plan to manage non-essential outdoor water usage. Public Water Suppliers that have not consistently met the 65 gpcd will be required to develop and implement MassDEP approved Compliance Plans including the use of Best Management Practices to meet the residential consumption standard. The 2008 through 2011 Annual Statistical Reports (ASR) indicate that the system has an average residential consumption of approximately 60 gpcd. However the range was from 50 to 73 gpcd, with both 2009 and 2010 consumption rates higher than the 65 gpcd standard.

Unaccounted-for Water

Unaccounted-for water (UAW) consists of unmetered water used for street cleaning, water main flushing, meter inaccuracy, unauthorized water uses, fire fighting and leakage in the distribution system. This term is typically expressed as a percentage of the total water supplied to the system. The UAW can be estimated by taking the difference between the total amount of water supplied and the total water billed and dividing by the total water supplied.

The 2008 through 2011 Annual Statistical Reports (ASR) indicate that the system has an average unaccounted-for water of approximately 14 percent. However, based on the Tata & Howard Water Audit Report of 2010, a comparison of the annual volume of water pumped from the sources to the amount of water metered by year concluded that the uncorrected UAW was approximately 14 percent in 2008, approximately 31 percent in 2009, and approximately 26 percent in 2010. The corrected average UAW based on the Water Audit Report indicates an average of 9.4 percent for the audit period (2008-2010), this includes adjustments for meter error and documented unmetered usage. The UAW for 2011 was approximately 10 percent.

Average Day Demand

Average day demand (ADD) is the total water supplied to a community in one year divided by 365 days. This term is commonly expressed in mgd. This demand includes all water used for domestic (residential), commercial, industrial, agricultural, and municipal purposes. The municipal component includes water used for system maintenance such as water main flushing and fire flows. In addition, the ADD includes unaccounted-for water attributed to unmetered water uses and system leakage. According to the 2008 through 2011 ASRs, the raw water ADD supplied for the system ranged from 3.10 mgd to 3.38 mgd.

The average raw water total usage of 3.33 mgd between 2009 and 2011 was used as a baseline system demand. Based on data from the ASR and the US Census population data, the water distribution system currently serves approximately 95 percent of the City. Based on this service percentage, the proposed 2032 service population is approximately 28,740. This includes the population projected by the PVPC and the proposed residential development in the proposed Village Hill and The Oaks developments. The additional projected flow of 0.74 mgd for known and planned developments and an additional assumed unaccounted for water of 10 percent results in a total 2032 projected demand of approximately 3.75 mgd.

The City of Northampton is also expected to have non-residential growth in the planning period. Projected wastewater demands for known and planned non-residential developments listed in the Comprehensive Wastewater Management Plan completed by Kleinfelder in September 2012. These demands include non-residential usage in the Village Hill development, the Route 110 Business Park, a Medical Center, the Clarke School, an Industrial Park, and additional growth at Coca Cola and the VA Hospital. It was assumed that the proposed water demand would be approximately 110 percent of the proposed wastewater demands. The proposed non-residential usage is approximately 0.7 mgd.

The projected population and demand proposed in the September 2012 Kleinfelder Comprehensive Wastewater Management Plan were utilized to project the water demands for the system.

The following criteria were used to develop the ADD for the design year 2032:

- Baseline ADD of 3.33 mgd
- Residential consumption of 65 gpcd
- Year 2032 service population of 28,740
- Maximum of 10 percent unaccounted for water
- Additional non-residential consumption of 0.70 mgd

The estimated ADD for the design year 2032 based on the above criteria is approximately 3.75 mgd, as shown in Table No. 3-1.

Summer Average Day Demand

MassDEP guidelines recommend that a system consider a projected summer ADD (SADD). The current SADD is estimated by averaging demands from the three maximum months for the past five years. As shown in Table No. 3-1, the raw water SADD ranged from 3.34 mgd to 3.96 mgd from 2008 to 2011. The SADD peaking factor is determined by dividing the SADD by the annual ADD for each of the past five years. These peaking factors are averaged to estimate the future summer peaking factor. Based on the 2008 through 2011 monthly demand data, the average summer peaking factor is 1.13. Based on the projected ADD of 3.75 mgd, the estimated 2032 SADD is approximately 4.23 mgd.

Table No. 3-1
Historic and Projected Water Use

Year	ADD (mgd)	SADD (mgd)	Peaking Factor (SADD/ADD)	MDD (mgd)	Peaking Factor (MDD/ADD)	Peak Hour (mgd)
2008	3.00	3.34	1.11	4.26	1.42	*
2009	3.37	3.63	1.08	4.39	1.30	*
2010	3.38	3.96	1.17	5.90	1.75	*
2011	3.23	3.71	1.15	5.28	1.63	*
-						
2022	3.72	4.20	1.13	6.51	1.75	10.42
2032	3.75	4.23	1.13	6.55	1.75	10.50

*Peak Hour information for 2008 through 2011 is not available.

Maximum Day Demand

Maximum day demand (MDD) is the maximum one-day (24-hour) total quantity of water supplied during a one-year period. This term is typically expressed in mgd.

MDD is a critical factor to be considered when determining the adequacy of a water supply system. The distribution system must be capable of meeting maximum day demands with coincident fire demands at a minimum pressure of 20 psi. Estimates of the projected maximum day demand and an allowance for the required fire flow are used to evaluate or design transmission mains, and pumping and storage facilities.

The projected MDD can be estimated by the MDD/ADD ratio. The MDD/ADD ratio provides a relationship between the two demands which can be used to estimate future demands. As shown on Table No. 3-1, the raw water MDD ranged from 4.26 mgd to 5.90 mgd from 2008 to 2011. Upon comparison of the MDD to the ADD, the ratios range from 1.42 to 1.75. To be conservative, the highest historical peaking factor was used to estimate future MDD. The resulting projected MDD for year 2032 is estimated to be 6.55 mgd based on the projected 2032 ADD of 3.75 mgd.

Peak Hour Demand

Peak hour demand is the maximum total quantity of water supplied in a single hour over a one-year period typically expressed in mgd. These demands are typically met by distribution water storage facilities. The peak flow rate recorded on July 13, 2012 was approximately 4,020 gpm (5.74 mgd). Based on the 2012 ADD of 3.32 mgd, the City observed a peak hour/ADD peaking factor of 1.75 in 2012. Because this only represents one year of data and the peak hour/ADD peaking factor is the same as the 2010 MDD/ADD ratio, a peaking factor based on typical historical consumption for communities of similar size was also considered. The MDD/ADD ratio for a community can be used to estimate the peak hour/ADD peaking factor. Using a MDD/ADD ratio of 1.75, the corresponding peak hour peaking factor for the system is estimated to be 2.8. Because this number is more conservative than the 2012 peaking factor it was used to estimate a projected peak hour flow. This conservative peaking factor also allows for flexibility in the demand projections. Using a 2032 ADD of 3.75 mgd, the projected peak hour flow for the year 2032 is estimated at 10.50 mgd.

3.4 Adequacy of Existing Storage Facilities

Distribution storage is provided to meet peak consumer demands such as peak hour demands and to provide a reserve for fire fighting. Storage also serves to provide an emergency supply in case of temporary breakdown of pumping facilities, or for pressure regulation during periods of fluctuating demand.

There are three components that must be considered for evaluating storage requirements. These components include equalization, fire flow requirements, and emergency storage. The three components of the storage evaluation were calculated under current and future demand conditions.

Equalization storage provides water from the tanks during peak hourly demands in the system. Typically, this quantity is a percentage of the maximum day demands. The percentages can range from fifteen to twenty-five percent, with fifteen percent used for a large system, twenty percent for a mid-sized system and twenty five percent used for a small system. A system is considered small if it has less than 3,300 customers, while a system is considered large if it has more than 50,000 customers. The Northampton system would be considered a medium sized system. As a result, twenty percent of maximum day demand was used for the equalization storage calculations.

The fire flow storage component is based on the basic fire flow requirement multiplied by the required duration of the flow. The basic fire flow is defined as a fire flow indicative of the quantities needed for handling fires in important districts, and usually serves to mitigate some of the higher specific flow. Within the Northampton system, a basic fire flow of 3,500 gpm for three hours was used for the storage evaluation.

The emergency storage component is typically equivalent to an ADD. However, if there is emergency power available at the sources, capable of supplying at least an ADD, the emergency storage component can be waived. The WTP is equipped with emergency power capable of supplying an ADD, therefore, the emergency storage component can be waived.

1. Water Distribution System Equalization

- Mid sized system = 20 percent of the Maximum Day Demand
- Maximum Day Demand in year 2010 = 5.85 mgd
- Estimated Maximum Day Demand in year 2032 = 6.55 mgd

- Equalization (2010) = 0.20 x 5.90 = 1.18 million gallons (mg)
- Equalization (2032) = 0.20 x 6.55 = 1.31 mg

2. Basic Fire Flow Requirement

- Representative fire flow for Northampton = 3,500 gpm
- Duration of 3 hours or 180 minutes
- Basic Fire Flow Requirement = 3,500 gpm x 180 min = 0.63 mg

3. Emergency
- Waived

The total required storage for any given year is the equalization component plus the basic fire flow requirement plus the emergency component. Therefore, the existing (year 2010) and projected (year 2032) total required storage for the system is as follows:

- Total Required Storage (2010) = 1.18+ 0.63= 1.81 mg
- Total Required Storage (2032) = 1.31 + 0.63 = 1.94 mg

Leeds Village High Pressure System

The Audubon Tank supplies the Leeds Village High Pressure System. Based on historical usage, the MDD in the Leeds Village High Pressure System is approximately 0.08 mgd. Because the Leeds Village High Pressure System represents such a small portion of the system, 25 percent was used for the equalization storage calculation. The Leeds Village High Pressure System is primarily residential users, with some larger users. Therefore, the basic fire flow for the service area was determined to be 2,500 gpm. Currently there is an emergency generator located at the Leeds Village Booster Pump Station, therefore, the emergency component was waived.

1. Equalization
- Small sized system = 25 percent of the Maximum Day Demand
- Leeds Village High Pressure System Maximum Day Demand in year 2011 = 0.08 mgd
- Leeds Village High Pressure System Maximum Day Demand in year 2032 = 0.08 mgd

- Equalization (2011) = 0.25 x 0.08 = 0.02 mg
- Equalization (2032) = 0.25 x 0.08 = 0.02 mg

2. Basic Fire Flow Requirement
- Representative fire flow for Leeds Village High Pressure System = 2,500 gpm
- Duration of 2 hours or 120 minutes
- Basic Fire Flow Requirement = 2,500 gpm x 120 min = 0.3 mg

4. Emergency
- Waived

The current (year 2010) and projected (year 2032) total required storage for the Leeds Village High Pressure System is as follows:

- Total Leeds Village High Pressure System Required Storage (2011) = 0.02 + 0.3 = 0.32 mg
- Total Leeds Village High Pressure System Required Storage (2032) = 0.02 + 0.3 = 0.32 mg

Available Storage Summary

Under existing and projected ADD, MDD and peak hour demands, a minimum pressure of 20 psi should be maintained throughout the distribution system. The highest customer in the Main Service System is at an elevation of approximately 350 feet. The WTP clearwell supplies the Main Service Area prior to the system through the Beaver Brook PRVs. Based on the operation of the PRVs and the tank base of 525 feet, the entire tank volume is usable.

Because the Turkey Hill Tank and the Reservoir Road Tanks are currently offline, these tanks were not considered in the total available storage. Recommendations involving these tanks are addressed in subsequent sections.

The total projected required storage for the 2032 design year in the Main Service System system is approximately 1.8 mg. Based on existing storage in the Main Service System, the City will have an estimated storage surplus of over 2 mg. Although there is a storage surplus in the City, the clearwell is located more than four miles from the first customers and even further from the downtown area. Recommendations involving storage in the City and in the downtown area are addressed in subsequent sections and included in final recommendations.

The highest customer in the Leeds Village High Pressure System is at an elevation of approximately 396 feet. The Audubon Storage Tank supplies the Leeds Village High Pressure System. To maintain a pressure of 20 psi, the tank can drop to an elevation of approximately 442 feet. The base of the tank is at an elevation of approximately 443 feet, which would make the entire tank volume usable.

The total projected required storage for the 2032 design year in the Leeds Village High Pressure System is approximately 0.3 mg. The City will have an estimated storage deficit of approximately 0.1 mg. Recommendations to address the storage deficit in the Leeds Village High Pressure System are described in subsequent sections.

SECTION 4 – Hydraulic Model Verification and Evaluation

4.1 General

A comprehensive computer model was utilized to evaluate Northampton's existing water distribution system and to obtain a basis for recommending water infrastructure improvements to the existing system. The City will be able to use the updated computer model as a planning tool to assess the potential impact of proposed developments and system improvements prior to their construction.

4.2 Model Verification

The hydraulic model was created in Bentley WaterGEMS modeling software based on the water main GIS database provided by the City. WaterGEMS allows the user to conduct hydraulic simulations using mathematical algorithms while in an ArcGIS environment. As part of this study, the hydraulic model was verified under steady state conditions based on fire flow testing completed by the City and information pertaining to the sources, pumping stations, storage facilities, and pressure reducing valve (PRV) settings provided by the City. Additionally, the model was verified over an extended period. Looking at the system over an extended period simulation (EPS) allows the hydraulic model to account for changes in the distribution system over time. These changes include tank levels, pump controls, and demand verification. EPS can be used to review potential water quality concerns such as water age and chlorine residuals. The hydraulic model can be used as a planning tool to assess the potential impact of proposed developments and system improvements.

