Water Master Plan Update

for the

Acton Water District

Acton, MA




August 2018

WRIGHT-PIERCE
Engineering a Better Environment™

ACTON, MASSACHUSETTS
WATER MASTER PLAN UPDATE

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SECTION 1

INTRODUCTION

1.1 GENERAL

Wright-Pierce has been working closely with the Acton Water District (AWD) for the past several years on a variety of projects ranging from studies, water main infrastructure all the way to state of the art water treatment facilities. The purpose of this master planning document is to evaluate the components of the AWD's Public Water Supply System, make recommendations, and present the needed improvements in a well thought out and useful Capital Improvement Plan (CIP) that the AWD 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 Supply and Facilities: The existing Acton 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 Acton's historical water use and the projections for its water use through the next 10-year planning period (2017 to 2026).

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 Evaluation and Assessment: This section presents the detailed evaluation performed of the Acton 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 Acton system is presented.

Section 7 - Demand Management: This section presents the AWD's efforts in demand management.

Section 8 - Asset Management: Due to the increasing complexities of the AWD's infrastructure and processes, this section presents an initial assessment of the AWD's current asset management processes and how it can be optimized or supplemented for increased efficiency.

Section 9 - Recommendations: This section summarizes the recommendations made within the other sections and presents the corresponding estimated costs for their implementation.

Section 10 - Recommended Capital Improvement Program: This section lays out a proposed Capital Improvement Program (CIP) to be used by the AWD over the next several years as a guide for improvements that will allow it to meet its identified needs.

SECTION 2

EXISTING SYSTEM SUPPLY AND FACILITIES

2.1 OVERVIEW OF WATER SYSTEM

The Acton Water District (AWD) serves the Town of Acton, located in Middlesex County, Massachusetts. The AWD is a separate municipality from the Town. Acton is bordered by the Towns of Westford and Carlisle to the north, the Towns of Littleton and Boxborough to the west, the Town of Concord to the east, and the Towns of Maynard, Stow, and Sudbury to the south. State Route 2 is the main transportation corridor in town and bisects the Town in an east to west direction. The Town has a population of approximately 22,300 people. The water system has service elevations ranging from approximately 125 feet to 375 feet above mean sea level.

The AWD owns and operates the water system which serves residential, commercial and municipal users. The District currently serves approximately 6,662 water customers consisting of 6,240 residential users, 320 commercial users, and 102 municipal users. Based on 2016 data, the average day demand is approximately 1,633,000 gallons per day (gpd) and the maximum day demand is approximately 2,660,000 gpd.

The Acton water system includes 11 active ground water sources (consisting of 36 wells) treated at five water treatment facilities, four water storage tanks (one with a booster pump station due to its lower hydraulic grade line), and approximately 135 miles of water main. An overview of the water supply is included as Figure 2-1. Figure 2-1 also presents the distribution system color coded by water main diameter, and Figure 2-2 presents the distribution system color coded by water main material type. A brief summary of each water system component is provided in the following sections.

Water Distribution System By Diameter
Acton Water District
Acton, Massachusetts

PROJ NO: 13748A | DATE: July 2018 | FIGURE: 2-1

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Water Distribution System By Material
Acton Water District
Acton, Massachusetts

PROJ NO: 13748A	DATE: July 2018	FIGURE: 2-2

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2.2 SUPPLY FACILITIES

The Acton Water District provides water to its customers from eleven active source locations consisting of thirty-six individual wells located throughout the Town of Acton. Design parameters and physical properties of each well are included in Table 2-1.

TABLE 2-1
EXISTING WATER SUPPLY SOURCES

Source	Address	No. Wells	Type	Size	Depth*	Year Constructed	Maximum Approved Withdrawal*	Source Code
Assabet No. 1 (inactive)	284 & 290 High Street	1	Gravel Packed	48” x 24”	68’	1970	350 gpm	2002000-05G
Assabet No. 1A (replacement well)	284 & 290 High Street	1	Gravel Packed	24” x 18”	61’	2004	350 gpm	2002000-26G
Assabet No. 2 (inactive-emergency)	330 & 332 High Street	1	Gravel Packed	12”	59’	1972	350 gpm	2002000-06G
Assabet No. 2A (replacement well)	330 & 332 High Street	1	Gravel Packed	18” x 12”	35’	2000	350 gpm	2002000-19G
Assabet No. 3 (inactive)	Powder Mill Road	1	Gravel Packed	24”x 18”	65’	1965	360 gpm	2002000-0BG
Christofferson Well	313 School Street	1	Gravel Developed	8”	40’	1964	278 gpm	2002000-04G
Clapp Wells 102 & 302	694 Mass. Avenue	2	Gravel Packed	18” x 12”	32’ & 34’	1976	245 gpm	2002000-24G-25G
Conant No. 1	595 & 605 Main Street	1	Gravel Packed	24”	31’	1955	325 gpm	2002000-02G
Conant No. 2, Wells No. 1-5	Post Office Square	5	Gravel Packed	18” x 12”	28’	1998	300 gpm	2002000-14G -18G
Kennedy Wells No. 1-4	960 & 962 Main Street	4	Gravel Packed	18” x 12”	31.5’-40’	1989	375 gpm	2002000-10G -13G
Lawsbrook Well	62 Lawsbrook Road	1	Gravel Packed	48” x 24”	53’	1960	105 gpm	2002000-03G
Marshall Tubular Wellfield	953 & 955 Main Street	15	Tubular Wellfield	2-1/2”	31’	1989	208 gpm	2002000-09G
Scribner Wellfield (inactive)	22 Lawsbrook Road	18	Tubular Wellfield	2-1/2”	26’-35’	1981	240 gpm	2002000-08G
Scribner Wells No. 1- 4 (replacement wells)	22 Lawsbrook Road	4	Gravel Packed	16” x 10”	37.5’ - 38’	2002	105 gpm	2002000-20G-23G
Whitcomb Well	693 Mass. Avenue	1	Gravel Packed	48” x 24”	34.5’	1970	245 gpm	2002000-01G

* As reported in the 2016 ASR.

The Christofferson, Lawsbrook, and Scribner sources are commonly referred to as the School Street sources (as this is where their previous common treatment facility was located, but these sources are currently being treated at the South Acton Water Treatment Plant) and the Assabet Wells (No. 1, 1A, 2, 2A, and 3) are commonly referred to as the Assabet sources.

All of the Town's sources are located in the Concord River Basin as designated by the 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 a yearly average volume. A copy of the WMA Registration Statement and Permit is included in Appendix A.

Registered sources are sources that were installed prior to January 1, 1988. These sources do not have individual restrictions beyond the safe yields of the supply and are not subject to the same conditions as permitted sources. WMA issued permits for water supplies that were permitted after 1988 and these permits limit the withdrawal for the water system and individual sources.

2.2.1 Assabet Wells

The Assabet sources consist of Wells No. 1, 1A, 2, 2A, and 3. Wells No. 1A and 2A are currently the only active wells and they were constructed in 2004 and 2000, respectively, as replacement wells for Wells No. 1 and 2. All of these wells are located in the south-east corner of Acton (north of the Assabet River).

The original Assabet Well No. 1, which is currently inactive, is a 24-inch by 48-inch gravel-packed well 68 feet deep, constructed in 1970. The well originally was capable of producing 350 gpm. However, due to a decline in well capacity and water quality, a replacement well was installed (Assabet Well No. 1A). Assabet Well No. 1 is shown in the picture to the right.

Assabet Well No. 1A was constructed in 2004 and was equipped with a submersible pump in a pitless adaptor. The well is a 24-inch by 18-inch gravel pack well. The well screen is 10 feet of 220/140 slot. The well has had manganese concentrations above the corresponding Secondary Maximum Contaminant Limit (SMCL) of 0.05 milligrams per liter (mg/L) for the past several years (as high as 0.555 mg/L in February 2017) as well as some elevated iron concentrations above the corresponding SMCL of 0.30 mg/L (as high as 0.57 mg/L in February 2016). The original pumping capacity of the well was 505 gallons per minute (gpm) and the original specific capacity was 36 gpm/ft. The source has a permitted pumping capacity of 0.499 Million Gallons per Day (MGD) or approximately 347 gpm.

The original Assabet Well No. 2 is 12 inches in diameter and 53.8 feet deep and is located adjacent to Turtle Pond (building shown in the photo to the left). Due to frequent plugging and decreased pumping capacity, a replacement well was installed 50 feet away. The replacement well, Assabet Well No. 2A, is an 18-inch by 12-inch diameter gravel-packed well approximately 35 feet deep that was constructed in May 2000. The well has had manganese concentrations above the corresponding SMCL of 0.05 mg/L for the past several years (as high as 0.27 mg/L in May 2011) as well as iron concentrations at or near the corresponding SMCL of 0.30 mg/L (as high as 0.14 mg/L in February 2016). The source also has a permitted pumping capacity of 0.499 MGD or approximately 347 gpm.

The existing Assabet Well No. 1 is currently inactive and would not be available in an emergency. The existing Assabet Well No. 2 is also inactive; however, it could be utilized in place of the replacement well in an emergency.

Flow from Assabet 1A is monitored through a magnetic flow meter at Assabet Well 1 Building and flow from Assabet 2A is monitored through a magnetic flow meter at Assabet Well 2 Meter Vault. The discharge piping from each well is combined into a 10-inch diameter transmission main into the South Acton WTP to be treated. An 8-inch magnetic flow meter located at the WTP is used to measure combined flow from the Assabet wells.

Assabet Well No. 3 (previously referred to as the W. R. Grace Well #3) was installed in 1965 as a 24-inch by 18-inch gravel packed well to a depth of 65 feet. It is still in its original state and has not been upgraded or connected to the AWD water system. Approval for Assabet Well No. 3 was received from MassDEP in February 2009. Based on available water quality data, the well is understood to have elevated manganese concentrations (above the corresponding SMCL of 0.05 mg/L) and that its 1,4-dioxane concentration [0.56 micrograms per liter (µg/L) in 2007] was also above the Massachusetts Drinking Water Guideline of 0.3 µg/L. Assabet Well No. 3 is currently inactive. The new South Acton WTP is also equipped to accommodate the flow from Assabet Well No. 3 in the future.

2.2.2 Clapp/Whitcomb Wells

The Clapp Wells No. 1 and 3 (shown in the picture on the right) consist of two 12-inch diameter gravel packed wells, one 32 feet deep and one 34 feet deep. The wells, installed in 2003, are currently permitted for a maximum authorized daily withdrawal rate of 245 gpm (0.35 MGD) based on the Zone II delineation. The two Clapp high lift pumps were replaced in 2016 (one in July and the other in October). Both pumps are a Goulds 9 RCHC 6 stage pump designed for 450 gpm at 285' TDH with a 50 HP motor.

Clapp Wells No. 1 and 3 are replacement wells for the Clapp satellite wells. The original Clapp satellite wells were replacement for the single gravel packed well formerly called the Erickson Well.

The Whitcomb Well (shown on the right) is a 48-inch by 24-inch gravel packed well having a depth of approximately 34.5 feet and was constructed in 1970. The Whitcomb Well site is also currently permitted for a maximum authorized daily withdrawal rate of 245 gpm (0.35 MGD) based on the Zone II delineation.

The Clapp/Whitcomb WTP (shown on the next page) treats raw water from the Whitcomb Well and the Clapp Wells. Treatment includes aeration for the removal of VOCs and pH adjustment followed by treatment with activated carbon for color and organics removal. After aeration the water is held in a clearwell, then water is pumped via two Goulds 6-stage pumps rated for 450 gpm at 285’ TDH through carbon filters in series (flow sequence is changed at each backwash) and to the distribution system. The Goulds pumps were recently replaced on October 2016. In addition, sodium hypochlorite and sodium fluoride are added for disinfection and fluoridation, respectively. In an effort to further raise the pH of the finished water, the AWD reinstalled a potassium hydroxide feed system at the sources in 2016. The facility has a combined pumping capacity of approximately 500 gpm (0.72 MGD).

Neither the wells (Clapp and Whitcomb) or the Clapp/Whitcomb WTP have any emergency generator provisions.

2.2.3 Conant Well No. 1

Conant Well No. 1 is located off of Main Street/Route 27 between Great Road and Brook Street. It is generally located in the central part of the AWD's distribution system, and this naturally developed well was constructed in 1955. Conant Well No. 1 has chemical treatment at the well and pumps directly to the distribution system via a 40 HP pump capable of up to 325 gpm. Treatment at the well includes potassium hydroxide for pH adjustment, sodium hypochlorite for disinfection, and sodium fluoride for fluoridation. It is about one mile away from the nearest source (Conant Well No. 2) and miles away from any other source and the two WTPs (North Acton and South Acton WTPs). The well has a maximum authorized daily withdrawal rate of 325 gpm (0.46 MGD) based on the Zone II delineation; however, due to poor water quality the well is generally pumped at a rate of approximately 120 gpm and has historically only been used during high demand periods. For the past several years, the Conant Well No. 1 has pumped between approximately 16.7 MG to 53.4 MG a year (approximately 3% to 9% of the annual total).

On July 7, 2015, the AWD received notice from MassDEP that the finished water manganese concentration in its Conant Well No. 1 was over the MassDEP's Office of Research and Standards Guidance Level (ORSGL) or 0.30 mg/L. The notification presented several actions that the AWD would need to take and included the requirement to prepare and submit a Compliance Plan and Corrective Action Plan to reliably and consistently reduce the manganese concentration to below the ORSGL at the entry point to the distribution system. This Compliance Plan was submitted in September of 2015 and the Corrective Action Plan was subsequently submitted in December of 2015. Several options were considered and evaluated to determine a feasible solution to reliably and consistently reduce the manganese concentration. The AWD is currently evaluating the potential to pilot suitable treatment processes in 2018.

The Conant Well No.1 pumping station does not have any emergency generator provisions.

2.2.4 Conant Well No. 2

Conant Well No. 2 consists of five 18-inch by 12-inch gravel packed wells with depths ranging from 25 to 32 feet. The wellfield was constructed in 1999 along with the WTP and is located off Route 27 near Brook Street (near Post Office Square). Submersible well pumps are installed in each of the five wells. The wells are currently permitted for a maximum authorized daily withdrawal rate of 150 gpm (0.216 MGD) in accordance with the WMA Permit. The combined raw water from all five wells is treated with aeration for VOC removal and pH adjustment, chlorine

for disinfection, and sodium fluoride for fluoridation. The treated water is pumped from the clearwell to the distribution system via a single Goulds 10 WLAC 7-stage pump pump capable of 300 gpm at 320’ TDH. The Goulds pump was recently replaced in May 2017. Potassium hydroxide for pH adjustment is not currently utilized at the source.

Conant Well No. 2 source also does not have any emergency generator provisions for the wells or treatment facility.

2.2.5 School Street Sources

The School Street Wells include the Christofferson Well, Lawsbrook Well, and Scribner Wellfield. The three wells are permitted with a total pumping capacity of approximately 487 gpm (0.702 MGD).

The Christofferson Well (shown on the right) is an 8-inch diameter gravel-developed well with a depth of 40 feet and was constructed in 1964. It has a submersible vertical turbine pump which can deliver 417 gpm. Of the three School Street sources, it has the largest permitted pumping capacity at 278 gpm (0.4 MGD). However, due to the high manganese concentration in the raw water, it has historically been used only during high demand periods. It was classified as groundwater under the influence (GWUI) of surface water as a result of two consecutive moderate ratings for microscopic particulate analysis (MPA) in May of 2009 and April of 2010, and an Administrative Consent Order (ACO) was issued in September 2011 by the MassDEP. The ACO required the implementation of treatment at the source that would comply with the surface water treatment rule regulations. This is discussed in more detail within later sections of this report.

The nearby Lawsbrook (shown on the left) and Scribner Well (shown on next page) sources have historically had better water quality than Christofferson, but the wells have experienced some intermittent iron, manganese, or color levels. The Lawsbrook Well was constructed in 1960 and is a 48-inch by 24-inch diameter gravel packed well with a total depth of 53 feet. The well has a permitted pumping capacity of 0.151 MGD (approximately 105 gpm). The Lawsbrook Well is located adjacent to the Scribner Wellfield on the Acton/Concord town line.

Originally constructed in 1981, the Scribner Wellfield supply was comprised of eighteen 2-1/2 inch diameter tubular well points ranging in depth from 26 feet to 35 feet. In 2001, a replacement wellfield was constructed to include four 16-inch by 10-inch gravel packed wells with an approximate depth of 38 feet. The source also has a permitted pumping capacity of 0.151 MGD (approximately 105 gpm).

Raw water from the Christofferson Well, Lawsbrook Well, and Scribner Wellfield is pumped first to the School Street Pump Station wet well. These three sources were previously combined and treated at the School Street WTP. However, with the GWUI classification of the Christofferson source, a new membrane filtration WTP was constructed in South Acton. Vertical turbine pumps at the station now transfer the combined raw water from School Street to this new WTP. The flow rate of each individual well is measured with electromagnetic flow meters located at each well station. The combined flow from the School Street Wells is then measured via an electromagnetic meter prior to leaving the station. All raw water pumps are controlled through variable frequency drives (VFDs) to deliver the operator set flow rate while maintaining the water level in each well.

The South Acton WTP (shown on the right) was constructed and put online in June of 2015. The WTP was designed primarily for Surface Water Treatment Rule compliance (due to Christofferson's GWUI classification) as well as for the removal of iron and manganese from the School Street Wells and the Assabet Wells (1A, 2A, and 3). The treatment process consists of groundwater pumping, aeration for VOC removal and pH adjustment, chemical oxidation of iron and manganese using potassium permanganate, provisions for coagulation and flocculation of particles and organic components using polyaluminum chloride, provisions for supplemental pH adjustment with potassium hydroxide, removal of particulate using microfiltration membranes, disinfection and maintaining free chlorine residual using sodium hypochlorite, fluoridation using sodium fluoride, expansion provisions for the advanced oxidation process, finished water pumping, wash water processing through settling and recycling a portion of water, and solids collection and long-term storage in lagoons.

The filtration system consists of three skids of microfiltration membranes (Pall Aria AP-6), feed tanks and reverse filtration tanks, ancillary piping, valves, pumps and blowers. The membrane system is designed to treat the desired flow of 1.7 MGD using three skids, but is capable of processing 1.7 MGD of flow with only two skids active. Additional space within the filtration system is provided to expand to approximately 2.0 MGD (1,400 gpm) in the future.

Due to potential for organics in the raw water (based on historical raw water color results), provisions for a coagulant feed system were part of the design. If needed, the future coagulant to be used at the South Acton WTP is assumed to be Polyaluminum Chloride (PACL) which is currently used at the North Acton WTP and is the coagulant preferred by Pall.

Finished water is delivered to the distribution system from the clearwell by two vertically mounted horizontal split case centrifugal pumps (one active and one standby).

Two lagoons are provided at the South Acton WTP with a combined capacity of approximately 55,300 gallons. The lagoons function as holding lagoons, allowing the solids concentration to build under quiescent conditions. As the solids settle in the lagoons, the supernatant overflows the weir at the end of the lagoons and is filtered through sand beds before discharging into the ground. Solids collected in the lagoon will be periodically removed to an off-site disposal location. Additionally, a connection to the Town of Acton sanitary sewer was completed as an alternate solids processing option.

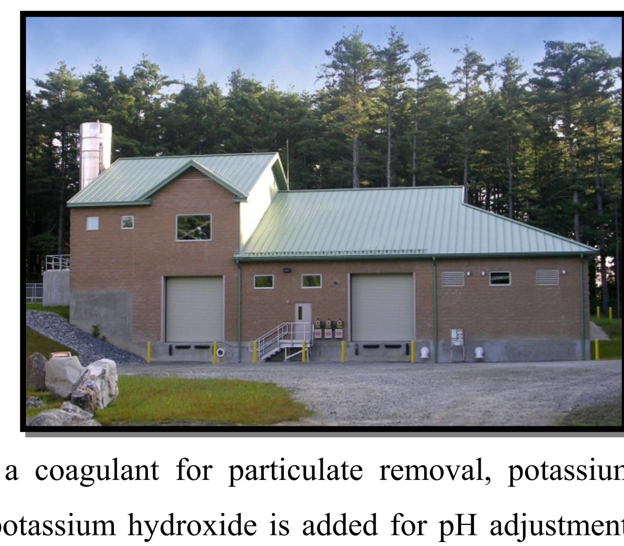
In the event of an electrical power outage at the facility, a 500-kilowatt (KW) Cummins diesel standby generator set, with an Automatic Transfer Switch was provided to automatically supply standby emergency electrical power to the WTP and Assabet Wells No. 1A and 2A.

Due to the presence of 1,4-Dioxane concentrations in the sources, additional space and process connection points were designed in case an Advanced Oxidation Process (AOP) would be required for its removal.

2.2.6 Kennedy and Marshall Wells

The Kennedy Wells consist of four 18-inch by 12-inch gravel packed wells that were installed in 1989. The wells range from approximately 31.5 to 40 feet deep and include a submersible well pump in each well. The Kennedy Wells have a WMA maximum permitted withdrawal of 0.540 MGD or approximately 375 gpm. The combined raw water from the Kennedy Wells is pumped to the North Acton WTP.

The Marshall Wellfield consists of eighteen 2½-inch tubular wells ranging in depth from 28 to 31 feet. The wells were installed in 1989 and include a vacuum prime system. Although the Marshall Wellfield has a WMA maximum permitted withdrawal of 0.300 MGD or approximately 208 gpm, the AWD is only able to pump approximately 75 gpm at this time which is possibly due to issues with the vacuum prime system and introduction of air into the system.



The North Acton WTP (shown on the right) is located at 960-962 Main Street. The WTP was constructed in 2009 to treat raw water from the Kennedy and Marshall Wellfields. The treatment process includes packed tower aeration for removal of VOCs and a Zenon ultrafiltration membrane for the removal of iron, manganese and natural organics (e.g., color). Polyaluminum chloride is added as a coagulant for particulate removal, potassium permanganate is added for metals oxidation, potassium hydroxide is added for pH adjustment, sodium hypochlorite is added for disinfection, and sodium fluoride is added for fluoridation. After filtration, the water enters the clearwell and the finished water is pumped to the distribution system by one of two 450 gpm 50 HP high head finished water pumps.

A 250 KW natural gas driven emergency generator was installed at the WTP that is capable of running one treatment train within the WTP along with the Kennedy Wells. No emergency generator provisions are located at the Marshall source.

2.3 DISTRIBUTION SYSTEM

2.3.1 Transmission and Distribution Mains

The distribution system consists of approximately 135 miles of water main predominantly ranging in diameter from 6-inch to 16-inch. A mix of 10, 12 and 16-inch main provide a north-south transmission grid. In general, the main service area is well looped; however, much of the pipe is small diameter (8-inch or smaller). A summary of the distribution system piping sorted by material type and pipe diameter is presented in Figures 2-3 and 2-4, respectively.

FIGURE 2-3
PIPE MATERIALS IN WATER DISTRIBUTION SYSTEM

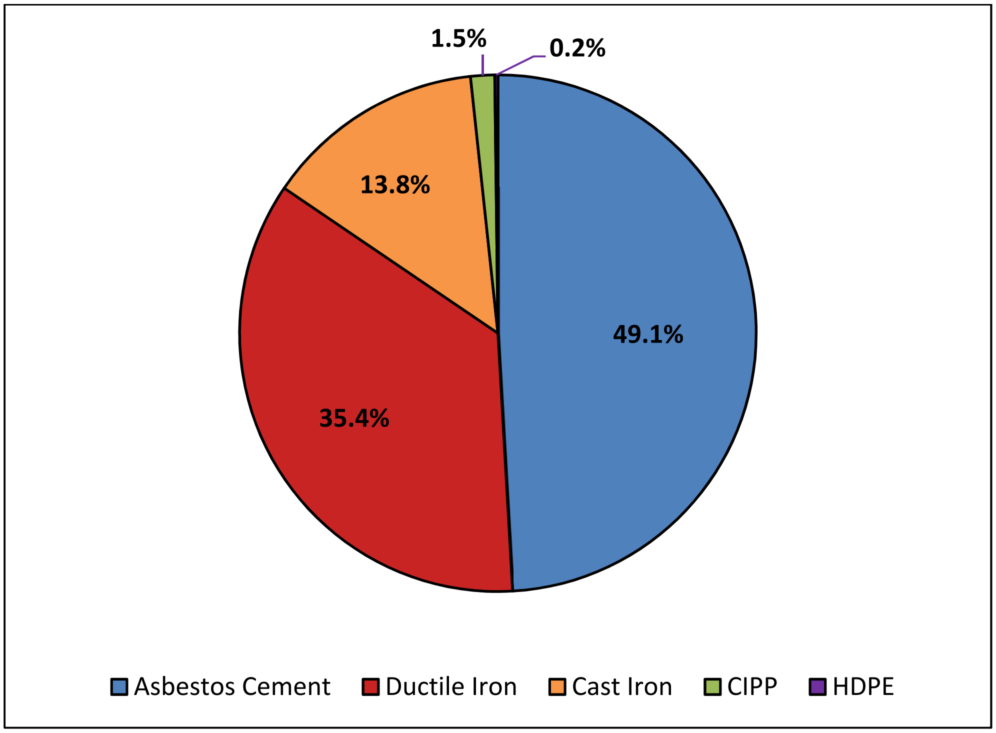
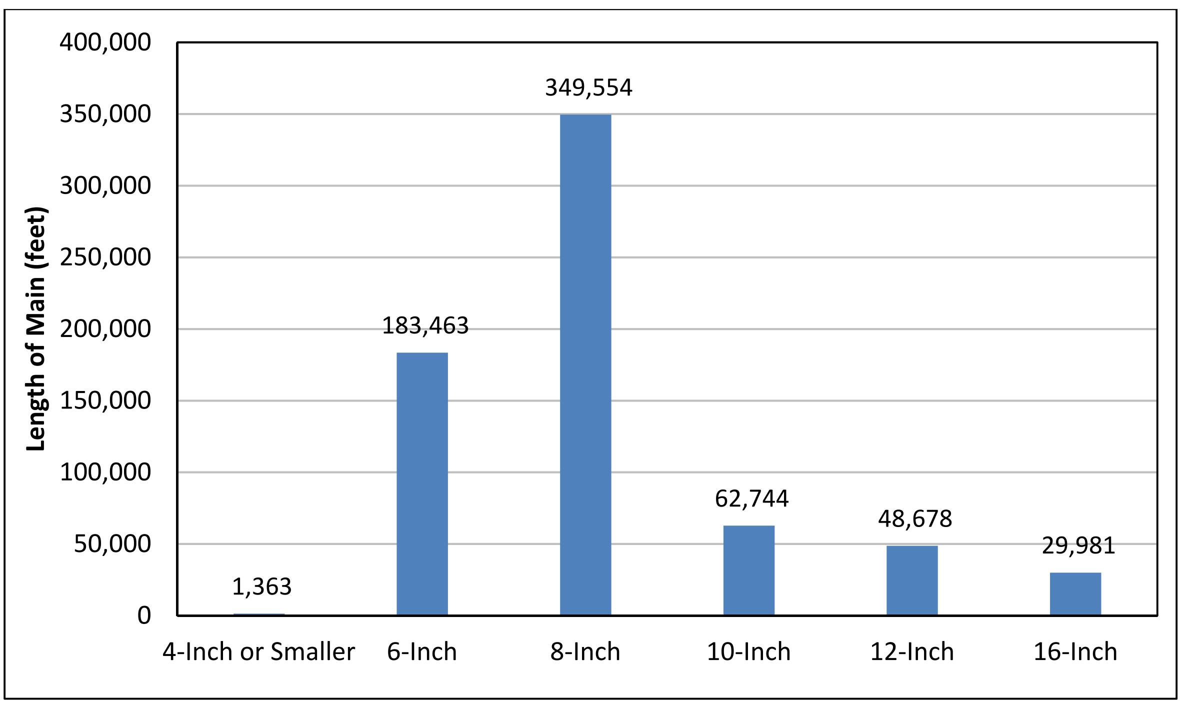


FIGURE 2-4
WATER DISTRIBUTION SYSTEM PIPE SIZE



The piping materials that are predominate in the Acton distribution system include the following:

- *Asbestos Cement (AC)* – Asbestos cement piping was readily available and typically installed in the 1960s and 1970s. As of 2017, approximately 49% of the distribution mains are AC including many primary transmission mains. 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 35% 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). The Acton Water System was established in 1912. At that time, unlined cast iron was installed. Over the years, some of the unlined cast iron has been field lined, or replaced with other materials. Based on system records, almost 14% 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.

- *Cured-In-Place-Pipe Lining (CIPP)* – The CIPP lining method consists of the reconstruction of existing water mains by the insertion of a flexible lining tube consisting of two concentric, tubular, woven seamless polyester jackets with a watertight polymeric membrane bonded to the interior layer. The AquaPipe liner was utilized for the CIPP method in AWD for approximately 10,160 linear feet within existing asbestos cement pipe in 2017. AquaPipe is a fully structural liner that complies with ASTM F1216 and ASTM F1743 Standards and is certified by NSF/ANSI Standard 61. This technology has been used within North America for the past 18 years and has been identified to be an economical and viable alternative to replacing water mains. Approximately 1.5% of AWD's distribution system is CIPP.

- *Other* – A minor amount of plastic and high density polyethylene (HDPE) piping exists within the system. This totals approximately 0.2% (1,306 feet) of the distribution system. Although not a part of the finished water distribution system, in 2015 approximately 10,700 linear feet of 10-inch PVC pipe was installed for raw water. This water main was installed in School Street, Parker Street, Independence Street, and Assabet Crossing in order to bring the raw water from the wells to the South Acton WTP to be treated.

Refer to the previous Figures 2-1 and 2-2 for Acton's water distribution system color coded by water main material type and pipe diameter.

2.4 INTERCONNECTIONS

2.4.1 Interconnections with Adjacent Communities

The Acton water system also has six emergency interconnections: two with Littleton, two with Maynard and two with Concord. One interconnection with Littleton is a single valve above ground at a booster pump station for Avalon Bay. The rest of the connections are two-valve assemblies buried in, or near, a public right-of-way. In general, the connections are closed valves at the Town lines and are opened on an as needed basis.

2.5 DISTRIBUTION STORAGE FACILITIES

Distribution storage facilities for the Acton Water District are comprised of one riveted-steel standpipe and three concrete reservoirs as summarized in Table 2-2.

TABLE 2-2
EXISTING DISTRIBUTION STORAGE FACILITIES

Name	Overflow Elev. (ft)	Height (ft)	Diameter (ft)	Capacity (MG)	Gal/ft	Type
Great Hill Standpipe	427.5	70.0	35	0.5	7,200	Steel
Flagg Hill Reservoir	427.5	25.0	117	2.0	80,500	Concrete
Nagog Hill Reservoir	427.5	52.5	99	3.0	57,150	Concrete
Wampus Hill Reservoir	317.0	26.0	140	3.0	115,150	Concrete

2.5.1 Great Hill Standpipe

The Great Hill Standpipe is a riveted steel tank constructed in 1916 that is located off Route 27 near Prospect Street. The 0.5 million gallon (MG) standpipe has an overflow elevation of 427.5 feet and is 35 feet in diameter and 70 feet high. The facility has an altitude valve. The Great Hill Standpipe is at the center of the distribution system and the oldest in the District.

The standpipe has two steel manways: one 24-inch by 18-inch inside diameter manway located 18 inches above the ground on the eastern side of the standpipe and one 30-inch inside diameter manway located 24 inches above the ground on the western side of the standpipe. The standpipe has a welded steel ladder with a safety cage from the roof dome to 16 feet above the ground. There is also a second ladder from the vent to the edge of the roof dome. The tank vent has an inside diameter of 24 inches and a height of 40 inches which located at the center of the dome roof. A screen and aluminum cap are installed over this vent. There are also two 24-inch diameter hatches on the roof.

The Great Hill Standpipe was last inspected on July 21, 2017 during its cleaning by Underwater Solutions Inc. Underwater Solutions provided recommendations which included the following:

- For the exterior of the standpipe:
    - Pressure wash the walls at 3,500 psi and at 3.5 gpm to remove accumulated mildew and chalking from these surfaces.

- A power tool cleaning and re-coating for the areas of coating fatigue found throughout the vertical panel joints. Re-coat with an epoxy/polyurethane flexible coating
    - Spot grind and re-coat the one area on the north side of the roof (adjacent to the northern interior access hatch) where the secondary coating has been removed.
    - Power clean and re-coat the corrosion and failed coating on the overflow pipe.
- For the interior of the standpipe:
    - The next time the tank is removed from service and de-watered, pressure wash the floor and wall surfaces (at 3,500 psi and at 3.5 gpm) and also perform a complete interior rehabilitation.
    - A power tool cleaning and re-coating of all panel lap joints and associated rivets showing coating fatigue (including the perimeter of the overflow cutout and the welds between the overhead panels).

Overall, the Great Hill Standpipe was noted to be found in generally good condition.

2.5.2 Flagg Hill Reservoir

The Flagg Hill Reservoir is a 2.0 MG precast prestressed concrete storage facility constructed in 1963. The facility has an overflow elevation of 427.5 feet and is 117 feet in diameter and 25 feet high. It is located in the western section of Acton on Ethan Allen Drive on the Boxborough/Acton town line. A mixing system (GridBee manufactured by SolarBee) was installed within the tank in 2012 to improve mixing and help maintain chlorine residual.

The reservoir has three 30-inch by 30-inch aluminum hatches on the roof dome. There is also a 24-inch inside diameter vent with a height of 24 inches at the center of the roof dome. A screen and 50-inch outside diameter aluminum cap are installed over this vent. The reservoir also has a 14-inch inside diameter overflow pipe that is cast within an 18-inch by 18-inch concrete box along the tank wall. The box is located approximately 12 inches below the top of the tank wall and terminates approximately 60 inches above the tank floor where the pipe continues through the floor.

