Water Supply and Treatment Evaluation

Hopkinton, MA

FINAL REPORT

July 2014

Weston & Sampson

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EXECUTIVE SUMMARY

The Town of Hopkinton faces some significant water supply questions:

- Do we have enough water?
- How much water do we need?
- Should we treat our ground water or purchase water from the MWRA?

The primary goal of this study and master plan was to determine the answers to these questions and to provide a road map for the Town in developing water supplies to serve the Town for the next 20 years.

Do we have enough water? The Town of Hopkinton is served through ground water supplies. One of the issues with ground water is that there is very little storage for peaking flow rates in the summer when demands are highest and water tables are typically at their lowest. The Town must have enough ground water supply for the summer peak day water use. *When the ground water table is low from lack of rainfall, the Town struggles to meet these summertime demands.* The Town is on the verge of not having adequate water supply in the summer during peak demands and low ground water levels. In wet years this problem is not as severe, in dry years, the problem can be drastic and future development and increased users will only worsen this impact. Unfortunately, these maximum day demands are the design point that Hopkinton must design a water supply around. As the water supply volume must be capable of serving these maximum day demands, there is great financial incentive to reducing these summer peak day demands. This can be performed through water conservation and education efforts. We recommend that Hopkinton continue and strengthen these conservation efforts, focused on reducing the peak demands that occur in the summer.

When we discuss water supply capacity in Hopkinton, we typically are referring to the well's safe yield. The safe yield of a well is typically determined on an individual well, theoretical basis, and does not generally account for well hydraulics, plugging, and other well interferences. Safe Yield defines the maximum dependable amount of water withdrawal that can be made continuously from a source during a period of years in which the probable driest period is likely to occur and incorporates environmental protection factors and hydrologic factors. For instance, the combined safe yield of the Alprilla wells No. 7 and 8 are 0.42 MGD. In order to obtain these volumes on a daily basis, the wells would need to be pumped 24 hours a day, which in the summertime when the water table is low and both wells are working against each other - may not be possible. This situation occurs at the Fruit Street wells also, which can lead to the summer water supply shortages. Even if the regulatory restrictions on pumping were lifted at the Fruit Street well site, the Town is limited in how much water they can physically get out of the ground due to reduced water table and well drawdown cones of influence. We have recommended additional ground water modeling in this site utilizing current data to identify if more water can be physically removed from this aquifer if new pumping equipment is installed or additional wells.

The Town of Hopkinton is also limited in how much they can pump through regulatory water management act (WMA) permitting restrictions. For instance, although the Fruit Street wells have a safe yield capacity of 1.35 MGD, the Town is limited through the WMA permit in how much they can pump from these wells to 0.75 MGD. The process of permitting new ground water sources and of permitting increased withdrawals from existing ground water sources in Massachusetts is currently undergoing a significant transformation through the Sustainable Water Management Initiative (SWMI). These new permitting regulations will have significant impacts on the Town of Hopkinton future withdrawals. The Town will need to synchronize their efforts toward future developments to take full advantage of all the mitigation and minimization measures being employed to protect water supply.

We recommend that the Town request an increase in the Fruit Street pumping limit of 0.75 MGD to 0.85 MGD through the WMA permit increase request and utilize the recharge from the Fruit Street wastewater treatment plant as mitigation and minimization measures. However, depending on well conditions (drawdown, water table, plugging) there may be times when it is not possible for the Town to pump more than the 0.75 MGD from the Fruit Street aquifer during the summer as the wells experience significant ground water level drawdowns and begin impacting each other during the dry conditions.

We have outlined other mitigation and minimization measures in Chapter 3 and in the Hopkinton Stormwater Recharge & Infiltration Planning (2013 SWMI Grant) prepared by Weston & Sampson. For instance, it is imperative that the Town continue with their efforts to reduce unaccounted for water use (UAW) to meet the 10% standard. This will continue to impede the Town in trying to lift some of the regulatory restrictions through the Water Management Act unless resolved. The Town has outlined a UAW Compliance Plan which identifies improvements that the Town can make to continue working toward the 10% UAW standard.

How much water does Hopkinton need? Weston & Sampson has outlined future water demand forecasts through 2033 utilizing the DCR Demand forecasting methodology. Weston & Sampson has performed the calculations assuming that growth will continue similar to the last 10 years for the next 10 years and that after that, the Town will continue to grow at a lesser rate. If the Town wants to continue to promote development, it will be imperative to have a robust water supply system.

The water quality in the Whitehall Wells No. 4 and No. 5 has high levels of iron and manganese. The safe yield of these wells represents a significant portion of the Town's water supply (0.83 MGD). The Town currently utilizes these wells during the summer peak flow days for supplemental water to meet demand, but the wells are only operated several days a year. Currently, iron and manganese are secondary contaminants and it is possible for the Town to use this water without regulatory fines. The DEP will begin regulating manganese at a limit below the manganese levels in No. 4 and No. 5 within the next several years, which will keep the Town from being able to pump these wells for supplemental summertime water supply without constructing a treatment plant. The other impact of the high iron and manganese in the ground water will be on the maintenance of these wells. These wells are not currently pumped daily. If the Town begins pumping these wells daily, the fouling will likely be significant which will cause increased maintenance. We have observed in other communities in the vicinity of Hopkinton with ground water wells with similar water quality to No. 4 and No. 5, that the water quality gets worse, as these wells are pumped more regularly.

Should we treat our ground water or purchase water from the MWRA? We have compared the alternative of treating the Whitehall wells versus connecting to the MWRA. If Hopkinton was to construct a connection directly to the MWRA (either a direct pipeline or a connection at the Southborough town line), the capital cost would be higher than constructing a water treatment plant to treat the Whitehall wells, but if Hopkinton can utilize MWRA water through their existing connection with Ashland, the capital costs to build facilities is relatively low for a good surface water supply. A surface water supply has potential to provide Hopkinton significant freedom from their summer ground water pumping shortages and the WMA permit. The down side will be the cost to purchase water, as MWRA water is expensive when compared with the cost to make water in Hopkinton. In comparing the MWRA alternative versus constructing a WTP to treat Wells No. 4 and 5, we have estimated that Southborough and Ashland will charge a 25% wheeling fee each and that the cost of water will approach the Southborough retail rate of water ($3.50/HCF) which is higher than Hopkinton's retail rate. If Hopkinton continues to utilize their well supplies for base volume and only utilizes MWRA water for the peak summer days, Hopkinton would only need to purchase a relatively small volume of water (60 to 70 MG/Year).

The Town of Hopkinton is working with Ashland for an arrangement that would allow Ashland to send more water from the Ashland WTP to Hopkinton, and meet their supply shortages caused by this through the MWRA. If this arrangement can be solidified, Hopkinton will never see MWRA water, so the mixing issues of chlorine and chloramines and the interbasin transfer issues are significantly reduced if not eliminated. Another significant benefit to an MWRA connection in Ashland will be the relief from the WMA permit reservoir shutoffs. This will significantly benefit Hopkinton as the volumes sent to Hopkinton can be reduced during the summer dry conditions.

In order to compare the MWRA water supply alternative with construction of a water treatment plant to treat the Whitehall wells, we included the operating costs with the capital over a 20-year period. The MWRA 20-year life-cycle cost is approximately $13.3 million (the majority of this cost is purchasing water) versus $18 million (the majority of this cost is capital) for a treatment plant located at the Fruit Street site to treat the Whitehall wells. These costs are outlined in detail in Chapter 8.

If a WTP is constructed to treat Whitehall Well No. 4 and No. 5 water, we recommend that the plant be constructed on the Fruit Street site. This will provide the following benefits over constructing a plant at the Whitehall site:

- The Whitehall site has land restrictions and would require that the Town get federal approval for land transfer;
- The water quality in the Whitehall wells is expected to worsen as the wells are pumped more regularly which will foul wells and increase maintenance costs;
- If a WTP is constructed on the Fruit Street site, wells No. 4 and No. 5 can be rested and the plant can still operate;
- Well No. 2 water can be treated through the same WTP as No. 4 and No. 5 to remove manganese;
- Construction of a WTP on the Fruit Street site will prevent having to construct a separate blending facility for Well No. 2 on the Fruit Street site;
- Recharge for residuals disposal at the Fruit Street site can be used through SWMI as mitigation / minimization measures.

If the Town chooses to connect to the MWRA, they will not need to bond for a water treatment plant. The costs for the MWRA can be more easily phased over the 20-year period (assuming wheeling water through Ashland). However given the high cost to purchase water through the MWRA connection, as the Town uses more water, the financials of this connection may shift and make a treatment plant for the Whitehall wells more financially beneficial. If demands increase higher than projected, and the Town is purchasing close to the safe yield of the Whitehall wells from the Ashland / MWRA connection throughout the year, they may choose to construct a plant at that time. If this situation occurs, the peaking capacity of MWRA surface water supply can still be utilized and may prove very instrumental in helping Hopkinton meet their peak summertime demands.

Water supply is not available at the same relative costs that it once was, and a robust water system that can support growth will be costly. We have outlined approximately $11 million in capital improvement projects in this study that will provide adequate water supply at good quality for the next 20-years. The impact of these capital improvements will be significant on Hopkinton water rates. Water rates will also be affected by the purchase of more expensive water from the MWRA. It will be imperative to identify the water rate impacts so that the Town can plan and mitigate cost impacts through future developments. Without proper planning, the impact of these developments causing the Town to need additional water supply will adversely impact existing rate payers and customers. The alternative of utilizing MWRA water, purchased through Ashland, will result in less impact to water rates than constructing a water treatment plant to treat the Whitehall wells. Water rates may continue to increase as more water is purchased through this connection, but the increases are expected to be more gradual with the MWRA alternative over constructing a water treatment plant.

Both Grove Street water storage tanks are in need of rehabilitation. The cost to rehabilitate these tanks is very high and the small Grove Street Tank, constructed in 1922, is at the end of its useful life. The small Grove Street Tank cannot be abandoned and removed, because the Large Grove Street Tank is also in need of rehabilitation and the Town cannot maintain water pressure during periods of high demand without at least one water storage tank at this site. These tanks are also short which creates pressure problems to customers who live on the hill when water level drops significantly. This hill has been targeted for construction of a high service pressure system. Construction of the high service system and an elevated tank, would allow the Town to drop the water level in the existing Grove Street tank to provide adequate mixing and turnover and would also improve water system storage from the existing water tanks to the Main Service System. We recommend that the Town construct a new low maintenance tank in the site of the existing small Grove Street Tank. This will allow both welded steel tanks to be abandoned from the site.

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1.0 EXISTING WATER SYSTEM

1.1 General

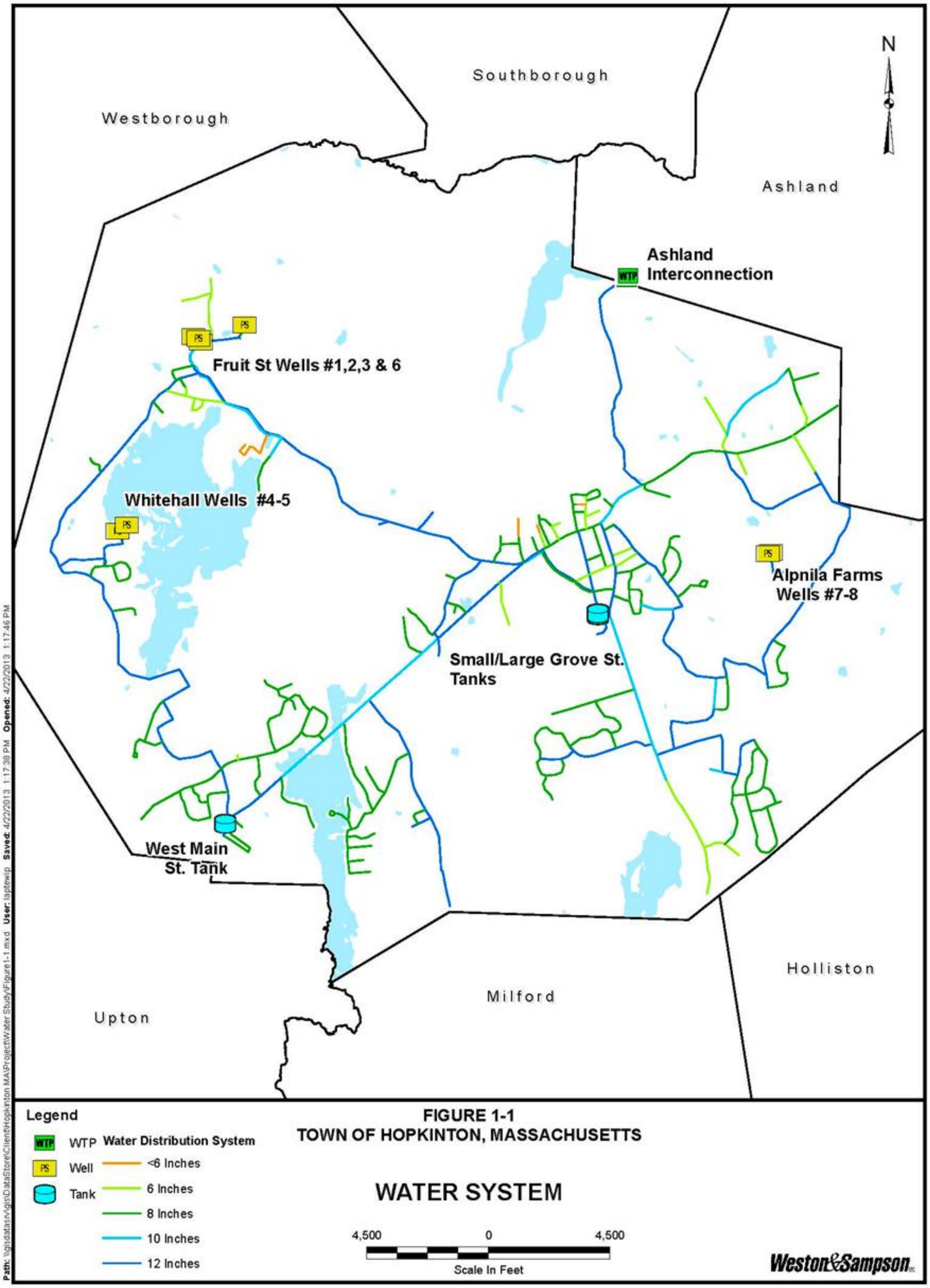
The Town of Hopkinton is a rural community approximately 26 miles west of Boston. Hopkinton has a current population of approximately 15,812 people based on the 2012 Hopkinton internal population census. According to the Town's 2011 Massachusetts Department of Environmental Protection (MassDEP) Annual Statistical Report (ASR), the Town served water to 8,900 customers, an estimated 56 percent of the 2012 Town population of 15,812.

The Hopkinton water system is comprised of one pressure zone with a hydraulic grade line of approximately 600 feet set by the overflow elevation of the Town's three water storage tanks. The small and large Grove Street Standpipes are adjacent to each other and have capacities of 0.32 million gallons (MG) and 1.5 MG respectively. The West Main Street Standpipe has a capacity of 0.793 MG. Figure 1-1 shows the Water Distribution System including sources, pumps, and standpipes.