Model Creation

The hydraulic model was created from the City's existing GIS data. The City provided a GIS shapefile of the system's water mains. The water main information included water main materials and installation dates. The data from the GIS files was imported into the WaterGEMS software to create the pipes in the model. In addition, tank data, pump curves, and PRVs were added to the model.

Demand Allocation

The hydraulic model represents the distribution system with a series of pipe segments and nodes. Demands are allocated to the nearest junctions to represent actual metered demand in the system. Demand allocation was achieved using 2011 usage data provided by the City. The data included water account number, account type, customer name, customer address, total usage for the year, average use per day, and average use per 12 months. The tabular data was organized to obtain estimated annual demand for each account. The account addresses were geocoded using ArcGIS software. Once the location of each meter was determined and demands were placed into a GIS shapefile, demands were then allocated to the junctions in the model using the Demand Allocator feature of the WaterGEMS software. The Demand Allocator assigns the water usage associated with each meter to the nearest junction.

Steady State Verification

The computer model is represented by node, pipe, tank, and PRV information. Initial C-values were assigned to water mains based on material, diameter, and installation year. Larger diameter water mains were assigned incrementally higher C-values than smaller diameter pipes. Asbestos cement, ductile iron, and PVC water mains were given higher C-values and unlined cast iron water mains were given lower C-values.

Initial flow tests were conducted by the City at twenty four locations throughout the distribution system in February and March 2012. Table No. 4-1 presents the results of the fire flow testing. The data obtained from the fire flow tests served as input data for the model verification under steady state conditions. This data included water levels in the storage tank, PRV settings, static and residual pressure readings, and measurement of flows from hydrants. It is important that each simulation reflect actual field conditions at the time of testing. Actual field conditions include current demands on the system, varying pressure settings at the PRVs, and varying water levels in the tank.

The PRV settings at the time of the February and March 2012 flow tests were not recorded during each test. Pressure loggers were placed on the PRVs for two days in May 2012 to observe pressure trends at the PRVs. Based on the data recorded by the data loggers, the downstream pressures from the PRVs ranged from 13 psi to 18 psi, depending on the time of day. The pressure setting through the PRVs was adjusted based on the time of the flow tests. An average value of approximately 300 gpm was utilized to represent the Coca-Cola demand at a given point in the day during the February and March flow tests.

When the results of the model simulations compared to within five percent of the hydraulic data collected from the fire flow tests, the computer model was considered verified under steady state conditions, and mathematically represented the physical operating conditions of the existing water distribution system. After the initial verification, some of the fire flow tests could not be verified without significant adjustment to C-values. This generally indicates that a pipe diameter or material is incorrect in the model, or a closed or partially closed valve or other restriction exists in the system. The areas in question were discussed with the City for verification of water main diameters and materials. The water main diameters and materials were verified to be correct in the hydraulic model based on the City's records. The City checked valves in the vicinity of these flow tests and found none to be closed. The City performed additional flow tests in July 2012 and September 2012 to verify the field results of the initial testing.

Table No. 4-1
2012 Flow Test Data

		Residual Hydrant			Flowing Hydrant			
Test No.	Date	Location	Static Pressure (psi)	Residual Pressure (psi)	Location	Static Pressure (psi)	Estimated Flow (gpm)	Estimated Flow at 20 psi (gpm)
1	2/28/12	Leonard Street H-2182	60	29	Leonard Street H-817	60	800	900
1A	7/17/12	Leonard Street H-2182	57	25	Leonard Street H-817	19	730	750
2	2/28/12	Chesterfield Road H-792	39	29	Chesterfield Road H-793	34	460	650
2A	7/17/12	River Road H-866	71	40	River Road H-1024	71	750	950
3	2/28/12	Rick Drive H-1154	70	61	Rick Drive H-1163	70	1,190	3,000
4	3/8/12	Chestnut Street H-621	60	44	Chestnut Street H-620	60	380	600
5	3/5/12	Ryan Road H-874	44	30	Ryan Road H-875	48	920	1,200
6	2/29/12	Platinum Circle H-743	32	28	Diamond Court H-507	28	730	1,300
7	2/29/12	Brookside Circle H-1173	34	24	Brookside Circle H-1175	34	750	900
8	3/5/12	Rocky Hill Road H-2377	58	45	Rocky Hill Road H-1377	56	1,660	2,950
9	2/29/12	Westhampton Road H-422	52	40	Westhampton Road H-1229	54	960	1,600
10	3/7/12	Easthampton Road H-1056	106	92	Easthampton Road H-1057	104	1,350	3,600
11	3/7/12	South Street H-452	110	10	South Street H-453	114	375	350
11A	8/8/12	South Street H-447	106	83	South Street H-448	105	1,350	2,750
12	3/14/12	Belmont Avenue H-385	108	96	Belmont Avenue H-384	110	1,400	4,100
13	3/14/12	Bridge Street H-19	120	92	Bridge Street H-20	120	2,400	4,750
13A	9/26/12	Bridge Street H-22	114	102	Bridge Street H-23	114	1,190	3,600
13B	10/3/12	Main Street H-12	120	118	Main Street H-13	124	1,600	3,200
14	3/7/12	Pomeroy Terrace H-105	110	90	Pomeroy Terrace H-104	116	1,450	3,300

Table No. 4-1Continued
2012 Flow Test Data

		Residual Hydrant			Flowing Hydrant			
Test No.	Date	Location	Static Pressure (psi)	Residual Pressure (psi)	Location	Static Pressure (psi)	Estimated Flow (gpm)	Estimated Flow at 20 psi (gpm)
15	3/7/12	Island Road H-658	120	26	Island Road H-659	122	440	450
15A	8/23/12	Mount Tom Road H-520	128	103	Mount Tom Road H-654	127	1,130	2,500
15B	8/23/12	Mount Tom Road H-1165	124	95	Mount Tom Road H-655	126	1,110	2,200
16	3/8/12	Paradise Road H-336	94	80	Paradise Road H-335	96	2,150	5,300
17	3/8/12	Maynard Road H-353	84	50	Maynard Road H-354	86	530	750
17A	9/26/12	Elm Street H-530	87	76	Elm Street H-662	87	1,390	3,700
18	3/7/12	Massasoit Street H-314	86	60	Massasoit Street H-315	88	840	1,400
19	3/7/12	Winter Street H-271	104	96	Winter Street H-272	108	1,190	4,200
20	3/14/12	King Street H-191	120	114	King Street H-193	122	1,240	5,650
21	3/14/12	Damon Road H-208	120	82	Damon Road H-209	118	1,990	2,570
21A	11/20/12	Damon Road H-207	112	93	Damon Road H-209	N/A	1,400	3,300
		Damon Road H-219	116	93				3,000
22	3/14/12	Industrial Drive H-1211	114	82	Industrial Drive H-1212	114	1,400	2,500
22A	11/20/12	Industrial Drive H-1211	112	94	Industrial Drive H-1212	N/A	1,520	3,600
		Industrial Drive H-1213	114	94				3,500
23	3/14/12	Bridge Road H-630	82	70	Bridge Road H-631	90	1,110	2,700

Table No. 4-1 Continued
2012 Flow Test Data

		Residual Hydrant			Flowing Hydrant			
Test No.	Date	Location	Static Pressure (psi)	Residual Pressure (psi)	Location	Static Pressure (psi)	Estimated Flow (gpm)	Estimated Flow at 20 psi (gpm)
24	3/7/12	Marian Street H-1168	84	20	Marian Street H-1169	78	500	500
24A	8/8/12	North King Street H-637	98	34	North King Street H-638	98	560	600
24B	8/8/12	Hatfield Street H-1155	107	81	Hatfield Street H-649	107	780	1,500
24C	8/8/12	Pine Brook Curve H-634	100	60	Pine Brook Curve H-635	110	900	1,300
25	9/26/12	Jackson Street H-288	92	85	Jackson Street H-287	92	920	3,200
26	9/26/12	King Street H-202	110	98	King Street H-2227	110	1,465	4,350
		King Street H-201	110	93				
27	9/7/12	Locust Street H-559	66	57	Locust Street H-561	64	520	1,260
28	9/7/12	Village Hill Road H-2258	85	80	Village Hill Road H-2265	85	1,150	4,600

Extended Period Simulations

Once the model was considered verified under steady state conditions, the model was simulated over an extended period. Analyzing a system under an extended period simulation (EPS) allows the hydraulic model to account for changes in the distribution system over time. These changes include tank levels, pump controls, and demand variations.

The EPS involved entering controls for the Leeds Booster Pump Station. In addition to controls, demand patterns had to be created. Two residential patterns were established: one for the Leeds Village High Service Area (LHSA) and one for the Main Pressure System. The demands in the LHSA are significantly less than the demands in the City. As a result of the LHSA being served by a separate pump station and water storage tank with demands significantly less than those of the Main Pressure System, the LHSA demands would fluctuate differently than the City overall. A smaller residential area would typically see less demand use in the overnight hours and slightly higher peak usage in the early morning and evening. A commercial pattern was assigned to the downtown area, Industrial Road, Damon Road, and North King Street. Actual usage data for individual commercial users was not available. It was assumed that typical commercial businesses would use water between 7:00 am and 6:00 pm. A fixed pattern was used for the hospital and for Coca Cola to represent continuous demands. The demand patterns established from the EPS are shown in Figure No. 4-1.

The residential demands follow a diurnal pattern, with peak water uses occurring once in the morning and once in the afternoon. The demand patterns were developed based on the tank level and pressure readings recorded at three hydrants located in the distribution system during the week of May 14, 2012. The pressure readings show significant activity throughout the day. Observed and modeled pressures for a location on Olander Drive are shown in Figure No. 4-2. The pressures are higher during the overnight hours when demands are lower. As demands increase in the morning and commercial users come online, the pressures in the system decrease. The demands were adjusted until the tank levels and pressures in the model matched the actual tank level and resembled the pressure trend from information provided by the City and the data collected by the data loggers. The model tank level in relation to actual tank level in the Audubon Road Tank for May 15, 2012 is represented in Figure No. 4-3.

4.3 Evaluation Criteria

The hydraulic evaluation facet of the Three Circle approach evaluates the system’s ability to meet varying demand conditions. In general, a minimum pressure of 35 pounds per square inch (psi) at ground level is required during average day, maximum day, and peak hour demand conditions. During MDD with a coincident fire flow, a minimum pressure of 20 psi is required at ground level throughout the system. To evaluate the system’s ability to meet these criteria, the following hydraulic simulations were run in the model:

Figure No. 4-1
Demand Patterns

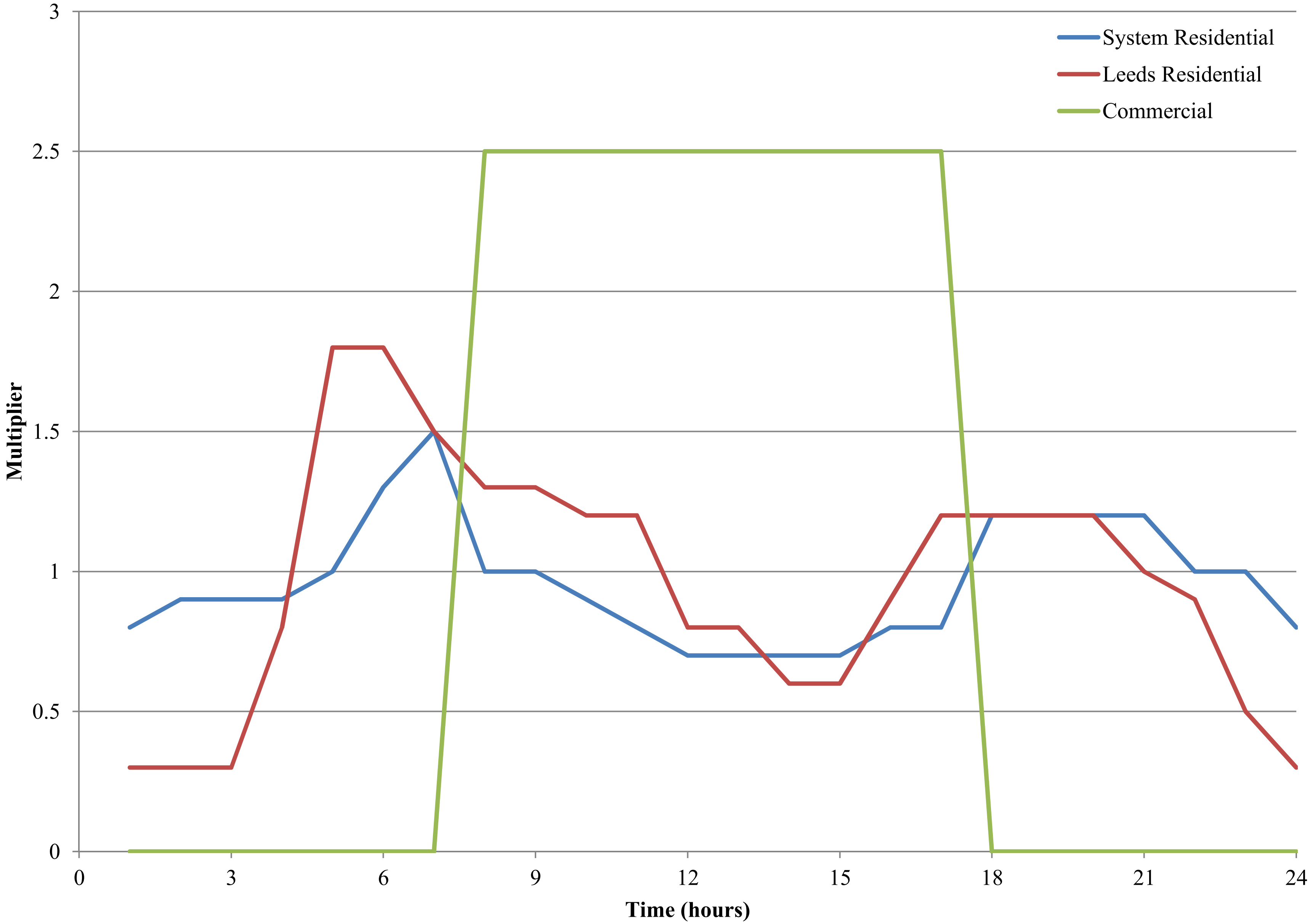

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Figure No. 4-2
Pressures at 91 Olander Drive