The Flagg Hill Reservoir was last inspected and cleaned on July 18, 2017 by Underwater Solutions Inc. and they provided recommendations which included the following:

- For the exterior of the reservoir:
    - Pressure wash the wall and roof surfaces at 3,500 psi and at 3.5 gpm to remove the accumulated mildew from the surfaces, the accumulated efflorescence from all cracks and any and all loose concrete from the spalls found throughout the roof.
    - Then resurface the areas of concrete spall found throughout roof with a suitable concrete filler.
    - Re-coat the wall and roof surfaces using an epoxy polyurethane flexible coating.
- For the interior of the reservoir:
    - Pressure wash the floor, wall, and overhead surfaces at 3,500 psi and 3.5 gpm to remove the staining from the floor and walls, the accumulated efflorescence from the cracks found throughout the interior wall and overhead surfaces and any and all loose shotcrete from the area of the walls having scour.
    - Then coat the wall and overhead surfaces using an elastomeric urethane flexible coating.
    - When the tank is out of service and de-watered, then complete the rehabilitation of the wall and overhead surfaces. Also, power tool clean and apply a protective coating the overflow piping to remove corrosion and prevent future corrosion.

Overall, the Flagg Hill Reservoir was noted to be found in fair condition.

2.5.3 Nagog Hill Reservoir

The Nagog Hill Reservoir is a 3.0 MG precast prestressed concrete storage facility constructed in 1974 and is located on Nagog Hill Road. The facility has an overflow elevation of 427.5 feet and is 99 feet in diameter and 52.5 feet high. The overflow pipe has an 8-inch inside diameter and is located on the north side of the tank.

The reservoir has one 24-inch by 18-inch inside diameter galvanized steel manway that is located 19 inches above the ground. There is also a 24-inch inside diameter vent with a height of 24 inches that is located within the center of the roof dome. A screen and a 42-inch outside diameter fiberglass cap is installed over the vent. One 48-inch by 30-inch aluminum hatch is also located on the roof dome.

The Nagog Hill Reservoir was last inspected and cleaned on July 21, 2017 by Underground Solutions Inc. and they provided recommendations which included the following:

- For the exterior of the reservoir:
    - Pressure wash the walls at 3,500 psi and at 3.5 gpm to remove the accumulated mildew from these surfaces and to remove the accumulated efflorescence from the cracks found throughout the exterior walls.
    - Chip/chisel with power and hand tools all of the loose shotcrete from the areas of spall found on the North-Westernmost side of the tank and then re-surface this area with a suitable concrete filler.
    - Coat wall and roof dome surfaces with an epoxy/polyurethane flexible coating.
- For the interior of the reservoir:
    - Pressure wash the floor, wall, overflow box, and overhead surfaces at 3,500 psi and at 3.5 gpm to remove the staining from these surfaces and to remove any and all loose concrete from the walls showing scour and all coating that has lost adhesion from the concrete filled wall panel joints, while removing the accumulated efflorescence from the cracks found within the overflow box.
    - Then coat the floor, wall, overflow box, and overhead surfaces using an elastomeric urethane flexible coating.
    - When tank is out of service and de-watered, complete the interior rehabilitation (including power tool cleaning the interior surface of the manway and pressure wash ladder) and then re-coat surfaces. Also replace the fall prevention device with a non-corrodible fall prevention device through the length of the ladder.

Overall, the Nagog Hill Reservoir was noted to be found in overall good condition.

2.5.4 Wampus Hill Reservoir and Booster Pump Station

The Wampus Hill Reservoir is 3.0 MG a precast prestressed concrete storage facility. The tank is 26 feet tall and 140 feet in diameter with an overflow elevation of 317 feet. The tank was constructed in 1989 on Windcliff Drive. Because the tank is at a lower hydraulic gradeline than the other tanks in the system, a booster pump station (shown on the right) is utilized to pump water from the tank into the distribution system. The booster pump station can supply water to the system through the use of three pumps: two pumps capable of pumping at 600 gpm for low demand periods and one pump capable of pumping 900 gpm for periods of higher demand.

The Wampus Hill Reservoir was generally used to provide additional supply to the system during high demand periods and was upgraded in 2012 with appropriate valving and controls for this to work through the supervisory control and data acquisition (SCADA) system.

A diesel driven emergency generator capable of running the booster station during a power outage is installed that is also capable of powering the various municipal Fire and Police communications equipment that is also located at the site.

The reservoir has one 36-inch by 30-inch inside diameter aluminum hatch on the roof dome that provides access to the interior of the tank. There is also a 24-inch inside diameter vent with a height of 24 inches at the center of the roof dome. A screen and 42-inch outside diameter fiberglass cap are installed over this vent. The reservoir also has an 8-inch inside diameter overflow pipe that is cast within a 20-inch by 22-inch concrete box along the tank wall. The box is located approximately 12 inches below the top of the tank wall and terminates approximately 60 inches above the tank floor. The pipe penetrates the wall on the Northernmost side of the reservoir located approximately 40 inches above the ground.

The Wampus Hill Reservoir was last inspected and cleaned on July 19, 2017 by Underground Solutions Inc. and they provided recommendations which included the following:

- For the exterior of the reservoir:
  - Pressure wash the wall and roof surfaces at 3,500 psi and 3.5 gpm to remove the accumulated mildew from these surfaces, the accumulated efflorescence from all cracks and any and all shotcrete that has spalled from the roof.
  - Resurface the area of concrete spall on the North side of the roof using a suitable concrete filler material and coat surfaces with an epoxy/polyurethane flexible coating.
- For the interior of the reservoir:
  - Pressure wash floor, wall, and overhead surfaces at 3,500 psi and 3.5 gpm to remove the staining and accumulated efflorescence from these surfaces.
  - Resurface area of concrete spall within the overhead on the North side of the reservoir using a suitable concrete filler material and then coat interior floor, wall, and overhead surfaces with an elastomeric urethane flexible coating.
  - Monitor the aluminum surfaces of the ladder through future scheduled inspections to ensure that fatigue does not occur.

Overall, the Wampus Hill Reservoir was noted to be found in overall good condition.

2.6 SCADA AND CONTROL SYSTEMS

The AWD currently uses a radio based SCADA system for communications with all of its remote locations except for the North Acton WTP and new South Acton WTP (which utilizes a higher capacity direct fiber optic communication). In general, all of the remote locations transmit their data to the Great Hill Tank which then retransmits it to the AWD's office. All stations can be controlled from the AWD Office or the back-up North Acton WTP location. The SCADA system components are Allen Bradley based and the software is Intellution.

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 Acton water system from 2012 through 2016. The discussion on water use is followed by a presentation of projections of future water demands. Data used in the analysis between 2012 through 2016 was obtained from Massachusetts Department of Environmental Protection (MassDEP) annual statistical reports and meter records provided by the Acton Water District (AWD). Additional population data was obtained from the United States (US) Census, UMass Donahue Institute (UMDI), Massachusetts Department of Transportation (MassDOT) Planning, Metropolitan Area Planning Council (MAPC), and the Town of Acton.

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 framework for the water supply and distribution system analyses. Updated projections of water-use needs through year 2026 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. Because Acton 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 Acton.

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

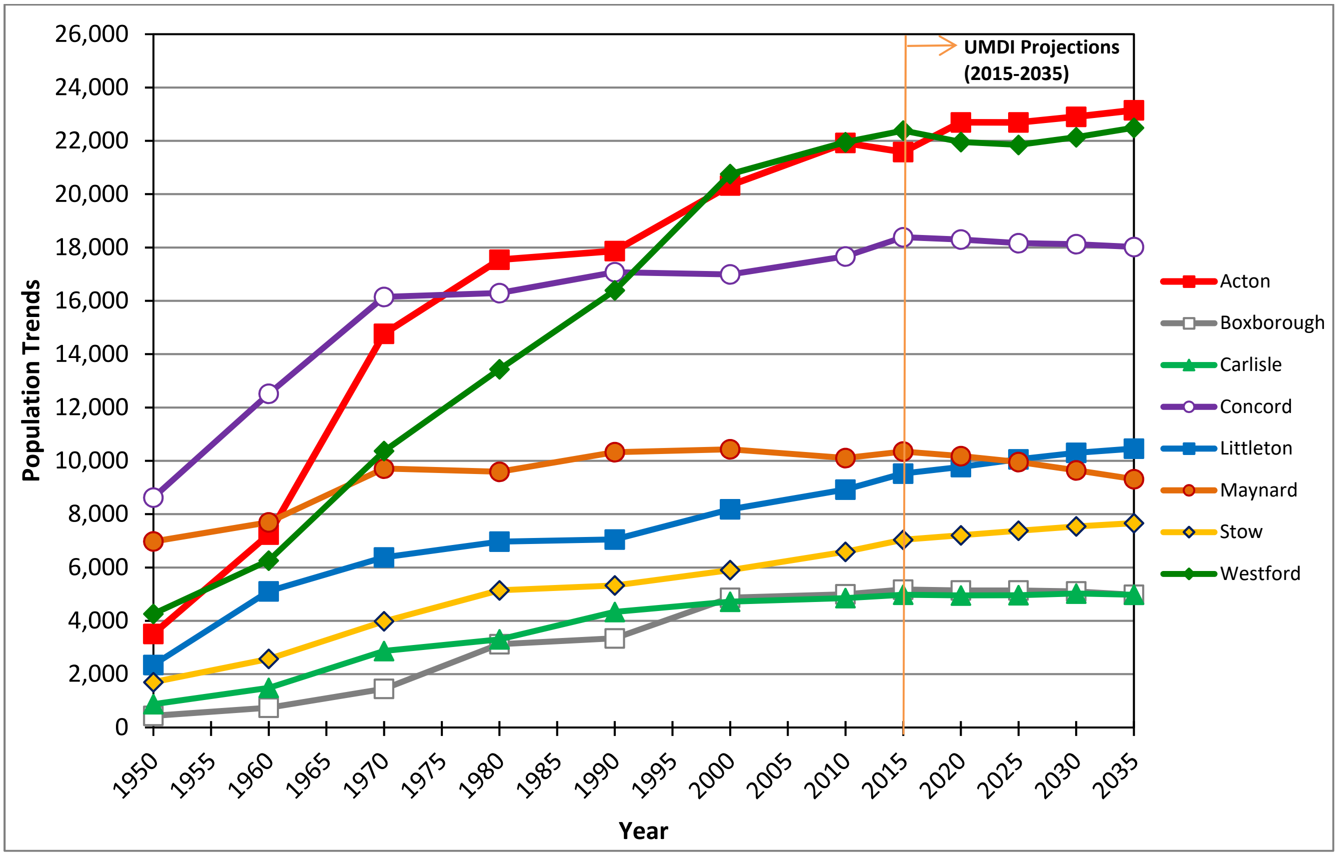
- US Bureau of Census Data
- UMDI
- MassDOT
- MAPC

The population trends in Acton and its neighboring communities from 1950 to 2015 are presented in Table 3-1 and graphically in Figure 3-1. UMDI population projections from 2015 to 2035 are also presented within Figure 3-1.

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

Town	1950	1960	1970	1980	1990	2000	2010	2015
Acton	3,510	7,238	14,770	17,544	17,872	20,331	21,924	21,582
Boxborough	439	744	1,451	3,126	3,343	4,868	4,996	5,174
Carlisle	876	1,488	2,871	3,306	4,333	4,717	4,852	4,978
Concord	8,623	12,517	16,148	16,293	17,076	16,993	17,668	18,387
Littleton	2,349	5,109	6,380	6,970	7,051	8,184	8,924	9,524
Maynard	6,978	7,695	9,710	9,590	10,325	10,433	10,106	10,349
Stow	1,700	2,573	3,984	5,144	5,328	5,902	6,590	7,039
Westford	4,262	6,261	10,368	13,434	16,392	20,754	21,951	22,385

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



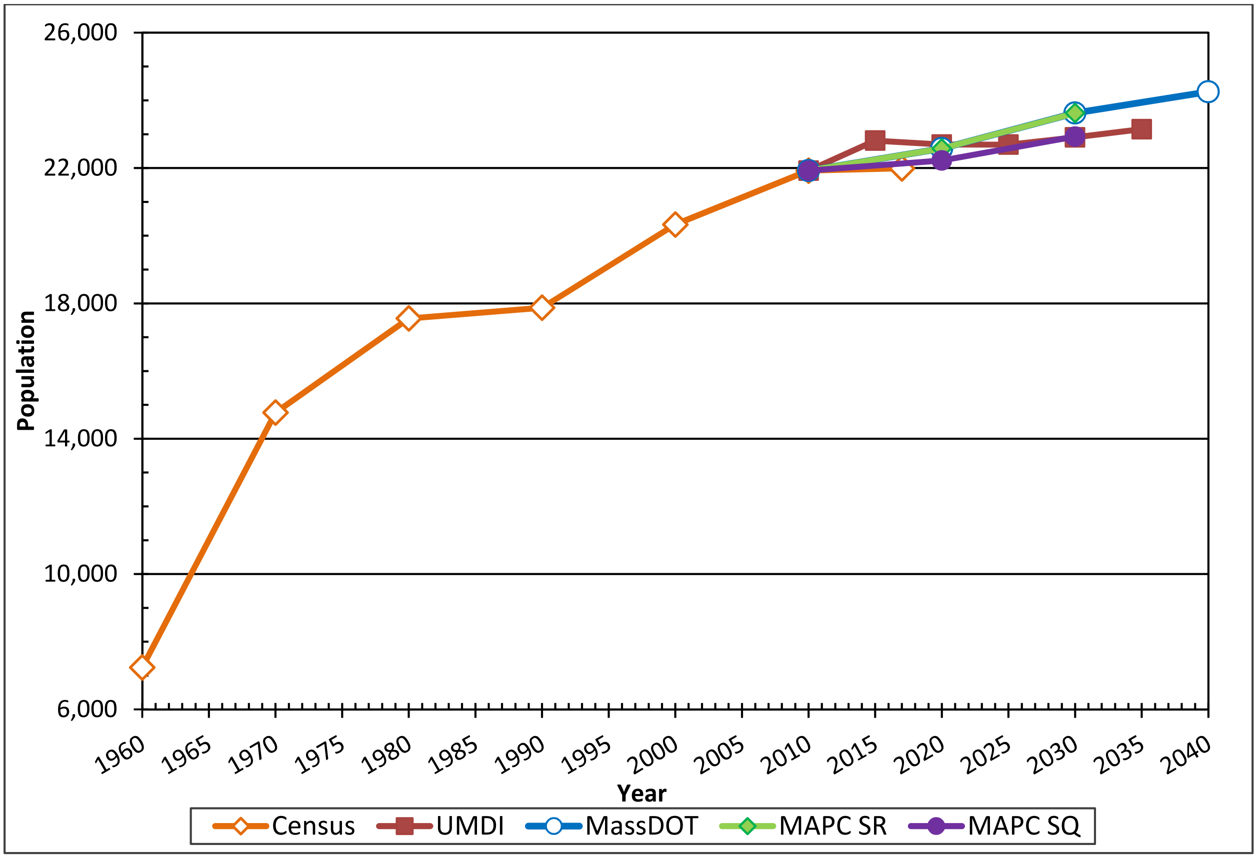
In general, the smaller communities in the suburbs experienced rapid growth during the post-World War II period from 1950's through the 1980's, when growth population began to level off in most communities. The most rapid growth during this period occurred in rural communities with abundant open space 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 to 30 years for most communities. In addition, escalating property values and high housing costs may have somewhat contributed to slower growth and development.

3.3 HISTORICAL AND PROJECTED POPULATION

According to the Census, the Town of Acton has experienced additional population growth since the early 1960s. From 1980 to 1990 the population was generally constant and then began to increase again in the early 1990s at a rate of approximately 1.3% per year through 2000. At that point, growth continued at a slower rate of approximately 0.78% per year through 2010. The 2017 population as reported by the Census is 21,990 residents (approximately 0.3% from 2010).

Population projections as reported by the US Census, UMDI, MassDOT, and MAPC were reviewed for this study. The historic populations from 1960 to 2010 were provided by the US Census and the Town provided the most recent census data for 2017. The UMDI projections were estimated in March of 2015 which provided projections from 2015 to 2035. The MassDOT projections were estimated in 2015 which provided projections from 2015 to 2040. Two sets of projections were used from MAPC: the Stronger Region (SR) scenario and the Status Quo (SQ) scenario. The Stronger Region Scenario determines its projection based upon the assumption that the trends of population, housing, workforce would be increased. The Status Quo Scenario determines its projection based upon constant existing rates of births, deaths, migration and housing occupancy. Both scenarios provided projections for years 2020 and 2030. These various historic and projected populations are presented in Figure 3-2.

FIGURE 3-2
HISTORIC AND PROJECTED POPULATION
ACTON, MASSACHUSETTS



As shown in Figure 3-2, the UMDI projected the greatest growth from 2010 to 2015 and then projects only a small increase of approximately 340 people from 2015 to 2035. The MAPC SR and MassDOT projections predict the same consistent growth rate from 2010 to 2030 of approximately 0.39% per year, since the MassDOT utilizes the MAPC SR projections as a basis for their projections. The MAPC SQ projects a reduced growth rate of approximately 0.23% from 2010 to 2030.

Since the 2017 Census population of 21,990 appears to align the closest to the MAPC SQ projection, the MAPC SQ was utilized in this study. Table 3-2 presents the MAPC SQ population projections from 2017 to 2026.

TABLE 3-2
MAPC SQ POPULATION PROJECTIONS
ACTON, MASSACHUSETTS

Year	Projected Population
2017	21,990
2018	22,068
2019	22,147
2020	22,225
2021	22,296
2022	22,366
2023	22,437
2024	22,507
2025	22,578
2026	22,649

In accordance to the MAPC SQ population projections, the projected population for the Town of Acton in 2026 is 22,649 people. This is an approximate 3.0% growth from 2017.

In regards to water service, it is understood that the Acton Water District provides water to approximately 95% of the Town's population according to the 2015 DCR Water Needs Forecast. This percentage is anticipated to increase up to 98% of the Town's population through the 2031 planning period for WMA.

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 Acton is currently supplied by eleven groundwater sources as discussed in Section 2. In general, demand from each individual source is metered, monitored, recorded, and reported by the AWD. Each of the individual well points for the Kennedy and Clapp sources are not metered individually.

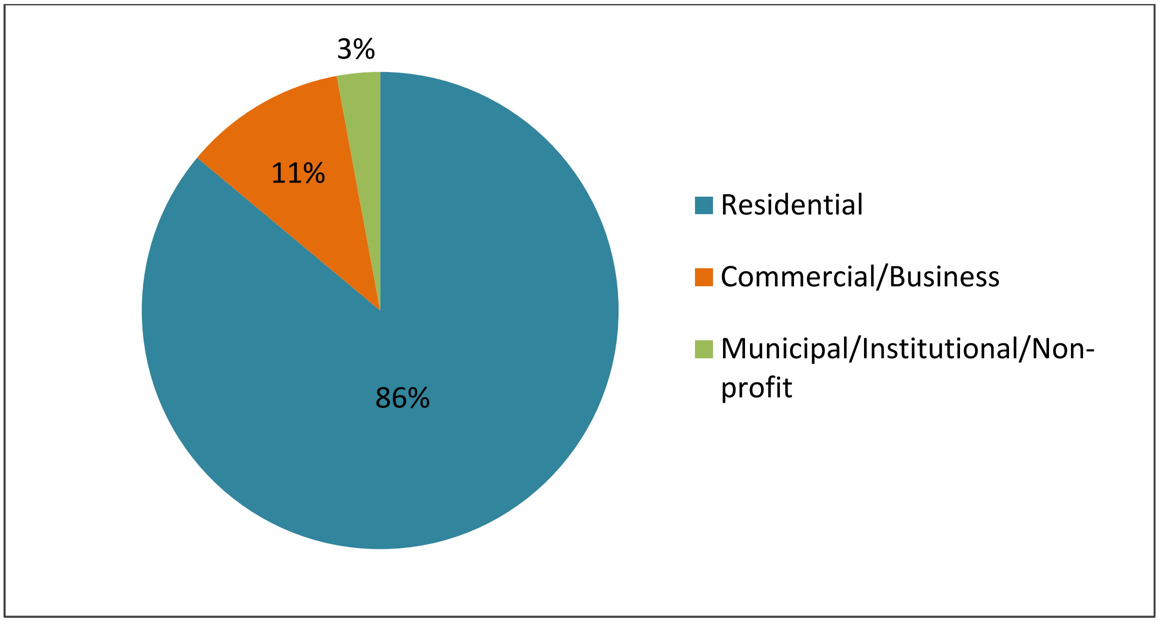
- Water *consumption* is defined as the quantity of water used or consumed by the customers or for the operations of the system. Water consumption consists of two components: revenue water and non-revenue water. Revenue or metered water is the sum of all individual water meter readings from customers. 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 theft, water used for fire-fighting and losses from storage tank overflows.

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

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

Year 2016 billing records indicate that the water system has 6,662 meter accounts. The approximate percentage of the total system demand by user type for 2016 is shown in Figure 3-3.

FIGURE 3-3
WATER CONSUMPTION BY DEMAND CATEGORY IN 2016
ACTON, MASSACHUSETTS



As presented in the figure above, the data indicates that the residential component accounts for the majority (approximately 86%) of the metered demands in the system. Commercial users account for approximately 11% and the municipal category accounts for approximately 3% of the demand.

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).

3.4.1 Year-Round Water Demand Trends

Table 3-3 below presents a summary of system-wide demands, average-day demands and maximum-day demands.

TABLE 3-3
WATER DEMAND TRENDS
ACTON, 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)
2012	612,600,000	1,678,356	2,430,000	1.45
2013	624,300,000	1,710,411	2,310,000	1.35
2014	627,660,000	1,719,616	2,500,000	1.45
2015	598,090,000	1,638,600	2,332,000	1.42
2016	596,040,000	1,632,986	2,660,000	1.63
Average	611,738,000	1,675,994	2,446,400	1.46

In general, the average day demand (ADD), maximum day demand (MDD) and demand ratio have been relatively consistent in the last five years and is lower than that of the previous five years. The average demand ratio of 1.46 was utilized for the future MDD demand calculations later in this report.

3.4.2 Seasonal Water Demand Trends

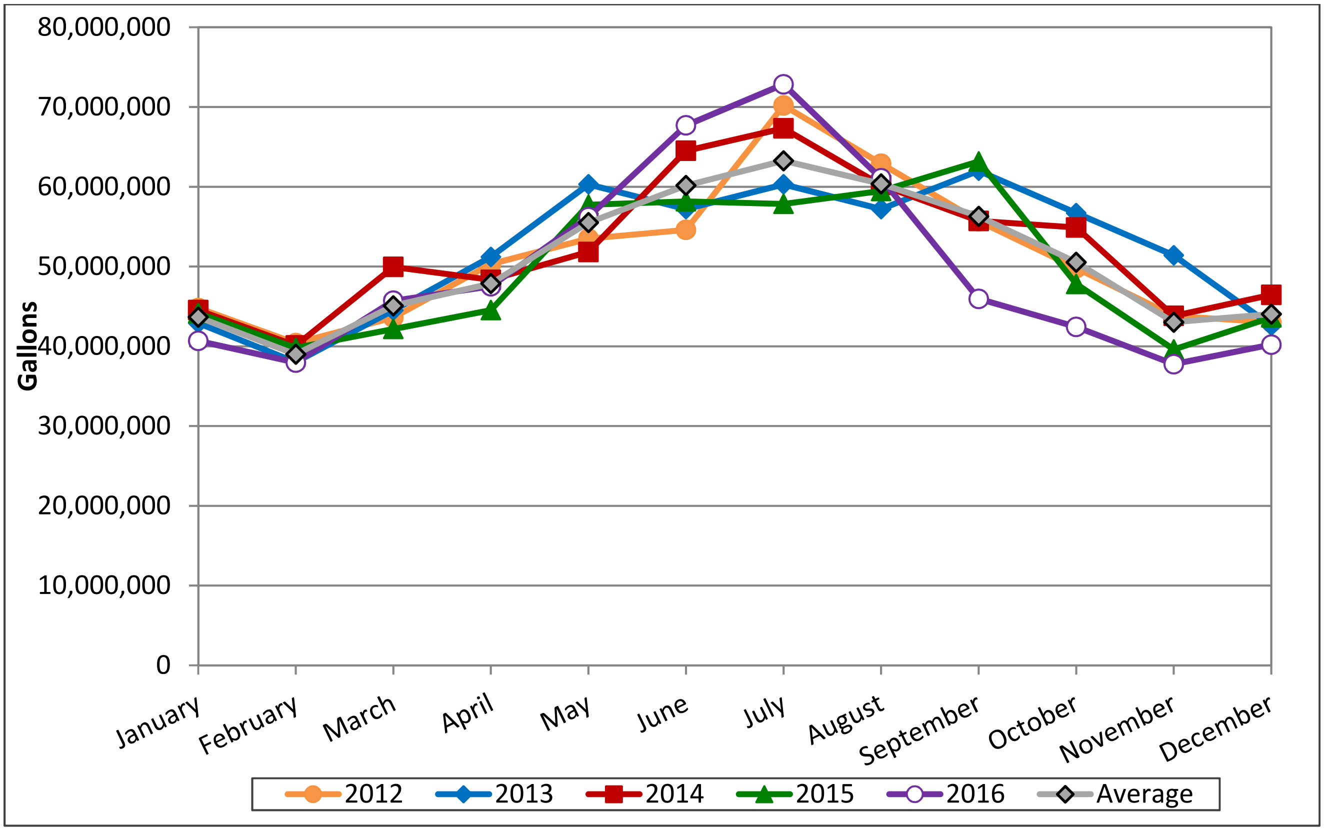
Water demand is typically a function of the time of year among other factors. In general, summer months have higher water demand due to the increased use of water for irrigation and recreation, in addition to seasonal population changes (if present in a particular community). Exceptions include industrial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

AWD's monthly production trends for years 2012 through 2016 are presented in Table 3-4 and graphically in Figure 3-4.

TABLE 3-4
WATER PRODUCTION TRENDS
ACTON, MASSACHUSETTS

Year	Total Water Production (gallons)					
	2012	2013	2014	2015	2016	Average
January	44,800,000	42,900,000	44,530,000	44,155,000	40,680,000	43,413,000
February	40,400,000	38,000,000	40,100,000	39,810,000	37,960,000	39,254,000
March	43,600,000	44,500,000	49,950,000	42,163,000	45,720,000	45,186,600
April	50,300,000	51,200,000	48,330,000	44,528,000	47,520,000	48,375,600
May	53,500,000	60,300,000	51,820,000	57,748,000	56,200,000	55,913,600
June	54,600,000	57,200,000	64,510,000	58,177,000	67,700,000	60,437,400
July	70,200,000	60,300,000	67,330,000	57,855,000	72,850,000	65,707,000
August	62,900,000	57,200,000	60,220,000	59,462,000	61,060,000	60,168,400
September	55,700,000	62,000,000	55,720,000	63,170,000	45,950,000	56,508,000
October	49,800,000	56,700,000	54,890,000	47,807,000	42,430,000	50,325,400
November	43,800,000	51,400,000	43,810,000	39,605,000	37,760,000	43,275,000
December	43,000,000	42,600,000	46,450,000	43,609,000	40,210,000	43,173,800
Total	612,600,000	624,300,000	627,660,000	598,089,000	596,040,000	611,737,800

FIGURE 3-4
SEASONAL WATER DEMAND TRENDS
ACTON, 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 September) before dropping again in the winter months. The highest and lowest average demand from 2012 to 2016 was 65,707,000 gallons in July and 39,254,000 gallons in February, respectively, which correlates to approximately a 40% drop in demand during the winter. 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. This is covered in additional detail later within Section 7 (Demand Management) of this report.

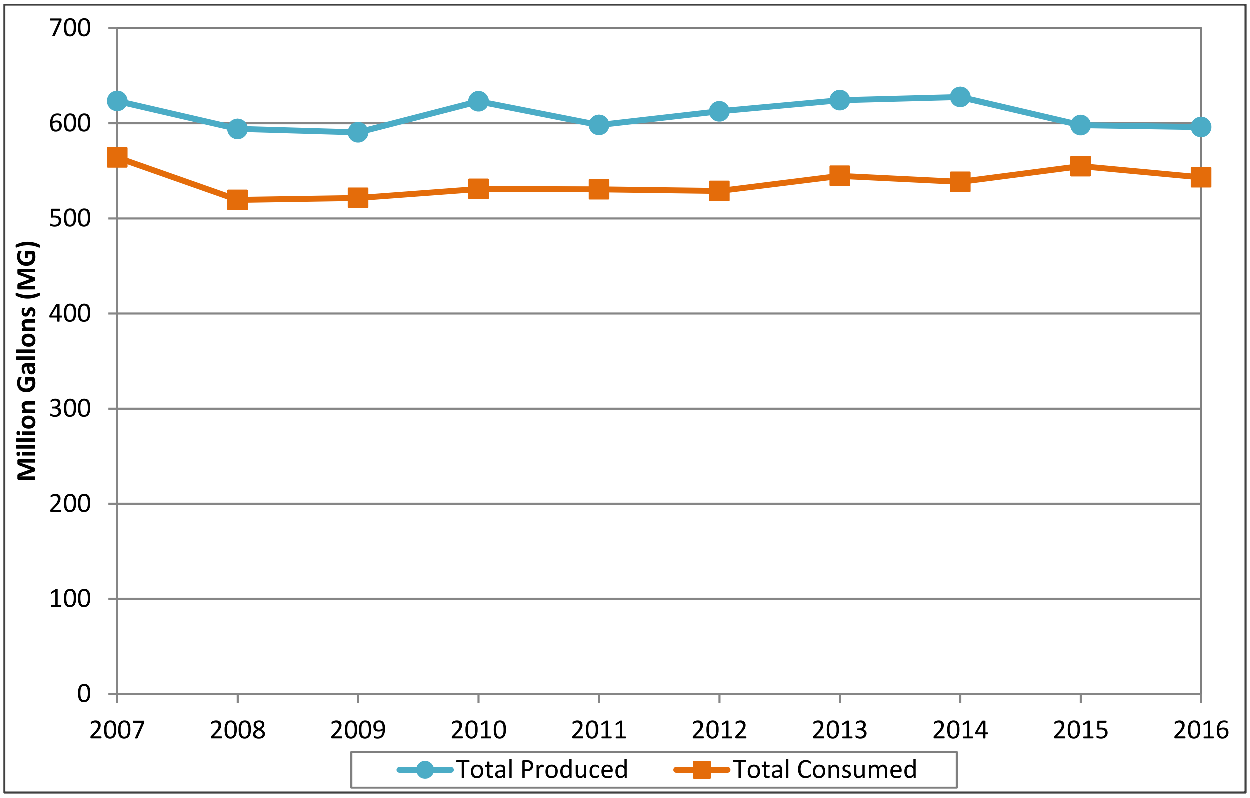
3.4.3 Water Production Trends

Water production and consumption trends are summarized in Table 3-5 and Figure 3-5. Water production is the total volume of treated 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 large difference between water produced and water consumed is considered unaccounted-for water. Additional details and concepts regarding non-revenue and unaccounted-for water are presented in the sections that follow.

TABLE 3-5
WATER PRODUCTION AND CONSUMPTION TRENDS
ACTON, MASSACHUSETTS

Year	Total Water Production/Consumption (Million Gallons)		
	Production	Consumption	Difference
2012	612.6	528.8	83.8
2013	624.3	544.7	79.6
2014	627.7	538.4	89.3
2015	598.1	554.9	43.2
2016	596.0	543.2	52.8
Average	611.7	542.0	69.7

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



As noted previously, the difference between water produced and water consumed is considered unaccounted-for water (further discussed within the next section). Recently in 2015 and 2016, the AWD has been able to achieve a reduction in unaccounted-for water by approximately 50% in comparison to previous years.

Beginning with calendar year 2013, AWD has been conducting a water audit in accordance with the American Water Works Association Manual 36 Water Audits and Loss Control Programs. These audits, with the exception of Calendar Year 2015, were funded through grants provided by MassDEP.

3.4.4 Revenue and Non-Revenue Water-Use Trends

Records from the production sources were used as the baseline for determining the AWD'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. Table 3-6 presents a breakdown of typical revenue and non-revenue sources in a system according to a standard American Water Works Association water balance. 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.

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

- <u>Total Production Volume</u> - The annual volume input to the water supply system.
- <u>Authorized Consumption</u> - The annual volume of metered and/or unmetered water taken by any user authorized to do so.
- <u>Water Losses</u> - The difference between Total Production Volume and Authorized Consumption, consisting of Apparent Losses plus Real Losses.
- <u>Apparent Losses</u> - Unauthorized Consumption, all types of metering inaccuracies and data handling errors.
- <u>Real Losses</u> - 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.
- <u>Revenue Water</u> - Those components of Total Production Volume which are billed and produce revenue.
- <u>Non-Revenue Water (NRW)</u> - The difference between Total Production Volume and Billed Authorized Consumption.

TABLE 3-6
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.**

Table 3-7 presents data comparing AWD’s production water volume to the revenue water volume.

TABLE 3-7
REVENUE AND NON-REVENUE WATER USE
ACTON, MASSACHUSETTS

Year	Total Production (MGY)	Total Revenue Water (MGY)	Non-Revenue Water (MGY)	% Non-Revenue Water
2012	612.6	495.9	116.7	19.0%
2013	624.3	508.0	116.3	18.6%
2014	627.7	502.0	125.7	20.0%
2015	598.1	514.6	83.5	14.0%
2016	596.0	517.1	78.9	13.2%
Average	611.7	507.5	104.2	17.0%

The data indicates that non-revenue water has averaged approximately 17% over the past five years (which is higher than the previous 5-year period) but has been decreasing since 2014. In particular, the non-revenue water has dropped by approximately 35% from 2014 to 2016.

Sources of non-revenue water reported in the AWD's MassDEP Annual Statistical Reports (2012 to 2016) 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 service breaks and resulting repairs. This volume has been calculated based on the known number of service breaks per year, an assumed loss rate, and period of water loss duration.
- Lost water from bleeders 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 to 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 could restrict unaccounted-for water to 10% or less.