1.2 Ground water Supply Facilities

The Town's water supply system includes eight operating gravel packed well sources located at three different Town owned sites and water supplied from an interconnection with the Town of Ashland. The Town's well sources are located in the Concord River Basin of the Sudbury-Assabet-Concord (SuAsCo) River Watershed:

- Fruit Street Wells No. 1, 2, 3, and 6
- Whitehall Wells No. 4 and 5
- Alprilla Farm Wells No. 7 and 8


Southborough
Westborough
Ashland
N
Ashland
Interconnection
WTP
PS
Fruit St Wells #1,2,3 & 6
Whitehall Wells #4-5
Alpnila Farms
Wells #7-8
Small/Large Grove St.
Tanks
West Main
St. Tank
Holliston
Milford
Upton
Legend
WTP
Well
Tank
Water Distribution System
<6 Inches
6 Inches
6 Inches
8 Inches
10 Inches
12 Inches
FIGURE 1-1
TOWN OF HOPKINTON, MASSACHUSETTS
WATER SYSTEM
4,500
0
4,500
Scale In Feet
Weston&Sampson

1.2.1 Fruit Street Wells

The Fruit Street Wells consist of Wells No. 1, 2, 3 and 6 with Wells No. 1, 2, and 3 located on the west side of Fruit Street and Well No. 6 constructed more recently on the east side of Fruit Street. The Town owns 85.75 acres around the Fruit Street Wells (Figure 1-2 and 1-3). The Main Fruit Street Building was constructed in 1958 on the west side of Fruit Street near Wells No. 1, 2, and 3. The Main Fruit Street Building is used to house a natural gas Olympian 150 kW, 3 phase, 240 volt, generator, which provides standby power for Wells No. 1, 2, and 3, and the sodium hydroxide chemical feed system for Well No. 1.

1.2.1.1 Well No. 1

Well No. 1 is a gravel-packed well with a depth of 33 feet, 18-inch diameter casing and 10 feet of 18-inch diameter screen. The Well No. 1 pump is a vertical turbine pump with an 8-inch diameter pump column and a design flow of 530 gallons per minute (gpm) at 345 feet of total dynamic head (TDH). The pump is equipped with a 60 horsepower (HP) premium efficiency motor manufactured by General Electric (GE). The Well No. 1 pump, motor, and 8-inch discharge piping are housed in an underground vault with a sump pump to prevent the build-up of water within the vault and a unit heater for temperature control. The Well No. 1 pump discharge piping runs through the Main Fruit Street Building where the water is treated for corrosion control by pH adjustment with sodium hydroxide. Sodium hypochlorite is not added directly to Well No. 1 water.

1.2.1.2 Well No. 2

Well No. 2 is a gravel packed well with a depth of 42.5 feet, 18-inch diameter casing and 10 feet of 18-inch diameter screen. The Well No. 2 pump is a vertical turbine pump with an 8-inch diameter pump column and a design flow of 500 gpm at 350 feet of TDH. The pump is equipped with a 60 HP premium efficiency motor manufactured by GE. The Well No. 2 pump, motor, and associated equipment are contained within a two level block building. The pump house is equipped with a unit heater for temperature control and stores the sodium hydroxide and sodium hypochlorite chemical feed systems for treatment of the well water. The water from

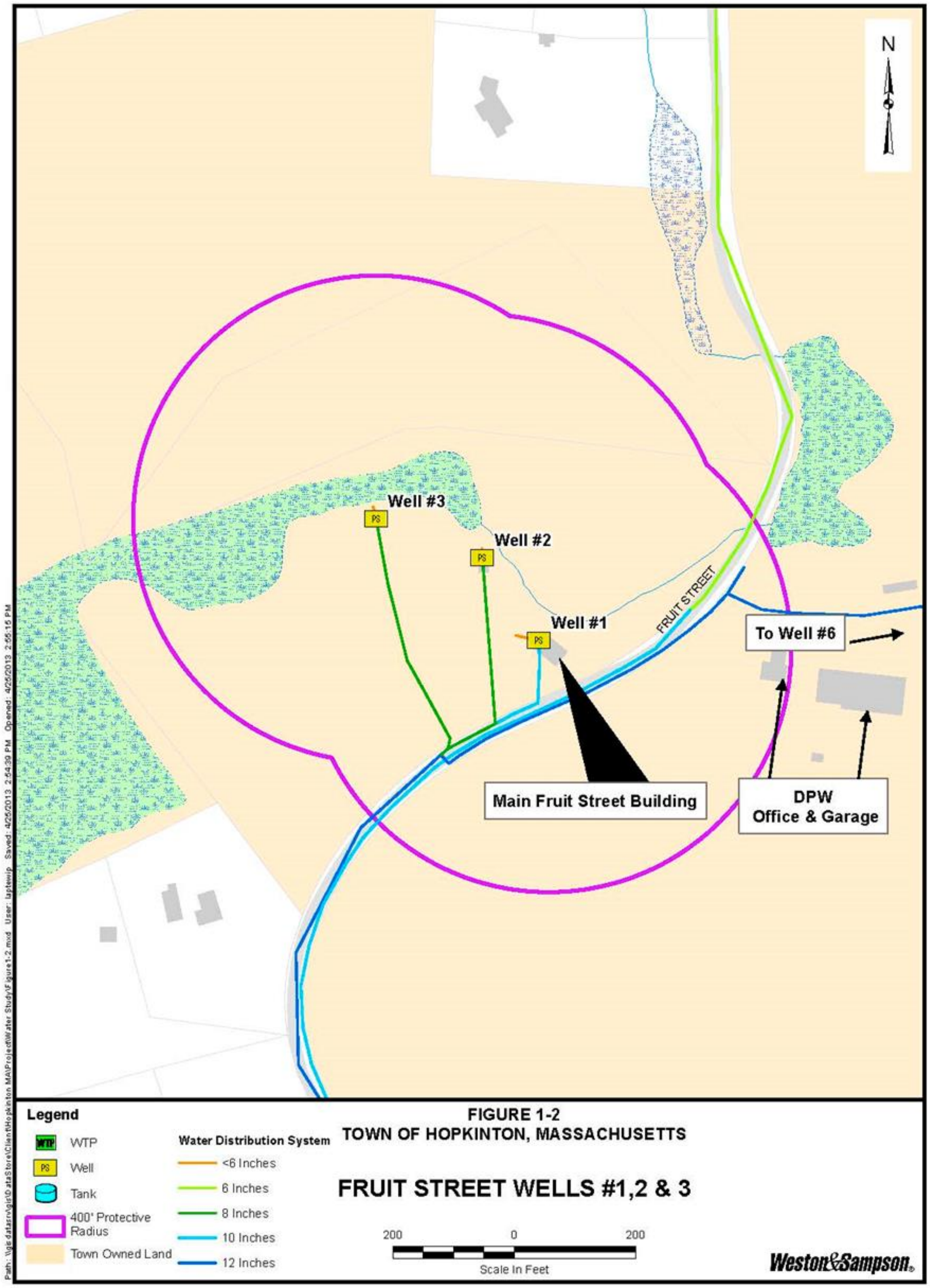


FIGURE 1-2
TOWN OF HOPKINTON, MASSACHUSETTS

FRUIT STREET WELLS #1,2 & 3

Well No. 2 is treated for pH adjustment and corrosion control using sodium hydroxide and for disinfection using sodium hypochlorite stored in the well building. The sodium hypochlorite dose is increased at Well No. 2 such that after the water is mixed with the water discharged from Well No. 1 a target chlorine residual of 1.0 mg/L is maintained in the distribution system

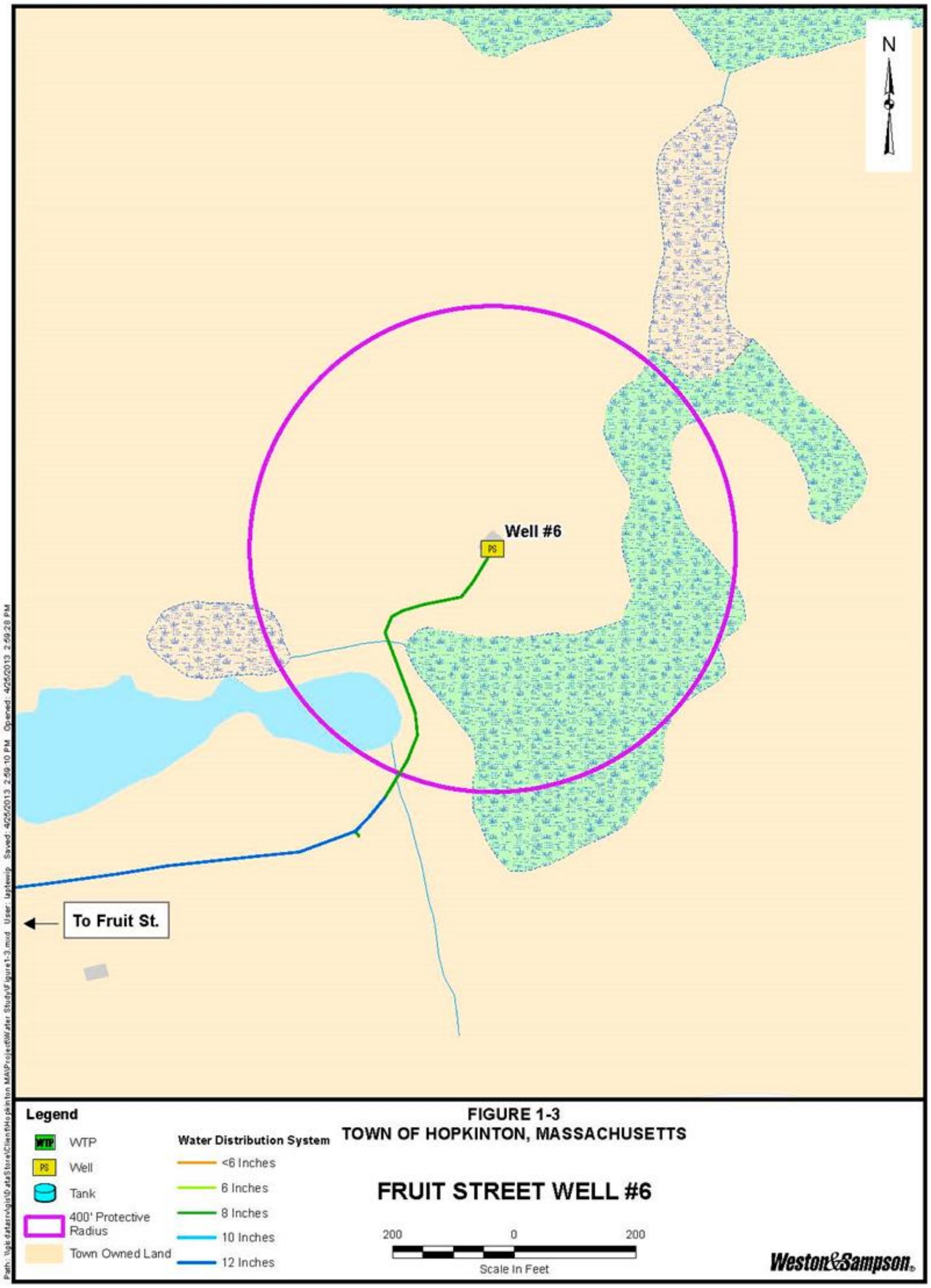
1.2.1.3 Well No. 3

Well No. 3 is a gravel-packed well with a depth of 36.7 feet, 24-inch diameter casing and 10 feet of 24-inch diameter screen. The Well No. 3 pump is a vertical turbine pump with a 6-inch diameter pump column and a design flow of 210 gpm at 320 feet of TDH. The pump is equipped with a 25 HP premium efficiency motor manufactured by GE. The Well No. 3 pump and motor are contained within a concrete underground vault. The vault is equipped with a unit heater for temperature control and a dehumidifier. Some of the instruments associated with Well No. 3, such as the flow and water level recorders, are located in the Main Fruit Street Building. The Well No. 3 pump discharge is not treated with chemical addition. The emergency power supply is supplied by the generator located in the Main Fruit Street Station.

Due to the water quality in the well, and in particular the presence of iron and manganese, the available yield has been reduced, and Well No. 3 has been reclassified by the MassDEP from an active to an emergency water source. Well No. 3 was not activated in 2011 or 2012 based on the MassDEP ASRs. Well No. 3 has not been included in the water supply evaluations completed in this study due to the low initial safe yield (0.12 MGD), the further reduction in yield due to water quality, and the reclassification by the MassDEP from an active to an emergency source.

1.2.1.4 Well No. 6

Well No. 6 is a gravel packed well, constructed in 2009, with a depth of 70 feet, 18-inch diameter casing. The Well No. 6 pump is a vertical turbine pump with a 6-inch diameter pump column and a design flow of 500 gpm at 374 feet of TDH. The pump is equipped with a 75 HP premium efficiency motor manufactured by Emersen. The Well No. 6 pump, motor, chemical feed systems, and associated equipment are contained within the Well No. 6 Pump Station. The pump station is equipped with an electric unit heater and exhaust fan for temperature control. A propane gas powered generator located in the Well No. 6 Pump Station is used for emergency power. The water from Well No. 6 is treated for pH adjustment and corrosion control with potassium hydroxide and for disinfection using sodium hypochlorite.


N
Well #6
PS
To Fruit St.
Legend
WTP
Well
Tank
400' Protective Radius
Town Owned Land
Water Distribution System
<6 Inches
6 Inches
8 Inches
10 Inches
12 Inches
FIGURE 1-3
TOWN OF HOPKINTON, MASSACHUSETTS
FRUIT STREET WELL #6
200
0
200
Scale In Feet
Weston&Sampson

1.2.2 Whitehall Wells

The Whitehall Wells No. 4 and 5 were constructed in 1987 and are located off of Charles McIntyre Lane and Donna Pass (Figure 1-4). The Town owns 3.7 acres near the wells, and the remaining land in the Zone 1 is owned by the Department of Conservation and Recreation (DCR). Each well has its own separate building where the pump, motor, associated equipment and chemical feed systems are located.

Due to a decrease in pumping capacity from the original wells, the Town constructed two satellite wells, one to supplement the water supply from each existing well. The new satellite wells, Wells No. 4A and 5A, are 12-inch diameter natural wells with submersible pumps that run in conjunction with the operation of the main well pumps. Each satellite well pump discharge ties into the main well with the discharge located between the well casing and the pump column.

1.2.2.3 Well No. 4

Well No. 4 is a gravel packed well with a depth of 38 feet, 24-inch diameter casing and 5 feet of 24-inch diameter, 160 slot stainless steel screen. The Well No. 4 pump is a vertical turbine pump with 6-inch diameter pump column and a design flow of 250 gpm at 300 feet of TDH. The pump is equipped with a 40 HP premium efficiency motor manufactured by US Motor. Based on the 2012 MassDEP Annual Statistical Report, Well No. 4 was only used during the months of April through August. Due to the poor water quality with elevated levels of iron and manganese, the well is only used to supplement the other Town's sources during periods with high water demands.