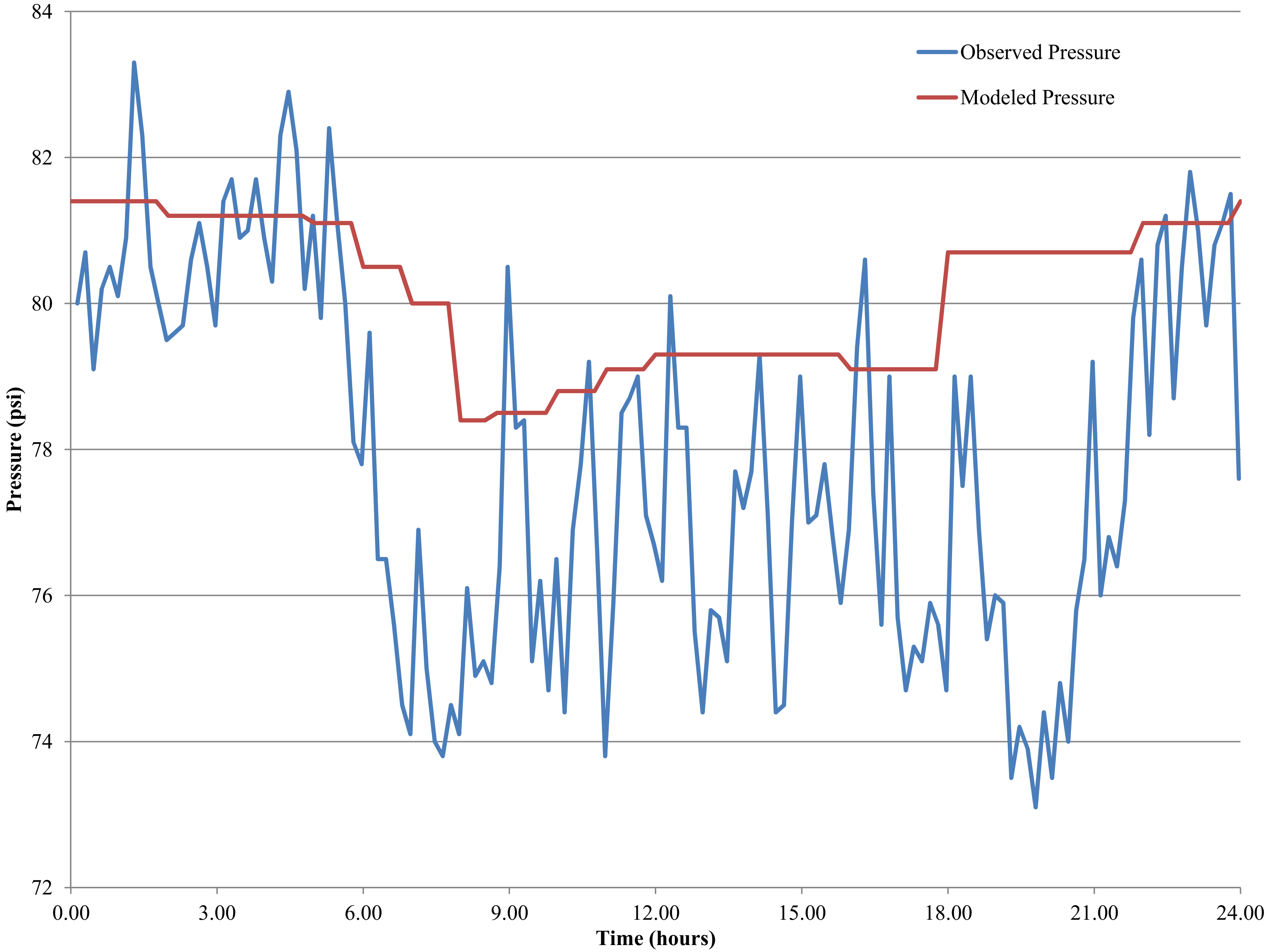

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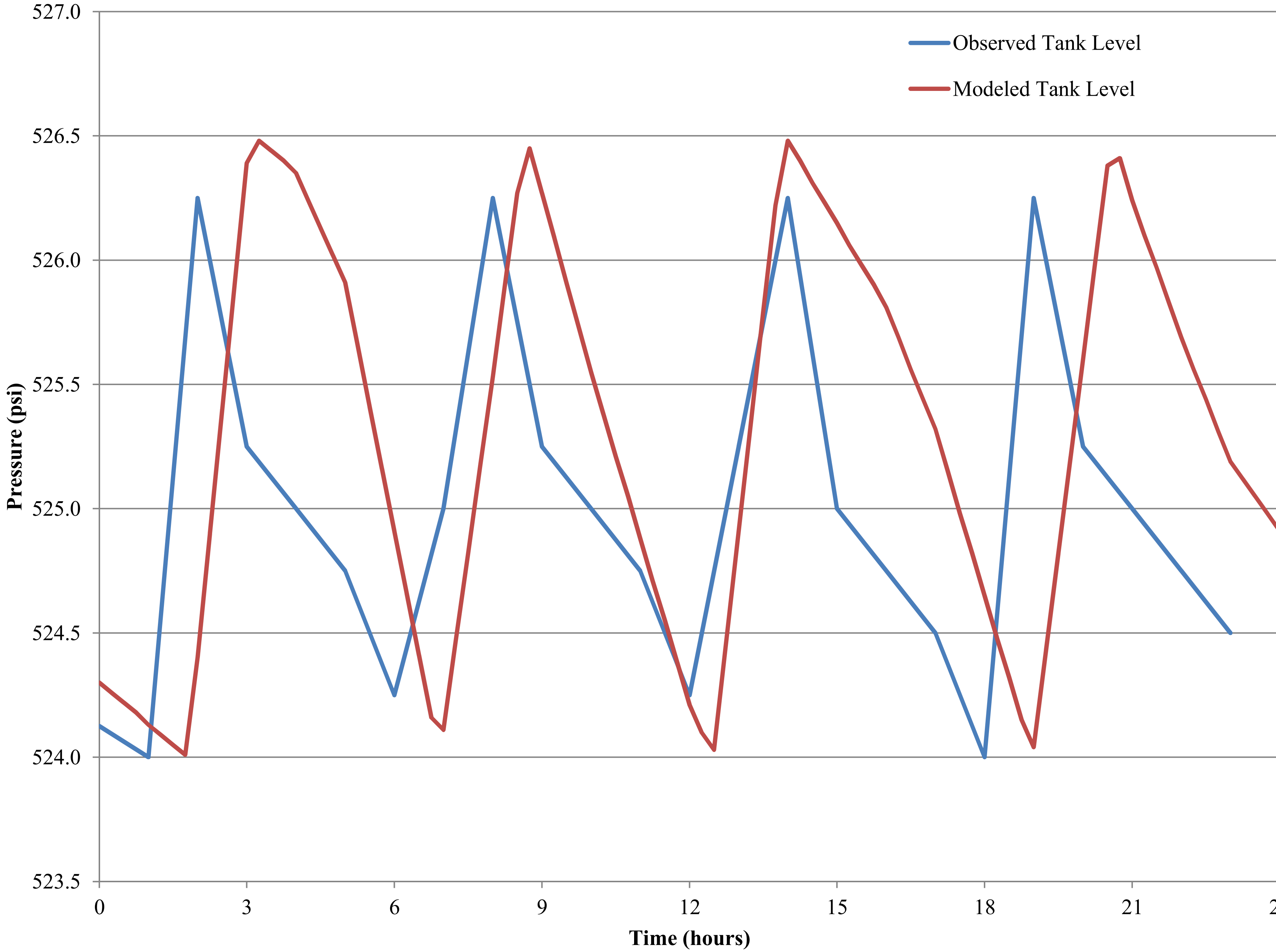
Figure No. 4-3
Audubon Tank Data



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Minimum/Maximum Pressure

During a projected year 2032 ADD, MDD, and peak hour demand condition (no coincident fire flow), a minimum pressure of 35 psi is recommended through the distribution system at street level. An upper limiting pressure of 120 psi is generally recommended, as older fittings in the system are generally rated at 125 to 150 psi. Pressures above this level can result in increased water use from fixtures and also increased leakage throughout the distribution system. Also, plumbing code states that water heaters in homes can be affected when pressures exceed 80 psi.

Under existing ADD conditions, static pressures are between 35 psi and 120 psi in most areas of the system. Some services in the low lying downtown area exceed 120 psi and some locations with higher elevations can experience pressures lower than 35 psi. Based on the current operating condition, with the HGL at the Beaver Brook PRV ranging from 400 to 430 feet, the City does not guarantee adequate static pressure above 320 feet in the Main Service System. System pressures in the Leeds Village High Pressure System range between 15 psi and 95 psi. The Leeds Village High Pressure System can provide 35 psi to service elevations up to approximately 440 feet. Areas in the Leeds Village High Pressure System above 440 feet are located west of the Audubon Road Tank. Based on the operating HGL, customers below an elevation of approximately 150 feet could experience pressures greater than 120 psi. Individual pressure reducing valves should be recommended to these customers.

Under existing and projected MDD and peak hour conditions, the system cannot maintain 35 psi throughout much of the western portion of the Main Service System. The operating HGL of the system cannot be adjusted without significantly increasing the static pressures downtown.

Insurance Services Office (ISO) Fire Flow Guidelines

The recommended fire flow in any community is established by the Insurance Services Office (ISO). The ISO determines a theoretical flow rate needed to combat a major fire at a specific location; taking into account the building structure, floor area, and the building contents. In general, the flows recommended for proper fire protection are based on maintaining a residual pressure of 20 psi. This residual pressure is considered necessary to maintain a positive pressure in the system to allow continued service to the customers and avoid negative pressures that could introduce groundwater into the system.

ISO testing was completed in September 2002. The test results indicate the available flow and estimated recommended fire flow in various sections of the distribution system at the time of the tests. The estimated recommended fire flows established by ISO varied from 500 to 6,000 gpm, depending on the location and the structure. It should be noted that a water system is only required to provide a maximum of 3,500 gpm at any point in the system.

The estimated recommended fire flows, as determined by the ISO, were simulated on the computer model and improvements were developed to meet the recommended needed fire flows for the deficient locations.

Additional Recommended Fire Flows

A review of the distribution system was completed to identify areas where larger buildings exist that were not identified in the latest ISO evaluation. Examples include condominiums, apartment complexes, schools, and commercial or industrial buildings. Recommended flows were estimated for these areas using the ISO published Guide for Determination of Needed Fire Flow. The guide uses factors including building size, material, location, and contents to establish a recommended fire flow. Larger multi-story buildings with larger areas have a higher recommended fire flow and buildings constructed of brick or concrete have a lower recommended fire flow than wood frame buildings of similar size. The recommended fire flow also factors in the proximity to neighboring buildings as well as the building material and size of the neighboring building. The contents of the buildings are classified based on the combustibility. The Guide for Determination of Needed Fire Flow gives examples of different types of buildings and the associated combustibility class. For example, a building with metal products storage would have a less combustible content than a paper products manufacturing plant. The factors used to estimate the recommended fire flows were estimated based on aerial photos and street level observations. The estimated recommended fire flows established based on this methodology are used as a guideline. Inspections were not performed on individual buildings and when assumptions were necessary the more conservative options was selected. Also, this methodology does not account for fire protection systems.

According to the American Water Works Association (AWWA), the minimum recommended fire flow in residential areas with one and two family dwellings not exceeding two stories in height is based on the distance between buildings. The recommended fire flow in areas with homes more than 100 feet apart is approximately 500 gpm, between 31 feet and 100 feet apart is approximately 750 gpm, between 11 feet and 30 feet is approximately 1,000 feet and 10 feet or less is approximately 1,500 gpm. The residential areas in the Northampton system were reviewed to determine the recommended residential flow in different areas.

4.4 Hydraulically Deficient Areas

The minimum and maximum recommended pressure conditions and estimated recommended fire flows were simulated on the computer model. Areas that do not meet the minimum and maximum recommended pressures were considered first. Hydraulic improvements were developed to alleviate pressure and fire flow deficiencies. Priority 1 improvements are intended to mitigate system pressure deficiencies and strengthen the transmission capabilities from the WTP. Priority 2 improvements are intended to strengthen the transmission capabilities in the system or provide the recommended ISO fire flows. Priority 3 improvements were identified as part of a system wide evaluation to improve fire flows.

Alternative Service Area Configuration

The existing system's Main Service Area HGL operates above the existing Turkey Hill and Reservoir Road storage tank overflow elevations and as a result, the facilities are offline. Additionally, in the western portion of the City pressures are low at the higher elevations, indicating that a higher operating HGL would be beneficial. Alternative scenarios were evaluated to improve low pressure concerns, including the possible use of the offline storage tanks. The existing transmission mains from the treatment facility to the system provide for an opportunity to increase the operating pressures in the western portion of the City without added pumping, as excess head is dissipated through a pressure reducing valve on each main.

