Water Master Plan

WARE DEPARTMENT OF PUBLIC WORKS
Ware, MA

October 2016

WRIGHT-PIERCE
Engineering a Better Environment

Water
Wastewater
Infrastructure

WATER MASTER PLAN
WARE, MASSACHUSETTS

TABLE OF CONTENTS

TABLE OF CONTENTS (CONT.)

TABLE OF CONTENTS (CONT.)

SECTION	DESCRIPTION	PAGE

TABLE OF CONTENTS (CONT.)

LIST OF APPENDICES

LIST OF TABLES

LIST OF FIGURES

Section 1

SECTION 1

INTRODUCTION

1.1 GENERAL

The purpose of this master planning document is to evaluate the components of the Ware Department of Public Works' (WDPW) Public Water Supply System, make recommendations, and present the needed improvements in a well thought out and useful Capital Improvement Plan (CIP) that the WDPW will be able to effectively use moving forward.

1.2 REPORT ORGANIZATION

This Master Plan is organized as follows:

Section 1 - Introduction: This section introduces the purpose of the master plan and presents a brief summary of its organization.

Section 2 - Existing System and Facilities: The existing Ware water system and its facilities are presented and reviewed in the section.

Section 3 - Historical and Projected Water Use: This section presents a review of Ware's historical water use and the projections for its water use through the next 10-year planning period (2016 to 2025).

Section 4 - Water Supply Evaluation and Assessment: An overview of the existing water supply evaluation and an assessment of its adequacy though the planning period are presented.

Section 5 - Distribution System and Storage Evaluation and Assessment: This section presents the detailed evaluation performed of the Ware distribution system infrastructure that was also analyzed by a comprehensive hydraulic water model.

Section 6 - Regulatory Review: An overview of the regulations applicable to the Ware system is presented.

Section 7 - Asset Management: Due to the increasing complexities of the WDPW's infrastructure and processes, this section presents an initial assessment of the WDPW's current asset management processes and how it can be optimized or supplemented for increased efficiency.

Section 8 - Recommendations: This section summarizes the recommendations made within the other sections and presents the corresponding estimated costs for their implementation.

Section 9 - Recommended Capital Improvement Program: This section lays out a proposed Capital Improvement Program (CIP) to be used by the WDPW over the next several years as a guide for improvements that will allow it to meet its identified needs.

Section 2

SECTION 2

EXISTING SYSTEM SUPPLY AND FACILITIES

2.1 OVERVIEW OF WATER SYSTEM

The Ware Department of Public Works (WDPW) serves the Town of Ware, located in Hampshire County, Massachusetts. Ware is bordered by the Towns of New Salem, Petersham, and Hardwick to the north, the Town of Belchertown to the west, the Towns of New Braintree, West Brookfield, and Warren to the east, and the Town of Palmer to the south. State Route 9 is the main transportation corridor in town and bisects the Town in a north to south direction. The Town has a population of approximately 9,880 people. The water system has service elevations ranging from approximately 384 feet to 647 feet above mean sea level (msl).

The WDPW owns and operates the water system which serves residential, commercial and municipal users. The WDPW currently serves approximately 2,360 water customers consisting of 2,145 residential users, 158 commercial users, 1 agricultural user, 27 industrial users and 29 municipal users. Based on 2015 data, the average day demand is approximately 652,200 gallons per day (gpd) and the maximum day demand is approximately 1,061,000 gpd.

The Ware water system includes four active ground water sources (consisting of six wells) treated at two water treatment facilities, two water storage tanks, a booster pump station, and approximately 42 miles of water main. An overview of the water system is included as Figure 2-1. A brief summary of each water system component follows.

2.2 SUPPLY FACILITIES

The Ware Department of Public Works provides water to its customers from four active source locations consisting of six individual wells located throughout the Town of Ware. Available design parameters and physical properties of each well are included in Table 2-1.

Overview of Water System

Ware, MA

PROJ NO: 13471A | DATE: 10/18/2016

WRIGHT-PIERCE
Engineering a Better Environment

FIGURE: 2-1

TABLE 2-1
EXISTING WATER SUPPLY SOURCES

Source	Address	No. Wells	Type	Size	Depth	Year Constructed	Maximum Approved Withdrawal	Source Code
Wellfield (Wells No. 1, 2 & 3)	Barnes Street	3	Gravel Packed	8” x 18”	48’-51’	1978	660 gpm	1309000-01G
Well No. 4 (Giard Well)	Barnes Street and Greenwich Road	1	Gravel Packed	18” x 24”	51’	1965	500 gpm	1309000-02G
Dismal Swamp Well	Gilbertville Road	1	Gravel Packed	12” x 18”	68’	1998	405 gpm	1309000-03G
Cistern	Near Muddy Brook and Barnes St	1	Dug	42’ wide	23’	1886	330 gpm	1309000-04G

All of Town’s sources are located in the Chicopee River Basin as designated by the Massachusetts Department of Environmental Protection (MassDEP). Withdrawals from each of the sources and in aggregate are limited and are permitted under the Massachusetts Water Management Act (WMA). The permit specifies pumping limitations on two conditions; a maximum daily volume and an annual average volume. A copy of the WMA Registration Statement and Permit is included in Appendix A.

2.2.1 Wellfield

The Wellfield consists of three gravel packed wells (Wells No. 1, 2, and 3) that are located off of Barnes Street. Well No. 1 is approximately 100 feet west of Barnes Street, Well No. 2 is approximately 100 feet west of Well No. 1 and Well No. 3 is approximately 200 feet west of Barnes Street. All three wells are adjacent to the Muddy Brook with Well No. 3 being the closest to the brook. This area was first developed in 1893 and there was a 41-point tubular well field utilized at this location until 1978. In 1978, the three gravel packed wells were installed which were located on the perimeter of the previous wellfield.

Each of the three wells is an 8-inch by 18-inch gravel packed well that is located within a pit with a hatch that contains a water meter and a sump pump. These pits need sump pumps since they tend to fill with groundwater. In 2010, pitless adapters were installed in each well and all of the electrical controls and panels were moved above grade in accordance to a previous MassDEP sanitary survey.

Well No. 1 has a depth of 51 feet (54.2 feet from the top of the pit) with a well screen length of 7 feet. The well is furnished with a Goulds submerged single stage 6-inch, 200 gallons per minute (gpm) pump (model 6CHC) with 80 feet of Total Dynamic Head (TDH). The pump is driven by a Franklin 5 horsepower (HP), 3 phase motor that is rated for 3,460 revolutions per minute (RPM). The pump intake setting is 28 feet. The original pumping capacity of the well was 300 gpm with 14.8 feet of drawdown. Well No. 1 is currently permitted to withdraw up to 220 gpm.

Well No. 2 has a depth of 50 feet and the length of the well screen is 8 feet. The well is furnished with a Goulds submerged single stage 6-inch, 200 gpm pump (model 6CHC) with 80 feet of TDH. The pump has a Franklin 5 HP, 3 phase motor that is rated for 3,460 RPM. The pump intake setting is 28 feet. Well No. 2 is currently permitted to withdraw up to 220 gpm although the original pumping capacity of the well was 300 gpm.

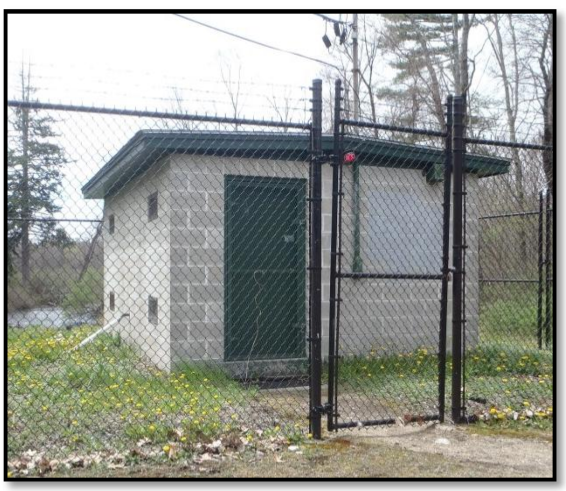
The total depth of Well No. 3 is 48 feet, but from the top of the pit, the depth is 47.5 feet. The well screen has a length of 8 feet and the well is furnished with a Goulds submerged two stage 6-inch, 100 gpm pump (model 100H05-2) with 100 feet of TDH. The pump motor is a Franklin 5 HP, 3 phase motor which is rated for 3,460 RPM. The pump intake setting is 33 feet. The original pumping capacity of the well was 300 gpm with 13.2 feet of drawdown. Well No. 3 is currently permitted to withdraw up to 220 gpm.

All three wells are permitted to have a combined pumping rate of 660 gpm (0.95 MGD). However, due to a decline in well capacity over the past ten years, two new replacement wells were constructed (Wells No. 2R and 3R). It was reported in 2014, that Wells No. 2 and 3 were pumping at approximately 150 gpm and 105 gpm, respectively. It is understood that the intent is to fully replace the current existing Wells No. 2 and 3 with these replacement wells. The replacement wells were recently approved by MassDEP in 2016 (BRP WS 19 – New Source Approval). Activation and pump installation (e.g. low or high head) is currently contingent upon a separate Treatability Study.

Wells No. 2R and 3R are 12-inch by 18-inch gravel and silica media packed wells that have a depth of 50 feet and 42 feet, respectively. The well screen for each well is 6 feet of 12-inch stainless steel, 0.140-inch slot. Recent water quality sampling in 2015 determined that the nitrate, nitrite, iron, and manganese were all under each respective SMCL.

The Wellfield source and the Well No. 4 source (described later) water are pumped directly into the Cistern with low lift pumps. High lift pumps in the Cistern pumps this water through the Pump House for treatment (as discussed later). In 2009, a bypass line was installed so the water can be pumped directly to the Pump House. This bypass line could be used once higher head pumps are installed at the wells due to the increased pressures. The combined withdrawal rate of Wells No. 1, 2, and 3 is controlled with Hand/Off/Auto switches located within the Pump House.

2.2.2 Well No. 4



Well No. 4, which is also called the "Giard Well", was constructed in 1965. Well No. 4 is an 18-inch by 24-inch gravel packed well that has a depth of approximately 51 feet and a well screen of 10 feet. The well is equipped with a vertical turbine pump that is powered by a 10 HP motor. The well is located approximately 800 feet southwest of Snow Pond off Pleasant Street, and the well is enclosed within a block well house that is protected with a black fence with barbed wire for security.

Well No. 4 is currently permitted for a maximum authorized withdrawal of 500 gpm. As noted previously, this well source is pumped to the Cistern with the Wellfield source.

The well does not have any emergency standby power.

A residential house located at 116 Pleasant Street is currently located within Well No. 4's Zone I. Therefore, it would be advantageous for the WDPW to acquire this land when/if it should become available for purchase. The parcel is approximately 1,515 square feet. Acquisition of the property would allow the WDPW to have more ownership within their Zone I.

2.2.3 Dismal Swamp Well

The Dismal Swamp Well, also referred to as Well No. 5, was constructed in 1998 and is located

approximately 1,000 feet northwest of Gilbertville Road (Route 32). The well is a 12-inch by 18-inch gravel packed well that has a depth of approximately 68 feet and a well screen of 13 feet. The Dismal Swamp Well is protected by a concrete structure that is built around the casing since the well is located within the 100 year flood plain (as shown in picture above). The well has a pitless adapter. The well vent is located approximately 100 feet west of the well head. The Dismal Swamp well is currently permitted for a maximum authorized withdrawal of 405 gpm. The water is pumped through an 8-inch water main to the Control Building and then into the distribution system. The Control Building is also referred to as the Gilbertville Road Pump Station (shown in the picture to the right) which is where the pump controls and chemical treatment are located.

The Dismal Swamp Well Control Building is equipped for corrosion control treatment and disinfection. The WDPW uses a 45%

solution of potassium hydroxide (KOH) to adjust the pH and a 12.5% solution of sodium hypochlorite (NaOCl) to disinfect the raw water. The KOH feed system includes one 1,550 gallon storage tank and flow paced metering pumps. The WDPW has recently upgraded the KOH feed system with a day tank setup in conformance with MassDEP Guidelines. The NaOCl feed system currently includes one 1,000-gallon storage tank, one 100-gallon day tank, flow paced metering pumps, and high and low level chlorine residual and pH alarm system. The NaOCl feed system is currently not being used due to manganese issues.

The well is equipped with a Goulds 6-inch, 300 gpm pump (model 7CLC-3) rated for 53.5 feet of TDH. The pump is driven by a Grundofs 30 HP motor.

The Dismal Swamp Well Control Building does not have any emergency generator provisions due to flood zone concerns.

2.2.4 Cistern

The Cistern was constructed in 1886 and was the Town's original water supply. The Cistern is a 42 foot wide by 23 foot deep dug well that is located near the Muddy Brook on Barnes Street which is adjacent to the Pump House (described later in this section). The well is located within a concrete building with brick walls that is enclosed within a fence for security. Additional security with the use of a security camera is intended to be installed in the future. The floor of the well is a natural bottom that consists of stone and sand. The Cistern is currently permitted for a maximum authorized withdrawal of 330 gpm (0.475 MGD). The Cistern holds approximately 230,000 gallons when full.

The Cistern is equipped with two 75 HP, 1,780 RPM five stage high lift pumps. The two pumps are located on a steel walkway over the well. A 10-inch pump column extends down from each pump to approximately 36-inches from the bottom of the well. The maximum capacity of each pump is 1,100 gpm. A surge relief valve is installed downstream of the pumps.

As presented previously, the Wellfield (Wells No. 1, 2, and 3) and Well No. 4 pump directly into this Cistern. The Cistern has float switches that activate the two sources when needed. Water from the Cistern is pumped through the Pump House for treatment and into the distribution system.

The Cistern has a temporary chemical feed system set up for emergency chlorination which is no longer used. In 2007, a permanent chemical feed system for disinfection was installed in the Pump House.

The Cistern was last inspected in July of 2014 during its cleaning by Underwater Solutions Inc. It was reported to be in good condition.

2.2.5 Pump House

The Pump House was constructed in 1886 along with the Cistern. The Pump House was most recently upgraded in 2007 and part of the upgrade was the installation of a permanent disinfection feed system. At the Pump House, the Wellfield, Well No. 4, and the Cistern sources

are chemically treated with potassium hydroxide to raise the pH and sodium hypochlorite for disinfection. Although not yet required, it is understood that the disinfection would not be Ground Water Rule compliant (for 4-log inactivation of viruses) due to the close proximity of the nearest downstream taps.

The Pump House is equipped with telemetry for all sources, chart recorders for the storage tanks, chemical monitoring equipment, chemical analyzers, alarms, chemical storage tanks, chemical feed pumps, and a generator. The Kohler 180 kW diesel generator provides standby power for the Pump House, the Wellfield, and one of the high lift pumps in the Cistern. Two fuel storage

tanks for the generator are stored in a small room upstairs within a brick containment wall.

The Pump House does not have a SCADA system, but there are Hand/Off/Auto (HOA) switches that control the well sources (Wellfield, Well No. 4, and Cistern) which are turned on or off based upon the level of the water storage tanks. This system can operate from either the Anderson Road tank or the Church Street tank level. A Verizon phone line is used as part of the system which is also located at the Pump House. The Dismal Swamp Well can also be operated at the Pump House with an old PLC from 1998 that is located within the Pump House. This PLC has old telemetry lines and is recommended to be replaced.

The potassium hydroxide feed system includes one 1,500-gallon Chem-Tainer bulk storage tank, a 500-gallon PolyProcessing day tank on a concrete pad, and two 0.5 HP Milton Roy metering pumps. The sodium hypochlorite feed system includes two 50-gallon day tanks on a concrete pad and two LMI metering pumps.

2.3 DISTRIBUTION SYSTEM

2.3.1 Transmission and Distribution Mains

The distribution system consists of approximately 47 miles of water main predominantly ranging in diameter from 6-inch to 12-inch. Approximately 136,300 feet of water main is composed of iron (ductile and cast) pipe and approximately 110,500 feet is composed of asbestos-cement (AC) pipe. A summary of the distribution system piping sorted by material type and pipe diameter is presented in Figures 2-2 and 2-3, respectively.

tanks for the generator are stored in a small room upstairs within a brick containment wall.

The Pump House does not have a SCADA system, but there are Hand/Off/Auto (HOA) switches that control the well sources (Wellfield, Well No. 4, and Cistern) which are turned on or off based upon the level of the water storage tanks. This system can operate from either the Anderson Road tank or the Church Street tank level. A Verizon phone line is used as part of the system which is also located at the Pump House. The Dismal Swamp Well can also be operated at the Pump House with an old PLC from 1998 that is located within the Pump House. This PLC has old telemetry lines and is recommended to be replaced.

The potassium hydroxide feed system includes one 1,500-gallon Chem-Tainer bulk storage tank, a 500-gallon PolyProcessing day tank on a concrete pad, and two 0.5 HP Milton Roy metering pumps. The sodium hypochlorite feed system includes two 50-gallon day tanks on a concrete pad and two LMI metering pumps.

2.3 DISTRIBUTION SYSTEM

2.3.1 Transmission and Distribution Mains

The distribution system consists of approximately 47 miles of water main predominantly ranging in diameter from 6-inch to 12-inch. Approximately 136,300 feet of water main is composed of iron (ductile and cast) pipe and approximately 110,500 feet is composed of asbestos-cement (AC) pipe. A summary of the distribution system piping sorted by material type and pipe diameter is presented in Figures 2-2 and 2-3, respectively.

tanks for the generator are stored in a small room upstairs within a brick containment wall.

The Pump House does not have a SCADA system, but there are Hand/Off/Auto (HOA) switches that control the well sources (Wellfield, Well No. 4, and Cistern) which are turned on or off based upon the level of the water storage tanks. This system can operate from either the Anderson Road tank or the Church Street tank level. A Verizon phone line is used as part of the system which is also located at the Pump House. The Dismal Swamp Well can also be operated at the Pump House with an old PLC from 1998 that is located within the Pump House. This PLC has old telemetry lines and is recommended to be replaced.

The potassium hydroxide feed system includes one 1,500-gallon Chem-Tainer bulk storage tank, a 500-gallon PolyProcessing day tank on a concrete pad, and two 0.5 HP Milton Roy metering pumps. The sodium hypochlorite feed system includes two 50-gallon day tanks on a concrete pad and two LMI metering pumps.

2.3 DISTRIBUTION SYSTEM

2.3.1 Transmission and Distribution Mains

The distribution system consists of approximately 47 miles of water main predominantly ranging in diameter from 6-inch to 12-inch. Approximately 136,300 feet of water main is composed of iron (ductile and cast) pipe and approximately 110,500 feet is composed of asbestos-cement (AC) pipe. A summary of the distribution system piping sorted by material type and pipe diameter is presented in Figures 2-2 and 2-3, respectively.

tanks for the generator are stored in a small room upstairs within a brick containment wall.

The Pump House does not have a SCADA system, but there are Hand/Off/Auto (HOA) switches that control the well sources (Wellfield, Well No. 4, and Cistern) which are turned on or off based upon the level of the water storage tanks. This system can operate from either the Anderson Road tank or the Church Street tank level. A Verizon phone line is used as part of the system which is also located at the Pump House. The Dismal Swamp Well can also be operated at the Pump House with an old PLC from 1998 that is located within the Pump House. This PLC has old telemetry lines and is recommended to be replaced.

The potassium hydroxide feed system includes one 1,500-gallon Chem-Tainer bulk storage tank, a 500-gallon PolyProcessing day tank on a concrete pad, and two 0.5 HP Milton Roy metering pumps. The sodium hypochlorite feed system includes two 50-gallon day tanks on a concrete pad and two LMI metering pumps.

2.3 DISTRIBUTION SYSTEM

2.3.1 Transmission and Distribution Mains

The distribution system consists of approximately 47 miles of water main predominantly ranging in diameter from 6-inch to 12-inch. Approximately 136,300 feet of water main is composed of iron (ductile and cast) pipe and approximately 110,500 feet is composed of asbestos-cement (AC) pipe. A summary of the distribution system piping sorted by material type and pipe diameter is presented in Figures 2-2 and 2-3, respectively.

FIGURE 2-2
PIPE MATERIALS IN WATER DISTRIBUTION SYSTEM

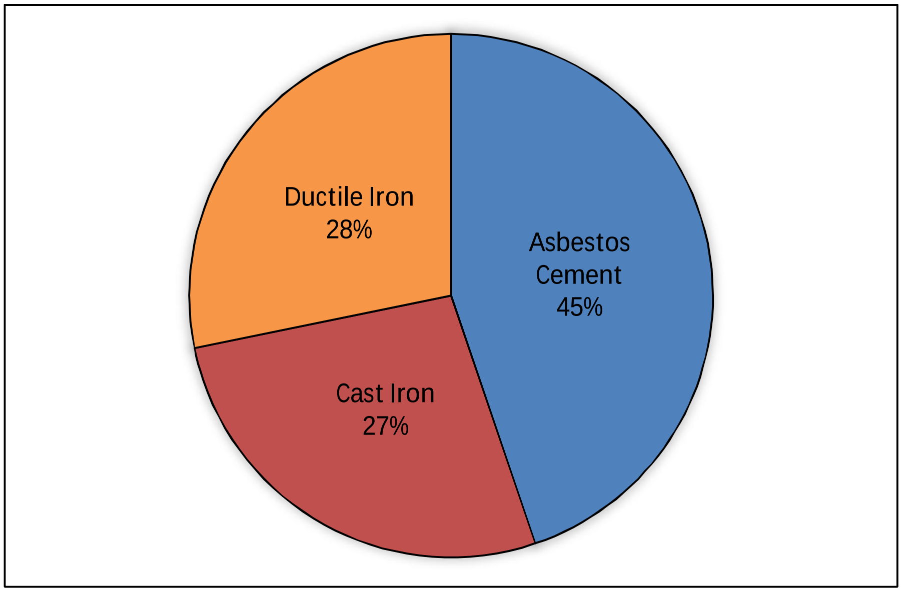
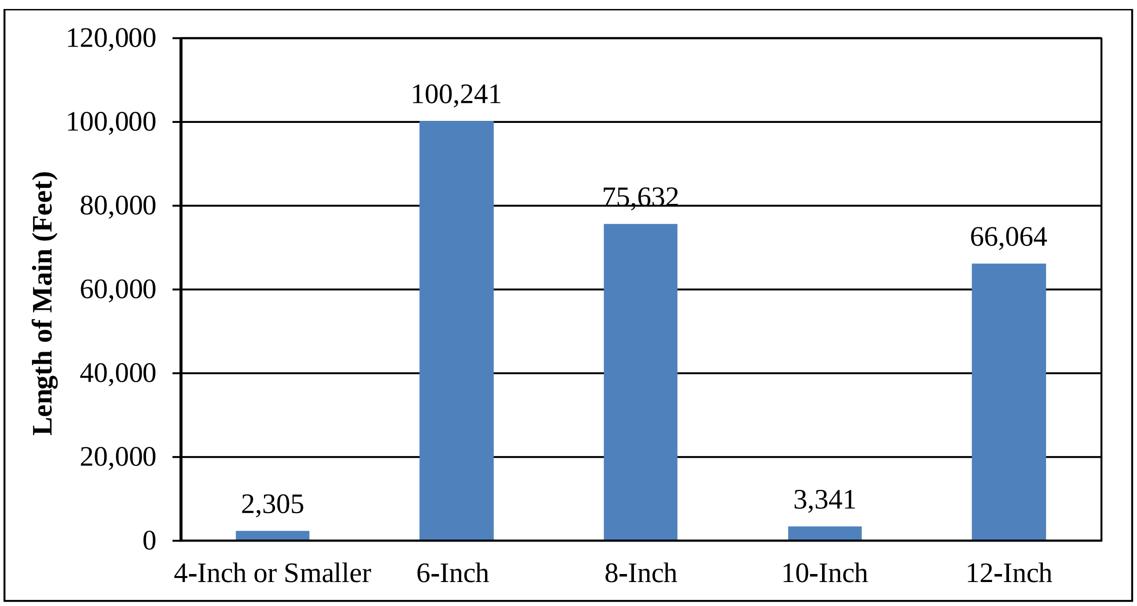


FIGURE 2-3
WATER DISTRIBUTION SYSTEM PIPE SIZE



Three primary piping materials predominate in the Ware distribution system:

- *Asbestos Cement (AC)* – Asbestos cement piping was readily available and typically installed in the 1960s and 1970s. As of 2016, approximately 45% of the distribution mains are AC. It is unknown at this time how much, if any, of the AC mains are vinyl-lined (TCE).

- *Ductile Iron Piping (DI)* – Cement lined ductile iron pipe is typically the piping of choice in today's distribution systems. It offers superior strength characteristics, is readily available, manufactured in a variety of thickness, and can be supplied with a variety of jointing systems. Approximately 28% of the distribution system is ductile iron pipe.

- *Cast Iron Piping (CI)* **–** Cast iron piping was the predecessor to DI and was typically installed from the late 1800s to the late 1960s. It is thought that the oldest CI pipes, dating to the late 1800s, have an average life expectancy of 100 to 120 years. Because of changing materials and manufacturing techniques, pipes laid in the 1920s have an average life expectancy of 100 years, while those laid in the post-World War II era are expected to last only about 75 years (source MIIC Infrastructure Report: Massachusetts Drinking Water, May 2007). Based on the Ware system records, approximately 27% of the system is currently unlined cast iron. Unlined cast iron water mains are typically the primary source of diminished hydraulic capacity in most distribution systems due to their internal tuberculation. Additionally, they can be the cause of discolored water complaints and microbiological problems.

Appendix B includes overviews of the Ware water distribution system that are color coded by water main material type and pipe diameter.

2.4 INTERCONNECTIONS

2.4.1 Interconnections with Adjacent Communities

The Ware water system does not have any interconnections with adjacent communities.

2.5 DISTRIBUTION STORAGE FACILITIES

Distribution storage facilities for the Ware Department of Public Works are comprised of two ground level storage tanks that are on the same hydraulic grade line as summarized in Table 2-2.

TABLE 2-2
EXISTING DISTRIBUTION STORAGE FACILITIES

Name	Overflow Elev. (ft)	Height (ft)	Diameter (ft)	Capacity (MG)	Type
Anderson Road Storage Tank	659	65	52	1.0	Steel
Church Street Storage Tank	659	24	100	1.5	Steel

2.5.1 Anderson Road Storage Tank

The Anderson Road Storage Tank is a welded steel standpipe constructed in 1978 that is located off Route 9 at 122 Anderson Road. The 1.0 million gallon (MG) tank has an overflow elevation of 659 feet and is 52 feet in diameter and 65 feet high. The facility has an altitude valve. The Anderson Road Storage Tank is at the western edge of the distribution system.

The tank has two 24-inch inside diameter manways; one on the northern side and one on the southern side of the tank. They are located approximately 17 inches above the tank base. The tank also has a welded steel ladder from the roof dome to 16 feet above the ground. The ladder has a fall prevention device and a welded safety cage. The tank vent has a diameter of 10 inches and a height of 31 inches which is located at the center of the dome roof. A galvanized steel screen and cap are installed over this vent. There are also two 24-inch diameter hatches on the roof.

The tank was last inspected in December of 2015 during its cleaning by Underwater Solutions

Inc. It was found to be in generally good condition. A high-pressure wash for the exterior wall, roof dome, and associated exterior components and also a re-coat to these surfaces within the next five years were recommended.

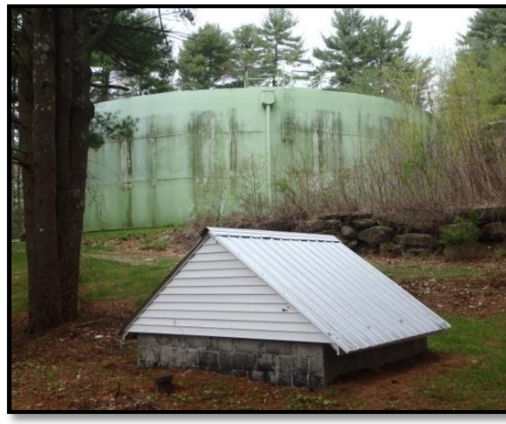
2.5.2 Church Street Storage Tank

The Church Street Storage Tank is a welded steel tank constructed in 1978 that is located in the northern part of town at 123 Church Street. The 1.5 MG tank has an overflow elevation of 659 feet and is 100 feet in diameter and 24 feet high. The facility has an altitude valve.

The tank has a 24-inch inside diameter manway on the north-eastern side and on the south-western side of the tank. They are located approximately 17 inches above the tank base. The tank also has a welded steel ladder on the north-eastern side of the tank from the roof dome to 16 feet above the ground. The ladder has a fall prevention device. The tank vent has a diameter of 10 inches and a height of 24 inches which is located at the center of the dome roof. A steel screen and cap are installed over this vent. There are also two 24-inch diameter hatches on the roof.

The tank was last inspected in December of 2015 during its cleaning by Underwater Solutions Inc. It was found to be in generally good condition. A high-pressure wash for the exterior wall, roof dome, and associated exterior components and to re-coat these surfaces within the next five years were recommended.

2.5.3 Booster Pump Station



Adjacent to the Church Street Storage Tank is a booster pump station located in a small below grade structure. Because the tank is at a lower hydraulic grade line than four houses nearby along Gilbertville Road, this booster pump station is utilized to pump water at an increased pressure from the tank to these houses. The building is equipped with three 100 gallon Well-X-Trol hydropneumatic storage tanks that is manufactured by Amtrol, and a 4 HP pump.

The booster pump station does not have any emergency generator provisions.

2.6 SCADA AND CONTROL SYSTEMS

The WDPW currently does not have a modern Supervisory Control and Data Acquisition (SCADA) system. All of the sources are run by Hand/Off/Auto (HOA) switches and are controlled by tank level telemetry. The sources are utilized based upon the level of the water storage tanks and the system can operate on either the level from the Church Street tank or the Anderson Road tank. When the HOA switch is turned on Auto, the sources will turn on by a tank level signal which is through an old pulse telemetry phone line. HOA switches for the Wellfield, Well No. 4 and Cistern are located at the Pump House. The Pump House also has a PLC to operate the Dismal Swamp Well.

The water system in Ware should be upgraded with a modern SCADA system for increased reliability, a higher level of service for consumers, increased efficiency, and optimized labor.

Section 3

Wright-Pierce
Engineering a Better Environment™

SECTION 3

HISTORICAL AND PROJECTED WATER USE

3.1 GENERAL

The purpose of this section is to present an analysis of water use in the Ware water system from 2011 through 2015. The discussion on water use is followed by a presentation of projections of future water demands. Data used in the analysis between 2011 through 2015 was obtained from Massachusetts Department of Environmental Protection (MassDEP) annual statistical reports and meter records provided by the Ware Department of Public Works (WDPW). Additional population data was obtained from the United States (US) Census, UMass Donahue Institute (UMDI), Massachusetts Department of Transportation (MassDOT) Planning, Pioneer Valley Planning Commission (PVPC), and the Town of Ware.

In order to plan for future needs of water system facilities and infrastructure, it is very important to understand future growth within the service area. An important aspect of the planning process is to plan for upgrades and/or additional water works facilities in advance of the impending increases in demand. The findings and recommendations presented herein will serve as the frame-work for the water supply and distribution system analyses. Updated projections of water-use needs through year 2025 were developed and are discussed in this section.

Numerous factors can impact water-use projections, including economic conditions, development (business, industrial, commercial and residential), and conservation efforts. As Ware is a mostly residential Town, residential water use is likely to be the most significant factor that will affect the water demand estimates. It is difficult at best to predict the impacts that the economy can have on a community. However, it is fair to assume that economic development generally leads to increases in population.

3.2 POPULATION DEMOGRAPHICS AND HISTORIC TRENDS

The population data discussed herein will serve as the basis for projecting water-use needs within the Town of Ware.