The Well No. 4 Pump Station is equipped with an electric unit heater for temperature control. A propane gas unit heater fed from the propane tank outside of Well No. 5 provides a backup. There is no standby power supply for the well pump at Well No. 4. The pump station is used to house the sodium silicate and sodium hypochlorite chemical feed systems for treatment of the

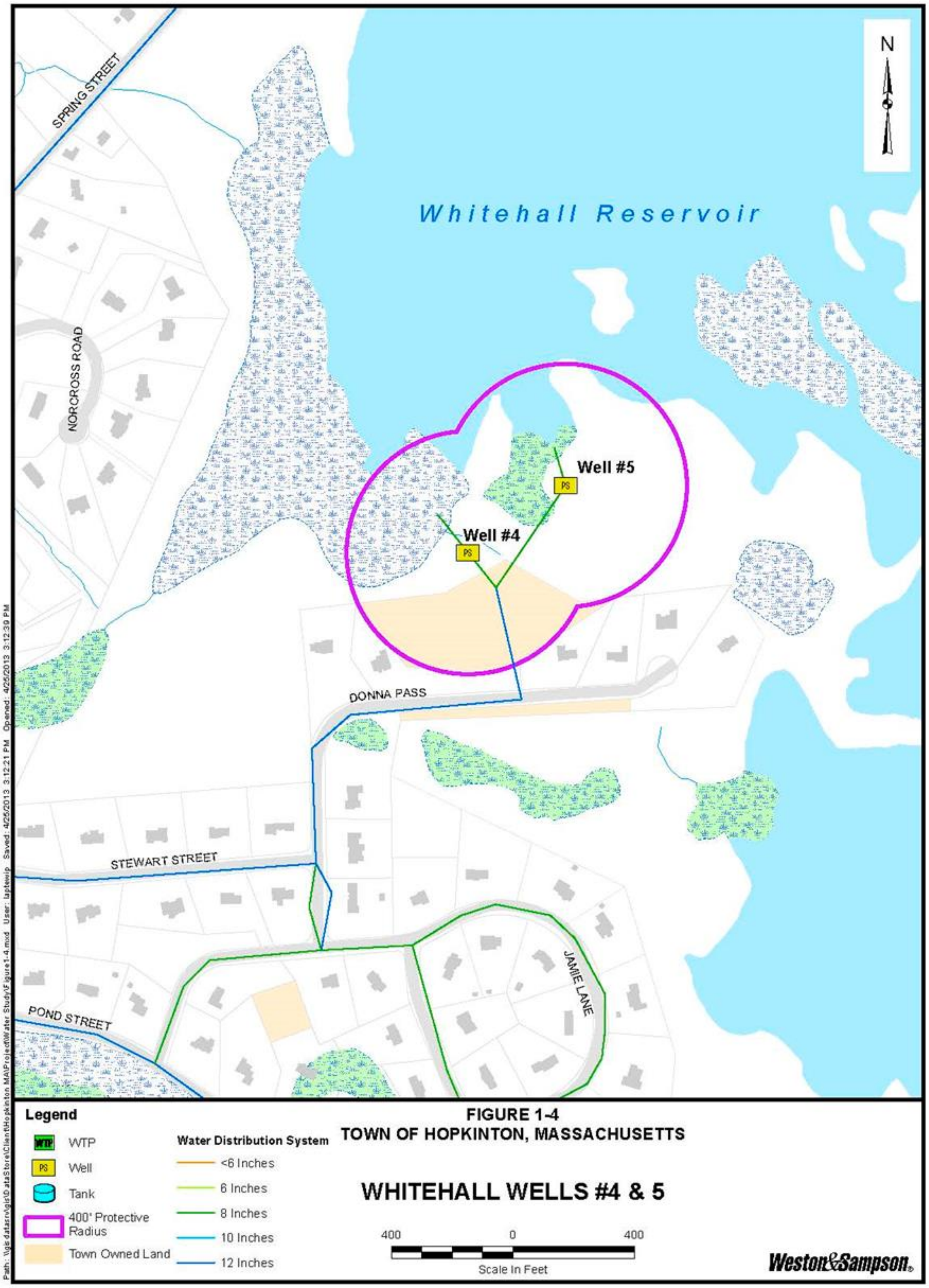


FIGURE 1-4
TOWN OF HOPKINTON, MASSACHUSETTS

WHITEHALL WELLS #4 & 5

well water. The water is treated for pH adjustment, corrosion control and the sequestering of iron and manganese using sodium silicate and for disinfection using sodium hypochlorite.

1.2.2.2 Well No. 5

Well No. 5 is a gravel packed well with a depth of 44 feet, 24-inch diameter casing and 5 feet of 24-inch diameter, 160 slot stainless steel screen. The Well No. 5 pump is a vertical turbine pump with a design flow of 325 gpm at 300 feet of TDH. The pump is equipped with a 40 HP premium efficiency motor manufactured by US Motor. Based on the 2012 MassDEP Annual Statistical Report, Well No. 5 was only operated for a total of 7 days during the months of June and July. Due to the poor water quality with elevated levels of iron and manganese, the well is used to supplement the other Town's sources during periods with high water demands and is activated after Well No. 4 in the Town's sequence of operation.

The Well No. 5 Pump Station is equipped with a propane unit heater for temperature control from a propane tank located within the surrounding fenced area. A Ford auxiliary engine, Model 11423 C-18-TT, and a right angle gear manufactured by Johnson provide emergency power for the pump at Well No. 5. The pump station is used to house the sodium silicate and sodium hypochlorite chemical feed systems for treatment of the well water. The water is treated for pH adjustment, corrosion control and the sequestering of iron and manganese using sodium silicate and for disinfection using sodium hypochlorite.

1.2.3 <u>Alprilla Farm Wells</u>

The Alprilla Farm Wells No. 7 and 8 were constructed in 2012 and are located off Alprilla Farm Road (Figure 1-5). The Town owns the land in the Zone 1. Each well has its own separate building where the pump, motor, and associated equipment are located. The water from Well No. 7 is pumped into the Well No. 8 pump station where it is combined with the Well No. 8 pump discharge, treated, and discharged to the water distribution system.

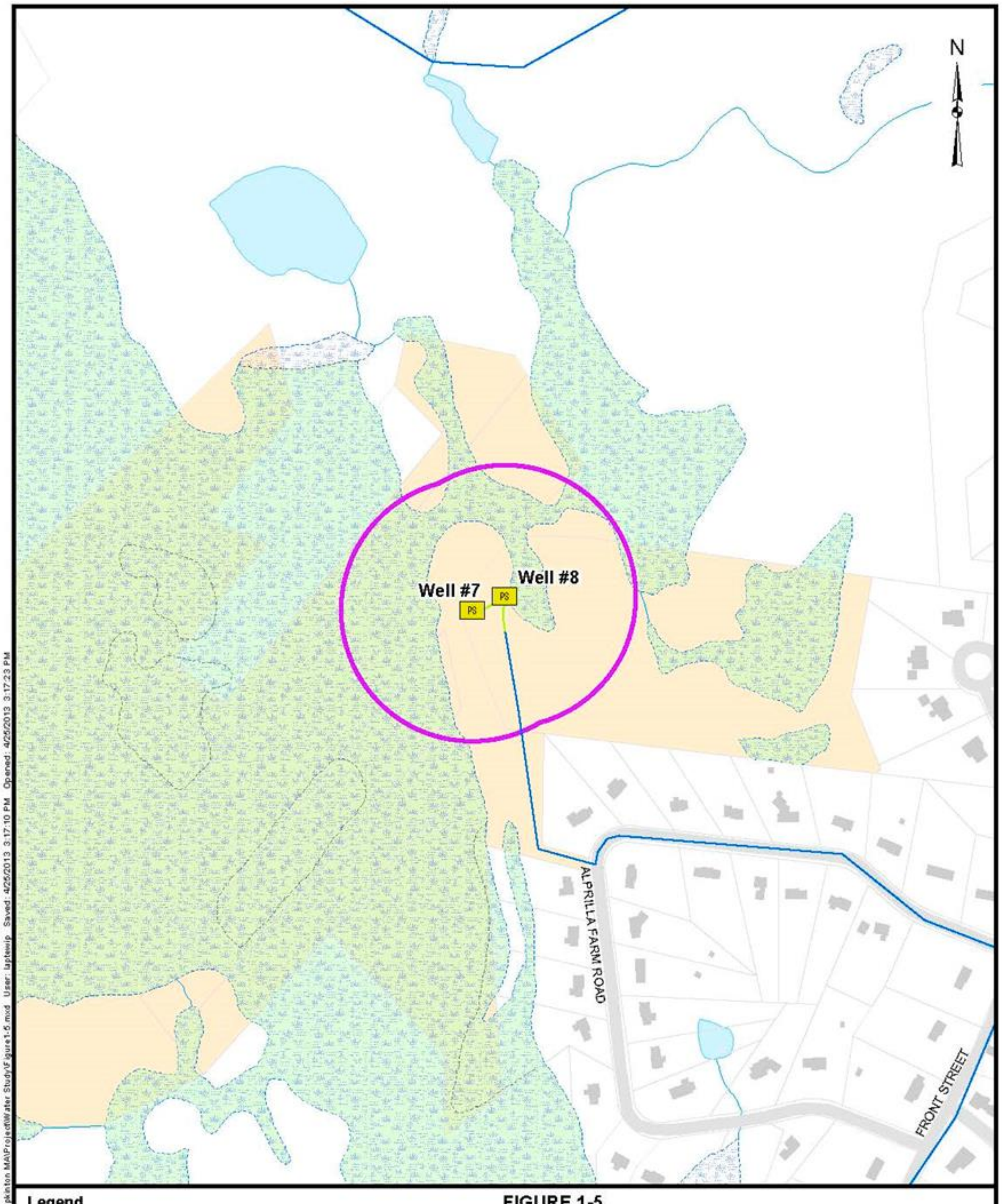


FIGURE 1-5
TOWN OF HOPKINTON, MASSACHUSETTS

ALPRILLA WELLS #7 & 8

1.2.3.1 Well No. 7

Well No. 7 is a gravel packed well with a depth of 55.5 feet, 18-inch diameter casing and 5 feet of 18-inch diameter, 180 slot stainless steel screen. The Well No. 7 pump is a vertical turbine pump with 6-inch diameter pump column and a design flow of 200 gpm at 370 feet of TDH. The pump is equipped with a 25 HP premium efficiency motor manufactured by US Motor. The pump house is equipped with an electric unit heater and an exhaust fan for temperature control. A natural gas generator located in the Well No. 8 Pump Station is used for emergency power at Well No. 7. The water from Well No. 7 is treated for pH adjustment and corrosion control with sodium hydroxide and for disinfection using sodium hypochlorite after it is combined with the pump discharge from Well No. 8.

1.2.3.2 Well No. 8

Well No. 8 is a gravel packed well with a depth of 46.8 feet, 18-inch diameter casing and 5 feet of 18-inch diameter, 200 slot stainless steel screen. The Well No. 8 pump is a vertical turbine pump with 4-inch diameter pump column and a design flow of 100 gpm at 360 feet of TDH. The pump is equipped with a 15 HP premium efficiency motor manufactured by US Motor. The pump house is equipped with a gas fired unit heater and an exhaust fan for temperature control. A natural gas generator located in pump station is used for emergency power. The pump discharge from Well No. 8 is combined with the Well No. 7 pump discharge in the Well No. 8 Pump Station and treated for pH adjustment and corrosion control with sodium hydroxide and for disinfection using sodium hypochlorite.

1.2.4 Town of Ashland Inter-Municipal Agreement

Due to lack of water supply in Hopkinton, the Town decided to connect to the Ashland Water Treatment Plant (WTP) in the spring of 2002. The agreement between the two Towns requires Hopkinton to purchase a minimum annual average of 0.3 million gallons per day (MGD), and the Water Management Act (WMA) permit sets the authorized maximum annual average to 0.5 MGD. The agreement permits Hopkinton to purchase a maximum one-day volume of 1.0 MGD. Town water operators have indicated that demands in Ashland during the peak summer months make it difficult to pump 1.0 MGD and that a maximum pumping rate of 0.8 MGD is more representative of what can be provided.

The WMA permit issued for the Town of Ashland includes the water sold to Hopkinton. The WMA permit for Ashland requires the Town to maintain a minimum water level in the reservoir of 3 feet below the spillway crest between June 1 and August 31 each year. If the water level in the reservoir drops below this minimum level, then the Ashland Wells No. 7 and No. 8 supplying the Ashland WTP are shut down, and the sale of water to Hopkinton will be banned until the reservoir level increases.

The Town of Hopkinton connected to the Ashland WTP by extending a 12-inch diameter water main on Wilson Street. The Ashland WTP has a 0.4 MG clearwell with two vertical turbine pumps, each with a capacity of 700 gpm, to supply Hopkinton's distribution system. One pump typically runs at a time with the second pump used as a spare unless there is a large demand requiring both pumps to be active. The pumps at the Ashland WTP are controlled at the Hopkinton Department of Public Works located at 85 Wood Street in Hopkinton.

Based on the 2012 MassDEP Annual Statistical Report, Hopkinton purchased approximately 148 MG of water from Ashland throughout the year. Water was pumped from the Ashland WTP into the Hopkinton distribution system every day throughout the year with an average day supply of 0.405 MGD and a maximum day supply of 0.7 MGD pumped on July 17, 2012.

TABLE 1-1
WATER SUPPLY WELLS

Well No.	WELL							WELL PUMP					Treatment	Auxiliary Power
	Well Name	Type	Year Installed	Well Depth (ft.)	Casing Diam. (ft.)	Screen Length x Diameter (ft. x in.)	Last Time Re-Developed	Year Installed	Horse Power (hp)	Total Dynamic Head (ft.)	Design Pump Capacity (gpm)	Safe Yield (MGD)		
1	Fruit Street	Gravel Packed	1958	33	18	10 x 18	2003		60	345	530	0.36	Sodium Hydroxide	Main Fruit Street Station generator
2	Fruit Street	Gravel Packed	1963	42.5	18	10 x 18	2009		60	350	500	0.27	Sodium Hypochlorite, Sodium Hydroxide	Main Fruit Street Station generator
3*	Fruit Street	Gravel Packed	1973	36.7	24	10 x 24		1976	25	312	150	0.12	None	Main Fruit Street Station generator
4	Whitehall	Gravel Packed	1987	38	24	5 x 24	2011	1986	40	300	250	0.36	Sodium Silicate, Sodium Hypochlorite	None
5	Whitehall	Gravel Packed	1987	44	24	5 x 24	2002	1986	40	300	325	0.47	Sodium Silicate, Sodium Hypochlorite	None
6	Fruit Street	Gravel Packed	2009	70	18		2009	2009	75	374	500	0.72	Potassium Hydroxide	Propane Generator
7	Alprilla	Gravel Packed	2012	55.5	18	5 x 18	2012	2012	25	370	200	0.28	Sodium Hypochlorite, Sodium Hydroxide	Natural Gas Generator
8	Alprilla	Gravel Packed	2012	46.8	18	5 x 18	2012	2012	25	360	100	0.14	Sodium Hypochlorite, Sodium Hydroxide	Natural Gas Generator

*Due to the reduced production, Well No. 3 has been reclassified by the MassDEP from an active to an emergency water source.