If the existing 20-inch diameter transmission main was dedicated to a newly established pressure zone in this area, the HGL elevation of the new service area could be increased to mitigate the low pressures and allow for a newly constructed storage facility to serve the higher elevations. It has been estimated that an operating hydraulic gradeline elevation of 440 feet would be most appropriate for the new pressure zone. The existing 36-inch diameter main would continue to serve the main low service area at the lower elevations, and a newly installed pressure reducing valve between the western and eastern service areas would operate under extreme demand conditions in the main system, such as during high fire flow demands.

Under this scenario, the Main Service System would be divided into two pressures zones, a High Service Area (HSA) and a Low Service Area (LSA). The HSA will include the western portion of the City and would be supplied by the 20-inch diameter transmission main. The pressures in the 20 inch diameter transmission main are currently reduced at the Mountain Street PRV to a HGL of approximately 455 feet. This PRV will be used to control the HGL in the proposed HSA. The PRV will be controlled and operated based on water levels in a new water storage tank located off Ryan Road. Additionally, a PRV would be located between the HSA and the LSA on Pine Street. This PRV would be set to open during lower pressures, typically observed during emergency flow conditions. The HSA and LSA will be separated by isolation valves.

Currently both Well No. 1 and Well No. 2 would be located in the proposed High Service Area. Wells No. 1 and No. 2 pumps should be evaluated to determine if they can operate under the desired flow while maintaining the higher HGL in the proposed HSA.

Hydraulically, it would be ideal to have a tank in the downtown area to provide storage at the opposite end of the system source. However, due to the elevations and limited property resources of the downtown area, this option was not considered feasible. Alternative locations were considered for the proposed water storage tank in the proposed HSA. These locations include the existing tank location, Baker Hill Road, and Ryan Road. Two options were considered at the existing tank location. The first was to utilize a booster pump station to properly operate the existing tank. The cost of the construction and operating costs of a booster pump station made this option not feasible. The other option was to construct a new tank with a higher overflow elevation or to increase the existing overflow elevation. Previous studies have indicated adding rings to increase height of the existing tank was not feasible due to structural considerations. Due to the proximity of the tank to existing homes, the City did not want to consider constructing a higher tank at this location.

A new water storage tank on Baker Hill Road was designed in 1970, but was never constructed. The City does not own property in this area, and several residential homes have been constructed in this area since 1970. Based on ground elevations, the tank would have to be elevated. Due to the required tank elevation, and the potential difficulty with land acquisition, this location was not considered.

The City currently owns property off Ryan Road. Based on ground elevations, a tank at this location could have a base elevation of approximately 400 feet with an overflow elevation of 450 feet. This location is near an existing 12-inch diameter water main on Ryan Road and would require a short extension of the water main to connect to the existing system. Because of the location in the system, the ground elevations, and the City owned land, this is the most ideal tank location. The highest customer in the proposed HSA has a ground elevation of approximately 350 feet, with the proposed overflow elevation of 450 feet and a tank base elevation at 400 feet, the entire tank would be usable while maintaining 20 psi at the highest customer. Based on the storage evaluation completed in Section 3, there is adequate storage in the Northampton system. However, to provide redundancy and to provide better fire protection for the proposed HSA and LSA, the proposed tank should be sized to provide 3,500 gpm for three hours, or approximately 0.63 million gallons.

As discussed with the City, the additional hydraulic evaluations completed in this section have been evaluated under the assumption that the Main Service System would be divided into two separate pressure zones, a Low Service Area and High Service Area in conjunction with the existing Leeds Village High Pressure System.

The LHPS would remain at a HGL of approximately 533 feet and be served by the Audubon Road Tank with a connection directly to the 36-inch diameter transmission line at Leonard Street. The existing PRV on the 36-inch line will be relocated downstream of the connection to the LHPS. A PRV with controls will have to be installed at the connection to the LHPS. This PRV will be controlled by the water level in the Audubon Road Tank and will allow the tank to fluctuate. The Leeds Booster Pump Station (BPS) will have to remain in service as a backup supply to the LHPS. The pumps should be set to operate at an extreme low water level in the Audubon Road Tank in the event that the tank cannot be filled directly from the 36-inch transmission main.

The 20-inch diameter transmission main will feed the HSA and operate at the HGL currently set by the Mountain Street PRV. This allows for removal of the existing 20-inch PRV. This configuration will increase pressures south of the existing PRV in the existing 20-inch diameter water main. Due to the transmission main's age and its difficulty to access in many areas, consideration should be made to improve the reliability and accessibility for long term, dependable use. The proposed phasing and locations of the 20-inch diameter water main is addressed later in this section.

Because the HGL in the proposed LSA will remain the same, some areas at higher elevations could experience pressure less than 35 psi under projected peak hour conditions. These areas include Baker Hill Road, Warner Street, Lexington Avenue, Russelwood Ridge, Village Hill, and Coles Meadow Road. Low pressures in these areas are due to high ground elevations and headlosses in the water mains between the PRV's and these locations. Individual booster pumps could be recommended to customers in these areas to improve pressures.

Priority 1 Hydraulic Improvements

Priority 1 hydraulic improvements are intended to mitigate system pressure deficiencies and strengthen the transmission capabilities from the WTP. These improvements including replacing the transmission main from the WTP and developing an additional service area.

Replace 20-Inch Diameter Water Transmission Main

1. The existing 20-inch diameter water main is in areas that are difficult to access and runs cross country. Rehabilitating the water main with a structural lining will increase dependability, but will not improve the mains accessibility. Replacing the water main will increase dependability and make the water main more accessible by relocating to roadways, where feasible. The project would be from the Mountain Street PRV to Mountain Street and follow the existing roadways in Williamsburg and Haydenville to the existing 20-inch diameter water main on River Road in Northampton. The portion of the project south of the existing Beaver Brook PRV should be completed first. This is the portion of the main with the most difficult access and pressures will be increased in this portion of the main with the removal of the Beaver Brook PRV. The northern portion of the relocation project will put the 20-inch and the 36-inch diameter mains in the same roadway. Connections between the two mains, separated with closed valves, should be made wherever possible to provide redundancy in the event of an emergency.

Implement Three Service Areas

2. The system improvements necessary to divide the Main Service System into two service areas and to supply the LHPS from the 36-inch diameter transmission main are summarized below. This improvement will help maintain adequate pressures in the western portion of the City, improve transmission, and help improve the inherent fire flow capacity in the system. The City may have to reevaluate the Initial Distribution System Evaluation with the HSA and the new tank. This may result in new total coliform sampling sites in the HSA.

   To adjust the pressures of the divided system, we recommend the installation of three pressure reducing valves. The existing PRV on the 36-inch diameter water main will be relocated downstream of the new 8-inch diameter connection to the LHPS at Leonard Street. An additional PRV will be installed on the connection into the LHPS at Leonard Street, which will be controlled by the water level in the Audubon Road Tank. The existing PRV on the 20-inch diameter line will be eliminated to operate the HSA off the Mountain Street PRV. The third PRV should be installed on the 24-inch diameter water main on Pine Street near Nonotuck Street to allow water to flow from the HSA to the LSA during an emergency. The portion of water main on Pine Street, crossing the river, should be relocated onto the road to provide secure accessibility and increase reliability. Wells No. 1 and No. 2 pumps should be evaluated to determine if they can produce the desired flow while maintaining the higher HGL in the proposed HSA.

New Water Storage Tank

3. A new water storage tank is recommended in the proposed HSA to provide fire protection and redundant storage. We recommend a 0.75 mg water storage tank off

Ryan Road with a base elevation of approximately 400 feet, an overflow elevation of approximately 445 feet, and an operating range between 430 feet to 440 feet. The Mountain Street PRV that feeds the HSA will be controlled from the water level in the proposed tank to allow the tank to fluctuate. Once the configuration is completed, it is recommended that the Turkey Hill and Reservoir Road tanks be dismantled. The Turkey Hill Tank is currently out of service and the Reservoir Road Tank has been emptied and is also out of service due to the low overflow elevation.

Priority 2 Hydraulic Improvements

Priority 2 hydraulic improvements are intended to strengthen the transmission capabilities in the system or provide the recommended ISO fire flows. These improvements include replacing undersized water mains with larger mains or rehabilitating existing mains to improve the hydraulic capacity.

4. It is recommended that the water main on Federal Street and College Lane from Elm Street to Main Street be cleaned and cement mortar lined to improve overall transmission, system pressures, and flow. The water main runs along Federal Street, College Lane, Green Street, West Street, Paradise Road, Dryads Green, Wishing Avenue, and Washington Place. C-value tests, conducted by the City, indicate a low C-factor in portions of the water main. Further investigation is recommended to determine if this project can be phased and to determine the structural integrity of the water main.

5. Based on elevation, the pressures along Warner Street and Lexington Avenue constrict the available flow throughout the LSA. To improve flow and pressure, the existing 6-inch diameter water mains on Hinckley Street from South Main Street to Maplewood Terrace, Maplewood Terrace from Hinckley Street to Warner Street, Warner Street from Maplewood Terrace to Liberty Street, Liberty Street from Warner Street to Wood Avenue, Wood Avenue from Liberty Street to Lexington Avenue, and Lexington Avenue from Wood Avenue to the end should be replaced with 8-inch diameter water main.

6. To maximize the flow out of the Audubon Tank and improve available flow for the LHPS, a new 12-inch diameter water main is recommended to replace the existing 8-inch diameter water main from the Audubon Tank to River Road.

7. A new 12-inch diameter water main is recommended to replace the existing 6-inch diameter water main on Day Avenue that connects the 12-inch diameter water main on Bates Street to the 12-inch diameter water main on Bridge Street. This would eliminate the bottleneck created by the 6-inch diameter water main and improve transmission capacity.

8. The existing field lined cast iron water main on Conz Street and Mount Tom Road from Wright Avenue to the end of the cement lined cast iron water main should be replaced with a larger diameter main, and unlined portions of the existing main should be cleaned and cement lined, to improve fire flows in this area. This replacement

would only provide a portion of the estimated recommended fire flow in the area. To fully provide the recommended fire flow a 16-inch diameter water main is recommended from Wright to the end of the main. It is also important to note that increasing the diameter of the water main could cause potential water quality issues due to increased water age. The recommended fire flow was based on a desktop review of the building and conservative estimates. The ISO has not completed an evaluation of this area. Also, the recommended fire flow estimate does not factor in fire protection systems. A fire protection system could reduce the recommended fire flow in this area. Further evaluation by the ISO in this area could better define the recommended fire flow.

9. The estimated needed fire flow at the Rockridge Retirement Community cannot be met. We recommend replacing the existing water main on Cooke Avenue, Hatfield Street from Cooke Avenue to North King Street, and on North King Street from Hatfield Street to Coles Meadow Road with new 12-inch diameter water main. New 12-inch diameter water main is also recommended on Coles Meadow Road from North King Street to the Rockridge Retirement Community. This replacement would only provide a portion of the needed fire flow. To fully provide the recommended fire flow, a 16-inch diameter water main or booster pump station would be required. The pump station would have to be located on North King Street in order to have adequate suction pressure to provide the increased flow. The recommended fire flow was based on a desktop review of the building and a conservative estimate. The ISO has not completed an evaluation in this area. Also, the recommended fire flow estimate does not factor in fire protection systems. A fire protection system could reduce the recommended fire flow in this area. Further evaluation by the ISO in this area could better define the recommended fire flow.

Priority 3a Hydraulic Improvements

Priority 3 hydraulic improvements were identified as part of a system wide evaluation to improve fire flows. Priority 2 hydraulic improvements have been separated into two categories. Priority 3a improvements include water mains recommended for replacement due to the existing water mains being undersized. Priority 3b improvements include water mains that are recommended to be cleaned and lined to improve the hydraulic capacity of the mains.

10. A new 8-inch diameter water main on Pine Street from the 10-inch diameter water main on Maple Street to the intersection of Mann Terrace and Beacon Street is recommended to provide the recommended residential flow in this area.

11. A new 8-inch diameter water main is recommended on Henry Street from the intersection with Hockanum Road to the end of the main to provide the recommended residential flow in this area.

12. A new 8-inch diameter water main is recommended on North King Street from the end of the 10-inch diameter water main to the end of the main to improve residential fire flow in this area.

13. Table No. 4-2 identifies water mains which would need to be replaced to provide the recommended residential fire flow. Based on Chapter 9 of the MassDEP May 2010 Guidelines for Public Water Systems, water mains providing fire protection and serving fire hydrants shall be 8-inch diameter or larger. The hydraulic recommendations in these recommendations follow these guidelines. However, 6-inch diameter water main could be considered for water mains less than 500 feet.

Table No. 4-2
Priority 3a - New 6-inch diameter and 8-inch Water Main

Location	Length (feet)	Installation Date	Material
Kary Street	250	1915	CI
Water main off of Center Street	300	1919	CI
Dimock Street and Chesterfield Road	1,900	1919 - 1931	CI
Grove Avenue	500	1900	CI
Blackberry Lane	300	1940	CI
Bright Avenue	200	1932	CI
Kearney Field to hydrant	300	1940	CI
Hebert Avenue	950	1928	CI
Warner Row	350	1900	CI
Hockanum Road	1,100	1915/1960	CI/AC

Priority 3b Hydraulic Improvements

14. There are some water mains in the system that cannot meet the recommended residential fire flow due to water main condition and internal corrosion. Table No. 4-3 identifies the water mains that could potentially be cleaned and lined to improve the hydraulic capability of the water main. The structural integrity and condition of these water mains would have to be considered prior to cleaning and lining the water main.