To better understand the population demographics in the Town of Ware, the following primary sources of information were collected and analyzed:

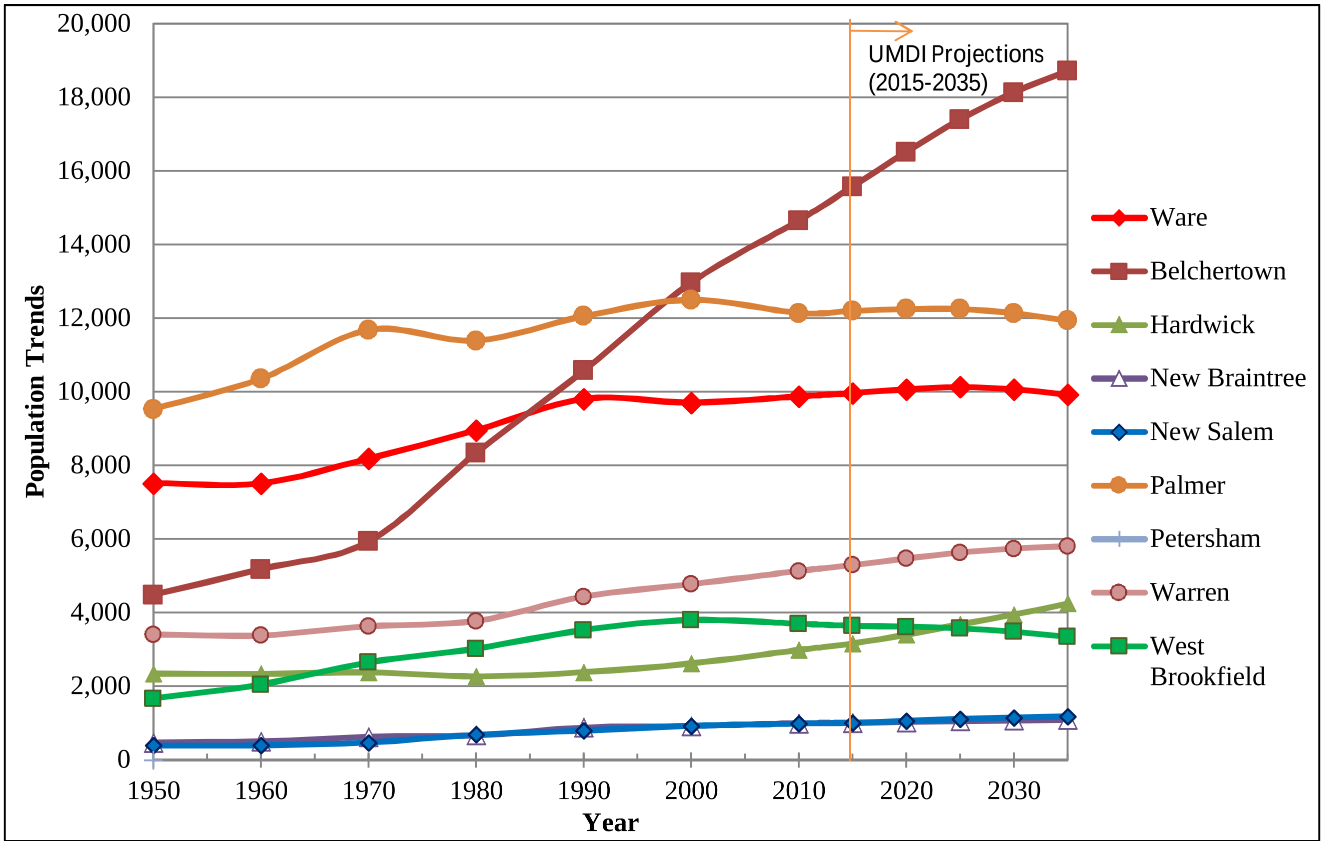
- US Bureau of Census Data
- UMDI
- PVPC
- MassDOT

The Census data includes population trends for each community in Massachusetts extending back to 1950. The population trends in Ware and its neighboring communities are presented in Table 3-1 and graphically in Figure 3-1.

TABLE 3-1
POPULATION TRENDS FOR WARE AND NEIGHBORING COMMUNITIES
WARE, MASSACHUSETTS

Town	1950	1960	1970	1980	1990	2000	2010
Ware	7,517	7,517	8,187	8,953	9,808	9,707	9,872
Belchertown	4,487	5,186	5,936	8,339	10,579	12,968	14,649
Hardwick	2,348	2,340	2,379	2,272	2,385	2,622	2,990
New Braintree	478	509	631	671	881	927	999
New Salem	392	397	474	688	802	929	990
Palmer	9,533	10,358	11,680	11,389	12,054	12,497	12,140
Petersham	814	890	1,014	1,024	1,131	1,180	1,234
Warren	3,406	3,383	3,633	3,777	4,437	4,776	5,135
West Brookfield	1,674	2,053	2,653	3,026	3,532	3,804	3,701

FIGURE 3-1
POPULATION TRENDS AND PROJECTIONS FOR WARE AND NEIGHBORING COMMUNITIES
WARE, MASSACHUSETTS



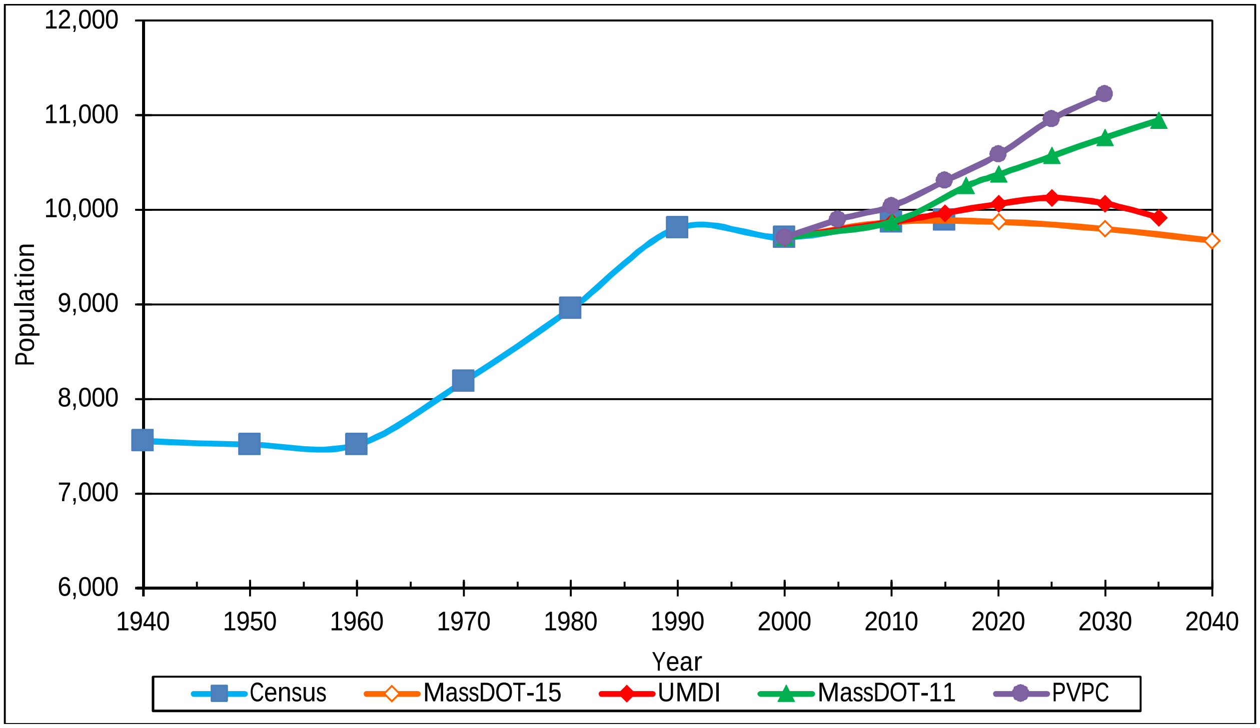
In general, the smaller communities in the suburbs experienced growth during the post-World War II period from 1950's through the 1980's, when growth population began to level off in most communities. The most rapid growth during this period occurred in rural communities with abundant open space and land available for development. In response to this growth, improved land-use planning, growth management and stricter development standards led to more sustained, managed growth over the last 20-30 years for most communities. In addition, escalating property values and high housing costs may have somewhat contributed to slower growth and development in certain communities.

3.3 HISTORICAL AND PROJECTED POPULATION

According to the Census, the Town of Ware has experienced additional population growth since the early 1960s. From 1960 to 1990 the population growth was strong and generally constant at the rate of 9.1% until 2000 when growth slowed significantly and became negative. At that point, growth resumed, but increased at a slower rate of approximately 0.18% per year through 2015. The current 2015 population as reported by UMDI is approximately 9,967 residents and the Census estimated a total population of approximately 9,888 residents in 2015.

Population projections as reported by the US Census, UMDI, MassDOT, and PVPC were reviewed for this study. The historic populations from 1940 to 2010 were provided by the US Census along with an estimated population in 2015. The UMDI projections were estimated in March of 2015 which provided projections from 2015 to 2035. Two sets of projections were used from MassDOT; an older projection from 2011 and an updated projection from 2015. The PVPC projections are from 2003. These various historic and projected populations are shown in Figure 3-2.

FIGURE 3-2
HISTORIC AND PROJECTED POPULATION
WARE, MASSACHUSETTS



As shown in the figure above, the MassDOT (2011) and PVPC projections have been higher than the actual 2010 population and increase at a rapid rate until 2030, while the UMDI and MassDOT (2015) projections only increase slightly until 2025 and then decreases until 2035 and 2040, respectively. Since the most recent projections show much slower growth, they are likely more realistic. Out of MassDOT (2015) and UMDI, UMDI is more conservative and likely more applicable for this Master Plan. Therefore, the UMDI projections for the next ten years, included in Table 3-2, were utilized in this study as they appear be more closely aligned with actual population trends.

TABLE 3-2
UMDI POPULATION PROJECTIONS
WARE, MASSACHUSETTS

Year	Projected Population
2016	9,986
2017	10,006
2018	10,025
2019	10,045
2020	10,064
2021	10,077
2022	10,090
2023	10,103
2024	10,116
2025	10,129

The UMDI projections show a slowing of growth over the next twenty years with an increase of 143 in population from 2016 to 2025. In regards to water service, the WDPW provides water to approximately 72% of the Town's population per the ASRs.

3.4 HISTORICAL WATER DEMAND TRENDS

The following discussion presents characteristics as it relates specifically to water demands. An analysis of historical water-use patterns is necessary to evaluate existing system capabilities and to understand future water supply and infrastructure needs. Within the context of this Report, a number of water industry terms will be used that are outlined below.

- Water *demand* and *production* is defined as the quantity of water which is pumped or produced from all sources of supply. Drinking water in Ware is currently supplied by the four active groundwater sources as discussed in Section 2. In general, demand from each individual source is metered, monitored, recorded, and reported by the WDPW.
- Water *consumption* is defined as the quantity of water used or consumed by the customers or for the operations of the system. Water consumption consists of two components: revenue water and non-revenue water. Revenue or metered water is the sum of all individual water meter readings from customers. Non-revenue water is water

which has been produced and delivered to the distribution system but is not billed to customers. Categories of non-revenue water include water used for un-metered accounts, bleeders, hydrant and main flushing, system leaks, water used for firefighting and losses from storage tank overflows.

MassDEP classifies all water users into seven account or user types as follows:

1. Residential
2. Residential Institutions
3. Commercial/Business
4. Agricultural
5. Industrial
6. Municipal/Institutional/Non-profit
7. Other

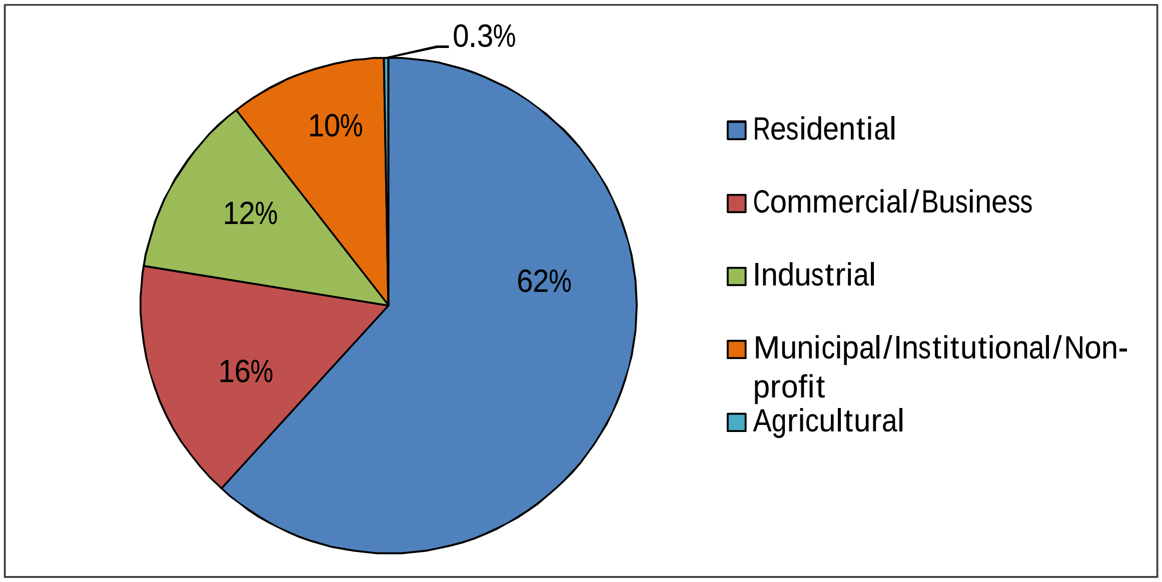
Table 3-3 presents Ware's historic average day demand for each category from 2006 through 2015.

TABLE 3-3
HISTORIC AVERAGE-DAY DEMANDS (MGY)
WARE, MASSACHUSETTS

Category	2006	2007	2008	2009	2010	2011	2012	2013	2014	2015
Residential	187.4	144.4	135.5	126.2	134.3	119.8	121.8	125.8	112.7	121.2
Commercial/Business	25.2	20.0	18.6	20.5	16.2	14.9	15.3	16.0	14.7	30.9
Agricultural	0	2.0	0.3	0.5	0.7	0.5	0.4	0.5	0.5	0.6
Industrial	35.5	38.0	27.7	22.3	27.2	26.2	30.1	31.6	26.0	23.4
Municipal/Institutional/ Non-profit	2.85	8.8	12.1	13.9	18.8	17.3	15.4	15.1	20.3	20.1
Other	4.87	5.8	4.4	1.1	1.2	1.6	1.7	1.9	1.1	0
Total Metered Use	255.8	218.9	198.6	184.6	198.4	180.2	184.7	191.0	175.4	196.2
Total Supplied	345.8	305.5	291.8	267.1	265.3	252.9	228.6	211.6	217.8	238.1

Year 2015 billing records indicate that the water system has 2,360 meter accounts. The approximate percentage of the total system demand by user type for 2015 is shown in Figure 3-3.

FIGURE 3-3
WATER CONSUMPTION BY DEMAND CATEGORY IN 2015
WARE, MASSACHUSETTS



As shown in the figure, the residential component accounts for the majority (approximately 62%) of the metered demands in the system. Then, the Commercial/Business have the second highest demand at approximately 16%.

Knowledge of average and maximum-day demands of a water system is required in order to evaluate the adequacy of the existing system. The annual average daily flow is useful in estimating total water demand, chemical needs associated with treatment, electric power consumption required for pumping, and long-term supply capacity (Safe Yield or Permitted Withdrawal). Average-day demand is defined as the total water-use in a year divided by 365 days.

The maximum-day demand is defined as the maximum day of water-use that occurs during a given year. The maximum daily demand is generally used to size pumping units, transmission mains, treatment processes, and storage facilities. The ratio of the maximum to average-day demand provides a general indication of the demand fluctuation over a typical day.

A third demand component useful in engineering design is the peak-hour demand. Peak-hour demand is the maximum demand that occurs over a one-hour period. Peak-hour demand is the maximum volume that must be provided by all sources in the system (water supply and storage). If data is not available to determine this component, it can be estimated.

3.4.1 Year-Round Water Demand Trends

Table 3-4 below presents a summary of system-wide demands, average-day demands and maximum-day demands for the last five years.

TABLE 3-4
WATER DEMAND TRENDS
WARE, MASSACHUSETTS

Year	Total Production (gallons/year)	Average Daily Demand (gallons/day)	Maximum Daily Demand (gallons/day)	Ratio (Maximum-day/Average-day)
	(A)	(B)	(C)	(C/B)
2011	252,928,000	692,953	1,228,000	1.77
2012	228,578,000	626,241	939,000	1.50
2013	211,624,500	579,793	1,177,000	2.03
2014	217,837,000	596,814	1,047,000	1.75
2015	238,055,000	652,205	1,061,000	1.63
Average	229,804,500	629,601	1,090,400	1.74

In general, the average day demand (ADD), maximum day demand (MDD), and demand ratio have been relatively consistent in the last five years. Therefore, the average demand ratio of 1.74 was utilized for the future MDD demand calculations later in this report.

3.4.2 Seasonal Water Demand Trends

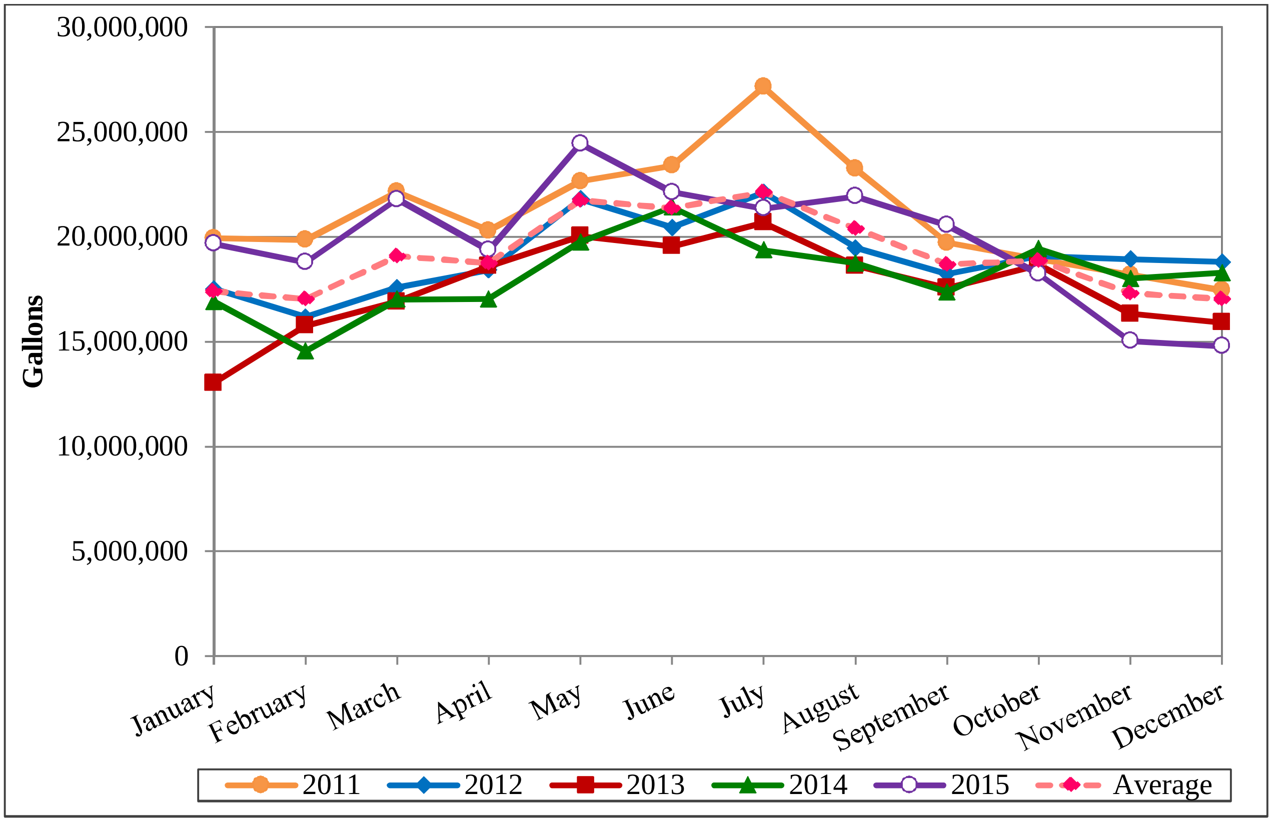
Water demand is typically a function of the time of year among other factors. In general, summer months have higher water demand due to the increased use of water for irrigation and recreation, in addition to seasonal population changes (if present in a particular community). Exceptions include industrial demands, which may follow demand patterns that result in higher average demands during the winter as opposed to the summer months.

The WDPW production trends by month for years 2011 through 2015 are presented in Table 3-5 and graphically in Figure 3-4.

TABLE 3-5
WATER PRODUCTION TRENDS
WARE, MASSACHUSETTS

Year	Total Water Production (Gallons)					
	2011	2012	2013	2014	2015	Average
January	19,932,000	17,518,000	13,028,500	16,902,000	19,676,000	17,411,300
February	19,858,000	16,180,000	15,769,000	14,555,000	18,790,000	17,030,400
March	22,136,000	17,579,000	16,904,000	17,018,000	21,790,000	19,085,400
April	20,299,000	18,412,000	18,599,000	17,028,000	19,361,000	18,739,800
May	22,640,000	21,805,000	20,038,000	19,751,000	24,437,000	21,734,200
June	23,392,000	20,453,000	19,540,000	21,413,000	22,128,000	21,385,200
July	27,140,000	22,112,000	20,673,000	19,358,000	21,350,000	22,126,600
August	23,241,000	19,467,000	18,610,000	18,735,000	21,923,000	20,395,200
September	19,709,000	18,232,000	17,563,000	17,368,000	20,549,000	18,684,200
October	18,938,000	19,088,000	18,658,000	19,419,000	18,237,000	18,868,000
November	18,191,000	18,939,000	16,333,000	18,008,000	15,032,000	17,300,600
December	17,452,000	18,793,000	15,909,000	18,282,000	14,782,000	17,043,600
Total	252,928,000	228,578,000	211,624,500	217,837,000	238,055,000	229,804,500

FIGURE 3-4
SEASONAL WATER DEMAND TRENDS
WARE, MASSACHUSETTS



As expected for a New England town, the general trend in the data shows that the demand increases from the winter months into the spring months and peaks during the summer months (June through August) before dropping again in the winter months. Variability in production between years can be seen during this same period which is expected due to the variability in precipitation from year to year.

3.4.3 Water Production Trends

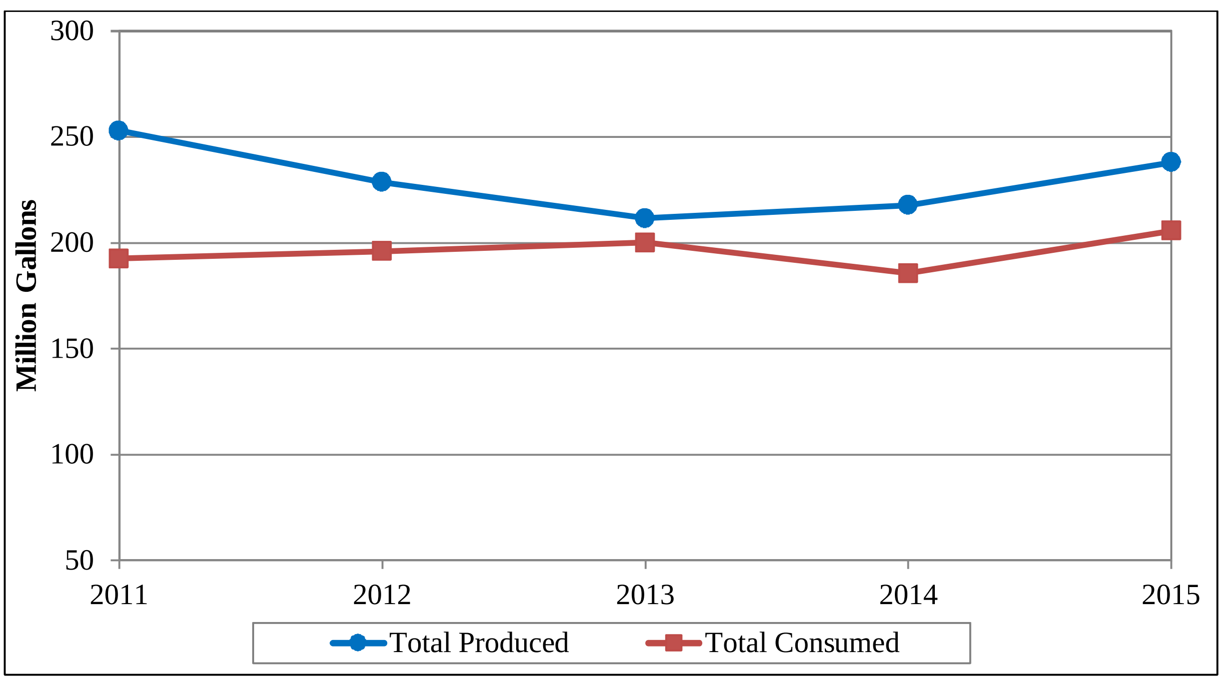
Water production is the total volume of raw water pumped from the well supply into the distribution system whereas water consumption is the actual volume of metered water billed to customers or other non-revenue water that is quantified. The difference between water produced and water consumed can be considered unaccounted-for water. Additional details and concepts regarding non-revenue and unaccounted-for water are presented in the sections that follow.

The WDPW water production and consumption trends for years 2011 through 2015 are presented in Table 3-6 and graphically in Figure 3-5.

TABLE 3-6
WATER PRODUCTION AND CONSUMPTION TRENDS
WARE, MASSACHUSETTS

Year	Total Water Production/Consumption (Million Gallons)		
	Production	Consumption	Difference
2011	252.9	192.6	60.3
2012	228.6	196.0	32.5
2013	211.6	200.1	11.6
2014	217.8	185.7	32.2
2015	238.1	205.7	32.3
Average	229.8	196.0	33.8

FIGURE 3-5
WATER PRODUCTION AND CONSUMPTION TRENDS
WARE, MASSACHUSETTS



3.4.4 Revenue and Non-Revenue Water-Use Trends

Records from the production sources were used as the baseline for determining the WDPW's revenue and non-revenue water-use. In general, revenue water is water-use that has been metered and billed to customers while non-revenue water is water-use that is not metered or results from inaccuracies of metering and other sources previously described. Sources of non-revenue water may include that which is needed for water operations, such as hydrant and water main flushing, leaks in the distribution system, accuracy of meters, un-metered or non-functioning services, lost water, water main breaks, unauthorized use, drainage of storage facilities for maintenance or repair, or accounting errors. Table 3-7 presents a breakdown of typical revenue and non-revenue sources in a system.

TABLE 3-7
REVENUE AND NON-REVENUE WATER USE CATEGORIES*

<table>
<tr><td rowspan="10">Total Production Volume (corrected for known errors)</td><td rowspan="4">Authorized Consumption</td><td rowspan="2">Billed Authorized Consumption</td><td>Billed Metered Consumption
(Including water exported)</td><td rowspan="2">Revenue Water</td></tr>
<tr><td>Billed Unmetered Consumption</td></tr>
<tr><td rowspan="2">Unbilled Authorized Consumption</td><td>Unbilled Metered Consumption</td><td rowspan="8">Non-Revenue Water (NRW)</td></tr>
<tr><td>Unbilled Unmetered Consumption</td></tr>
<tr><td rowspan="6">Water Losses</td><td rowspan="3">Apparent Losses</td><td>Unauthorized Consumption</td></tr>
<tr><td>Customer Metering Inaccuracies</td></tr>
<tr><td>Data Handling Errors</td></tr>
<tr><td rowspan="3">Real Losses</td><td>Leakage on Transmission and Distribution Mains</td></tr>
<tr><td>Leakage and Overflows at Utility's Storage Tanks</td></tr>
<tr><td>Leakage on Service Connections up to point of Customer metering</td></tr>
</table>

*** From AWWA M36.**

Following is a list of definitions for the various terms used herein.

- Total Production Volume - The annual volume input to the water supply system.
- Authorized Consumption - The annual volume of metered and/or unmetered water taken by any user authorized to do so.
- Water Losses - The difference between Total Production Volume and Authorized Consumption, consisting of Apparent Losses plus Real Losses.

- Apparent Losses - Unauthorized Consumption, all types of metering inaccuracies and data handling errors.
- Real Losses - The annual volumes lost through all types of leaks, breaks and overflows on mains, service reservoirs and service connections, up to the point of customer metering. Commonly referred to as lost water.
- Revenue Water - Those components of Total Production Volume which are billed and produce revenue.
- Non-Revenue Water (NRW) - The difference between Total Production Volume and Billed Authorized Consumption.

Table 3-8 presents data comparing WDPW's production water volume to the revenue water volume.

TABLE 3-8
REVENUE AND NON-REVENUE WATER USE
WARE, MASSACHUSETTS

Year	Total Production (MGY)	Total Revenue Water (MGY)	Non-Revenue Water (MGY)	% Non-Revenue Water
2011	252.9	180.2	72.7	28.74%
2012	228.6	184.7	43.9	19.18%
2013	211.6	191.0	20.6	9.74%
2014	217.8	175.4	42.5	19.50%
2015	238.1	196.2	41.9	17.60%
Average	229.8	185.5	44.3	18.95%

The data from the table above indicates that non-revenue water has averaged approximately 19% over the past five years.

Sources of unaccounted for water reported in the WDPW's MassDEP Annual Statistical Reports (2011 - 2015) include:

- Water used for system-wide hydrant and main maintenance flushing.

- Water required for new water main construction purposes. This includes water used for filling and flushing new mains, chlorinating, and flushing chlorinated water.
- Water used for fire protection and training (includes flow tests).
- Water used for sewer and stormwater system flushing.
- Water used for street cleaning.
- Tank overflow and drainage.
- Lost water as a result of water main breaks and resulting repairs.
- Lost water from bleeders and blow offs to improve water quality in portions of the system.

Some non-revenue water uses can be confidently estimated by the water supplier and are therefore considered "authorized uses" of water. The remaining volume is considered water losses.

Industry standards suggest that the total lost water volume should be no higher than 20% of the total production volume while real losses, true unaccounted-for water, should be no more than 10% of total production volume. Many states, including Massachusetts, have made or are considering making unaccounted-for water a condition of approval for new supply sources and require communities to maintain unaccounted-for water at 10% or less. Massachusetts requires that water systems reduce unaccounted-for water use to less than 10% in order to move forward with developing new sources of water supply. In addition, MassDEP has established performance standards for all water systems that restricts unaccounted-for water to 10% or less.

Leaks are often the largest contributor to unaccounted-for water. Leaks can originate from anywhere in the system. The largest sources of leakage typically occur on main lines or through valves. Other sources of leaks include service-lines, residential meter boxes, residential leakage on the customer side of the service and other miscellaneous types.

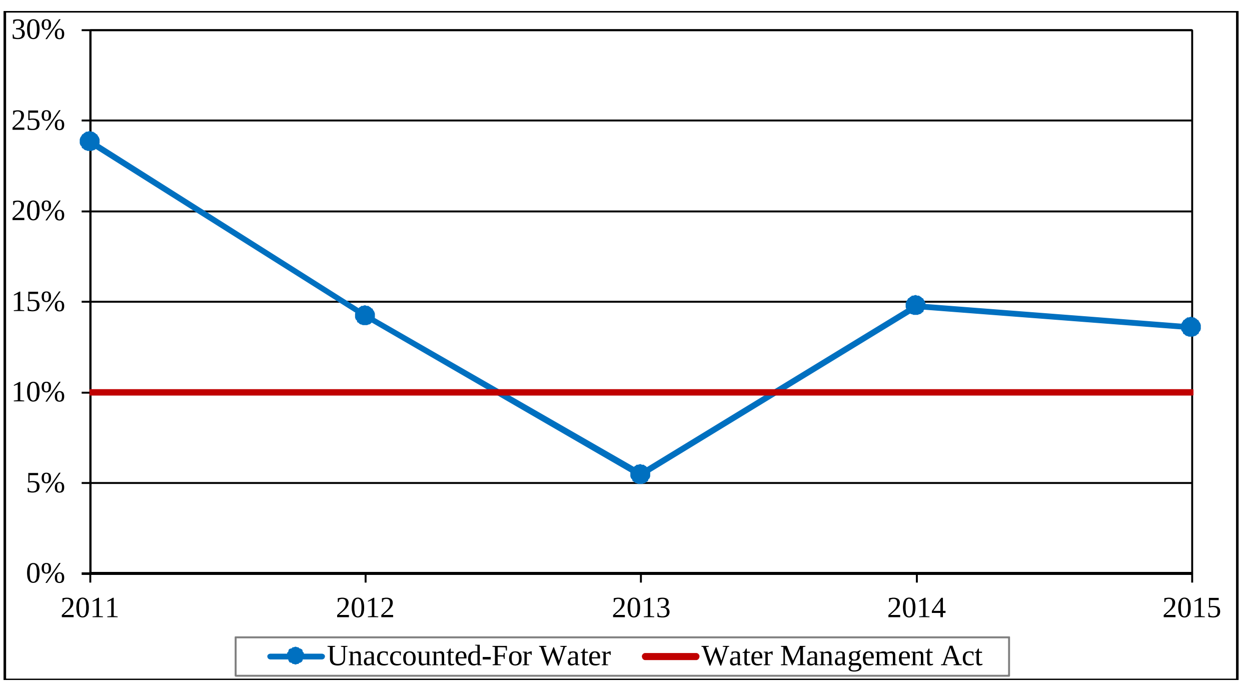
Table 3-9 and Figure 3-6 presents data as reported in the MassDEP Annual Statistical Reports related to lost water also known as unaccounted-for water (UAW) in the Ware system. The

UAW ranged from approximately 5% to 24% with an average of 14.4%. This is higher than the Water Management Act performance standard of 10%.