1.3 Water Distribution System

The Town of Hopkinton's distribution system includes approximately 62 miles of water mains installed throughout the Town. The Town's distribution system includes approximately 3,081 service connections based on the 2012 MassDEP ASR. Table 1-2 presents a summary of the water mains in the Town's water distribution system by pipe size.

The Town's GIS database was created utilizing the Town's hydraulic modeling database. The hydraulic model pre-dates 2003 when it was converted from an EPA Net model. The GIS and hydraulic model have the Hazen Williams C-values populated, but no information on installation date or material of construction. Based on the C-values in the GIS and model, we have identified the unlined water main in Table 1-3 (assuming that C-value less than 80 is unlined pipe).

TABLE 1-2
WATER DISTRIBUTION SYSTEM

Main Size (inches)	Linear Footage of Pipe (ft)	% of Distribution System
< 6	3,386	1%
6	28,299	9%
8	136,105	41%
10	29,959	9%
12	131492	40%
Total	**329,241**	**100%**

TABLE 1-3
DISTRIBUTION SYSTEM PIPE MATERIAL SUMMARY

Main Size (inches)	% Unlined*	% Cement Lined
< 6	0.7	0.4
6	4.1	4.5
8	4.1	37.2
10	4.6	4.5
12	0.0	39.9
Total	**13.4**	**86.6**

1.3.1 Water Storage Facilities

This section of the report provides a brief description of each of the three water storage tanks in the Hopkinton water system. This information was taken from the inspection reports completed by Haley & Ward Inc. Table 1-5 presents the Town's water storage information.

TABLE 1-4
DISTRIBUTION STORAGE FACILITIES

Location	Year Built	Diameter (ft)	Height (ft)	Capacity (MG)	Overflow Elev. (ft)[1]
Small Grove Street	1922	35	60	0.32	600
Large Grove Street	1965	65	61	1.5	600
West Main Street	1954	44	70	0.793	600

1. USGS mean sea level datum.

1.3.1.3 Grove Street Tanks

Small Grove Street Standpipe

The Small Grove Street Standpipe was constructed using riveted steel in 1922 by Hodge Boiler Works. The tank stands approximately 60-feet tall with a diameter of 35-feet and a capacity of 0.32 MG. The tank is located within a chain link fence in a highly populated area between Grove Street and the Hopkinton Middle School parking lot, situated adjacent to the Large Grove Street Standpipe. There is minimal space within the fenced in area, but space for parking and equipment is available outside of the enclosed area. The interior and exterior of the tank was last painted in 1995. The tank was last inspected by Haley & Ward Inc. on October 9, 2012.

According to the 2012 inspection report, the overflow screen is satisfactory for now but may need modification to meet the MassDEP Guidelines and Policies during the next rehabilitation program.

Large Grove Street Standpipe

The Large Grove Street Standpipe is welded steel constructed in 1965 by the Chicago Bridge and Iron Company (CB&I). The tank stands approximately 61-feet tall with a diameter of 65-feet and a capacity of 1.5 MG. The tank is located within a chain link fence in a highly populated area between the Small Grove Street Standpipe and the Hopkinton Middle School. There is minimal space within the fenced in area, but space for parking and equipment is available outside of the enclosed area. The top half of the interior and exterior of the tank was last painted in 1995. The bottom half of the tank was not painted due to the inability to drain the tank. The tank was last inspected by Haley & Ward Inc. on October 9, 2012. No sanitary deficiencies were identified in the report.

1.3.1.4 West Main Street Tank

The West Main Street Standpipe is welded steel constructed in 1954. The tank stands approximately 70-feet tall with a diameter of 44-feet and a capacity of 0.793 MG. The tank is located with a chain link fence at the intersection of West Main Street and School Street in a rural part of Town. There is minimal space inside the fenced area for equipment and vehicles, and limited space available to park along the street. Haley & Ward Inc. last inspected the tank in April of 2012. The interior and exterior of the tank was last painted in 2009.

1.3.2 Emergency Connections

The Town of Hopkinton does not have an emergency connection or pump station installed and available to supply or receive water from an adjacent Town (not including the Ashland connection). The Hopkinton distribution system runs along the Town line bordering the Town of Milford. The distribution system in the Town of Milford runs along the Town line as well. There is a Hopkinton hydrant and Milford hydrant in close proximity to each other that could be connected in an emergency situation. The hydraulic gradeline of the water system in the Town of Milford is approximately 40 feet higher than the hydraulic gradeline in Hopkinton of 600 feet. Based on the gradelines, water from the Milford system could be fed into the Hopkinton distribution system in an emergency situation through a connection between the adjacent hydrants. This connection would result in an increased pressure in Hopkinton of approximately 17 psi, which if this was a concern, the water could be fed through a pressure reducing valve.

2.0 WATER SUPPLY REQUIREMENTS

2.1 General

The purpose of this section is to estimate the amount of water that the Town requires to meet present and future water demands. To provide a conservative estimate of future needs, water supply requirements are estimated through a 20-year planning period or the year 2033. In order to accurately project future water supply requirements, it is necessary to analyze historical water production and consumption records. Projections for future water use are then calculated based upon the projected population to be served and per capita water usage. Water demand projections were developed using the methodology detailed in Water Resources Commission's, "Policy for Developing Water Needs Forecasts for Public Water Suppliers and Communities and Methodology for Implementation."

2.2 Population

Population data for the Town of Hopkinton as reported by the U.S. Census Bureau is shown in Table 2-1 and graphically depicted on Figure 2-1.

TABLE 2-1
U.S. CENSUS HISTORICAL POPULATION DATA

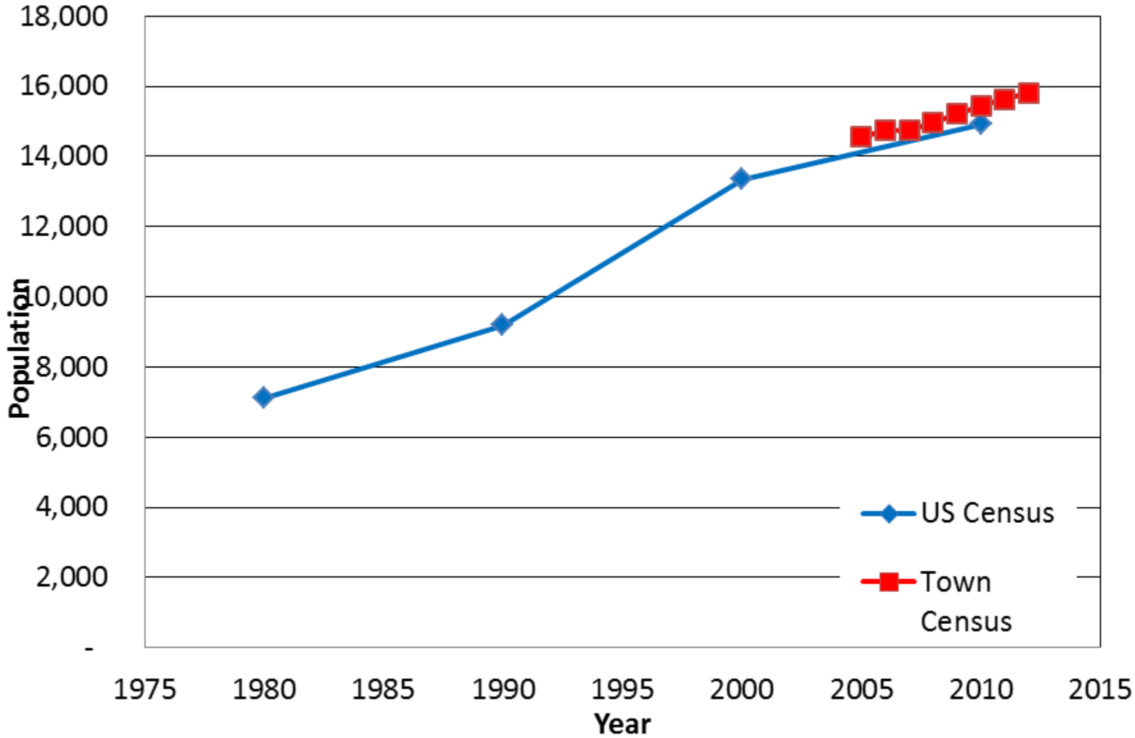
Year	Population
1980	7,114
1990	9,191
2000	13,346
2010	14,925

The estimated total population for Hopkinton from the year 2009 through 2012 based on an internal Town census is presented in Table 2-2 and Figure 2-1. The population of Hopkinton grew by approximately 1.5 percent between 2009 and 2010, 1.2 percent between 2010 and 2011, and 1.2 percent between 2011 and 2012. The Town's entire population is not served by the water distribution system. Each year the Town of Hopkinton completes the MassDEP annual statistical report (ASR) in which the population served by the water distribution system is estimated based on the number of residential water services and the average number of people per service. The average capita per service connection is approximately 2.99 people based on the 2010 U.S. Census. The estimated population served by the water distribution system as reported in the MassDEP ASR is shown in Table 2-2 and is calculated by multiplying the number of service connections by the household census data.

TABLE 2-2
TOWN OF HOPKINTON ESTIMATED POPULATION DATA

Year	Town Population	Population Served by Distribution System	% of Population Served
2009	15,216	8,409	55.3
2010	15,448	8,559	55.4
2011	15,630	8,610	55.1
2012	15,812	8,901	56.3

FIGURE 2-1
HISTORICAL POPULATION DATA



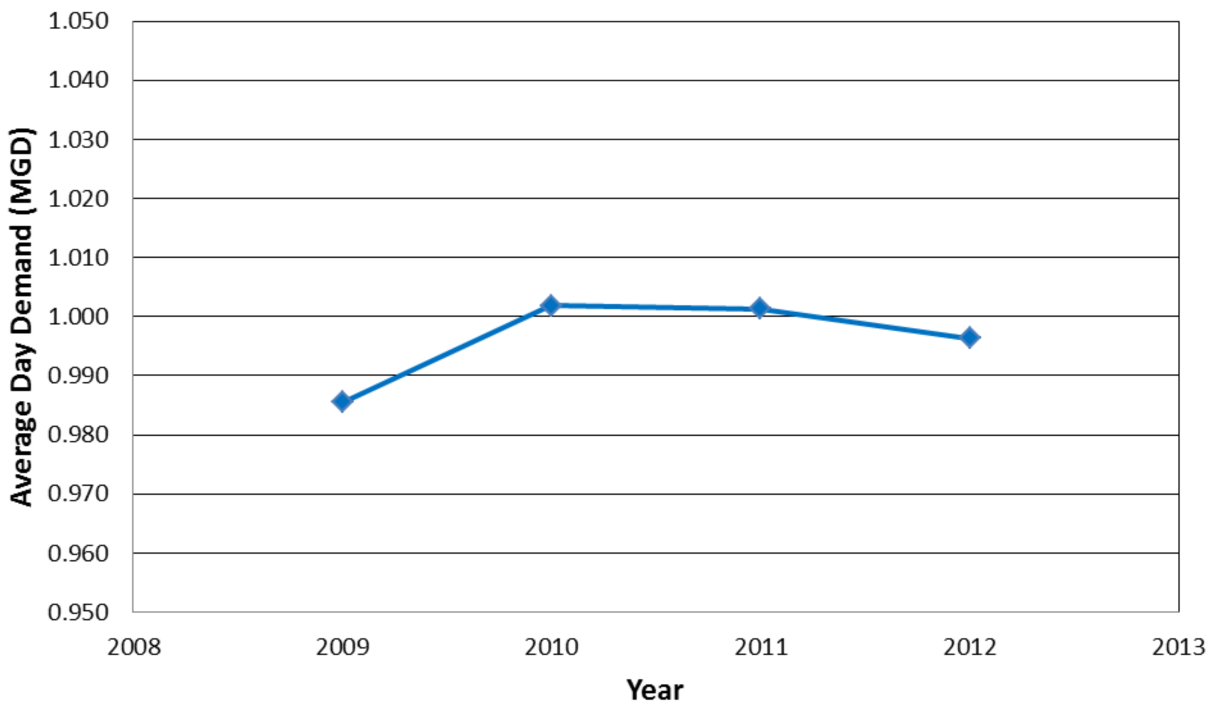
2.3 System Demands

Water production data for the Town's water system for the years 2009 through 2012 was taken from the MassDEP ASRs. This data was analyzed to determine trends in the annual water production for the Town and is presented in Table 2-3 and graphically shown in Figure 2-2.

TABLE 2-3
AVERAGE DAILY WATER PRODUCTION

Month	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	Annual Average
2009 (MGD)	0.92	0.95	0.94	0.97	1.09	1.08	1.02	1.16	1.09	0.91	0.87	0.84	**0.99**
2010 (MGD)	0.82	0.82	0.84	0.90	1.06	1.17	1.35	1.20	1.11	0.95	0.89	0.90	**1.00**
2011 (MGD)	0.89	0.87	0.87	0.92	1.09	1.23	1.30	1.13	1.03	0.92	0.88	0.88	**1.00**
2012 (MGD)	0.87	0.87	0.86	0.97	1.06	1.15	1.27	1.14	1.05	0.93	0.91	0.86	**1.00**

FIGURE 2-2
ANNUAL AVERAGE DAY DEMAND



In the MassDEP ASR, the system demand is broken down by demographic type (residential, commercial, agricultural, industrial, municipal, institutional, and other), estimated usage within the Town for municipal purposes (street cleaning, hydrant flushing, etc.), and the amount of unaccounted-for water (UAW). Based on review of the data, the average daily water demand has been consistent over the past four years ranging from 0.99 MGD to 1.00 MGD. Water use restrictions enforced by the Town each summer have helped to manage and limit water usage/demands.

Table 2-4 shows the MassDEP ASR reported maximum day demand for the Town. The maximum day demand occurred in August in 2009, in July in 2010, and in June in both 2011 and 2012. The average ratio from maximum to average day demands is 1.63. We utilized this demand ratio of 1.63 to project future maximum day demands.

TABLE 2-4
MAXIMUM DAILY WATER PRODUCTION

Year	Average Day Demand (MGD)	Maximum Day Demand (MGD)	Ratio
2009	0.99	1.35	1.37
2010	1.00	1.72	1.72
2011	1.00	1.73	1.73
2012	1.00	1.70	1.71
Average	**1.00**	**1.63**	**1.63**

The peak hour demands are the highest hourly demands that occur during a 24-hour period and generally occur in conjunction with the maximum day demand. Because peak hour demands can vary anywhere from 1.0 to 3.0 times the maximum day demands, and are short-term demands, they can and should be met from distribution storage rather than from supply facilities. The exact peak hour water system demand is not known for Hopkinton. Towns in Massachusetts with development similar to Hopkinton generally experience a peak factor of 1.5 to 2.0 times maximum day demand. This results in a peak hour factor of approximately 2.5 times the average day demand.

2.4 Future Population Projections

Future average and maximum day demands are projected based upon the projected residential population, expected trends in the per-capita water use, and the projected commercial and industrial growth of the Town. Hopkinton is expected to continue to grow in terms of population as multiple residential and commercial developments are under construction, approved, and/or planned over the 20-year planning period. The Town's list of residential and commercial projects under construction and planned over the 20-year planning period is shown in Table 2-5. The Town has listed projected water demands for these developments which will be discussed later in this chapter. We have compiled population projections for the Town of Hopkinton over the 20-year planning period of this report, through the year 2033.