Table No. 4-3
Priority 3b - Water Mains to be Cleaned and Lined

Location	Length (feet)	Installation Date	Material
Fair Street	2,700	1914	CI
Langworthy Road	700	1938	CI
North Farms Road	3,200	1926	CI
Grove Avenue	500	1930	CI
Grant Avenue	800	1917	CI
Island Road	2,350	1921	CI

SECTION 5 – Critical Component Assessment

5.1 General

A critical component assessment was performed for the water distribution system to evaluate the impact of potential water main failures on the water distribution system. The critical component assessment includes identification of critical areas served, critical water mains and the need for redundant mains.

5.2 Evaluation Criteria

Critical areas served are locations in the distribution system that require continual water supply for public health, welfare or financial reasons. Examples of critical service areas include hospitals, nursing homes, schools, and business districts. All water mains within 500 feet of a critical area are considered a critical component. Because water storage tanks and sources provide water and maintain pressure to critical service areas, tanks and primary sources are also considered critical areas. Generally, any water main within 500 feet of a water storage tank or primary source is considered a critical component.

Critical water mains are those mains that are the sole transmission main from a source or tank. In addition, main transmission lines that do not have a redundant main are considered critical. The evaluation included a visual review of the water mains leading into and out of the critical areas and the transmission grid.

The critical components and critical water mains were prioritized based on input from the City. Priority 1 water mains are the most critical, with Priority 3 water mains the least critical. Water mains that were not identified as critical are given a priority of 0.

5.3 Critical Components

Critical areas served, critical supply mains, and redundant mains were evaluated in the water system based on the criteria described above.

Critical Areas Served

A system-wide review of critical areas served such as health care facilities, and schools was completed. Other critical areas were identified during the workshop with the City staff. Fifty critical users were identified in the City.

Critical Water Mains

Critical water mains include raw water lines, primary transmission lines as well as mains connecting water storage tanks and sources to the system. In addition, primary distribution system water mains that do not have a redundant main are considered critical. Water mains that cross major highways, major rivers or active railroad tracks are also considered critical because of the difficulty in construction and permitting involved in replacing or rehabilitation of the water main.

Critical water mains were identified based on a review of the distribution system model, and by using the WaterGEMS criticality feature. The criticality feature runs multiple simulations that break each pipe in the model. The model calculates if the system can still be served with adequate flow and pressures after a pipe is taken out of service. This feature can identify areas served by multiple mains, but would no longer be able to serve customers if one of the mains were taken out of service. This feature identified the transmission lines as impacting more than five percent of the system and therefore critical.

Critical water mains given a priority rating of 1 include water mains within the 500 foot radius of Priority 1 critical components, raw water mains, transmission mains, water mains that are difficult to access, water mains that cross under a river, City water mains identified using the criticality feature, and water mains in the downtown area of the City. Water mains given a priority rating of 2 include water mains within the 500 foot radius of Priority 2 critical components, and water mains that intersect with railroads or water bodies. Water mains given a priority rating of 3 include water mains that are within the 500 foot radius of Priority 3 critical components, water mains that were identified by the City as deep mains that may be difficult to access or water mains that intersect with the Dam layer provided by the City.

SECTION 6 – Asset Management

6.1 General

The Northampton water distribution system includes approximately 154 miles of water mains varying in size and material. A number of factors including age, material, break history, soil conditions, static pressure, potential water hammer, and shallow water mains affect the decision to replace or rehabilitate a water main. Using an asset management approach, each water main in the system was assigned a grade based on these factors. The grades were then used to establish a prioritized schedule for water main replacement or rehabilitation.

6.2 Data Collection

Information regarding the water main diameters, material, and installation year was obtained from the City's existing ArcGIS water main database. Information, regarding break history and soil conditions, was obtained from system records and other information provided by the City.

6.3 Evaluation Criteria

To prioritize water main replacement or rehabilitation, a water main grading system has been established. The grading system uses the water main characteristics such as age, material, break history, potential water hammer, diameter, static pressure, soil characteristics, and water main depth to assign point values to each pipe in the system. Each category is assigned a rating between zero and 100 with zero being the most favorable and 100 being the worst case within the category. Each category is then given a weighted percentage, which represents priorities within the system. It is at the Owner's discretion to adjust the weight based on system performance and condition. Our recommendation is to assign a maximum of 30 percent to any one category. The rating is then multiplied by the weight. The weighted rating for each performance criteria will be utilized to determine the overall rating per pipe. Those pipes with the highest grade are most in need of replacement or rehabilitation.

To establish a rating system specific to the Northampton water system, a workshop was held with the system management and operators. During the discussion, it was determined that history of breaks, material, and age are of primary concern to the City. The grading system is shown in Table No. 6-1 and discussed in detail later in this section.

Age/Material

The water industry in the United States followed certain trends over the last century. The installation date of a water main correlates with a specific pipe material that was used during that time as shown on Table No. 6-2. For example, up until about the year 1958, unlined cast iron water mains were the predominant pipe material installed in water systems. Factory cement lined cast iron mains were manufactured from the late 1940's to about the mid 1970's, when pipe manufacturers switched primarily to factory cement lined ductile iron pipe.

Table No. 6-1
Asset Management Rating

Weight	Performance Criteria	Rating	Weighted Rating
25%	Break History		
	2 or more breaks per 1,000 ft	100	25
	Fewer than 2 breaks per 1,000 ft	80	20
	No history of breaks	0	0
18%	Material		
	Asbestos Cement	100	18
	Unlined Cast Iron	80	14.4
	Field or Factory Lined Cast Iron	70	12.6
	Ductile Iron	5	0.9
15%	Installation Date		
	1900 or earlier	85	12.8
	1901-1914	80	12
	1915-1919	75	11.3
	1920-1929	70	10.5
	1930-1939	100	15
	1940-1947	90	13.5
	1948-1975	100	15
	1976-1979	20	3
	1980-1999	5	0.8
	2000-2012	0	0
15%	Diameter		
	4-inch water main or less	100	15
	6-inch water main	90	13.5
	8-inch water main	50	7.5
	10-inch water main	15	2.3
	12-inch water main	10	1.5
	14-inch water main	7	1.1
	16-inch water main	5	0.8
	24-inch water main or larger	3	0.5
12%	Static Pressure		
	Greater than 125 psi	100	12
	100 to 125 psi	80	9.6
	80 to 100 psi	60	7.2
	Less than 80 psi	0	0
7%	Soils		
	Pipe on rock	100	10
	Potentially corrosive soil	90	9
	Cinders	80	8
	Stray current	70	7
	Landfills	60	6
	Gravel/Sand	0	0

Table No. 6-1 (continued)
Asset Management Rating

Weight	Performance Criteria	Rating	Weighted Rating
5%	<u>Potential Water Hammer</u>		
	Above 250 psi	100	5
	Below 250 psi	0	0
3%	<u>Shallow Water Mains</u>		
	Shallow Water Mains	100	3
	Not Shallow Water Mains	0	0

Table No. 6-2
Pipe Material by Installation Year (feet)

Installation Year	Asbestos Cement	Cast Iron	Copper	Ductile Iron	Field Lined or Factory Lined Cast Iron	PVC	Total
1900 or earlier		67,722					67,722
1901-1914		66,787			21,070		87,857
1915-1919		39,867					39,867
1920-1929		77,033	13				77.046
1930-1939	10,066	53,122					63,188
1940-1947	7,906	12,674					20,580
1948-1975	137,680	183			3,525		145,327
1976-1979	6,756			18,496			25,252
1980-1999			17	207,712		320	208,049
2000-2012			54	90,117			90,171
Total	162,408	317,388	84	316,325	24,595	320	825,059

Note: All units in Linear Feet

Cast iron water mains consist of two types; pit cast and sand spun. Pit cast mains were generally manufactured up to the year 1930 while sand spun mains were generally manufactured between 1930 and 1976. Pit cast mains with diameters between 4-inch diameter and 12-inch diameter do not have a uniform wall thickness but are generally thicker and stronger than spun cast mains. However, pit cast mains in this range of sizes may have "air inclusions" as a result of the manufacturing process. This reduces the overall strength of the main, which makes it more prone to leaks and breaks. Although sand spun mains have a uniform wall thickness, the overall wall thickness was thinner than the pit cast mains. The uniformity provided added strength, however, the thin wall thickness made it more susceptible to corrosion and breaks. Pit cast mains 16-inch diameter and larger have very thick pipe walls and are generally stronger than the thinner walled sand spun cast mains. While the transition to factory cement lined cast iron mains had begun in the late 1940's, prior to the year 1958, most cast iron water mains that were manufactured were still unlined. Unlined cast iron mains increased the potential for internal corrosion. By 1958 the majority of cast iron mains manufactured had a factory cement lining. Rubber gasket joints were also introduced around 1958. Prior to this date, joint material was jute (rope type material) packed in place with lead or a lead-sulfur compound, also known as "leadite" or "hydrotite." Leadite type joint materials expand at a different rate than iron due to temperature changes. This can result in longitudinal split main breaks at the pipe bell. Sulfur in the leadite can promote bacteriological corrosion that can lead to circumferential breaks of the spigot end of the pipe. In the case of the Northampton water system, the changeover from cast iron pipe was recorded as 1947. Therefore, with the exception of a small group of water mains, all the water mains installed prior to 1947 were unlined cast iron.

Factory lined cast iron was manufactured and installed up until about 1973. Overlapping this period, factory cement lined ductile iron main was manufactured from the 1950's, and continues to be manufactured today, although most New England water utilities did not begin to install ductile iron pipe until the late 1960's. In the Northampton water system, very little factory cement lined cast iron water mains were installed. The minimal amount that was installed was completed between 1948 and 1975. Factory cement lined cast iron and ductile iron pipe provided increased protection against internal corrosion. The City has also field cement lined some of the water mains. Approximately three percent of the system consists of water mains that are factory cement lined cast iron water mains (CLCI) or field cement lined cast iron (FLCI). Unlined cast iron (CI) water mains make up approximately 39 percent of the water system.

Between the 1930's and 1970's, the water industry also utilized asbestos cement (AC) pipe for their expanding water systems. An advantage of AC pipe is that it resists tuberculation build up, resulting in less system head loss. However, depending on the water quality, the structural integrity of AC mains can deteriorate over time, thereby becoming sensitive to pressure fluctuations and/or nearby construction activities. In addition, external influences such as soil type and high groundwater can corrode AC mains, thus reducing the strength further. Most water mains installed between from 1949 through 1974 are AC. A majority of the AC mains installed were added to the system by private developers. Approximately 20 percent of the system is composed of AC mains. The City has had problems with the existing

AC water mains, especially in areas with higher static pressures. The AC water mains have the highest rating score of all materials.

Polyvinyl Chloride (PVC) pipe was first used in the United States in the early 1960s. Due to its resistance to both chemical and electrochemical corrosion, PVC pipe is not damaged by aggressive water or corrosive soils. In addition, the smooth interior of PVC pipe is resistant to tuberculation. The 1994 "Evaluation of Polyvinyl Chloride (PVC) Pipe Performance" by the AWWA Research Foundation, found that utilities have experienced minimal long term problems with PVC pipe. Generally, problems with PVC occurred when the area surrounding the pipe was disturbed after installation of the pipe, indicating that PVC pipe is not as strong as ductile iron with disturbances such as being hit by excavation equipment after installation. It should be noted that PVC is a permeable material. Low molecular weight petroleum products and organic solvents can permeate PVC pipe if the contaminants are found in high concentrations in the soil surrounding the pipe. Less than one percent of the system is PVC.

A small percentage of the system is small diameter copper pipe. Typically this pipe would be considered a water service and not be included in this study. Some of these copper mains in the system may serve multiple homes and were therefore, considered as part of this study.

Approximately 38 percent of the system is cement lined ductile iron water main. This material was introduced in the United States in 1950's, however, was not widely used until the 1970's. According to the Ductile Iron Pipe Research Association (DIPRA), ductile iron pipe retains all of cast iron's qualities such as machinability and corrosion resistance, but also provides additional strength, toughness, and ductility.

In general, the oldest water mains in the system received a high rating of 100, while the newest received a rating of zero. A significant rating decrease occurs around 1975, which represents the timeframe when ductile iron water main was introduced into the Northampton system.

Diameter

The Northampton system consists of water mains ranging in diameter from less than 4-inch diameter to 36-inch diameter. Approximately 36 percent of the system is comprised of 8-inch diameter pipes and approximately 21 percent is 6-inch diameter pipes.

In general, as the diameter of a pipe increases, the strength increases. In most cases, failure occurs in the form of ring cracks. This is primarily the result of bending forces on the pipe. Pipes that are 6-inches in diameter are more likely to deflect or bend than a larger diameter main. Pipes that are 8-inches in diameter are less likely to break from bending forces than 6-inch diameter mains due to their increased diameter and resulting increased moment of inertia.

In addition, the pipe wall thickness typically increases as the pipe diameter increases. Pipes that are 16-inches in diameter and larger have significantly thicker walls than 12-inch diameter pipe and smaller diameter mains, such that in addition to superior bending resistance, they also are much more resistant to failure from pipe wall corrosion. The rating system for the diameter of the water mains follows the concept that 4-inch diameter water mains are not as strong as 24-inch diameter water mains. Therefore, a rating of 100 was given to 4-inch diameter and smaller water mains and a rating of zero was given to the 36-inch diameter water mains. Table No. 6-1 shows a significant drop in the rating score between a 6-inch diameter water main (90) and 8-inch diameter water main (50). This is due to greater bending strength. An 8-inch diameter water main has proven to have nearly twice the bending strength of a 6-inch diameter water main. In general, 8-inch diameter water mains are stronger and less likely to break than 6-inch diameter pipes.