TABLE 3-9
UNACCOUNTED FOR WATER USE
WARE, MASSACHUSETTS

Year	Non-Revenue Water (MGY)	% of Total Production	Estimate of Non-Revenue which has been Accounted-for	Remaining Unaccounted which has NOT been Accounted-for (UAW)
2011	72.7	28.7%	4.9%	23.8%
2012	43.9	19.2%	5.0%	14.2%
2013	20.6	9.7%	4.3%	5.5%
2014	42.5	19.5%	4.7%	14.8%
2015	41.9	17.6%	4.0%	13.6%
Average	44.3	19.0%	4.6%	14.4%

FIGURE 3-6
UNACCOUNTED FOR WATER USE
WARE, MASSACHUSETTS



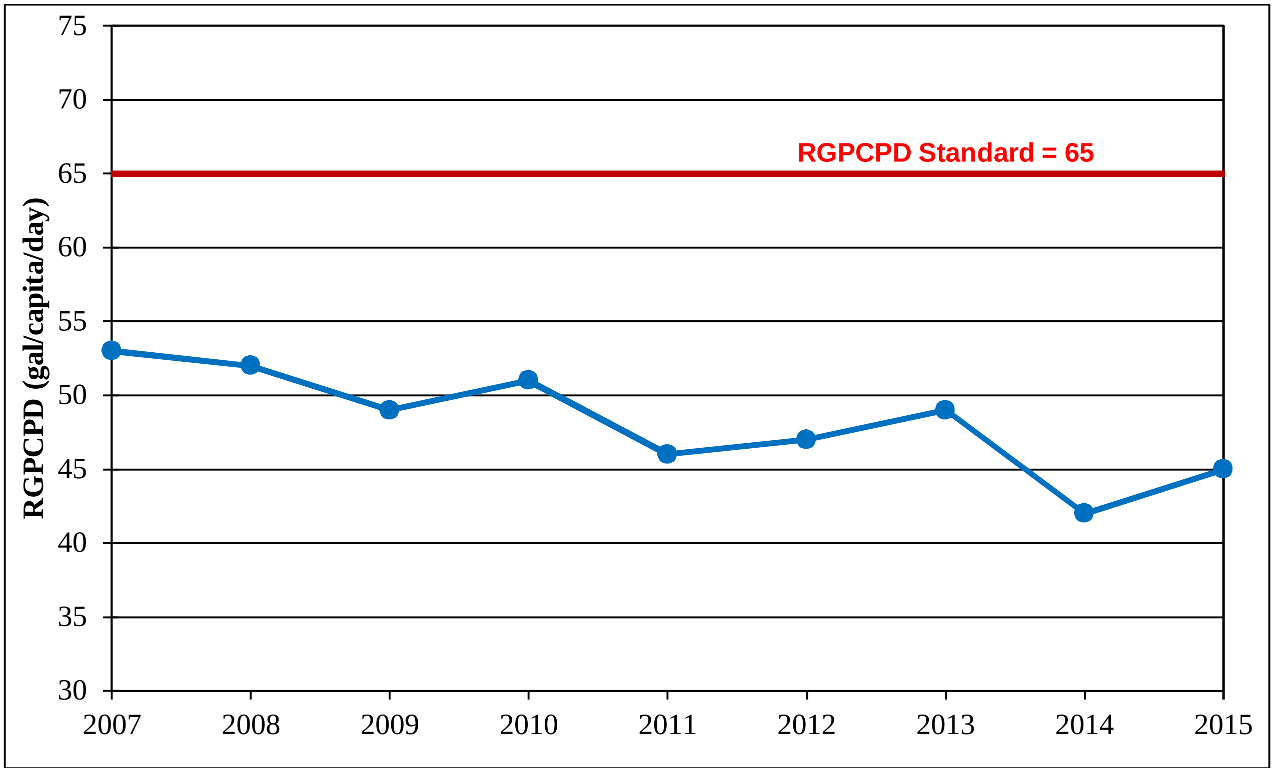
It should be noted that Table 3-9 and Figure 3-6 list and present the UAW values from WDPW's ASRs. After reviewing each year's ASR, MassDEP corrected the WDPW's reported UAW values based upon their own calculations and analysis. According to MassDEP, only one of the UAW (%) values was corrected within this time frame. The corrected value is 10% for year 2013.

In order to comply with MassDEP's performance standard, it will be important for WDPW to gain a clear understanding of the true magnitude of the lost water component of water use. The biggest gains in reducing lost water typically will come from one of several sources: (1) improving accuracies in master and customer meters, (2) controlling where possible variations in water demand, particularly that of large customer users, (3) reduction in main leakage, and (4) improving the accounting, estimation and reporting procedures for non-metered use.

3.4.5 Residential Gallons per Capita per Day Water Consumption

As presented in Figure 3-7 per capita residential water-use in Ware has ranged between 42 and 53 residential gallons per capita per day (rgpcpd) over the past nine years. The WMA permit limits residential consumption to 65 rgpcpd on an annual basis.

FIGURE 3-7
HISTORICAL WATER-USE TRENDS
RESIDENTIAL GALLONS PER CAPITA PER DAY
WARE, MASSACHUSETTS



The values for the last several years are excellent by any standard and are indicative of a well-managed system. It is likely that water use restrictions, conservation requirements, and other provisions in the permit are leading to lower water use. To be conservative however, future water-use projections will be based on 65 rgpcpd for residential water customers. Also, the Massachusetts Department of Conservation and Recreation (DCR) utilizes 65 rgpcpd for their water demand projections to determine the WMA permitted withdrawal rates.

3.4.6 Largest Water-Use Customers

The ten largest water users were identified from the billing database. This data is presented within Table 3-10. These customers and their demands were assigned specific nodes in the hydraulic model developed for this Report. Large water users can have a significant impact on water demand and alterations in the water use patterns for the larger customers could significantly influence future water use.

TABLE 3-10
2015 LARGEST WATER USERS
WARE, MASSACHUSETTS

Rank	Account	Customer Name	Description	Service Address	Gallons/Year	Gallons/Day
1	05-1683	Kanzaki Papers	Industrial Manufacturing	60 Cummings Street - Boiler House	2,005,080	5,493
2	02-2606	Waste Water Treatment Plant	Municipal	30 Robbins Road	1,172,610	3,213
3	05-2525	Kanzaki Papers	Industrial Manufacturing	38 Cummings Street	761,530	2,086
4	05-0021	Baystate Mary Lane Hospital	Hospital	85 South Street	483,120	1,324
5	01-1787	Norcor Auto Wash Inc.	Carwash	134 West Street	223,480	612
6	03-2487	Walmart	Retail Store	352 Palmer Road	211,770	580
7	06-2588	Quabbin Wire and Cable	Industrial Manufacturing	10 Maple Street	207,188	568
8	05-1681	Baystate Mary Lane Hospital	Hospital	60 South Street	176,907	485
9	01-1666	Sean Madigan	Laundromat	142 West Street	115,990	318
10	02-2046	Town of Ware	School Building (Elementary)	4 Gould Road	109,100	299

As shown, the top water users are within the industrial, municipal, and commercial categories. In 2015, the top ten water users consumed approximately 5.5 million gallons of water, or approximately 2.8% of the total metered water use. This small percentage indicates that the largest water users have a minimal impact on the overall system performance.

3.5 WATER USE PROJECTIONS THROUGH THE PLANNING PERIOD

An understanding of current and future average and maximum daily demands of a water system is required in order to evaluate the existing system and plan for future needs. The annual average daily flow is useful in estimating total water demand, chemical needs associated with treatment, electric power consumption required for pumping, and long-term supply capacity (safe or permitted yield). The maximum daily demand is generally used to size transmission mains, treatment processes and equipment, and storage facilities.

3.5.1 Water Demand Projection Methodology

3.5.1.1 Residential

Residential water-use is the result of residential demand by populations living within the Ware water system. Residential users include single family and multifamily dwellings, as well as apartments. On average, the residential component of the total revenue-water is about 62% of the total water-use.

MassDEP performance standards set a residential per capita demand goal of 65 residential gallons per capita per day (rgpcd). The calculated average-per capita water consumption in the Ware water system over the last five years is approximately 46 rgpcd, which is well below the MassDEP standard. However, in order to account for potential fluctuations in demand due to annual changes in weather and rainfall, the MassDEP per capita goal of 65 rgpcd was utilized in the demand projections. Additionally, as only approximately 72% of the Town's population is served by the WDPW, 72% of the projected population was also utilized for the residential demand projection.

3.5.1.2 Commercial

Commercial water-use consists of business parks, restaurants, retail stores, car washes, banks, etc. located within the service area. In just the last year in 2015, commercial demand increased by almost double compared to 2014. Since 2006, the lowest annual demand took place in 2014 at approximately 14.7 million gallons and the highest demand took place in 2015 with approximately 30.9 million gallons. The average commercial water-use since 2006 has been 18.4 MGY.

Employment projections from the Department of Conservation and Recreation (DCR) estimate approximately 2,836 employees in 2017 and 2,884 employees in 2023 which is an increase in employment by about 1.7%. It is assumed that the employment increase will directly correlate with the commercial demand. Therefore, an increase of 0.28% per year was utilized for the commercial demand projections.

3.5.1.3 Agricultural

In the last five years, agricultural demand (by the one noted user) has ranged from 0.4 to 0.6 MGY and averaged approximately 0.5 MGY. Agricultural demand has had an average annual increase of approximately 0.07 MGY from 2012 to 2015, and therefore this demand is expected to increase over the planning period. The average increase of 0.07 MGY was utilized for the agricultural demand projections.

3.5.1.4 Industrial

In the last five years, industrial demand has ranged from 23.4 to 31.6 MGY and averaged approximately 27.5 MGY. Industrial demand is not expected to increase over the planning period. Therefore, the average demand of 27.5 MGY was utilized for the industrial demand projections.

3.5.1.5 Municipal

Municipal water-use is water used by schools, government offices, etc. located within the Ware system. In the last five years, municipal demand has ranged from 15.1 to 20.3 MGY and averaged approximately 17.6 MGY. Municipal demand is not expected to increase over the planning period. Therefore, the average demand of 17.6 MGY was utilized for the municipal demand projections.

3.5.1.6 Unaccounted-For Water

As discussed, UAW ranged from approximately 5% to 24% with an average of 14.4%. MassDEP requires that water systems work to achieve a maximum of 10% unaccounted-for water. The Ware system is close to meeting the MassDEP requirement; however, the 14.4% average for unaccounted-for water was utilized for the projections to be more conservative.

3.5.2 Average Day Water Demand Projections

Table 3-11 presents the projected average daily demands based on the methodology described above.

TABLE 3-11
PROJECTED AVERAGE-DAY DEMANDS (MGY)
WARE, MASSACHUSETTS

Category	2016	2017	2018	2019	2020	2021	2022	2023	2024	2025
Residential	170.6	170.9	171.3	171.6	171.9	172.1	172.4	172.6	172.8	173.0
Commercial/Business	31.0	31.1	31.2	31.3	31.4	31.44	31.5	31.6	31.7	31.8
Agricultural	0.61	0.68	0.75	0.82	0.89	0.96	1.03	1.10	1.17	1.24
Industrial	27.5	27.5	27.5	27.5	27.5	27.5	27.5	27.5	27.5	27.5
Municipal/Institutional/ Non-profit	17.6	17.6	17.6	17.6	17.6	17.6	17.6	17.6	17.6	17.6
Other	0	0	0	0	0	0	0	0	0	0
Total Metered Use	247.9	247.8	248.3	248.8	249.3	249.6	250.0	250.4	250.8	251.2
Unaccounted-For Water (14.4%)	35.7	35.7	35.8	35.8	35.9	35.9	36.0	36.1	36.1	36.2
Total Water Use	283.6	283.5	284.0	284.6	285.1	285.6	286.0	286.4	286.9	287.3

3.5.3 Maximum and Peak Hourly Flow Demand Projections

As previously discussed, the average peaking factor for the last five years of 1.74 was utilized to estimate the future maximum daily demands. Due to the unavailability of daily demand data for the maximum day to calculate the peak hourly demand, the peak hourly demand will be estimated. Communities of similar size to Ware tend to have a peak hour demand between 2 to 3 times the average day hourly demand. Therefore, a peak hour peaking factor of 3 was utilized to estimate future peak hour demands to be conservative. The resultant projected maximum day and peak hour demands are presented in Table 3-12.

TABLE 3-12
PROJECTED MAXIMUM-DAY DEMANDS
WARE, MASSACHUSETTS

Year	ADD (MGD)	MDD (MGD)	Peak Hour (MGH)
2016	0.78	1.35	0.097
2017	0.78	1.35	0.097
2018	0.78	1.35	0.097
2019	0.78	1.36	0.097
2020	0.78	1.36	0.098
2021	0.78	1.36	0.098
2022	0.78	1.36	0.098
2023	0.78	1.37	0.098
2024	0.79	1.37	0.098
2025	0.79	1.37	0.098

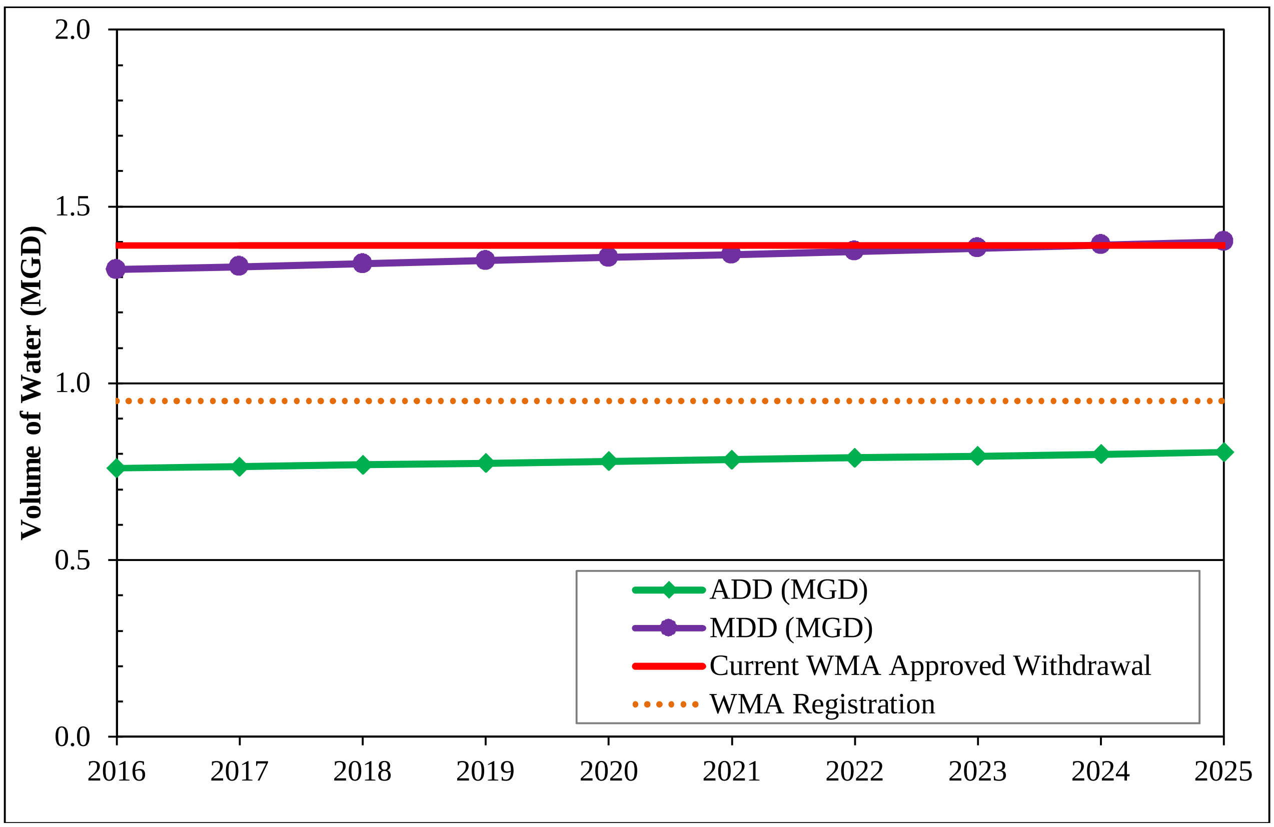
The projected maximum day and average day demand in 2025 is 1.37 MGD and 0.79 MGD, respectively, with a peak hour of 0.098 MGH.

3.6 WATER MANAGEMENT ACT

The Massachusetts Water Management Act (WMA) places water withdrawal limits on water supply sources in part to control water withdrawals from watersheds to ensure the adequate natural water supply needs of flora and fauna that inhabit the watersheds. The WDPW has five registered water supply wells (Well No.1, Well No.2, Well No.3, Well No.4/Giard Well, and the Cistern) and one permitted supply well (Dismal Swamp Well). The WMA registration authorizes withdrawal of 0.95 MGD on average over the calendar year. The current WMA permit authorizes an additional withdrawal of 0.44 MGD for a total authorized withdrawal of 1.39 MGD (through 5/31/2015). The most recent copies of the WDPW's registration statement and WMA Permit are included within Appendix A. It should be noted that the WMA Permit expired on May 31, 2015. Until a new permit is issued to the WDPW, compliance and analysis within this report is based on the most recent authorized annual withdrawal volumes (i.e., Period 5).

The withdrawal limits and projected water demands through year 2025 are shown in Figure 3-8.

FIGURE 3-8
PROJECTED WATER DEMANDS
WARE, MASSACHUSETTS



The data indicates that the WDPW generally has adequate water supply capacity through year 2025 based on the projections presented herein. It should be noted that the WDPW currently has mandatory non-essential outdoor water use restrictions in place that help to reduce the average and maximum daily demands in the system. Therefore, it will be important to continue these restrictions to keep demands below the WMA registered withdrawal volume. A detailed review of the existing sources ability to meet the projected demands is presented in Section 4.

3.6.1 SWMI

Ware's WMA permit will be renewed shortly and as a basis for this permit, the Massachusetts Department of Environmental Protection (MassDEP) utilizes the Massachusetts Department of Conservation and Recreation (DCR) water demand projections to determine the WMA permitted withdrawal volumes. The DCR provided a draft of the Town's water needs forecast in January of 2016 which is presented in Table No. 3-13.

Also, the WMA regulation has now started to integrate the Sustainable Water Management Initiatives (SWMI). The SWMI would impose additional regulations onto a Town based upon the Town's permitted withdrawal volume.

TABLE 3-13
DRAFT WATER NEEDS FORECAST FROM DCR
WARE, MASSACHUSETTS

	2017	2023	2028	2033
ADD Projection (MGD)[1]	0.78	0.79	0.79	0.78
ADD Projection (MGD)[2]	0.66	0.67	0.67	0.67

[1] Assuming 65 RGPCD and 10% UAW. Includes 5% buffer of +0.04.
[2] Assuming water delivery continues at current RGPCD and UAW. Includes 5% buffer of +0.03.

In accordance with the new SWMI regulations that are now included within the WMA, each applicant is assigned a Baseline for water use. The Baseline is a parameter that MassDEP developed in order to determine an applicant's applicability for a requested volume for their permit renewal. The Baseline water use is calculated by determining the volume withdrawn in 2005 plus 5%, the average annual volume withdrawn from 2003 through 2005 plus 5%, or the registered amount. Whichever option provides a greater value is determined as the Baseline. Ware's Baseline is 1.09 MGD which is based off of the volume withdrawn during 2005 plus 5%. As previously provided in Table No. 3-11, the future estimated average day demand for 2025 could reach a total of 287.3 MGY (0.79 MGD). DCR's draft water needs forecast projects a total demand of 0.78 MGD by year 2033 in its first scenario. Both of these projections are below the established Baseline.

In accordance with the new SWMI regulations, MassDEP has established review categories called “tiers” for all water supply systems as part of the permit requirement. The calculated Baseline along with the requested water withdrawal volume is ultimately the threshold for determining an applicant’s tier. There are a total of three tiers and each tier has specified requirements that Ware would be required to fulfill based on a variety of categories established by the WMA. If the Town’s water demand surpasses the Baseline, then the Town would fall into a tier where there will be additional requirements placed upon the Town. Additional requirements would include submitting a minimization plan, performing additional conservation measures, optimizing withdrawal, and returning water to the sub-basin(s). The projected average day future demand that was previously calculated (as well as DCR’s projection) does not surpass the Baseline. Therefore, the Town should not expect to have any additional requirements from the new SWMI regulations related to increased water withdrawal.

Section 4

SECTION 4

WATER SUPPLY EVALUATION AND ASSESSMENT

4.1 GENERAL

As presented within the previous two sections of this report, the Ware Department of Public Works (WDPW) utilizes four active groundwater sources for its water supply. Withdrawal from each source of supply is permitted through the Massachusetts Water Management Act (WMA). The WDPW's current permit includes the five previously registered groundwater wells (Well No.1, Well No.2, Well No.3, Well No.4/Giard Well, and Cistern) and one permitted supply well (Dismal Swamp Well). The registration authorizes a withdrawal of 0.95 million gallons per day (MGD) on average over the calendar year and the WMA permit authorizes an additional average daily withdrawal of 0.44 MGD. This results in a total authorized average daily withdrawal of 1.39 MGD for all sources (through 5/31/2015 as noted in the previous section).

This section presents the evaluation and assessment of those sources' ability to reliably meet the forecasted water use needs for the system.

4.2 ADEQUACY OF EXISTING WATER SUPPLY CAPACITY

A water system is considered to have adequate long-term supply if it can meet the following system conditions:

- Design Condition No. 1 - The permitted annual average-day pumping rate of the source of supply should exceed the projected average-day demand, and;
- Design Condition No. 2 - The pumping capacity of the system with the largest source (or pumping unit) out of service should be greater than or equal to the projected maximum-day demand.

Both conditions should be met in order to assure the reliability of service to the customers. Each of these conditions has been evaluated on a system-wide basis for the WDPW and the results are presented in the following sections of the report.

Table 4-1 summarizes the WMA's maximum authorized daily withdrawal volumes for each well individually as well as a registered and permitted total. The individual withdrawals included for the registered sources are based on the approved maximum daily pumping volume that was assigned to the source in accordance with its Zone II or pump test. The individual withdrawals for the permitted sources are taken from the WMA permit.

TABLE 4-1
MAXIMUM AUTHORIZED DAILY WITHDRAWAL VOLUMES
WARE, MASSACHUSETTS

Source	PWS Source ID	Maximum Authorized Annual Average		Individual Withdrawals (MGD)	Maximum Daily Rate (MGD)
		Registered	Permitted		
Well No. 1	1309000-01G	Yes	No	0.317	0.95
Well No. 2		Yes	No	0.317	
Well No. 3		Yes	No	0.317	
Well No. 4/Giard Well	1309000-02G	Yes	No	0.720	0.72
Cistern	1309000-04G	Yes	Yes[1]	0.475	1.08[3]
Dismal Swamp Well	1309000-03G	No	Yes	0.583	0.583
Total (MGD):		**0.95**	**0.35 - 0.44[2]**	**2.729**	**2.383**
TOTAL (Registered & Permitted) (MGD):		**1.30 - 1.39[2]**			

[1] Rate Limitation (Max Day)
[2] Daily Average (per period as noted in permit)
[3] Combined rate for Cistern and Wells No. 1, 2, and 3.

Due to permitting restrictions, it is noted that the total authorized withdrawal amounts by the WMA permit do not match the sum of all individual sources. The total authorized withdrawal is currently at 1.39 MGD (0.95 MGD registered and 0.44 MGD permitted) and the total individual withdrawals add up to 2.7 MGD (almost double of the total withdrawal).

As presented within Section 2 of this report, the WDPW treats its sources at two water treatment plants (WTPs) which can also be referred to as chemical feed facilities; the Pump House and the Dismal Swamp Well Control Building. The Pump House treats the water from the Wellfield

(Wells No. 1, 2, and 3), Well No. 4, and the Cistern. Table 4-2 presents the pumping capacities of the WDPW's current wells and associated WTPs.

TABLE 4-2
WELL AND WTP PUMPING CAPACITIES
WARE, MASSACHUSETTS

Source	Well Capacity (MGD)	WTP Capacity (MGD)
Well No. 1	0.317	1.584
Well No. 2	0.317	
Well No. 3	0.317	
Cistern	0.475	
Well No. 4/Giard Well	0.720	
Dismal Swamp Well	0.583	0.583
Total:	**2.729**	**2.167**

It is noted that the actual capacity of a well is dynamic as wells lose capacity over time and can regain some of that lost capacity after a cleaning. Therefore, the design pumping capacity is more often used when evaluating the adequacy of a groundwater system unless extreme circumstances to the contrary are known.

4.2.1 Average-Day Demand Analysis

As presented previously (Design Condition No. 1), the first analysis of the ability for a water system to meet anticipated demands is to confirm whether or not the sources can meet the projected average-day demands with all available sources. As it is good waterworks practice to run the wells on a 16 hour on and 8 hour off basis over a regular period of 24 hours, the available capacities based on 16 hours of runtime (available safe yield) were calculated and used for the analysis.

Table 4-3, which follows, presents the summarized results of average-day demand analysis.

TABLE 4-3
AVERAGE-DAY DEMAND ANALYSIS RESULTS
WARE, MASSACHUSETTS

Source	Well Capacity (MGD)	WTP Capacity (MGD)	Available flow @ 16-hours of Pumping (MGD)
Well No. 1	0.317	1.584	1.056
Well No. 2	0.317		
Well No. 3	0.317		
Cistern	0.475		
Well No. 4/Giard Well	0.720		
Dismal Swamp Well	0.583	0.583	0.389
Total:	**2.729**	**2.167**	**1.445**

By comparing the projected average-day required total of 0.79 MGD for 2025, it can be seen that the WDPW system would have adequate water capacity under this analysis.

4.2.2 Maximum-Day Demand Analysis

Also as discussed previously (Design Condition No. 2), the second analysis of the ability for a water system to meet anticipated demands is to confirm whether or not the sources can meet the projected maximum-day demands with the largest available source considered to be off-line (i.e., unavailable). As it is good waterworks practice to run the wells on a 16 hour on and 8 hour off basis over a regular 24 hour period, the available capacity based on 16 hours of runtime (available safe yield) was also used as the starting point for this analysis.

Since the WDPW has all of its wells connected to WTPs, the analysis was run under two scenarios. The first was performed to assess the impact of losing the largest connected source (i.e., well) and the second was performed to assess the impact of losing the largest connected WTP. Both of these scenarios were run for the system as it currently exists.

Table 4-4 presents the summarized results of the first maximum-day analysis that assessed the loss of the largest source (Well No. 4).

TABLE 4-4
MAXIMUM-DAY DEMAND RESULTS – LARGEST SOURCE OFF-LINE
WARE, MASSACHUSETTS

Source	Well Capacity (MGD)	WTP Capacity (MGD)	Available flow @ 16-hours of Pumping (MGD)
Well No. 1	0.317	1.426	0.951
Well No. 2	0.317		
Well No. 3	0.317		
Cistern	0.475		
Well No. 4/Giard Well	0.000		
Dismal Swamp Well	0.583	0.583	0.389
Total:	**2.009**	**2.009**	**1.339**

By comparing the projected maximum-day required total of 1.37 MGD for 2025, it can be seen that the WDPW system would be in a slight deficit of 0.031 MGD (1.37 – 1.339 MGD) under this analysis scenario. Although this analysis indicates a small deficit, it could be overcome with additional pumping (as analysis utilizes 16 hours) and/or from storage in the system. For example, running the analysis with 17 hours would indicate sufficient capacity at 1.423 MGD.

Table 4-5 presents the summarized results of the first maximum-day analysis that assessed the loss of the largest WTP (the Pump House).

TABLE 4-5
MAXIMUM-DAY DEMAND RESULTS – LARGEST WTP OFF-LINE
WARE, MASSACHUSETTS

Source	Well Capacity (MGD)	WTP Capacity (MGD)	Available flow @ 16-hours of Pumping (MGD)
Well No. 1	0.317	0.000	0.000
Well No. 2	0.317		
Well No. 3	0.317		
Cistern	0.475		
Well No. 4/Giard Well	0.720		
Dismal Swamp Well	0.583	0.583	0.389
Total:	**2.729**	**0.583**	**0.389**

By comparing the projected maximum-day required total of 1.37 MGD for 2025, it can be seen that the WDPW system would not have adequate water capacity under this analysis scenario even if the remaining sources were temporarily run non-stop for 24-hour operation (assuming all other sources were operable).

However, it should be noted that the Pump House currently has a generator for emergency power and also the Cistern has two pumps available to pump water through the WTP. One pump is for back-up in case the other pump goes down. Nonetheless, this scenario should still be considered a possibility as a potentially catastrophic event could occur that renders the Wellfield, Well No. 4, and the Cistern sources inoperable (e.g., loss due to unforeseen contamination that cannot be treated). Other potential reasons for loss of capacity can include failure or temporary loss of treatment equipment, regulatory actions limiting use, scheduled and unscheduled maintenance, etc.

4.3 OPPORTUNITIES FOR EXPANDED WATER SUPPLY

Based on the analyses presented in the previous section, the WDPW has sufficient supply capacity to meet its projected average-day demands but not for its projected maximum-day demands when the largest well is considered to be off-line and pumping is limited to 16 hours of operation. However, this condition can be easily met when pumping is limited to 17 hours.

Under the most extreme scenario, the WDPW cannot meet its maximum-day demands when the WTP is considered inoperable. This would likely have a low probability, as there are two pumps feeding into the facility and it has emergency power provisions. However, other catastrophic events that render the WTP unusable should also be considered. Therefore, in order for the WDPW to more reliably meet the maximum-day demands under the more extreme scenario, other reliable sources of supply should be considered for implementation to make up the difference in an emergency. Based on the scenario that considered the largest WTP to be off-line, a deficit of approximately 0.981 MGD (1.37 MGD – 0.389 MGD) is identified.

The following sections present available options to the WDPW for this.

4.3.1 Interconnections

A possible source of additional supply would be an interconnection with a neighboring community (or communities) via an intermunicipal agreement (IMA) or a large water supplier such as the Massachusetts Water Resources Authority (MWRA). The following two sections present these options further.

4.3.1.1 Neighboring Communities

As presented previously within Section 2 of this report, the Town of Ware is surrounded by eight neighboring communities, but the WDPW does not have interconnections with any of these communities.

If the establishment of a suitable interconnection and IMA for the purchase of water from a neighboring community be desired, then at a minimum, the following major conditions would need to be satisfied for this option to be viable:

- Adequate and guaranteed supply quantity from the supplier;
- Proper hydraulics for the transfer of the water supply into the WDPW system;
- A permanent, reliable, and redundant interconnection;
- Acceptable and compatible water quality; and
- No impacts to the WDPW's distribution system.

Should a formal interconnection be desired, it is important to understand each contributing cost factor in a neighboring community's cost structure to determine if an interconnection makes sense for each community. The economic decision to purchase water from an adjacent utility requires consideration of two costs:

- <u>Marginal or Production Cost:</u> The bare or production cost of water at a utility to produce, treat and deliver water to the distribution system; and

- Avoided Cost: The cost to develop or treat a similar supply within the receiving utility's service area.