The Metropolitan Area Planning Council (MAPC) has developed population projections for the Town of Hopkinton. The MAPC projections have not been updated recently to reflect the actual current population in the Town so the population numbers listed by the MAPC are lower than the Town census. In developing our population projections, we have evaluated the MAPC projections for growth rates and not actual population values and used the trending to project future population.

The MAPC has two separate projections that focus on different factors concerning population, economic, and commercial growth. The first projections known as "Current Tends" were developed in 2005 and focus solely on Hopkinton and the trends that can be generated from the 1970 to 2000 census data. "Current Trends" are derived from the historical growth and continuation of development, population, and economic trends. The "Current Trends" do not take into consideration the recent population growth after 2005 or the developments recently completed or under construction in Town as shown in Table 2-5. As a result of the changes in population growth in Hopkinton since the projections were developed, the "Current Trends" MAPC projections underestimate the future population for the Town.

TABLE 2-5
TOWN OF HOPKINTON APPROVED AND PLANNED DEVELOPMENTS

	Retail	Flow (gpd)	Office	Flow (gpd)	Fitness	Flow (gpd)	Day Care	Flow (gpd)	Assisted Living	Flow (gpd)	Medical	Flow (gpd)	Industrial	Flow (gpd)	Rental Units	Bedrooms	Flow (gpd)	Residential	Bedrooms /Unit	Total Bedrooms	Flow (gpd)	Total Flow (gpd)
Recently Constructed/Under Construction																						
Farm Stand	6,400	320	1,000	75																		454
Angels Garden	432	22																				25
Mayhew Court															12	29	22,620					22,620
Legacy Farms[1]																						170,000
Hopkinton Square	55,000	2,750	15,000	1,125														9	3	27	1,755	6,211
Highland Park Subdivision																		24	4	96	6,240	6,240
Connely Hill Subdivision																		20	4	80	5,200	5,200
Peppercorn Village (40B)[2]																		11		44	2,860	8,555
Sanctuary Lane (40B)[2]																		13		52	3,380	6,950
Stagecoach Heights (40B)[2]																		32		128	8,320	9,900
Golden Pond Resident Care[3]							75	600			8,000	2,000			1	58	3,770					6,760
Bridle Path Subdivision																		6	4	24	1,560	1,560
																					Subtotal Recently Constructed Projects	**244,475**
Approved/Pending Projects																						
Hopkinton Village Center	14,000	700	15,000	1,125																		1,825
81-83 Main - Office Building			6,020	452																		519
Maspenock Woods																		30				6,820
Elmwood III Subdivision																		24	4	96	6,240	6,240
78 West Main - Dunkin Donuts	3,000	150																				173
Hayden Woods, 215 Hayden Rowe																		18	2	36	2,340	2,340
Hunter's Ridge																		19	4	76	4,940	4,940
																					Subtotal Approved/Pending Projects	**22,857**
Planned Projects																						
42 Main St.	6,442	322	6,442	483																		805
Crossroads Project															375	2	48,750					48,750
151 Hayden Rowe Subdivision																		15	4	60	3,900	3,900
Hayden Rowe 40B																		16	4	64	4,160	4,160
149 Hayden Rowe 40B																		20	4	80	5,200	5,200
Drugstore (West Main/Lumber)			10,000	750																		750
Capital Group - Cedar Street													25,000	1,250								1,250
Governor's Crossing - Wilson St.																		26	2	52	3,380	3,380
Medical - 77 Main street[3]											40,000	10,000										10,000
Hayden Woods, 215 Hayden Rowe St.															18	2	2,340					2,340
Hopkinton Mews, Lumber St. 40B															250	1.8	29,835					29,835
Reservoir View, Spring St.																		3	4	12	780	780
Christian Estates, Pond St.																		3	4	12	780	780
Golden Pond phase 3									30,000	1,500												1,725
Leonard St. subdivision																		7	4	28	1,820	1,820
203 Pond St. subdivision																		12	4	48	3,120	3,120
Lumber St./West Main St. (Mastroianni)	40,000	2,000			70,000	2,500																4,500
																					Subtotal Planned Projects	**123,095**
																					Assume 50% of Planned Projects	**61,548**

1. Legacy Farms Water Use based on total projected water demand of 170,000gpd
2. Total flow is based on permitted volume, not initial project demand
3. Medical office building estimated 1,000sf/doctor
4. 20-year system expansion based on 10 new service connections per year at 4 persons per connection at 65gpdpc
5. Total Flow for all non-residential use is based on Title V

Subtotal Planned/Approved/Recently Constructed Projects **328,880**

20-yr Potential Existing System Expansion (gpd)[4] **52,000**

Total Estimated Additional Average Day Demand (gpd) **380,880**

Current Permit 980,000

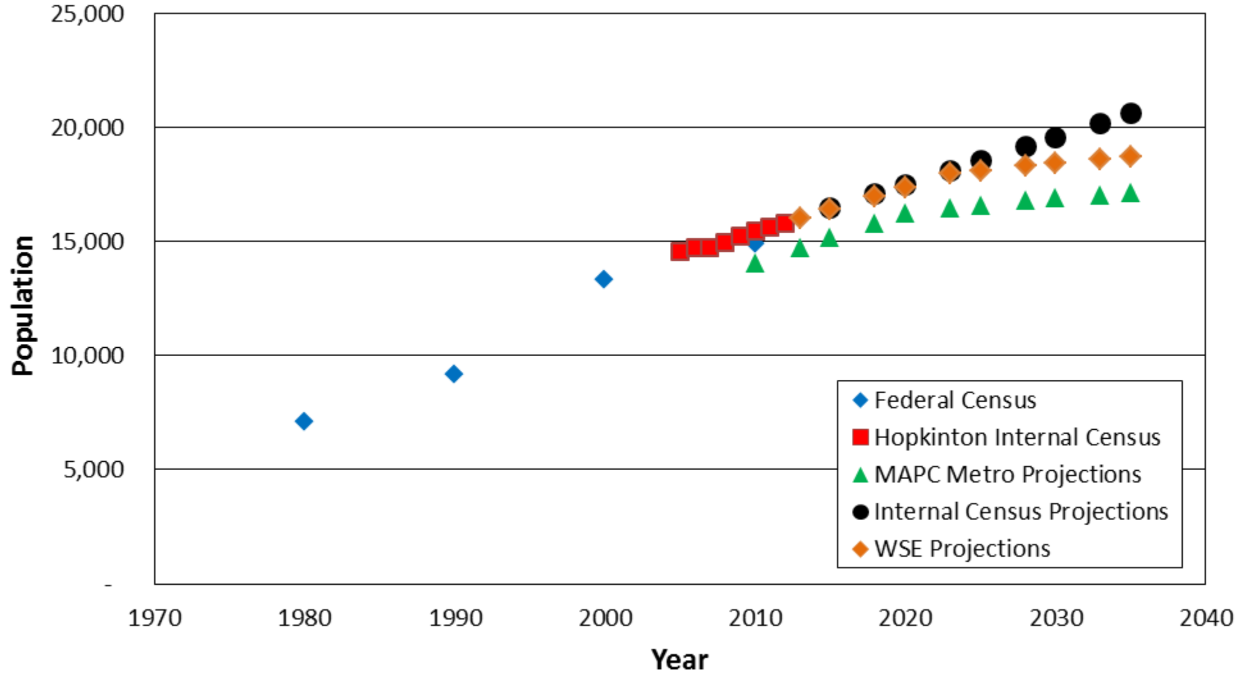
The second projections are developed from “Metro Trends”, which take the region as a whole into consideration. The “Metro Trends” are derived based on the plan of the region for sustainable and equitable growth as well as preservation. These trends take the Massachusetts Department of Transportation’s (MassDOT) transportation expansion plan into consideration in making the projections. The “Metro Trends” were developed in 2007 to project population through 2030 and were revised in 2011 to extend the projections through 2035. However, when the projections were revised, the starting population was not increased to reflect the actual current population in Hopkinton at the start of the projections.

The “Metro Trends” projections are more aggressive than the “Current Trends” and more reflective of the growth rate we expect to see in Hopkinton. However, the “Metro Trends” over the first ten years of the 20-year planning period may still underestimate the future population, when compared to local population census, due to the incorrect starting population and several developments, including Legacy Farms, under construction and planned for the Town as shown in Table 2-5. Projected construction and potential residential occupancy for part of the Legacy Farms development planned over the next three years includes 240 apartment units with an average of 1.5 bedrooms per unit and 275 units with an average of 2.5 bedrooms per unit. Beyond the first ten years of the planning period, we expect the population growth to follow a trend similar to that developed by the MAPC. The “Current Trends” and “Metro Trends” MAPC projections are presented in Table 2-6, and the “Metro Trends” are illustrated in Figure 2-3. The “Current Trends” are not shown in Figure 2-3 since these projections underestimate the future population and do not factor into the Weston & Sampson projections.

TABLE 2-6
MAPC POPULATION PROJECTIONS

Year	Population (Current Trends)	Population (Metro Trends)
2013	14,486	14,728
2018	14,857	15,817
2023	15,184	16,453
2028	15,483	16,786
2033	16,536	17,063

FIGURE 2-3
HISTORICAL POPULATION AND POPULATION PROJECTIONS



Both population projections completed by the MAPC show an increase in the population over the 20-year planning period, but the growth rate included in the projections is less than the rate experienced by the Town over the last four years. We developed population projections for the 20-year planning period, through 2033, based on the Town's internal census and population growth trend observed between 2009 and 2012. The projected growth rate for the first ten years of the 20-year planning period was estimated to be the same as the average growth rate observed between 2009 and 2012.

The population growth for the second ten years of the 20-year planning period was projected to slow down since the developments under construction and planned for construction in the near future will be occupied within the first ten years. The population growth for the second ten years was projected to match the increase shown by the "Metro Trends" of the MAPC projections. In summary, we expect Hopkinton will observe an increase in population by approximately 2,700 people over the next 20-year planning period. The projected population was similar to the expected population growth determined by the Town's build-out plan shown in Table 2-5. Weston & Sampson's population projections are shown in Table 2-7 and Figure 2-3.

TABLE 2-7
WESTON & SAMPSON POPULATION PROJECTIONS

Year	Population
2013	16,019
2018	17,004
2023	17,989
2028	18,322
2033	18,599

Based on the 2004 Water Assets Study prepared for the Massachusetts Executive Office of Environmental Affairs, the Town of Hopkinton population build-out based on available land that can still be developed is estimated to be 25,945.

2.5 Per Capita Water Use

The population projections and the anticipated per capita water use were used to prepare water demand projections. Table 2-8 shows the average daily per capita demand for the Town from the years 2009 to 2012. The per capita demand is calculated based on the residential average day demand and the estimated population of the Town that is served by the water distribution system (residential average day demand divided by the population served). According to the MassDEP ASR, the population of the Town served by the water distribution system is estimated from the number of households connected to the distribution system as determined by Town records and the average household size for the Town of 2.99 people as listed in the 2010 U.S. Census.

The average residential daily per capita demand value based upon the 3-year average (2009 to 2011) was approximately 55.1 gpcd. According to the standards and conditions set forth by the Massachusetts Water Resources Commission (WRC) in the *Water Conservation Standards* (Updated June 2012), a public water supply (PWS) shall meet or demonstrate steady progress toward meeting a residential gallons per capita day (gpcd) water use performance standard of 65 gpcd, especially in those communities in a basin with a higher level of stress classification. Hopkinton's water supplies are located within the Sudbury Assabet Concord (SuAsCo) Basin and more specifically the Concord River Basin. The Concord River Basin is a medium stressed basin increasing the need for the Town to meet the residential water use standards set by the Massachusetts WRC.

TABLE 2-8
AVERAGE PER CAPITA WATER DEMAND

Year	Town Census Population	Population Served	Average Day Demand (MGD)	Residential Demand (MGD)	Daily Per Capita Residential Demand (gpcd)	Commercial/ Industrial Demand (MGD)
2009	15,216	8,409	0.986	0.451	53.6	0.365
2010	15,448	8,559	1.002	0.493	57.6	0.291
2011	15,630	8,610	1.001	0.467	54.2	0.243
2012	15,812	8,901	0.996	0.490	63.3	0.302
3-Yr Average (2009-2011)	**15,527**	**8,690**	**0.996**	**0.475**	**55.1**	**0.300**

Hopkinton's residential per capita day water use over the past four years has been below the guidelines established by the Massachusetts WRC. Hopkinton implements seasonal water use restrictions from May 1st through September 30th to limit non-essential outdoor water use in the summer which has resulted in the Town consistently maintaining a residential gallons per capita day (RGPCD) less than the performance standard of 65 gallons. With the proposed developments planned and under construction in Hopkinton, the Town may observe an increase in the residential water use and RGPCD. For this reason, we developed two scenarios to project future residential water demands for the Town. We utilized a conservative residential per capita day water use of 65 gpcd in one scenario and the Town's 3-year average RGPCD, 55.1 gpcd, in the second scenario. At the time of the projections, the 2012 data was not available.

2.6 Un-Accounted For Water Use

The difference between the water supply pumping data and the sum of the residential, commercial and municipal demands (less known losses) is categorized as unaccounted-for water (UAW). UAW is caused by leaks in broken water mains and services, old meters not registering correctly, unauthorized hydrant openings, illegal connections, standpipe overflows, and data processing errors. Table 2-9 lists the UAW in Hopkinton from 2009 to 2012. According to the standards and conditions set forth by the Massachusetts Water Resources Commission (WRC) in the *Water Conservation Standards* (Updated June 2012), the UAW performance standard is ten (10) percent of the average day demand.

TABLE 2-9
HISTORICAL UNACCOUNTED-FOR WATER

Year	Unaccounted For Water (MGD)	Percentage of Unaccounted For Water (%)
2009	0.170	17.21
2010	0.218	21.74
2011	0.291	29.06
2012	0.204	20.49
Average	**0.221**	**22.12**

Hopkinton's UAW exceeded the ten (10) percent performance standard in each year between 2009 and 2012. Hopkinton has completed master meter calibrations, leak detection surveys on the distribution system, and customer water meter replacements in an attempt to reduce its UAW. When the leak detection survey was conducted in 2009, the UAW was reduced to nearly 17 percent. Leak detection surveys were not conducted in 2010 and 2011, and the UAW increased again to over 29 percent in 2011. When a leak detection survey was completed in 2012, the UAW was reduced to approximately 20 percent. In order to project future water demands for the Town of Hopkinton we assumed a UAW of 20 percent through the year 2015 and that this would be reduced to 15 percent for all years thereafter.

2.7 Water Demand Projections

In order to project future (2033) average day water demands, we look at the historical population, historical water use, per capita demands, projected populations, and the projected population served by the water distribution system.

The population served by the water distribution system has remained relatively constant for the past four years at approximately 55 percent. As previously mentioned, the population of the Town served by the water distribution system is estimated each year in the MassDEP ASR from the number of households connected to the distribution system and the average household size for the Town, 2.99 people. We have observed in other communities where the water system does not serve the entire Town, that as new large developments are constructed and the water system is extended to serve that development, that over time, existing houses will tend to connect to the Town water system for water service. We anticipate that this type of growth will occur in Hopkinton over the 20-year planning period.