Break History

Based on conversations with the City staff and the main failure records, the Northampton water system experiences an average of approximately seven breaks per year. In relation to the total miles of water main in the system, this equates to approximately five breaks per 100 miles per year. In comparison to the national average of 25 breaks per 100 miles per year, the Northampton water system is in good condition. However, each water main break costs the City time, materials, and labor. They also cause disruption to the public and water consumers. At some point, it becomes more efficient to replace the main then to continue repairing it. Based on the water main break records, there are several areas in the system that have experienced frequent breaks. Areas that have experienced two or more breaks per 1,000 feet have a score of 100 and areas that have experienced one break per 1,000 feet have a score of 80. Water mains that have had no breaks have a score of zero.

Soils

Water main degradation can occur both internally and externally. Factors that increase the rate of external corrosion include high groundwater, clay soils, contaminated soils, soils with low calcium carbonate, or soils with high acidity or sulfate. Wetlands areas have greater potential to cause external corrosion of water main than other soil conditions. Wetlands areas and areas with potentially corrosive soils are scattered throughout the distribution system. Areas where the water system and the potentially corrosive soils coincide were considered areas of potential exterior corrosion. There were also areas identified by the City with known poor soils and with known corrosion issues. These areas were given the highest rating (100) because they were known areas of potential corrosion. Areas identified as wetlands or potentially corrosive soils through soils maps were given a rating of 80. All other pipe was assigned a rating of zero.

Pressures

The static pressures of the system were evaluated under existing demands and conditions, assuming conservative PRV settings of 21 psi. Approximately 54 percent of the Northampton water system has a pressure above 80 psi. Plumbing code states that water heaters can be affected by pressures greater than 80 psi. Pressures above 100 psi can result in increased water use from fixtures and also increased leakage throughout the distribution system.

MassDEP Guidelines and Policies for Public Water Systems states that normal working pressures should be approximately 60 psi and not less than 35 psi. Areas with pressures exceeding 125 psi are required to have pressure reducing valves on the water mains. These areas are more susceptible to water main breaks. In addition, main failures in areas of higher pressures typically cause more disruption, and result in more costly repairs for damages.

All areas with pressures above 125 psi received the most points for static pressures, while pipes with pressures under 80 psi were given a pressure rating of zero.

Pressure Range	Percentage in System
Less than 80 psi	46
80 – 100 psi	18
100 – 125 psi	31
Greater than 125 psi	5

Potential Water Hammer

Water hammer occurs due to a sudden change in water velocity causing a high pressure wave to propagate throughout the system. This can happen if a line valve or hydrant valve is rapidly closed or a pump stops pumping due to a power outage. A sudden pump stop often results in the rapid closure of the pump's check valve. Estimated water hammer potential was developed using the hydraulic model to determine static pressure and pipeline velocity during the peak hour of 2032 maximum day. A potential water hammer of 250 psi approximates water traveling through a main at three feet per second with a static pressure of 100 psi. Locations with an estimated potential water hammer of 250 psi and greater were assigned 100 points. Water mains with an estimated potential water hammer of less than 250 psi were given a score zero.

Shallow Water Mains

Water mains located less than five feet below the ground surface are considered to be shallow water mains. These water mains are more susceptible to failure as a result of frost or increased stress due to live loads at the ground surface. The City has identified certain locations where water mains are considered to be shallow. Water mains considered to be shallow were assigned an asset rating of 100 and water mains that were not shallow were given a score of zero.

6.4 Asset Management Areas of Concern

Based on the asset management ratings, there are several areas of concern in the system. Water mains with a total rating less than 35 are considered to be in good to excellent condition. Areas with a total rating ranging from 35 through 55 are considered to be in fair to good condition, and areas with a total rating of 56 or greater are considered to be in poor to fair condition.

SECTION 7 – Recommendations and Conclusions

7.1 General

The following summarizes the findings of the study and presents a prioritized plan for recommended improvements and associated costs. The prioritization of improvements allows for constructing the necessary improvements over an extended period of time as funds allow. Costs are based on the January 2013 Engineering News Record (ENR) construction cost index for Boston, MA of 12025.13 and include costs associated with water services, hydrants, and permanent and temporary trench pavement and a 25 percent allowance for engineering and contingencies. Estimates do not include costs for land acquisition, easements, or legal fees.

The capital improvement projects considered by this study will provide a direct benefit to the overall level of service to the Northampton customers, reduce operation and maintenance costs by reducing the frequency of water main failures and the damage they cause, as well as improve fire protection to the homeowners and businesses.

The Water Research Association's (formerly the American Water Works Research Foundation) study on "Cost of Infrastructure Failure," which was completed in 2002, found that in addition to direct costs paid by water utility ratepayers for water main failures, there are also societal costs, which are paid by the public. Examples of the direct costs include outside contractor costs, engineering costs, police assistance, fire department assistance, electrical, telephone, and gas utility damage costs, landscaping restoration costs, and laboratory costs. Examples of societal costs included the cost of traffic impacts, business customer outage impacts, public health impacts (including loss of life), property damage not covered by direct costs, and the cost of reduced fire fighting capability during the failure event.

Replacement of one percent of a system each year (a 100 year replacement cycle) is a reasonable guideline based on industry experience and analysis. For the Northampton distribution system, this would equate to approximately 8,000 linear feet of water main replacement each year as a guideline. Regular rehabilitation of water mains reduces main failures, leakage, and water quality issues. Water main rehabilitation can also provide socio-economic benefits by reducing operational costs associated with chemical and energy usage. Also, rehabilitation or replacement of water mains that are inadequately sized to provide needed fire protection will improve public safety.

7.2 General Recommendations

To maintain a comprehensive database of the condition of the system, it is recommended that the City continue to update the GIS based water main database and include water main failure data as well. Currently, the City maintains a list of breaks, leaks, and replacements with the nearest street address and the properties of the failed main such as diameter, material, and type of break. In addition, the City should record the joint type, type of lining, and type of failure such as ring crack, lateral split, hole in the pipe, "punky" AC pipe failure, or joint leak.

If possible, the City should include the apparent cause of the failure such as frost load, traffic load, direct contractor damage, settlement, water hammer, external soil corrosion, or stray current. This data can be used to create a Water Main Failure Map for identifying areas of concern in the system on an ongoing basis. The map can be used to easily identify break locations and determine if streets or areas have a higher frequency of failures and to view any patterns in the location, type, pipe manufacturer, or other patterns in occurrences of failure. The water main failure database will aid the City in making water main rehabilitation and replacement decisions in the future. In addition, it is recommended that the City maintain data on pipe crushing results from water mains that have failed. At the time of a main failure, a one foot section of the water main should be cut from the pipe that will remain in place (adjacent to the repair). The sample will then need to be marked with date, installation date, diameter, location and any information regarding the type of main failure. It is then recommended that this coupon is analyzed further and data is recorded on results. It is also recommended that the City continue to update the database of new or rehabilitated water mains. The database should include water main diameter, material, lining, joint type, soil conditions, and date of installation.

It is recommended that prior to installation of all new ductile iron water mains, the City test the soils in the area of the new main to determine corrosion potential. If the soil is found to be potentially corrosive, the City should consider wrapping the main with polyethylene to protect against external corrosion. Wrapping is a relatively inexpensive practice that can extend the life of new ductile iron pipe. In addition, wrapping helps to protect the pipe from stray currents that may develop near the main.

The City of Northampton should continue to perform regularly scheduled maintenance programs, including hydrant flushing, inspection and maintenance at the wells and pump stations, pressure reducing valves, and meter testing/calibration. An annual unidirectional flushing program should be implemented. A unidirectional flushing program starts at a point of origin, usually a source or tank, and works outward flushing each portion of water main through clean water mains. The budgetary cost for design of a unidirectional flushing program is $50,000. The City should continue the existing replacement program during which hydrants and valves that do not function as intended are identified and replaced. These deficiencies are normally identified through routine operation and during the system-wide flushing program.

Audubon Road Tank

Based on the Underwater Solutions, Inc. inspection report dated June 2010 of the Audubon Road Tank, the tank is in fair to good condition only requiring a few repairs. The recommended repairs include the following: coating all exterior surfaces of the tank to protect and maintain the integrity of the steel and improve the aesthetic value of the tank, sealing the gap between the roof and the walls around the tank circumference, and permanently repair the two penetrations within the roof dome. The painting is not an immediate need, therefore an inspection is recommended within the next two years to reevaluate the needs of the tank. The estimated probable construction cost of completing the minor repairs outlined by Underwater

Solutions is approximately $10,000 to complete the listed items and an additional $2,000 for the recommended inspection.

Water Treatment Facility Clearwell

Based on the Underwater Solutions, Inc. inspection report dated June 2010 of the Water Treatment Facility Clearwell, the storage tank is generally sound and free of obvious leakage. A high pressure washing and coating for the exterior of the tank to seal cracks and protect the concrete was recommended. The need for exterior coating should be reevaluated once the tank is cleaned. It is recommended that the surface cracks found throughout the floor within the outer cell, the overhead, and the concrete support beams of the outer cell be inspected every two years to confirm they are not increasing in size or depth resulting in exposure of the underlying re-enforcement steel.

Radio Path Survey

A report was completed by Tata & Howard dated April 25, 2012 providing recommendations to upgrade communications and control between the Mountain Water Treatment Plant, the Mountain Low Lift Pumping Station (LLPS), Ryan Reservoir and the West Whately Reservoir. A radio path study was conducted and concluded that there are two alternatives to resolving the communication issues with LLPS. A new RTU is recommended inside the Water Filtration Plant and would include a cabinet, radio, PLC, UPS, telephone modem, and antenna. It was recommended that the RTU be linked to the existing SCADA MTU computer and the programming and LLPS screen be updated, as needed. An RTU will be located inside the LLPS and will include a cabinet, radio, PLC, Operator Interface Terminal, antenna, wiring to existing instrument panel and programming. The estimated probable construction cost for the WTP RTU and the LLPS RTU is approximately $65,000. A second alternative for communication between the Water Treatment Plant and the Low Lift Pump Station included the installation of a fiber optic cable installed between the two facilities. This would increase the cost by approximately $15,000.

The study also looked at linking the Ryan and West Whately sites to the Water Treatment Plant. This link would involve an internet connection established via a DSL phone line. A separate RTU would need to be located at the Ryan and West Whately meter pits and will include above grade cabinets, telephone modems, PLCs, Operator Interface Terminals, wiring to existing instruments, and programming. The estimated construction cost for the Ryan and West Whately RTUs is approximately $40,000. This cost does not include upgrade of the electric service, panel, and equipment at the meter pits. This work is being evaluated under a separate capital improvement project.

The total estimated cost for design and construction of the WTP, Mountain LLPS, Ryan Reservoir, and West Whately Reservoir communication upgrades as outlined above, with the fiber optic alternative, is $160,000 including engineering and contingencies.

Raw Water Main Evaluation

In December of 2012, Tata & Howard, Inc. completed a Raw Water Main Evaluation of the raw water supply system to comply with the conditions of the Water Management

Registration Statement and Permit. It was recommended that the City cut coupons from the 20-inch diameter main in order to determine the structural condition of the pipe, and if it was adequate, to then proceed to cleaning and lining the water main if needed. Based on the evaluations of the coupon, the water main appears to be in good condition. However, the C-factor tests indicate a C-value of 40. The City is currently evaluating for air entrapment and closed valves, and it was recommended that improvements on the water main be postponed until more information is available. Assuming the water main is found to be in poor condition, the estimated cost to clean and line the water main is approximately $1,250,000. Upon obtaining the results of the raw water transmission line evaluation above, the means of transporting water from the West Whately Reservoir to the Water Filtration Plant will be further evaluated for recommendations.

A low lift booster pump station was also recommended at the connection between the West Whately and Ryan Reservoir transmission mains, to allow the City to close the connection between the West Whately Reservoir and the drainage channel leading to the Mountain Reservoir. This would conserve energy by reducing the amount of head needed to lift water to the WTP. Additionally, it would reduce the potential risk of contamination from the roadway adjacent to Borowski's Brook. The approximate cost of the low lift booster pump station is $650,000.

It is recommended that the valve into Borowski's Brook from the West Whately Reservoir be maintained so it can be utilized in the event of a failure of the 24-inch diameter water main from the interconnection to the WTP. The cost effectiveness of the pump station will be evaluated once the transmission main assessment is complete.

To accomplish active flow management from the reservoirs, it was recommended that the City consider the following improvements:

First, rehabilitate the meter pits located on the West Whately and Ryan transmission mains. This would include installing new electrical panels and wiring, new service cables, replacing the existing venturi meters with magnetic flow meters, and installing flow control valves. The estimated cost to design and construct this improvement is approximately $120,000.

Second, we recommend the City install water stage recorders at the Ryan, West Whately, and Mountain Reservoirs to monitor surface water level and connect the recorders to the City's existing SCADA system. To best achieve the reservoir management described in the Analysis of Reservoir Safe Yield repot completed by Metcalf & Eddy dated October 1995, the City must have a means of accurately measuring the pool stage in the reservoirs and allocate flow from the reservoirs to the WTP in response. The estimated cost to design and construct this improvement is approximately $30,000 for the three reservoirs, including engineering and contingencies.