A utility considering an interconnection with an adjacent community to purchase water should be willing to pay somewhere between the avoided cost to develop its own independent supply and the selling community's marginal production cost. If the price of purchasing water is greater than the community's ability to develop or treat its own supply at a lower cost, then no incentive exists to purchase water from an adjacent water system.

The WDPW has noted that there would be no incentive to purchase water from a neighboring community since there would be a high cost associated with this. Therefore, it would not be recommended for WDPW to establish an interconnection.

Additional effort would need to be expended by the WDPW should it desire to pursue a formal interconnection with one of its neighboring community water systems which is beyond the scope of this Master Plan.

4.3.1.2 MWRA

Another long term water supply alternative would be an emergency interconnection to the MWRA system. The nearest communities served by MWRA water include Chicopee, South Hadley, and Wilbraham. These three towns are fully served by the MWRA. Therefore, access to the MWRA for the WDPW would require a wheeling agreement through the Chicopee, South Hadley, or Wilbraham distribution systems.

Additional effort would need to be expended by the WDPW should it desire to pursue a formal emergency interconnection with the MWRA which is beyond the scope of this Master Plan.

4.3.2 New Sources

If desired, another alternative for improved long term water supply would be the implementation of a new groundwater well source or sources. However, this solution is not guaranteed due to many unknowns.

4.3.2.1 New Source Approval Process

If the WDPW desired to implement a new groundwater source(s), then the WDPW would need to follow the New Source Approval (NSA) process. The NSA, in conjunction with the Water Management Act Withdrawal Permit application process, requires applicants to evaluate potential impacts caused by the proposed withdrawals. MassDEP receives comments from the Executive Office of Environmental Affairs (EOEA) through the Massachusetts Environmental Policy Act (MEPA) (301 CMR 11.00) review process to ensure protection of natural resources.

The process of exploring, testing, permitting, and developing a new water supply source can be a difficult and costly endeavor. The following state-level permits, at a minimum are required:

- MassDEP New Source Approval (NSA)
- Massachusetts Environmental Policy Act (MEPA) Environmental Notification Form (ENF)
- MassDEP Water Management Act (WMA)
- Potentially, MEPA Environmental Impact Report (EIR)
- Massachusetts Natural Heritage and Endangered Species Program (NHESP)
- And others potentially identified in the process.

In addition, local permits from the conservation commission, for example, may be needed depending upon the location of the proposed water supply.

The NSA process is involved, requires many steps, and can't be completed until the other state permits are successfully approved. The following outlines the various steps, in a roughly chronological order, required to navigate the new source development process (from the beginning). Fortunately, much of the same data can be used to support the various permit applications.

- **Step #1 – Conduct Groundwater Exploration Program**

  The Groundwater Exploration process begins with a desktop hydrogeological study of potential well sites utilizing existing information from the United States Geological Survey (USGS), MassDEP, and private consultant's work in or near areas under consideration.

  Following the desktop study, sites that the WDPW wishes to pursue further should be the subject of a limited field investigation to confirm the hydrogeologic suitability of the site for water supply development. In some cases, this process may begin with geophysical investigations to identify aquifer extents and other broad hydrogeologic characteristics.

  Next a relatively small-scale pumping test should be conducted to gain an initial assessment of aquifer and water quality characteristics and potential well yield before instigating the MassDEP Site Exam Process.

- **Step #2 – Submit Request for Site Exam**

  Once initial testing has shown a site likely to be suitable for the development of a public water supply, a request is made to invite the MassDEP to come and investigate the site suitability themselves. The Request for Site Exam is submitted as a report that summarizes all of the initial investigations and presents the case for why the subject site is considered suitable for public water supply. The Request for Site Exam must include:
  - A characterization of land use in a half-mile radius around the well;
  - A map showing current land uses, other existing private and public water withdrawals, zoning, and potential contamination sources;
  - An evaluation of potential impact to the proposed public water supply from contamination sources;
  - A boring and construction log for the test well at the site, an estimate of yield from that well, and water quality testing results;
  - Locations and boring logs for other exploratory wells;
  - A preliminary conceptual model of the aquifer including stratigraphic cross-sections, boundary conditions, and initial estimates of the Zones 2 and 3 areas;

- Description of any potential contamination sources in the estimated Zone 2 area;
- An initial estimate of the final production well proposed yield;
- Water Quality results obtained during initial test well testing;
- A wellhead protection plan including local contact persons, a plan for drafting needed regulatory and zoning controls, and a timeframe for achieving those controls; and
- A surveyed site plan showing the Zone 1, well locations, and elevations.

· **Step #3 – Conduct MassDEP Site Exam**

After the Request for Site Exam has been reviewed and accepted, the MassDEP will make a site visit. This visit will include:

- A land use/sanitary survey of the preliminary Zone 2 area;
- A discussion of proposed observation well locations and any special requirements for the forthcoming prolonged pumping test; and
- The identification of any potentially hydrologically connected surface water features.

To be approved for further testing after the Site Exam, the MassDEP must be satisfied that:

- The site is not at significant risk from floods or other disasters;
- The site will be readily accessible at all times;
- The site is not subject to undue short circuiting from surface waters;
- The site meets Zone 1 protection and ownership requirements; and
- The site is not located within one half mile of potentially serious sources of pollution.

· **Step #4 – Submit Prolonged Pumping Test Proposal**

Following a satisfactory review of the Request for Site Exam report and the Site Exam itself, MassDEP will provide written approval to proceed with the submittal of a Pumping Test Proposal. The Prolonged Pumping Test must be conducted at a pumping rate of at least half that of the requested permit rate for the final production wells. Specific guidelines for the number and placement of observation wells, the delivery of discharge water, water level monitoring criteria, water quality monitoring criteria, and flow monitoring must be followed and described in the proposal. Further guidelines resulting from the Site Exam may also need to be followed. A draft of proposed zoning and regulatory controls must also be submitted at this time, as well as a description of the status of other necessary permit applications and regulatory review.

- **Step #5 – Conduct Pumping Test**

  Once the Prolonged Pumping Test Proposal has been approved, the Prolonged Pumping Test and all associated monitoring will be conducted following the criteria outlined in the proposal and any other specific instructions received from MassDEP. Special monitoring requirements may be required to assess specific hydrologic or water quality questions at MassDEP discretion. The pumping test must proceed for a minimum of 5 consecutive days and onwards until no more than a half-inch fluctuation is observed at a proximal observation well over the final 24-hours of pumping. Recovery of the aquifer must be monitored until water levels have recovered to 95% of pre-test levels or until recovery time equals the total duration of pumping.

- **Step #6 – Submit Source Final Report**

  The final step in the NSA process is to submit a Source Final Report describing all of the pertinent information collected to date, the methods, analyses, and results of the Prolonged Pumping Test, a full description of the area hydrogeology, a final delineation of the Zones 2 and 3 for the proposed well, an analysis of water quality data, an analysis of potential hydraulic connections to surface waters, a discussion of the well's proposed period and rate of operation and expected groundwater impacts from that operation, a groundwater monitoring plan to protect the quality of water derived from the proposed well, and an approvable wellhead protection bylaw. Detailed numerical modeling will be required to adequately delineate the Zone 2 area for the proposed well. The 1997 MassDEP Zone 2 model should be utilized. The Source Final Report must also include a detailed discussion of the methods and results of the Zone 2 modeling effort.

Final NSA will not be granted until all other permitting and regulatory goals are achieved, ownership and control of the Zone 1 is adequately demonstrated, an approved wellhead protection bylaw is in place, and a groundwater monitoring program has been accepted.

- **Step #7 - MEPA ENF Submittal**

  An environmental notification form (ENF) submittal is required for any new withdrawal or expansion of withdrawal of 100,000 gallons per day or greater requiring new construction. The ENF is a relatively simple form and letter describing the proposed project, any potential impacts, and proposed mitigation. Following review of the ENF, the MEPA office may grant a MEPA certificate for the proposed project or request the submittal of an Environmental Impact Report (EIR) to provide a more detailed description of the proposed project and potential impacts. An EIR is mandatory for proposed groundwater withdrawals of 1,500,000 gallons per day or greater or the construction of 10 or more miles of water main. The issues considered by the MEPA office when evaluating an ENF for a new proposed water supply will include proximity to water resources and rare, water-dependent species habitat, potential interference with other withdrawals, and potential for water quality issues. The lower the potential for any of those issues to be significant, the less likely the MEPA office will be to require a full EIR. A successful review of the proposed new water supply source by the MEPA office is a prerequisite for the receipt of a WMA permit and a NSA permit.

- **Step #8 - WMA Permit Application**

  A WMA permit is required for any new withdrawal or expansion of withdrawal of 100,000 gallons per day or greater. Although similar and interlinked with the NSA process, the WMA permit is entirely focused on potential water quantity impacts to water resources and other, pre-existing water users. The water quality component, which figures prominently in the NSA process for drinking water supplies, is not part of the WMA permit. Much of the data required to satisfy WMA requirements that no significant drawdown or water quantity impacts are likely from the proposed new water supply source are the same as those needed for NSA analyses. However, the WMA requires that the data be used in a different way and submitted in a different format.

As with the MEPA permit process, the WMA process can be made simpler by minimizing the potential for any impacts to water resources, water-dependent, rare species habitat, and other water withdrawals. The effort to prove that no significant impacts are likely to occur from the proposed new water supply is made simpler if the new supply is located greater than 1,000 feet from any surface water resources and one half mile from other water withdrawals or potential contamination sources.

- **Step #9 – Submit Design Plan for Permanent Works**

  Once the MassDEP has granted NSA for the proposed water supply site, the site is permitted and approved for a specified withdrawal rate. The next step is to apply for and receive permits for the actual physical apparatus used to withdraw, treat, store, and transmit the water. The proponent submits detailed design drawings to MassDEP specifying exactly what will be built and how the construction will proceed. After MassDEP review and commentary, approval of the Permanent Works Plan allows construction of the proposed new water supply to proceed.

- **Step #10 – Construct Permanent Works for Water Supply**

  Once approval of the design documents has been granted, the project is advertised for public bids in accordance with State bidding law. Throughout construction, independent construction oversight must be provided by the applicant.

- **Step #11 – MassDEP Inspection of Permanent Works**

  Final MassDEP Inspection and approval of the constructed Permanent Works must occur before the new water supply source is allowed to operate. The inspection will include whether construction was completed in conformance with the approved plans, sanitary conditions, and other items pertinent to public safety.

4.3.3 Existing Sources

As discussed earlier in this report, the WDPW provides water to its customers from four active source locations consisting of six individual wells located within the Town of Ware. The four active sources are reported to have been installed as early as 1886 with the Cistern and more recently with the replacement wells for the Wellfield in 2016.

In general, well performance over time is influenced by many factors that can contribute to a steady and sometimes rapid decline in hydraulic performance. Well screen plugging and deterioration in yield can occur from encrustation and biofouling of the well screen surface, between the slot openings, gravel pack, and within the surrounding aquifer formation. In addition, the migration of silt, clay and fine sand over time can steadily decrease the soil pore space openings in the adjacent gravel pack and aquifer formation.

Well redevelopment entails the removal of the materials plugging the well screen via mechanical and chemical rehabilitation of the well and well screen. As most of the WDPW's well sources contain elevated concentrations of iron and manganese, loss of pumping capacity over time is common and well cleanings/redevelopments are routinely practiced.

Cleaning and redevelopment of each well is recommended when the specific capacity of the well drops no more than 10% from the last cleaning. Therefore, it's very important that the specific data be proactively tracked and recorded as it's possible that lost capacity may not be regained.

Although the exact method of cleaning and redevelopment varies for every source due to a variety of conditions (e.g., age, construction, screen type, water quality, surrounding formation, etc.), a comprehensive and routine well maintenance program should include the following:

- Prior to the well redevelopment process, a pre-cleaning pump test should be performed on each well utilizing the existing equipment to establish baseline performance data.
- After the initial performance test is completed, the pump equipment should be removed and the well televised for a record of its existing condition.

- After the removal of the pumping equipment, the well should be cleaned and redeveloped in accordance with the program that was specifically tailored for it. The traditional approaches used historically throughout New England may be suitable under certain circumstances. However, it is highly recommended that the technique selected avoid the use of any process which introduces a food source for bacteria growth (i.e., regrowth after cleaning).
- After the well is cleaned and redeveloped, the well should be televised again for a record of its rehabilitated condition and to identify any issues that were not visible prior to the first televised recording.
- Upon confirmation that all is acceptable from the second televised recording, a post-cleaning pump test should be performed on each well utilizing the existing equipment (cleaned and rehabilitated as necessary) to establish the new performance data.

In summary, the ultimate effectiveness of the chemical and/or mechanical cleaning is determined by the previously mentioned factors which resulted in the well's reduction in yield. The effectiveness of a well cleaning is also reduced when the well yield is allowed to decline for a longer period (i.e. increasing time between well cleanings). This often results in the inability of the well to regain its original construction hydraulic performance. Therefore, when significant well performance is lost and/or the cleaning frequency becomes too costly, a replacement well needs to be considered.

Although no sources are currently understood to be significantly under capacity, the WDPW should routinely clean and redevelop its existing sources to maintain its capacity.

4.4 SOURCE TREATMENT

As was presented within Section 2 of this report, the Wellfield, Well No. 4, and the Cistern are all chemically treated at the Pump House and the Dismal Swamp Well is chemically treated individually. At both locations, the water is treated with potassium hydroxide (KOH) for pH adjustment and sodium hypochlorite (NaOCl) for disinfection.

Historically, iron and manganese have been causing water quality problems and chronic consumer complaints in Ware. Both the iron and manganese concentrations have been exceeding their corresponding SMCL of 0.30 mg/L and 0.05 mg/L, respectively. It should be noted that the injection of NaOCl oxidizes any iron and manganese present in the water while the injection of KOH speeds up the process. This oxidation is what causes the minerals to become visible and cause consumer complaints. It should also be noted that even concentrations that are below their corresponding SMCLs will oxidize and slowly accumulate within the distribution system over time. These sediments will then be re-suspended during increased demands or with a flow reversal (e.g. use of hydrant) and cause dirty water complaints.

Additional information related to a regulatory review of these and other water quality constituents is presented later within Section 6 of this report.

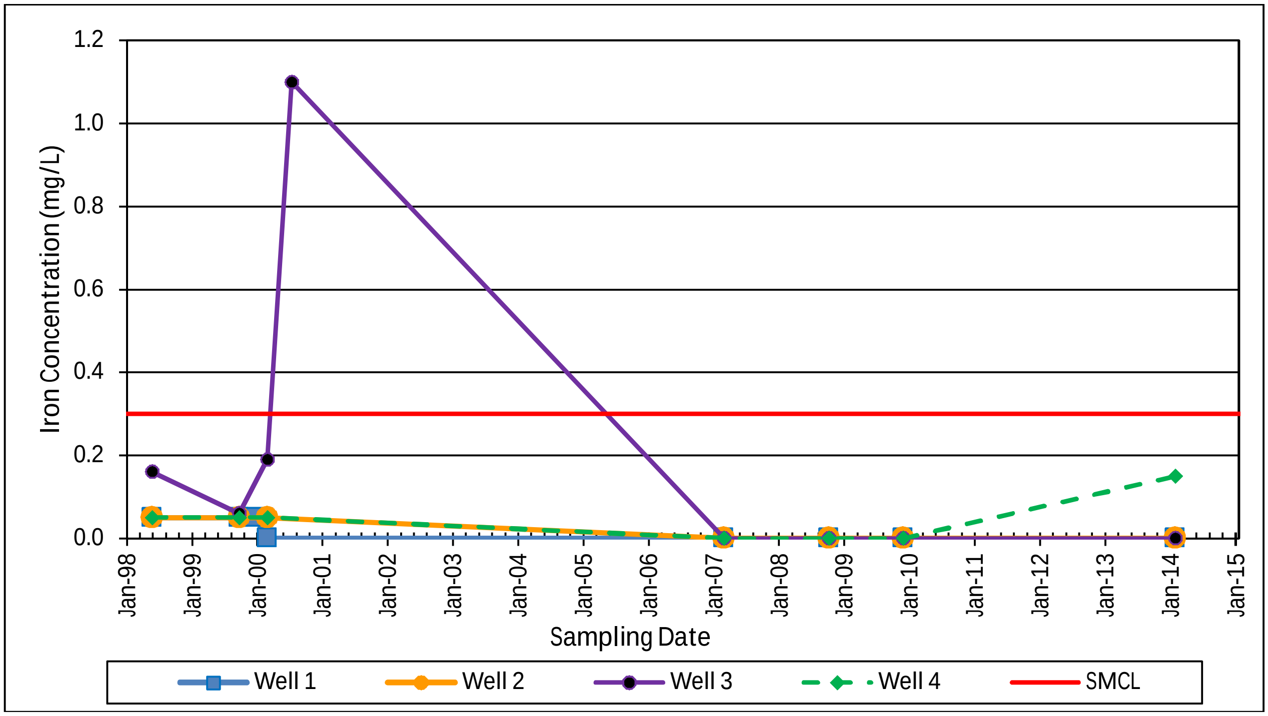
Figures 4-1 through 4-6 within this section present the available historic water quality data for iron and manganese from the Wellfield, Well No. 4, Cistern, and Dismal Swamp Well sources.

4.4.1 Barnes Street Sources

The Barnes Street Sources consist of Wells No. 1 through 4 and the Cistern. These sources are blended together at the Cistern and then chemically treated at the Pump House. Limited grab samples from each well source have been historically tested for iron and manganese.

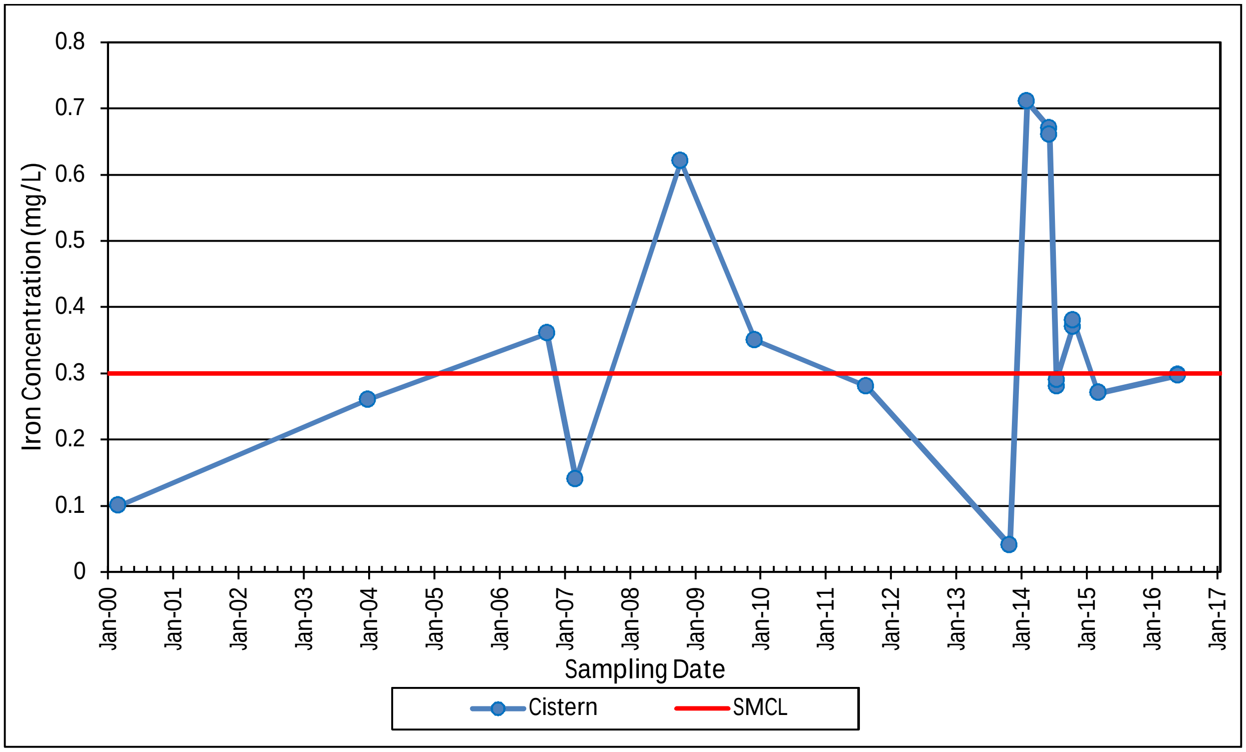
The available iron concentrations in Wells No. 1 through 4 between the years of 1998 and 2014 are presented in Figure 4-1.

FIGURE 4-1
IRON CONCENTRATIONS FOR WELLS NO. 1-4
WARE, MASSACHUSETTS



Besides the one exceedance of 1.1 mg/L for Well No. 3 in 2000, all of the wells (Wells No. 1 through 4) have been below the iron SMCL of 0.30 mg/L. Although the data is limited, the iron concentrations in Well No. 4 may be increasing. The blended iron concentrations at the Cistern are presented in Figure 4-2.

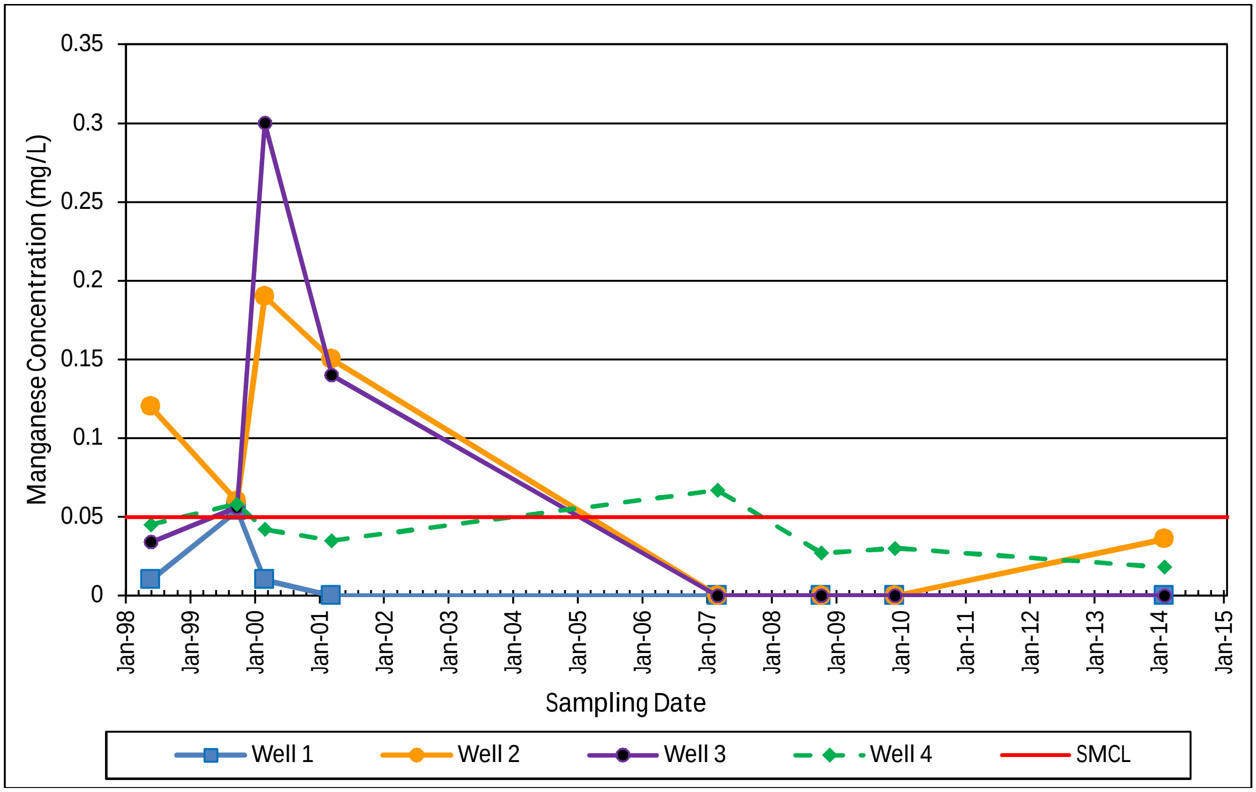
FIGURE 4-2
IRON CONCENTRATIONS FOR THE CISTERN
WARE, MASSACHUSETTS



Based on historic water quality since 2000, Wells No. 1 through 4 have not shown iron concentrations above the SMCL of 0.3 mg/L while the Cistern has had several exceedances since 2006. Therefore, it can be determined that the Cistern source is likely the cause of the iron exceedances during those pumping conditions. Furthermore, it can be assumed that the Cistern has a higher concentration than what is presented in the results (Figure 4-2) since all the sources were likely blended during sampling. These concentrations of iron (ranging to 2+ times the SMCL) will contribute to consumer complaints about “dirty water”.

The manganese concentrations in Wells No. 1 through 4 between the years of 1998 and 2014 are presented in Figure 4-3.

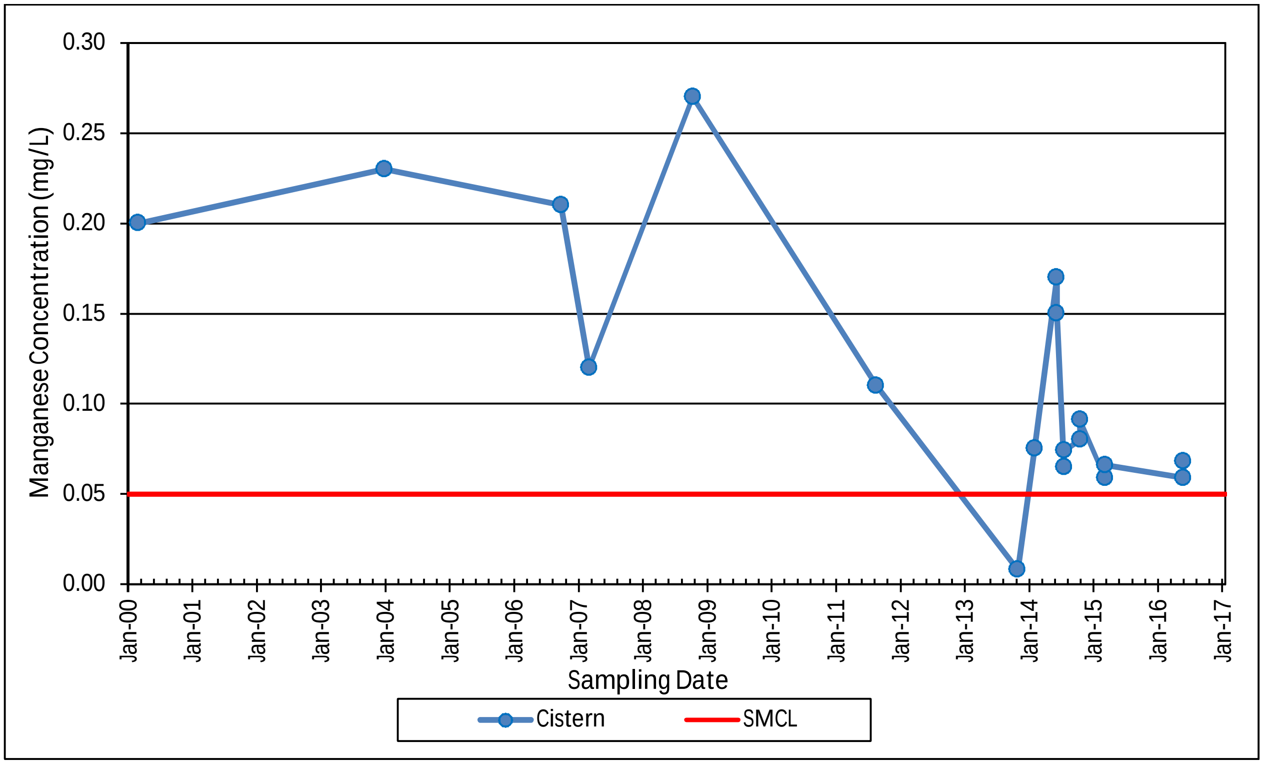
FIGURE 4-3
MANGANESE CONCENTRATIONS FOR WELLS NO. 1-4
WARE, MASSACHUSETTS



There have been several manganese exceedances from the four wells between 1998 and 2007. Well No. 1 has had one exceedance of 0.054 mg/L in 1999. Well No. 2 has had four exceedances (0.12 mg/L, 0.06 mg/L, 0.19 mg/L, and 0.15 mg/L) between 1998 and 2001. Well No. 3 has had three exceedances (0.056 mg/L, 0.30 mg/L, and 0.14 mg/L) between 1999 and 2001. Well No. 4 has had a total of two exceedances; 0.058 mg/L in 1999 and 0.067 mg/L in 2008. Since 2007, all of the manganese concentrations have been below the SMCL of 0.05 mg/L. However, Well No. 2 may be increasing based on the most recent data (2014).

The blended manganese concentration from the Cistern source is presented in Figure 4-4.

FIGURE 4-4
MANGANESE CONCENTRATIONS FOR THE CISTERN SOURCE
WARE, MASSACHUSETTS



Similarly, the concentration of manganese at the Cistern appears to also have come down recently but the concentrations are still above the manganese Secondary Maximum Contaminant Limit (SMCL) of 0.05 mg/L. The blended source has consistently had manganese concentrations well above the corresponding SMCL of 0.05 mg/L (ranging to approximately 5+ times the SMCL) from 2000 to 2016.

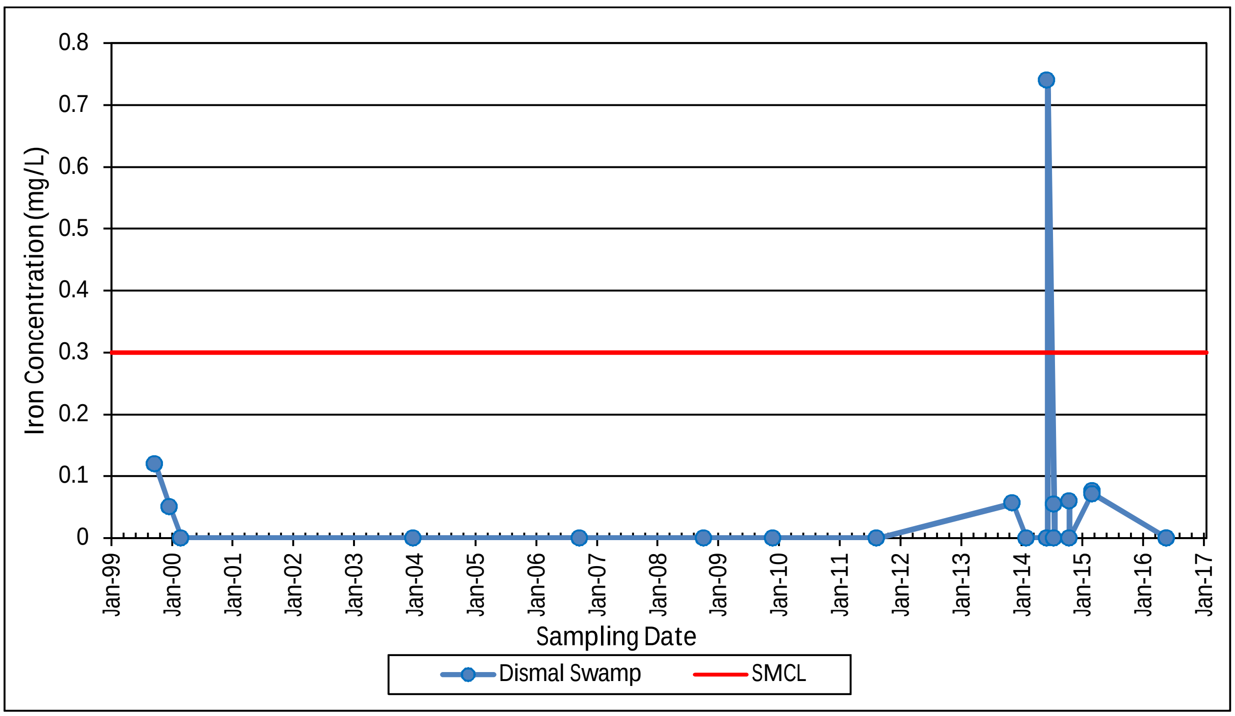
Although the manganese concentrations from Wells No. 1 through 4 have been below the SMCL since 2007, the Cistern's combined concentrations were still exceeding the SMCL. Therefore, it can be determined that the Cistern source is contributing to the elevated concentrations of manganese. These concentrations of manganese will also contribute to any water quality complaints.