Based on the historical percentage of the Town's population served by the water distribution system (Table 2-2), and in order to account for future water system connections from existing houses currently served by wells, we project that 68 percent of the population will be served by the water system by the year 2033 (an increase of 0.65 percent per year). This projection assumes that all population increases will be served by the distribution system and approximately 10 services per year (serving 4 people per service for a total of 40 people per year) will be installed to connect existing residents onto the Town's water distribution system.

Between 2009 and 2012, the water demand of commercial and industrial accounts has comprised between 24 to 37 percent of the total average day water demand (average of 30 percent). In accordance with the WRC's, "*Policy for Developing Water Needs Forecasts for Public Water Suppliers and Communities and Methodology for Implementation*," the MAPC employment projections for the Town of Hopkinton were used to project the future commercial and industrial demands. The MAPC employment projections for the Town of Hopkinton estimate an increase in the number of employees by 0.6 percent per year and 12 percent over the 20-year planning period. The commercial/industrial water demand was projected to increase in demand by 12 percent over the 20-year planning period.

Table 2-10 shows Weston & Sampson's average day water demand projections for the 20-year planning period through the year 2033 for Scenario 1. As previously identified, the projected residential average day demand is estimated from the projected population served and the Massachusetts WRC performance standard of 65 gpcd. The projections were developed using the methods outlined by the WRC's, "*Policy for Developing Water Needs Forecasts for Public Water Suppliers and Communities and Methodology for Implementation*," which is included in Appendix C. The commercial/industrial water demand was projected to increase by 12 percent over the 20-year planning period, and the UAW was estimated at 20 percent through 2015 and 15 percent between 2016 and 2033.

TABLE 2-10
AVERAGE DAY WATER DEMAND PROJECTIONS SUMMARY (65 RGPCD)

Date	W&S Population Projections	% of Distribution System Projected to be Served	Projected Population to be Served	Projected Residential Avg. Day Demand (MGD)	Projected Comm./Ind. Demand (MGD)	Projected Unaccounted-For Water Demand (MGD)
2013	16,019	57%	9,152	0.595	0.304	0.225
2018	17,004	61%	10,337	0.672	0.314	0.174
2023	17,989	64%	11,522	0.749	0.323	0.189
2028	18,322	66%	12,055	0.784	0.332	0.197
2033	18,599	67%	12,532	0.815	0.341	0.204

Table 2-11 shows Weston & Sampson's average day water demand projections for the 20-year planning period through the year 2033 for Scenario 2. In this scenario, the projected residential average day demand is estimated from the projected population served and the Town's 4-year average RGPCD of 55.1 gpcd. As we assumed for Scenario 1, the commercial/industrial water demand was projected to increase by 12 percent over the 20-year planning period, and the UAW was estimated at 20 percent through 2015 and 15 percent between 2016 and 2033.

TABLE 2-11
AVERAGE DAY WATER DEMAND PROJECTIONS SUMMARY (55.1 RGPCD)

Date	W&S Population Projections	% of Distribution System Projected to be Served	Projected Population to be Served	Projected Residential Avg. Day Demand (MGD)	Projected Comm./Ind. Demand (MGD)	Projected Unaccounted-For Water Demand (MGD)
2013	16,019	57%	9,152	0.468	0.304	0.193
2018	17,004	61%	10,337	0.528	0.314	0.149
2023	17,989	64%	11,522	0.589	0.323	0.161
2028	18,322	66%	12,055	0.616	0.332	0.167
2033	18,599	67%	12,532	0.640	0.341	0.173

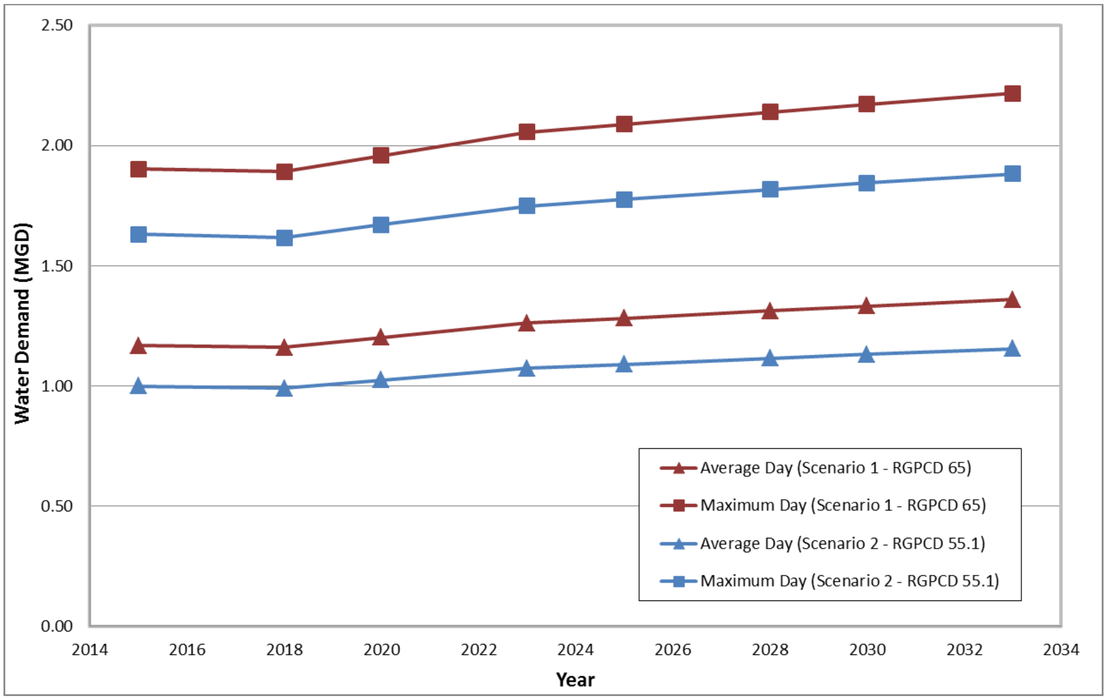
Table 2-12 and Figure 2-4 show the future average day and maximum day demand projections for the Town through the year 2033 for Scenario 1 and 2. The historical average ratio of maximum day to average day demand for the Town of 1.63 was used to estimate the future maximum day demands.

For the purposes of determining the magnitude of the improvements necessary to the water system, the projected demands will be used. This information will be used to recommend improvements to the water system. Figure 2-4 shows the water demands increasing slightly over the next 20-years. However, it is possible that the Town could experience additional commercial/industrial water users over the next 20-years that could increase these projections. It is therefore important to review demands and projections for large proposed developments regularly.

TABLE 2-12
FUTURE WATER DEMAND PROJECTIONS

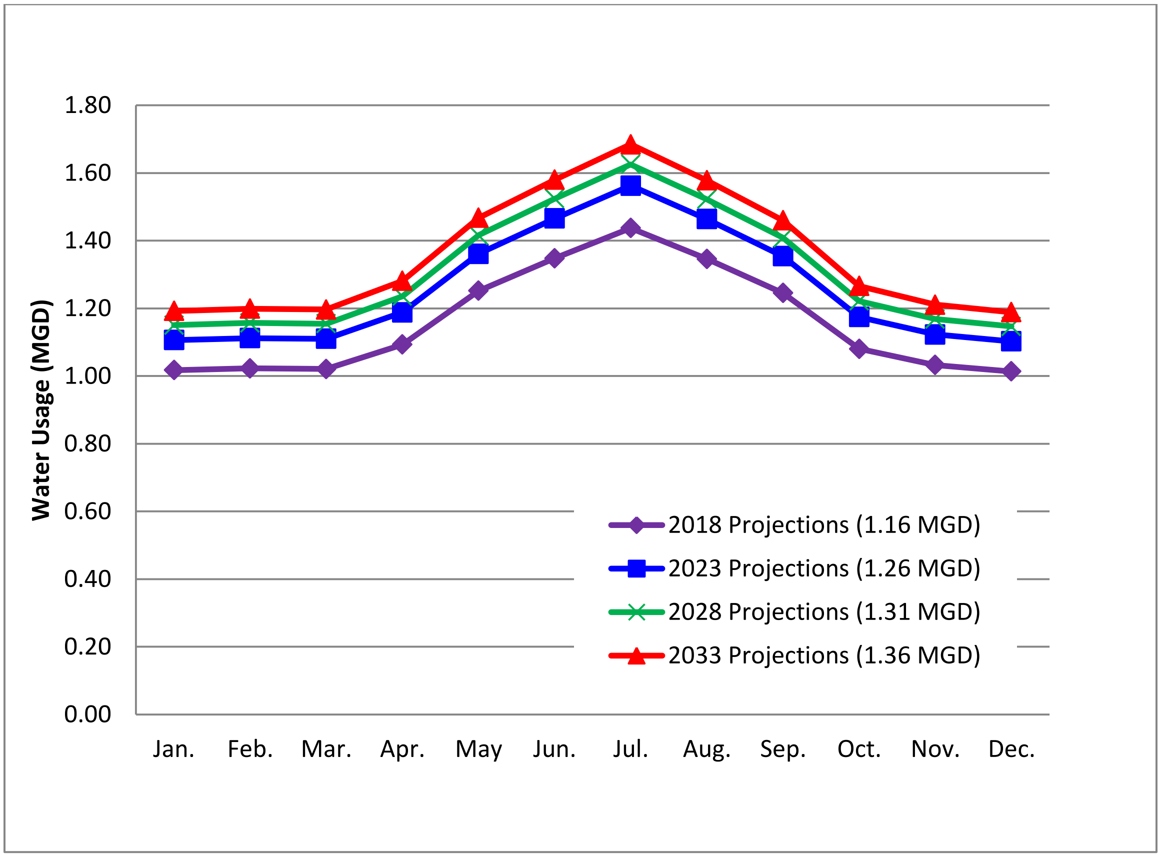
Year	Scenario 1 (65 RGPCD)		Scenario 2 (55.1 RGPCD)	
	Average Day Demand (MGD)	Maximum Day Demand (MGD)	Average Day Demand (MGD)	Maximum Day Demand (MGD)
2013	1.12	1.83	0.96	1.57
2018	1.16	1.89	0.99	1.62
2023	1.26	2.06	1.07	1.75
2028	1.31	2.14	1.11	1.82
2033	1.36	2.22	1.15	1.88

FIGURE 2-4
WATER DEMAND PROJECTIONS



As you can imagine and will notice as you continue through this report, that the projected water demands, significantly impact the water supply alternatives. It is important to understand how the ground water supplies in Hopkinton serve water demands. There is very little water storage in a ground water supplied system, and in fact, water supply is generally at its lowest, when demands are at their highest here in New England as is the case in Hopkinton. Figure 2-5 demonstrates the typical bell curve of projected average monthly water use in Hopkinton for the projected water demands. As Hopkinton is primarily served by ground water sources, the pumping capacity of the wells needs to be sized to serve the maximum summer day demand. This figure demonstrates that there is actually extra water capacity in the wells during the winter months when the demands are lower.

FIGURE 2-5
AVERAGE MONTHLY REPORTED IN MGD WATER USE PROJECTIONS – SCENARIO 1



3.0 SUSTAINABLE WATER MANAGEMENT

3.1 General

The process of permitting new ground water sources and of permitting increased withdrawals from existing ground water sources in Massachusetts is currently undergoing a significant transformation. Through the Sustainable Water Management Initiative (SWMI), the State has sought to develop new policy and permitting requirements that comprise a comprehensive approach to balancing water supply needs with the environmental sustainability of freshwater rivers and streams. Since 2010, the Massachusetts Executive Office of Energy and Environmental Affairs (EEA) has developed a permitting framework ("the SWMI framework"), testing and revising it through the course of a 5-community pilot program and two additional grant programs, one of which Hopkinton participated in. The final draft of the SWMI framework, approved in late 2012, will guide the approval of new and increased ground water withdrawal through Water Management Act (WMA) permit applications beginning in 2014.

This new SWMI-based approach to WMA permitting will impact Hopkinton if and when the Town looks to apply for increased water withdrawals through their WMA permit. Due to the new SWMI framework, it is likely that all future water withdrawals will require some form of mitigation / impact reduction. These measures may take many forms, such as water demand reductions and water conservation programs, water loss reductions, recharge of wastewater, and/or recharge of stormwater, as was studied by the Town through a 2013 SWMI grant program. The new SWMI framework and its implications to Hopkinton are discussed in the following sections.

3.2 The SWMI Framework

Tasked with safeguarding the sustainable management of the State's water resources for both human and ecological needs, the SWMI framework evaluates potential increases in a community's water withdrawals, through three key elements, including: Baseline conditions, Safe Yield, and Seasonal Streamflow Criteria. Baseline conditions refer to greater of the 2005 withdrawal rate plus 5% or the 2003-2005 average withdrawal rate plus 5%. Applications for withdrawals in excess of the Baseline will trigger the new stringent SWMI-based permitting requirements. Safe Yield defines the maximum amount of water withdrawal that can be allowed in a basin during drought conditions, incorporating environmental protection factors and hydrologic factors. To complement Safe Yield, which establishes an upper limit on water withdrawals on an annual basis and on a major basin scale, the SWMI framework developed the Seasonal Streamflow Criteria to guide WMA permitting decisions over a seasonal time-frame and at sub-basin scales. Under SWMI, the State is divided into more than 1000 sub-basins, and each one is assigned a grade based on the quality of its aquatic habitat and the degree to which nearby ground water withdrawals alter August streamflow. Proposed ground water withdrawals in sub-basins with poor aquatic habitat or strongly altered streamflow regimes will trigger relatively more intensive minimization and mitigation requirements through the MWA permitting process.

To satisfy the goal of sustainable management of water resources for both human and ecological needs, the SWMI framework will be used to assess all future ground water withdrawal requests through WMA permits against Baseline conditions, Safe Yield, and Seasonal Streamflow Criteria. Based on those comparisons, the permittee will be placed into one of three permit review tiers of increasing stringency. The permitting review thresholds and requirements are described in great detail in the 2013 SWMI Framework, but are summarized briefly in Figure 3-1

In general, Tier 1 permittees are required to conduct demand management; Tier 2 permittees must minimize impacts associated with their proposed additional withdrawals; and Tier 3 permittees must minimize such impacts, but also mitigate the requested additional withdrawal by returning an equivalent volume back to the sub-basin or otherwise offsetting the hydrologic impact of the withdrawal.

FIGURE 3-1

SWMI FRAMEWORK

NOTE: All permits require Standard Permit Conditions for all surface and groundwater withdrawals. These include conditions such as 65 rgpcd, 10% UAW, outside water use restrictions, and standard conservation BMPs.