Third, we recommend the City install flow gauging on the discharge spillways of each reservoir and connect them to the existing SCADA system. The estimated cost to design and construct this improvement is approximately $65,000 for the three reservoirs. The cost provided includes installation of equipment for communication using cellular technology and power using solar technology for each site, as they are not currently outfitted with power or communication equipment.

Forth, we recommend that the City install a weather station at the WTP and connect it to the existing SCADA system to monitor and record precipitation events. The estimated cost to design and construct this improvement is approximately $15,000.

Mountain Reservoir Low Lift Pump Station Generator

Tata & Howard completed a letter report dated May 29, 2012, analyzing the alternatives and standby generator size for the Mountain Reservoir Low Lift Pump Station. The alternatives evaluated included a liquid propane generator and a diesel fueled generator.

Based upon the existing electrical load at the pump station, it was recommended that a 500 kilowatt (KW) diesel fueled generator, 800 amp automatic transfer switch, associated appurtenances, and prefabricated concrete building are installed at the Mountain Reservoir Low Lift Pump Station to power the station during power failures. The estimated project cost including engineering and contingencies is $455,000.

7.3 Prioritization of Water Distribution System Improvements

Based on the Three Circles Approach, a prioritized list of improvements was created. Improvements were separated into three phases. The Phase I and Phase II Improvements are prioritized based on hydraulic needs, location in the distribution system, and the condition of the water main. In general, the Phase I Improvements include water mains that fall into all three of the circles. Phase II Improvements generally include water mains that fall into two of the three circles. These improvements strengthen the transmission grid, eliminate potential asset management concerns, and provide redundancy.

Phase III Improvements generally fall into one circle. These improvements include the remaining hydraulic recommendations from Section 4 and areas with high asset management ratings. Phase III Improvements should be completed as funds become available and considered when reviewing road paving schedules.

It should be noted that the list of improvements is extensive due to the nature of this report. This results in a high associated cost if all of the suggested improvements were constructed. The intent of the prioritization, therefore, is to serve as a guide for implementation from the most needed to the least needed improvements based on the prioritization and weighted criteria established jointly by the City and Tata & Howard. These improvements would most logically be constructed over an extended period of time.

Table No. 7-1, at the end of this section, includes a prioritized list of Phase I Water Distribution System Improvements and the linear footage with the estimated cost of each Phase I Improvement. Table No. 7-2 includes a prioritized list of Phase II Water Distribution System Improvements with the linear footage and estimated cost of each Phase II

Improvement. It should be noted that paving schedules or Highway Department improvements were not evaluated as part of this study. The City may reprioritize the recommendations if paving or road work is scheduled on any of the roads recommended for water main improvements. It should also be noted that for any cleaning and lining project, it is recommended that a pipe section be tested or a coupon of the water main be evaluated prior to designing the project.

Phase I Improvements

Replace 20-Inch Diameter Water Transmission Main

1. The existing 20-inch diameter water transmission main is in areas that are difficult to access and runs cross country. Replacing or rehabilitating the water main will increase dependability and make the water main more accessible. The water main would be relocated to roadways where feasible. The project would be from the Mountain Street PRV to Mountain Street and follow the existing roadways in Williamsburg and Haydenville to the existing 20-inch diameter water main on River Road in Northampton. Prior to construction, it is recommended that alternative locations are assessed to include easement evaluation and use of old Railroad bed. Additionally, it is recommended that various non-destructive tests are completed to assess the condition of the water main including but not limited to integrity tests, C-value tests and coupon evaluations. Evaluation of the water main including engineering service, and nondestructive testing is approximately $113,000. The estimated probable construction cost of approximately 20,000 linear feet of 20-inch diameter water main is $6,388,000. This cost includes estimates for three bridge crossings and an additional cost associated with River Road having known areas of ledge between Audubon Road and Bridge Road in Haydenville.

Implement New Service Area Configuration

2. As discussed in Section 4 of the report, it is recommended that the Main Service System be divided into two service areas and the LHPS be supplied from the 36-inch diameter transmission main as summarized below. This improvement will help maintain adequate pressures in the western portion of the City, improve transmission, and help improve the inherent fire flow capacity in the system. It is also recommended that the City reevaluate the Initial Distribution System Evaluation (IDSE) with the HSA and the new tank. This may result in additional total coliform sampling sites.

   To adjust the pressures of the divided system, we recommend the installation of three pressure reducing valves. The existing PRV on the 36-inch diameter line will be relocated downstream of the new 8-inch diameter connection to the LHPS at Leonard Street. An additional PRV will be installed on the new connection into the LHPS at Leonard Street, which will be controlled by the water level in the Audubon Road Tank. The existing PRV on the 20-inch diameter line will be eliminated to operate the HSA off of the Mountain Street PRV. The third PRV shall be installed on the 24-inch diameter water main on Pine Street near Nonotuck Street to allow water to flow from the HSA to the LSA during an emergency. The portion of water main on Pine Street, crossing the river, should be relocated onto the road to provide secure accessibility and increase reliability. Well No. 1 and Well No. 2 pumps should also be evaluated to determine if they can produce the desired flow while maintaining the higher HGL in the proposed HSA. The estimated probable construction cost of each PRV, pit, valve, and associated SCADA is approximately $200,000. The estimated probable construction cost of relocating the 24-inch diameter water main on Pine Street from the bridge crossing to the roadway is approximately $103,000. The total estimated cost for this improvement is approximately $853,000.

New Water Storage Tank

3. A water storage tank is recommended in the proposed HSA to provide fire protection and redundant storage. We recommend construction of a 0.75 mg water storage tank off Ryan Road with a base elevation of approximately 400 feet, an overflow elevation of approximately 445 feet, and an operating range between 430 feet to 440 feet. A new 12-inch diameter water main from the tank to the system is also included in this recommendation. The Mountain Street PRV that feeds the HSA will be controlled from the water level in the proposed tank to allow the tank to fluctuate. Once the configuration is completed, it is recommended that the Turkey Hill and Reservoir Road tanks are dismantled. The estimated probable construction cost of a new 0.75 mg water storage tank including mixing, fence, valve and pit, SCADA, approximately 2,600 linear feet of new 12-inch diameter water main, engineering and contingencies would be approximately $1,875,000. An additional $300,000 is estimated for the probable construction cost of dismantling the Turkey Hill and Reservoir Road tanks. The total estimated probable cost of construction for this project is approximately $2,175,000.

Water Main Recommendations

4. To improve the available fire flow and transmission capabilities along North King Street, a new 8-inch diameter water main is recommended on North King Street from Laurel Park near the end of Ashbury Avenue to the end of the water main. This water main has asset management scores ranging between 73 and 75 and is considered critical. This improvement will help meet recommended fire flows on King Street. The estimated probable construction cost of approximately 2,900 linear feet of 8-inch diameter water main is $653,000.

5. Fair Street has been identified by the City as critical and is in potentially poor condition based on asset management ratings of between 56 and 59. It is recommended that the existing water main be replaced with a new 8-inch diameter water main. The estimated probable construction cost of approximately 2,600 linear feet of 8-inch diameter water main is $488,000.

6. As discussed in Section 5, it is recommended that the existing 8-inch diameter water main on Audubon Road from the Audubon Road Tank to River Road be replaced with 12-inch diameter water main. To maximize the flow out of the Audubon Tank and

improve available flow for the LHPS, a new 12-inch diameter water main is recommended to replace the existing 8-inch diameter water main from the Audubon Tank to River Road. This water main has an asset management rating of 41 and is considered to be in fair to good condition. As it is the only feed from the tank, it is important that the main is hydraulically sufficient to provide the necessary flows for the service area. The estimated probable construction cost of approximately 2,200 linear feet of 12-inch diameter water main is $544,000.

7. The existing 12-inch diameter field lined cast iron water main on Conz Street and Mount Tom Road from Wright Avenue to the end of the cement lined cast iron water main should be replaced with a new 16-inch diameter water main and the existing 12-inch diameter water main from the end of the cement lined cast iron water main to the end of the water main should be cleaned and lined to provide the estimated recommended fire flow located at the PCA facility at the end of Mount Tom Road. This water main is hydraulically deficient, critical, and has a high asset management score. The water main is considered hydraulically deficient based on recommended estimated fire flow needed at the end of the water main for PCA. Without any improvements, approximately 1,500 gpm is available. With a cement lined or new 12-inch diameter water main from the end of the field lined water main to the end of the water main, approximately 2,300 gpm is available. However, to provide the recommended fire flow of approximately 3,500 gpm to PCA the 16-inch diameter water main from Wright Avenue to the end of the field lined cast iron main is needed. It is also important to note that increasing the diameter of the water main could cause potential water quality issues due to increased water age. The recommended fire flow was based on a desktop review of the building and conservative estimates. The ISO has not completed an evaluation of this area. Also, the recommended fire flow estimate does not factor in fire protection systems. A fire protection system could reduce the recommended fire flow in this area. Further evaluation by the ISO in this area could better define the recommended fire flow. This water main is considered critical. The asset management ratings range between 35 and 46, indicating the water main is in good to fair condition for the northern portion of the water main, however the cast iron main in the southern portion of the road is in poor condition with asset management ratings ranging between 64 and 67. Do to the condition of this main, cleaning and lining may not be an option. The estimated probable construction cost of replacing approximately 3,100 linear feet of 12-inch diameter water main is $388,000. The estimated probable construction cost of constructing approximately 4,700 linear feet of 16-inch diameter water main is $1,169,000. The total estimated probable construction cost of the project is approximately $1,557,000.

Phase II Improvements

8. It is recommended that the 16-inch diameter water main on Federal Street and College Lane from Elm Street to Main Street is cleaned and lined to improve overall transmission capacity, system pressures, and flow. The water main runs along Federal Street, College Lane, Green Street, West Street, Paradise Road, Dryads Green, Wishing Avenue, and Washington Place. This water main is critical. The asset

management scores range between 23 and 44. C-value tests, conducted by the City, indicate a low C-factor in portions of the water main. Further investigation is recommended to prioritize which portions of the main shall be cleaned and lined. The estimated probable construction cost to clean and line approximately 10,900 linear feet of 16-inch diameter water main is approximately $1,635,000.

9. A new 8-inch diameter water main is recommended on Henry Street from Hockanum Road to the end of the water main. The existing water main has asset management ratings between 55 and 61, which is considered to be in poor condition. The water main is hydraulically deficient and the improvement is recommended to provide the recommended residential fire flows at the end of the water main. The estimated probable construction cost of replacing approximately 1,100 linear feet of 8-inch diameter water main is $207,000.

10. The water main on Damon Road from King Street to the end of the 12-inch diameter cast iron water main is critical and in poor condition. The water mains have high asset management ratings ranging between 42 and 70. The estimated probable construction cost of approximately 3,800 linear feet of 12-inch diameter water main is $894,000.

11. The water main on North Farms Road from North Maple Street to Country Way and North Maple Street from Bridge Road to North Farms Road is recommended to be replaced. Hydraulically, the existing water main could be cleaned and lined to provide adequate fire flow on Country Way. However, the existing main is in poor condition with high asset management ratings ranging from 53 to 66. The estimated probable construction cost of approximately 3,200 linear feet of 8-inch diameter water main is $601,000.

12. A recommended fire flow of approximately 3,500 gpm was estimated at the Rockridge Retirement Community located on Coles Meadow Road. Based on the condition of the 10-inch diameter water main on North King Street and elevations in the area, recommended fire flows cannot be met. We recommend replacing the 10-inch diameter water main on Cooke Avenue, Hatfield Street from Cooke Avenue to North King Street, and on North King Street from Hatfield Street to Coles Meadow Road with new 12-inch diameter water main. New 12-inch diameter water main is also recommended on Coles Meadow Road from North King Street to the Rockridge Retirement Community. This replacemen t would only provide a portion of the recommended fire flow. To fully provide provide the recommended 3,500 gpm, a 16-inch diameter water main or booster pump station would be required. The pump station would have to be located on North King Street in order to have adequate suction pressure to provide the increased flow. The recommended fire flow was based on a desktop review of the building and a conservative estimate. The ISO has not completed an evaluation in this area. Also, the recommended fire flow estimate does not factor in fire protection systems. A fire protection system could reduce the recommended fire flow in this area. Further evaluation by the ISO in this area could better define the recommended fire flow. The asset management scores of these mains

range from 37 to 58. The estimated probable construction cost of approximately 8,800 linear feet of 12-inch diameter water main is $1,915,000.

13. A new 12-inch diameter water main is recommended on Day Avenue to improve flow from the 12-inch diameter water main on Industrial Drive to the 12-inch diameter water main on Bridge Street. The water main is considered to be in fair to good condition. The estimated probable construction cost of approximately 900 linear feet of 12-inch diameter water main is $197,000.

14. Table No. 7-3 lists the improvements that have high asset management scores and are located in the City identified downtown area. All water mains in this area are considered critical. All water mains less than 8-inches in diameter are recommended to be replaced with 8-inch diameter water mains.

15. The existing 12-inch diameter water main on Easthampton Road from the intersection with Earle Street south to the end of the water main is recommended for replacement due to the high asset management score and criticality of the main. The estimated probable construction cost of approximately 7,600 linear feet of water main on a state road is $2,070,000.

16. It is recommended that the water main on Hockanum Road is replaced from the end of the 8-inch diameter water main to the end of the main. The water main has an asset management ratings ranging between 58 and 66. The improvement would provide the recommended residential fire flow at the end of the water main. The estimated probable construction cost of replacing approximately 1,400 linear feet of 12-inch diameter water main is $263,000.