4.4.2 Dismal Swamp Well Source

The Dismal Swamp Well source is being treated at the Gibertville Road Pump Station with potassium hydroxide (KOH) for pH adjustment and sodium hypochlorite (NaOCl) for disinfection. It is understood that the NaOCl feed system is not being used due to elevated manganese concentrations in the raw water. The WDPW will reactivate system with chlorination in the future or as required by MassDEP.

The historic iron concentrations for the Dismal Swamp Well source from 1999 to 2016 are presented in Figure 4-5.

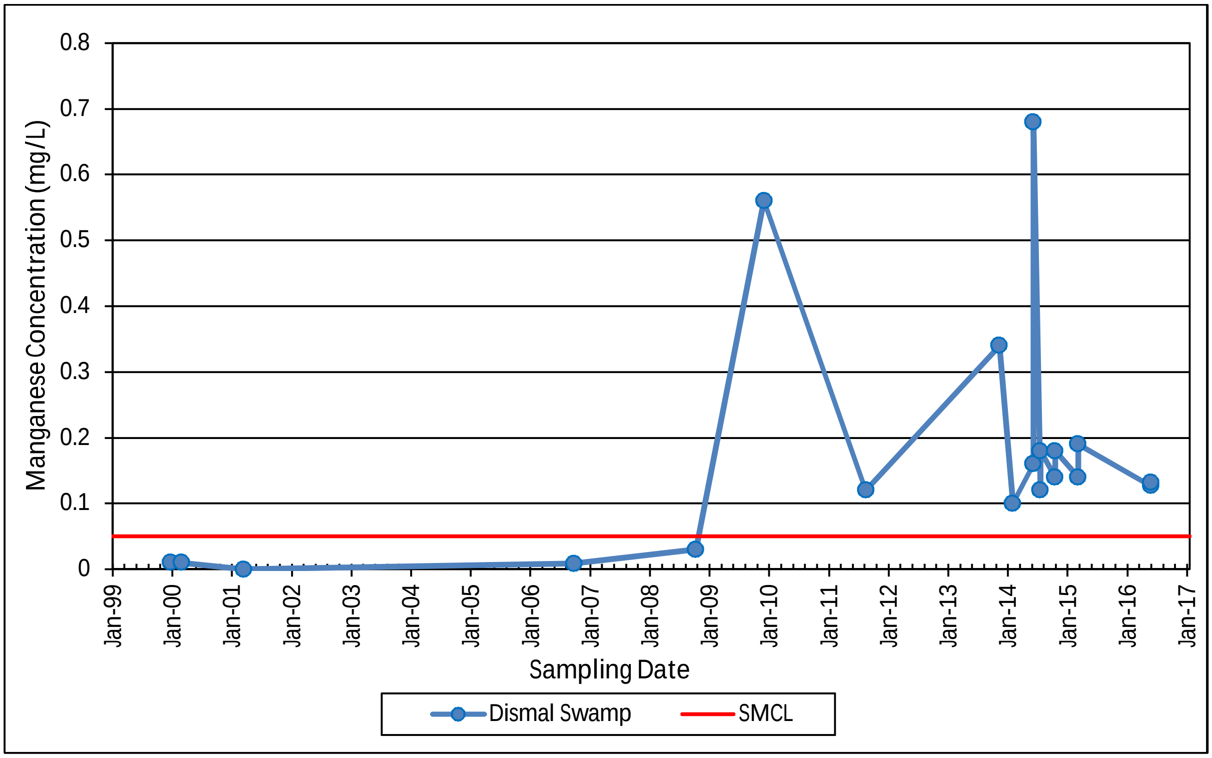
FIGURE 4-5
IRON CONCENTRATIONS FOR DISMAL SWAMP WELL SOURCE
WARE, MASSACHUSETTS



As can be seen from the data, the iron concentrations for the Dismal Swamp Well has been primarily below the corresponding SMCL with the exception of the one exceedance of 0.74 mg/L on June 16, 2014.

The historic manganese concentrations from 1999 to 2016 are presented in Figure 4-6.

FIGURE 4-6
MANGANESE CONCENTRATIONS FOR DISMAL SWAMP WELL SOURCE
WARE, MASSACHUSETTS



All of the sampling data since 2008 has been above the corresponding SMCL at the Dismal Swamp Well. Three of the data points were also above the ORSGL established by the MassDEP (further discussed in Section 6). The "Barnes Street Water Quality Evaluation" by Tata & Howard on April 18, 2012 evaluated potential causes for these exceedances since 2008. The report compared the concentrations and pumping rate to see if increased pumping from the well would contribute to increased concentrations and determined that there was not a direct correlation. The report noted that a check valve at the pump station was not working correctly when they were writing the report and suspected that it may have contributed to the manganese exceedances since distribution water could have flowed back into the well. Another factor that was considered for the high concentrations was potential changes in the aquifer and natural groundwater fluctuations.

4.4.3 Treatment Options

Due to chronic water quality issues, Wright-Pierce and WDPW acquired grant funding from USDA and supplemented it with Town funds to study the implementation of treatment at its sources. Based on information presented, four options are available.

4.4.3.1 Option No. 1

The first option is to only treat the Dismal Swamp Well source. This source has historically had the highest exceedances of manganese out of the six sources in Town. If treated, the Dismal Swamp Well would be capable of providing up to 0.583 MGD of treated water. Since the Town is projected to need 0.79 MGD for average daily demand (ADD) and 1.37 MGD for maximum daily demand (MDD) by 2025, this source would not be able to meet either of these demands. Water from the other sources would need to be utilized. Since the other sources would not be treated under this option, then the Town would still have iron and manganese issues.

4.4.3.2 Option No. 1a

As previously discussed, the Cistern source is suspected to be the cause for the elevated iron and manganese concentrations. Therefore, Option No. 1a would be similar to Option No. 1 but would also include the removal of the Cistern source. This option would treat the water at the Dismal Swamp Well source and then only utilize Wells No. 1 through 4 to fulfill demand. The combined approved maximum daily rate would be 1.67 MGD which would meet ADD and MDD.

Within the past several years, Wells No. 1 through 4 have had low concentrations of iron and manganese (below their corresponding SMCLs). In the future it is possible for these concentrations to increase due to increased pumping without the use of the Cistern. Eventually these sources may also need to be treated.

4.4.3.3 Option No. 2

Option No. 2 is to treat all of the Town's sources. Since the Dismal Swamp Well is not in a close proximity to the Barnes Street sources, it would have to be treated separately. Therefore, the installation of two water treatment plants would be required. A total combined maximum daily rate of 2.383 MGD could be provided to the Town from all of the sources. Since the ADD and MDD that would be needed by 2025 is only 0.79 MGD and 1.37 MGD, respectively, the total rate of 2.383 MGD would not be necessary and likely too costly.

4.4.3.4 Option No. 3

The last option is to only treat the water from the Barnes Street sources. The Barnes Street sources consist of the Wellfield (Wells No. 1, 2, and 3), Well No. 4, and the Cistern which when combined could supply the Town with an approved maximum daily rate of 1.80 MGD. This would meet the projected ADD and MDD of 0.79 MGD and 1.37 MGD, respectively. The Dismal Swamp Well source then could be used as a back-up source in case of an emergency. The Barnes Street sources are located in close proximity to each other and already are pumped through a common point with available land nearby and is in close proximity to the sewer system. Treatment could easily be provided at this common point.

Option No. 3 is overall the best plausible option. The Barnes Street sources alone can meet the Town's water demand needs and should the system grow significantly, treatment can be added at the Dismal Swamp Well (if needed). Therefore, Wright-Pierce recommends that the WDPW proceed with Option No. 3.

4.5 MISCELLANEOUS SUPPLY ISSUES AND RECOMMENDATIONS

The following section presents some other supply related issues that should be noted.

4.5.1 Emergency Power Provisions

Having appropriate emergency power provisions to maintain an adequate supply capacity during a loss of power event is an important consideration for water suppliers. The following is an excerpt from MassDEP's Guidelines and Policies about required emergency (standby) power provisions for water suppliers:

"*Standby power is required at all water treatment facilities and other facilities as may be required by MassDEP, unless it can be demonstrated that the facility has the ability to provide the maximum daily demand for up to 24 hours by other means. This may include the combined ability of other sources to provide the maximum daily demand through existing or new emergency power generation at those sources, from storage tanks, or through a viable interconnection with another public water supplier that is part of an emergency plan approved by MassDEP.*"

As was previously presented within Section 2 of this report, the WDPW has emergency power provisions only installed at the Pump House. This generator provides emergency power for the Pump House, the Wellfield, and one of the high lift pumps in the Cistern. With emergency power at this location, the WDPW has the capability to provide 1.08 MGD when utilizing the Cistern and Wellfield sources. The WDPW is also has an additional usable volume of 0.43 MG from their two water storage tanks (as calculated in detail within Section 5.5). Combined, these would add up to 1.51 MGD. The projected MDD for 2025 is 1.37 MGD; therefore, the WDPW currently has adequate provisions for emergency power according to the MassDEP requirements presented for the ability to provide the maximum daily demand for up to 24 hours.

There are no emergency power provisions provided at any other of the locations. Should the WDPW desire to have full emergency power provisions, suitable generators would need to be installed at all of its other source locations. These locations include the Dismal Swamp Well, Well No. 4, and the Booster Pump Station at the Church Street Tank. It is noted that emergency power provisions shall also be incorporated into the future Barnes Street WTP.

Section 5

Wright-Pierce
Engineering a Better Environment™

SECTION 5

DISTRIBUTION SYSTEM EVALUATION AND ASSESSMENT

5.1 PURPOSE/SCOPE OF SYSTEM ANALYSIS

The purpose of the distribution system analysis is to assess the hydraulic adequacy of the Ware Department of Public Works' (WDPW) pumping and storage facilities, transmission mains, and distribution piping and its ability to satisfy both existing and projected demand conditions. The scope of the evaluation will be focused on the following:

A. Distribution System Hydraulics
   - Maximum and Minimum System Pressures
   - Adequate Fire Flows
   - Reliable Pipe Looping and Redundancy, Pipe Velocities and Pipe Sizing
   - Interconnections to Adjacent Utilities

B. Storage Analysis
   - Adequate Storage Volume
   - Location of Storage
   - Storage Redundancy
   - Adequate Emergency, Fire Storage and Peak-Hour Storage Volumes

Water systems are analyzed, planned and designed primarily through the application of basic hydraulic principles. The existing computer hydraulic model developed in 2012 by another consultant was supplied to Wright-Pierce by the Town to be used as the hydraulic tool for analyzing the condition of the Ware water system under existing and projected demands. The evaluation was based on compliance with Commonwealth of Massachusetts code requirements and standard engineering practice. A variety of options were considered as part of this Study. Specific recommendations are discussed in this section and summarized with cost estimates in Section 8.

5.2 DISTRIBUTION SYSTEM COMPUTER MODEL

A computerized hydraulic model of the Ware water distribution system was developed in 2012 by a previous consultant for the WDPW. Wright-Pierce (WP) was supplied with this model for the analysis and it is understood to have been previously calibrated. The model was originally developed using the InfoWater hydraulic modeling software as manufactured by Innovyze and was also used as the software modeling tool for this Master Plan. The element features or attributes assigned to the water system utilities included: pipe material, pipe diameter, pipe friction coefficient (Hazen-Williams C-Value), storage tank operating elevations, pump and tank level controls, and water system pump operation parameters.

5.2.1 Stress Conditions

Several stress conditions are run in order to evaluate the adequacy of the system to meet existing and projected demand conditions. This is done by simulating the following two demand conditions, using the computer hydraulic model:

- <u>Peak Hour on Maximum Day in the Year 2025</u>
  Under peak-hour conditions, a water system is considered adequate if a minimum pressure of 35 pounds per square inch (psi) can be provided to the entire service area.
- <u>Maximum Day in the Year 2025 Plus Various Fire Flow Requirements</u>
  Under maximum-day *plus* fire flow demand conditions, a system must be capable of providing the needed fire flow during maximum-day demands, while maintaining a minimum residual pressure of 20 psi coincidental throughout the distribution system.

Each of these conditions are evaluated under varying demands, and where the system does not meet the criteria set forth, alternative improvements are modeled and recommendations are made based on the hydraulic and cost effectiveness of the improvements.

5.3 WATER SYSTEM CHARACTERISTICS AND ADEQUACY

The approach used to evaluate the Ware distribution system was to first, identify the hydraulic requirements of the system, and secondly to identify the adequacy and limitations of the system under the existing and projected demand conditions.

Several factors are normally considered in the evaluation of the adequacy of a water distribution system. These include: system pressures, velocity of water in the pipelines, headloss, pipe looping, redundancy, piping reliability and adequacy, and future fire flow capabilities. Following is a discussion of each of these factors, as well as how they apply to both existing and projected demand conditions.

The following discussion presents the findings from the analysis and offers various options for resolving deficiencies.

5.3.1 Piping Validation

It is critical that actual details of the subsurface piping network be clearly understood in order to validate the necessity of improvements. The hydraulic model and system piping configuration was obtained from the existing hydraulic model provided by the Ware Department of Public Works. The piping network within the model is understood to be current.

5.3.2 Water System Pressure

A water system should be designed to accommodate a range of pressures within minimum and maximum guidelines (40 to 80 psi). Low system pressures result in customer complaints, may affect the accuracy of meters, and will restrict available flow for firefighting. Higher pressures can contribute to increased water loss from leakage (i.e., unaccounted-for water), can increase maintenance on equipment, lead to higher energy costs, and tend to increase consumption.

Approximately 64 percent of Ware's water system has static pressures between 80 and 120 psi, and approximately 33 percent of nodes have static pressures between 40 and 80 psi. The remaining

3% is below 40 psi. Figure 5-1 represents a color coded static pressure node map for various pressure ranges. As shown in the figure, the system is predominantly made up of pressures between 80 to 120 psi. There are only a few nodes that are less than 20 psi and these are located adjacent to the Church Street Tank. It is understood that these residential services in the immediate vicinity of Church Street Tank are on a small local boosted system.

Variations in customer demand, changes in elevation and proximity to pumping facilities and sources of supply will cause water pressure to vary throughout the service area. In general, when customer demands increase, pressure will decrease. Areas with higher elevations typically have lower pressures.

Massachusetts Guidelines for Public Water Systems states that normal working pressure in the distribution system should be approximately 60 to 80 psi and not less than 35 psi. Standard water works practice generally allows a normal maximum system pressure of 80 to 100 psi. State Plumbing Code requires that household pressures must be lower than 100 psi. This can be achieved locally and is not a municipal requirement. Pressures throughout the system during fire flow events should be maintained above 20 psi at all locations. Services in areas where pressures exceed 80 psi should be considered for installation of pressure reducing valves.

5.3.3 Pipe Velocities and Head Loss

Water velocities in pipelines can have either a positive or negative impact on operations and water quality throughout the system. Pipes with velocities that exceed 5 feet per second (fps) contribute to increased headloss which in turn requires pumps to work harder and energy costs to increase. Higher velocities can also scour the interior of the pipe, which reduces its useful life. High velocities are common in smaller diameter piping. On the other hand, pipes having velocities below 2 fps present a risk of depositing sediment which could contribute to poor water quality and poor hydraulics. Generally, velocities in the system under all existing and future conditions were found to be adequate. The transmission mains from the Barnes Street sources (via the Cistern) will also experience velocities between approximately 2 to 3 fps depending on number of wells in operation.


Data from MassGIS and Town of Ware
HARDWICK
W. BROOKFIELD
PALMER
Church St. Tank
Capacity: 1.5 MG
Overflow: 659 FT
Barnes Street Sources
(Wells 1,2,3,4, & Cistern)
Anderson Rd Tank
Capacity: 1.0 MG
Overflow: 659 FT
Legend
Junction
PRESSURE
less than 20 PSI
20 - 40 PSI
40 - 80 PSI
80 - 120 PSI
Tank
Well Source
Pipe
Diameter
less than 4"
6"
8"
10"
12"
N
0 250 500 1,000 1,500 2,000
Feet
Sources: Esri, HERE, DeLorme, USGS, Intermap, increment P Corp., NRCAN, Esri Japan, METI, Esri China (Hong Kong), Esri (Thailand).
Static Pressure Node Map
Ware, MA
PROJ NO: 13471A
DATE: 10/18/2016
FIGURE: 5-1
WRIGHT-PIERCE
Engineering a Better Environment

5.3.4 Dead-End Mains and Pipe Looping

Dead-end mains in a water system present a number of operational issues. First, because water cannot pass through a dead-ended pipe, velocities in these pipes tend to be very low. This condition can cause sediment build-up and contributes to poor water quality. In winter months, pipes having low velocities can be prone to freezing. Generally, the only way to improve this condition is to regularly flush the ends of these pipes, add bleeders, or loop the pipe into another location in the distribution system.

Flushing can be labor intensive and if not done on a regular basis, will have little effect in improving conditions. Bleeders, can be effective in improving water quality and help prevent freezing. But this method increases the unaccounted-for water component and electrical pumping costs. Looping requires capital investment in new piping. In some cases it may not be practical to loop pipes.

Measurable improvements in water quality, pressure and flow characteristics can be made by eliminating dead-ends. Not only would pipe looping improve hydraulics, it would also provide redundancy to the system. The WDPW distribution system is generally well looped, with the majority of the dead ends being 6-inch diameter water mains located on side streets. The longest dead-end in the system is a stretch of approximately 14,000 linear feet of 8-inch water main that runs north on Greenwich Road to the Hardwick Town line. Due to the isolated location of this water main relative to adjacent mains, no opportunities for looping this dead-end are available at this time.

5.3.5 Fire Flow

The ability to provide fire protection is a valuable asset for a community. Guidelines for fire flow requirements are provided by the Insurance Services Office (ISO). ISO is an insurance organization responsible for evaluating and classifying communities for insurance rating purposes. Periodically, the ISO will visit a community, perform fire flow tests and develop a fire insurance rate for that community. The rate assigned ranges from 1 to 10 with 1 being the best rating. The rating is based on the total firefighting capability of the community including such factors as water supply, fire department structure and available communication systems.

Specific fire protection requirements at a given locale vary with the physical characteristics of a building. ISO assigns a required fire flow based on the worst case premise in a general location using the following factors: (1) materials of construction, (2) its occupancy use, (3) proximity to other structures, (4) height and size of building, (5) the existence of fire walls, (6) presence or absence of sprinklers, as well as others. Some special use buildings may have required fire flow as high as 12,000 gallons per minute (gpm). Table 5-1 presents typical fire flow requirements for various building types and uses.

TABLE 5-1
TYPICAL FIRE FLOW REQUIREMENTS

Land-Use or Building Type	Range of Required Fire Flows and Flow Duration
SINGLE AND TWO FAMILY DWELLINGS	
Over 100 feet Building Separation	500 gpm for 2 hours
31 to 100 feet Building Separation	700 gpm for 2 hours
11 to 30 feet Building Separation	1,000 gpm for 2 hours
10 feet or less Building Separation	1,500 gpm for 2 hours
MULTIPLE FAMILY RESIDENTIAL COMPLEXES	2,000 to 3,000 gpm for 2-3 hours
AVERAGE DENSITY COMMERCIAL	1,500 to 2,500 gpm for 2-3 hours
HIGH VALUE COMMERCIAL	2,500 to 3,500 gpm for 2-3 hours
LIGHT INDUSTRIAL	2,000 to 3,500 gpm for 2-3 hours
HEAVY INDUSTRIAL	2,500 to 3,500 gpm for 2-3 hours

Municipal fire insurance ratings are partially based on a water utility's ability to provide needed fire flows up to a maximum flow of 3,500 gpm. The ISO requirement of 3,500 gpm is the criteria used for all non-residential land uses. This is the largest fire flow that the ISO recognizes as necessary for a system to provide even if a specific building within the community requires a greater fire flow. Many areas in Ware are considered to have fire flow requirements of 3,500 gpm.

The Ware public water system is predominately comprised of residential customers (91%). However, there are many locations throughout the system where the ISO requirement is 3,000 gpm or greater. The basis of our analysis considers the latest available ISO hydrant flow requirements and testing data completed in 2015. Table 5-2 lists the results of the model simulations of the available fire flows coincident with the projected year 2025 maximum-day demand for ISO locations throughout the service area.

The estimated available fire flows shown in Table 5-2 differ from the ISO field testing results completed in 2015 because of varying pumping rates, system demands and tank elevations during the testing period along with system pressure constraints used for the analysis. The available fire flows presented are based on maintaining a minimum 20 psi residual in all areas of the distribution system. The three locations adjacent to Church Street Tank that are boosted were not factored into the analysis. Normal field testing procedures do not take into account pressures in the distribution system other than at a test hydrant, which typically result in higher estimated available fire flow.

It should be noted that Table 5-2 presents a second set of estimated available fire flows which excludes an additional two nodes on Upper Church Street in proximity to the storage tank with elevations over 600 feet. These nodes are not understood to be boosted and due to their elevation were found to be the critical node in the majority of the fire flow simulations. The critical node being the first node in the system to drop to 20 psi during the simulation. By excluding these nodes from the analysis, the estimated available fire flow would represent a system with these nodes incorporated into the boosted area near the tank.

A discussion of piping replacement options to improve fire flows in deficient areas of the system follows.

TABLE 5-2
AVAILABLE FIRE FLOWS AT 2015 ISO TEST LOCATIONS
PROJECTED 2025 MAXIMUM-DAY DEMANDS

Test No.	Land-Use Description	Test Location	Available Fire Flow (gpm) Year 2015[1]	Estimated Available Fire Flow[2] (gpm) 2025	Estimated Available Fire Flow[3] (gpm) (Excluding >600' Elevation) 2025	ISO Required Fire Flow (gpm)[4]	Adequate (Yes/No)
1	Commercial	Palmer Road at Belchertown Road	1,800	1,010	1,625	1,500	**No**
2	Residential	Belchertown Road at Greenwich Plains Road	1,800	1,840	1,840	500	Yes
3	Commercial	R Palmer Road at Gould Road	2,500	900	1,960	3,000	**No**
4	Commercial	West Street at HomeCrest Avenue	2,500	810	1,770	3,500	**No**
5	Commercial	Warebrook Drive at Eagle Street	2,000	790	1,460	2,250	**No**
6	Commercial	Cresent Street at Greenwood Road	1,700	780	1,180	3,500	**No**
7	Commercial	Convent Hill Road at North Street	2,000	760	1,850	2,250	**No**
8	Commercial	Church Street at Park Street	2,500	770	1,770	3,000	**No**
9	Commercial	E. Main Street at Canal Street	2,300	775	1,745	2,000	**No**
10	Commercial	71 South Street	2,300	785	1,750	2,250	**No**
11	Commercial	Mechanic Street at Desmond Avenue	1,800	780	1,690	3,000	**No**
12	Commercial	East Street at Ross Avenue	2,300	775	1,750	1,750	**No**
13	Residential	Greenwich Road at Lee Road	600	670	670	500	Yes
14	Commercial	Gilbertville Road at East Street	1,000	780	1,000	3,000	**No**

[1] Available Flows per reported 2015 ISO Hydrant Test Data does not consider maintaining 20 psi residual system pressure.
[2] Estimated available fire flows based on tank levels 2 feet down from overflow and well supply pumping off, minimum system pressure of 20 psi (excluding boosted nodes around Church Street Tank).
[3] Estimated fire flows assume an expanded boosted zone around Church Street Tank which excludes all nodes above 600 feet from the analysis.
[4] Flows greater than 3,500 gpm are not considered in evaluating system compliance with ISO fire suppression rate schedule.

5.3.5.1 Fire Flow Deficiencies

In general, Ware has adequate hydraulic capacity to meet its residential fire flow demand requirements, however there are numerous areas where commercial fire flows are inadequate. Table 5-2 displays a total of ten inadequate fire flow areas under current maximum day demand conditions.

Figure 5-2 displays each ISO node within the system and whether it has adequate available fire flow to meet the required ISO demand assuming a minimum system pressure constraint of 20 psi. The AFF run was based on the existing system infrastructure utilizing current projected 2025 Maximum Day Demands. The status of all well supplies is off, and storage tank levels were set to 2 feet below overflow elevation (overflow elevation: 659 feet). This elevation most accurately represents the operating zone of the storage tanks based on existing information. The following sections discuss options that have been considered to resolve the apparent fire flow deficiencies.

<u>Residential Fire Flow</u>

Of the 14 ISO test locations, only two are classified as residential. The first location (ISO #2) is located at the intersection of Belchertown Road and Greenwich Plains Road, while the second location (ISO #13) is located at the intersection of Greenwich Road and Lee Road. The results of the hydraulic simulation estimated adequate fire flow available at both locations to meet the 500 gpm requirement as shown on Table 5-2.

<u>Commercial Fire Flow</u>

The remaining 12 ISO test locations are all categorized as commercial with required fire flow demands ranging from 1,500 gpm to 3,500 gpm. Of the 12 commercial ISO test locations evaluated, all identified as having inadequate fire flow based on the hydraulic analysis. A discussion of each deficient ISO location along with potential improvements follows.

Data from MassGIS and Town of Ware
HARDWICK
PALMER
W. BROOKFIELD
ISO No. 1
ISO No. 2
ISO No. 3
ISO No. 4
ISO No. 5
ISO No. 6
ISO No. 7
ISO No. 8
ISO No. 9
ISO No. 10
ISO No. 11
ISO No. 12
ISO No. 13
ISO No. 14
Anderson Rd Tank
Capacity: 1.0 MG
Overflow: 659 FT
Church St. Tank
Capacity: 1.5 MG
Overflow: 659 FT
Barnes Street Sources
(Wells 1,2,3,4, & Cistern)
Legend
Node
Does Not Meet ISO Demand
Meets ISO Demand
Tank
Well Source
Pipe
Diameter
less than 4"
6"
8"
10"
12"
N
0 250 500 1,000 1,500 2,000
Feet
ISO Test Locations
Ware, MA
PROJ NO: 13471A
DATE: 10/18/2016
FIGURE: 5-2
WRIGHT-PIERCE
Engineering a Better Environment
Sources: Esri, HERE, DeLorme, USGS, Intermap, increment P Corp., NRCAN, Esri Japan, METI, Esri China (Hong Kong), Esri (Thailand).

Palmer Road at Belchertown Road (ISO #1)

Fire flows at this location were found to be deficient by approximately 490 gpm. This section of Palmer Road is located on the west side of the distribution system and is currently served by a 12-inch ductile iron water main with a parallel 6-inch diameter cast iron water main. Fire flow is limited at this location due the low system pressures at the high elevations around Church Street Tank that are not currently boosted. When running the fire flow simulation with an increased boosted zone on Upper Church Street to include all nodes with elevations 600 feet or higher, the ISO fire flow demand is met. Therefore, expanding the boosted zone adjacent to Church Street Tank to include the additional high elevation homes on Upper Church Road is the recommended improvement to address this fire flow deficiency.

R Palmer Road at Gould Road (ISO #3)

Fire flows at this location were found to be deficient by approximately 2,100 gpm. This section of Palmer Road is currently served by a 12-inch asbestos cement main and a parallel 6-inch cast iron main. Fire flow is limited at this location due the low system pressures at the high elevations around Church Street Tank during the simulation. When removing the system pressure constraint of 20 psi from the simulation, there is adequate available fire flow to meet the ISO demand of 3,000 gpm, which would indicate that hydraulic restrictions are not the limiting factor but that elevational restrictions are. When running the fire flow simulation with the expanded boosted zone improvement on Upper Church Street, the available fire flow increases to approximately 1,960 gpm, however it is still deficient by approximately 1,000 gpm. The Anderson Road Tank is located just over a mile away from this ISO location, with all 12-inch diameter water main along the route. Elevational restrictions in the system can be improved by installing booster stations or increasing the hydraulic grade line of the system (i.e. raising tanks). Although expanding the boosted pressure zone to include the houses on Upper Church Street will not increase flow enough to meet the required ISO demand, this improvement is still recommended based on the increase of 1,000 gpm in additional fire flow it provides.

West Street at HomeCrest Avenue (ISO #4)

Fire flows at this location were found be deficient by approximately 2,690 gpm. This ISO location is approximately 2,500 linear feet east of ISO #3 where Palmer Road transitions to West Street.

Similar to ISO #3, this section of West Street is served by a 12-inch diameter asbestos concrete main with a parallel 6-inch cast iron main. Available fire flow is limited at this location due to low system pressures at the high elevations around Church Street Tank. When removing the system pressure constraint of 20 psi from the simulation, there is adequate available fire flow to meet the ISO demand of 3,500 gpm, which would indicate that hydraulic restrictions are not the limiting factor but that elevational restrictions are. Similar to ISO #3, when running the fire flow simulation with the increased boosted zone improvement on Upper Church Street, the available fire flow increases; however, it is still deficient by approximately 1,730 gpm. For similar reasons noted for ISO #3, the expansion of the boosted zone to include Upper Church Street is recommended.

Warebrook Drive at Eagle Street (ISO #5)

Fire flows at this location were found to be deficient by approximately 1,460 gpm. This location is located on the west side of the distribution system and is currently served by a 12-inch ductile iron main off W. Main Street from the south. An 8-inch ductile iron main on Eagle Street feeds this location from the north (i.e. looped). Fire flow is limited at this location due the low system pressures at the high elevations around Church Street Tank during the simulation. When removing the system pressure constraint of 20 psi from the simulation, there is adequate available fire flow to meet the ISO demand, which would indicate that hydraulic restrictions are not the limiting factor but rather elevational. Similar to previous ISO locations, available fire flow increases by approximately 670 gpm when running the fire flow simulation with the expanded boosted zone on Upper Church Street; however, it still does not meet the ISO required demand of 2,250 gpm. Although the ISO fire flow demand cannot be met, the expansion of the boosted zone is still recommended for similar reasons noted in previous locations.

Cresent Street at Greenwood Road (ISO #6)

Fire flows at this location were found to be deficient by approximately 2,720 gpm. This ISO location is fed by an 8-inch asbestos cement water main from Pleasant Street and is located just north of the Barnes Street well sources. This ISO location is also looped via Eagle Street by an 8-inch water main. When removing the system pressure constraint of 20 psi from the simulation, there is adequate available fire flow to meet the ISO demand at this location. When running the fire flow simulation with the expanded boosted zone on Upper Church Street, the available fire flow increased by approximately 400 gpm, which is still approximately 2,320 gpm less than the 3,500 gpm required by ISO. Unlike previous ISO locations which were located on existing 12-inch diameter mains, this location is fed from 8-inch diameter mains. A second simulation was run after increasing the diameter of the existing 8-inch mains on Pleasant Street and Eagle Street to 12 inches in diameter along with the expanded boosted zone. This improvement would create a loop of 12-inch pipe feeding the ISO location. The results of this simulation found an increase in available fire flow of approximately 300 gpm, which is still less than the required 3,500 gpm at this location. Given the minimal benefit at this location of expanding the boosted zone or increasing the pipe diameters, we do not recommend any improvements for this location.

Convent Hill Road at North Street (ISO #7)

Fire flows at this location was found to be deficient by approximately 1,490 gpm. North Street is served by a 12-inch ductile iron pipe. The fire flow location is looped through 12-inch diameter water mains on Highland Village to the north and Walnut Street to the south. These two mains are fed from another 12-inch diameter ductile iron main on Church Street which creates a loop. The Church Street Tank is located just north of the fire flow location. When running the fire flow simulation with the expanded boosted zone on Upper Church Street, the available fire flow increases by approximately 1,000 gpm, which reduces the deficit to approximately 400 gpm. The model indicated higher head-loss though the existing sections of 10-inch and 12-inch cast iron main that run along Church Street from Pleasant Street to the tank. The low C-factor (60) which has been assigned to this stretch of cast iron pipe would indicate that the piping may be restricted due to heavy tuberculation over time or potentially a partially closed valve; however, the installation date is unknown. A second simulation was run after installing new piping on Church Street from Pleasant Street to the storage tank (approximately 3,800 linear feet) while also incorporating the expanded boosted zone. The results of the simulation increased the available fire flow by approximately 650 gpm, which meets the ISO flow requirement of 2,250 gpm. In addition to expanding the boosted pressure zone, we also recommend that the Church Street piping is rehabbed/replaced between Pleasant Street and the tank. Additional investigation by the WDPW is recommended on this stretch of pipe to determine the cause of the hydraulic restriction. If it is determined that tuberculation is the cause, then it will be much more cost effective to clean and line this pipe rather than replace it entirely.