	PERMIT REVIEW TIERS	REVIEW THRESHOLDS	SPECIAL CONDITIONS: RESOURCE SPECIFIC CONDITIONS AND AGENCY CONSULTATION	SPECIAL CONDITIONS: SEASONAL GROUNDWATER WITHDRAWAL LEVELS 4 and 5
No Change in Groundwater Level[B] or Biological Category[C]	Tier 1	No additional withdrawal request above baseline	If a CFR is present in GWL 4/5, conduct a desktop pumping evaluation and consult with agencies to minimize impact of withdrawals on CFR	Overall Concept: **Minimize** existing impacts to the greatest extent feasible[A] I. Evaluate the following potential actions to develop a plan based on improvement and feasibility: 1) optimization of existing resources; 2) use of alternative sources, including sources available to meet seasonal needs; 3) interconnections with other communities or suppliers; 4) releases from surface water impoundments; 5) outdoor water restrictions tied to streamflow triggers; 6) implementation of reasonable conservation measures consistent with health and safety; 7) New England Water Works Assoc. BMP toolbox; 8) other measures that return water to the sub-basin or basin intended to improve flow. II. Implement the plan
	Tier 2	Additional withdrawal request above baseline	Consult with agencies if CFR is present or if in BC 1, 2, or 3 to evaluate and implement feasible mitigation[D], commensurate with the impact from the additional withdrawal to ensure that streamflow criteria are met	**Minimize** impacts by implementing Tier 1 Conditions. **Mitigate** impacts commensurate with impact from additional withdrawal[D], in consultation with agencies Demonstrate no feasible alternative source that is less environmentally harmful[E], if additional withdrawal is greater than 5% unimpacted August median flow

	PERMIT REVIEW TIERS	REVIEW THRESHOLDS	SPECIAL CONDITIONS
If Backsliding is Proposed	Tier 3	Additional withdrawal request above baseline, AND Seasonal Groundwater Withdrawal Level[B], and/or Biological Category change	All Groundwater Levels - Demonstrate no feasible alternative source that is less environmentally harmful[E]
			Groundwater Levels 4 and 5 - Tier 1 Conditions apply. **Mitigate** impacts commensurate with impact from additional withdrawal[D], in consultation with agencies
			In Natural Resource areas such as BC 1, 2 or 3, or CFR - Evaluate and implement feasible **mitigation**[D], commensurate with impact from additional withdrawal, based on consultation with agencies

A) In determining if an action is feasible, the following should be taken into consideration: level of improvement; costs; the purview that is under the authority of the permitee, and adaptive management

B) Groundwater Withdrawal Level is Seasonal Streamflow Criteria - see Table 3

C) Biological Categories - see Table 2

D) From Offsets/Mitigation Table - see Table 6

E) ".....source that is less environmentally harmful" is defined as a source that is not in a groundwater level 4 or 5, and with excess capacity where additional withdrawal would not result in backsliding to a more altered groundwater level (e.g., groundwater level 2 to groundwater level 3).

3.3 WMA Permitting of Future Withdrawals

Prior to 2012, the Town of Hopkinton was permitted through the Water Management Act to withdraw 0.99 million gallons per day. To accommodate the additional demand associated with the Legacy Farms development, the Town sought to increase their WMA permitted withdrawal rate. Given the fact that the Town currently exceeds the WMA limit of 10% unaccounted for water (UAW), the Town applied for and received an interim amendment to their WMA permit, increasing their withdrawal volume from 0.99 MGD to 1.21 MGD. The Town subsequently tested and installed two additional wells, Well Nos. 7 & 8, at the Alprilla Farms site, and Well No. 6 in the Fruit Street wellfield. However, as discussed elsewhere in this report, additional proposed development of Legacy Farms and other areas within Town are expected to increase the daily water demand over the next 20 years to approximately 1.36 MGD. The projected increase in water demand will likely cause the Town to pursue additional ground water withdrawals and to apply for an additional increase to their WMA permitted volume.

As discussed further in Chapter 6 of this report, the Town's most promising alternative for providing additional ground water is to increase the pumping rate of wells within the Fruit Street wellfield, particularly that of the new well, No. 6. The wellfield is currently operated below its safe yield capacity because of the current WMA permitted volume for the wellfield, 0.75 MGD. In fact, the sum of the tested safe yields of Well Nos. 1, 2, and 6 (No. 3 is no longer in use) is 1.35 MGD. The safe yield of a well is typically determined on an individual well, theoretical, basis and does not generally account for well hydraulics, plugging and other well interferences. While the safe yield of the entire aquifer has not yet been determined, it very likely exceeds 0.75 MGD. Increasing the withdrawal rate of Well No. 6 and/or other wells in the Fruit Street wellfield would likely allow the Town to meet the 20-year projection of 1.36 MGD of water use. If not, the two most promising locations for new ground water withdrawals are WH-3 and WH-4, located along the Sudbury River near the Southborough Town line and on the eastern shore of Whitehall Reservoir, respectively. Whether the Town is able to increase capacity of the Fruit Street wellfield or install new wells at WH-3 or WH-4, the Town will need to apply to increase their WMA permitted withdrawal volume from 1.21 to 1.36 MGD.

All three potentials sites for additional water – the Fruit Street wellfield, WH-3, and WH-4 – are located within the SuAsCo basin. Further, all three sites are located within sub-basins with a Ground water Withdrawal Level category of GWL4 or GWL5. Therefore, if new or additional ground water withdrawals are to be pursued at one or more of these sites, the SWMI-based permitting process will likely entail mitigating the impacts of the additional withdrawn volume through up to 0.16 MGD of offsets within the SuAsCo basin. These offsets may take the form of stormwater recharge, wastewater recharge, water conservation measures and water demand reductions, and/or water loss reductions.

3.4 Stormwater Recharge and the 2013 SWMI Grant Study

Sustainable water management is already being promoted in Hopkinton through the Sustainable Water Management Initiative. Hopkinton is committed to managing their water resources in a fashion that balances the needs of a community with the environmental needs of a watershed. Sustainable water management is about reducing water withdrawal needs and improving ground water infiltration to improve watershed health. Weston & Sampson was successful in assisting the Town in obtaining a 50/50 grant to perform a study in Hopkinton looking at the potential for stormwater recharge.

The study included a review of the Town's stormwater-related bylaws, development of a stormwater recharge training video, a GIS-based analysis of areas within Town most favorable for stormwater recharge, and an analysis of the potential benefit of retrofitting 262 existing stormwater systems within Town. As indicated in the 2013 study, retrofitting all 262 existing systems to capture and recharge stormwater runoff would result in an offset of approximately 0.6 MGD. The majority of that volume, approximately 0.4 MGD, would be recharged within the SuAsCo basin, where the Fruit Street wellfield is located.

In addition to evaluating the potential for stormwater recharge, the 2013 SWMI grant study also identified additional offsets or mitigation sources for use in satisfying the SWMI framework and new WMA permitting requirements. Those additional offsets included in-stream flow improvements, habitat improvements, wastewater discharge improvements, increased water supply management, and increased demand management.

3.5 Wastewater Recharge

Recent conversations between the Town and MassDEP revealed that those offsets may be satisfied or partially satisfied by ground water discharge at the Town's primary Wastewater Treatment Plant (WWTP), located within the SuAsCo basin, near the Fruit Street Wellfield. While the WWTP is permitted to discharge up to 0.35 MGD to ground water, it currently discharges approximately 0.11 MGD. The current discharge rate of 0.11 MGD is not sufficient to account for the full 0.16 MGD of offsets required by an increase to the WMA permitted volume. However, given that water demand is projected to increase over the next 20 years, it is possible that discharge from the WWTP would increase to 0.16 MGD or higher. Other opportunities for mitigating offsets exist as well as evaluated through a recent SWMI-related stormwater recharge study conducted by the Town.

3.6 Water Conservation Standards

The Water Conservation Standards set statewide goals for water conservation and water use-efficiency, and provide guidance on effective conservation measures. The goal is to move everyone to more pragmatic water use that will tighten our infrastructure, reduce water waste, help ensure sustained water supply, protect aquatic ecosystems, and provide financial savings for the cost of water. The Water Conservation Standards, written by the Massachusetts Executive Office of Energy and Environmental Affairs (EEA) and Water Resources Commission was published in July 2006 and updated in June 2012, include recommendations and standards for the following 10 areas; Comprehensive Planning and Drought management Planning; Water Audits and Leak Detection; Metering; Pricing; Residential Use; Public Sector Use; Industrial, Commercial and Institutional Use; Agricultural Use; Lawn and Landscape; and Public Education and Outreach.

TABLE 3-1
AVERAGE INDOOR WATER USE IN CONSERVING AND NON-CONSERVING NORTH AMERICAN SINGLE-FAMILY HOMES

Conservation Level	RGPCD
Nonconserving Home	69.3
Conserving Home – 2001	45.2
Conserving Home – 2005	36.2

This table summarized from the WCP Table 6-7 (Vickers, 2001 MA Water Conservation Standards).

3.7 UAW Compliance Plan

Hopkinton's unaccounted for water (UAW) average for the last 4 years is 22%, greater than the 10% performance standard presented by Massachusetts WRC. If Hopkinton were able to reach the 10% performance standard that would save 43.8 million gallons of water every year (120,000 gallons per day). There would be savings in resources for pumping, treating, and distributing water as well as environmental health benefits to the wetlands that will be protected. In addition, the amount of water purchased from other sources could be reduced.

In 2010 Hopkinton filed a UAW Compliance Plan with MassDEP, this plan was updated in December 2012. Hopkinton is working diligently to replace aged water meters, replace and calibrate master water meters, and install radio-read registers so that they can go to quarterly billing. They are continuing with their leak detection efforts and are actively looking for and repairing leaks. It is imperative for Hopkinton to continue working diligently to reduce their UAW. The MassDEP introduced the concept of achieving functional equivalent compliance in the Town's most recent WMA permit. The DEP will consider Hopkinton for functional equivalency if they are unable to meet the 10% UAW performance standard within 5 years of receiving its permit if they:

- Are complying with the Water Conservation requirements included in the permit,
- Have implemented the required limits on nonessential outdoor water use, and
- Are making demonstrable efforts to finance, implement and enforce a MassDEP approved compliance plan.

The Town may need to conduct a water audit to continue identifying contributing factors to the high UAW. We estimate the cost to prepare this audit to be approximately $30,000.

3.8 Summary & Recommendations

Given the hydrogeologic landscape of Hopkinton, if and when the Town pursues additional water withdrawals, it will surely face the need to mitigate those additional withdrawals through offsets. Fortunately the Town has many promising opportunities for such offsets, including wastewater discharge at their existing WWTP at Fruit Street, stormwater recharge throughout Town as was studied through the 2013 SWMI grant program, and many others in the form of water conservation, demand reduction, and water loss reduction strategies. While it will be important for the Town to actively pursue these many offset opportunities, it will be equally important for those offsets to be recorded in a reliable manner and for Town departments to communicate effectively regarding the need and implementation of those offsets. More than ever, it will be important for the Town to coordinate ongoing and proposed development with the Town's need for more water and for corresponding mitigating offsets. As the SWMI framework will require detailed accounting of those offsets, the Town will need to develop a reliable way to record the impact of the various mitigation measures that are implemented.

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4.0 DISTRIBUTION STORAGE SYSTEM EVALUATION

4.1 General

The primary purpose for water storage in a water distribution system is to provide water and stabilize pressures to residents during the peak water demand periods (usually in the summer), provide adequate storage to meet fire flow needs, and provide water during periods of emergency and during pumping facility failures. There has been a historical tendency in the water industry to oversize water storage tanks. Within the last several years, with changes to the disinfection by-product rule, we have identified these large water storage tanks to be areas where water quality deteriorates through improper mixing and can result in significant water aging. Our goal in reviewing the Town's water storage requirements is to recommend adequate water volume at proper heights to meet system needs.

4.2 Water Storage Requirements

When evaluating the water storage needs of a system, the storage assessment is typically broken into three components; equalization, fire and emergency storage. The following describes how each storage classification is defined when assessing the storage needs of a water system and how water storage is calculated:

- Equalization storage - furnish 15% of Max Day demand available at 35 psi or greater to the water service provided to the highest ground elevation.
- Fire storage - the maximum volume the Town would be required to store is 630,000 gallons; which equates to 3,500 gpm for three hours. According to a 1983 ISO report, the high and middle schools, located near Hayden Rowe Street and Colonial Avenue, were identified as an area requiring a 3,500 gpm fire flow for three hours. The bottom of the designated fire storage volume should be provided at a storage height that provides 20 psi pressure in the water system to the water service located at highest ground elevation.
- Emergency storage – Although it is good industry practice to have emergency storage, there is no requirement for furnishing a certain volume of emergency storage. The

volume of emergency storage is dependent upon available emergency power, number of sources and other emergency measures (e.g. interconnections).

The dimensions and volumes of the three existing water storage tanks in the water system are as follows:

TABLE 4-1
EXISTING TOWN WATER STORAGE

Storage Tank	Small Grove Street Tank	Large Grove Street Tank	West Main Street Tank	Total
Storage Volume (gals)	320,000	1,500,000	793,000	2,613,000
Overflow Elevation (ft)	600	600	600	--
Height (feet)	60	60	70	--
Diameter (feet)	30	65	44	--
Volume/foot (gal/ft)	5,330	25,000	11,370	41,700
Usable Storage (gals)[1]	75,000	350,000	159,000	584,000

1 – Usable storage is defined as all storage located at a storage height that provides 20 psi pressure (586.2 feet) in the water system to the water service located at highest ground elevation (540 feet).

As witnessed in Table 4-2, the Town has a deficit in equalization storage. This situation is created because the Town has customers located at high ground elevation. This makes much of the water in these tanks un-useable, makes it difficult to get proper turnover in the tanks, and creates a situation where the Town needs to keep these tanks full to maintain pressure.

TABLE 4-2
AVAILABLE WATER STORAGE

	Equalization Storage[1] (gals)	Fire Storage[2] (gals)	Emergency Storage[3] (gals)	Total Existing Storage (gals)
Available	0	584,000	2,029,000	2,613,000
Surplus/ (Deficit)	(567,250)	(46,000)	N/A	--

[1] Equalization storage is the water stored above 35 psi elevation provided to the highest house in the service area (540-feet).
[2] Fire storage is the water stored above 20 psi elevation provided to the highest house in the service area (540-feet).
[3] Emergency storage is the water stored below 20 psi elevation provided to the highest house in the service area (540-feet).

We recommend the Town provide 15% of Maximum Day demand for equalization storage. *Given the emergency power present at all but one of the well sites and balancing the appropriate amount of emergency storage with water age concerns, we recommend that we not calculate for a required emergency storage as this may serve to increase the size of the tank without just reason*.

The following table lists the amount of equalization, and fire storage required in the existing tanks to meet the guidelines defined above for future water demands as presented in this report. As noted in Table 4-2, the Town does not currently have any equalization storage available and has 584,000 gallons of fire storage available to the highest houses. Per these criteria, the Town of Hopkinton water storage is deficient in serving the water system.