17. The water mains on Williams Street from Hockanum Road to Montview Avenue and on Hockanum Road from William Street to Pleasant Street are recommended for replacement. The water mains range in asset management scores from 56 to 59. The estimated probable construction cost of replacing approximately 1,200 linear feet of 8-inch diameter water main is approximately $226,000.

18. It is recommended that the water mains on Winslow Avenue and on Hinckley Street are replaced with 8-inch diameter water main. The asset scores of these water mains range from 47 to 67, indicating the water mains are in poor condition. The water mains are also considered critical. The estimated probable construction cost of replacing approximately 3,500 linear feet of 8-inch diameter water main is approximately $662,000.

19. The water main on Valley Street from Hockanum Road to Montview Avenue is recommended for replacement with a new 8-inch diameter water main. The existing main has an asset management score of 57. The water main is considered to be in poor condition. It also falls within the critical area. The estimated probable

construction cost of replacing approximately 600 linear feet of 8-inch diameter water main is approximately $113,000.

20. The 4-inch and 6-inch diameter water mains on Hebert Avenue from South Street to the end of main cannot provide adequate residential fire flow and are considered to be in poor condition. The asset management ratings range from 50 to 59. It is recommended the water mains be replaced with a new 8-inch diameter water main. The estimated probable construction cost of replacing 950 linear feet of 8-inch diameter water main is approximately $179,000.

21. The water main on Florence Road from Rocky Hill Road to the 10-inch diameter water main is considered critical. The water main has a high asset score of 61. It is recommended that this water main is replaced with a new 12-inch diameter water main. The estimated probable construction cost of 1,900 linear feet of 12-inch diameter water main is $420,000.

22. The water main on Coles Meadow Road from Northampton Street to end of water main is recommended for a new12-inch diameter water main. The water main has a high asset management rating of 63. This water main is also considered critical. A new 12-inch diameter water main is recommended to replace the existing main. The estimated probable construction cost of 4,300 linear feet of 12-inch diameter water main is approximately $941,000.

23. Chestnut Street from Bridge Road to Locust Street has asset rating ranging between 59 and 65, and is considered critical. The estimated probable construction cost of replacing approximately 3,200 linear feet of 8-inch diameter water main $663,000.

24. Ferry Avenue from Island Road to the end of the water main is hydraulically deficient and does not meet the recommended fire flow at the end of the street. The asset score of the existing water main is 82. A new 8-inch diameter is recommended to replace the existing water main and provide recommended fire flow. The estimated probable construction cost of replacing approximately 400 linear feet of 8-inch diameter water main is $75,000.

25. The water main on Blackberry Lane is recommended for replacement. The water main is considered to be in fair and poor condition with asset management ratings ranging between 46 and 57. This improvement would also provide the recommended residential fire flow at the end of the 4-inch diameter water main. The estimated probable construction cost of replacing approximately 300 linear feet of 8-inch diameter water main is $57,000.

Phase III Improvements

26. Table No. 7-4 shows all new 8-inch diameter water mains that are recommended due to hydraulic deficiencies in providing recommended fire flows. It is recommended

that these water mains are replaced based on available funding or roadwork schedules.
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27. Table No. 7-5 shows all new 8-inch diameter water mains that are recommended due to high asset management ratings. The water mains are in poor condition and have asset management ratings greater than 56. It is recommended that these water mains are replaced based on available funding or roadwork schedules.

28. In general, it is recommended that all dead end water mains are looped to eliminate dead ends, provide redundancy and improve water quality throughout the system.

7.4 Summary of Estimated Improvements Costs

A summary of all estimated probable costs can be found in Table No. 7-6. The total estimated probable construction costs for the general recommendations included the unidirectional flushing program; rehabilitation of the Audubon Toad Tank; the radio path survey; evaluation of the raw water main (assuming cleaning and lining); the new raw water low lift booster pump station, and the mountain reservoir generator is approximately $2,767,000. The total estimated probable construction costs for the prioritized water distribution system improvements including Phase I – III is approximately $34,312,000.

Table No. 7-1
Prioritization of Improvements – Phase I

Item No.	Location	From	To	Water Main Diameter (in)	Length (LF)	Estimated Cost
1	New 20-inch water main			20	20,000	$ 6,388,000
2	PRVs with vaults (3)					$ 750,000
	Relocate Pine Street Bridge Crossing			24	200	$ 103,000
3	New 12" main from tank	Audubon Road Tank	River Road	12	2,600	$ 1,875,000
	New water storage tank					
	Dismantle water storage tanks (2)					$ 300,000
4	North King Street	Laurel Park	End of Main	8	2,900	$ 653,000
5	Fair Street	Bridge Street	Cross Path Road	8	1,950	$ 366,000
	Cross Path Road	Fair Street	DI Main	8	650	$ 122,000
6	Audubon Road	Audubon Tank	River Road	12	2,200	$ 544,000
7	Conz Street and Mount Tom Road	Wright Avenue	End of FLCI Main	16	4,676	$ 1,169,000
	Mount Tom Road	End of FLCI Main	End of Main	C&L	3,100	$ 388,000
					Total Estimated Phase I Cost:	**$12,658,000**

Table No. 7-2
Prioritization of Improvements – Phase IIa

Item No.	Location	From	To	Water Main Diameter (in)	Length (LF)	Estimated Cost
8	Federal Street and College Lane	Elm Street	Main Street	C&L	10,900	$ 1,635,000
9	Henry Street	Hockanum Road	End of Main	8	1,100	$ 207,000
10	Damon Road	King Street	12-inch Ductile Iron Water Main	12	3,800	$ 894,000
11	North Farms Road	North Maple Street	Country Way	8	1,800	$ 338,000
	North Maple Street	Bridge Road	North Farms Road	8	1,400	$ 263,000
12	Cooke Avenue	Bridge Road	Hatfield Street	12	1,200	$ 263,000
	Hatfield Street	Cooke Avenue	North King Street	12	1,700	$ 372,000
	North King Street	Hatfield Street	Ashbury Avenue	12	5,000	$ 1,094,000
	Coles Meadow Road	North King Street	Northampton Street	12	850	$ 186,000
13	Day Avenue			12	900	$ 197,000
15	Easthampton Road	Earl Street	End of Main	12	7,600	$ 2,070,000
16	Hockanum Road	End of 8-inch	End of Main	8	1,400	$ 263,000
17	Williams Street	Hockanum Road	Montview Ave	8	500	$ 94,000
	Hockanum Road	William Street	Pleasant Street	8	700	$ 132,000
18	Hinckley Street	South Main Street	Riverside Drive	8	2,726	$ 512,000
	Winslow Avenue			8	800	$ 150,000

Table No. 7-2 (continued)
Prioritization of Improvements – Phase IIa

Item No.	Location	From	To	Water Main Diameter (in)	Length (LF)	Estimated Cost
19	Valley Street	Hockanum Road	Montview Ave	8	600	$ 113,000
20	Hebert Avenue	South Street	End of Main	8	950	$ 179,000
21	Florence Road	Rocky Hill Road	10-inch Water Main	10	1,920	$ 420,000
22	Coles Meadow Road	Northampton Street	End of Main	12	4,300	$ 941,000
23	Chestnut Street	Bridge Road	Locust Street	8	3,200	$ 663,000
24	Meadow Road and Ferry Avenue	Island Road	End of Main	8	400	$ 75,000
25	Blackberry Lane			8	300	$ 57,000
					Total Estimated Phase IIa Cost:	**$11,118,000**

Table No. 7-3
Item No. 14 Downtown Improvements – Phase IIb

Location	From	To	Water Main Diameter (in)	Length (LF)	Estimated Cost
Pleasant Street	Wright Avenue	Main Street	12	2,500	$ 732,000
Wright Avenue	Conz Street	Pleasant Street	12	600	$ 132,000
Main Street	Market Street	4-inch near State Street	12	2,100	$ 627,000
Fruit Street	South Street	Conz Street	8	1,300	$ 244,000
Wilson Avenue	Conz Street	End of Main	8	500	$ 94,000
Maple Avenue	Conz Street	End of Main	8	500	$ 94,000
Fulton Avenue	Conz Street	Pleasant Street	8	400	$ 75,000
Wright Avenue	Pleasant Street	End of Main	8	500	$ 94,000
Cracker Barrel Alley	Main Street	Masonic Street	8	600	$ 113,000
Crafts Avenue	Main Street	8-inch water main	8	100	$ 19,000
4" off of Old S. Street	Old S. Street	Between Crafts Avenue	8	100	$ 19,000
Bedford Terrace	Elm Street	8-inch water main	8	500	$ 94,000
Armory Street	Pleasant Street	Pleasant Street	8	1,200	$ 225,000
Merrick Lane	King Street	End of Main	8	300	$ 57,000
New South Street	Main Street	6-inch Water Main	8	350	$ 76,000
Kingsley Street	Pleasant Street	8-inch water main	8	200	$ 38,000
Michelman Lane	Pleasant Street	8-inch water main	8	350	$ 66,000
Trumbull Road	Slate Street	Pleasant Street	8	700	$ 132,000
4" off of center street	Center Street	End of Main	8	300	$ 57,000
Park Avenue	Trumbull Road	End of Main	8	200	$ 38,000
				Total Estimated Phase IIb Cost:	**$3,026,000**

Table No. 7-4
Item No. 26 Hydraulic Improvements – Phase IIIa

Location	From	To	Water Main Diameter (in)	Length (LF)	Estimated Cost
Kearney Field			8	300	$ 57,000
4-inch on Grove Hill			8	500	$ 94,000
Kary Street			8	250	$ 47,000
Langworthy Road			C&L	700	$ 66,000
Grant Avenue			C&L	800	$ 88,000
Chesterfield Road			8	1,450	$ 272,000
Dinock Street			8	450	$ 85,000
Grove Avenue			C&L	500	$ 47,000
Pine Street	Maple Street	Beacon Street	8	700	$ 132,000
Reservoir Road	Water Street	End of Main	8	850	$ 160,000
Maplewood Terrace	Hinckley Street	Warner Street	8	1,400	$ 263,000
Warner Street	Maplewood Terrace	Liberty Street	8	150	$ 29,000
Liberty Street	Warner Street	Wood Avenue	8	800	$ 150,000
Wood Avenue	Liberty Street	Lexington Avenue	8	350	$ 66,000
Lexington Avenue	Wood Avenue	Riverside Drive	8	350	$ 66,000
Warner Row			8	350	$ 66,000
Island Road			C&L	2,350	$ 221,000
Total Estimated Phase IIIa Cost:					**$1,909,000**

Table No. 7-5
Item No. 27 Asset Management Improvements – Phase IIIb

Location	From	To	Water Main Diameter (in)	Length (LF)	Estimated Cost
6-inch off of Damon Road			8	200	$ 38,000
Holyoke Street			8	400	$ 75,000
Gleason Road			8	1,300	$ 244,000
Riverbank Road	Bridge Street	End of Main	8	2,100	$ 394,000
Fair Street Extension	North of Old Ferry Road	End of Main	8	2,200	$ 413,000
River Road	Audubon Road	End of 8-inch water main	8	800	$ 221,000
Garfield Avenue			8	900	$ 169,000
Burncoat Road and Leeno Terrace			8	1,400	$ 263,000
Stonewall Drive			8	550	$ 104,000
Laurel Lane			8	600	$ 113,000
Old Ferry Road	Bridge Street	Fair Street Ext	8	2,250	$ 486,000
Terrace Lane			8	700	$ 132,000
Crosby Street	Bates Street	Hubbard Avenue	8	1,200	$ 225,000
Elizabeth Street			8	1,100	$ 207,000
Woodmond Road	North Street	8-inch water main	8	1,100	$ 207,000
Winter Street			8	600	$ 113,000
Stoddard Street	Prospect Street	8-inch water main	8	1,000	$ 188,000
Glendale Avenue			8	400	$ 75,000

Table No. 7-5 (continued)
Item No. 27 Asset Management Improvements – Phase IIIb

Location	From	To	Water Main Diameter (in)	Length (LF)	Estimated Cost
Hillside Road, Crescent Street and Bancroft Road			8	4,000	$ 750,000
Arlington Avenue			8	300	$ 57,000
Sanders Avenue			8	300	$ 57,000
Riverside Drive and Landy Avenue			8	2,100	$ 394,000
Burts Pit Road and Pine Valley Road			8	1,700	$ 319,000
Fairview Avenue	South Street	Olive Street	8	1,150	$ 216,000
Graves Avenue	Market Street	8-inch water main	8	750	$ 141,000
				Total Estimated Phase IIIb Cost:	**$5,601,000**

Table No. 7-6
Summary of Improvement Costs

Location	Estimated Cost
General Recommendations	
Unidirectional Flushing Program	$ 50,000
Audubon Road Tank	$ 12,000
Radio Path	$ 120,000
Raw Water Main Evaluation (Assuming Cleaning and Lining)	$1,250,000
Low Lift Booster Pump Station	$ 650,000
Flow Monitoring and Spillway Flow Gauging	$ 230,000
Mountain Reservoir Low Lift Pump Station Generator	$ 455,000
Total Estimated Probable Construction Costs:	**$2,767,000**
Prioritized Water Distribution System Improvements	
Phase I	$12,658,000
Phase IIa	$11,118,000
Phase IIb	$ 3,026,000
Phase IIIa	$ 1,909,000
Phase IIIb	$ 5,601,000
Total Estimated Probable Construction Costs:	**$34,312,000**