Church Street at Park Street (ISO #8)

Fire flows at this location were found to be deficient by approximately 2,320 gpm. This section of Church Street at Park Street is served by a 10-inch cast iron water main. The Church Street Tank is located to the north and the piping transitions to 12-inch cast iron approximately one block from the ISO location and continues as 12 inches all the way to the tank. It should also be noted that when removing the system pressure constraint of 20 psi from the simulation, there is adequate available fire flow to meet the ISO demand. When running the fire flow simulation with the expanded boosted zone on Upper Church Street, the available fire flow increases by approximately 1,000 gpm, which reduces the deficit to approximately 1,230 gpm. When running the simulation with the proposed improvements on Church Street (see ISO #7) along with the expanded boosted zone, the available fire flow increased to approximately 2,375 gpm; however, it is still not adequate to meet the ISO flow 3,000 gpm. Although the ISO fire flow demand cannot be met, the expansion of the boosted zone along with the piping upgrades on Church Street are still recommended because they increase available fire flow by approximately 1,600 gpm.

East Main Street at Canal Street (ISO #9)

Fire flows at this location were found to be deficient by approximately 1,225 gpm. This location is served by a 12-inch ductile iron pipe on East Main Street. Supply to this location is primarily fed via the 12-inch diameter cast iron main on Church Street. When removing the system pressure constraint of 20 psi from the simulation, there is adequate available fire flow to meet the ISO demand. When running the fire flow simulation with the expanded boosted zone on Upper Church Street, the available fire flow increased by approximately 1,000 gpm, reducing the deficit to approximately 250 gpm below the ISO required 2,000 gpm. When running a third simulation with the proposed improvements on Church Street (see ISO #7) along with the expanded boosted pressure zone, the available fire flow increases to approximately 2,340 gpm which meets the required ISO demand of 2,000 gpm. Therefore, we would recommend implementing these two improvements in order to meet the ISO fire flow demand.

71 South Street (ISO #10)

Fire flows at 71 South Street were found to be deficient by approximately 1,465 gpm. This location is served by a 12-inch ductile iron main on South Street. This end of South Street is also fed from West Street via an 8-inch water main off Homecrest Avenue, which provides additional looping. When removing the system pressure constraint of 20 psi from the simulation, there is adequate available fire flow to meet the ISO demand. When running the fire flow simulation with the expanded boosted zone on Upper Church Street, the available fire flow increased by approximately 1,000 gpm, reducing the deficit to approximately 500 gpm below the ISO required flow of 2,250 gpm. When running a third simulation with the proposed improvements on Church Street (see ISO #7) along with the expanded boosted pressure zone, the available fire flow increased to approximately 2,300 gpm which meets the required ISO demand of 2,250 gpm. Therefore, we would recommend implementing these two improvements in order to meet the ISO fire flow demand.

Mechanic Street at Desmond Avenue (ISO #11)

Fire flows at this location were found be deficient by approximately 2,220 gpm. This location is currently served by an existing 6-inch cast iron water main on Mechanic Street. When removing the system pressure constraint of 20 psi from the simulation, there was still inadequate available fire flow to meet the ISO demand. When running the fire flow simulation with the expanded boosted zone on Upper Church Street, the available fire flow increased by approximately 900 gpm, reducing the deficit to approximately 1,300 gpm below the ISO required flow of 3,000 gpm. The commercial fire flow requirement of 3,000 gpm is a large flow for a 6-inch pipe to accommodate. A second improvement scenario was performed in which the pipe on Mechanic Street was increased to 8 inches in diameter in addition to incorporating the expanded boosted zone. The results of this scenario increased the available fire only marginally (approximately 100 gpm), indicating that the majority of the headloss is occurring elsewhere in the distribution system. A third improvement scenario was run which incorporated the upgrades from the previous scenario along with the upgrades on Church Street (ISO #7) and the expanded pressure zone. These improvements increased the available fire flow to approximately 2,340 gpm; however, it did not meet the required ISO flow of 3,000 gpm. The minimum recommendation is to upsize the pipe on

Mechanic Street to 8-inch in diameter. If the other noted upgrades are also incorporated it will increase available fire even closer to the required ISO flow of 3,000 gpm.

East Street at Ross Avenue (ISO #12)

Fire flows at this location were found to be deficient by approximately 975 gpm. This location is currently served by a 12-inch ductile iron main on East Street. When running the fire flow simulation with the expanded boosted zone on Upper Church Street, the available fire flow increased by to approximately 1,750 gpm which meets the required ISO flow at this location. Therefore, we recommend the expansion of the boosted zone to meet the ISO demands.

Gilbertville Road at East Street (ISO #14)

Fire flows at this location were found be deficient by approximately 2,220 gpm. This location is currently served by an existing 8-inch asbestos cement water main. This location is on a long stretch of 8-inch water main that serves as the primary feed into the system from Dismal Swamp Well. When running the fire flow simulation with the expanded boosted zone on Upper Church Street, the available fire flow increased only marginally (approximately 220 gpm) which is still well below the required ISO flow of 3,000 gpm. Due to the isolated geographic location of this water main compared to the rest of the distribution system, no opportunities for looping are feasible at this time. An increase in pipe diameter will also not make a significant improvement. Therefore, no recommendations are made for this location.

5.3.6 Summary

A variety of hydraulic criteria were used to evaluate the adequacy of the distribution system. In many regards, the water system is strong and in relatively good condition. However, a number of deficiencies exist throughout the system that should be addressed as funding allows. Following is an overview of the areas of identified deficiencies. Specific detail can be found in the previous sections. Summary recommendations for distribution system piping improvements can be found in Section 8 of this report.

5.3.6.1 Water System Pressure

Pressures throughout the system are generally adequate, however because of the rolling terrain of Ware, the pressures in the system range vary significantly. As is typical of most systems, areas of low pressure exist in the immediate vicinity of storage tanks (Church Street Tank) and in the highest elevations of the system. Little can be done about these conditions unless the tank overflow is raised or individual booster systems are placed on the service lines of the affected customers. Currently there is a booster pump station at the Church Street Tank since the tank is at a lower hydraulic grade line than four houses nearby along Gilbertville Road. It is recommended based on the fire flow analysis that this boosted zone be expanded to include an additional 10 to 11 houses along Upper Church Street with elevations above 600 feet. Under the projected maximum day demand in 2025 pressure will range between 20 to 116 psi. The use of localized pressure reducing valves is recommended for pressures above 100 psi.

5.3.6.2 Pipe Velocities and Headloss

A higher velocity of water in a pipeline increases headloss and subsequently increases pumping costs. In general, velocities throughout the system were adequate under 2025 maximum day demand conditions with the pumps off. Velocities were not evaluated during fire flow analysis (as this is an extreme situation).

5.3.6.3 Dead-End Mains and Pipe Looping

The entire system generally appears to be well looped with the exception of a long 14,000 linear foot stretch of 8-inch water main on Greenwich Road which extends north to the Hardwick town line. However, due to its geographic location relative to the rest of the system, no opportunities for looping are available at this time. The majority of the dead-ends consist of small diameter asbestos cement and cast iron piping. In general, older un-lined cast iron dead-end mains should be targeted for long term replacement and included in the yearly pipe replacement program.

5.3.6.4 Fire Flow

In general, the Ware water system is adequate in terms of being able to provide the needed residential fire flows. However, it should be noted that approximately 40% of the pipe in the system is 6 inches or smaller in diameter, which may limit fire flow capacity (standard water works practice recommends 8 inches as a minimum). Given the amount of 6-inch pipe in the system, replacement should be prioritized to locations with hydraulic deficiencies. These residential areas identified through the hydraulic analysis were found to be the most hydraulically deficient (i.e. <500 gpm available fire flow):

1. Mountainview Drive and Oakridge Circle (3,700 linear feet of 6-inch)
2. Canal Street (280 linear feet), Clinton Street (550 linear feet), and Maple Street (290 linear feet) (all 4-inch dead-ends)
3. Dunham Avenue (80 linear feet of a 2-inch dead-end)

A number of commercial locations of the system are deficient. In total, 12 commercial ISO fire flow test locations were evaluated using the hydraulic water model and all 12 were found to be deficient. The analysis was performed using standard engineering practice where the available fire flow represents the total flow available while maintaining a minimum of 20 psi throughout the system, not just at the fire flow node. Due to the various high elevations in the system, the available fire flows at these locations were limited by pressure drops below 20 psi elsewhere in the system. Although the improvements described previously did not solve all the commercial ISO flow deficiencies, the following provided the largest increase in available fire flow and should be considered:

1. Expansion of the boosted zone around Church Street Tank to include the 10 to 11 additional homes on Upper Church Street (ISO #1).
2. Replacement of approximately 800 linear feet of 10-inch cast iron pipe on Church Street between Pleasant Street and Prospect Street with new 12-inch ductile iron pipe (ISO #7).
3. Replacement of approximately 3,000 linear feet of 12-inch cast iron pipe on Church Street between Prospect Street and the Church Street Tank. Confirmation of pipe condition is

recommended prior to replacement to determine if hydraulic restriction is related to another cause (i.e. partially closed valve, mislabeled pipe size, or etc.) (ISO #7).

4. Replace approximately 2,000 linear feet of 6-inch cast iron pipe on Mechanic Street with new 8-inch ductile iron (ISO #11).

As noted previously, little can be done about the low pressures in high elevation unless the tank overflow is raised or individual booster systems are placed on the service lines of the affected customers. Therefore, if meeting the ISO demands at the large commercial locations is critical, it would be most effective to incorporate local booster systems on a case-by-case basis at these locations. It should also be noted that the estimates provided are with all well pumps off. Increased flows would be provided with pumps on (but is not part of typical fire flow analyses).

5.4 WATER MAIN INVENTORY

Water mains in particular have been identified as the largest component of drinking water systems requiring attention. In fact, the 2011 Drinking Water Infrastructure Needs Survey Assessment (DWINSA) report by the EPA identified the transmission and distribution component to be over 64.4% of the total need for the next twenty years. This corresponds to an amount of $247.5 billion dollars.

The water works industry is moving towards a practice of maintaining an on-going replacement program where 1% to 2% of the total system length is replaced annually. Doing this would help assure that the distribution system is fully replaced every 50 to 100 years to improve and maintain reliability. As this approach would require large annual capital expenditures that could have proportionately larger rate impacts to smaller systems, replacing 2% of a distribution system annually could be very difficult without financial assistance. Taking into consideration the size of the WDPW system, we will assume replacing 0.5% of the system annually. With a current system size of approximately 47 miles, this would equate to approximately 1,240 linear feet per year of water main replacement. Assuming a unit capital cost of $175 per linear foot of 8-inch water main installed, the total cost per year for WDPW calculates to be approximately $220,000. Under this scenario, the distribution system would be fully replaced in 200 years. It is acknowledged that as priorities change and funding better understood, the annual replacement program can be re-assessed and modified as necessary.

Within the annual replacement program budget, the WDPW plans to complete a phased project to remove an older existing 6-inch cast iron water main on West Street with poor hydraulic capacity. Currently this street has 6-inch and 12-inch water mains that supply water to the customers. The project includes relocating the services from the 6-inch main to the 12-inch main and then eliminating the 6-inch main and any interconnections from the system.

5.4.1 Method of Analysis

The Ware water distribution system is comprised of several types of water main installed between 1912 and the present. Each type of water main will reach the end of its useful life at a different time depending on the age, diameter, materials of construction, installation, and working pressure. Therefore, it is important to have a comprehensive inventory of all water mains in the system. Based on data provided by the WDPW the following data was compiled and tabulated for all water main segments:

- Diameter;
- Material of Construction;
- C-value;
- Static Pressure;
- Break History.

In future analyses, the installation date (if available) and areas of water quality complaints (after WTP construction) should be included.

A weighted ranking system was then developed for the data and used to calculate a numerical value (sum) for each segment and prioritization of the future water main improvements. In general, the higher the weighted value, the more important that criteria is for determination of replacement need. The values and weighting factors determined for each of the criteria are presented below.

Diameter - In general, the smaller the diameter of the installed water main, the less likely it may be able to provide adequate supply. Larger diameter water mains have thicker walls, and are therefore stronger as well. In general, 8-inch diameter pipe is the accepted minimum water main diameter recommended for water distribution systems. Accordingly, the criteria values for diameter were established as follows:

TABLE 5-3
DIAMETER CRITERIA VALUES

Diameter	Value
2-inch	100
4-inch	100
6-inch	100
8-inch	40
10-inch	20
12-inch	10
16-inch	5

The corresponding weighting factor selected for diameter was 20%.

Material of Construction - The typical water main materials of construction have a variety of differences based on their strength, corrosion resistance, flow characteristics, etc. that can be correlated to their useful life expectancies. However, it is noted that even the same materials (such as cast iron) have different life expectancies based on their period of manufacture. A recent study by the American Water Works Association (AWWA) titled "Buried No Longer: Confronting American's Water Infrastructure Challenge" utilized a pipe failure probability model, extensive research and professional experiences to estimate the typical service life for various types of pipe as shown in Table 5-4.

TABLE 5-4
ESTIMATED SERVICE LIFE BY MATERIAL

Material	Service Life (Years)
Asbestos Cement	100
Cast Iron	115
Ductile Iron	110
HDPE	100
PVC	100

It should be noted that due to changing materials and manufacturing techniques, pipe installed through the 1920s has a longer useful life than installed after World War II. In addition, the data provided in Table 5-4 is for pipes that were installed in suitable ground conditions and modern laying practices. Pipes that were installed in poor ground conditions or improperly installed may have shorter expected service lives.

Based on the expected service life and current age of the water main in the Ware system, the following criteria values were utilized for the pipe material:

TABLE 5-5
MATERIALS CRITERIA VALUES

Material	Value
Asbestos Cement	100
Cast Iron	70
Ductile Iron	5
HDPE	5
PVC	5

A weighting factor of 30% was selected for the material of construction.

Static Pressure - Based on the current hydraulic model, static pressures within the water distribution system can vary from a high of approximately 120 psi down to a low of approximately 40 psi. Massachusetts Guidelines for Public Water Systems states that normal working pressure in the distribution system should be approximately 60 to 80 psi and not less than 35 psi. Standard water works practice generally allows a normal maximum system pressure of 80 to 100 psi. Although common in New England, higher pressures can lead to increased water loss at leaks and more frequent breaks as water mains approach the end of their useful life. For the static pressure criteria, the following values were established.

TABLE 5-6
PRESSURE CRITERIA VALUES

Pressure (psi)	Value
Greater than 120	100
100 - 120	80
80 - 100	60
Less than 80	20

The weighting factor of 20% was selected for static pressure.

Break History - Historical water main break records offer one of the clearest indications of past and likely future, problem areas within a water distribution system. Although highly undesirable, breaks can be a regular occurrence within water distribution systems that must be dealt with immediately. Several factors can contribute to breaks including poor installation, shallow burial depths, corrosion, environmental factors, and many of the other criteria discussed. Accordingly, the criteria values for break history were established as follows:

TABLE 5-7
BREAK HISTORY CRITERIA VALUES

Breaks	Value
4+	100
3	80
2	60
1	40
0	0

Due to its highly undesirable impacts, a weighting factor of 30% was selected for break history.

5.4.2 Prioritization of Water Main Projects

Utilizing the criteria and weighting factors discussed above a pipe condition score was calculated for each pipe in the distribution system. These scores were then sorted from highest to lowest as an initial means of upgrade prioritization (as a higher sum indicated a greater need for upgrade/replacement). Two water main inventory spreadsheets were developed from this exercise. The first includes the alphabetized list of pipes by street and their associated physical characteristics (no pipe condition scores). The second spreadsheet sorts the pipes according to their pipe condition scores, from highest to lowest and also highlights the pipes recommended for replacement. These spreadsheets are included in Appendix H.

Figure 5-3 includes pipe condition scores for all water mains in the system. Pipe rankings were colored as follows:

- Red: Pipe ranking from 75-90.
- Orange: Pipe ranking from 60-74.
- Blue: Pipe ranking from 40-59.
- Green: Pipe ranking from 0-39.

Data from MassGIS and Town of Ware
HARDWICK
W. BROOKFIELD
PALMER
Church St. Tank
Capacity: 1.5 MG
Overflow: 659 FT
Barnes Street Sources
(Wells 1,2,3,4, & Cistern)
Anderson Rd Tank
Capacity: 1.0 MG
Overflow: 659 FT
Greenwich Plns Rd
Old Poor Farm Rd
Hutchinson Rd
King George Dr
Greenwich Plains Rd
Fisherdick Rd
Cummings Rd
E Greenwich Plains Rd
Osborne Rd
Winslow Rd
Howard Rd
Walker Rd
Crescent St
Lee Rd
Campbell Rd
Greenwich Rd
Hardwick Pond Rd
Anna St
King St
King Cir
Sorel Rd
Old Stagecoach Rd
Old Gilbertville Rd
Upper Church St
Ware Rd
Sheehy Rd
Pond Brook Park
Gilbertville Rd
Walter Dr
W Main St
Belchertown Rd
Doane Rd
Babcock Tavern Rd
Desantis Dr
Anderson Rd
Crescent Ter
Berkshire Dr
Willow St
Jovewell St
Highland Village
Hazel Ave
Convent Hill Rd
Walnut St
School St
Church St
Aspen St
Dale St
Cottage St
Prospect St
Park St
High St
Park Ave
Pleasant St
Eddy St
W Main St
Main St
E Main St
Pine St
East St
Ross Ave
Morse Ave
Maple Ave
Quarry St
Prendiville Rd
Old Warren Rd
Mattson Blvd
Pleasant Ter
Viomis Ave
Eagle St
Clinton St
Barnes St
Parker St
Webb St
North St
Pulaski St
Gould St
Vernon St
Gilbert Ct
Maple St
Knox Ave
Mechanic St
Demond Ave
Elm St
Glendale Cir
Wildflower Dr
Gould Rd
Monroe St
Williams St
South St
Greenway Ave
Homecrest Ave
Robbins Rd
Ashley St
Marjorie St
Lois St
Bond St
Chestnut St
Dowell Rd
Dunham Ave
Fourth Ave
3rd Ave
2nd Ave
1st Ave
West St
Richfield Ave
Towne St
Hillside Dr
Mountainview Dr
Laurel Dr
Malboeuf Rd
Oakridge Cir
Skyview Dr
W Warren Rd
Palmer Rd
Kingsberry Ln
Woodland Hts
Westbrook Ave
Pinecrest Cir
Dugan Rd
Williston Dr
Hillside Ter
Juniper Hill Rd
Klug St
Belchertown Rd
Bacon Rd
Shady Path
Shady Hill Rd
Bel Air Dr
Old Warren Rd
N
0 250 500 1,000 1,500 2,000
Feet
Legend
Junction
Tank
Well Source
Pipe
Ranking
0 - 39
40 - 59
60 - 74
75 - 90
Water Main Ranking
Ware, MA
PROJ NO: 13471A
DATE: 10/18/2016
FIGURE:
5-3
WRIGHT-PIERCE
Engineering a Better Environment
Sources: Esri, HERE, DeLorme, USGS, Intermap, increment P Corp., NRCAN, Esri Japan, METI, Esri China (Hong Kong), Esri (Thailand).

The piping upgrades included in the Capital Improvement Plan were selected based on two primary factors: pipe condition score and available funding per year. In general, those pipes with a pipe condition score greater than 60 are considered to be in fair to poor condition. However, because of limitations in funding, all piping with scores of 60 or higher cannot be replaced within a 10-year improvement period. As stated earlier in this section, we are assuming a replacement schedule of approximately 1,240 LF of pipe per year at $175/LF which correlates to approximately $220,000 per year for replacement costs. Over the course of the 10-year capital improvement period, this correlates to approximately 12,400 linear feet of new pipe construction. In general, water mains with pipe condition scores of 60 or more were initially selected for replacement that added up to a total of approximately 36,192 linear feet. Since this value exceeds the proposed budgeted amount for repairs, we would recommend the WDPW replace these pipes each year as budget allows. The specific water main replacement recommendations and associated costs are included in Section 9 and within the Capital Improvements Plan (CIP) in Section 9.

5.5 DISTRIBUTION STORAGE

Distribution storage is used for and provides a number of important functions to a water system. This includes establishing and sustaining adequate pressure throughout the system, fire fighting capabilities, and short-term emergency purposes. Storage also provides a "cushion" to equalize peak fluctuations, improves service reliability, provides operational flexibility, and allows intermittent operation of pumping equipment. Ware has two distribution storage facilities on a single pressure zone. As part of this study, a storage analysis was conducted, and is presented in the following section.

5.5.1 Storage Analysis

In general, system storage is necessary to satisfy the following three conditions:

- Storage should be provided to satisfy all demands which exceed the maximum day flow rate. In general, the volume of storage which is depleted during the typical daytime, peak flow periods is then refilled during the lower demand, early morning hours.

- Storage should be provided for fire protection. If a fire occurred during the maximum day demand, the water used to fight the fire would be drawn from storage volume.
- Storage can also be provided to meet emergency conditions such as power failures, transmission main breaks, other potential disruptions in service, etc.

The primary criteria used to evaluate storage requirements include: average and peak water usage, water supply capabilities, as well as fire protection and reserve or emergency needs. Each of these criteria is used to establish three components of storage: (1) peak-hour volume, (2) fire volume, and (3) emergency volume. The total of these components is referred to as the active or available usable storage volume. All storage components described should be available while still providing at least 20 psi of pressure throughout the system. This pressure is equivalent to the volume of water stored 46 feet above the highest service. It is also desirable for storage tanks to be dispersed appropriately throughout the distribution system to deliver flows from multiple locations to reduce pipe velocities and provide flows to a fire location.

Peak-hour storage is the volume of water required during peak demand periods above the maximum available pumping capacity. This volume should be provided independent of the required fire or emergency volumes in order to assure sufficient reserve volume in the event of a fire or emergency during a peak demand period.

Fire storage is that component set aside solely for the purpose of fire fighting. Properly sized storage will include a sufficient volume of water for fire protection on days of maximum demands while maintaining a minimum pressure of 20 psi throughout the distribution system.

Emergency storage is desirable and is recommended for other purposes above and beyond that required for equalizing and fire volumes. This may include storage desired as a factor of safety for emergencies or where demands are unpredictable and fluctuate widely. Determining emergency storage is somewhat arbitrary and generally depends on the level of safety a utility desires. Emergency storage is often simply calculated as the volume necessary to supply the system during repair or maintenance work, or in the event that the pumping facilities do not have emergency back-up power equipment. In most cases, this is calculated as a specified number of hours of the average-day demands.

Storage in the Ware system is provided by two storage facilities. The storage facilities are located throughout the system and have a maximum hydraulic grade-line of 659 feet. Storage components for these two tanks were calculated as follows:

1. Equalization Storage for Peak-Hour Storage Fluctuation - The storage volume necessary to provide the system hourly fluctuation demands was estimated to be 25 percent of the maximum day total demand. Twenty-five percent of the projected year 2025 maximum-day demand is approximately 0.34 MG (0.25 * 1.36).
2. Fire Protection Storage Volume - The maximum required available fire flow which is generally recommended to be provided in this system is 3,500 gpm for 3 hours, equal to 0.63 MG. This rate was chosen based on the commercial fire flow requirements established by the Insurance Services Office (ISO).
3. Emergency Storage - Emergency storage volume provides a short term water supply during emergencies such as transmission main failures, equipment failures, power failures and natural disasters. Emergency storage is typically estimated to be one average day demand. However, the emergency storage component can be waived if back up power is provided at sources capable of providing the average daily demand. The Cistern has backup power that is utilized at the Pump House and also the Wellfield which is capable of providing the average daily demand. Therefore, the emergency component is waived.

The calculation for the current available active storage volume is summarized on Table 5-8 and the storage analyses developed within Table 5-9.

TABLE 5-8
EXISTING AVAILABLE ACTIVE STORAGE VOLUME

Storage Component	Anderson Road Tank	Church Street Tank
Total Capacity (MG)	1.0	1.5
Diameter (ft)	52	100
Overflow Elevation (ft)	659	659
Base Elevation (ft)	594	635
Unit Volume (gal/ft)	15,885	58,748
Highest User Served (ft)	607	607
Minimum Tank Elevation to Maintain 20 psi System Pressure (ft)	653.2	653.2
Total Active Storage (MG)	**0.09**	**0.34**

To determine the adequacy of the existing active storage volume available, an analysis of each of the storage components described was made using projected demands through year 2025. Table 5-9 presents the storage component analysis.

TABLE 5-9
STORAGE COMPONENT ANALYSIS

	2016	2025
Projected Average-Day Demand (MGD)	0.78	0.79
Projected Maximum-Day Demand (MGD)	1.35	1.37
Peak Hour Storage (25% MDD)	0.34	0.34
Fire Protection Storage	0.63	0.63
Emergency (waived)	N/A	N/A
Total Storage Needed	**0.97**	**1.0**
Available Usable Storage	0.43	0.43
Surplus or (Deficit)	**-0.54**	**-0.57**

The existing active storage volume in the system is approximately 0.43 MG (0.09 MG + 0.34 MG) and the total required active storage volume for the previously described components is 1.0 in year 2025. Based on this analysis, the Ware water system will have an increased storage deficit of approximately 0.57 MG in year 2025.

Additional usable storage can be achieved by expanding the boosted zone near the Church Street Tank to include the additional users on Upper Church Road with elevations over 600 feet. The revised calculation for available active storage with the expanded booster zone is summarized in Table 5-10 and the revised storage analysis developed within Table-5-11.

TABLE 5-10
AVAILABLE ACTIVE STORAGE VOLUME
WITH EXPANDED BOOSTED PRESSURE ZONE

Storage Component	Anderson Road Tank	Church Street Tank
Total Capacity (MG)	1.0	1.5
Diameter (ft)	52	100
Overflow Elevation (ft)	659	659
Base Elevation (ft)	594	635
Unit Volume (gal/ft)	15,885	58,748
Highest User Served (ft)	585	585
Minimum Tank Elevation to Maintain 20 psi System Pressure (ft)	631	631
Total Active Storage (MG)	**0.44**	**1.63**

TABLE 5-11
STORAGE COMPONENT ANALYSIS
WITH EXPANDED BOOSTED PRESSURE ZONE

	2016	2025
Projected Average-Day Demand (MGD)	0.78	0.79
Projected Maximum-Day Demand (MGD)	1.35	1.37
Peak Hour Storage (25% MDD)	0.34	0.34
Fire Protection Storage	0.63	0.63
Emergency (waived)	N/A	N/A
Total Storage Needed	**0.97**	**1.0**
Available Usable Storage	2.07	2.07
Surplus or (Deficit)	**1.1**	**1.07**

Under this scenario, the existing active storage volume in the system is approximately 2.07 MG and the total required active storage volume for the previously described components is 1.0 in year 2025. Based on this analysis, the Ware water system will have storage surplus of approximately 1.07 MG in year 2025 if they expand their boosted zone to include the users over 600 feet in elevation. Otherwise an additional water storage tank would be required.

5.5.2 Storage Tank Operations

One of the potential drawbacks of surplus storage is the increased detention time that is created when adequate turnover is not present. The current tanks operations obtained from the hydraulic model have an operating range of only a few feet. Furthermore, all of the WDPW's tanks have one inlet/outlet pipe. This configuration can result in stratified water within the tank because the last water to enter the tank when it is filling is typically the first water to leave the tank when it is emptying. Over time, this "last in, first out" configuration causes the ageing of water in the top portion of the tank. Old water can result in stagnation, loss of chlorine residual, increase in disinfection byproducts, and increased microbiological activity (i.e. total coliform) within the tank.

Therefore, it is good practice to minimize water age in the tanks as much as possible. This can be accomplished by operating the system to allow the tank levels to fluctuate over a greater range, by adding internal tank mixing systems, or both.

Implementation of tank mixing is recommended to be implemented at both of the WDPW's tanks. Therefore, the following section provides a background for the various forms of mixing systems.

5.5.2.1 Storage Tank Mixing Systems

In general, there are two types of tank mixing systems currently available for most tanks: (1) passive and (2) active. Some of the most common system types for each along with their typical advantages and disadvantages are discussed in the following sections.

Passive Type Mixing System

Passive systems mix a tank through the use of specialized valving, which take advantage of the existing flows into and out of a tank.

Elastomeric Check Valve Tank Mixing System

The TideFlex tank mixing system is a passive system consisting of inlet piping and a series of elastomeric check valves that ensure fill and draw from the tank are at different elevations, increase jet velocities to promote mixing and turnover in the tank. This system includes the installation of vertical or horizontal piping inside the tank (depending on tank geometry) that would extend from the existing common inlet/outlet at the bottom of the tank. Water is dispersed into the tank via multiple check valves along the inlet pipe at multiple elevations and/or locations. These inlet check valves are designed to have a high jet velocity that promotes mixing in the tank during tank filling. The outlet check valves are typically located near the bottom of the tank. The effective mixing action generated by this system occurs when the tank is filling.

Advantages and disadvantages of this type of passive mixing system include the following:

Advantages:

1. This mixing system has the lowest operation costs because no new pumps or motors are typically required.
2. Ice formation within the tank should be reduced as the surface water is agitated during each fill cycle.
3. This system is essentially maintenance-free as the only components of this system that require maintenance are the check valves. The manufacturer claims that the valves have a 25-year operation life.

Disadvantages:

1. The tank only mixes when filling. No mixing occurs during periods of inactivity and may require a minimum operational flow rate to achieve mixing.
2. The mixing system requires internal piping and pipe supports. Depending on tank materials, the piping manifold could need to welded (or attached via other means) to the tank walls and/or floor.
3. Depending on required layout (size and number of valves), the additional head loss created by the valves may increase pumping costs slightly.
4. Cannot be used for integral chlorine boosting. A separate booster station would be required.

Active Type Mixing Systems

Active mixing systems use mechanical means to mix a tank that do not depend on the existing flows into and out of a tank. There are currently two common types of active mixing systems in the municipal water works industry.

SolarBee Recirculation System

The first SolarBee Recirculation System introduced to the market is an active type system that consists of a solar powered pump that floats on the water surface in the center of the storage tank. The intake for the pump is set just above the tank floor and is curved upward to reduce the potential for redisturbing the sediment that has settled on the bottom of the tank. Water is drawn from the lower portion of the tank and distributed at the water surface to promote mixing in the tank. A photovoltaic panel that can be mounted to the top of the tank (or elsewhere) supplies the required power during the daylight and a rechargeable battery supplies energy during the night. There is an optional electric input for periods of extended overcast weather or during low solar conditions. Operational information about the status of the SolarBee unit is communicated to a local control panel and can also be transmitted to a Supervisory Control and Data Acquisition (SCADA) location using existing telemetry. There are no specific operations and maintenance (O&M) costs related to the SolarBee mixing system except for maintenance required to keep the photovoltaic cell clean. There is no electric power required to mix the tank with the photovoltaic cell in full operation.

Advantages and disadvantages of this type of active mixing system include the following:

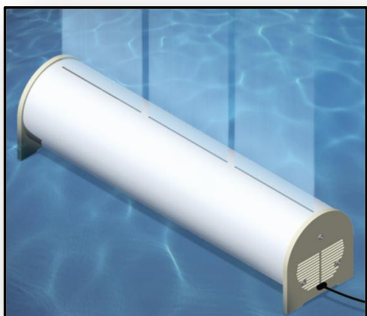
Advantages:

1. The tank is continuously mixed (as long as the system is in operation) as it does not depend on the tank to be filling.
2. The system can be maintained without taking the tank out of service.
3. The system is designed to fit through roof hatches for removal and maintenance purposes.
4. Ice formation within the tank should be minimal as water movement is continuous as long as the unit is functioning.
5. Low operation costs as power is supplied by solar equipment.
6. No internal piping manifold is required (i.e., no welding or attachment via other means to the tank walls and/or floor).
7. No additional head loss is created.