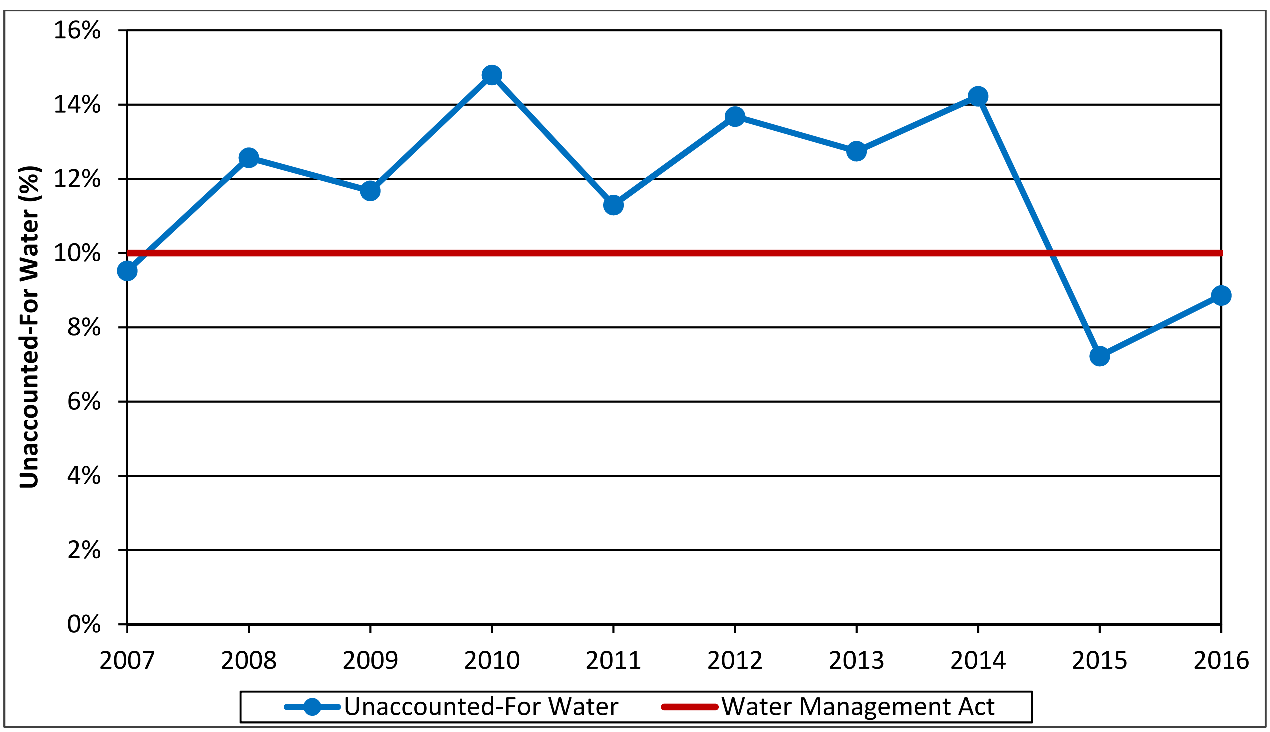
Leaks are often the largest contributor to unaccounted-for water. Leaks can originate from anywhere in the system. The largest sources of leakage typically occur on main lines or through valves. Other sources of leaks include service-lines, residential meter boxes, residential leakage on the customer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

Table 3-8 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 Acton system.

TABLE 3-8
UNACCOUNTED FOR WATER USE
ACTON, 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)
2012	116.7	19.0%	5.4%	13.7%
2013	116.3	18.6%	5.9%	12.7%
2014	125.7	20.0%	5.8%	14.2%
2015	83.5	14.0%	6.7%	7.2%
2016	78.9	13.2%	4.4%	8.9%
Average	104.2	17.0%	5.6%	11.3%

FIGURE 3-6
UNACCOUNTED FOR WATER USE
ACTON, MASSACHUSETTS



As shown in the table and figure, the UAW ranged from approximately 7% to 14% with an average of 11.3%. This is higher than the Water Management Act performance standard of 10% but is noted to have been below the standard for the last two years. This significant drop in 2015 and 2016 is presumed to be due to the upgrade (increased accuracy) of the master metering when the SAWTP was commissioned.

It should be noted that Table 3-8 and Figure 3-6 lists and presents the UAW values extracted from AWD's ASRs. After reviewing each year's ASR, MassDEP corrected the District's reported UAW values based upon their own calculations and analysis. According to MassDEP, the correct UAW (%) values are 14, 11, 15, an 8 for the respective years 2012, 2013, 2014, and 2015. This report will only present and discuss the ASRs' values provided by the AWD.

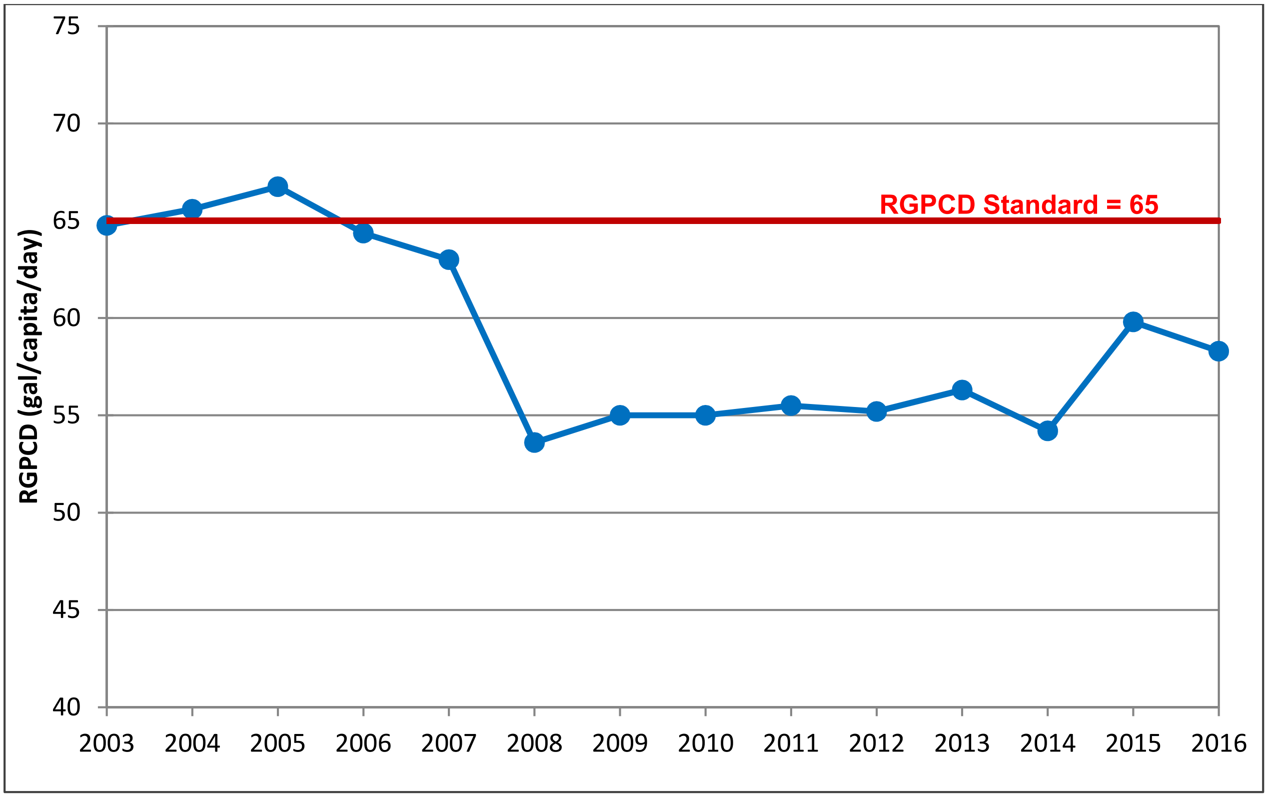
In order to comply with MassDEP's performance standard, it will be important for AWD to continue to gain a clear understanding of the true magnitude of the lost water component of water use. The biggest gains in reducing lost water typically will come from one of several sources: (1) improving accuracies 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. Specific recommendation for reducing unaccounted for water use will be included in other sections of this report.

As mentioned earlier, AWD has been conducting a water audit in accordance with the American Water Works Association Manual 36 Water Audits and Loss Control Programs since calendar year 2013.

3.4.5 Residential Gallons per Capita Day Water Consumption (RGPCD)

As presented in Figure 3-7 per capita residential water-use in Acton has ranged between 53.6 and 66.8 RGPCD over the past nine years. The WMA permit limits residential consumption to 65 RGPCD on an annual basis.

FIGURE 3-7
HISTORICAL WATER-USE TRENDS
RESIDENTIAL GALLONS PER CAPITA DAY
ACTON, 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 and efficiency requirements, and other provisions in the permit are leading to lower water use. To be conservative, future water-use projections will be based on 65 RGPCD for residential water customers.

3.4.6 Largest Water-Use Customers

The ten largest water users were identified by AWD from their billing database and this information is presented within Table 3-9. 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 3-9
2016 LARGEST WATER USERS
ACTON, MASSACHUSETTS

Rank	Account	Customer Name	Description	Service Address	Gallons/Year	Gallons/Day
1	558	Life Care Center of Acton	Nursing Home	1 Great Road	3,738,063	10,241
2	6024	Acton Estates	Apartment Complex	53-55 Brook Street	3,416,886	9,361
3	6333	Avalon Bay Communities	Condo Complex	1000 Avalon Drive	3,027,448	8,294
4	2291	Concordian Motel	Motel	71 Hosmer Street	2,416,511	6,621
5	4114	Acton-Boxboro High School	High School	16 Charter Road	2,074,024	5,682
6	4478	Acton Housing	Apartment Complex	68 Windsor Avenue	2,020,902	5,537
7	1003	Briarbrook	Condo Complex	9 Davis Road	1,825,584	5,002
8	5751	Haartz Corporation	Industrial	87 Hayward Road	1,815,703	4,975
9	1005	Briarbrook	Condo Complex	15 Davis Road	1,803,017	4,940
10	1072	Goulds Clothing	Commercial/ Shopping Center	260 Great Road	1,679,761	4,602

As shown, the majority of the top water users are large residential users such as assisted living communities and apartment/condo complexes. In 2016, the top ten water users consumed approximately 23.8 million gallons of water, or approximately 4.6% 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 Acton 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 86% of the total water-use.

MassDEP performance standards set a residential per capita demand goal of 65 residential gallons per capita day (RGPCD). According to AWD's ASR, the average-per capita water consumption in the Acton water system over the last five years (2012 through 2016) is approximately 57 RGPCD, 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 96% of the Town's population is served by the AWD, 96% of the projected population was also utilized for the residential demand projection for the planning period.

3.5.1.2 Commercial

Commercial water-use consists of business parks, restaurants, hotels, banks, golf-courses, etc. located within the service area. In the last five years, commercial demand has increased annually from a low of approximately 53 million gallons per year (MGY) to a high of approximately 57 MGY and averaged approximately 55 MGY. There was a growth of approximately 7.3% from 2012 to 2016 which equates an annual growth rate of approximately 1.8%. Therefore, an annual growth rate of 1.8% was utilized to estimate the continued demand growth for the commercial usage component for the planning period.

3.5.1.3 Municipal

Municipal water-use is water used by schools, government offices, etc. located within the Acton system. In the last five years, municipal demand has ranged from 6.6 to 17 MGY and averaged approximately 13.5 MGY. Municipal demand is not expected to increase over the planning period. Therefore, the average demand of 13.5 MGY was utilized for the municipal demand projections.

3.5.1.4 Other

In the last five years, the other category was utilized in 2012 and 2014 which included a demand of 6.2 MGY and 5 MGY, respectively. According to the 2012 ASR, this demand was for seasonal services, construction meters, and commercial and residential. According to the 2014 ASR, this demand was used for temporary construction meters, sample taps, abatements, and water to waste for water quality. It was assumed that these services will continue through the planning period. Therefore, the average demand of 5.6 MGY from 2012 and 2014 will be utilized as an annual "Other" demand.

3.5.1.5 Unaccounted-For Water

As discussed, UAW ranged from approximately 7% to 14% with an average of 11.3%. MassDEP requires that water systems work to achieve a maximum of 10% unaccounted-for water. The Acton system is close to meeting the MassDEP requirement; however, the 11.3% average for unaccounted-for water was utilized to be more conservative.

3.5.2 Average Day Water Demand Projections

Table 3-10 includes the projected average daily demands from 2017 to 2026 based on the methodology described above.

TABLE 3-10
PROJECTED AVERAGE-DAY DEMANDS (MGY)
ACTON, MASSACHUSETTS

Category	2017	2018	2019	2020	2021	2022	2023	2024	2025	2026
Residential	500.8	502.6	504.4	506.2	507.8	509.4	511.0	512.6	514.2	515.8
Commercial/Business	58.0	59.1	60.1	61.2	62.3	63.44	64.6	65.7	66.9	68.1
Municipal/Institutional/ Non-profit	13.5	13.5	13.5	13.5	13.5	13.5	13.5	13.5	13.5	13.5
Other	5.6	5.6	5.6	5.6	5.6	5.6	5.6	5.6	5.6	5.6
Total Metered Use	578.0	580.8	583.6	586.5	589.2	592.0	594.7	597.5	600.3	603.1
Unaccounted-For Water (11.3%)	64.2	64.5	64.8	65.2	65.5	65.8	66.1	66.4	66.7	67.0
Total Water Use	642.2	645.3	648.5	651.7	654.7	657.7	660.8	663.9	667.0	670.1

The total projected average-day demand for 2026 is 670.1 MGY which is a growth of approximately 4.3% from 2017.

3.5.3 Maximum and Peak Hourly Flow Demand Projections

As previously discussed, the average to maximum day ratio of 1.46 from the last five years was utilized to estimate the future maximum daily demands. In addition, previously provided daily demand data for the maximum day in 2014 (July 3) was utilized to estimate the peak hourly demand for the system. In 2014 the peak hour demand was approximately 1.49 times the average day hourly demand. This is generally consistent with expected peaking factors for similarly sized communities. Therefore, a peak hour peaking factor of 1.49 was utilized to estimate future peak hour demands. The resultant projected maximum day and peak hour demands are presented in Table 3-11.

TABLE 3-11
PROJECTED AVERAGE-DAY AND MAXIMUM-DAY DEMANDS
ACTON, MASSACHUSETTS

Year	ADD (mgd)	MDD (mgd)	Peak Hour (mgh)
2017	1.76	2.57	0.109
2018	1.77	2.58	0.110
2019	1.78	2.59	0.110
2020	1.79	2.61	0.111
2021	1.79	2.62	0.111
2022	1.80	2.63	0.112
2023	1.81	2.64	0.112
2024	1.82	2.66	0.113
2025	1.83	2.67	0.113
2026	1.84	2.68	0.114

It should be noted that peak hour usage (from storage) is discussed later in this report within Section 5 – Distribution System Evaluation and Assessment.

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 and to ensure adequate natural water supply needs of flora and fauna that inhabit the watersheds. The AWD has eight registered water supply wells (Assabet 1A & 2A, Christofferson, Clapp, Conant 1, Lawsbrook, Scribner, and Whitcomb) and four permitted supply wells (Conant 2, Kennedy, Marshall, and Assabet 3). The WMA registration authorizes withdrawal of 1.56 MGD on average over the calendar year. The WMA permit authorizes an additional withdrawal of 0.38 MGD for a total authorized withdrawal of 1.94 million gallons per day (MGD). The most recent copies of the District's registration statement and WMA Permit are included within **Appendix A**.

3.6.1 SWMI

As a basis for the AWD's WMA permit, MassDEP utilizes the Massachusetts Department of Conservation and Recreation (DCR) water demand projections to determine the WMA permitted withdrawal volumes. The DCR provided the Town's final water needs forecast on September 29, 2015 which is presented in Table 3-12 (and also included within **Appendix A**).

TABLE 3-12
FINAL WATER NEEDS FORECAST FROM DCR (2015)
ACTON, MASSACHUSETTS

	2021	2026	2031
ADD Projection (MGD)[1]	2.00	2.05	2.10
ADD Projection (MGD)[2]	1.79	1.83	1.87

[1] Assuming 65 RGPCD and 10% UAW. The 5% buffer is +0.11.
[2] Assuming water delivery continues at current RGPCD (54.5) and UAW (12.6%). The 5% buffer is +0.09.

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

In accordance with the new SWMI requirements, 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. AWD's Baseline is 1.79 MGD which is based off of the volume withdrawn during 2005 plus 5%. As previously provided in Table 3-11, the future estimated average day demand for 2026 could reach a total of 1.84 MGD. DCR's water needs forecast projects a total demand of 2.10 MGD by year 2031 under its first scenario and 1.87 MGD under its second scenario. All three of these projections are above the established Baseline.

The historical and projected water demands through year 2026 are presented in Figure 3-8.

FIGURE 3-8
HISTORICAL AND PROJECTED WATER DEMANDS
ACTON, MASSACHUSETTS

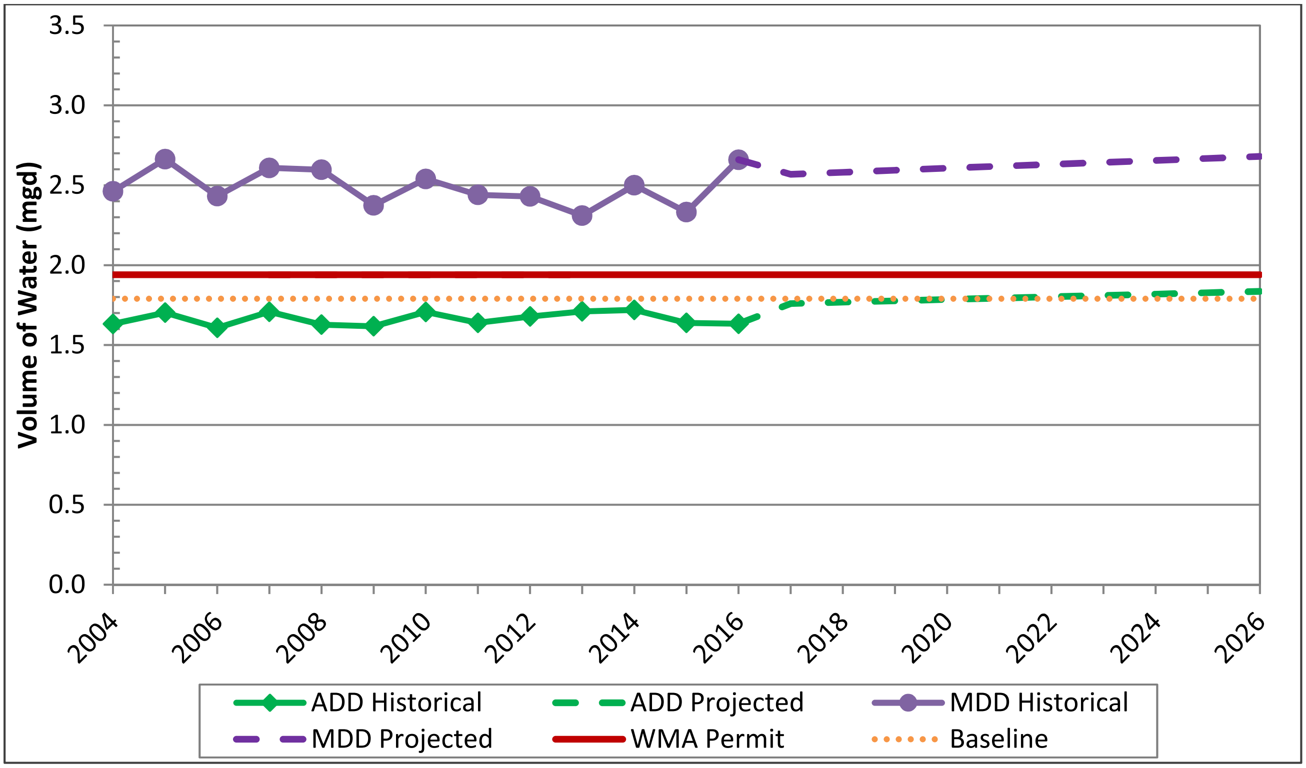


Figure 3-8 illustrates the historical water demands from 2004 to 2016 (solid green for ADD and solid purple for MDD) and the projected water demands from 2017 to 2026 (dashed lines in their respective colors). These projected demands were previously calculated in Table 3-11.

In comparison with the current WMA permit, the data indicates that the AWD generally has adequate permitted water supply capacity through year 2026 based on the projections presented herein. However, in accordance with the new SWMI regulations, the water demand is conservatively projected to reach the Baseline by 2017 and then surpass the Baseline by 2019. It should be noted that the AWD 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 permitted withdrawal volume. A detailed review of the existing sources ability to meet the projected demands is presented in Section 4.

In accordance with the new integrated 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. Other factors that determine the tier is the Groundwater Withdrawal Level and Biological Category for the water supply sub-basins. There are a total of three tiers and each tier has specified requirements that the AWD would be required to fulfill based on a variety of categories established by the WMA. The three tiers are specified as the following:

- **Tier 1**: No additional groundwater withdrawal request above the Baseline and no change in Groundwater Withdrawal Level or Biological Category
- **Tier 2**: Additional groundwater withdrawal request above Baseline and no change in Groundwater Withdrawal Level or Biological Category
- **Tier 3**: Additional groundwater withdrawal request above Baseline and change in Groundwater Withdrawal Level and/or Biological Category

The permit conditions for each tier is further discussed within the Massachusetts Sustainable Water Management Initiative – Framework which can be found on the MassDEP website.

The Biological Category (BC) was established by MassDEP as an indicator for the SWMI regulations to help determine whether withdrawn water would affect the functions and values of any streams and/or rivers. The BCs range from 1 to 5 and represent the least impacted to the more impacted, respectively. Each of the five categories represents a percentage range that was determined from the alteration of the fluvial fish community characteristics in accordance with four specific parameters (impervious cover, stream channel slope, cumulative groundwater withdrawal, and percent wetland within stream buffer area).

The Groundwater Withdrawal Category (GWC) was determined by MassDEP by calculating the ratio of the groundwater pumping to unaffected streamflow. The GWCs range from 1 to 5 and represent the least impacted to the most impacted, respectively.

Since the projected average day future demand is predicted to surpass the Baseline within the next few years (the DCR's projections also surpass the Baseline), the AWD will fall into a tier where there will be additional requirements placed upon the District (either Tier 2 or 3). Along with the standard conditions that apply for all permitted groundwater and surface water withdrawals, these additional requirements would include submitting a minimization plan, performing additional conservation measures, optimizing withdrawal, and returning water to the sub-basin(s).

3.6.1.1 SWMI Criteria

The three new criteria that are now being implemented within the WMA (determined by the tier) are the following:

- Minimization
- Mitigation
- Coldwater Fisheries Resource

Minimization: Minimization correlates to the groundwater extraction at source locations. MassDEP has set a limit of 25% for August Net Groundwater Depletion (NGD) and exceeding this limit requires permittees to determine and implement ways to reduce withdrawals or return groundwater to basins or sub-basins to improve streamflow. The minimization guidelines include:

- Additional conservation measures that go beyond the Standard Conditions.
- Optimizing withdrawal points located in sub-basins that are less groundwater depleted, if possible, to minimize depletion.
- Releasing and returning water to the sub-basin to improve streamflow.
- Additional restrictions on nonessential outdoor water use that go beyond the Standard Conditions.
- 18 additional measures are provided as options within MassDEP's Water Management Act Permit Guidance Document (such as more frequent billing, seasonal rate structure, etc.).

Mitigation: Withdrawals requested above the PWS permit's baseline will require mitigation in order to receive approval for the increased withdrawal by receiving "credits" to compensate for the impact. It is calculated by subtracting the savings through enhanced demand management and all applicable wastewater adjustments from the increase over baseline. SWMI provides six categories of options for mitigation:

- Instream flow improvement (e.g., Department of Fish and Game approved releases)
- Habitat improvement (e.g., install a fish ladder, remove a dam or a flow barrier, replace culverts)
- Wastewater improvement (e.g., infiltration and inflow removal, wastewater recharge)
- Stormwater/impervious cover (e.g., implement MS4 requirements, recharge stormwater)
- Water supply management (e.g., adopt enterprise account)
- Demand management (e.g. ban non-essential water use, conservation water rates, reuse wastewater, water saving devices, rebates, new meters)

Each option is classified as either direct mitigation (quantitative credit system) or indirect mitigation (qualitative credit system). Direct mitigation will directly result in enhanced streamflow, streamflow contributions, or surface water releases and indirect mitigation are environmental improvements that will help compensate for streamflow impacts resulting from withdrawals. Some examples of a direct mitigation are surface water releases, stormwater discharge, or infiltration and inflow removal. Indirect mitigation is not amenable to volumetric calculation and some examples are installing and maintaining a fish ladder, removal of a dam or flow barrier, or replacing a culvert.

Coldwater Fisheries Resource (CFR): CFR are waters identified with cold water fish (according to 321 CMR 5.00) determined by the MA Division of Fish and Wildlife (DFW). According to MassDEP and the DFW, the CFR are dependent with groundwater and withdrawals greatly impact the CFR. Any applicant that is withdrawing water from a sub-basin that contains a CFR is required to consult with MassDEP and the Executive Office of Energy and Environmental Affairs (EEA) regarding the minimization of impacts to this CFR.

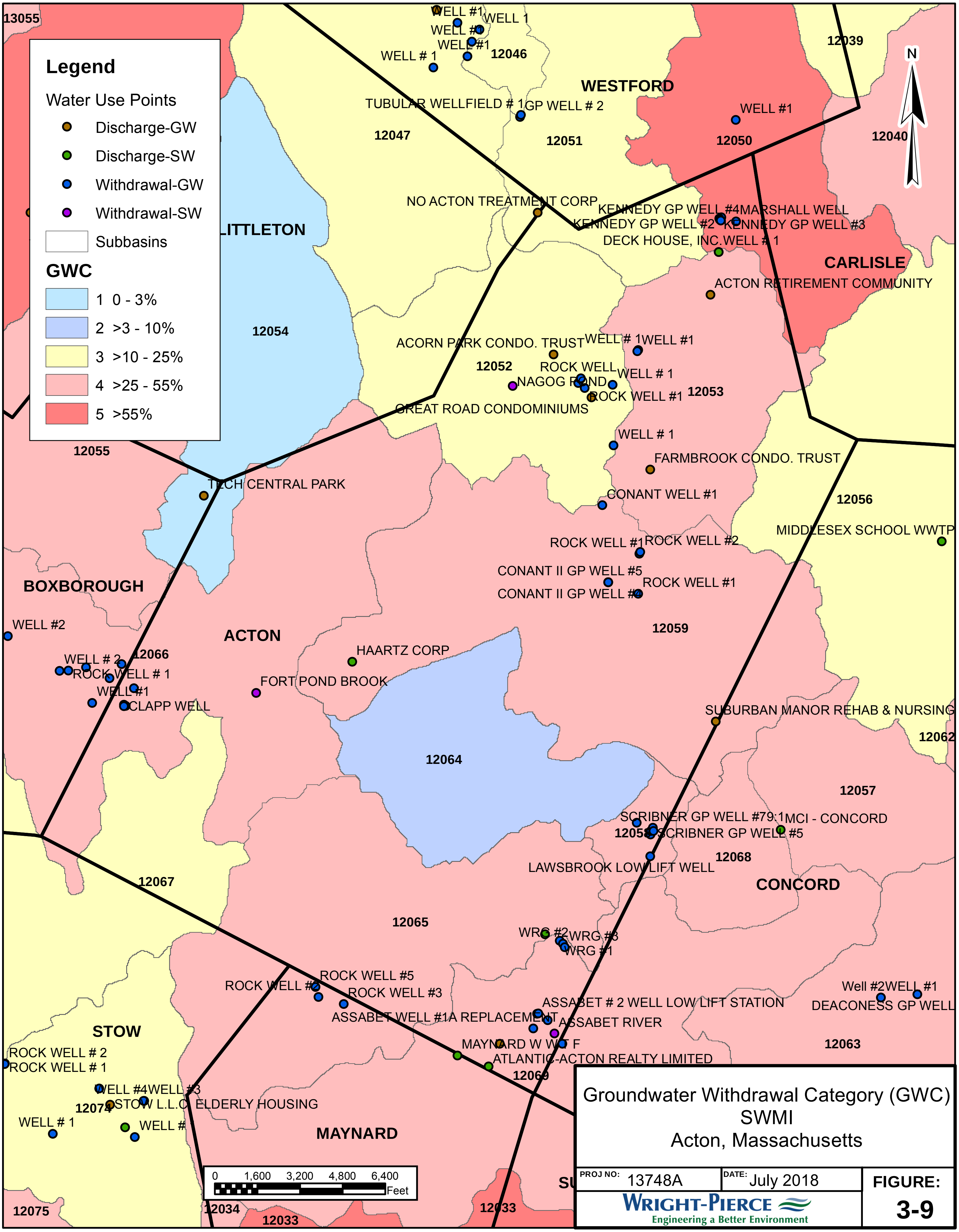
3.6.1.2 Sub-basins

The entire Town of Acton is located within the Concord Basin which is comprised of about 33 Towns. All of AWD's groundwater sources are located within 5 of the 13 sub-basins that are located within the Town of Acton (Sub-basins No. 12050, 12058, 12059, 12066, and 12069). Each water supply sub-basin is discussed below with their corresponding GWC, BC, NGD, and CFR. Figures 3-9 through 3-12 also illustrate each of these categories for all of the sub-basins in Acton.

Sub-basin 12050: Sub-basin 12050 has an area of approximately 2.81 square miles and about 8.6% of this area is impervious cover. Within this sub-basin, the AWD utilizes five groundwater sources (Marshall Well and Kennedy Wells 1-4). Within the GWC, this sub-basin is located within Category 5 which associates to the range of alteration of unimpacted August median flows due to groundwater withdrawal being 55% or greater. Within the BC, the sub-basin is also located within Category 5 which associates to 65% or greater alteration of the Range of Fluvial Fish Relative Abundance. This represents fish communities that have undergone severe changes to their structure and function.

Sub-basin 12050 is also listed positively for NGD. All permitees above 25% for August NGD (regardless of tier) are required to determine and implement ways to reduce withdrawals or return groundwater to basins or sub-basins to improve streamflow. Since AWD utilizes groundwater sources within this sub-basin which has an August NGD of 115.8% which is much greater than the 25%, AWD will need to submit a minimization plan (described in 310 CMR 36.22(5)).

There are no CFRs within this sub-basin.