8. Can also be used for chlorine boosting (with equipment add on).

Disadvantages:

1. Maintenance is required at the photovoltaic cell to ensure a clean surface for solar energy gain. Snow or ice may impact the photovoltaic cell.
2. Any work or maintenance on the unit requires a confined space entry permit into the top of the tank with a raft.
3. A crane is required when/if retrieval of the SolarBee unit is required.
4. Electricity may be required to maintain mixing during extended overcast periods.



Grid powered models (referred to as GridBee) are also now available from the same manufacturer when utilization of solar power is not feasible or desired. Unlike the SolarBee (which floats on the water surface), the GridBee unit is mounted on the tank's floor.

Mechanical Mixing System

The PAX System is another active type mechanical mixing system that consists of a submersible motor and impeller system connected to the top of a tripod which is placed on the tank's bottom/floor. The unit is relatively compact and its tripod legs are collapsible to make installation through the smaller 18-inch openings possible. The motor is a water-filled, water lubricated, brushless DC type that is powered off a 120 Volt alternating current (VAC) circuit. The unit is typically set in the center of the tank and is 4-feet in height. The unit's impeller rotates at a rate of up to 1,200 revolutions per minute (rpm) and is set at the appropriate rate determined by the Manufacturer for the particular tank size. The unit's control center is of stainless steel construction. Status outputs include an on or off status and a common fault. It is understood that solar panel options are also available for powering the units.

Advantages and disadvantages of this type of active mixing system include the following:

Advantages:

1. The tank is continuously mixed (as long as the system is in operation) as it does not depend on the tank to be filling.
2. The system is designed to fit through small openings for removal and maintenance purposes.
3. Ice formation within the tank should be minimal as water movement is continuous as long as the unit is functioning.
4. No internal piping manifold is required.
5. No additional head loss is created.

Disadvantages:

1. The tank must be taken out of service system for maintenance.
2. A crane is likely required when retrieval of the PAX unit is required (through a roof hatch).
3. The unit's legs would need to be welded and/or restrained if installed on an uneven floor.
4. If the solar option is selected, maintenance would also be required at the photovoltaic cell to ensure a clean surface for solar energy gain. Snow or ice may impact the photovoltaic cell.
5. If the solar option is selected, electricity may be required to maintain mixing during extended overcast periods.
6. Cannot be used for integral chlorine boosting. A separate booster station would be required.

In summary, as the water level within the WDPW's water storage tanks do not currently fluctuate significantly, the use of active mixing systems is recommended for all tanks. Both of the tanks should be individually evaluated for proper sizing.

5.5.3 Tank Evaluation and Maintenance

As described with Section 2 of this report, the current condition of Ware's water storage tanks are generally acceptable with some cleaning and miscellaneous repairs recommended. That section, as well as individual inspection reports, should be referred to for additional detail.

5.6 INTERCONNECTION WITH ADJACENT WATER SYSTEMS

Interconnections with surrounding communities are valuable from an emergency response perspective, but the Town of Ware currently does not have any interconnections with adjacent communities. If ever determined to be needed or desired, the WDPW currently has existing water mains close to the borders of Hardwick, West Brookfield, and Palmer.

5.7 DISTRIBUTION SYSTEM MAINTENANCE

5.7.1 Unaccounted-for Water Reduction

As discussed in Section 3, non-revenue water in the Ware system was estimated to be an average of approximately 19.0% of the total water production. Approximately, 60% of all non-revenue water is attributed to leakage in water systems in the US.

Water leakage can be divided into two broad categories: (1) Unavoidable Leakage and (2) Underground Leakage, as described below.

- Unavoidable Leakage - Unavoidable leakage includes the numerous minor water leaks that normally exist in any water system. However, because of their number and size, they are more costly to repair than to simply allow them to exist.
- Underground Leakage - Underground leakage occurs from factors such as earth settlement and corrosive water or corrosive soil, which cause deterioration of pipes and joints. It also includes serious water main breaks and service-line breaks. The cost of wasted water from underground leakage often makes leak repair economical.

Unfortunately, most underground leakage is never seen reaching the surface since the individual leaks, although numerous, are spread throughout the system and have relatively low flows. Due to the large amount of older piping in the Ware distribution system, low volume underground leakage is most likely a major contributor to the unaccounted-for water.

MassDEP requires that a leak detection survey be performed on the entire Ware water system every two years. Comprehensive water audits can be useful in determining water usage that is above normal in various areas, providing target areas for leak detection or system maintenance.

5.7.2 Comprehensive Water Audit

A water audit is a process whereby a detailed accounting of all water use is made. It quantifies usage to various categories over a certain period of time. The audit can often pinpoint uses within the system that are above normal limits. An audit involves quantifying water from all production sources, all metered users, and all non-metered authorized users. It also requires making estimates of potential water losses, unavoidable leakage and total leakage. From analysis of the data, a priority listing can be developed to target specific areas of abnormal usage in the system.

5.7.3 Valve Maintenance

Since operation of valves within a distribution system is usually required only in emergencies (water main breaks), valves are often installed and then forgotten until such an emergency arises. Like other mechanical devices, valve operability is adversely affected by neglect. As a result of this neglect, valves can be found to be inoperable at the worst possible time.

Typically valves within any water system are of the sliding disk type (gate valves). This type of valve, which permits an unobstructed flow when fully opened, is hydraulically very efficient. However, when gate valves are left in the open position, deposits may settle and accumulate on the valve seats and prevent tight closure.

To prevent these problems, a valve exercising and maintenance program is recommended. The Insurance Services Office (ISO) recommends that valves be inspected and operated annually. We recommend that the transmission main valves, those valves located on the larger diameter pipes between the supplies and storage, be inspected semi-annually, once in the spring and again in the fall. The fall operation will discover any problems before the onset of winter. In the spring, inspect these valves by making sure a valve wrench can be put on the operating nut. This inspection will uncover any problems that have been caused by the previous winter and spring rains. All data should be logged and recorded in a data management system. If an asset management system is implemented, it should include custom designed queries that will allow selection of valves by age, condition and type. The water system capital budget should include repair or replacement of a fixed number of valves each year based on condition or operational problems.

The following valve inspection program steps should be included in an asset management system:

A. The data file for each valve should contain at least the following information:
   - Valve Size
   - Opening direction
   - Manufacturer of valve
   - Number of turns to open
   - Date of installation
   - Both general and specific descriptions of valve location including valve ties
   - Date of last maintenance - parts replaced and condition of valve
   - Valve Status (Open/Closed)

B. Prepare a master sheet which would be used to summarize the work performed and man hours involved. The actual valve maintenance program should use a checklist to determine:
   - Condition of gate box
   - Obstructions in gate box that might prevent gate wrench from seating on valve operating nut
   - Operability of valve
   - Number of turns to close and open the valve

- Any leaks detected

Altitude valves at the storage facilities and surge relief valves should also be incorporated into the annual valve exercising and maintenance program. Failure of altitude valves in an open position could result in the tank overflowing resulting in wasted water and potential damage to property. Failure in the closed position could cause a deficit in available fire protection or equalization volume by removing the volume of water in the tank from the active storage volume. Failure of the relief valves at the pump stations could cause damage to the pumps and motors, resulting in costly repair bills. Altitude valves should be serviced and settings should be checked and logged annually.

As part of the Town’s annual flushing program, operators must open and close all required main and hydrant valves on a routine. This program can also help identify a closed valve. A closed or partially closed valve can drastically reduce the system's hydraulics and available fire flow. We recommend electronic logs of valve status and maintenance history be tracked as part of the asset management system.

The most important part of the maintenance program is to evaluate the inspection reports and to implement the necessary repairs. The Fire Department should be notified whenever it is necessary to shut down a portion of the distribution system for such repairs.

Power valve operators are the preferred method for exercising valves for the following reasons. First, water system personnel are able to operate more valves per day, thus reducing the total time allotted for valve operation, and second, reduce the potential of physical injuries caused by valve operation. For increased efficiency, the WDPW may want to consider the purchase and use of this equipment.

5.7.4 Hydrant Maintenance

The distribution system contains approximately 344 active hydrants. Routine hydrant maintenance is essential and should be coordinated with active involvement from the Fire Department. The

ISO recommends that fire hydrants be inspected twice a year. The best time for these inspections is in the spring and in the fall. The fall inspection enables detection of problems before winter conditions. The spring inspection may uncover any problems which may have been caused by the previous winter (e.g., frost heaves).

In addition to semi-annual inspections, non-draining hydrants should be pumped dry immediately after use and checked for:

- Loose or missing caps,
- Missing gaskets,
- Damaged operating nuts or nozzle threads, and
- Corroded breakaway bolts at ground level.

Similar to a valve management program, hydrant maintenance activities should be recorded and the results evaluated and integrated into an asset management database. The water system budget should include replacement of a fixed number of hydrants each year, and maintain a hydrant flushing/inspection program.

5.7.5 Water Main Maintenance

In general, the velocity of water steadily decreases as it leaves the source of supply and approaches the consumer. This decreasing velocity permits the formation of precipitates and allows them to settle out inside the pipe. To remove most of these deposits, a high velocity flushing (Unidirectional Flushing) program is needed. The objective of a unidirectional flushing program is simply to create a high velocity in the pipeline to re-suspend the deposits and to scour the interior surface of the pipe. The water is then flushed out of a hydrant. The optimum times of year for flushing are in the spring and in the fall.

The accumulation of precipitates not only results in reduced flow capacity but also increases pumping costs and/or reduces system pressure. A flushing program will also reduce color and taste complaints from the customers, improve water quality overall and decrease the age of the water in the distribution system.

It is understood that the WDPW currently implements a unidirectional flushing program. If found to be effective, this program should continue to be implemented going forward with improvements to the program as necessary. In general, as the WDPW implements treatment at its sources (to remove potential precipitates), the effectiveness of the flushing program will increase, while the corresponding effort required to perform the program will likely decrease. As improvements to the system are made the flushing program should be reassessed to confirm its applicability and/or increase its effectiveness.

Section 6

SECTION 6

REGULATORY REVIEW

6.1 GENERAL

The Ware Department of Public Works (WDPW) supplies drinking water to the residents of the Town of Ware from four active groundwater sources that have some water quality concerns and the sources require treatment. Over the past few years, the Massachusetts Department of Environmental Protection (MassDEP) and the United States Environmental Protection Agency (EPA) have undertaken significant rule making activity, including:

- A new Office of Research and Standards Guideline (ORSG) for manganese.
- Incorporation of the new federal Revised Total Coliform Rule (RTCR).
- Updates to the Stage 2 Disinfectants and Disinfection By-Product Rule.
- Additional requirements from the federal Reduction of Lead in the Drinking Water Act.
- Updates to the Unregulated Contaminant Monitoring Rule 3 (UCMR 3).
- The addition of the Unregulated Contaminant Monitoring Rule 4 (UCMR 4).

In addition, several pending regulations are anticipated in the near future including the Radon Rule.

6.2 OVERVIEW OF DRINKING WATER REGULATIONS

The purpose of this regulatory review is to assist WDPW in identifying major regulatory topics that might influence long-term decision making regarding supply or treatment strategies. This review highlights important new rules, but does not explore their implications for WDPW in great detail as they are still in their early stages.

The purpose of the Safe Drinking Water Act (SDWA) of 1974 (amended in 1984 and 1996) is to ensure that public water systems meet national standards that protect consumers from the harm of contaminants in drinking water, by requiring EPA to regulate contaminants that present health risks and which are known to, or are likely to, occur in public drinking water supplies. For each regulated contaminant, EPA sets a legal limit on the amount allowed in drinking water. Limits set by States must be at least as strict as those established by EPA.

The Massachusetts Department of Environmental Protection (MassDEP) Drinking Water Program is the primacy agency which regulates Massachusetts water systems under 310 Code of Massachusetts Regulations, Chapters 22 and 36. Chapter 36 is the State's Well Head Protection Regulation and Water Management Act Program.

Existing and future regulations that may impact the WDPW include:

- Ground Water Rule (GWR)
- Total Coliform Rule (TCR)
- Lead and Copper Rule (LCR)
- Stage 2 Disinfectants/Disinfection Byproduct Rule (Stage 2 D/DBPR)
- Radon Rule
- Surface Water Treatment Regulations
- Unregulated Contaminant Monitoring Rule (UCMR)

In 2002, Congress amended the Safe Drinking Water Act (SDWA) by enacting the Public Health Security and Bioterrorism Preparedness and Response Act, which added several important sections to the SDWA to address water system security.

6.2.1 National Primary Drinking Water Regulations

National Primary Drinking Water Regulations (or primary standards) are legally enforceable standards that apply to public water systems for primary contaminants. Primary standards limit the levels of contaminants in drinking water that adversely affect the public's health. Currently, the primary contaminant standards are divided into the following six categories:

- Microorganisms;
- Disinfectants;
- Disinfection Byproducts;

- Inorganic Chemicals;
- Organic Chemicals; and
- Radionuclides.

The concentrations allowed for the primary contaminants are quantified with a maximum contaminant level (MCL) due to the fact that each can compromise public health through chronic or acute exposure. A complete listing of the national primary drinking water standards published by the EPA is included within Appendix C.

6.2.2 National Secondary Drinking Water Regulations

National Secondary Drinking Water Regulations (NSDWRs) (or secondary standards) are non-enforceable guidelines regulating contaminants in drinking water. These contaminants may cause cosmetic effects (such as skin or tooth discoloration) or aesthetic effects (such as color, taste, or odor). The EPA recommends secondary contaminant standards to water systems but does not require systems to comply. However, individual states may choose to adopt them as enforceable standards.

A complete listing of the national secondary drinking water standards as published by the EPA is included within Appendix D.

6.2.3 Massachusetts Drinking Water Standards

Under the SDWA, a state may be granted primacy for implementing the provisions of the SDWA. The MassDEP has primacy for administering the SDWA in the Commonwealth of Massachusetts. Within the MassDEP, the Office of Research and Standards (ORS) is charged with establishing public health standards and guidelines for contaminants in drinking water. This involves adoption of standards established by the EPA, or the adoption of a more stringent standard or guideline.

In general, the Massachusetts drinking water standards follow the national primary and secondary standards. A complete listing of the Massachusetts drinking water standards and guidelines is included within Appendix E. MassDEP has established MCLs not currently in the

National Primary Drinking Water Regulations for total Nitrate/Nitrite, Perchlorate and Radon. MassDEP has also established health guidelines for 32 additional contaminants as well as one additional SMCL for Methyl tertiary butyl ether (MTBE) not covered in the National standards.

In general, Nitrate has been detected in some of the WDPW well sources at levels below the regulated limit. Should the concentration of these compounds increase and exceed their corresponding limits, new treatment processes may need to be considered.

Of particular note to this project is MassDEP's inclusion of manganese with an ORS Guideline Limit of 0.3 mg/L in the Massachusetts Drinking Water Standards. In general, MassDEP is requiring community water systems to implement removal treatment when the ORS Guideline is exceeded.

6.2.3.1 Manganese

MassDEP has been taking a much closer look at raw water and distribution system manganese (Mn) concentrations as a 2004 report by the EPA advised about potential impacts to infants/children from consuming water with manganese concentrations in excess of 0.3 mg/L for sustained periods of time. It is understood that MassDEP is in the process of assembling a more formalized policy on a recommended manganese strategy.

The U.S. Environmental Protection Agency (EPA) originally set a Secondary Maximum Contaminant Level (SMCL) of 0.05 mg/L. This was set to avoid aesthetic concerns such as stains on plumbing and laundered clothes. Each state however can choose to adopt the standard or set a more stringent one. In 2004 the EPA issued a report titled Drinking Water Health Advisory for Manganese to provide guidance to communities that may be exposed to high Mn concentrations.

MassDEP's Guidelines for Public Water Systems state that if the Mn concentration in the raw water exceeds 0.30 mg/L then removal is required. If the Mn concentration is between 0.05 mg/L and 0.30 mg/L, then MassDEP requires the water system to consult with their local MassDEP Office.

Some recent studies have identified the public health risks associated with the ingestion of elevated levels of Mn and MassDEP's recent ORS guideline for Mn closely follows the EPA's Health Advisory for Mn. It is understood that the MassDEP has recently provided a notice on manganese monitoring to Public Water Suppliers along with a Manganese Monitoring Information Sheet. This can be found in Appendix F.

Historically, Mn has been causing water quality problems and chronic consumer complaints in Ware. Mn concentrations have been exceeding its corresponding SMCL of 0.05 mg/L. Additional information regarding Ware's historical Mn concentrations since 1998 can be found in Section 4.

6.2.4 Ground Water Rule

The Ground Water Rule (GWR) which pertains to groundwater sources NOT under the influence of surface water was finalized on November 8, 2006. Compliance requirements of the GWR began in 2010. The purpose of the GWR is to better identify systems at risk for fecal contamination, and to provide the primacy agency a flexible range of tools to better protect the public health.

The GWR has the following four major components:

1. Periodic sanitary surveys of ground water systems that require the evaluation of eight critical elements and the identification of significant deficiencies (e.g., a well located near a leaking septic system). States must have completed the initial survey by December 31, 2012 for most community water systems (CWSs) and then by December 31, 2014 for CWSs with outstanding performance and for all non-community water systems.

2. Source water monitoring to test for the presence of *E. coli*, enterococci, or coliphage in the sample. There are two monitoring provisions:

a. Triggered monitoring for systems that do not already provide treatment that achieves at least 99.99 percent (4-log) inactivation or removal of viruses and that have a total coliform-positive routine sample under Total Coliform Rule (TCR) sampling in the distribution system.
b. Assessment monitoring - As a complement to triggered monitoring, a State has the option to require systems with sources that seem susceptible to fecal contamination, to conduct source water assessment monitoring to help identify high risk systems.

3. Corrective actions required for any system with a significant deficiency or source water fecal contamination. The system must implement one or more of the following correction action options:
    a. correct all significant deficiencies,
    b. eliminate the source of contamination,
    c. provide an alternate source of water, or
    d. provide treatment which reliably achieves 99.99 percent (4-log) inactivation or removal of viruses.

4. Compliance monitoring to ensure that treatment technology installed to treat drinking water reliably achieves at least 99.99 percent (4-log) inactivation or removal of viruses.

A sanitary survey by the State primacy agency would be required every 3 years, and would review eight critical components to the extent that they apply to the individual water system being surveyed:

1. Source
2. Treatment
3. Distribution System
4. Finished Water Storage
5. Pumps, Pump Facilities and Controls
6. Monitoring, Reporting, and Data Verification

7. System Management and Operation
8. Operator Compliance with State Requirements

Survey frequency may be reduced to five years if the system either treats to 4-log inactivation of viruses or has an outstanding performance record in the eight performance elements documented in previous inspections and has no history of TCR MCL or monitoring violations since the last sanitary survey.

Significant deficiencies in groundwater systems include, but are not limited to, the following types:

- Unsafe source (e.g., septic systems, sewer lines, feed lots nearby)
- Improper well construction
- Fecal indicators present
- Lack of proper cross-connection control for treatment chemicals
- Lack of redundant mechanical components where chlorination is required for disinfection
- Improper venting of chemical storage tanks
- Overflow and drain pipes not properly screened
- Holes in storage tank roof, improper hatch construction, improper clearwell hatch construction
- Inadequate internal cleaning and maintenance of storage tank
- Unprotected cross connection (e.g., hose bib without vacuum breaker)
- System leakage that could result in the introduction of contaminants
- Inadequate monitoring of disinfectant residuals and TCR MCL or monitoring violations

The GWR uses the existing TCR monitoring as one trigger for identifying whether a system should be defined as high risk and requiring source monitoring. A groundwater system that does not disinfect to 4-log virus inactivation which has a distribution system TCR sample that tests positive for total coliform is required to conduct "triggered source water monitoring" to evaluate whether the total coliform presence in the distribution system is due to fecal contamination in the groundwater source. Within 24-hours of receiving the total coliform positive notice, the system must collect at least one groundwater sample from each groundwater source and test it for fecal indicators.

If any monitoring sample is fecal indicator-positive, the system must notify the State immediately, and then take corrective action. Corrective action is required to correct the significant deficiency, provide an alternate source of water, or provide treatment which reliably achieves at least 99.99 percent (4-log) inactivation or removal of viruses before or at the first customer. The 4-log virus inactivation can be achieved through Treatment Technique. One available Treatment Technique is to maintain a disinfectant residual for a prescribed length of contact time. The required contact time is dependent upon the type of disinfectant used and the water pH and temperature.

Systems serving 3,300 or more people per day must monitor the disinfection continuously. When a system continuously monitors chemical disinfection, the system must notify the State any time the residual disinfectant concentration falls below the state-determined residual disinfectant concentration and is not restored within four hours. If any sample does not contain the required residual concentration, the system must take follow-up samples every four hours until the required residual disinfectant concentration is restored.

6.2.5 Revised Total Coliform Rule

On February 13, 2013, the Revised Total Coliform Rule (RTCR) was published in the Federal Register which was then followed by some minor corrections on February 26, 2014. The corrections became effective on April 28, 2014. As of April 1, 2016, all public water systems have been required to comply with the RTCR requirements. Provisions of the RTCR include:

- A maximum contaminant level goal (MCLG) and maximum contaminant level (MCL) for E. coli for protection against potential fecal contamination was set.
- A total coliform treatment technique (TT) requirement was set.
- Monitoring total coliforms and E. coli according to a sample siting plan and schedule specific to the PWS was added to the requirements.

- Allowing PWSs to transition to the RTCR using their existing Total Coliform Rule (TCR) monitoring frequency were added in the provisions.
- Monitoring and certifying the completion of a state-approved start-up procedure for seasonal systems were added to the requirements.
- Assessments and corrective action when monitoring results show that PWSs may be vulnerable to contamination were added to the requirements.
- Public notification requirements for violations.
- Specific language for CWSs to include in their Consumer Confidence Reports when they must conduct an assessment of if they incur an E. coli MCL violation.

In general, the existing TCR establishes an MCL based on the presence or absence of total coliforms (fecal coliform and E. coli). Compliance is based on the presence or absence of total coliforms on a monthly basis and the total number of samples required is a function of population served. Under the current TCR, a system the size of Ware's (approximately 2,360 water consumers) would take fewer than 40 samples per month and a violation triggered when one routine/repeat sample per month is total coliform positive. Under the RTCR, there is no longer a MCL violation for multiple total coliform detections (E. coli only). Instead, the RTCR requires systems that have indication of coliform contamination in the system to assess the problem and take corrective action. The level of assessment is based on the severity or frequency of the contamination. Currently, WDPW complies with all provisions of the RTCR.

6.2.6 Lead and Copper Rule (LCR)

The Lead and Copper Rule (LCR) was promulgated in 1991 is currently in effect for all community water systems and non-transient, non-community water systems. The purpose of the LCR is to protect public health by minimizing lead and copper levels in drinking water, primarily by reducing water corrosivity.

The LCR establishes action levels (AL) of 0.015 mg/L for lead and 1.3 mg/L for copper based on 90th percentile results of tap water samples. An AL exceedance is not a violation, but can trigger other requirements that can include the following:

- Water quality parameter monitoring;
- Corrosion control treatment;
- Source water monitoring/treatment;
- Public education; and
- Lead service line replacement.

Most water systems have incorporated the Rule's requirements. However, often it is difficult for utilities to remain in compliance or to remain on reduced monitoring as source water conditions change over time, or when a new treatment is implemented for the sake of other important water quality goals. Because lead and copper solubility are so sensitive to water quality, anytime a water system makes a change in water chemistry, the change should be brought about very gradually, if possible, and monitoring sampling should be conducted in distribution taps to detect changes in lead and copper levels.

Changes to the LCR were made on October 10, 2007 that addressed the requirements for monitoring, treatment processes, reporting, public notification and education requirements, and lead service line replacement.

Additional changes were made in 2011 which reduced the maximum allowable lead content. This content that is considered to be “lead-free” is a weighted average of 0.25 percent calculated across the wetted surfaces of pipes, pipe fittings, plumbing fittings, and fixture and 0.2 percent for solder and flux. Section 1417 of the SDWA established this definition of “lead-free”. In 2013, the SDWA Section 1417 was amended by the Community Fire Safety Act to include fire hydrants within the list of exempted plumbing devices.

Currently, WDPW complies with all the provisions of the lead and copper rule.

6.2.7 Stage 2 Disinfectants and Disinfection Byproducts Rule (Stage 2 D/DBPR)

The Stage 2 Disinfectants and Disinfection By-Products Rule (Stage 2 D/DBPR) was finalized as of January 4, 2006. The purpose of the rule is to increase public health protection by reducing the presence of disinfection by-products in drinking water. The Stage 2 Rule applies to all community water systems that add a primary or a residual disinfectant. The WDPW system serves less than 10,000 people and is therefore a "Schedule 4" system under the Stage 2 D/DBPR.

While the Stage 2 D/DBPR rule does not change the MCL values for TTHMs and HAA5s that were established under the Stage 1 D/DBPR, it does change the way sampling results are averaged to determine compliance. Compliance determination for Stage 2 will be based upon a Locational Running Annual Average (LRAA) as opposed to the system-side running annual average (RAA) used in Stage 1. LRAAs must be met at every monitoring location while the RAA allows the system to average results over all monitoring locations. Since WDPW is a “Schedule 4”, then they were required to begin LRAA TTHM and HAA5 monitoring by October 1, 2013.

The Stage 2 D/DBP required systems to complete an Initial Distribution System Evaluation (IDSE) to identify new Stage 2 monitoring locations that best represent high-DBP locations.

The WDPW is currently in compliance with this regulation.

6.2.8 Radon Rule

Radon-222 is a naturally occurring volatile gas which forms from the radioactive decay of uranium-238 in the ground. Radon is colorless, odorless, tasteless, chemically inert, and radioactive. Radon can move through air or dissolve into water occurring in soil pores. Radon commonly enters homes through soil gas entering basement and crawl spaces, or when water containing radon is used for cooking or washing it is released into the air of the house where it can be inhaled.

The Radon Rule was proposed on November 2, 1999 but has not yet been finalized. It was re-scheduled to be promulgated in late 2004, but it still remains delayed. The rule is unique in that for the first time, the EPA seeks to address a health risk caused by an air and water-borne contaminant with one rulemaking.

MassDEP has already established an MMCL for Radon of 10,000 picocuries per liter (pCi/L). USEPA originally proposed an MCL of 300 pCi/L and an alternative MCL (AMCL) of 4,000 pCi/L for governments or utilities that have implemented a "multi-media mitigation (MMM) program" to lower indoor air radon from all sources. This means that treatment would not be required for supplies with radon levels between 300 and 4,000 pCi/L if either the State or WDPW were to develop and implement a MMM program. With or without a MMM program, sources with radon levels above 4,000 pCi/L would be required to provide treatment. The volatile nature of radon makes it easy to remove with exposure to the atmosphere, usually during aeration, which EPA has designated as the Best Available Technology (BAT) for radon removal.

6.2.9 Surface Water Treatment Regulations

The WDPW system is supplied entirely by groundwater and has never been classified as groundwater under the influence (GWUI) of surface water. If any of the sources become classified as GWUI in the future, then there are a number of regulations that specifically apply to surface water sources as well as to groundwater sources determined to be GWUI.

These surface water treatment regulations include the following:

- Surface Water Treatment Rule (finalized in 1989)
- Interim Enhanced Surface Water Treatment Rule (finalized in 1998)
- Filter Backwash Recycling Rule (finalized in 2001)
- Long Term 1 Enhanced Surface Water Treatment Rule (finalized in 2002)
- Long Term 2 Enhanced Surface Water Treatment Rule (promulgated in 2006)

The major requirements for these regulations can be summarized as follows:

- Pathogens:
  - 99.9% (3-log) inactivation and/or removal of *Giardia lamblia*.
  - 99.99% (4-log) inactivation and/or removal of viruses.
  - 99% (2-log) removal of *Cryptosporidium* (additional removal could be required based on *Cryptosporidium* monitoring results obtained from source monitoring required as part of the Long Term 2 Enhanced Surface Water Treatment Rule). The WDPW was to comply with the *Cryptosporidium* treatment requirements by October 1, 2012.
- Residual Disinfectants:
  - Disinfectant residual $\geq$ 0.20 mg/L at entrance to distribution system.
  - Detectable disinfectant residual in the distribution system.
- Turbidity Performance:
  - Combined filter effluent turbidity $\leq$ 0.30 Nephelometric Turbidity Units (NTU) 95% of time.
  - Maximum level of 1 NTU.
- Filter Backwash Water:
  - Required to be returned to the head of the plant for full treatment if recycling is practiced.

6.2.10 Unregulated Contaminant Monitoring Rule 3

The Contaminant Candidate List (CCL) was created under the 1996 SDWA Amendments to change the process by which priorities are set in establishing drinking water regulations. The first Contaminant Candidate List was issued in March 1998. Every five years the EPA is required to publish a list of currently unregulated contaminants in drinking water that may pose risks, and make determinations on whether or not to regulate at least five contaminants on a five year cycle, or 3½ years after each CCL is published, if EPA finds that such regulation would present a meaningful opportunity for health risk reduction.

On July 18, 2003, EPA made final determinations for a subset of contaminants on the 1998 CCL, which concluded that sufficient data and information were available to make the determination that regulation was not appropriate for the following nine (9) contaminants: Acanthamoeba, aldrin, dieldrin, hexachlorobutadiene, manganese, metribuzin, naphthalene, sodium, and sulfate.

On April 2, 2004 EPA announced its preliminary decision to carry over 51 contaminants (nine microbiological and 42 chemical contaminants or contaminant groups) from the first contaminant candidate list (CCL1), which was finalized on February 24, 2005 (70 FR 9071) into CCL2. The comment period for draft CCL2 ended on June 1, 2004 and EPA published CCL2 in February 2005.

In the process of creating the final CCL2, EPA removed a group of 23 contaminants suspected of being endocrine disruptors and 35 pesticides, because both groups of chemicals were the focus of additional data collection efforts under other programs at EPA. Both groups of chemicals have been included in the preliminary CCL (PCCL), which is the precursor to CCL3 screening and evaluation process.

Methyl-tertiary dibromoethylene (MTBE) and perchlorate are currently on the second Contaminant Candidate List. EPA did not make a regulatory determination on either perchlorate or MTBE in its CCL2 Preliminary Determinations; Proposed Rule (May 1, 2007). MTBE was not regulated at that time because EPA's health risk assessment had not been finalized. For Perchlorate, EPA is still examining whether it is appropriate to regulate based upon occurrence in public water systems, although there are currently efforts in Congress to force the regulation of Perchlorate. Occurrence and health effects data have justified the inclusion of N-nitrosodimethylamine (NDMA), enterotoxigenic *E. Coli,* in CCL3. MassDEP has already established an MMCL for Perchlorate of 2.0 µg/L. MassDEP has also included MTBE in its listing of health guidelines and SMCLs.

EPA announced the draft CCL3 in February 2008 and described the process used to develop it. This new multi-step process builds on evaluations used for previous CCLs and was based on substantial expert input and recommendations from the National Academy of Science’s National Research Council (NRC) and the National Drinking Water Advisory Council (NDWAC). The draft CCL3 includes 93 chemicals or chemical groups and 11 microbiological contaminants which are known or anticipated to occur in public water systems. The list includes chemicals used in commerce, pesticides, biological toxins, disinfection byproducts, and waterborne pathogens. EPA evaluated approximately 7,500 chemicals and microbes and selected 104 candidates for the CCL3 that have the potential to present health risks through drinking water exposure. The CCL3 was officially published in October 2009 and was established in May 2012. The final CCL 3 includes 104 chemicals or chemical groups and 12 microbiological contaminants.

The complete CCL1, CCL2, and CCL3 list of contaminants are presented in Tables 6-1, 6-2, and 6-3.

TABLE 6-1
LIST 1 CONTAMINANTS FOR UCMR 3

Assessment Monitoring (List 1 Contaminants)	
Contaminant	**Analytical Methods**
Volatile Organic Compounds	EPA 524.3
1,2,3-trichloropropane	
1,3-butadiene	
chloromethane (methyl chloride)	
1,1-dichloroethane	
bromomethane (methyl bromide)	
chlorodifluoromethane (HCFC-22)	
bromochloromethane (halon 1011)	
Synthetic Organic Compounds	EPA 522
1,4-dioxane	
Metals	EPA 200.8 Rev 5.4, ASTM D5673-10, Standard Methods 3125 (1997) (excluding chromium-6)
vanadium	
molybdenum	
cobalt	
strontium	
chromium*	
Chromium-6	EPA 218.7
chromium-6	
Oxyhalide Anion	EPA 300.1, ASTM D6581-08, Standard Methods 4110D (1997)
Chlorate	
Perfluorinated Compounds	EPA 537 Rev 1.1
perfluorooctanesulfonate acid (PFOS)	
perfluorooctanoic acid (PFOA)	
perfluorononanoic acid (PFNA)	
perfluorohexanesulfonic acid (PFHxS)	
perfluoroheptanoic acid (PFHpA)	
perfluorobutanesulfonic acid (PFBS)	

*Monitoring for total chromium- in conjunction with UCMR 3 Assessment Monitoring- is required under the authority provided in Section 1445(a)(1)(A) of SDWA

TABLE 6-2
LIST 2 CONTAMINANTS FOR UCMR 3

Screening Survey (List 2 Contaminants)	
Contaminant	**Analytical Methods**
Hormones	EPA 539
17-β-estradiol	
17-α-ethynylestradiol (ethinyl estradiol)	
16-α-hydroxyestradiol (estriol)	
equilin	
estrone	
testosterone	
4-androstene-3,17-dione	

TABLE 6-3
LIST 3 CONTAMINANTS FOR UCMR 3

Pre-Screen Testing (List 3 Contaminants)	
Contaminant	**Analytical Methods**
Microbiological	EPA 1615
enteroviruses	
noroviruses	
Microbiological Indicators	
total coliforms	
E. coli	
Enterococci	
bacteriophage	
aerobic spores	

Unregulated Contaminant Monitoring Rule 3 (UCMR 3) incorporates 30 contaminants (28 chemicals and 2 viruses). UMCR 3 was published in the Federal Register (FR) on April 16, 2012. Sampling for the UCMR 3 List 3 will occur between 2013 and 2015 for a selected 800 represented PWSs by the EPA. All samples taken from systems with 10,000 people or fewer will be paid for by the EPA.

EPA recently announced the draft CCL4 in January of 2015 which includes 100 chemicals or chemical groups and 12 microbial contaminants. The list includes chemicals used in commerce, pesticides, biological toxins, disinfection byproducts, pharmaceuticals and waterborne pathogens. The changes that were made to the final CCL3 within the draft CCL4 include:

- The addition of two nominated contaminants. These contaminants are manganese and nonylphenol.
- The removal of Perchlorate.
- The removal of five contaminants (1,3-dinitrobenzene, dimenthoate, terbufos, terbufos sulfone, and strontium).

The draft CCL4 chemical contaminants can be found in Appendix G.

6.2.11 Unregulated Contaminant Monitoring Rule 4

The fourth Unregulated Contaminant Monitoring Rule (UCMR 4) was proposed on December 11, 2015 and is anticipated to be finalized by the end of this year (2016). Then, implementation for UCMR 4 is expected to begin in 2017 with monitoring beginning in January 2018.

In accordance with the 1996 SDWA, the EPA is required issue a new list of at least 30 unregulated contaminants every five years. This list of contaminants would need to be monitored by PWSs. The UCMR 4 outlines these chemical contaminants that will need to be monitored between 2018 and 2020.

The List 1 of contaminants for UCMR 4 is presented in Table 6-4.

TABLE 6-4
LIST 1 CONTAINANTS FOR UCMR 4

Assessment Monitoring (List 1 Contaminants)	
Contaminant	**Analytical Methods**
Cyanotoxin Chemical Contaminants	
Total microcystin	ELISA
microcystin-LA	EPA 544
microcystin-LF	EPA 544
microcystin-LR	EPA 544
microcystin-LY	EPA 544
microcystin-RR	EPA 544
microcystin-YR	EPA 544
Nodularin	EPA 544
anatoxin-a	EPA 545
cylindrospermopsin	EPA 545
Metals	EPA 200.8, ASTM D5673-10, SM 3125
Germanium	
Manganese	
Pesticides and One Pesticide Manufacturing Byproduct	EPA 525.3
alpha-hexachlorocyclohexane	
Chlorpyrifos	
Dimethipin	
Ethoprop	
Oxyfluorfen	
Profenofos	
tebuconazole	
total permethrin (cis- & trans-)	
Tribufos	
Brominated Haloacetic Acid (HAA) Groups[1]	EPA 552.3 or EPA 557
HAA5	
HAA6Br	
HAA9	
Alcohols	EPA 541
1-butanol	
2-methoxyethanol	
2-propen-1-ol	
Other Semivolatile Chemicals	EPA 530
butylated hydroxyanisole	
o-toluidine	
Quinoline	

[1] Regulated HAAs (HAA5) are included in the proposed monitoring program to gain a better understanding of co-occurrence with currently unregulated disinfection byproducts. (a) HAA5 includes: dibromoacetic acid, dichloroacetic acid, monobromoacetic acid, monochloroacetic acid, trichloroacetic acid. (b) HAA6Br includes: bromochloroacetic acid, bromodichloroacetic acid, dibromoacetic acid, dibromochloroacetic acid, monobromoacetic acid, tribromoacetic acid. (c) HAA9 includes: bromochloroacetic acid, bromodichloroacetic acid, chlorodibromoacetic acid, dibromoacetic acid, dichloroacetic acid, monobromoacetic acid, monochloroacetic acid, tribromoacetic acid, trichloroacetic acid.

UCMR 4 incorporates 30 contaminants (10 cyanotoxin chemical contaminants, 2 metals, 8 pesticides, 1 pesticide manufacturing byproduct, 3 brominated haloacetic acid groups, 3 alcohols, and 3 other semivolatile chemicals). Sampling for the UCMR 4 List 1 will occur from March 2018 through November 2020 for a randomly selected 800 surface water (SW) or groundwater under the direct influence of surface water (GWUDI) systems by the EPA. These systems would be selected for one component of UCMR 4 sampling (10 cyanotoxins or 20 additional chemicals). All samples taken from systems with 10,000 people or fewer will be paid for by the EPA.

Section 7

SECTION 7

ASSET MANAGEMENT

7.1 INTRODUCTION

Drinking water systems are comprised of many visible and hidden (i.e., buried) assets. Like many other communities, the Town of Ware has an aging water infrastructure system that is in the need of attention, whether it be through replacement and/or rehabilitation. The water system has grown to meet the needs of the town as they have arisen and continues to evolve, whether it be in response to growth, contraction, regulatory drivers or other reasons. As such, the management of the system is getting more complicated while at the same time the finances and human resources required for its management are stretched thin.

Due to this ever growing complexity, the Town should strongly consider the implementation of a formal asset management (AM) program. Although the Town is currently undergoing this master planning process, an asset management program would add another level of functionality to help it understand what it has and meet its level of service goals more efficiently.

Therefore, the purpose of this section is to introduce the concept of AM and identify some initial items the Town could begin to implement for a future AM program.

7.2 ASSET MANAGMENT

In general, an AM program would help you answer the following questions about your water system.

- What do I have?
- Where is it located?
- What condition is it in?
- How much life remains in the particular asset?
- How much will it cost to maintain and/or replace the asset?
- What Level of Service (LOS) does the system need to provide?

- Which of my assets are critical?
- What happens if they fail?
- How much capital do I need to maintain my desired LOS and when do I need it?
- How can I finance the capital needs?

If these questions cannot be easily answered, a well done and comprehensive AM plan will provide the means to do so with the flexibility to continue to do so into the future as well. Combining that with a computerized maintenance management system (CMMS) would also help a municipality provide its critical services more efficiently and cost effectively.

In summary, AM programs incorporate life-cycle cost analysis, level of service (LOS) planning and criticality (via likelihood of failure and consequence of failure analyses) to build a capital improvement plan that is sustainable and affordable. There are many customized software programs (such as Viewworks™), that are true AM programs that can help integrate these objectives too.

This master plan has used the traditional waterworks practice analyses to prioritize current and future needs in a 10-year capital improvement plan (CIP). This approach delivers a comprehensive, prioritized CIP for that period that will need to be routinely updated as improvements are made and/or new needs identified. However, the WDPW may wish to expand this methodology in the future via an AM program. To assist Town's like Ware begin or implement an AM program, the MassDEP has a recently implemented a competitive grant program that is understood to provide funding over the next several years.

7.3 AREAS FOR ASSET MANAGEMENT

Based on our initial understanding of the WDPWs system, the following items were initially identified for consideration to gather information that would be needed for the efficient creation of an AM program:

- Hydraulic Model – The Town currently has a hydraulic model which was created with the GIS compatible InfoWater hydraulic modeling software as manufactured by

Innovyze. The Town should consider further populating the model's database of water system components to also include attributes for the age of the water mains (i.e., year installed), location of valves, location of hydrants, etc. Additional information could also be included for the valves and hydrants such as their year installed, manufacturer, model, opening direction, etc.

- Geographic Information System (GIS) - Building out a GIS system with links to record documents. In doing this, the Town could efficiently access record drawings, water service tie cards, etc. from an efficient interface. Laptops, tablets, or other mobile devices could implemented be for field personnel to easily access and/or modify the information. This technique would allow for hyperlinking of engineering plans and long-term preservation of old paper document records through scanning. The GIS system could then be expanded in the future to incorporate and hyperlink photographic records, construction documents and other desirable information when resources are available.
- Computerized Maintenance Management System (CMMS) - A CMMS is a newer innovation to improve inventory management, real-time maintenance and sustainability of treatment and distribution system assets. A CMMS system is a software package that is configured to track run-time operation of assets and to plan preventative maintenance. Many vendors offer customized CMMS packages. Often a CMMS module can be added to a supervisory control and data acquisition (SCADA) system in a treatment facility or at a central operating node to track real-time operation time and data to plan preventative/routine maintenance, inventory management and operations budgeting. As the WDPW transitions into the construction of a new water treatment plant, a formal CMMS should be considered for incorporation into the project.

Section 8

WRIGHT-PIERCE
Engineering a Better Environment™

SECTION 8

RECOMMENDATIONS

8.1 GENERAL

The intent of this section is to provide an overview of the recommendations made for the Ware Department of Public Works' (WDPW's) system within the previous sections of this report along with their estimated costs where applicable. The details of each recommendation can be found in the corresponding sections within this report. The prioritization and scheduling of recommendations into a ten-year Capital Improvement Program (CIP) is presented within Section 9.

8.2 WATER SUPPLY

As presented within Section 4 of this report, the WDPW's existing sources were evaluated under various scenarios utilizing standard water works practices. All sources were determined to be capable of meeting the projected average day demand (ADD) and maximum-day demand (MDD) for the planning period. But if the largest well was considered to be off-line, then the system would not be capable of meeting the projected 2025 maximum-day demands for the planning period with pumping limited to 16 hours of operation. There would be a deficit of approximately 0.031 million gallons per day (MGD). They were however, determined to be capable of meeting the projected maximum-day demands for the planning period when operated for 17 hours.

8.2.1 New Source of Supply

Possibilities for additional supply included interconnections with neighboring communities, an interconnection with a large water supplier (e.g., the Massachusetts Water Resources Authority), or the implementation of a new well source or sources. As previously noted within Section 4, the Town does not currently have any interconnections with neighboring communities or an interconnection with a large water supplier. Due to the high cost associated with the implementation of these two options, these interconnections are not desired nor recommended at this time. The last option for a new source of supply would be to implement a new well source or sources. Since the Town is currently operating their existing well sources with enough capacity to supply the projected ADD and the MDD for 2025, this option is currently not recommended. Also, it is not guaranteed that the new well source would have better water quality than the existing wells.

8.2.2 Optimization of Existing Supply

Over time, well performance is influenced by many factors that can contribute to a steady and sometimes rapid decline in hydraulic performance. When this occurs, cleaning and well redevelopment is required to remove the materials plugging the well and screen via mechanical and chemical rehabilitation. Cleaning and redevelopment of each well is recommended when the specific capacity of the well drops no more than 10% from the last cleaning. The effectiveness of a well cleaning is also reduced when the well yield is allowed to decline for too long between cleanings. This often results in the inability of the well to regain its original construction hydraulic performance. Therefore, when significant well performance is lost and/or the cleaning frequency becomes too costly, a replacement well needs to be considered.

As previously discussed in Section 2, Wells No. 2 and 3 from the Wellfield source experienced a decline in capacity over the past ten years. Therefore, the WDPW constructed two replacement wells (Wells No. 2R and 3R) in 2015. The intent is to fully replace the current existing Wells No. 2 and 3 with these replacement wells. The replacement wells were recently approved by MassDEP in 2016 and they will be connected to the water system based on the recommendations per the ongoing Treatability Study being performed by Wright-Pierce (i.e., piloting of the Barnes Street sources).

The remaining sources in the Ware water system have not shown any decline in hydraulic performance at this time, but it is recommended that the WDPW continue with a routine well cleaning and redevelopment program for its wells on an as needed basis.

8.2.3 Treatment Needs

As described within the previous sections, the Wellfield, Well No.4, and the Cistern sources are currently being chemically treated at the Pump House and the Dismal Swamp Well is being chemically treated individually. At both locations, the water is treated with potassium hydroxide (KOH) for pH adjustment and sodium hypochlorite (NaOCl) for disinfection.

Historically, iron and manganese have been causing water problems and chronic consumer complaints within the Town of Ware. Both the iron and manganese concentrations have been exceeding their corresponding SMCL of 0.30 mg/L and 0.05 mg/L, respectively. Therefore, additional treatment is desired to be implemented at the sources. Currently, Wright-Pierce is performing a Treatability Study to determine design parameters for a new water treatment plant (WTP) that would remove these secondary constituents. The process would include piloting for technology verification, then proceed with permitting and design, and finally to construct the WTP. For this entire process, the WDPW should plan for a period of approximately two to three years until the WTP is in operation.

As previously discussed in Section 4, it is recommended that the WDPW proceed with Option No. 3 which consists of treating only the Barnes Street sources. The Barnes Street sources includes the Wellfield (Wells No. 1, 2, and 3), Well No. 4, and the Cistern which when combined could supply the Town with an approved maximum daily rate of 1.80 MGD. Based on the water use projections for the planning period from Section 3 of this report, the new WTP for the Barnes Street sources would be able to reliably provide for the system's projected average-day demand of 0.79 MGD and also the system's projected maximum-day demand of 1.37 MGD.

As part of the Treatability Study, the Barnes Street sources will be pilot tested with a GreensandPlus™ media for removal of the excess iron and manganese in October of 2016. If successful results are obtained, the new WTP will be designed around this treatment.

The Dismal Swamp Well source shall remain as it currently exists and could be used as a back-up source. As noted previously, the Barnes Street sources alone can meet the Town's water demand needs and should the system grow significantly, treatment can be added at the Dismal Swamp Well (if needed).

If treatment is not provided (for removal of secondary constituents) at the sources that are used, iron and manganese would still be an issue in the Town's water system and consumer complaints will continue.

A budget for the cost to construct a new WTP to treat the Barnes Street sources will be prepared as part of the Treatability Study. However, a range of possible costs that should be anticipated for the construction of a new 1.8 mgd WTP that utilizes GreensandPlus™ media for treatment is estimated to be between $3.5M to $5.0M.

8.3 DISTRIBUTION SYSTEM

The WDPW's distribution system was evaluated to assess its hydraulic adequacy utilizing its computerized hydraulic model. Various improvements were recommended to improve fire flow capacity, water main replacements, water storage tank mixing, and SCADA upgrades. Each is summarized in the sections that follow. Figure 8-1 presents an overview of the recommended distribution system improvements.

Data from MassGIS and Town of Ware
ISO Improvement 1
Expansion of the existing boosted pressure zone to include Upper Church Street.
Church St. Tank
Capacity: 1.5 MG
Overflow: 659 FT
Barnes Street Sources
(Wells 1,2,3,4, & Cistern)
ISO Improvement 3
Replacement of existing 12-inch cast iron water main with 12-inch ductile iron on Church St. between Prospect St. and Church St. Tank (~3,000 LF)
ISO Improvement 2
Replace existing 10-inch cast iron water main with 12-inch ductile iron on Church St. between Pleasant St. and Prospect St. (~800 LF)
ISO Improvement 4
Replace existing 6-inch cast iron water main with 8-inch ductile iron on Mechanic St. (~2,000 LF)
Legend
Tank
Well Source
Pipe
Diameter
less than 4"
6"
8"
10"
12"
Green St
Sorel Rd
Walker Rd
Sheehy Rd
Pond Brook Park
Old Gilbert
Upper Church St
Walter Dr
Greenwich Rd
Crescent St
Hazel Ave
Pearl St
Highland Village
Boulder Dr
Wrin St
Stoney Point Dr
Belmont St
Convent Hill Rd
Church St
Sherwin St
Walnut St
Smith Ave
Gareau Ave
Aspen St
School St
Laurier Ave
Cottage St
Castle St
Bellevue Ave
Dale St
Aspen Ct
Spring St
Prospect St
Vigeant St
North St
High St
Pleasant St
Park St
Park Ave
Barnes St
Eddy St
Storrs St
Parker St
Bank St
W Main St
Main St
E Main St
Otis Ave
Canal St
Grove St
Pine St
Cherry St
East Ct
East St
Prendville Rd
Ross Ave
West St
Webb Ct
Buckley Ct
Pulaski St
Maple St
Knox Ave
Maple Ct
Maple Ave
Milner St
Morse Ave
Quarry St
Off Monroe St
Clifford Ave
Elm St
Mechanic St
Demond Ave
South St
Union St
Greenway Ave
Hospital St
Chestnut St
Glendale Cir
N
0 250 500 Feet
Sources: Esri, HERE, DeLorme, USGS, Intermap, increment P Corp., NRCAN, Esri Japan, METI, Esri China (Hong Kong), Esri (Thailand).
Proposed Improvements
Ware, MA
PROJ NO: 13471A
DATE: 10/18/2016
WRIGHT-PIERCE
Engineering a Better Environment
FIGURE: 8-1

8.3.1 ISO Fire Flows

As presented within Section 5 of this report, the system's residential Insurance Services Office (ISO) locations were adequate in terms of being able to provide the needed residential fire flows. However, a number of locations within the system were determined to be deficient. In total, the 14 ISO fire flow test locations were evaluated using the hydraulic water model and 10 were found to be deficient. By implementing the improvements for each ISO described previously in Section 5, all of the commercial deficiencies would not be solved. In order to solve these deficiencies and to provide the largest increase in available fire flow, then the following recommendations with associated capital costs are recommended to be implemented:

1. Expansion of the boosted zone around Church Street Tank to include the 10 to 11 additional homes on Upper Church Street. A range of possible costs to implement this improvement would be estimated to be between $150,000 and $200,000.
2. Replacement of approximately 800 linear feet of 10-inch cast iron pipe on Church Street between Pleasant Street and Prospect Street with new 12-inch ductile iron pipe (ISO #7). This would have an estimated capital cost of $160,000 assuming a unit capital cost of $200 per linear foot for 12-inch water main installed.
3. Replacement of approximately 3,000 linear feet of 12-inch cast iron pipe on Church Street between Prospect Street and the Church Street Tank (ISO #7). Confirmation of pipe condition is recommended prior to replacement to determine if hydraulic restriction is related to another cause (i.e. partially closed valve, mislabeled pipe size, or etc.). A total cost of $600,000 would be estimated to install the new water main assuming a unit capital cost of $200 per linear foot for 12-inch water main installed.
4. Replace approximately 2,000 linear feet of 6-inch cast iron pipe on Mechanic Street with new 8-inch ductile iron (ISO #11). This would have an estimated capital cost of $350,000 assuming a unit capital cost of $175 per linear foot for 8-inch water main installed.

A total estimated cost to complete all of these recommendations would be approximately $1,260,000 to $1,310,000. This estimate does not include any engineering design/permitting/ construction administration costs as they would vary based on actual scope. However, a 25% contingency would be suitable for initial estimating purposes.

8.3.2 Water Main Improvement Program

As previously noted in Section 5, it is recommended in the water industry to maintain an on-going water main replacement program where 1% to 2% of the total system length is replaced annually. As this approach would require large annual capital expenditures, replacing 2% of a distribution system annually can be very difficult financially. Taking into consideration the size of the WDPW system, it is recommended that at least 0.5% of the system is replaced annually. With Ware's currently system size of approximately 47 miles, approximately 1,240 linear feet per year would be recommended. Assuming a unit capital cost of $175 per linear foot of 8-inch water main installed, the total cost per year would be approximately $220,000. Based on this estimation, approximately 12,400 linear feet of new 8-inch pipe construction would be recommended to be completed over the next ten years. This would correspond to a total of approximately $2,170,000 within the a 10-year capital improvement plan period. This estimate does not include any engineering design/permitting/construction administration costs as they would vary based on actual scope. However, a 25% contingency would be suitable for initial estimating purposes.

Within the annual replacement program budget, the WDPW plans to complete a phased project to remove an older existing 6-inch cast iron water main on West Street with poor hydraulic capacity. Currently this street has 6-inch and 12-inch water mains that supply water to the customers. The project includes relocating the services from the 6-inch main to the 12-inch main and then eliminating the 6-inch main and any interconnections from the system.

8.3.3 Water Storage Tanks

Distribution storage is a valuable asset and critical component to a water distribution system. As previously discussed, adequate storage is required for a variety of operational needs such as to buffer peak demands of the system, provide volume for firefighting purposes, and volume for other emergency purposes. Properly designed storage facilities should incorporate each category of storage as required and be sited properly within the system to provide the greatest benefit to users and operators. When possible, systems should consider redundancy in storage to facilitate maintenance.

The WDPW system would have sufficient (i.e., excess) storage volume and adequate redundancy with its two water storage tanks if the booster pump station expansion is implemented as previously noted within Section 5. If not, then a new water storage tank with a usable storage volume of at least 0.57 MG would be needed. A construction cost for such a tank could vary between $1.0M and $2.0M or more depending on project specifics.

8.3.3.1 Mixing Systems

Due to the storage volume present within the WDPW's system in combination with the WDPW's operational practice of minimal tank level fluctuation, high detention times (i.e., water age) are created within the storage tanks. As high detention times can lead to detrimental water quality, it was recommended that a mixing system be implemented at each tank. As previously discussed in Section 5, there are two common types of tank mixing systems currently available for most tanks: passive and active. For passive systems, there is the TideFlex system and for active systems, there is the SolarBee Recirculation System, GridBee Recirculation System, and the PAX Mixing System. Based on each system's installation, estimated cost, and ease of future maintenance, the GridBee System would be preliminarily recommended for installation in Ware's two existing tanks as it can be easily installed in an undrained tank.

Estimated costs to implement the GridBee mixing process within the tanks is estimated to be as follows:

- 1.0 MG Anderson Road Standpipe: $50,000
- 1.5 MG Church Street Tank: $50,000

This estimate does not include any engineering design/permitting/construction administration costs.

8.3.3.2 Tank Repairs

The WDPW's two water storage tanks were last inspected in December of 2015 by Underwater Solutions Inc. (as noted within Section 2 of this report). It was determined through these inspections that both of the tanks were overall in generally good condition. There were a variety of items requiring maintenance and/or repair were identified for each of the tanks.

Summarized cost estimates from the inspection reports are as follow:

- 1.0 MG Anderson Road Standpipe: $20,000
- 1.5 MG Church Street Tank: $20,000

Underwater Solutions Inc. noted that they anticipate the interior and exterior surfaces of the tanks to be acceptable for approximately 4 to 5 more years, but after this time, the tanks would most likely require a recoating. It would cost approximately $800,000 to recoat each of the tanks (without engineering costs).

As painting of existing welded steel tanks can be costly, another option for the WDPW to consider would be the replacement of the two tanks with a newer tank. This would also allow the WDPW to determine if changing the volume of the tanks or increasing the hydraulic gradeline of the system would be possible/beneficial. It is understood that the Town has received conceptual cost estimates for two new tanks (two 800,000 gallon tanks) for approximately $1.4M (not including engineering). If desired, the Town should further pursue this analysis in additional detail.

8.3.3.3 SCADA Upgrades

Ware's water system currently does not have a modern SCADA system. As discussed previously, each source is run by a Hand/Off/Auto (HOA) switch which is ultimately controlled by a soon to be obsolete tank level telemetry. Therefore, the water system should be upgraded with a modern SCADA system. This would provide the Town with increased reliability, a higher level of service for consumers, increased efficiency, and optimized labor. A budget of $280,000 is estimated for the implementation of the new SCADA system. This budget includes communication panels with radio telemetry being implemented at each of the wells and both of the tanks, a control panel at the new Barnes Street WTP, a panel with radio telemetry at the DPW Office to be tied in remotely, and the installation and licensing costs. The budget can vary based on the scope. This estimate does not include any engineering design/permitting/construction administration costs.

In general, all costs are estimated based on limited information that is currently available and are presented as year 2016 dollars. All costs should be re-visited and revised as necessary when additional detail is available and prior to when the project is anticipated to move forward.

Section 9

Wright-Pierce
Engineering a Better Environment

SECTION 9

RECOMMENDED CAPITAL IMPROVEMENT PROGRAM

9.1 OBJECTIVE

The Ware Department of Public Works (WDPW) has an aging infrastructure that is in need of attention, either through replacement or rehabilitation. As there are many needs that have been identified in the WDPW's future, a well laid out Capital Improvement Plan (CIP) will help the WDPW prioritize the new needs and plan for their implementation. This final section is the culmination of all others from this report and presents a ten-year CIP for the WDPW's moving forward. The estimated capital costs for the newly identified needs are presented. Routine costs for operation and maintenance are not included.

9.2 CAPITAL IMPROVEMENT PROGRAM

The proposed CIP has been developed from analyses presented in this report. A summarized description of each improvement was previously presented within Section 8. The improvements and recommendations are prioritized later in this section.

In addition to the categories of priority discussed below, the improvements can simply be classified as either Maintenance driven or Demand driven (as a result of anticipated growth). In general,

- Maintenance driven improvements are projects recommended which specifically address deficiencies in the system. The treatment of existing sources or meeting regulatory needs can be considered to be within this category.
- Demand driven improvements are projects which will be required to satisfy projected growth and associated demands.

At this time, the majority of the recommendations for the WDPW's system are maintenance driven. Over time, the WDPW may have to shift the priority of projects in order to respond to the needs of the community and/or to take advantage of opportunities such as roadway reconstruction projects or new developments as they are identified. It is important that the WDPW revisit the recommendations yearly to re-prioritize, schedule and budget the recommended projects as needs are confirmed or modified.

All of the identified improvements have been prioritized within high, intermediate, and low-priority categories that are described in more detail in the following sections.

The proposed 10-year CIP is presented within Table 9-1 at the end of this section. The initial layout is spread out within the next 10-year window based on our current understanding of needs and is subdivided within the Supply and Distribution categories. Due to their magnitude in cost, the currently on-going Treatability Study and thereafter, the new water treatment plant (WTP) for the Barnes Street sources is included.

9.2.1 High Priority Improvements

The highest priority improvements are generally the projects which have been identified for completion during the next three years and include the following:

- ***Barnes Street WTP*** - The Barnes Street WTP as it is currently underway with piloting (and a high priority driven by regulatory need for manganese removal).
- ***Tank Mixing Systems and Repairs*** - The implementation of mixing systems and tank repairs are identified as high priorities since improved mixing and turnover within the existing tanks will be important once the new water treatment plant is put on-line and significantly improved water quality is pumped into the distribution system. The repairs are also recommended to be performed at the same time as the mixing system installation so as to avoid two separate costlier periods of down time. These should be scheduled to occur prior to completion of the WTP
- ***Security Camera at Cistern*** **–** For additional security at the Cistern source, a security camera is intended to be installed within the next few years. This improvement can be incorporated within the Barnes Street WTP project.
- ***Well No. 4 House Acquisition*** **–** Currently there is a residential house located at 116 Pleasant Street that is within Well No. 4's Zone I. The house has been recently put on the

market to be sold and it would be advantageous to acquire this land. The acquisition of the house would be considered a high priority improvement since this acquisition would help protect the Town's source.

9.2.2 Intermediate Priority Improvements

The intermediate-term improvements identified are generally recommended for completion during the middle portion of the 10-year CIP (from approximately year three to year six). The intermediate term projects include the following:

- ***Tank Recoating*** **–** Both of the water storage tanks will need to be recoated in approximately 4 to 5 years. Since painting the existing tanks can be costly, another option for WDPW to consider would be the replacement of the two tanks. The cost would be comparable and it would also allow the WDPW to determine if changing the volume of the tanks or increasing the hydraulic gradeline of the system would be possible/beneficial.
- ***SCADA*** - The implementation of a new SCADA system is classified as an intermediate priority improvement. All of the sources are currently run by Hand/Off/Auto (HOA) switches and are controlled by tank telemetry. Upgrading the water system with SCADA would provide increased reliability, a higher level of service, increased efficiency, and reduced labor. However, this could be implemented more quickly if tied in with the WTP project.
- ***Generator at Dismal Swamp Well, Well No. 4, and Booster Pump Station*** **–** Should the WDPW desire to have full emergency power provisions, suitable generators would need to be installed at the Dismal Swamp Well, Well No. 4 and the Booster Pump Station at the Church Street Tank. This is considered an intermediate priority since the WDPW currently has adequate provisions for emergency power according to MassDEP requirements.

9.2.3 Low Priority Improvements

Although improvements to fire flow capabilities and other distribution system piping projects can be considered to be intermediate-term improvements, they have been allocated to the lower-priority improvements for the time being due to the large capital expenditures that the WDPW will be incurring over the next few years.

ISO Improvements:

As discussed in Section 5, four recommendations were provided to solve all of the commercial deficiencies and to provide the largest increase in available fire flow in Town. These recommendations include the expansion of the boosted zone around the Church Street Tank and the replacement of approximately 5,800 linear feet of water main. At a total value of approximately $1,260,000 to $1,310,000, these costs have been equally distributed between years 5 through 10 to provide flexibility in phasing.

Annual Water Main Improvement Program (WMIP):

Taking into consideration the size of the WDPW system, it is recommended that at least 0.5% of the system is replaced annually. With Ware's current system size of approximately 47 miles of water main, approximately 1,240 linear feet per year would be recommended. Assuming a unit capital cost of $175 per linear foot of 8-inch water main installed, the total cost per year would be approximately $220,000.

In summary, the improvement program is intended to be flexible and subject to adjustment and modification as needs change and evolve in the water system. Long-term projections should be reviewed and reevaluated periodically to assure that initial assumptions remain relevant and accurate. Specific annual scheduling of improvements within each major priority period should be reassessed annually with the WDPW's Capital Planning Committee to assure maximum financial benefit in any given year.

TABLE 9-1
RECOMMENDED TEN-YEAR CAPITAL IMPROVEMENT PROGRAM
WARE, MASSACHUSETTS

	Total Estimate	Year 1 2016	Year 2 2017	Year 3 2018	Year 4 2019	Year 5 2020	Year 6 2021	Year 7 2022	Year 8 2023	Year 9 2024	Year 10 2025
SUPPLY											
Existing Sources											
Dismal Swamp -Generator	$200,000					$200,000					
Cistern -Security Camera	$5,000		$5,000								
Well No. 4 -Generator	$200,000						$200,000				
Well No. 4 -House Acquisition	$200,000		$200,000								
Treatment											
Barnes Street WTP	$5,000,000			$3,000,000	$2,000,000						
DISTRIBUTION											
ISO Improvements											
Various	$1,260,000					$210,000	$210,000	$210,000	$210,000	$210,000	$210,000
WMIP Improvements											
Various	$1,320,000					$220,000	$220,000	$220,000	$220,000	$220,000	$220,000
Storage Improvements											
Mixing Systems											
Anderson Road	$50,000				$50,000						
Church Street	$50,000				$50,000						
Repairs											
Anderson Road	$20,000				$20,000						
Church Street	$20,000				$20,000						
Repainting											
Anderson Road	$800,000					$800,000					
Church Street	$800,000						$800,000				
BPS - Generator	$50,000						$50,000				
SCADA Upgrades	$280,000				$280,000						
TOTAL	**$10,255,000**	**$0**	**$205,000**	**$3,000,000**	**$2,420,000**	**$1,430,000**	**$1,480,000**	**$430,000**	**$430,000**	**$430,000**	**$430,000**

Note: As noted in text, costs presented do not include engineering phase services budgets. All cost estimates are presented in 2016 dollars.