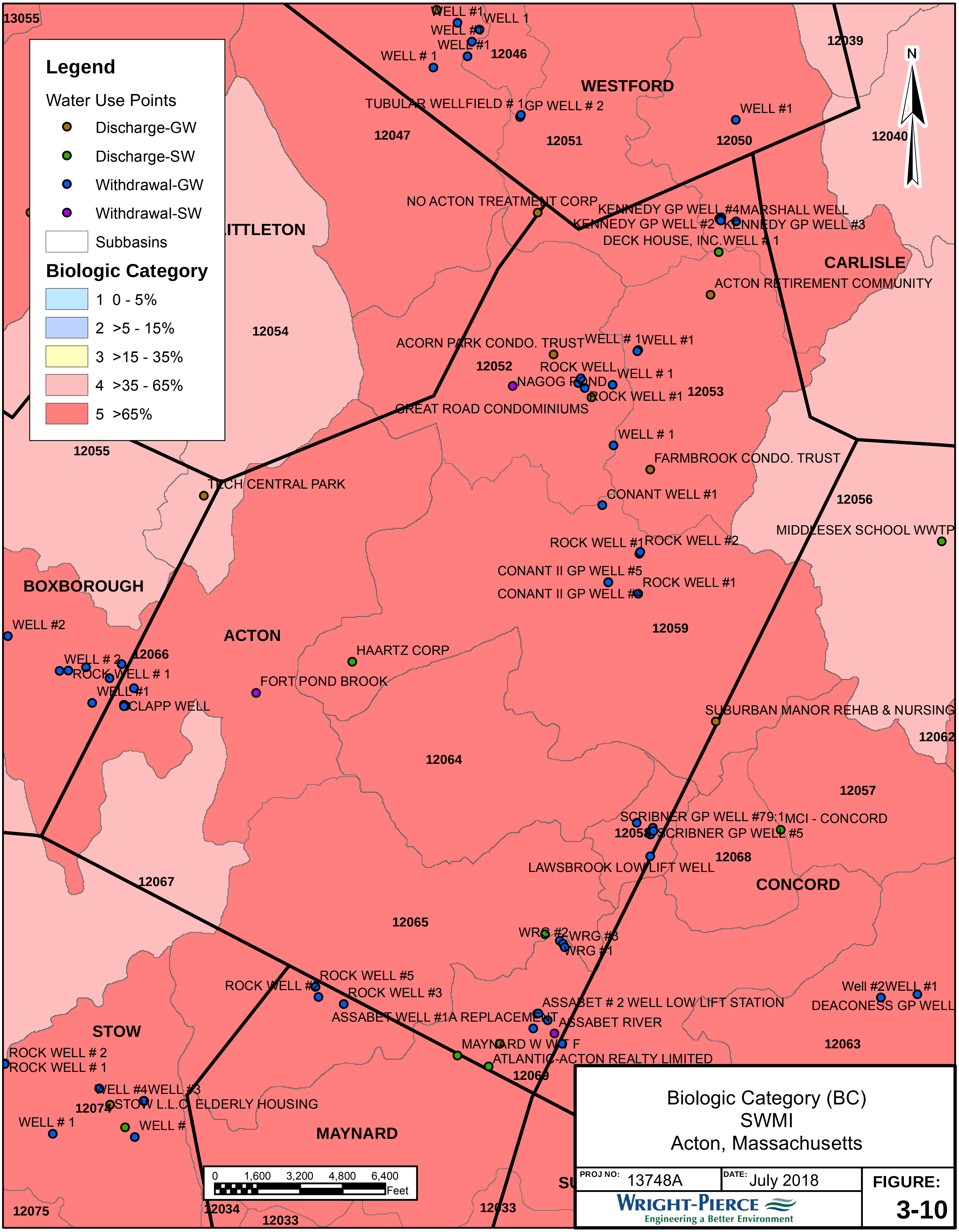
Groundwater Withdrawal Category (GWC)
SWMI
Acton, Massachusetts

PROJ NO: 13748A

DATE: July 2018

FIGURE: 3-9

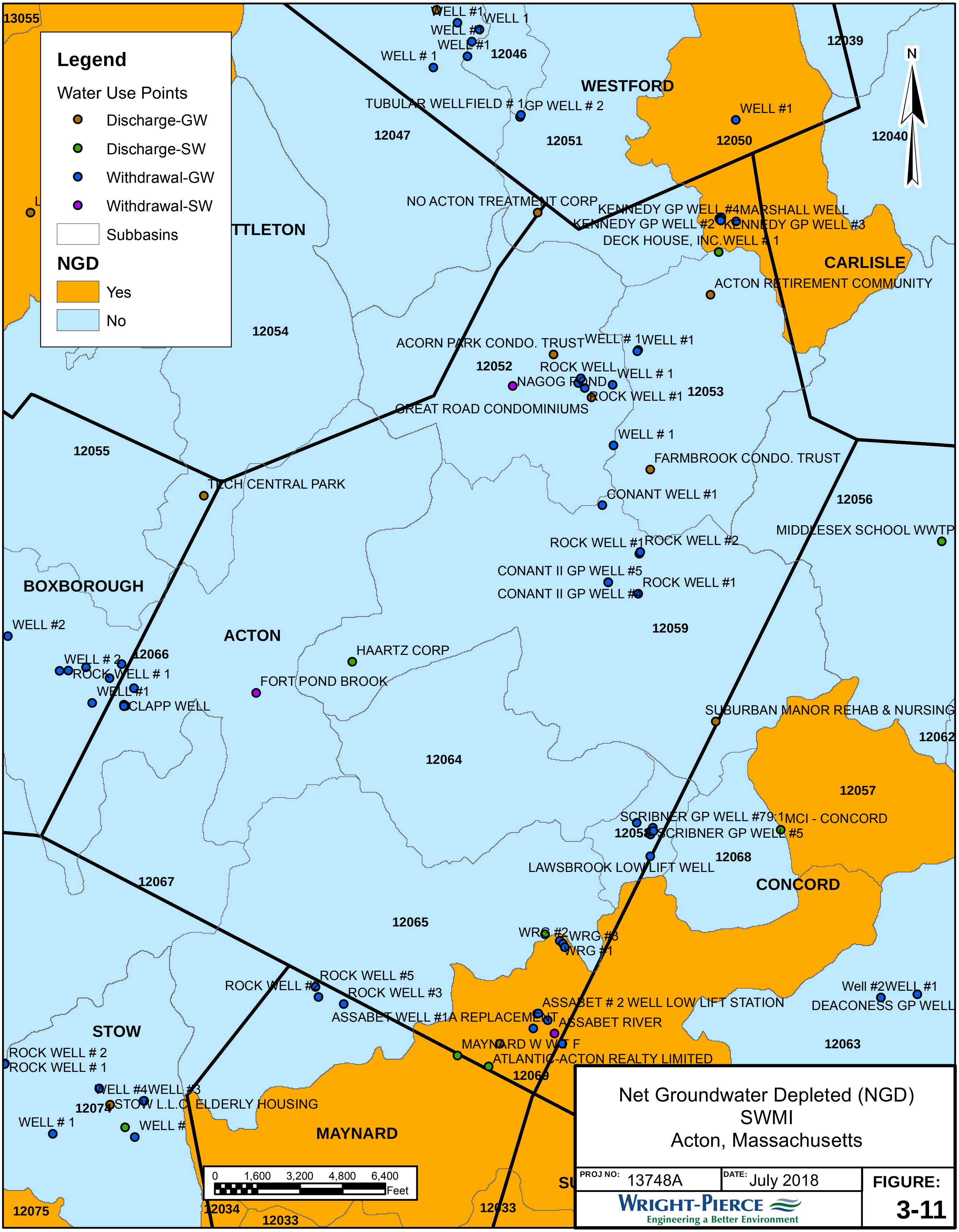
WRIGHT-PIERCE
Engineering a Better Environment



Biologic Category (BC)
SWMI
Acton, Massachusetts

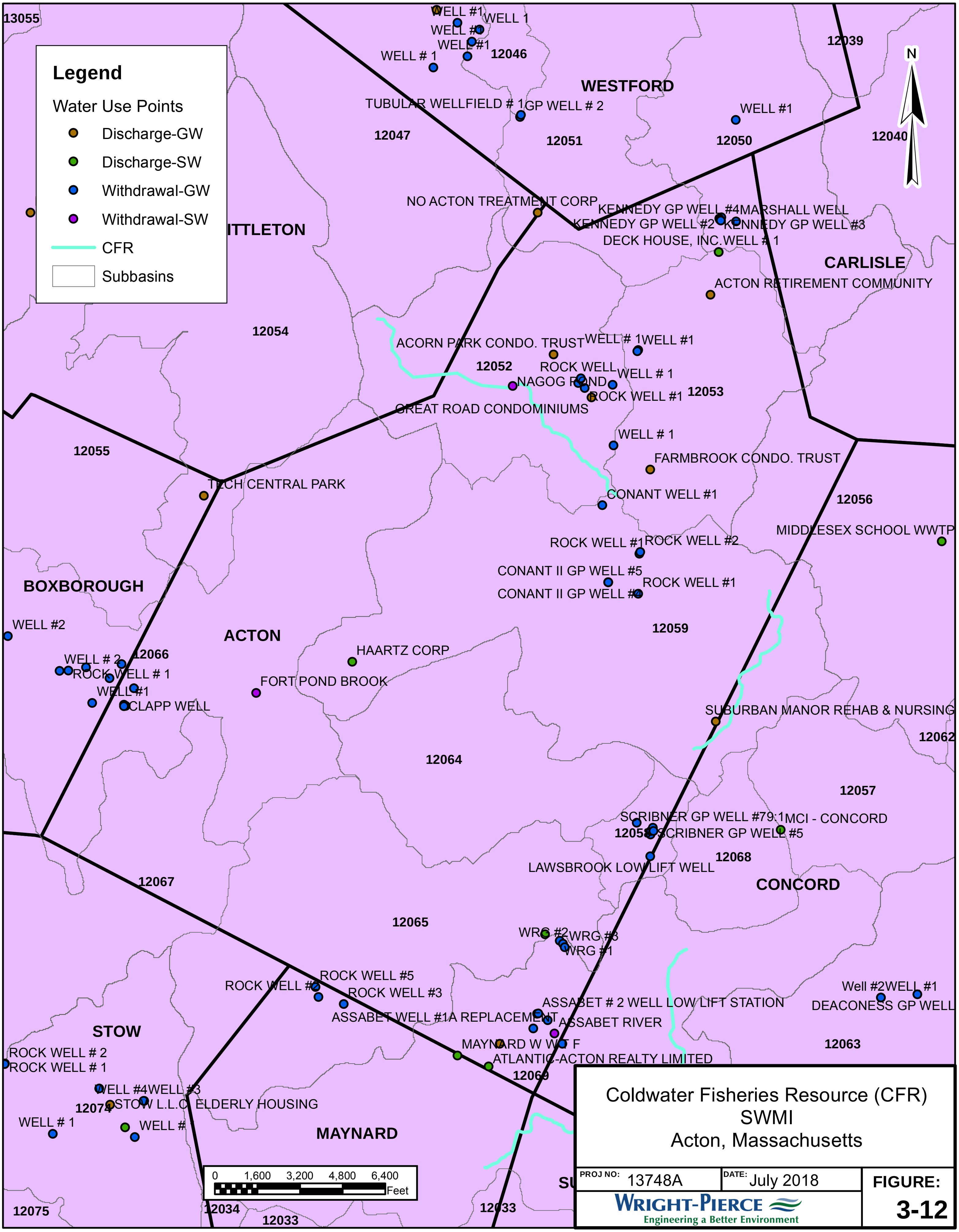
PROJ NO: 13748A
DATE: July 2018
FIGURE: 3-10

WRIGHT-PIERCE
Engineering a Better Environment



Net Groundwater Depleted (NGD) SWMI
Acton, Massachusetts

PROJ NO: 13748A	DATE: July 2018	FIGURE:
WRIGHT-PIERCE Engineering a Better Environment		3-11


Legend
Water Use Points
Discharge-GW
Discharge-SW
Withdrawal-GW
Withdrawal-SW
CFR
Subbasins
WESTFORD
LITTLETON
CARLISLE
BOXBOROUGH
ACTON
CONCORD
STOW
MAYNARD
13055
12039
12040
12046
12047
12050
12051
12052
12053
12054
12055
12056
12057
12058
12059
12062
12063
12064
12065
12066
12067
12068
12069
12074
12075
12034
12033
WELL #1
WELL 1
WELL # 1
TUBULAR WELLFIELD # 1
GP WELL # 2
NO ACTON TREATMENT CORP
KENNEDY GP WELL #4
MARSHALL WELL
KENNEDY GP WELL #2
KENNEDY GP WELL #3
DECK HOUSE, INC.
ACTON RETIREMENT COMMUNITY
ACORN PARK CONDO. TRUST
ROCK WELL
NAGOG POND
ROCK WELL #1
GREAT ROAD CONDOMINIUMS
FARMBROOK CONDO. TRUST
CONANT WELL #1
TECH CENTRAL PARK
ROCK WELL #2
MIDDLESEX SCHOOL WWTP
CONANT II GP WELL #5
CONANT II GP WELL #4
WELL #2
WELL # 2
ROCK WELL # 1
CLAPP WELL
HAARTZ CORP
FORT POND BROOK
SUBURBAN MANOR REHAB & NURSING
SCRIBNER GP WELL #7
SCRIBNER GP WELL #5
MCI - CONCORD
LAWSBROOK LOW LIFT WELL
WRG #2
WRG #3
WRG #1
ROCK WELL #5
ROCK WELL #3
ASSABET # 2 WELL
LOW LIFT STATION
ASSABET WELL #1A REPLACEMENT
ASSABET RIVER
MAYNARD W WT F
ATLANTIC-ACTON REALTY LIMITED
Well #2
DEACONESS GP WELL
ROCK WELL # 2
WELL #4
WELL #3
STOW L.L.C. ELDERLY HOUSING
0
1,600
3,200
4,800
6,400
Feet
N
Coldwater Fisheries Resource (CFR)
SWMI
Acton, Massachusetts
PROJ NO: 13748A
DATE: July 2018
FIGURE:
3-12
WRIGHT-PIERCE
Engineering a Better Environment

Sub-basin 12058: Sub-basin 12058 has an area of approximately 24.85 square miles and about 9.5% of this area is impervious cover. Within this sub-basin, the AWD utilizes six groundwater wells (Christofferson Well, Lawsbrook Well, and the Scriber Wells). Within the GWC, this sub-basin is located within Category 4 which associates to the range of alteration of unimpacted August median flows due to groundwater withdrawal being between 25 to 55%. If the sub-basin withdraws an additional 0.102 MGD, then the category would change to Category 5. Within the BC, the sub-basin is located within Category 5 which associates to 65% or greater alteration of the Range of Fluvial Fish Relative Abundance. This represents fish communities that have undergone severe changes to their structure and function.

Sub-basin 12050 is listed as a positive value for NGD which indicates the sub-basin is depleted, but since this sub-basin has an August NGD of 22.4% which is below the 25% limit for August NGD, AWD does not need to submit a minimization plan.

There are no CFRs within this sub-basin.

Sub-basin 12059: Sub-basin 12059 has an area of approximately 21.11 square miles and about 11.9% of this area is impervious cover. Within this sub-basin, the AWD utilizes six groundwater wells (Conant No. 1 and Conant No. 2 Wells). Under the GWC, this sub-basin is located within Category 4 which associates to the range of alteration of unimpacted August median flows due to groundwater withdrawal being between 25 to 55%. If the sub-basin withdraws an additional 0.476 MGD, then the category would change to Category 5. Under the BC, the sub-basin is located within Category 5 which associates to 65% or greater alteration of the Range of Fluvial Fish Relative Abundance. This represents fish communities that have undergone severe changes to their structure and function.

Sub-basin 12059 is listed as negative for NGD which indicates the sub-basin is surcharged. This sub-basin currently has an August NGD of -0.6%.

CFRs do exist within this sub-basin. Since AWD is withdrawing from a sub-basin with a CFR, AWD will be required to have a consultation with MassDEP and EEA to discuss the potential to reduce any impacts to the CFR (regardless of tiers). Also after AWD's consultation, an implementation plan (described in 310 CMR 36.22(4)) may need to be submitted. The implementation plan consists of an evaluation of options to transfer withdrawals to other withdrawal points to minimize any impacts at the CFR.

Sub-basin 12066: Sub-basin 12066 has an area of approximately 12.58 square miles and about 8.8% of this area is impervious cover. Within this sub-basin, the AWD utilizes four groundwater sources (Whitcomb Well and the Clapp Wells). Under the GWC, this sub-basin is located within Category 4 which associates to the range of alteration of unimpacted August median flows due to groundwater withdrawal being between 25 to 55%. If the sub-basin withdraws an additional 0.263 MGD, then the category would change to Category 5. Under the BC, the sub-basin is located within Category 5 which associates to 65% or greater alteration of the Range of Fluvial Fish Relative Abundance. This represents fish communities that have undergone severe changes to their structure and function.

Sub-basin 12066 is listed as a positive value for NGD which indicates the sub-basin is depleted, but since this sub-basin has an August NGD of 2%, which is below the 25% limit for August NGD, AWD does not need to submit a minimization plan.

There are no CFRs within this sub-basin.

Sub-basin 12069: Sub-basin 12069 has an area of approximately 121.03 square miles and about 12.0% of this area is impervious cover. Within this sub-basin, the AWD utilizes four groundwater sources (Assabet Wells No. 1, 2, 1A and 2A). Under the GWC, this sub-basin is located within Category 4 which associates to the range of alteration of unimpacted August median flows due to groundwater withdrawal being between 25 to 55%. If the sub-basin withdraws an additional 2.182 MGD, then the category would change to Category 5. Under the BC, the sub-basin is also located within Category 5 which associates to 65% or greater alteration of the Range of Fluvial Fish

Relative Abundance. This represents fish communities that have undergone severe changes to their structure and function.

Sub-basin 12069 is listed positively for NGD. All permitees above 25% for August NGD (regardless of tier) are required to determine and implement ways to reduce withdrawals or return groundwater to basins or sub-basins to improve streamflow. Since AWD utilizes groundwater sources within this sub-basin which has an August NGD of 32.2%, which is greater than the 25% limit, AWD will need to submit a minimization plan (described in 310 CMR 36.22(5)).

CFRs do exist within this sub-basin. Since, AWD is withdrawing from this sub-basin, AWD will be required to have a consultation with MassDEP and EEA to discuss the potential to reduce any impacts to the CFR (regardless of tiers). Also after AWD's consultation, an implementation plan (described in 310 CMR 36.22(4)) may need to be submitted. The implementation plan consists of an evaluation of options to transfer withdrawals to other withdrawal points to minimize any impacts at the CFR.

3.6.1.3 Tiers

AWD's Baseline is 1.79 MGD which was determined from the volume withdrawn during 2005 plus 5%. If AWD does not request additional groundwater withdrawal above this Baseline and no change in GWC or BC, then AWD would be classified as a Tier 1. The WMA permit conditions for permittees that stay below this Baseline would include the following:

- Minimize impacts of existing withdrawals through demand management.
- Any permittees above 25% NGD must further minimize impacts to the greatest extent feasible.
- If CFR are present, permittee must conduct a desktop pumping evaluation and consult with agencies to minimize impact of withdrawals.

If AWD requests additional groundwater withdrawal above the Baseline and there is no change in the GWC or BC, then AWD would be classified under a Tier 2. The WMA permit conditions for permittees that stay below this Baseline would include the following:

- Continue demand management.
- If GWC is Category 4 or 5 or BC is Category 1, 2, or 3, develop and implement a Mitigation/Offsets plan.
- If above 25% NGD, demonstrate no feasible alternative source.
- If above 25% NGD, there is a CFR, or sources are within SWMI-defined natural resource areas, consultation with an agency may be required.

If AWD requests additional groundwater withdrawal above the Baseline and there is a change in the GWC or BC, then AWD would be classified under a Tier 3. The WMA permit conditions for permittees that stay below this Baseline would include the following:

- Continue demand management.
- Demonstrate there is no feasible alternative source that is less environmentally harmful.
- If GWC is Category 4 or 5 or BC is Category 1, 2, or 3, develop and implement a Mitigation/Offsets plan.
- If above 25% NGD, there is a CFR, or sources are within SWMI-defined natural resource areas, consultation with an agency may be required.

The exact details on what the requirements will be for the AWD will be known/defined when the AWD renews its WMA Permit.

SECTION 4

WATER SUPPLY EVALUATION AND ASSESSMENT

4.1 GENERAL

As presented within the previous two sections of this report, the Acton Water District (AWD) utilizes eleven active groundwater sources for its water supply. Withdrawal from each source of supply is permitted through the Massachusetts Water Management Act (WMA). The AWD's current permit includes the eight previously registered groundwater wells (Assabet 1A & 2A, Christofferson, Clapp, Conant 1, Lawsbrook, Scribner, and Whitcomb) and four permitted supply wells (Conant 2, Kennedy, Marshall, and Assabet 3). The registration authorizes a withdrawal of 1.56 million gallons per day (MGD) on average over the calendar year and the WMA permit authorizes an additional average daily withdrawal of 0.38 MGD. This results in a total authorized average daily withdrawal of 1.94 MGD for all sources.

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 AWD 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 (as per discussions with MassDEP). The individual withdrawals for the permitted sources are taken from the WMA permit.

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

Source	Individual Registered/Permitted Withdrawals (MGD)	Total Authorized Registered/Permitted Withdrawals (MGD)
Assabet No.1A	0.499	
Assabet No.2A	0.499	
Christofferson	0.400	
Lawsbrook	0.151	
Scribner	0.151	
Whitcomb	0.352	
Clapp	0.352	
Conant No. 1	0.468	
Registered Total:	**2.872**	**1.560**
Conant No. 2 Wells	0.216	
Kennedy Wells	0.540	
Marshall Wellfield	0.300	
Permitted Total:	**1.056**	**0.380**
COMBINED TOTAL:	**3.928**	**1.940**

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.

As presented within Section 2 of this report, the AWD treats its sources at five water treatment plants (WTPs) or just chemically at the source for Conant 1. Table 4-2 presents the pumping capacities of the AWD's current wells and associated WTPs.

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

Source	Well Capacity (MGD)	WTP Capacity (MGD)
Assabet No.1A	0.499	
Assabet No.2A	0.499	
Christofferson	0.400	
Lawsbrook	0.151	
Scribner	0.151	1.700
Whitcomb	0.352	
Clapp	0.352	0.704
Conant No.1	0.468	0.468
Conant No.2 Wells	0.216	0.216
Kennedy Wells	0.540	
Marshall Wellfield	0.108	0.500
Total:	**3.736**	**3.588**

It is noted that the actual capacity of a well is dynamic as wells lose capacity over time and regain 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. Although originally capable of pumping 208 gallons per minute (gpm) or 0.3 MGD, Marshall is currently understood to only be capable of pumping approximately 75 gpm or 0.108 MGD. Therefore, these well pumping capacities shall be the basis of analysis for determining if there is adequate water supply capacity.

4.2.1 Average-Day Demand Analysis

As discussed 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. It should be noted that other factors may prevent this practice from being strictly followed, such as water quality, regulatory compliance, and other operational considerations.

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

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

Source	Well Capacity (MGD)	WTP Capacity (MGD)	Available flow @ 16-hours of Pumping (MGD)
Assabet No.1A	0.499		
Assabet No.2A	0.499		
Christofferson	0.400		
Lawsbrook	0.151		
Scribner	0.151	1.700	1.133
Whitcomb	0.352		
Clapp	0.352	0.704	0.469
Conant No.1	0.468	0.468	0.312
Conant No.2 Wells	0.216	0.216	0.144
Kennedy Wells	0.540		
Marshall Wellfield	0.108	0.500	0.333
Total:	**3.736**	**3.588**	**2.392**

By comparing the projected average-day required total of 1.87 MGD for 2026, it can be seen that the AWD 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 AWD has the majority 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 (the Kennedy Wells).

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

Source	Well Capacity (MGD)	WTP Capacity (MGD)	Available flow @ 16-hours of Pumping (MGD)
Assabet No.1A	0.499		
Assabet No.2A	0.499		
Christofferson	0.400		
Lawsbrook	0.151		
Scribner	0.151	1.700	1.133
Whitcomb	0.352		
Clapp	0.352	0.704	0.469
Conant No.1	0.468	0.468	0.312
Conant No.2 Wells	0.216	0.216	0.144
Kennedy Wells	0.000		
Marshall Wellfield	0.108	0.108	0.072
Total:	**3.196**	**3.196**	**2.131**

By comparing the projected maximum-day required total of 2.74 MGD for 2026, it can be seen that the AWD system would not have adequate water capacity under this analysis scenario.

However, the AWD system would have adequate water capacity if the sources were operated longer than 16-hours (as 24-hour operation in the short-term could provide up to 3.196 MGD if needed and all other sources were operable).

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

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

Source	Well Capacity (MGD)	WTP Capacity (MGD)	Available flow @ 16-hours of Pumping (MGD)
Assabet No.1A	0.499		
Assabet No.2A	0.499		
Christofferson	0.400		
Lawsbrook	0.151		
Scribner	0.151	0.000	0.000
Whitcomb	0.352		
Clapp	0.352	0.704	0.469
Conant No.1	0.468	0.468	0.312
Conant No.2 Wells	0.216	0.216	0.144
Kennedy Wells	0.540		
Marshall Wellfield	0.108	0.500	0.333
Total:	**3.736**	**1.888**	**1.259**

By comparing the projected maximum-day required total of 2.74 MGD for 2026, it can be seen that the AWD 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 AWD currently has full emergency power provisions at the South Acton WTP site. Therefore, although possible, an extended loss of the sources via common interruptions such as loss of power is less likely to impact the source. Nonetheless, this scenario should still be considered a possibility as a potentially catastrophic event could occur that renders the largest sources inoperable. 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 AWD 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). Furthermore, the maximum-day demand could not be met under any scenario when the largest WTP was considered to be off-line.

Under the more stringent scenario (i.e., when a WTP is considered to be off-line), it should be noted that the water supply could be supplemented via the use of the Wampus booster station for a short period of time. With its highest capacity pump (900 gpm) operating, another approximately 1.3 MGD could be provided from the Wampus storage tank to supplement the shortfall for a short-period (just over two days) to meet the projected maximum-day demand. Since the Wampus site is not a constant source of supply (like a well), it must be routinely refilled. Its use would also be generally limited to its starting volume of water. Therefore, its use was not included in the previous analyses and should not be considered as such.

In order for the AWD to more reliably meet the maximum-day demands (if 24-hour operation is determined to be undesirable), other reliable sources of supply would need to be implemented to make up the difference. Based on the scenarios that considered the largest source to be off-line, a deficit of approximately 0.61 MGD (2.74 MGD – 2.131 MGD) is identified at the end of the planning period. When limited to a 16-hour pumping operation, this amount would correspond to a source having at least a 0.91 MGD capacity.

The following sections present available options to the AWD 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 AWD currently has emergency interconnections with the neighboring communities of Littleton, Maynard, and Concord. All of the interconnections are isolated with manual gate valves and are not metered. A transfer would need to be initiated manually.

Sudbury and Westford are the only other neighboring towns with a sizeable community water system that the AWD does not have an emergency interconnection with. The bordering towns of Boxborough, Carlisle, and Stow do not have their own water systems.

The establishment of a suitable interconnection and IMA for the purchase of water from a neighboring community would be required. 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 AWD system;
- A permanent, reliable, and redundant interconnection;
- Acceptable and compatible water quality; and
- No impacts to the AWD'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:

- Marginal or Production Cost: 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.

Additional effort would need to be expended by the AWD should it desire to pursue a formal interconnection with one of its neighboring community water systems.

4.3.1.2 MWRA

Another long term water supply alternative is an interconnection to the MWRA system. With the impact of water conservation and leak repair, the MWRA has a surplus of drinking water to provide to communities in need. This surplus has become an appropriate solution to Eastern Massachusetts communities that may have water quality and/or quantity issues. The MWRA's policies and terms have become increasingly favorable and the opportunity to extend the service area is more user-friendly.

The MWRA Board of Directors has modified their policy to allow the wheeling of water through adjacent towns. Previously, the policy required direct connections. The nearest communities served by MWRA water include Lexington and Bedford. Lexington is fully served by the MWRA while Bedford is only served water partially or for emergency purposes. Therefore, access to the MWRA for the AWD would require a wheeling agreement through the Lexington or Bedford distribution systems. Since AWD does not border either of these towns, either Concord and/or Carlisle would have to be involved in this interconnection. One possible scenario could be to install a pip up the I-495 corridor from Marlborough and then down the MA-111 through Boxborough to Acton.

In addition to specific local or regional issues, the following submittals and events are necessary to successfully progress through the MWRA admission process (in no specific order):

- MWRA - OP.10, Admission of New Community to MWRA Water System
- Massachusetts Environmental Policy Act (MEPA) – Environmental Notification Form (ENF), Draft and Final Environmental Impact Report
- Water Resources Commission – Inter-basin Transfer Act (IBTA) submittal & approval
- MassDEP – approvals based on actions requested
- District Meeting vote to support
- Massachusetts House and Senate approval
- Inter-municipal agreement to allow wheeling of water

The MWRA charges a one-time connection fee to new members to recover a portion of the capital costs already paid by existing members through the rate structure. This pro-rated fee varies from approximately \$5.0M to \$5.3M per million gallons per day of supply requested and is typically financed over a 20 year period. The current MWRA metered rate for water demanded by a community is pro-rated at \$3,582.09 per million gallons (FY2018), equivalent to \$2.68 per 100 cu/ft.

The MWRA membership process can take up to a few years for an eligible community to seek and acquire membership through the approval of the MWRA Advisory Board and Board of Directors. The variability of this timeline is caused by several factors, some of which are outside of the control of the applicant. These reasons include:

- The community's commitment to get it done
- The attitude of the specific interest and stakeholder groups
- The objective and timely review of regulatory applications
- The cooperation of member communities

- The availability of MWRA water and
- The experience of the strategic advisor and consultant

Currently, the nearest fully served MWRA water customer is Lexington, Massachusetts. For the AWD to connect to the MWRA, many technical challenges would require further study including:

- Water quality issues from mixing a chloraminated water with the AWD's water.
- Modeling to determine hydraulic constraints or improvements needed in the Lexington system (as well as the system's supplying Lexington) to deliver the needed flow and pressure to Acton.
- Wheeling costs including distribution and other costs on top of the cost to purchase the water from the MWRA.

A dedicated raw water main from the MWRA system in Lexington (or possibly Bedford) to Acton would be required. Not knowing the exact route (or size) that the transmission main would need to be, this connection is preliminarily estimated to be approximately 10+ miles in length. A direct, dedicated main, would eliminate the complexity of wheeling water through the Bedford distribution system but would be costlier.

An estimated cost to just construct a dedicated main as described above would be $10-$12M and this would be in addition to the cost to build an appropriate pumping station (or stations), any upstream hydraulic improvements, purchase of water, and for MWRA access fees.

Another potential route with similar challenges could be through the southern communities for a connection with Framingham or possibly Marlborough. Without the cooperation of communities along the needed route or interest to connect themselves, similar costs can be expected.

Therefore, the cost for an MWRA connection may be prohibitively expensive to consider for additional supply.

In addition to the immediate up front capital costs to be incurred for an MWRA connection (which are more readily quantifiable), other items which are harder to quantify include the following:

- Loss of local control over the water supply.
- Loss of control with pricing (annual rate increases over the next four years are expected to rise to an average 3.9% through 2020 according to MWRA).
- Community desire for a chloraminated water supply.

4.3.2 New Sources

Another alternative for improved long term water supply would be the implementation of a new groundwater well source or sources. Potential sources in town include potential bedrock wells, a potential well(s) on the Flannery-O'Toole property in West Acton, and a surface water supply option from Nagog Pond.

4.3.2.1 Bedrock Wells

In 2000, the AWD had a fracture trace and lineament study performed by D.L. Maher Company to determine the possibility of bedrock well sources in the north Acton area. Based on their initial review, D.L. Maher identified several sites in the north Acton area that were recommended to be further investigated.

Although no potential capacities were identified within the report, the capacities for other high-yield bedrock wells in the town of Acton were noted to range from 40 to 80 gpm. Bedrock wells can have poorer water quality (e.g., iron, manganese, radon, etc.) than traditional wells; however, their proximity to the newer North Acton WTP (designed with aeration and iron and manganese removal) or any other existing WTP in Acton may make development of the wells more attractive should similar treatment be required. Therefore, the AWD may want to consider moving forward with the investigation to further determine the viability of bedrock wells as a supplemental supply.

4.3.2.2 Flannery-O'Toole Property

Another location within the town that had previously been identified as a potential new source is the Flannery-O'Toole property in West Acton. It is understood that previous investigations (also by D.L. Maher) had indicated that a tubular wellfield with a capacity of approximately 350 gpm (0.5 MGD) would be possible. However, the source was not pursued at that time due to its marginal water quantity.

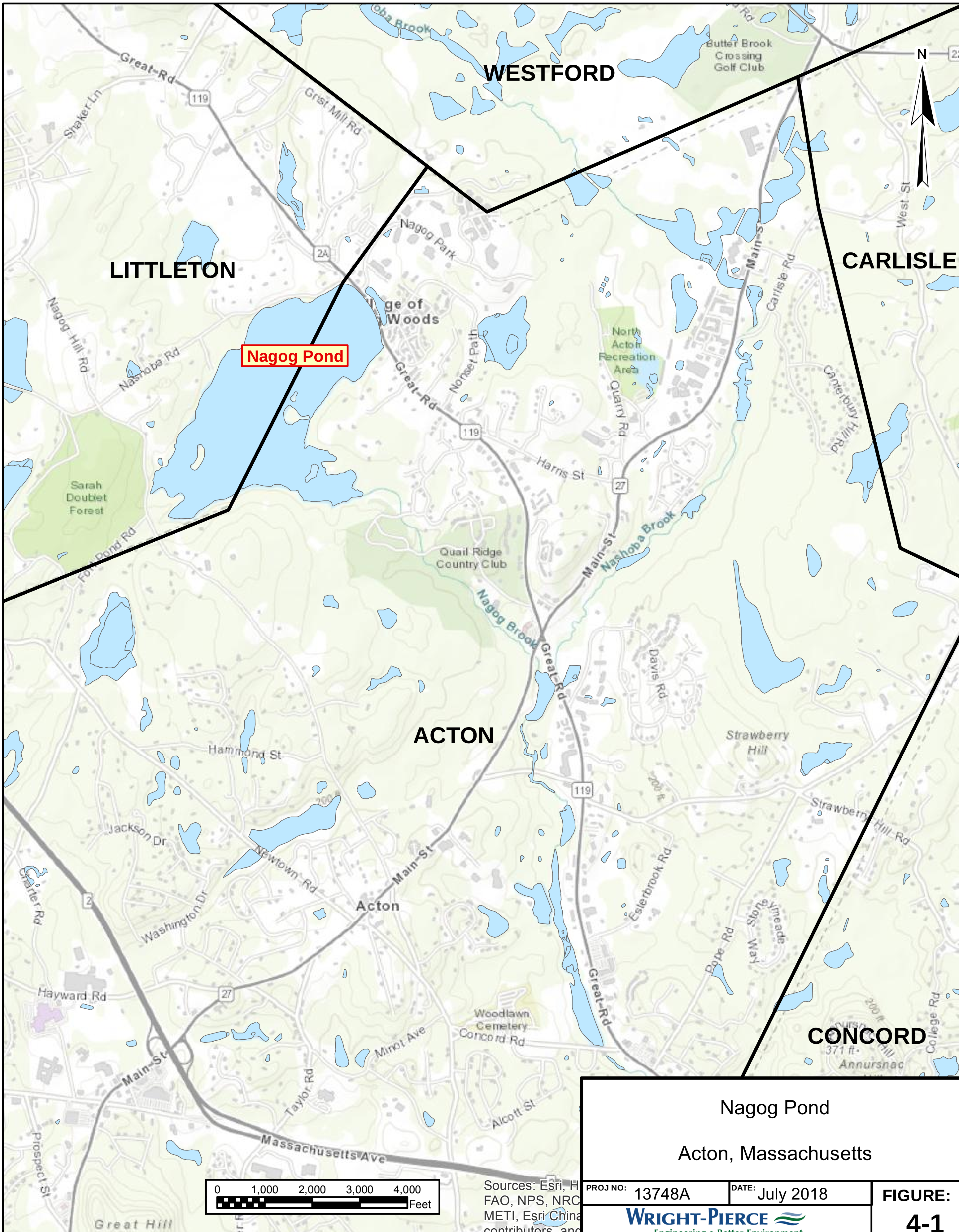
It is noted that good water sources of high quantity and/or high quality (i.e., no treatment required) are very rare. Therefore, the AWD may want to consider further investigation of the property for additional supply capacity. If water quality treatment is needed, it may be a candidate for combined water treatment with the Whitcomb and Clapp sources if a facility is determined to be required in the future.

The ability to successfully permit a new well supply is not certain under the Water Management Act and is getting ever more involved. The process can be very costly, complicated, and likely take several years if successful. A description of the typical New Source Approval (NSA) process is presented later within Section 4.3.2.5.

4.3.2.3 Nagog Pond

As described within Section 1 of this report, the District currently acquires all of its drinking water from groundwater supply wells. When and if the District needs or desires to pursue additional supply, a surface water supply option could be further investigated.

Nagog Pond, which is located within both the towns of Acton and Littleton as shown on Figure 4-1 is a potential surface water option. The source however is currently used for water supply by the Town of Concord but is understood to be potentially available for use by Acton (and/or Littleton) per Section 10 of the 1884 Massachusetts General Laws Chapter 201.



Nagog Pond

Acton, Massachusetts

PROJ NO: 13748A	DATE: July 2018	FIGURE:
WRIGHT-PIERCE Engineering a Better Environment		4-1

Based on discussions with MassDEP, Nagog Pond has a "firm yield" of 0.86 MGD averaged over the period of a year. The Town of Concord is also understood to be in the process of designing a WTP upgrade for the source with a capacity of 1.5 MGD (nearly twice the firm yield) at an estimated construction cost of $12.5 Million. Concomitant and compelling, prudent concern for maintaining the volume of Nagog Pond will include a reliable monitoring system that would provide assurance that the level of the pond remains stable.

The cost to construct a surface water treatment plant is often more expensive than that of a typical groundwater source. Based on the construction estimate for Concord's proposed WTP, a unit cost of approximately $8.3 Million per MGD of treated water capacity can be estimated. Depending on the identified need for Acton, a similar if not higher unit cost should be expected as a lower design capacity (e.g., 0.5 MGD) would have a higher unit cost per million gallons of treated water capacity. This is because the majority of the initial cost for a treatment plant (e.g., the building and supporting infrastructure) is required nonetheless and is less dependent on the increasing treatment capacity.

Additionally, should the District want to pursue the source on its own, the pilot study process would first need to be undertaken (e.g., if a different treatment process like membrane filtration was desired). This would also be more expensive than the others completed in the past as surface water pilot studies require a minimum two season duration (i.e., during warm and cold water conditions).

It is also understood that the District does not currently own any land in the general vicinity of the source that would be suitable for the construction of a surface water treatment plant. Additional land acquisition costs and/or raw water pumping station and transmission main costs would be incurred to convey the water to a suitable site.

Some potentially lower cost options for the District to consider should Nagog Pond water be needed or desired include the following (each with their own pros and cons beyond the scope of this document):

- Enter an intermunicipal agreement with the Town of Concord to purchase treated water.
- The potential for the future WTP to be a regional resource (i.e., a shared supply between the three Towns of Concord, Acton, and Littleton).

4.3.2.4 Assabet Well No. 3

Although approved as a source for the AWD, Assabet No. 3 is currently only considered a redundant source (i.e., additional volume has not been approved). As discussed within Section 2 of this report, the well has excessive manganese concentrations and has had 1,4-Dioxane concentrations above the Massachusetts Drinking Water Guideline. Because of this, it has not yet been connected to the water system. Depending on MassDEP's requirements, the source may be able to be used if dilution with other sources continues to be effective at managing concentrations in a manner protective of public health (or if 1,4-Dioxane treatment is provided). As previously discussed, the South Acton WTP has designated space for future treatment should its connection to the WTP be implemented (or 1,4-Dioxane be detected at higher concentrations in the other connected sources). Additionally, the SAWTP has been designed for the additional capacity should Assabet 3 be permitted for additional volume. It is also understood that the AWD is considering to perform a long term pump test on the source in the near term.

4.3.2.5 New Source Approval Process

The New Source Approval (NSA) process, 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 hydrogeologic 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 AWD 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 Acton Water District provides water to its customers from eleven active source locations consisting of twenty-two individual wells located throughout the Town of Acton. The eleven active sources are reported to have been installed as early as 1955 with Conant Well No. 1 and more recently with the replacement wells at the Scribner Wellfield in 2002.

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 AWD'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. Available performance data for each well from previous cleaning operations is presented in Table 4-6.

TABLE 4-6
WELL CLEANING DATA
ACTON, MASSACHUSETTS

Source	Date	Before Redevelopment		After Redevelopment		% Improvement
		Specific Capacity (GPM/FT)	Pumping Rate (GPM)	Specific Capacity (GPM/FT)	Pumping Rate (GPM)	
Assabet No.1	11/00	13	200	19	350	46%
Assabet No.1A	02/10	42.7	329	45.7	448	7%
Assabet No.2	06/15	4.4	30	16.5	317	275%
Assabet No.2A	06/12	23	174	36	180	57%
Christofferson	07/18	22	172	26	300	18%
Lawsbrook	03/01	48	239	64	236	33%
Scribner Wellfield	10/01		200			
Conant No.1	11/11	21.57	304	26.34	299	22%
Conant No.2						
*Well No.1**	10/98	22.8	210			
*Well No.2**	10/98	6.9	90			
*Well No.3**	10/98	21.3	200			
*Well No.4**	10/98	34.4	169			
*Well No.5**	10/98	3.78	75			
Kennedy No.1						
Well No.1	02/16	42.96	116	40.17	200	72%
Well No.2	02/16	9.15	110	11.68	125	28%
Well No.3	02/16	17.03	90	24.35	179	43%
Well No.4	02/16	37.2	160	48.61	175	31%
Clapp No.1	02/18	4.9	50	7.26	90	48%
Clapp No.3	02/18	7.4	96	14.89	140	101%
Whitcomb	07/11			140	210	
Marshall						

* Before treatment data is original specific capacity and pumping rate when well was first constructed.

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.

At this time, the Marshall Source cannot consistently pump its permitted amount and its use has been limited. The source is comprised of an older manifolded wellfield and vacuum prime type pumping system. A corroded manifold system (that is likely introducing air into the system under pumping conditions) as well as the small 2½" tubular wells that cannot be cleaned has likely caused the loss of capacity. In 2011, Wright-Pierce evaluated the hydrogeologic conditions of the wellfield site and determined that four gravel packed wells should be able to provide a combined flow of approximately 200 gpm (if not the original 208 gpm). Based on the results from the field investigation, a Proposal for Replacement Wells was submitted to MassDEP and approval received on August 22, 2011. At this point, the design, permitting, bidding and construction phases remain for the replacement wells and related pumping station modifications. A budget of approximately $400,000 was preliminarily estimated for the completion of these remaining tasks.

Based on the identified need for additional supply per the previous analyses, it is recommended that the AWD proceed with the rehabilitation of the Marshall source in the near term to strengthen the reliability of its system by optimizing its current sources. This short-term supply improvement in conjunction with a parallel, but longer-term, pursuit of a new source is important to ensure the reliability of its current water supply while planning to meet the community's growing demand.

4.4 SOURCE TREATMENT

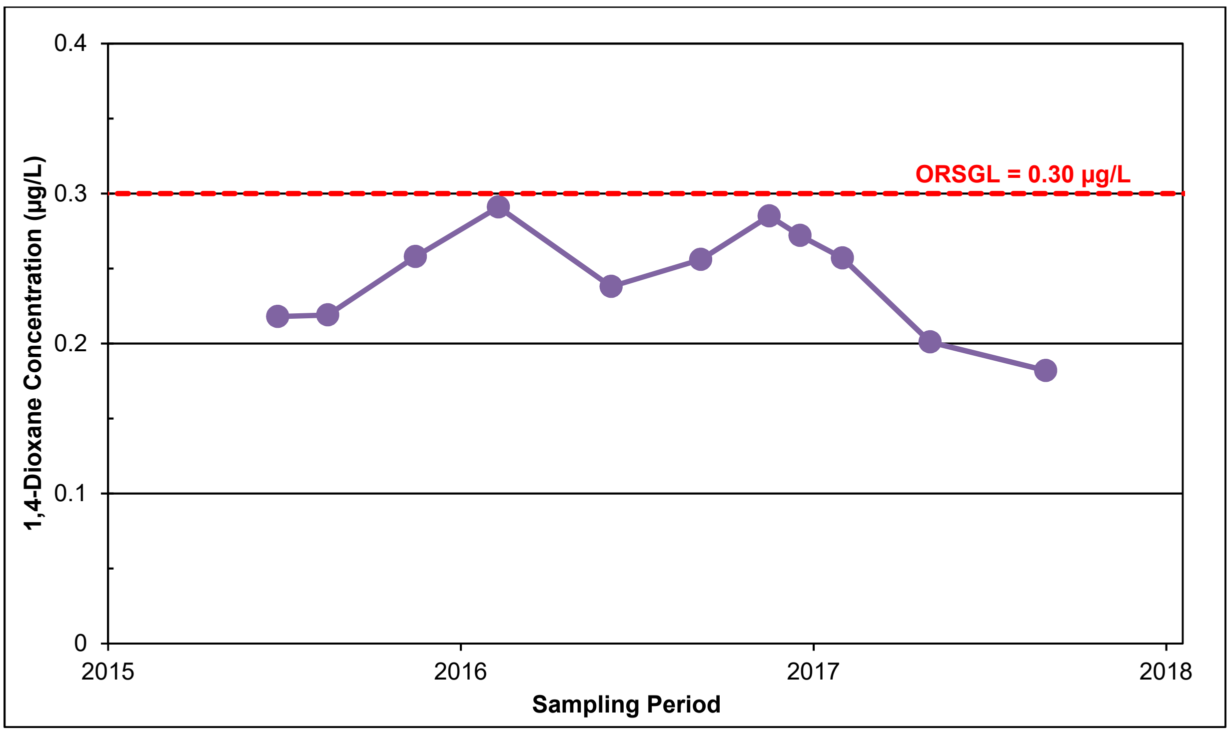
As was presented within Section 2 of this report, all eleven of the AWD's sources are treated but to a varying degree. In summary:

- The Kennedy and Marshall sources are fully treated at the North Acton WTP (aeration, membrane filtration, and chemical conditioning).
- The Assabet 1A, Assabet 2A, Christofferson, Lawsbrook, and Scribner sources are treated at the recently constructed South Acton WTP (aeration, membrane filtration, and chemical conditioning).
- The Clapp and Whitcomb sources are treated at the Clapp/Whitcomb WTP (aeration, granular activated carbon filtration, and chemical conditioning).
- The Conant No. 1 source is only chemically conditioned at its pumping station.
- The Conant No. 2 source is treated at the Conant WTP (aeration and chemical conditioning).

Due to the prevalence of nuisance secondary constituents (e.g., naturally occurring iron, manganese, and organic color) at varying concentrations within all of its sources, Wright-Pierce performed a Desk-Top Well Treatability Analysis for the AWD in 2008. The Wright-Pierce report evaluated and presented treatment options for all of the sources in order of recommended priority.

The South Acton WTP was recently constructed to treat the secondary constituents for the Assabet and School Street sources. Treatment of all five sources is comprised of aeration, membrane filtration, and chemical conditioning. Room for enhanced treatment is available at the WTP should treatment for 1,4-Dioxane ever be determined to be required. The available water quality data for 1,4-Dioxane at the South Acton WTP is presented within Figure 4-2.

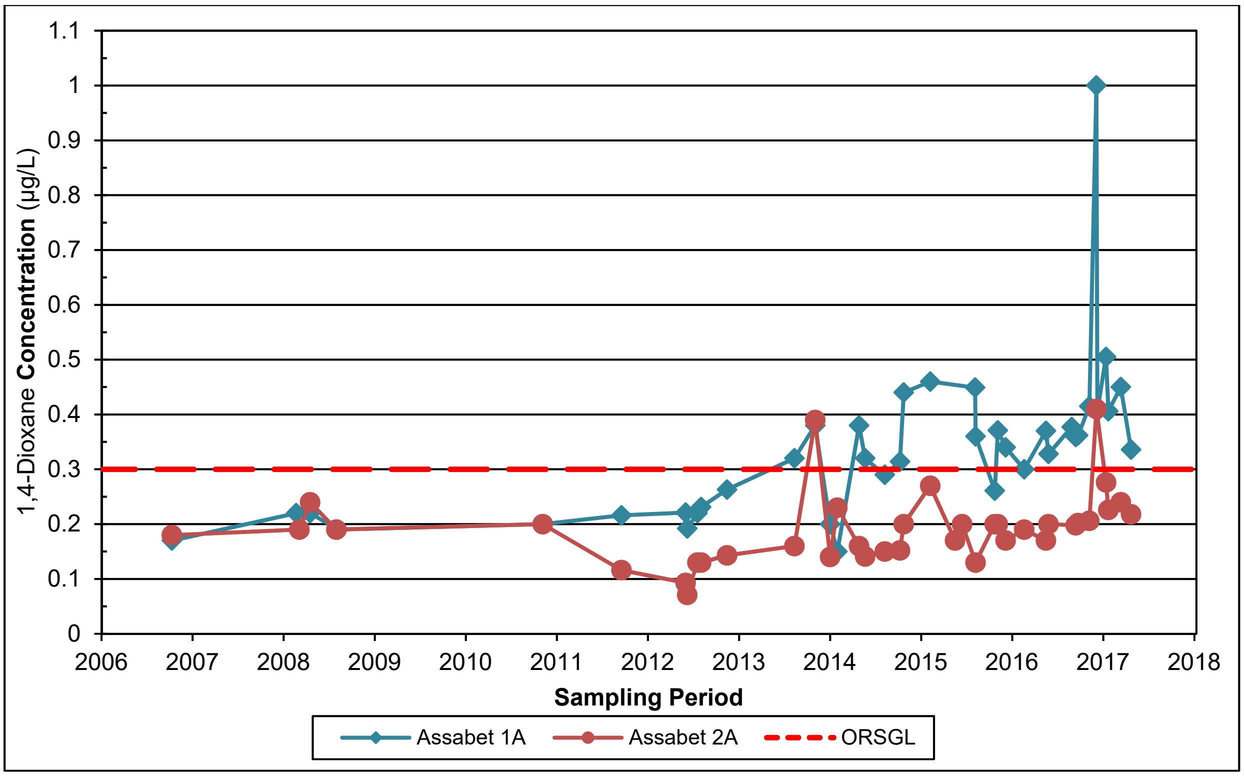
FIGURE 4-2
1,4-DIOXANE CONCENTRATIONS AT SOUTH ACTON WTP
ACTON, MASSACHUSETTS



The available data presented in the figure above indicate that there have not been any 1,4-Dioxane exceedances above the ORSGL at the South Acton WTP. The concentrations have ranged from a low of 0.182 µg/L on August 31, 2017 to a high of 0.291 µg/L on February 10, 2016.

Samples have been taken since 2006 at each of the well sources (Assabet and School Street) for 1,4-Dioxane and the available data is presented within Figures 4-3 and 4-4.

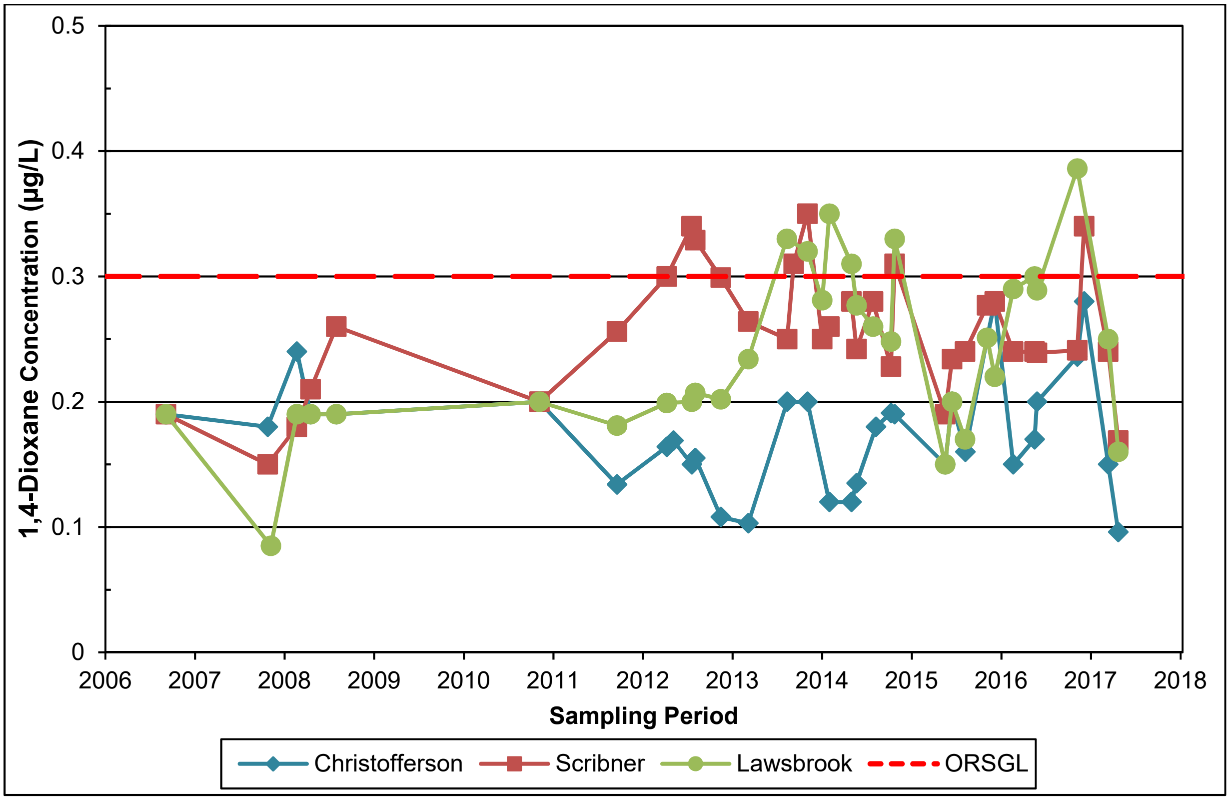
FIGURE 4-3
1,4-DIOXANE CONCENTRATIONS FOR ASSABET SOURCES
ACTON, MASSACHUSETTS



The 1,4-Dioxane concentrations at Assabet 1A and 2A appear to have been slowly increasing since 2006. As shown in the figure, the concentrations at Assabet 1A have recently been exceeding the 1,4-Dioxane ORSGL of 0.30 µg/L (0.0003 mg/L) over the past few years, with the highest concentration reaching 1 µg/L on December 15, 2016.

The historic data available for 1,4-Dioxane at the School Street sources (Christofferson, Scribner, and Lawsbrook) is presented in Figure 4-4.

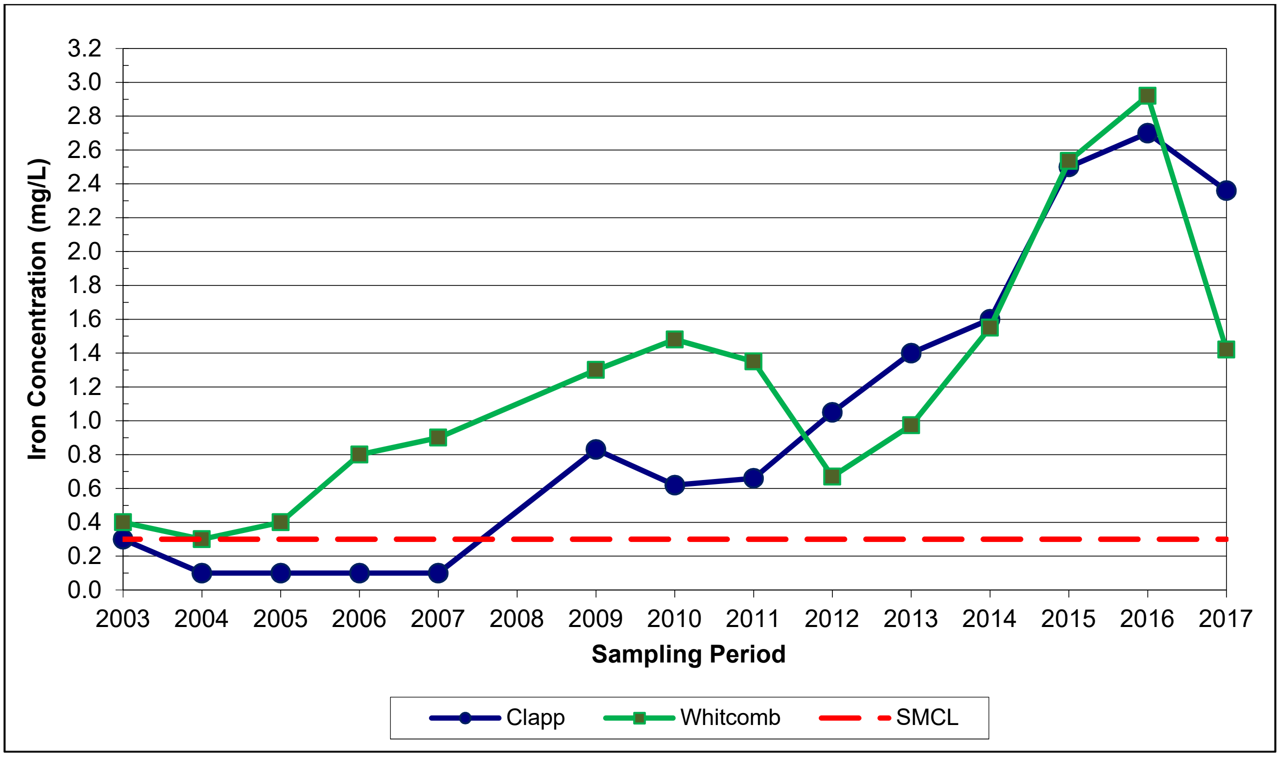
FIGURE 4-4
1,4-DIOXANE CONCENTRATIONS FOR SCHOOL STREET SOURCES
ACTON, MASSACHUSETTS



The Scribner and Lawsbrook Wells have had several 1,4-Dioxane exceedances over the ORSGL, while the Christofferson Well has been consistently below the ORSGL.

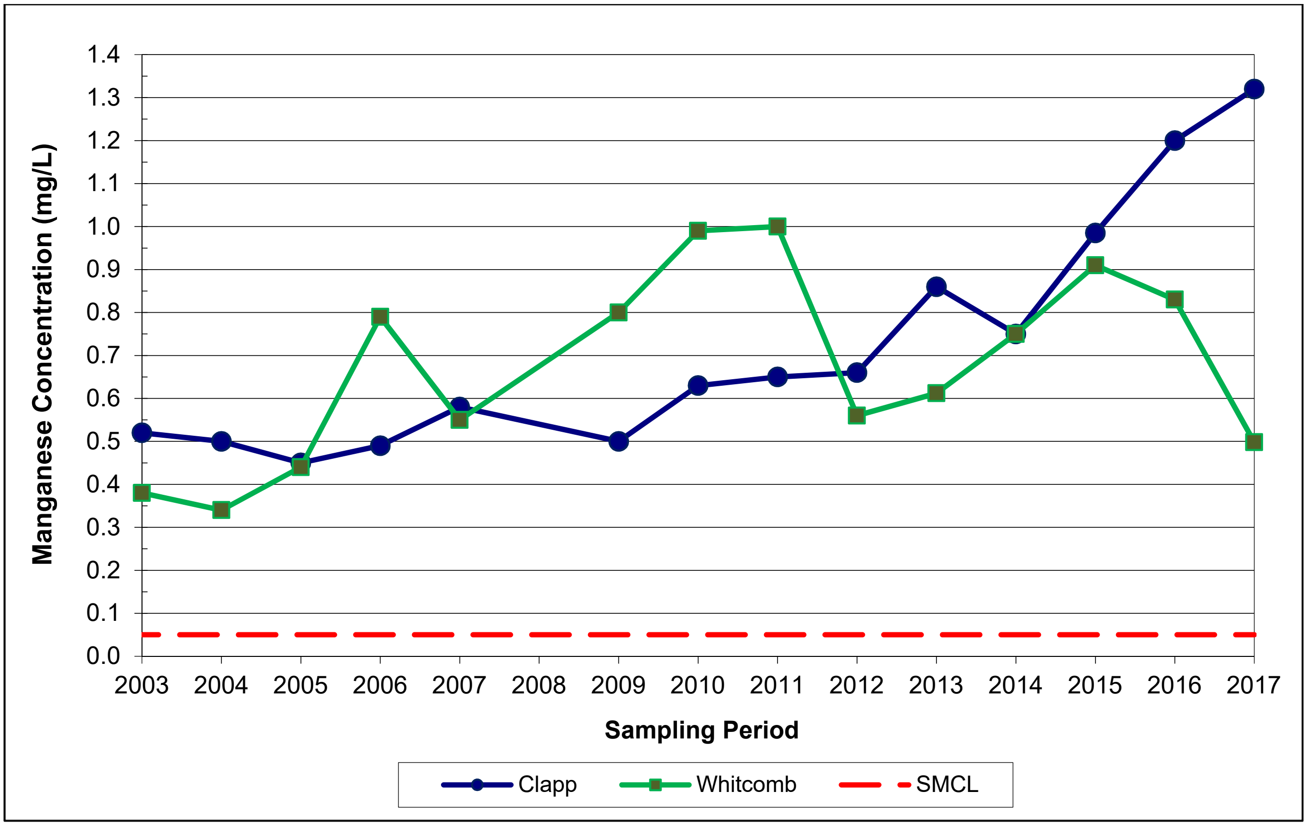
Only the Clapp, Whitcomb, and Conant sources currently remain without treatment for the removal of the same secondary constituents that have caused consumer complaints throughout the District. Therefore, these five sources are further discussed within this section regarding these same secondary constituents. Additional information related to a regulatory review is presented later within Section 6 of this report. Figures 4-5 through 4-10 present the available data on raw water iron, manganese, and color for the five sources.

FIGURE 4-5
IRON CONCENTRATIONS FOR WHITCOMB AND CLAPP SOURCES
ACTON, MASSACHUSETTS



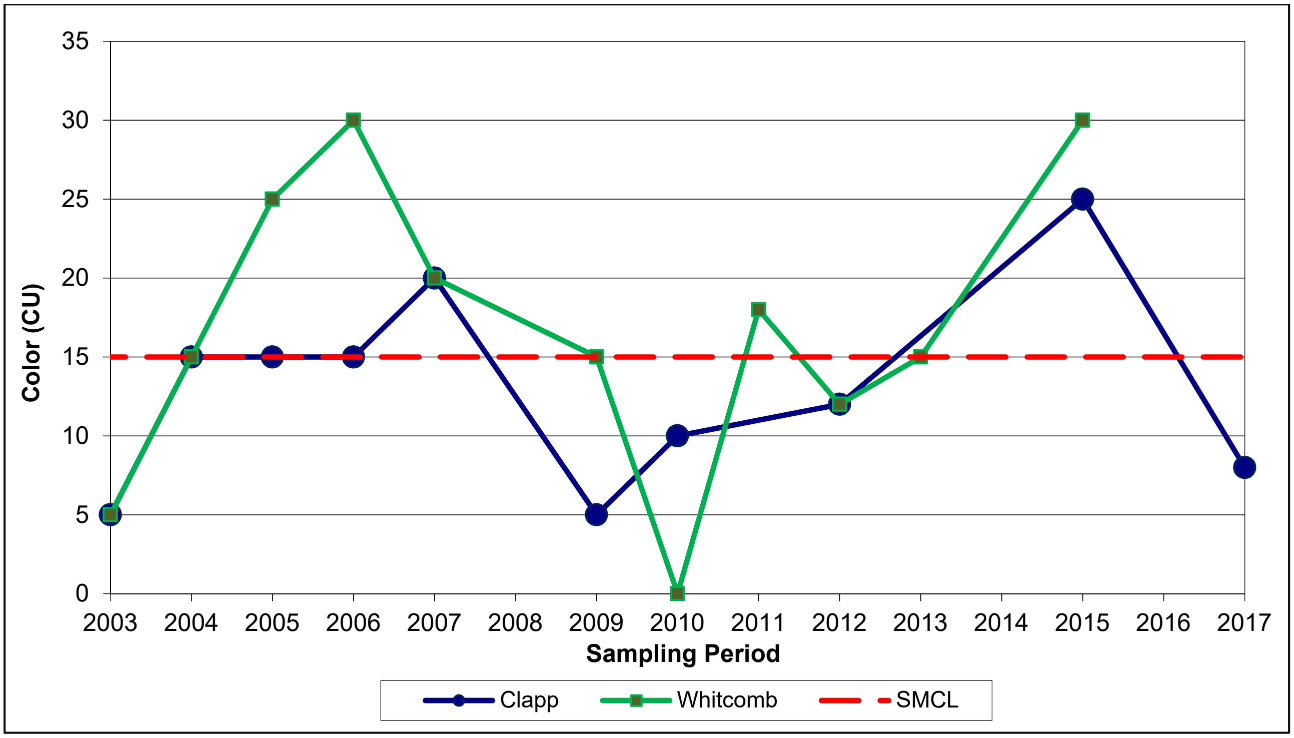
The iron concentration at both sources reached their peak concentration in 2016 with concentrations of 2.7 mg/L and 2.92 mg/L for Clapp and Whitcomb, respectively. These concentrations are well above the corresponding Secondary Maximum Contaminant Limit (SMCL) of 0.30 mg/L and have been exceeding this SMCL since 2008 for Clapp and 2004 for Whitcomb. Without removal, these concentrations of iron (ranging from approximately 4 to 9 times the SMCL) will continue to contribute to consumer complaints about “dirty water”.

FIGURE 4-6
MANGANESE CONCENTRATIONS FOR WHITCOMB AND CLAPP SOURCES
ACTON, MASSACHUSETTS



As shown in Figure 4-6, both sources have consistently had manganese concentrations well above the corresponding Secondary Maximum Contaminant Limit (SMCL) of 0.05 mg/L. The manganese concentration at the Clapp Well has just recently reached its peak concentration of 1.32 mg/L on March 31, 2017. Without removal, these concentrations of manganese (ranging from 7 to 26 times the SMCL) will continue to contribute to consumer complaints about "dirty water".

FIGURE 4-7
COLOR MEASUREMENTS FOR WHITCOMB AND CLAPP SOURCES
ACTON, MASSACHUSETTS

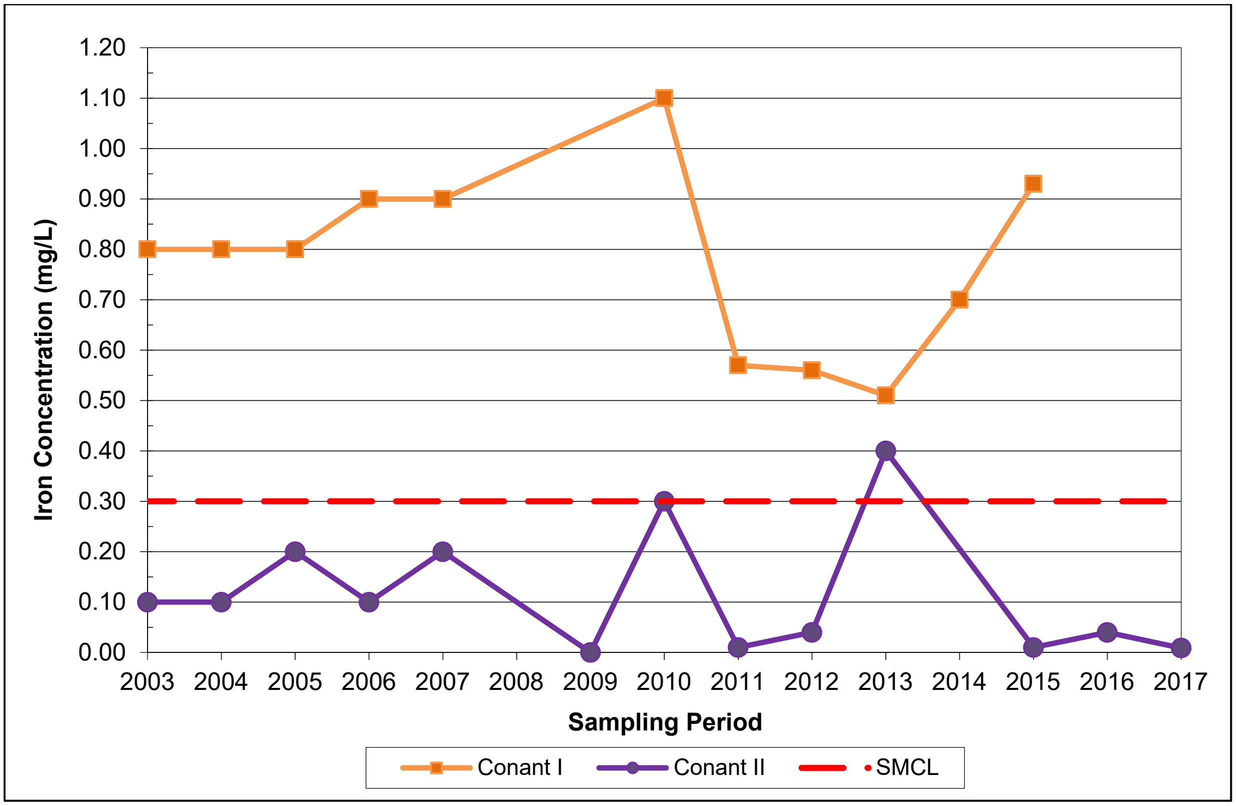


The color levels for both the Clapp and Whitcomb wells have varied from below to higher than the corresponding SMCL of 15 CU for color. Although not determinable from the data, it is likely that the color is inorganic in nature and a result of the already oxidized iron and manganese within the raw water.

If the raw water from the Clapp and Whitcomb wells is left untreated (for iron and manganese removal), it will continue to significantly contribute to "dirty water" complaints from consumers.

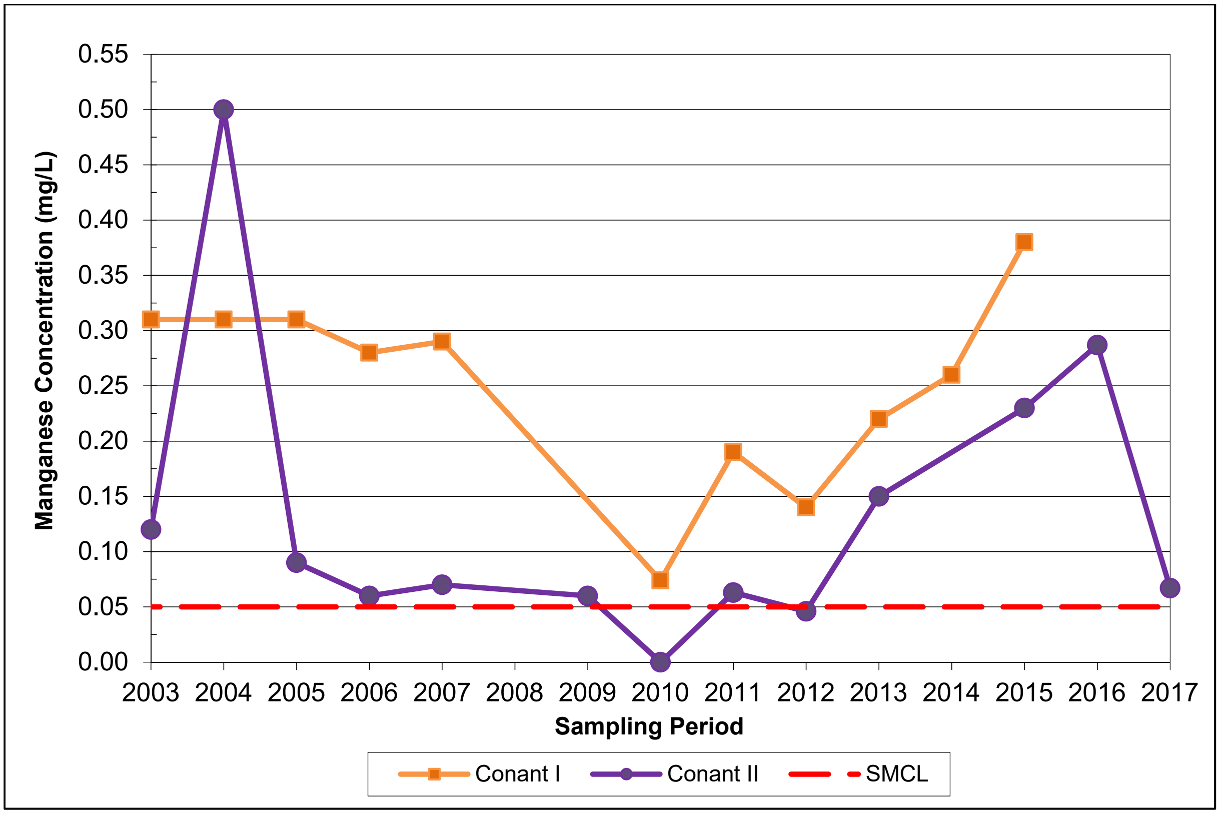
The historic data available for iron, manganese, and color for the Conant sources are provided in Figures 4-8 to 4-10.

FIGURE 4-8
IRON CONCENTRATIONS FOR CONANT SOURCES
ACTON, MASSACHUSETTS



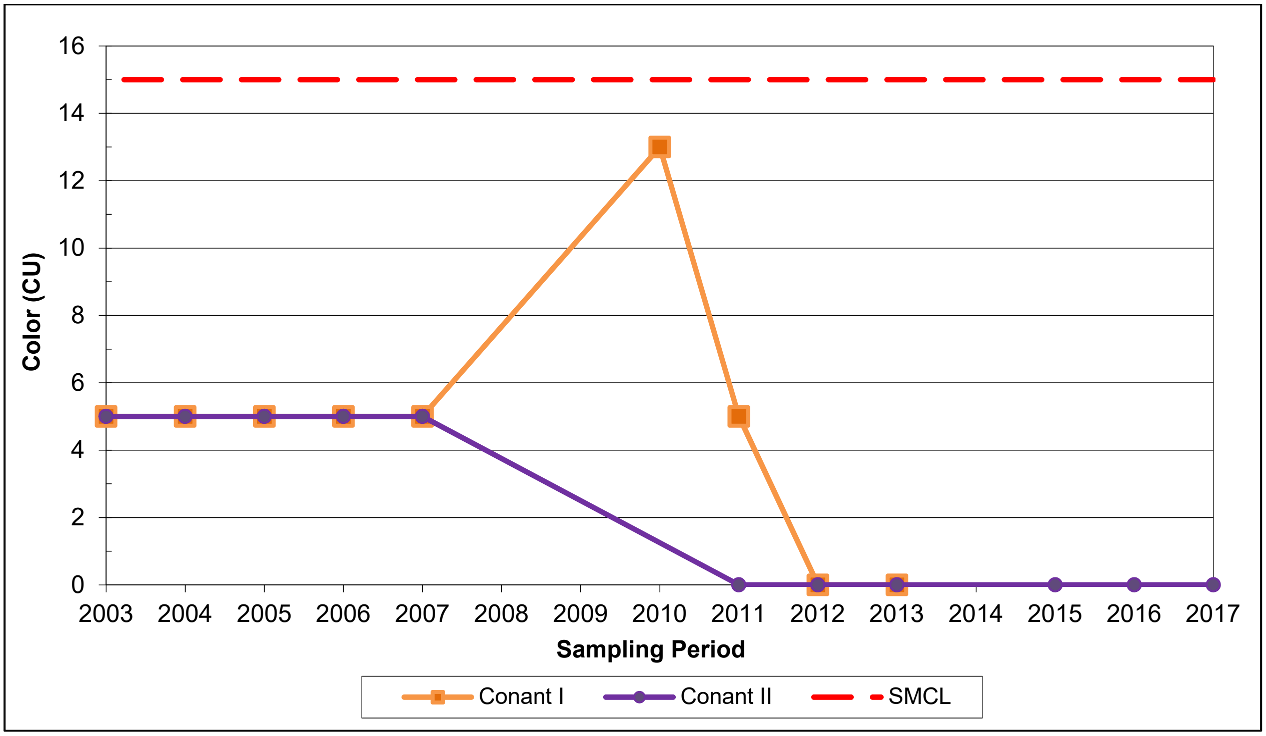
As previously presented within Section 2, the Conant 2 source is of better water quality than that of Conant 1, and as such is used more regularly. As can be seen from the data, the iron concentrations for Conant 2 are typically below the corresponding SMCL while those for Conant 1 are consistently above (and vary from approximately 2 to 4 times the SMCL).

FIGURE 4-9
MANGANESE CONCENTRATIONS FOR CONANT SOURCES
ACTON, MASSACHUSETTS



Although the manganese concentrations for both Conant sources are predominantly above the corresponding SMCL, those for Conant 1 are significantly higher (and have ranged from approximately 2 to 6 times the SMCL). With the exception of one data point in 2017, the manganese concentrations for both Conant sources have been consistently increasing since 2010.

FIGURE 4-10
COLOR MEASUREMENTS FOR CONANT SOURCES
ACTON, MASSACHUSETTS



The color concentrations for both Conant 1 and 2 have been consistently below the SMCL.

Although much better in water quality than that of Clapp and Whitcomb wells, if the raw water from the Conant 1 well is utilized and left untreated (for iron and manganese removal), it will also continue to contribute to "dirty water" complaints from consumers. Based on the available data presented, continued use of the Conant 2 source should be expected to contribute to "dirty water" complaints with its increasing manganese concentrations.

Currently there is a combined treated water output capacity of 2.2 MGD that includes removal of secondary constituents available (1.7 MGD from the South Acton WTP and 0.5 MGD from the North Acton WTP). Based on the water use projections for the planning period from Section 3 of this report, this would be able to provide for the system's projected average-day demand of 1.87 MGD, but not the system's projected maximum-day demand of 2.74 MGD. Therefore, it will be important for the AWD to supplement the volume from other sources during the higher demand period. If treatment is not provided (for removal of secondary constituents) at the supplemental sources that are used, consumer complaints will continue during the higher demand periods as oxidized minerals are reintroduced into the water system.

Based on well capacity (Table 4-1), it is recommended that the next WTP be planned to treat either the Clapp and Whitcomb sources or the Conant 1 and Conant 2 sources. With a registered withdrawal of 0.352 MGD from each the Clapp and Whitcomb sources, an additional 0.70 MGD of treated water quality would be made available. This would make a combined treated water output capacity of 2.9 MGD available (1.7 MGD + 0.5 MGD + 0.70 MGD) for the AWD that would be sufficient to meet the system's projected maximum-day demand of 2.74 MGD. As for the Conant sources, a combined 0.68 MGD (0.468 MGD from Conant 1 and 0.216 MGD from Conant 2) of treated water quality would be made available. This would make a combined treated water output capacity of 2.88 MGD available (1.7 MGD + 0.5 MGD + 0.68 MGD) for the AWD that would also be sufficient to meet the system's projected maximum-day demand of 2.74 MGD. Furthermore, implementation of treatment for the Clapp and Whitcomb sources or the Conant sources would provide the following:

- A third WTP for increased redundancy
- A WTP in the western or eastern portion of Acton
- Provide the AWD with flexibility in its treatment operations so that the treated wells are allowed to rest and not solely rely on the North Acton and South Acton WTPs

The process for the next WTP would need to be started with piloting for technology verification, and proceed with permitting & design, through construction. For this entire process, the AWD should be plan for an approximate three year period.

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 AWD has emergency power provisions installed at the following water supply locations:

- Kennedy WTP (0.50 MGD with only the Kennedy Wells)
  - Kennedy Wells
- Assabet WTP (0.998 MGD)
  - Assabet Wells 1A and 2A
- School Street WTP (0.4 MGD with only the Christofferson Well)
  - Christofferson Well
- South Acton WTP (1.0 MGD)
- Wampus Hill Booster Pump Station (up to 1.3 MGD)

With these five locations, the AWD has the capability to provide its projected maximum day demand for the planning period during a short-term loss of power with a pumping capacity of approximately 4.20 MGD. Not including the Wampus Hill Tank volume, the AWD also has an additional useable volume of 2.65 MG in its other three storage tanks (as presented later within Section 5 of this report). Therefore, the AWD currently has adequate provisions for emergency power according to the MassDEP requirements presented for the ability to provide the maximum daily demand.

Should the AWD desire to have full emergency power provisions, suitable emergency generators would need to be installed at all of its other source locations.

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 Acton Water District (AWD) 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. A computer hydraulic model was originally developed in 2010 and has been updated as required to reflect any known changes in the distribution system since 2010. This model was used as the hydraulic tool for analyzing the condition of the Acton 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 9 - Recommendations.

5.2 DISTRIBUTION SYSTEM COMPUTER MODEL

A computer hydraulic simulation model of the Acton water distribution system was developed in 2010 for the AWD to assist in the evaluation of the adequacy of the distribution system under various conditions. Since its development, Wright-Pierce (WP) has utilized the model to assist the AWD evaluate the causes of isolated water quality complaints in addition to select hydraulic analysis. This model was also used by WP for the last Master Plan Update completed in 2012. WaterGEMS V10 hydraulic modeling software as manufactured by Bentley Systems Inc. (formally Hastead Methods) was used as the software modeling tool for the master plan. The element features or attributes assigned to the water system utilities included: pipe material and diameter, pipe friction coefficient (Hazen-Williams C-Value), storage tank operating elevations, pump and tank level controls, altitude valve controls, and water system pump operation parameters. Calibration of the model was described in detail in a memorandum prepared by WP and submitted to the AWD in November of 2010.

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 three demand conditions, using the computer hydraulic model:

- <u>Peak Hour on Maximum Day in the year 2026</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 2026 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 Acton 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. Recommended improvements are presented in Section 7.

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 record information provided by the Acton Water District in addition to Town GIS data.

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 fire fighting. 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.

Currently there are no areas served with static pressures below 40 psi. Approximately 45 percent of Acton's water system has static pressures between 80 and 100 psi, and approximately 25 percent of nodes have static pressures of greater than 100 psi. Figure 5-1 represents a color coded static pressure node map for various pressure ranges. Areas where pressure exceeds 120 psi are predominantly located in the southeast portion of the system where elevations are lowest.

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.

Static Pressure Node Map
Acton Water District
Acton, Massachusetts

PROJ NO: 13748A	DATE: Jan 2018	FIGURE: 5-1

WRIGHT-PIERCE
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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 which have velocities which 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. Transmission mains from the wells will also experience velocities between approximately 1.8 to 3.0 fps depending on number of wells in operation.

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 AWD distribution system is generally well looped, with very few long dead-end mains. For purposes of this analysis only large diameter dead-ends were identified. The only large diameter water main dead-end identified is approximately 3,200 linear feet of 10-inch DI main on Pope Road from the Wingate Lane to Proctor Street. 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 fire fighting 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 Acton are considered to have fire flow requirements of 3,500 gpm.

The Acton public water system is predominately comprised of residential customers (87%), 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 1999. Table 5-2 lists the results of the model simulations of the available fire flows coincident with the projected year 2026 maximum-day demand for ISO locations throughout the service area. It should be noted that hydrant flow testing was completed in 2010 by Wright-Pierce at representative locations throughout the system in order to calibrate the hydraulic model. The details of these test results were included as part of the Hydraulic Model Calibration memorandum submitted to the District in 2010.

The available fire flows shown in Table 5-2 differ from the ISO field testing results completed in 1999 because of varying pumping rates, system demands and tank elevations during the testing period along with distribution system improvements performed since the testing. In addition, the available fire flows presented are based on maintaining a minimum 20 psi residual in all areas of the distribution system. Normal field testing procedures do not take into account pressures in the distribution system other than at a test hydrant. A discussion of piping replacement options to improve fire flows in deficient areas of the system follows.

5.3.5.1 Fire Flow Deficiencies

In general, Acton 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 eleven inadequate fire flow areas under current maximum day demand conditions. However, it is noted that all locations experienced an increase in estimated fire flow since the last Master Plan. This is likely attributed to the new system hydraulics with the new South Acton WTP location and water main upgrades.

TABLE 5-2
AVAILABLE FIRE FLOWS AT 1999 ISO TEST LOCATIONS
PROJECTED 2026 MAXIMUM-DAY DEMANDS

Test No.	Land-Use Description	Test Location	Available Fire Flow (gpm) Year 1999[1]	Estimated Available Fire Flow[2] (gpm)	Required Fire Flow (gpm)[3]	Adequate (Yes/No)
1	Commercial	Central Street at Nashoba Road	950	1,550	2,250	**No**
2	Commercial	Willow Street at Pearl Street	3,000	3,500+	1,250	Yes
3	Commercial	Elm Street at Elm Court	1,200	1,900	1,750	Yes
4	Residential	Robbins Street at Prescott Road	1,700	2,070	750	Yes
5	Commercial	Massachusetts Ave at Charter Road	1,600	2,700	3,000	**No**
6	Commercial	Piper Road at Discovery Way	3,300	2,800	3,000	**No**
7	Commercial	River Street at Chadwick Street	2,300	3,500+	3,500	Yes
8	Commercial	Powder Mill Road at Sudbury Road	2,200	2,220	3,500	**No**
9	Commercial	Craig Road at Russell Road	2,500	3,130	3,000	Yes
10	Commercial	Pope Road at Great Road	2,100	2,630	3,500	**No**
11	Commercial	Concord Road at Wood Lane	1,900	1,840	3,000	**No**
12	Residential	Hammond Street at Evergreen Road	3,700	3,500+	750	Yes
13	Commercial	Great Road at Brook Street	3,400	1,720	3,000	**No**
14	Commercial	Great Road at Harris Street	3,000	1,580	3,000	**No**
15	Commercial	Nagog Park Road at Great Road	2,300	1,440	3,000	**No**
16	Commercial	Main Street at Sawmill Road	3,900	1,640	3,500	**No**
17	Commercial	Main Street at North Street	3,000	1,640	3,500	**No**

[1] Available Flows per reported 1999 ISO Hydrant Test Data does not consider maintaining 20 psi residual system pressure.
[2] Simulated available fire flows based on tank levels 5 feet down from overflow and well supply pumping off, minimum system pressure of 20 psi.
[3] Flows greater than 3,500 gpm are not considered in evaluating system compliance with ISO fire suppression rate schedule.

Figure 5-2 displays the available fire flow (AFF) at each ISO node within the system in addition to pipe diameter. The AFF run was based on the existing system infrastructure utilizing current Projected 2026 Maximum Day Demands. The status of all well supplies is off, and storage tank levels were set to 5 feet below overflow elevation (overflow elevation: 427.5 feet). Wampus Hill Tank pumping station was also turned off. The following sections discuss options that have been considered to resolve the apparent fire flow deficiencies.

<u>Residential Fire Flow</u>

Of the 17 ISO test locations, only two are classified as residential. The first location (ISO 4) is located at the intersection of Robbins Street and Prescott Street, while the second location (ISO 12) is located at the intersection of Hammond Street and Evergreen Road. The results of the hydraulic simulation estimated adequate fire flow available at both locations to meet the 750 gpm requirement.

<u>Commercial Fire Flow</u>

The remaining 15 ISO test locations are all categorized as commercial with required fire flow demands ranging from 1,250 gpm to 3,500 gpm. Of the 15 commercial ISO test locations evaluated, eleven were identified as having inadequate fire flow based on the hydraulic analysis. Twelve were noted to have inadequate fire flow within the last Master Plan. A discussion of piping replacement options to improve fire flows in deficient areas of the system follows:

Central Street at Nashoba Road (ISO #1)

Fire flows at this location were found to be deficient by approximately 700 gpm. This section of Central Street is currently served by an 8-inch water main; however the model results indicated areas of higher head loss on Central Street south of Elm Street where the section is served by a 6-inch main. In order to address this deficiency, a new 10-inch water main was modeled on Central Street from Nashoba Road to Massachusetts Avenue and Windsor Avenue (approximately 6,250 linear feet) to replace the existing 6-inch and 8-inch AC mains on Central Street. This improvement increased available fire flows on Central Street at Nashoba Road above 2,250 gpm.

Notes:
1. Simulated available fire flows based on tank levels 5 feet below overflow well supply pumping off and minimum system pressure at 20 psi.
2. Fire flow simulations run under 2026 projected maximum day demand.

W:\GIS_Development\WaterModel\MA\Acton\MasterPlan Figures\5-2 ISO Map.mxd

Available Fire Flows at ISO Test Locations Acton Water District Acton, Massachusetts

PROJ NO:	13748A	DATE:	Jan 2018	FIGURE: 5-2

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Massachusetts Avenue at Charter Road (ISO #5)

Fire flows at this location were found to be deficient by approximately 300 gpm. This section of Massachusetts Avenue is currently served by an existing 6-inch cast iron pipe. The model indicated higher head loss on this existing section 6-inch pipe restricting flow to the test location. In order to address this deficiency, a new 8-inch water main was modeled on Massachusetts Avenue from Main Street (Route 27) to Charter Road (approximately 600 linear feet) to replace the existing 6-inch main. This improvement increased available fire flow at this location above 3,000 gpm.

This section of Massachusetts Avenue has also been identified by the Town as having an increase in development and other infrastructure is likely to be improved in this area. Within Section 9 of this report, this ISO #5 location is presented in Table 9-1 for recommended ISO improvements (from Charter Road to Main Street). East of Main Street up to Piper Road is also listed as CIP #14 within Table 9-2 as part of the Recommended 10-Year Water Main Improvement Plan.

Piper Road at Discovery Way (ISO #6)

Fire flows at this location were found be deficient by approximately 200 gpm. This location is currently served by an existing 6-inch water main on Piper Road just south of Massachusetts Avenue. The model indicated higher head loss on this existing section of 6-inch pipe. In order to address this deficiency, the a new 8-inch water main was modeled on Piper Road from Massachusetts Avenue to Discovery Way (approximately 200 linear feet) to replace the existing 6-inch AC main. This improvement increased available fire flow above 3,000 gpm.

Powder Mill Road at Sudbury Road (ISO #8)

Fire flows at this location were found to be deficient by approximately 1,280 gpm. Sudbury Road is located at the southeast edge of distribution system bordering the towns of Maynard and Concord. This location is currently served by an existing 8-inch water main. A new 12-inch main on Old High Street to Powder Mill Road was modeled to address the fire flow deficiency in this area. The new 12-inch water main on Old High Street increased fire flow capacity at this location to approximately 2,900 gpm, however this is still below the required flow of 3,500 gpm. As the location is on an extremity of the water system and a large distance from Acton's existing water storage tanks, the improvements necessary to increase capacity to provide the required fire flow would be prohibitively expensive and not cost effective. Although the fire flow simulations were run assuming tank levels five feet below overflow and all well supplies off; a second simulation was run at this location with the new South Acton Water Treatment Plant (WTP) pumping at 750 gpm in addition to the improvements on Old High Street described above. The results of the simulation indicated that with the South Acton WTP operating at a minimum of 750 gpm, available fire flow was increased above the 3,500 gpm requirement at this location.

Pope Road at Great Road (ISO #10)

Fire flows at this location was found to be deficient by approximately 900 gpm. Great Road at Pope Road is currently served by an existing 10-inch AC main. The model indicated high head loss in sections of 6-inch AC pipe on Nagog Hill Road and Concord Road south of Main Street. This bottleneck appears to be restricting larger flows from the Nagog Hill Storage Tank to the eastern section of the distribution system. In order to address this deficiency, two different options were modeled.

Option 1:

A new 12-inch water main was modeled from Main Street along Nagog Hill Road and Concord Road to Pope Road (via Great Road) to replace the existing 6-inch and 10-inch mains (approximately 7,650 linear feet). This improvement increased available fire flow at this location above 3,500 gpm.

Option 2:

Alternatively, by upsizing the entire section of Concord Road from Main Street to Great Road instead of the 6-inch section of Nagog Hill Road, the fire flow requirements at Great Road and Pope Road will also be met. This improvement includes approximately 7,650 linear feet of pipe replacement, similar to Option 1.

Option 3:

A third alternative was modeled which included a new 12-inch main on Great Road between Strawberry Hill Road and Concord Road where currently no water main exists. The model indicated that this improvement would increase available fire flow to approximately 2,900 gpm, however still falling below the ISO requirement of 3,500 gpm.

The improvements described in Option 2 are recommended based on the existing age of the Concord Road piping (1940s) compared to the Nagog Hill Road segment (1960s). In addition, this upgrade will raise the available fire flow at ISO #11 above the required flow rate. This location is described in more detail below.

Concord Road at Wood Lane (ISO #11)

Fire flows at this location were found to be deficient by approximately 1,100 gpm. The model indicated high head loss in the existing 6-inch cast iron main on Concord Road. To address this deficiency, a new section of 12-inch main was modeled between Main Street and Wood Lane to replace the existing 6-inch on Concord Road. This improvement increased the available fire flow at this location above 3,000 gpm. This section of Concord Road is included in the improvements described in Option 2 above.

Great Road north of Brook Street (ISO #13, #14, and #15)

Fire flows on Great Road were found be deficient at Brook Street (1,300 gpm), Harris Street (1,400 gpm) and Nagog Park Road (1,600 gpm). These locations are in the northern section of distribution system along sections of existing 12-inch and 16-inch mains. To address these deficiencies, new 16-inch pipe was modeled on Main Street and Great Road to replace the existing 12-inch mains (7,300 linear feet). By upsizing the existing 12-inch mains on Main Street and Great Road to the Littleton interconnection, a continuous 16-inch transmission is provided from Nagog Hill storage tank. In addition, a new 12-inch water main was modeled on Great Road between Concord Road and Brook Street to improve looping (5,400 linear feet). The existing 10-inch main on Brook Street was also upsized to 12-inch. This scenario assumes and requires that the improvements on Concord Road described above for ISO location #10 are implemented. These improvements increased available fire flow at Great Road and Brook Street

(ISO #13) above 3,000 gpm. The available fire flow at ISO#14 and #15 on Great Road increased by 1,000 gpm, however the required 3,000 gpm at these locations still was not met.

A possible scenario to address the deficiencies at ISO locations #14 and #15 would be to utilize the existing pumping station at the Wampus Hill storage tank during fire flow events which is located off Main Street in the northern section of the distribution. The station is currently equipped with a 900 gpm fire pump to provide additional fire flow in this area. A fire flow simulation was run with this fire pump turned on and the results indicated that available fire flow at ISO #14 and #15 would increase to approximately 2,400 gpm and 2,100 gpm respectively. However, these values are still below the 3,000 gpm ISO requires.

A probable cause of these deficiencies is the lack of nearby gravity storage facilities to the ISO test locations. Wampus Hill storage tank is located in vicinity; however the tank is below the hydraulic grade line (HGL) of the system and therefore is considered pump storage. Under this condition, fire flows are limited by the capacity of the pumps. To increase flow to this area from the Town's gravity storage facilities, one potential option is to install a 12-inch diameter cross-country water main from the Nagog Hill storage tank to the existing 12-inch main on Acorn Park Drive. This connection would provide additional looping to this area and increase fire flow capacity. This option was simulated in the model and the results indicated that the increase in available fire flow would meet the ISO requirements at location #14, but not at location #15. However, by running the same scenario with the Wampus Hill pumping station on, fire flow requirements would be met at both locations.

Main Street north of Wampus Hill Tank (ISO #16 and ISO #17)

Fire flows on Main Street were found to be deficient at Sawmill Road (ISO #16) and North Street (ISO #17) by approximately 1,900 gpm at both locations. This section of Main Street is located in the northeast corner of the distribution system north of Wampus Hill storage tank. These locations are currently served by an existing 16-inch water main. The model indicated high head loss in a stretch of existing 12-inch diameter pipe on Main Street between Nagog Hill Road and the Conant Well No. 1. In order to address these deficiencies, a new 16-inch main was modeled on this section of Main Street to replace the existing 12-inch (4,500 linear feet). This improvement increased available fire flow on Main Street at Sawmill Road and North Street by approximately 1,000 gpm, however the 3,500 gpm flow requirement at these locations was still not met.

Similar to ISO locations #14 and #15, a second simulation was run with the Wampus Hill tank pumping station operating. The results of this simulation indicated an increase in available fire flow of approximately 1,000 gpm, but still below the 3,500 gpm ISO requirement at these locations. However, by incorporating the proposed cross country water main connection described above, fire flow requirements at both locations were able to be met.

The improvements required to provide the required fire flow at ISO locations #14 through #17 are anticipated to be complex projects with high associated construction costs. In addition, many of the proposed hydraulic improvements would require the replacement of newer ductile iron water main that was installed in the 1970s or later. Based on this, the District's input on these potential improvements will be needed in developing recommendations for future versions of the master plan. Due to the large current expenditures and other identified improvements, it may be prudent for the District to focus its efforts on these improvements and then reevaluate these more extreme locations (ISO locations #14 through #17) at a later date when conditions may be different.

Furthermore, the AWD should consider these and the other identified deficiencies as part of its water impact reviews and developer approvals. Opportunities for cost recovery or cost sharing from the developer may be identified based on the size and impact of the proposed development.

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 9 of this report.

5.3.6.1 Water System Pressure

Pressures throughout the system are generally adequate. As is typical of most systems, isolated areas of low pressure exist in the immediate vicinity of storage tanks (Ethan Allen Drive) and in the highest elevations of the system (typically 40 to 45 psi). 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. Under the projected maximum day demand in 2026 pressure will range between 40 to 125 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 2026 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. The majority of the dead-ends consist of small diameter asbestos cement piping. In general, dead-end mains should be targeted for long term replacement and included in the yearly pipe replacement program. The only large diameter water main dead-end identified is approximately 3,200 linear feet of 10-inch ductile iron main on Pope Road from the Wingate Lane to Proctor Street. No opportunities for looping this dead-end are available at this time.

5.3.6.4 Fire Flow

Although the system is adequate in terms of being able to provide the needed residential fire flows, a number of commercial locations of the system are deficient and require upgrade. In total, 17 ISO fire flow test locations were evaluated using the hydraulic water model and 11 were found to be deficient. By implementing the improvements described previously, the available fire flow at seven locations was increased to values at or above the ISO requirement. Available fire flow at the four remaining deficient locations improved as a result of the upgrades, however the ISO flow requirements were still unable to be met. In order to meet the required fire flow at these locations under the analysis performed, large scale improvements to the existing system would be needed.

5.4 WATER MAIN INVENTORY

Approximately 25% of the Acton water system was installed prior to 1950 and is at or nearing its end of useful life. 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 64.4% of the total need for the next twenty years. For Massachusetts alone, this corresponds to an amount of $5.64 billion dollars.

The water works industry is moving towards a practice of maintaining an on-going replacement program where up to 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.

The AWD currently budgets $500,000 per year for these improvements. This would correlate into approximately 2,750 LF of 8-inch water main replaced per year assuming a unit cost of $180 per linear foot. With a current system size of approximately 135 miles, this would account for only 0.39% of the system annually and correlate into fully replacing the system approximately every 260 years. It is acknowledged that as priorities change and funding better understood, the annual replacement program can be re-assessed and modified as necessary.

5.4.1 Method of Analysis

The Acton 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 AWD, existing GIS information, and follow-up interviews with the AWD staff, the following data was compiled and tabulated for all water main segments:

- Diameter;
- Year Installed;
- Material of Construction;
- C-value;
- Static Pressure;
- Water Quality Issues; and
- Break History.

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 15%.

Year Installed - Simplistically, the older the water main the more likely it is reaching the end of its useful life due to a variety of factors such as fatigue, changes in materials and manufacturing techniques, etc. Based on these factors, the corresponding criteria values for year of installation were established as follows:

TABLE 5-4
INSTALLATION YEAR CRITERIA VALUES

Year	Value
Pre 1950	100
1950-1969	80
1970-1979	60
1980-1989	40
1990-1999	20
2000-2009	10
2010-2016	5
2017+	0

Due to the importance of age, a corresponding weighting factor of 25% was selected for year of installation.

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 5-5.

TABLE 5-5
ESTIMATED SERVICE LIFE BY MATERIAL

Material	Service Life (Years)
Asbestos Cement	100
Cast Iron	115
CIPP*	50+
Ductile Iron	110
HDPE	100
PVC	100

*Cured in Place Pipe (CIPP) is a newer technology that is being used to cost effectively rehabilitate water mains at a lower cost than with full replacement. As the liners are relatively new to the water works industry, their design life is expected to be a minimum of 50 years.

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 5-5 is for pipes that were installed in benign 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 Acton system, the following criteria values were utilized for the pipe material:

TABLE 5-6
MATERIALS CRITERIA VALUES

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

A weighting factor of 20% was also 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 5-7
PRESSURE CRITERIA VALUES

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

The weighting factor of 5% was selected for static pressure.

Water Quality Issues - Water quality problems can result from a variety of issues but can be attributed to distribution system related items (e.g., unlined CI water mains, low flows, dead ends, etc.). AWD staff identified customer water quality complaints in various areas of the system which were logged into the piping database. As water quality issues can have immediate impacts to the consumers, the criteria were rated highly for all as follows.

TABLE 5-8
WATER QUALITY CRITERIA VALUES

Water Quality Complaints	Value
Yes	100
No	0

Due to the relative importance of noted water quality issues, a corresponding weighting factor of 15% was selected.

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. In particular, AWD has identified numerous breaks on Main Street, Arlington Street, Stow Street, Oakwood Road, and Hosmer Street (among others). Accordingly, the criteria values for break history were established as follows:

TABLE 5-9
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 25% 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 B**. Figure 5-3 includes pipe condition scores for all water mains in the system.

Water Main Ranking
Acton Water District
Acton, Massachusetts

PROJ NO: 13748A	DATE: Jan 2018	FIGURE: 5-3

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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, the AWD budgets an annual amount of $500,000 for pipe upgrades. This is approximately the amount required to replace approximately 2,750 linear feet (LF) of 8-inch water main per year assuming a unit cost of $180 per linear foot. Over the course of the 10-year capital improvement period, this correlates to approximately 27,500 LF 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 27,500 LF.

After further evaluation of this initial grouping, it was determined that some of these segments scored higher because of poor water quality which was not attributed by the pipe, but rather the water itself. Since additional treatment of the AWD's sources has been progressing, its corresponding value was discounted. Additionally, if there was no break history with these associated segments (which would indicate they are currently structurally sound), then these segments were removed from the priority grouping and replaced with pipe sections having a score above 60 and a history of breaks. The specific water main replacement recommendations and associated costs are included in Section 9 and within the Capital Improvements Plan (CIP) in Section 10.

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. Acton has four 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 Acton system is provided by four storage facilities. The storage facilities are located throughout the system and have a maximum hydraulic gradeline of 427.5 feet, with the exception of Wampus Hill Tank, which is pumped storage. Storage components for these four 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 2026 maximum-day demand is approximately 0.67 MG (0.25 * 2.68).
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 South Acton WTP (Assabet Wells) and Kennedy Wells have backup power and are capable of providing the average daily demand. Therefore, the emergency component is waived.

The calculation for available active storage volume is summarized on Table 5-10 and the storage analyses developed within Table 5-11.

TABLE 5-10
EXISTING AVAILABLE ACTIVE STORAGE VOLUME

Storage Component	Flagg Hill Reservoir	Great Hill Tank	Nagog Hill Reservoir	Wampus Hill Tank*
Total Capacity (MG)	2.0	0.5	3.0	3.0
Diameter (ft)	117	35	99	140
Overflow Elevation (ft)	427.5	427.5	427.5	317
Base Elevation (ft)	402.5	357.5	375	291
Unit Volume (gal/ft)	80,400	7,200	57,550	115,100
Highest User Served (ft)	363	363	363	363
Minimum Tank Elevation to Maintain 20 psi System Pressure (ft)	409.2	409.2	409.2	N/A
Total Active Storage (MG)	**1.47**	**0.13**	**1.05**	**3.0**

*Pumped Storage

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 2026. Table 5-11 presents the storage component analysis.

TABLE 5-11
STORAGE COMPONENT ANALYSIS

	2016	2026
Projected Average-Day Demand (MGD)	1.63	1.79
Projected Maximum-Day Demand (MGD)	2.38	2.68
Peak Hour Storage (25% MDD)	0.59	0.67
Fire Protection Storage	0.63	0.63
Emergency (waived)	N/A	N/A
Total Storage Needed	**1.22**	**1.30**
Available Usable Storage	5.66	5.66
Surplus or (Deficit)	**4.43**	**4.36**

The existing active storage volume in the system is approximately 5.66 MG (2.65 MG without Wampus Hill) and the total required active storage volume for the previously described components is 1.30 MG in year 2026. Based on this analysis, the Acton water system will have adequate storage through the planning period.

Under this analysis scenario, the AWD's four tanks provide adequate redundancy as a surplus storage volume is present if the Wampus Hill Tank (the largest active storage volume) is considered to be off-line. Therefore, no additional storage volume would be recommended.

If more storage is desired, a larger replacement for the existing Great Hill Standpipe would be recommended for the following reasons:

- It is the oldest tank in the system and is approaching an age of 100-years (and possibly its end of life).
- It is the smallest volume tank in the AWD system.
- Because of its small volume and location in the center of the AWD system, its level can fluctuate significantly more than the other tanks (especially during flushing events) and

cause flow reversals (and possibly dirty water complaints) in the system as water in the other tanks drain to fill it back up.

If ultimately desired by the AWD, a tank of at least 2.0 MG in volume would be preliminarily recommended as a replacement for the Great Hill Standpipe.

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. Furthermore, all of the AWD'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, by adding internal tank mixing systems, or both.

Based on previously performed hydraulic simulations with the model (March of 2011) Wright-Pierce assessed the benefits of tank fluctuation and internal tank mixing. Recommendations included the following:

- Allowing the tank levels to fluctuate more than current operations;
- Utilization of the Wampus Hill Tank Booster Pump Station (BPS) more often, and;
- Implementation of mixing at the Flagg Hill Tank to improve turnover and water quality

As presented previously in Section 2 of this report, the AWD has made modifications to the Wampus Hill Tank BPS for improved operational control and installed mixing within the Flagg Hill Tank to help improve turnover and maintenance of good water quality.

Implementation of tank mixing is recommended to be implemented at all of the AWD'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.

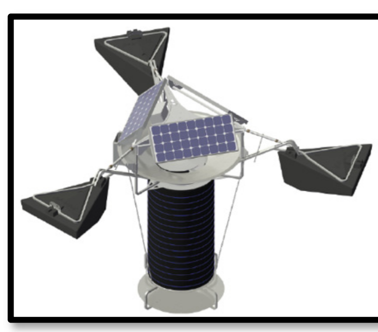
Disadvantages:

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.

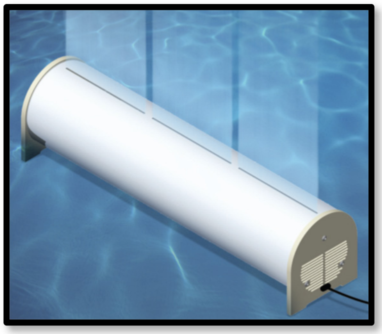
Advantages and disadvantages of this type of active mixing system include the following:

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. This is the type of mixing system that was installed within the Flagg Hill Reservoir in 2012. 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 AWD's water storage tanks do not fluctuate significantly, the use of active mixing systems is recommended for all tanks. As the AWD implements treatment at its sources, it will be more important to maintain the excellent water quality that is produced within its distribution system. Therefore, the remaining three tanks should be individually evaluated for proper sizing, but at a minimum, the AWD may want to consider implementation of the GridBee unit (recently installed at the Flagg Hill Tank) for standardization.

To assess the effectiveness of a tank mixing system, the following minimum water quality sampling program is recommended to be taken from the top, middle, and bottom portions of the tank on a quarterly basis prior to and subsequent to a unit's installation:

- Temperature

- pH
- Chlorine Residual
- TTHMs/HAA5s

Upon sufficient data acquisition (at least one year prior and one year after installation), criteria for a successful installation would include a more uniform temperature, pH, and chlorine residual profile throughout the water column and reduced disinfection byproducts.

5.5.3 Tank Evaluation and Maintenance

As described with Section 2 of this report, the current condition of Acton'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 OPPORTUNITIES FOR INTERCONNECTION WITH ADJACENT WATER SYSTEMS

Interconnections with surrounding communities are valuable from an emergency response perspective. The Town of Acton currently maintains interconnections with the Towns' of Littleton, Concord and Maynard. In general, the connections are closed valves at the Town lines that are opened on an as needed basis. Section 4 of this report presented a basic discussion on the use of the interconnections as potential sources of supplemental supply.

5.7 DISTRIBUTION SYSTEM MAINTENANCE

5.7.1 Unaccounted-for Water Reduction

As discussed in Section 3, non-revenue water in the Acton system was estimated to be an average of approximately 17.0% 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 Acton 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 Acton 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.

Over the last five years, the AWD has completed M36 audits and two years of component analysis.

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 District's annual flushing program, operators must open and close all required main and hydrant valves on a routine. During the calibration of the hydraulic model several valves were found closed. It is critical to log valve status as part of the flushing program. A closed or partially closed valve can drastically reduce the system's hydraulics, as was found during calibration of the hydraulic model. 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 AWD may want to consider the purchase and use of this equipment.

5.7.4 Hydrant Maintenance

The distribution system contains approximately 1,371 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, 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.

The AWD should be commended on its routine practice of a unidirectional flushing program. In general, as the AWD 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

REGULATORY REVIEW

6.1 GENERAL

The Acton Water District (AWD) supplies drinking water to the residents of the Town of Acton from a number of well sources with varied levels of water quality concerns and treatment requirements. 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 Guidelines (ORSG) for manganese.
- A new ORSGL of 0.30 µg/L for 1,4-Dioxane.
- Incorporation of the new federal Revised Total Coliform Rule (RTCR).
- Additional requirements from the federal Reduction of Lead in the Drinking Water Act.

In addition, other possible pending regulations are anticipated in the future including the Radon Rule.

6.2 CLASSIFICATION OF THE ACTON WATER DISTRICT SYSTEM

Water system classification is typically based upon the population served and the character of the water source (e.g., surface water versus groundwater). A water system's classification is used to establish which state and federal rules apply to a water system and the requirements for compliance. AWD is classified as a non-transient community water system because it is a public water system that supplies water to the same population year-round. AWD serves approximately 96% of the residents of Acton (which was reported to have a population of 22,925 in 2015), and is therefore considered to be a large sized community water system (>10,000 people). The AWD system is entirely served with groundwater (i.e., wells).

6.3 OVERVIEW OF DRINKING WATER REGULATIONS

A review of regulatory issues pertaining to large sized community groundwater systems such as AWD has been completed as part of this study. The purpose of this regulatory review is to assist AWD 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 AWD in great detail as they are still in their early stages.

The purpose of the Safe Drinking Water Act (SDWA) of 1974 (amended in 1986 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 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 identified as impacting AWD 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

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. AWD has complied with this regulation.

Tables 6-1 through 6-4 that follow include most recent available water quality data for the AWD's sources that was provided for use in the Master Plan.

TABLE 6-1
WATER QUALITY – SECONDARY CONTAMINANTS, VOC, IOC, AND 1,4-DIOXANE
ACTON, MASSACHUSETTS

	MCL, SMCL or Guideline	Assabet 1A	Assabet 2A	Clapp	Whitcomb	Conant 1	Conant 2	Christofferson	Lawsbrook	Scribner	Kennedy	Marshall
SECONDARY CONTAMINANTS												
Sample Date		5/29/2012	6/26/2015	3/31/2017	6/11/2015	5/14/2013	7/27/2017	5/3/2017	8/9/2017	8/9/2017	6/27/2017	5/2/2016
Iron	0.3 mg/L	0.28	0.053	**2.36**	**2.27**	**0.51**	0.009	**0.448**	0.047	ND	0.222	**0.33**
Manganese	0.05 mg/L	**0.32**	**0.054**	**1.32**	**0.9**	**0.22**	**0.067**	**0.81**	0.033	**0.052**	**0.867**	**0.51**
Alkalinity	NS	27	20	41	35	11	8	24	23	18	34	22
Calcium	NS	19.5	19.9	26.8	31	13.6	10.9	25.1	18.6	17.9	24.1	14.3
Magnesium	NS	4.6	5.3	5.5	6.6	4.5	3	4.6	3.8	3.7	5	3.4
Hardness	NS	68	71	90	105	52	40	82	62	60	81	50
Potassium	NS	3.3	3.7	4.5	5.3	ND	2.4	3.8	3.4	1.9	6.4	2.7
Turbidity	TT	0.2	0.75	0.8	4.5	0.35	0.15	0.8	0.15	0	0.35	0.65
Aluminum	0.05 - 0.2 mg/L	ND	ND	ND	0.05	ND	0.02	ND	0.007	ND	0.189	0.1
Chloride	250 mg/L	80.7	90.6	112	**263**	104	70.5	136	85.6	66.1	141	73.3
Color	15 CU	0	0	8	**30**	0	0	6	0	0	**40**	**18**
Copper	1 mg/L	ND	ND	0.006	0.01	0.12	0.113	ND	ND	0.028	0.172	0.033
Odor	3 TON	0	0	0	0	1	0	0	0	0	0	0
pH	6.5 - 8.5	**6.1**	**5.7**	**6.3**	**5.6**	**5.6**	**5.8**	**6.4**	**6.2**	**6.3**	**6**	**5.7**
Silver	0.1 mg/L	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND
Sulfate	250 mg/L	20.4	23.6	7.3	12.4	15.2	8.2	16.9	15	13.9	15	11
TDS	500 mg/L	218	262	280	**550**	262	168	316	230	178	324	190
Zinc	5 mg/L	0.071	0.018	0.012	0.006	0.033	0.008	ND	0.013	ND	ND	0.064

NS = No Standard

BOLD = Exceeds Standard

CU = Color Units

TON = Threshold Odor Number

ND = None Detected

TABLE 6-1 CONTINUED
WATER QUALITY – SECONDARY CONTAMINANTS, VOC, IOC, AND 1,4-DIOXANE
ACTON, MASSACHUSETTS

	MCL, SMCL or Guideline	Assabet 1A	Assabet 2A	Clapp	Whitcomb	Conant 1	Conant 2	Christofferson	Lawsbrook	Scribner	Kennedy	Marshall
VOLATILE ORGANIC COMPOUNDS (VOCs)												
Sample Date		9/7/2016	8/9/2017	7/12/2017	7/12/2017	7/25/2016	7/27/2017	8/9/2017	8/9/2017	8/9/2017	8/8/2016	9/22/2016
Total	Varies	-	ND	ND	ND	ND	-	ND	-	-	-	ND
1,1-Dichloroethylene	7 ug/L	-	-	-	-	-	-	-	0.9	1.9	-	-
MTBE	20 - 40 ug/L	0.62	-	-	-	-	-	-	-	-	-	-
INORGANIC COMPOUNDS (IOCs)												
Sample Date		5/29/2012	5/29/2012	8/14/2017	8/14/2017	6/28/2016	7/27/2017	5/3/2017	5/3/2017	5/3/2017	6/27/2017	6/2/2014
Antimony		-	-	-	-	-	-	-	-	-	-	-
Arsenic	0.010 mg/L	-	-	**0.012**	**0.018**	-	-	-	-	-	-	-
Barium	2 mg/L	-	-	-	-	-	-	-	-	-	-	-
Beryllium	0.004 mg/L	-	-	-	-	-	-	-	-	-	-	-
Cadmium	0.005 mg/L	-	-	-	-	-	-	-	-	-	-	-
Chromium	0.1 mg/L	-	-	-	-	-	-	-	-	-	-	-
Cyanide	0.2 mg/L	-	-	-	-	-	-	-	-	-	-	-
Fluoride	4.0 mg/L	-	-	-	-	-	-	-	-	-	-	-
Mercury	0.002 mg/L	-	-	-	-	-	-	-	-	-	-	-
Nickel	0.1 mg/L	-	-	-	-	-	-	-	-	-	-	-
Selenium	0.05 mg/L	-	-	-	-	-	-	-	-	-	-	-
Sodium	20 mg/L	**68.5**	**44.7**	-	-	**74.3**	**36.7**	**65.8**	**47.3**	**37.4**	**77.6**	**23**
Thallium	0.002 mg/L	-	-	-	-	-	-	-	-	-	-	-
1,4-DIOXANE												
Sample Date		5/3/2017	5/3/2017	5/3/2017	5/3/2017	5/3/2017	5/3/2017	5/3/2017	5/3/2017	5/3/2017	5/3/2017	5/3/2017
1,4-Dioxane	0.3 ug/L	**0.336**	0.218	-	-	-	-	0.096	0.16	0.169	-	-

BOLD = Exceeds Standard
ND = None Detected
- = No Data

TABLE 6-2
WATER QUALITY - NITROGEN
ACTON, MASSACHUSETTS

	MCL, SMCL or Guideline	Assabet WTP	Clapp	Whitcomb	Conant 1	Conant 2	Christofferson	Lawsbrook	Scribner	NAWTP
NITROGEN										
Sample Date		6/2/2014	6/19/2017	6/19/2017	6/28/2016	7/27/2017	5/3/2017	5/3/2017	5/3/2017	6/27/2017
Nitrate	10 mg/L	1.2	0.34	0.18	ND	0.24	1.6	4.2	1.5	0.18
Nitrite	1 mg/L	-	-	-	-	-	-	-	-	-

ND = None Detected

TABLE 6-3
WATER QUALITY - TTHM
ACTON, MASSACHUSETTS

Sample Location	Q1 2/15/2017	Q2 5/17/2017	Q3 8/16/2017	Q4 11/17/2017	LRAA
DISINFECTION BY-PRODUCTS (DBP) - TTHM (ppb)					
DBP1: 11 Breezy Point Way	46.9	90.9	29	28.5	49
DBP2: 5 Highland Street	12.6	18.9	28	21.7	21
DBP3: 210 Main Street	11.4	18.8	24	30.1	21
DBP4: 486 Main Street	24.7	28.4	38	29.0	30

TTHM = Total Trihalomethanes

TABLE 6-4
WATER QUALITY - HAA5
ACTON, MASSACHUSETTS

Sample Location	Q1 2/15/2017	Q2 5/17/2017	Q3 8/16/2017	Q4 11/17/2017	LRAA
DISINFECTION BY-PRODUCTS (DBP) - HAA5 (ppb)					
DBP1: 11 Breezy Point Way	6.3	11.5	2.7	1.3	5.6
DBP2: 5 Highland Street	0	0	0	2.7	0.68
DBP3: 210 Main Street	2.1	3.8	4.5	3.6	3.5
DBP4: 486 Main Street	2.9	2.9	5.1	3.9	3.7

HAA5 = Haloacetic Acids

6.3.1 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**.

6.3.1.1 Arsenic

Under the National Primary Drinking Water Regulations, Arsenic has a MCL of 0.010 mg/L. Over the past several years, this contaminant has been detected in the raw water of the AWD's Clapp and Whitcomb Wells. As shown in Table 6-1 under the Inorganic Compounds, the Arsenic concentrations from the Clapp and Whitcomb Wells were recently measured to be 0.012 and 0.018 mg/L, respectively, on August 14, 2017.

The raw water from the Clapp and Whitcomb Wells is treated at the Clapp/Whitcomb WTP which includes aeration for the removal of VOCs and pH adjustment followed by treatment with Granular Activated Carbon (GAC) for color and organics removal. It is understood that the GAC treatment is removing some of the Arsenic from the raw water since the finished water concentrations are being reduces to approximately 0.003 mg/L (this is the average concentration determined from available data since 2012). At this time, the AWD can be considered to be in compliance with the Arsenic MCL.

6.3.2 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**.

6.3.3 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 Spring 2017 Standards and Guidelines for Contaminants in Massachusetts Drinking Waters 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. The contaminants of particular concern to the AWD include manganese, 1,4-Dioxane, MTBE and Nitrate/Nitrite as these have been detected in the AWD well sources noted within the previous tables.

In general, Nitrate and MTBE have been detected in some of the AWD well sources at levels below the regulated limit. Possible solutions could include dilution with other sources or treatment for its removal if mandated by MassDEP. Aeration at the WTPs sized for MTBE (and VOC) reduction can also be utilized for treatment. Should the concentration of these compounds increase and exceed their corresponding limits, new treatment processes may need to be considered.

Under the Massachusetts Drinking Water Standards, 1,4-Dioxane has an ORSGL of 0.0003 mg/L (or 0.30 µg/L). As presented within Section 4, this contaminant has been detected in the AWD's School Street and Assabet well sources. Within the last few years, there have also been exceedances at the Assabet Wells, the Scribner Wells, and the Lawsbook Well.

Of particular note 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.

6.3.3.1 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 manganese concentrations.

MassDEP's Guidelines for Public Water Systems state that if the manganese concentration in the raw water exceeds 0.30 mg/L then removal is required. If the manganese 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 manganese and MassDEP's recent ORS guideline for manganese closely follows the EPA's Health Advisory for manganese. 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, manganese has been causing water quality problems and chronic consumer complaints in AWD. Manganese concentrations have been exceeding its corresponding SMCL of 0.05 mg/L as shown previously in Table 6-1. Additional information regarding Acton's historical manganese concentrations since 1998 can be found in Section 4.

6.3.3.2 1,4-Dioxane

1,4-Dioxane is an "emerging contaminant" that has been found in groundwater at sites throughout the United States. It is a likely human carcinogen, and it may also cause kidney and liver damage with long-term exposure. The physical and chemical properties and behavior of 1,4-Dioxane create challenges for its treatment. It is highly soluble in water, and is not readily biodegradable. EPA has determined that the drinking water concentration representing a 1×10^{-6} cancer risk level for 1,4-Dioxane is 0.35 μg/L.

No federal maximum contaminant level (MCL) has yet been established for 1,4-Doxane in drinking water, although states have health-based drinking water guidance values. 1,4-Dioxane was included on the third drinking water contaminant candidate list (CCL), which is a list of unregulated contaminants that are known to, or anticipated to, occur in public water systems, and may require regulation under the Safe Drinking Water Act. Out of all the Public Water Supplies in New England that were tested under the Unregulated Contaminant Monitoring Rule 3 (UCMR-

3), the state of Massachusetts had the most detections of 1,4-Dioxane (76%). Several state governments have set their own varying advisory levels for 1,4-Dioxane as presented in Table 6-5.

TABLE 6-5
1,4-DIOXANE GUIDELINES
ACTON, MASSACHUSETTS

Regulatory Guidelines for 1,4-Dioxane in Drinking Water for Selected States		
State	**Guideline**	**Concentration (µg/L)**
California	Notification Level	1.0
Colorado	Drinking Water Standard	3.2
Connecticut	Action Level	3.0
Maine	Maximum Exposure Guideline	4
Massachusetts	Guideline	0.3
New Hampshire	Proposed Remediation Value	3.0
New York	Drinking Water Standard	50

1,4-Dioxane is difficult to treat because it is very soluble in water, and it has low volatility so that aeration cannot effectively remove it. It has been shown that advanced oxidation process (AOP) treatment can be very effective for treating 1,4-Dioxane. AOP is a process that uses various combinations of ozone, ultraviolet light, and/or hydrogen peroxide to create highly reactive hydroxyl radicals (OH-) that break down the 1,4-Dioxane. Depending on the AOP implemented, other treatment steps may be required. For example, GAC contactors would be needed downstream of the AOP equipment to quench any remaining hydrogen peroxide that may not have been consumed by the reaction. Additionally, the AOP equipment may need to be installed downstream of a filtration step that would remove any competing or interfering constituents (like iron or manganese).

The School Street (Christofferson, Lawsbrook, and Scribner) and Assabet sources have had concentrations that exceeded the 1,4-Dioxane ORSGL of 0.30 µg/L. However, it is understood that the MassDEP may not require any treatment implementation until the concentration reaches 2.6 µg/L (none of the sources have reached this concentration). As discussed earlier in this report, provisions for this enhanced treatment is available at the South Acton WTP should treatment for

1,4-Dioxane ever be determined to be required. AWD's historical 1,4-Dioxane data can be found in Section 4 of this report.

The 1,4-Dioxane contamination can be primarily attributed to W. R. Grace and Nuclear Metals, Inc. (NMI). The W. R. Grace facility produced material used to make concrete and organic chemicals, container sealing compounds, latex products, and paper and plastic battery separators. The property was first acquired in 1954 and until their manufacturing ceased in 1991, their wastewater and solid industrial wastes from these operations were disposed of in unlined lagoons. The industrial solvent, 1,4-Dioxane, has migrated beyond the W. R. Grace site and is being detected in Acton's groundwater wells.

The Nuclear Metals, Inc. site is located at 2229 Main Street in Concord, MA (not far from the W. R. Grace site). According to EPA, NMI focused in large-scale production of depleted uranium (DU) armor penetrators, other DU products, and beryllium alloy parts. Manufacturing operations resulted in significant contamination to the soil, sediment, and groundwater at the 46-acre property. NMI first acquired the site in 1972, was renamed Starmet Corporation in 1997, and was then triggered to investigation by the EPA and MassDEP in 2001. Since then, the site was undergoing a multi-phase clean-up. The groundwater was contaminated with VOCs and 1,4-Dioxane. On July 7, 2016, EPA issued an Administrative Settlement Agreement and Order of Consent which includes requirements to install groundwater pumping and a treatment system to cut off the 1,4-Dioxane and VOC contamination before it reaches the Assabet Well 1A. In 2016, an extraction well was installed on AWD property with additional monitoring wells.

Three articles written by EPA regarding W. R. Grace and NMI along with a site plan are provided in **Appendix G**.

EPA also has an informative Technical Fact Sheet on 1,4-Dioxane (recently updated in November 2017) that is attached in **Appendix H**.

6.3.4 Acton Water District VOC Regulation

The AWD has adopted its own more protective action levels for VOCs in its water supply to provide the highest quality drinking water to its consumers. In summary, an action level of 1 part per billion (ppb) has been set for all regulated VOCs that have MCLs specified by the MassDEP or EPA. Upon detection of a regulated VOC at or above 1 ppb, the District will perform the follow up procedures listed within its regulation. A copy of the regulation and the corresponding list of applicable VOCs are included within **Appendix I**.

6.3.5 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 and beginning on December 1, 2009, all Massachusetts public water systems (PWS) using ground water must comply with the new federal GWR. 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. <u>Periodic sanitary surveys</u> 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). Sanitary survey requirements for all ground water systems is described in 310 CMR 22.26(2).

2. <u>Source water monitoring</u> 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, such as AWD, 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. AWD uses a free chlorine residual (through the addition of sodium hypochlorite) for disinfection. AWD is compliant with 4-log inactivation of viruses at its two WTPs.

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.

6.3.6 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 AWD's (approximately 6,662 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, AWD complies with all provisions of the RTCR.

6.3.7 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 [additional information can be found in MassDEP Regulation 310 CMR 22.04(8)]. 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, AWD is on a reduced monitoring schedule and complies with all the provisions of the Lead and Copper Rule.

6.3.8 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 AWD system serves greater than 10,000 people and is therefore a "Schedule 3" 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.

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

The requirements for the IDSE were met using a combination of the 40/30 Certification and distribution system modeling. The monitoring locations identified through this process are:

- 11 Breezy Point Road
- 5 Highland Road
- 486 Main Street
- 210 Main Street

AWD is currently in compliance with this regulation.

6.3.9 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 AWD 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.

Radon in the AWD sources have been measured at levels less than 4,000 pCi/L. In addition, the treated water at all sources was found to be below the anticipated MCL requirement of 300 pCi/L. AWD Policy is to provide aeration to address potential VOCs, which should be sufficient to address radon as well.

6.3.10 Surface Water Treatment Regulations

The AWD system is supplied entirely by groundwater. However, the Kennedy and Marshall Wells and the Christofferson Well were previously classified as groundwater under the influence (GWUI) of surface water. 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 AWD was to comply with the *Cryptosporidium* treatment requirements by October 1, 2012.
- Residual Disinfectants:
  - Disinfectant residual ≥ 0.20 mg/L at entrance to distribution system.
  - Detectable disinfectant residual in the distribution system.
- Turbidity Performance:
  - Combined filter effluent turbidity ≤ 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.

Treatment that meets these regulations is currently in place at the North Acton Water Treatment Plant (WTP) for the Kennedy and Marshall Wells and also at the South Acton WTP for the Christofferson Well. The South Acton WTP was recently constructed in 2015 and currently also treats the Assabet Wells, Lawsbrook Well, and Scribner Wellfield.

SECTION 7

DEMAND MANAGEMENT

7.1 GENERAL

The implementation of new water supply sources has historically been difficult. Recognizing this, the Acton Water District (AWD) has been very diligent in optimizing the use of water from its existing sources via a proactive demand management process.

Additionally, the process continues to evolve and become increasingly more difficult. In November of 2012 the Executive Office of Energy and Environmental Affairs (EOEA) released its final Sustainable Water Management Framework. This framework is a result of the Sustainable Water Management Initiative (SWMI) process and is now used by the Massachusetts Department of Environmental Protection (MassDEP) as a guide to incorporate changes into the Water Management Act (WMA) regulations. This was further discussed within Section 3.

This section presents a brief overview of the AWD's efforts in demand management.

7.2 SEASONAL DEMAND ANALYSIS

Comparing the average seasonal water usage between the lower water use periods (i.e., the winter months) with the higher water use periods (i.e., the summer months) provides a good understanding of the seasonal demand variations. Table 7-1 presents the AWD's calculated average daily usage for the winter (December through February) and summer (June through August) periods on an annual basis since 1995. The difference between the two periods as well as the corresponding summer to winter ratio is also presented.

TABLE 7-1
AVERAGE SEASONAL WATER USAGE
ACTON, MASSACHUSETTS

Year	Average Daily Usage (MGD)			
	Winter	Summer	Difference	Ratio
1995	1.249	1.823	0.574	1.460
1996	1.413	2.046	0.633	1.448
1997	1.669	2.315	0.646	1.387
1998	1.694	2.076	0.382	1.226
1999	1.743	2.077	0.334	1.192
2000	1.64	2.082	0.442	1.270
2001	1.923	2.385	0.462	1.240
2002	1.625	2.057	0.432	1.266
2003	1.433	1.777	0.344	1.240
2004	1.437	1.879	0.442	1.308
2005	1.483	1.958	0.475	1.320
2006	1.389	1.837	0.448	1.323
2007	1.389	2.142	0.753	1.542
2008	1.445	1.73	0.285	1.197
2009	1.405	1.811	0.406	1.289
2010	1.421	2.056	0.635	1.447
2011	1.382	2.021	0.639	1.463
2012	1.378	2.018	0.640	1.464
2013	1.328	1.878	0.550	1.414
2014	1.409	2.065	0.656	1.466
2015	1.372	1.887	0.515	1.375
2016	1.278	2.168	0.890	1.696

Since 1995, the seasonal demand difference has ranged from a low of 0.285 million gallons per day (MGD) in 2008 to a high of 0.890 MGD in 2016. For that same period, the summer to winter ratio ranged from a low of 1.192 in 1999 to a high of 1.696 in 2016. Seasonal fluctuations are generally expected from year to year, but in particular, the average daily demand for the summer period is expected to fluctuate the most since it is dependent on precipitation.

Table 7-2 presents precipitation data for the same period that was obtained from the Massachusetts Department of Conservation and Recreation (DCR) Rainfall Database. The data for the Town of Concord, MA rainfall gauging station (closest location) was used.

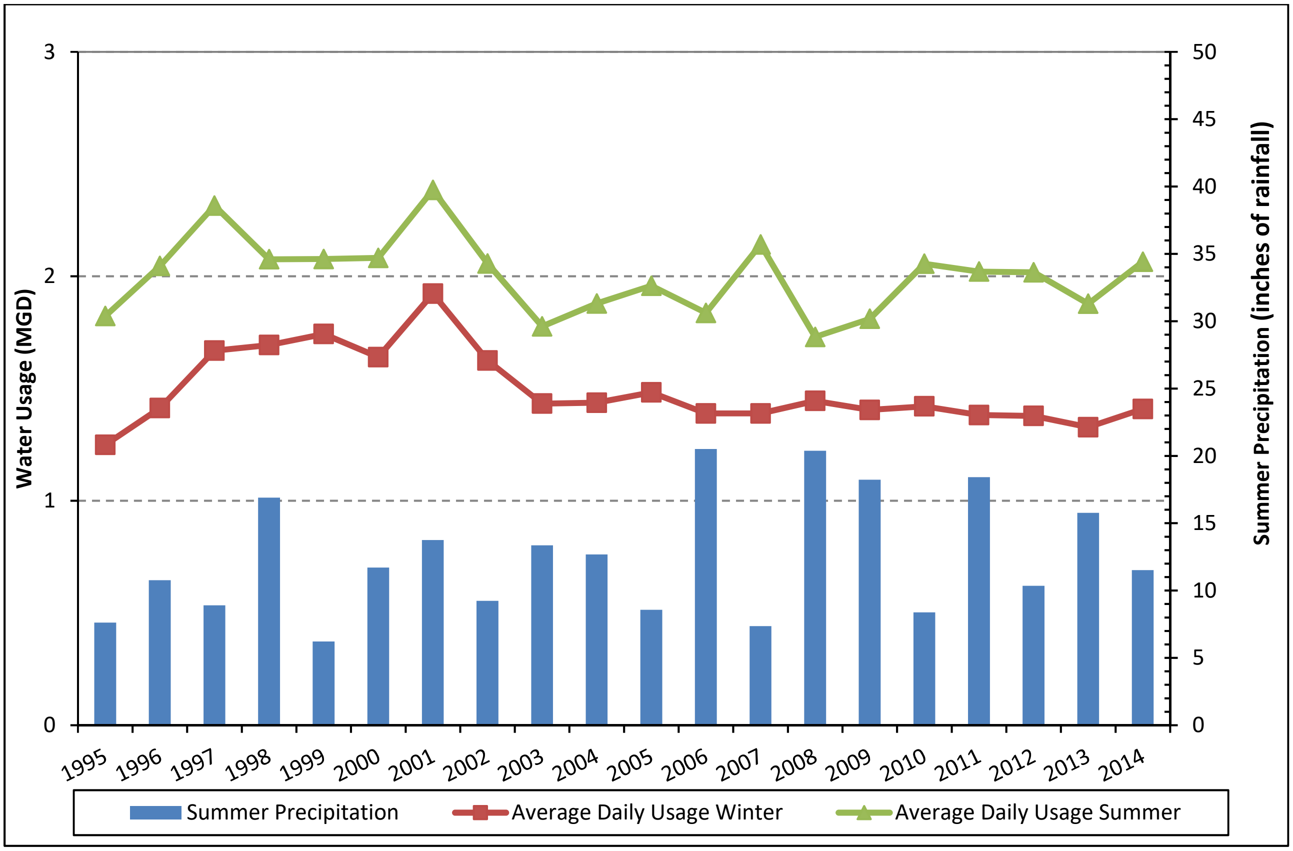
TABLE 7-2
PRECIPITATION DATA FROM CONCORD STATION
ACTON, MASSACHUSETTS

	January	February	March	April	May	June	July	August	September	October	November	December	Total Precipitation	Summer Precipitation
1995	3.52	2.92	1.96	2.05	2.71	2.20	4.17	1.24	3.23	6.57	5.15	2.42	**38.14**	**7.61**
1996	5.44	2.35	3.81	6.04	2.68	2.20	7.59	0.97	7.89	8.93	2.76	5.95	**56.61**	**10.76**
1997	2.51	1.72	5.39	3.59	2.23	1.15	2.63	5.12	1.66	2.06	5.87	2.62	**36.55**	**8.90**
1998	5.57	4.18	4.43	2.96	6.62	11.17	3.82	1.91	1.68	5.62	1.81	1.45	**51.22**	**16.90**
1999	5.26	3.66	3.14	0.67	2.90	0.81	2.72	2.68	10.06	4.01	2.43	2.12	**40.46**	**6.21**
2000	2.89	2.22	3.41	6.18	3.76	5.28	3.94	2.48	2.62	3.20	4.43	3.01	**43.42**	**11.70**
2001	1.54	2.21	8.00	1.09	2.53	6.41	3.53	3.81	1.98	0.96	0.74	2.77	**35.57**	**13.75**
2002	2.51	1.48	4.28	2.43	5.52	4.96	2.15	2.12	3.43	3.95	5.05	6.44	**44.32**	**9.23**
2003	1.82	3.55	4.11	3.32	4.45	5.80	2.21	5.34	3.34	5.72	2.01	4.59	**46.26**	**13.35**
2004	0.92	1.12	3.93	6.87	3.36	1.34	5.22	6.12	6.95	2.82	3.05	3.55	**45.25**	**12.68**
2005	3.31	2.89	2.72	4.70	4.90	1.75	2.86	3.96	1.24	13.43	4.45	2.05	**48.26**	**8.57**
2006	2.52	2.15	0.00	2.85	10.37	10.74	4.51	5.26	2.21	5.64	6.23	2.54	**55.02**	**20.51**
2007	2.25	0.65	3.91	6.65	4.62	3.30	3.34	0.72	2.43	3.02	3.68	4.00	**38.57**	**7.36**
2008	2.50	8.76	5.43	4.41	2.22	4.39	9.73	6.25	9.51	3.26	5.14	5.20	**66.80**	**20.37**
2009	3.68	1.50	2.90	4.04	2.60	5.21	9.14	3.87	2.32	5.11	4.62	4.85	**49.84**	**18.22**
2010	3.39	5.37	16.06	2.94	3.55	2.22	2.25	3.90	1.99	5.61	4.07	3.48	**54.83**	**8.37**
2011	4.06	4.79	3.01	3.77	3.59	6.48	2.03	9.90	5.44	6.89	3.90	4.04	**57.90**	**18.41**
2012	3.11	1.48	1.65	3.50	3.91	3.77	1.79	4.79	3.27	6.85	0.82	5.54	**40.48**	**10.35**
2013	1.56	3.97	2.74	1.62	4.27	9.82	3.83	2.12	3.12	0.95	2.77	NA	**36.77**	**15.77**
2014	2.58	4.21	5.24	4.60	2.36	2.81	4.74	3.96	1.45	6.06	NA	NA	**38.01**	**11.51**

NA: Not Available

Figure 7-1 presents the calculated seasonal demands along with the corresponding summer precipitation amounts for the same period.

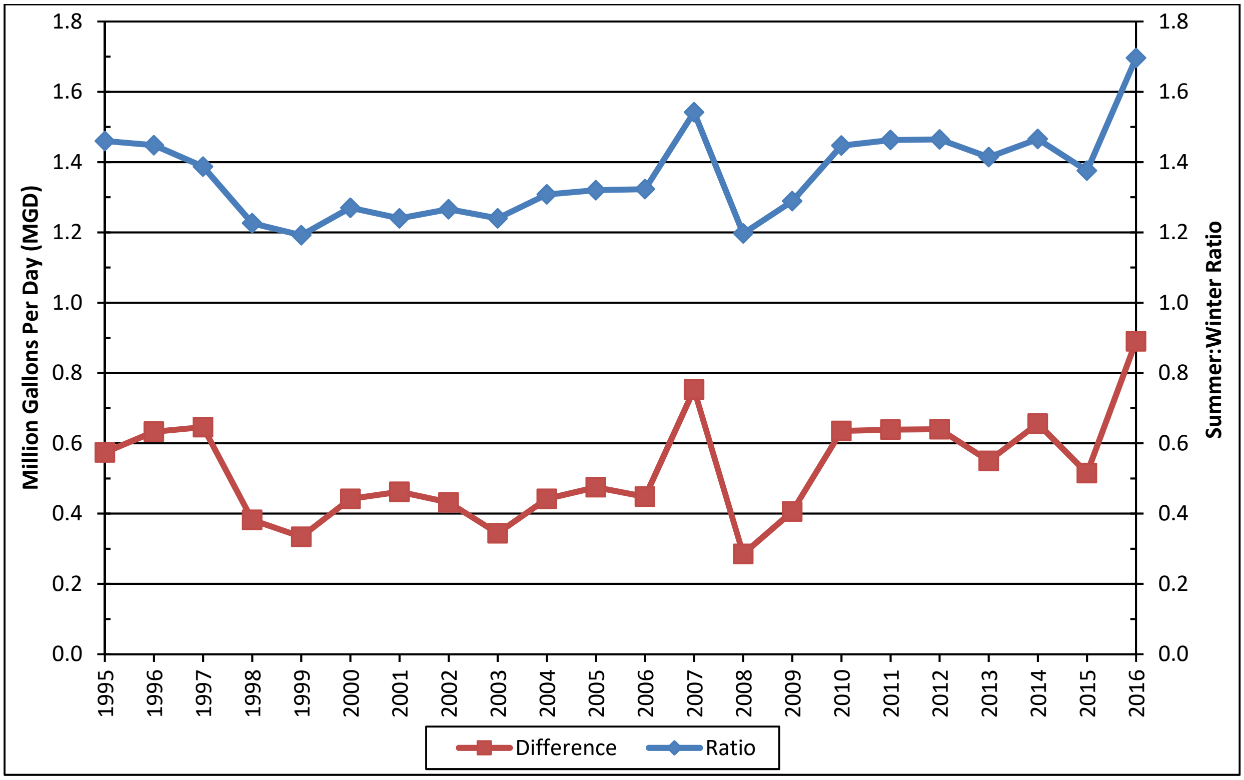
FIGURE 7-1
AVERAGE DAILY WINTER AND SUMMER USAGE
ACTON, MASSACHUSETTS



As expected, the summer demand is consistently higher than that of the winter and it typically increases when the summer precipitation decreases. It is noted that the winter demand has consistently decreased from its high in 2001and has remained relatively stable since 2003. Prior to that period the winter demand trend tended to more closely follow that of the summer.

To help identify any other trends for the seasonal usage, the seasonal differences and ratios are presented on Figure 7-2.

FIGURE 7-2
WINTER AND SUMMER USAGE DIFFERENCE AND RATIO
ACTON, MASSACHUSETTS



From this data, there appears to be an increasing trend in the seasonal difference as well as the summer to winter ratio that started around 1999. As the average winter daily water usage has remained relatively stable (with a slight increase) over the past several years, this trend could possibly be a result of increased development and use in irrigation systems.

7.3 EXISTING INITIATIVES

The AWD has implemented various rules, regulations and programs to help manage the increased seasonal demand of the summer months. Some of these include the following:

- ***Outdoor Water Use Restrictions***: The AWD has a mandatory water use restriction program that is effective every year from May 1st through October 1st. In general odd numbered houses are allowed to use water outdoors on Wednesdays, Fridays, and Sundays while even numbered houses are allowed to use water outdoors on Tuesdays, Thursdays, and Saturdays. No lawn watering is allowed from 7 AM to 7 PM. There is no outdoor water use allowed on Mondays (including new lawns).

- ***Permanent Outside Irrigation System Requirements***: The AWD has established minimum requirements for all irrigation systems that are connected to its public water supply. Requirements for timing devices (programmable in conformance with the AWD's Outdoor Water Use Restrictions), moisture sensing devices, and approved backflow preventers are included.

- ***Water Rate Structure***: The AWD utilizes an increasing rate block structure that charges higher rates for increased water use. Additionally, the AWD also has a higher summer rate schedule (for the July & September billings) that is higher than its winter rate schedule (for the December and March billings).

- ***Water Impact Report Requirements***: The AWD has also incorporated the requirement that a Water Impact Report be prepared and submitted to the AWD for approval by any person applying for water service that has a design demand greater than 2,500 gallons per day or requires a service line over 2-inches in diameter. Among other items, the report needs to present the proposed development's impact to the AWD's existing supply and distribution system while identifying the water conservation techniques to be incorporated that will mitigate the project's impact.

- ***Mitigation Fee***: The AWD charges a mitigation fee for any project that has a proposed increase in water use from its current use in excess of 200 gallons per day, for projects connecting to the District that were previously connected to another Public Water System, or projects that extend the existing distribution system. AWD may waive mitigation fees after review of the Water Impact Report.

Additional information on these (and other items) is incorporated within the AWD's "*Rules, Regulations and Rates*". A copy of which is included within **Appendix I**.

7.4 POTENTIAL INITIATIVES

Another important part of demand management is to adequately account for as much of the water produced as possible so that an accurate understanding can be made. For this the accuracy of the AWD's master meters (e.g., raw and/or treated water meters) and revenue meters (service meters used for billing) is very important. An imbalance on either side would lead to inaccuracy, skewed demand assessments, and undesired results. For example, having master meters that over-register (i.e., they indicate more water is pumped than actual) would lead to a higher than actual unaccounted for water (UAW) percentage. On the other extreme, having revenue meters under-register (indicate less water was consumed than actual) would lead to lost revenues as well as add to the UAW.

The AWD has standardized on the Badger brand of metering technology for its master and revenue meters. The AWD is currently in the process of upgrading all of its meters to be on the Orion communications protocol and approximately 26 meters remain. The meter reading system is of the automatic meter reading (AMR) type. Revenue meters are now read monthly for both commercial and residential accounts via the use of a drive-by system (a vehicle outfitted with the appropriate mobile reading system equipment).

It is understood that the AWD has their master meters tested and calibrated on an annual basis. Therefore, the measurement accuracy for the distribution system inputs can be considered acceptable.

In regards to revenue meters, these are typically broken down into larger (commercial type) and smaller (residential type) meters.

Over time, due to changes in ownership, growth, or changes in use the larger commercial meters are often a source of inaccuracy if they are not the correct size for the utilized flow rates. The term "right sizing" is often used to describe a process of assuring that the largest meters (and often the largest revenue producers) are sized appropriately to capture the maximum amount of revenue for the water supplier. As it is understood that the AWD does not have many large or commercial meters, it is unlikely that a "right-sizing" meter program would provide measurable benefits at this time.

The American Water Works Association (AWWA) recommends that 1-inch and smaller meters be tested at least every 10 years. Under the current water withdrawal permit, the MassDEP requires the AWD to comply with the following for metering:

- Continue to meter 100% of the water system with all meters of proper size and calibration to measure flow to within 5%, including public buildings and facilities.
- Continue the AWD's ongoing program to inspect individual service meters and that such program include sufficient funds in the annual budget to recalibrate, repair, or replace meters as needed.
- Calibrate the AWD's master meters annually.

As discussed later in Section 8, the AWD has partnered with WaterSmart software. Combined with the monthly meter reading, this provides the District with the ability to engage their customers on water use and notify them more quickly of any possible detected leaks and/or high usage alerts.

SECTION 8

ASSET MANAGEMENT

8.1 ASSET MANAGEMENT

Asset management can be simply described as the practice of managing infrastructure capital assets to minimize the total cost of owning and operating them, while delivering the service level customers desire.

Wright-Pierce previously facilitated a workshop with Acton Water District (AWD) management and technical support staff to review current Asset Management (AM). The purpose of the workshop was to discuss existing AM procedures at the AWD and to identify areas for improvement. A brief description of existing procedures and areas for possible enhancement are discussed as follows.

8.2 EXISTING PROCEDURES

First and foremost, the AWD should be commended for its efforts as it is far ahead of many utilities of its size in New England with regard to Geographic Information System (GIS), hydraulic modeling, and transition of paper records into digital formats. The AWD has developed and maintains a custom built database application to reference and log data pertaining to the water distribution system, operations and warrants. This is a unique and powerful system that the AWD staff can utilize for various search and lookup methods to bring up scanned paper records and database information. The AWD has scanned much of its historical paper records for service and system valve tie-cards. These scans are hyperlinked within the custom built database application "Water Operations Warrants". This database application is installed on Acer Netbook computers and deployed into the field with the operations crew.

The AWD has a quality mapping grade Global Positioning System (GPS) unit manufactured by Trimble and has been in the process of mapping nearly all its valves, hydrants and key water distribution infrastructure. As time permits, the AWD continues to collect data in the field including the newer water main and appurtenance infrastructure as part of the District's recent upgrades. Utilizing this GPS data the AWD is working toward developing a more complete GIS system and has dedicated staff to maintain and update this system. Data is being organized to allow various queries and searches of the water mains, valve, hydrants and service records for screening and sorting data on the system based on asset age, manufacturer, and other specific descriptive information.

The AWD's hydraulic model is currently maintained in WaterGEMS V10 which is a GIS (ESRI) based software. The pipes and associated data developed from the hydraulic model initially served as the pipes for the GIS. Generally, the detail depicted in the GIS is more accurate (and thus more detailed) than what is represented (or required) in the hydraulic model. There are various approaches to maintain synchronization between GIS and the hydraulic model if the AWD desires to develop a higher level of detail into the GIS pipes.

The AWD retains other water system data within its account information and billing software that it has developed on its own. The AWD collects water-use data from its residential and commercial meters monthly using an automatic meter read (AMR) system that collects real-time water use data with radio telemetry using a drive-by (vehicle mounted) type system. All of the meters in the AWD system are AMR compatible. At this time, the AWD has approximately 26 meters remaining to be replaced as part of its meter replacement program to convert the meter communication protocol to Orion (as the previous Trace system is no longer supported by Badger). This water use data is currently used to generate a quarterly water bill. Customers currently have the option to receive and pay their bill traditionally (paper via the mail) as well as paperless (via electronic billing and payment).

Since the last Master Plan Update, the AWD has partnered with WaterSmart Software to provide its customers with another tool for use as part of their water efficiency and outreach tool box. WaterSmart is a data driven customer engagement and analytics platform, that offers greater insight and control over water use in the customer's home or business. Some of the stated benefits from this program include:

- Improved water use information that is updated monthly with customized usage summary.

- Access to an interactive water-saving recommendation library that is customized specifically for that property.
- Improved customer awareness of their water usage.
- Timely leak and high-use alerts.
- Multiple ways to send and receive communications on water use and water supply issues (e.g., email and text)
- E-Bill Presentment and historical payment and use information.

8.3 AREAS FOR ASSET MANAGEMENT ENHANCEMENT

Traditional AM programs incorporate level of service (LOS) planning, criticality analysis, and life-cycle cost analysis, to build a capital improvement plan that is sustainable and affordable. The criticality component includes a risk analysis of the individual assets to determine their likelihood of failure (LOF) and consequence of failure (COF). Customized software programs such as Viewworks™, are true AM programs that can integrate these objectives.

This master plan has used an informal LOS and matrix analysis which considers criticality of assets to develop weighting factors to prioritize needs in the capital improvement plan (CIP). This current approach will deliver a comprehensive, prioritized CIP, although AWD may wish to expand this methodology in the future.

From the initial results of our previous AM workshop with the AWD, two areas for improvement were identified for the AWD consideration:

- Building out the GIS with links to the custom database application and record documents.
- Implementation of a comprehensive Computerized Maintenance Management System (CMMS)

The AWD has most all of the components needed to develop a useful AM system. By merging the "Water Operations and Warrants" database application, with the accurate GPS points and pipes from the hydraulic model and the AWD will have a more valuable GIS. By adding databases to track the maintenance, condition and costs of each of these assets the AWD will have a burgeoning AM system.

8.3.1 GIS Build Out

With a relatively straightforward effort the AWD could achieve this by linking the system valve, hydrant and service databases within this system to the GIS. Doing this would provide a full spatial interface to the data. Points could be added to the GIS based upon the physical address information within the database using a process called geocoding. The system valves and hydrants that have already been located accurately using GPS would be linked to the database by simply adding the database ID key value from the associated geocoded point to the GPS mapped asset in the GIS. The customer service curb stop points that have not yet been mapped with GPS could be added to the GIS as an approximate location using this same geocoding process. This effort would provide a direct link back to the data within the database. This simple linking would provide a powerful visual interface to the database, allowing the field crews to access all of that data, complete with the scanned paper records like tie cards, by simply clicking on the asset points on the map. Visual maps could also be created by symbolizing the assets with various colors, shapes or sizes based on the any of the data values, like water usage, size, material, condition, etc.

With the AWD's talented application development staff various possibilities exist to create these additional databases, link them to the assets, and provide an interface for staff to update, capture, process, and share the data.

It is also understood that the AWD continues to advance its GIS system and scan paper records as time permits and staff is available. In addition to the data that has been developed within the custom database application many large scale paper maps or engineering design plans exist. These too could be linked to the GIS system. Large scanners and GIS software could be employed to scan these records and spatially locate them in the map using a similar geocoding process. Doing this would create a scenario where clicking the map in an area (where more information was desired) would present the user with a list of any record documents that exist in the database for that particular area. The corresponding record could then be physically retrieved or immediate access provided to the document via a hyperlink if a scanned or digitized version of the document was available. This technique would allow for immediate hyperlinking of engineering plans and long-term preservation of these remaining old paper documents through scanning. The existing GIS system could be expanded in the future to incorporate and hyperlink photographic records, construction documents and other desirable information when resources are available.

8.3.2 Computerized Maintenance Management System

A Computerized Maintenance Management System (CMMS) system 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 can be 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.

For example, a finished water pump's operating hours can be tracked within a CMMS program using a SCADA interface. The required routine maintenance intervals from the manufacturer's operations manual can be imported into a CMMS, which triggers or predicts when run-time recommendations will be exceeded. Using this type of information, a utility can pre-plan within a given budget cycle appropriate funds and time to make predicted and needed maintenance. This approach will be more beneficial to the AWD as its system is now more complex with two state of the art membrane filtration water treatment plants, several wells and pumping stations, etc. and possibly more in the future.

In summary, the AWD should consider the following items to further enhance its AM Program:

- Building out its GIS system with the additional information presented. Other data to consider for this include the following:
    - The creation of a more formal water main break location and repair database. Having this data accurately recorded and available within the GIS will further assist

the AWD in cost effectively prioritizing its annual Water Main Improvement Program.

  - The creation of a more formal means of tracking and recording water quality (and other) complaints received by its consumers. As the AWD continues to be in a period of large expenditures for implementation of needed treatment at its sources, a comprehensive database of water complaints (by location) will give the AWD the ability to quantify the measurable improvements made after treatment is implemented.
  - The incorporation of unique identification codes for all system assets.
- Implementation of a CMMS system for its increasingly complex water system.

8.4 AVAILABLE FUNDING

AM planning and programs can often be perceived as cost prohibitive and delay their implementation. For the past few years, the MassDEP has been offering Water Infrastructure Assessment and Planning Grants to eligible applicants on a competitive basis. The MassDEP's intent is that the preparation of these AM plans will assist the grant recipients in meeting the regulatory requirements of the Federal government and Commonwealth of Massachusetts. The grant funded project, when completed, would also provide a worthwhile basis for the water system to consider annual budget appropriations and rate system adjustment to ensure regular and timely replacement of equipment prior to failure, thus ensuring the operating capability of their infrastructure.

Grants of up to $40,000 have been awarded by the MassDEP over the past four funding rounds with an in kind services and/or cash match requirement from the community based on its demographics. For the Acton Water District, the required match would be 25% (or up to $10,000).

This is a cost effective means for water suppliers to begin their AM programs and is recommended to be considered by the AWD when the next round of competitive funding is advertised (via a request for proposals from the MassDEP).

SECTION 9

RECOMMENDATIONS

9.1 GENERAL

The intent of this section is to provide an overview of the recommendations made for the Acton Water District's (AWD's) system within the previous sections of this report along with their estimated costs where applicable. Additional 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 10.

9.2 WATER SUPPLY

As presented within Section 4 of this report, the AWD's existing sources were evaluated under various scenarios utilizing standard water works practices. The sources were determined to be capable of meeting the projected average-day demands for the planning period. But the existing sources were not determined to be capable of meeting the projected maximum-day demands for the planning period when limited to 16-hours of operation. They were however, determined to be capable of meeting the projected maximum-day demands for the planning period when operated for up to 24 hours.

Therefore, in order for the AWD to more reliably meet its projected maximum-day demands, the pursuit of other reliable sources of supply was recommended. Based on the evaluation scenarios that considered the largest source to be off-line (for redundancy), a supply deficit of approximately 0.61 million gallons per day (MGD) was identified. When limited to 16-hours of pumping, this amount would correspond to a source having at least a 0.91 MGD capacity.

9.2.1 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 or expanded well source or sources. The AWD has previously identified potential new sources in town include potential bedrock wells, expanded permitted capacity at Conant 2, additional potential wells in the vicinity of Conant 2, permitted capacity at Assabet 3, a potential well(s) on the Flannery-O'Toole Property, and a surface water supply option from Nagog Pond. It is recommended that these sources be further investigated as part of a long-term plan to enhance the reliability of its water supply if a new source is desired. When the Conant 2 wells were approved, MassDEP approved a lower permitted withdrawal than what we applied for citing environmental concerns. With 20 years of pumping and 10 years of environmental monitoring, a case could be made for increasing the permitted withdrawal to the original pump test results. Additionally, if the AWD moves forward with utilization of the previously permitted Assabet 3 source, it would be recommended that additional permitted volume be requested as it is currently only considered as a redundant source. It is also noted that with the incorporation of the Sustainable Water Management Initiative (SWMI) principles into the Water Management Act (WMA) process, the addition of additional supply will get more complicated and require a more in depth analysis.

Based on this information, the AWD will need consider its desire/timing of an additional source.

9.2.2 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. The AWD has a routine well cleaning and redevelopment program for its wells on an as needed basis and it is recommend that this program continue.

Marshall Wellfield - The Marshall source in particular has not been able to consistently pump its permitted amount and its use has been limited. Wright-Pierce previously evaluated the hydrogeologic conditions of the site in 2011 and determined that four gravel packed wells should be able to provide a combined flow of approximately 200 gallons per minute (gpm). Approval from MassDEP was received on August 22, 2011 to proceed with the improvements. At this point, the design, permitting, bidding and construction phases remain for the replacement wells and related pumping station modifications.

Based on the previously identified need for additional supply, it is recommended that the AWD consider proceeding with the rehabilitation of the Marshall source in the near term to strengthen the reliability of its existing system by optimizing use of its current sources (while pursuing additional supply). Regaining this lost pumping capacity would ultimately reduce the amount of additional supply needed from a new source.

A budget estimate of approximately $450,000 is anticipated for the completion of these remaining tasks. However, it is understood that the Marshall source is within the worst SWMI categories for groundwater withdrawal, biological, and net groundwater depletion. As such its use may be significantly impacted/limited in the future and the AWD may not want to proceed with the rehabilitation until its final deposition is better defined.

9.2.3 Treatment Needs

As described within the previous sections, the AWD's sources are treated in a variety of ways. Removal of secondary constituents (that cause consumer complaints) as well as compliance with the surface water treatment regulations is currently being achieved at the North Acton Water Treatment Plant (WTP) and at the South Acton WTP. After which, only the Clapp, Whitcomb, and Conant sources remain without treatment for the removal of the same secondary constituents.

Based on the water use projections for the planning period from Section 3 of this report, the two WTPs would be able to provide for the system's projected average-day demand of 1.87 MGD, but not the system's projected maximum-day demand of 2.74 MGD. Therefore, it will be important for the AWD to supplement the volume from other sources during the higher demand period. If treatment is not provided (for removal of secondary constituents) at the supplemental sources that are used, consumer complaints will continue during the higher demand periods as oxidized minerals are reintroduced into the water system.

Based on well capacity, it is recommended that the next WTP be planned to treat either the Clapp and Whitcomb sources or the Conant 1 and Conant 2 sources. With a registered withdrawal of 0.352 MGD from each the Clapp and Whitcomb sources, an additional 0.70 MGD of treated water quality would be made available. This would make a combined treated water output capacity of 2.9 MGD available (1.7 MGD + 0.5 MGD + 0.70 MGD) for the AWD that would be sufficient to meet the system's projected maximum-day demand of 2.74 MGD. As for the Conant sources, a combined 0.68 MGD (0.468 MGD from Conant 1 and 0.216 MGD from Conant 2) of treated water quality would be made available. This would make a combined treated water output capacity of 2.88 MGD available (1.7 MGD + 0.5 MGD + 0.68 MGD) for the AWD that would also be sufficient to meet the system's projected maximum-day demand of 2.74 MGD.

Treatment for the Clapp and Whitcomb sources or the Conant sources would provide the following additional benefits:

- A third WTP for redundancy
- A WTP in the western or eastern portion of Acton
- Provide the AWD with flexibility in its treatment operations so that the treated wells are allowed to rest and not solely rely on the North Acton and South Acton WTPs

The process for the next WTP would need to be started with piloting for technology verification, and proceed with permitting & design, through construction. For this entire process, the AWD should be plan for an approximate three-year period.

Based on the elevated inorganic constituents (iron and manganese), color, and the recent microbiological history of the sources, an advanced treatment process such as membrane may be warranted. Therefore, the preliminarily estimated cost for this process is based on the previous experience with the North Acton and South Acton WTPs. For this, $150,000 is estimated for piloting and $8.5M is estimated for design and construction for either the Clapp and Whitcomb sources.

As iron and manganese are the primary constituents of concern at the Conant sources, a simpler GreensandPlus™ pressure filtration process would be appropriate. A budgetary level estimated cost for this process would be up to $100,000 for piloting (the two sites individually) and approximately $5.5M for a combined water treatment facility with connecting water main.

Should the newer biological filtration process (Ferazur/Magnazur) with a lower potential backwash residuals volume be desired for investigation, the budgetary level estimated cost to pilot the process would be up to an additional $200,000 (at the two sites individually).

9.3 DISTRIBUTION SYSTEM

The AWD's distribution system was evaluated to assess its hydraulic adequacy utilizing the computerized hydraulic model. Various improvements were recommended to improve fire flow capacity, water main replacements, and water storage tank mixing. Each is summarized in the sections that follow. Figure 9-1 presents an overview of the recommended distribution system improvements.

Notes:

1. Refer to Section 9 in report for improvement descriptions.

W:\GIS_Development\WaterModel\MA\Acton\MasterPlan Figures\9-1 Capital Improvement Plan.mxd

Capital Improvement Plan
Acton Water District
Acton, Massachusetts

PROJ NO: 13748A	DATE: Jan 2018	FIGURE:
WRIGHT-PIERCE Engineering a Better Environment		9-1

9.3.1 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 commercial locations within the system were determined to be deficient. In total, the 17 ISO fire flow test locations were evaluated using the hydraulic water model and 11 were found to be deficient. By implementing the improvements described previously, the available fire flow at seven locations was increased to values at or above the ISO requirement. Available fire flow at the four remaining deficient locations improved as a result of the noted possible upgrades, but the ISO flow requirements were not fully met. In order to meet the required fire flow at these locations, large scale improvements to the existing system would be needed and AWD input is requested prior to making recommendations for those locations.

The recommended ISO improvements are summarized and estimated capital costs for their construction are include within Table 9-1.

TABLE 9-1
RECOMMENDED ISO IMPROVEMENTS

No.	ISO Test Location	Street Name	From	To	Length (ft)	Existing Diameter (in)	Existing Material	Proposed Diameter (in)	Estimated Capital Cost*
1	ISO 1	Central Street	Nashoba Road	Elm Street	3,150	8	AC	10	$614,250
		Central Street	Elm Street	Mass Ave	3,100	6	AC/CI	10	$604,500
								Subtotal	**$1,218,750**
2	ISO 5	Massachusetts Ave	Charter Road	Main Street	600	6	CI	8	$108,000
								Subtotal	**$108,000**
3	ISO 6	Piper Road	Massachusetts Ave	Discovery Way	200	6	AC	8	$36,000
								Subtotal	**$36,000**
4	ISO 8	Old High Street	High Street	Powder Mill Road	1,300	8	CI	12	$279,500
								Subtotal	**$279,500**
5	ISO 10 & 11	Concord Road	Main Street	Hosmer Street	3,250	6	CI	12	$698,750
		Concord Road	Hosmer Street	Great Road/Pope Road	4,400	10	AC	12	$946,000
								Subtotal	**$1,644,750**
6	ISO 13	Main Street	Nagog Hill Road	Brook Street	2,600	12	DI	16	$624,000
		Great Road	Concord Road	Brook Street	5,400	No Ex WM	N/A	12	$1,161,000
		Brook Street	Main Street	Great Road	2,200	10	AC	12	$473,000
								Subtotal	**$2,258,000**
								ISO Improvement Total	**$5,545,000**

* Estimated capital costs include estimated construction costs only. Engineering costs are not included as they are variable due to project specifics that are unknown at this time.

9.3.2 Water Main Improvement Program

The AWD budgets $500,000 per year for pipe upgrades. In essence this is the amount required to replace approximately 2,750 linear feet (LF) of 8-inch water main per year assuming a unit cost of $180 per linear foot for cement-lined ductile iron pipe. Based on this estimation, approximately 27,500 LF of new pipe construction can be funded over a 10-year capital improvement plan period.

The specific water main replacement recommendations previously identified within Section 5 of this report are summarized along with their estimated capital construction costs within Table 9-2.

TABLE 9-2
RECOMMENDED 10-YEAR WATER MAIN IMPROVEMENT PLAN

No.	Street Name	From	To	Length (ft)	Existing Diameter (in)	Existing Material	Proposed Diameter (in)	Estimated Capital Cost*
1	Arlington Street	Birch Ridge Road	Notre Dame Road	2,750	6	AC	8	$495,000
2	Stow Street	Liberty Street	Maple Street	1,360	6	AC	8	$244,800
3	Billings Street	Robbins Street	(Dead End)	1,100	6	AC	8	$198,000
4	Robbins Street	Prescott Road	(Dead End)	790	8	AC	8	$142,200
5	Prospect Street	Massachusetts Avenue	Central Street	4,450	6	AC	8	$801,000
6	Spencer Road	Prospect Street	Flint Road	1,200	6	AC	8	$216,000
7	Charter Road	Massachusetts Avenue	-	860	6	AC	8	$154,800
8	Huron Road	Nashoba Road	Oneida Road	640	6	AC	8	$115,200
9	Nagog Hill Road	Main Street	Concord Road	2,010	6	AC	8	$361,800
10	Lawsbrook Road	-	Lisa Lane	200	6	AC	8	$36,000
11	Oakwood Road	Piper Road	Brucewood Road	2,280	8	AC	8	$410,400
12	Alcott Street	Concord Road	Emerson Drive	890	8	AC	8	$160,200
13	Pinewood Road	Brucewood Road	Oakwood Road	800	6	AC	8	$144,000
14	Massachusetts Avenue	Main Street	Piper Road	3,530	8	AC	8	$635,400
15	Marian Road	Squirrel Hill Road	Willow Street	1,510	6	AC	8	$271,800
16	Algonquin Road	Huron Road	Oneida Road	1,290	6	AC	8	$232,200
17	Mass. Ave Extension	Massachusetts Avenue	-	600	6	AC	8	$108,000
18	Arlington Street	Central Street	Spruce Street	1,320	6	CI	8	$237,600
19	Newtown Road	Minuteman Road	Main Street	700	6	AC	8	$126,000
			Total LF	28,280			**Total Capital Cost**	**$5,090,400***

* Estimated capital costs include estimated construction costs only. Engineering costs are not included as they are variable due to project specifics that are unknown at this time.

9.3.3 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. As noted previously, the AWD system has sufficient (i.e., excess) storage volume and adequate redundancy with its four water storage tanks under the industry standard evaluation.

However, as noted by the previous Master Plan update, the AWD has also experienced large water usage from storage during prolonged high demand periods. To better capture this event, a more extreme water storage evaluation was performed that identified a potential deficit of water storage when the largest tank was considered to be offline. For this, a minimum 2.0 million gallon (MG) water storage tank was recommended at the Great Hill site that would offer multiple benefits.

Should the AWD decide to implement this option, a budget of $2.55M is estimated for engineering and construction.

9.3.3.1 Mixing Systems

Due to the storage volume present within the AWD's system in combination with the AWD'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 active mixing systems be implemented at each tank. Since the Flagg Hill Tank already had a GridBee mixing system recently installed, it was recommended that the AWD consider standardizing around the product at its other three tanks.

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

- 0.5 MG Great Hill Standpipe: $30,000
- 3.0 MG Nagog Hill Reservoir: $60,000
- 3.0 MG Wampus Hill Reservoir: $60,000

Should the AWD decide to proceed with a larger tank at the Great Hill location, it is recommended that the GridBee mixing unit be sized so that it can then be reinstalled within the larger tank when it is constructed (under the assumption it's installed in the existing tank first).

9.3.3.2 Tank Repairs

The AWD's four water storage tanks were last inspected in July of 2017. In accordance with the Massachusetts Department of Environmental Protection (MassDEP) Guidelines and Policies, the tanks will need to be inspected again no later than July of 2022. As noted within Section 2 of this report, a variety of items requiring maintenance and/or repair were identified for each of the tanks.

An estimated cost (and ranges where appropriate) for each storage tank excluding recoating is as follow:

- 0.5 MG Great Hill Standpipe: $15,000
- 2.0 MG Flagg Hill Reservoir: $175,000 to $265,000
- 3.0 MG Nagog Hill Reservoir: $20,000
- 3.0 MG Wampus Hill Reservoir: $15,000

In general, all costs are estimated based on limited information that is currently available and are presented as year 2017 dollars. All costs should be re-visited and revised as neccesary when additional detail is available and prior to when the project is anticipated to move forward.

SECTION 10

RECOMMENDED CAPITAL IMPROVEMENT PROGRAM

10.1 OBJECTIVE

The Acton Water District (AWD) has and continues to undergo significant changes within its system. Treatment of its sources has become a priority and is demanding a majority of its financial resources. As there are many other needs that have been identified in the AWD's future, a well laid out Capital Improvement Plan (CIP) will help the AWD 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 AWD's moving forward. The estimated capital costs for the newly identified needs are presented. Routine costs for operation and maintenance are not included.

10.2 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 9 of the report. 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 AWD's system are maintenance driven. Over time, the AWD may have to shift the priority of projects in order to respond to the changing 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 AWD 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 10-1 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.

10.2.1 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:

- ***Conant 1 and Conant 2 Piloting*** - Due to the Corrective Action Plan (CAP) required by the MassDEP due to manganese concerns, the piloting of suitable treatment technologies should be planned for in the near term.
- ***Conant 1 and Conant 2 Treatment*** - Although important for consistently good water quality, the implementation of treatment for the Conant sources was identified for the earlier part of the next ten-year period to also account for the large capital expenditures that the AWD has incurred during the last several years.
- ***Tank Mixing Systems and Repairs*** - The implementation of mixing systems and tank repairs. These are identified as high priorities since improved mixing and turnover within the existing tanks will be important now that the two new water treatment plant are on-line and significantly improved water quality is pumped into the distribution system. The improvements for the three tanks without mixing currently installed (Great Hill, Nagog Hill, and Wampus Hill) are timed so that they are completed in the near term. 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.

Depending on AWD desires, the implementation of a new source or sources could likely be considered a high priority recommendation.

10.2.2 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:

- ***Marshall Wellfield*** – The improvements to recover the lost capacity of the Marshall Wellfield and make it more reliable was noted to be the first intermediate-term improvement as it was important. However, it was not identified as a high priority improvement due to the AWD's concern that its use may be limited by the Water Management Act's (WMA) Sustainable Water Management Initiative (SWMI) provisions.
- ***Whitcomb and Clapp Treatment*** – Although also important for consistently good water quality, the implementation of treatment for the Whitcomb & Clapp sources was also identified for the middle of the next ten-year period due to cost it was identified to occur after the Conant treatment project.
- ***Flagg Hill Repairs*** – As the Flagg Hill Tank already has a tank mixing system installed, its repairs were identified to occur after mixing is installed and repairs completed at the other three tanks.
- ***Great Hill Standpipe Replacement*** – The potential replacement of the existing Great Hill Standpipe with a larger 2.0 million gallon (MG) tank was identified for this period to allow for planning, budgeting, and so as to not coincide with other large expenditures.

10.2.3 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 AWD has and will continue to incur over the next several years.

As previously noted in the other sections, it is understood that the AWD has an annual budget of $500,000 for an on-going water main improvement program (WMIP). Accordingly, this amount (initially split in half between ISO and WMIP) has been allocated to the 10-year CIP presented within Table 10-1. Based on input from the AWD and as other needs arise, the priority and order can be adjusted.

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 AWD's Treasurer and Finance Committee to assure maximum financial benefit in any given year.

TABLE 10-1
RECOMMENDED 10-YEAR CAPITAL IMPROVEMENT PROGRAM
ACTON, MASSACHUSETTS

	Total Estimate	Year 1 2018	Year 2 2019	Year 3 2020	Year 4 2021	Year 5 2022	Year 6 2023	Year 7 2024	Year 8 2025	Year 9 2026	Year 10 2027
SUPPLY											
New Sources											
Bedrock Wells	TBD	TBD	TBD	TBD	TBD	TBD	TBD	TBD	TBD	TBD	TBD
Flannery-O'Toole	TBD	TBD	TBD	TBD	TBD	TBD	TBD	TBD	TBD	TBD	TBD
Assabet 3	TBD	TBD	TBD	TBD	TBD	TBD	TBD	TBD	TBD	TBD	TBD
Existing Sources											
Marshall Wellfield	$450,000						$450,000				
Treatment											
Conant 1 & Conant 2	$5,800,000	$300,000	$500,000	$3,000,000	$2,000,000						
Whitcomb & Clapp	$8,650,000					$150,000	$500,000	$4,000,000	$4,000,000		
DISTRIBUTION											
ISO Improvements											
Various	$2,500,000	$250,000	$250,000	$250,000	$250,000	$250,000	$250,000	$250,000	$250,000	$250,000	$250,000
WMIP Improvements											
Various	$2,500,000	$250,000	$250,000	$250,000	$250,000	$250,000	$250,000	$250,000	$250,000	$250,000	$250,000
Storage Improvements											
Mixing Systems											
Great Hill	$30,000	$30,000									
Nagog Hill	$60,000		$60,000								
Wampus Hill	$60,000			$60,000							
Repairs											
Great Hill	$20,000	$20,000									
Flagg Hill	$250,000				$250,000						
Nagog Hill	$25,000		$25,000								
Wampus Hill	$20,000			$20,000							
TOTAL	**$20,365,000**	**$850,000**	**$1,085,000**	**$3,580,000**	**$2,750,000**	**$650,000**	**$1,450,000**	**$4,500,000**	**$4,500,000**	**$500,000**	**$500,000**

Note: All cost estimates are presented in 2017 dollars.

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