TABLE 4-3
FUTURE WATER STORAGE REQUIREMENTS

Required Equalization Storage[1] (gals)	Required Fire Storage[2] (gals)	Required Emergency Storage[3] (gals)	Total Required Storage (gals)
340,280	630,000	N/A	970,280

[1] Required equalization storage is 15% of Future Max Day demand (2.27 MGD)
[2] Required fire storage is 3,500 gpm for 3 hours
[3] There is no requirement for emergency storage

4.3 Main Service System Tank Improvements

Both Grove Street Tanks are in need of maintenance. The Town recently completed an inspection of the tanks in December 2013. It determined that the large Grove Street Tank is in need of a full interior and exterior coating system estimated to cost $635,000. The small Grove Street Tank was observed to need approximately $435,000 of improvements. Given the age of the small Grove Street Tank, and the fact that it is at the end of its useful life, it doesn't make good fiscal sense to spend money rehabilitating it. We would recommend that the Town abandon the small Grove Street Tank and remove it from service, but the Town will not be able to provide adequate service to customers if they remove the Large Grove Street Tank from service for maintenance without either a second Main Service Tank on the site or a High Service Tank which would feed into the Main Service System. Based on the required storage volume outlined in Table 4-3, the Town could actually provide adequate storage with a smaller tank located at the Grove Street tank site than 1.5 MG. We reviewed the cost to replace the two steel water storage tanks on the site with one low/no maintenance water tank (i.e. glass fused to steel or pre-stressed concrete). This would relieve the pressure to have two tanks on the site for redundancy, but would not adjust the height of the tank nor construct a tank for the high service system. The cost to construct a concrete tank on the Grove Street Tank site would be several hundred thousand dollars higher than a glass-fused to steel tank. The space requirements for constructing a concrete tank are also significantly higher than the glass-fused to steel tank. Given that we would be constructing the new 1.0 MG standpipe in the location of the small Grove Street Tank while maintaining the large Grove Street Tank with a small site, we are recommending construction of a glass-fused to steel tank. The enclosed photo is of a glass-fused to steel tank we constructed in Medway which was constructed on a very tight site next to an existing bolted steel tank that was maintained in service for the duration of construction of the new tank. The cost to construct a new 1.0 MG, 60 foot tall glass-fused-to steel standpipe next to the Large Grove Street Tank and abandon the Large Grove Street Tank after construction is approximately $1.35 million which is approximately half the cost of rehabilitating the existing Large Grove Street Tank. This cost includes removal and abandonment of both tanks from the site, the small Grove Street Tank prior to construction and the Large Grove Street Tank after construction. The benefit to constructing a new glass-fused-to steel tank to replace the existing 1.5 MG tank would be a significant reduction in the annual maintenance cost. Given that these coating systems are only anticipated to last 15 to 20 years, the payback to constructing a new tank would be realized within the planning period of this study.

4.4 High Service System

Weston & Sampson prepared a High Service System Evaluation dated May 10, 2011 where we evaluated constructing a high service system around the Grove Street Tank site and High school complex encompassing the land of high ground elevation. The primary focus of the study was to determine if the Town needed a high service system and if a second water main should be installed in Main Street as part of the water main replacement project to provide looping for the high service system. The recommendations of the study was that the Town did need to construct a high service system, that it should include houses above ground elevation 500 feet that would provide approximately 50 psi to the highest ground elevation (585-feet), and that a parallel water main was not needed in Main Street. This represented an increase from the Earth Tech Water Master Plan which recommended a 450-feet ground elevation cut-off for the high service system boundaries. The primary reasons for modifying the recommendation were as follows:

- Customers between 450- and 500-feet of ground elevation are served well through the Main Service System with tank overflow elevation of 600-feet.
- There are portions of undeveloped land at high ground elevation that could be served by a higher service system grade line.

We further evaluated the volume, height and location of the tank and further laid out the high service area as part of this study. The High Service System Evaluation recommended a 750,000 gallon elevated water storage tank (reduced from Earth Tech Water Master Plan recommendation of 1.0 MG) with an overflow elevation of 700 feet. A water tank at this elevation would allow the Town to serve water supply to undeveloped areas of high ground elevation and allow the Town's water system to be expanded. After further reviewing the topography in the area near Lumber Street and Glen Road, we recommend lowering the overflow elevation of a tank serving the high service area to 650 feet. A tank at this elevation would adequately serve the majority of houses on the hill. It is possible that approximately a dozen houses on the peak of the hill in the vicinity of Glen Road and Breakneck Hill Road would require booster pumps if they were to connect to the Town water system.

Constructing a high service system and new high service tank would lower the ground elevation of the highest house in the Main Service System to approximately 500-feet, which would increase the amount of equalization storage available in the existing water storage tanks; would allow the Town to drop the water level in the tanks more consistently without impacting pressure in the system; and would allow the small Grove Street Tank to be removed from service.

It is time to remove the small Grove Street Tank from service and demolish it from the site. Maintaining two steel water tanks on the same site is costly, especially when amortized over the lifespan of a painting system. This tank is approaching 100 years old which is considered the useful life of a bolted steel tank.

If the high service system is constructed, the highest house elevation in the Main Service system would be located at 500-feet. Table 4-4 demonstrates the hypothetical available water storage in the Main Service System with the high service system constructed, the small Grove Street Tank out of service, and a new 1.0 MG glass-fused to steel standpipe replacing the large welded steel Grove Street Tank. The new Grove Street 1.0 MG tank would have approximately 18,000 gallons/foot. This indicates that the two water storage tanks would have adequate volume to service the entire Town water system.

TABLE 4-4
AVAILABLE WATER STORAGE MAIN SERVICE SYSTEM – FUTURE CONDITIONS

	Equalization Storage[1] (gals)	Fire Storage[2] (gals)	Emergency Storage[3] (gals)	Total Existing Storage (gals)
West Main St	227,400	386,580	185,000	798,980
New Grove St	360,000	530,000	110,000	1,000,000
Available	587,400	916,580	295,000	1,798,980

[1] Equalization storage is the water stored above 35 psi elevation provided to the highest house in the service area (500-feet).
[2] Fire storage is the water stored above 20 psi elevation provided to the highest house in the service area (500-feet).
[3] Emergency storage is the water stored below 20 psi elevation provided to the highest house in the service area (500-feet).

After the high service system is constructed, the Town will no longer have a water storage deficiency. We recommend that a tank be constructed in the high service system to provide fire flows and stabilize pressures to the high service system as well as to the Main Service System.

If the high service area was created today, we estimate that approximately 150 existing houses would be served by it. Assuming that another 100 homes would be constructed within this service area, we estimate the daily water use of the high service area to be approximately 40,000 gpd. Table 4-5 details the required storage volume for a water tank servicing the high service system.

TABLE 4-5

HIGH SERVICE SYSTEM WATER STORAGE TANK

Required Equalization Storage[1] (gals)	Required Fire Storage[2] (gals)	Required Emergency Storage[3] (gals)	Total Required Storage (gals)
10,000	216,000	N/A	226,000

[1] Required equalization storage is 15% of Future Max Day demand (60,000GPD)
[2] Required fire storage is 1,800 gpm for 2 hours
[3] There is no requirement for emergency storage

We recommend that the Town consider constructing a 300,000 gallon elevated composite tank for the high service system. A composite tank would include a concrete pedestal with a glass fused to steel elevated storage cylinder. A 300,000 gallon capacity tank of this type would be approximately 36-feet in diameter and the water cylinder would be approximately 40 feet tall. The tank would not require painting and would be considered a low maintenance alternative. The City of Peabody just completed a similar tank to this (see enclosed photo).

4.4.1 *Lumber Street Tank Site*

If a 300,000 gallon composite elevated tank is constructed on the Lumber Street tank site with ground elevation approximately 585-feet, with an overflow elevation of 650-feet, the tank would be approximately 65-feet tall. If the tank is constructed on this site, it will require the Town to construct approximately 4,500 feet of 12-inch water main to connect the tank to the existing water system through to Daniel Road. We recommend that the Town consider either utilizing the existing high school complex pump station (as that pump station will not be required when the high service system is constructed) to fill the tank or construct a new pump station on the Grove Street Tank site with appropriately sized pumps.

4.4.2 *Grove Street Tank Site*

It may be possible to construct the new elevated tank on the Grove Street Tank site if one of the existing tanks is demolished. The tank site is very tight, and space is limited which will make constructing a new tank with an existing tank is service challenging. If a 300,000 gallon elevated composite tank is constructed on the Grove Street tank site with overflow elevation of 650 feet, the tank will be approximately 130-feet tall (approximately twice the height of the existing tanks). The benefit to putting the high service tank on this site is that it would not require that 4,500 feet of water main be constructed as part of the tank construction.

The May 10, 2011 High Service Evaluation recommended installation of pressure reducing valve vaults on Granite Street and Pleasant Street that would operate to supplement water supply from the High Service System to the Main Service System. This is important to supplement fire flows and pressures in the vicinity of the Grove Street Tank. It will be necessary to construct some additional water main on Grove Street to loop the high service system and to allow the 12-inch water mains on Grove and Pleasant Street to serve the Main Service System from the Grove Street Tank. These water main improvements are discussed in more detail in Chapter 5. We suggest that the following water mains be installed to create the high service system. These improvements will be required regardless of which site is chosen for the high service tank.

TABLE 4-6
PROPOSED HIGH SERVICE WATER MAINS

Street Name	Limits	Proposed Diameter (in.)	Length (ft)	2013 Project Cost
Grove Street	Standpipe to Pleasant St	16	1,100	$330,000
Grove Street	Pleasant St to Maple St	8	700	$140,000
TOTAL			1,800	$470,000

It is possible that the cost of the high service system could be primarily born by developers looking to develop this land of high ground elevation that cannot be adequately served from the Main Service System. The Town does not appear to be getting complaints regarding water pressure in this area from existing customers implying that it is not an urgent need. The Town will not be able to extend the water service to serve areas of land of high ground elevation near I-495 without building the 12-inch pipeline. We have outlined the costs of constructing the high service system at the Grove Street site as it seems like the likely alternative, but the Town would need to spend an additional $900,000 if it is constructed at the Lumber Street tank site (Table 4-7).

TABLE 4-7
HIGH SERVICE SYSTEM PROJECT COSTS
(GROVE STREET TANK SITE)

Description	2013 Project Cost
Construct 0.3 MG Tank	$1,300,000
Construct New Pump Station	$550,000
Construct PRV Vaults (2)	$175,000
Install Water Main on Grove St	$470,000
TOTAL	$2,495,000

1. This does not include the pipeline to connect the high service system to the Lumber Street area and tank site.

4.5 Summary

We have attempted to outline the challenges facing the Town with regards to the high service system and water storage issues:

- The Town is in immediate need of performing maintenance on their existing Grove Street Tanks.
- It doesn't make sense to rehabilitate the Small Grove Street Tank as it is at the end of its useful life.
- The Town needs two tanks on the Grove Street tank site as long as one of them is painted steel due to the downtime for maintenance which is required every 15 to 20 years.
- The cost to rehabilitate the Large Grove Street Tank is approximately half the cost to replace it with a 1.0 MG low maintenance tank. A low maintenance tank on the site would allow the Town to remove both painted steel tanks and would pay for itself in 15 to 20 years when it needs to be painted again.
- It doesn't make sense to spend $2.5 million constructing the high service system when the Town is not getting pressure complaints. It makes sense to wait until a future development can help share in the cost.

We recommend that in lieu of spending $635,000 painting the Large Grove Street Tank, the Town construct a new low maintenance 1.0 MG tank in the location of the Small Grove Street Tank and remove the Large Grove Street standpipe. This would allow the Town to maintain water service with the Large Grove Street Tank during construction and abandon and demolish the tank after construction of the new tank. This work will need to be planned within the next five years, as the Large Grove Street Tank will begin failing without rehabilitation and is estimated to cost $1,350,000.

5.0 HYDRAULIC MODELING

5.1 General

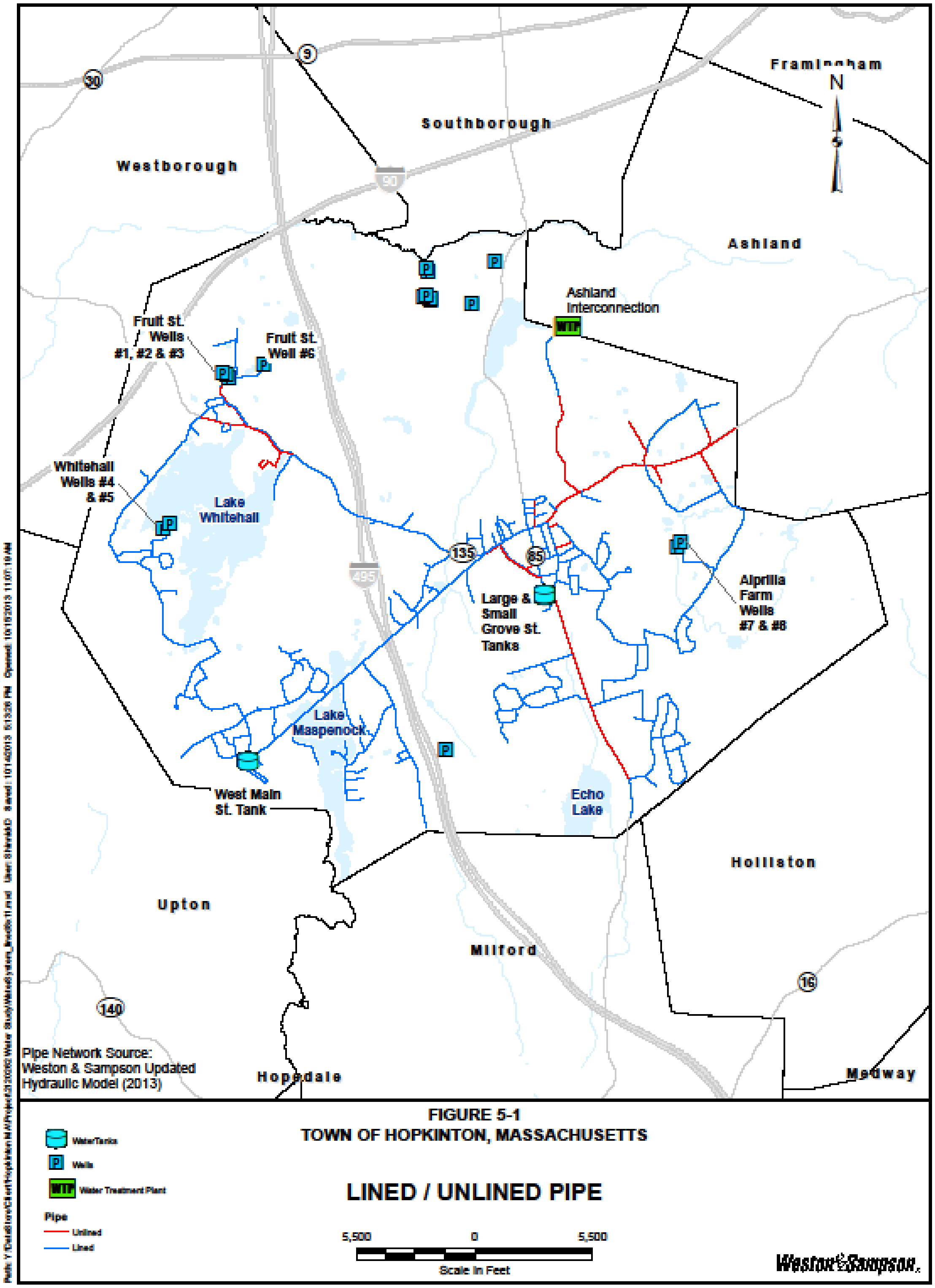
The primary focus of this Master Plan was to evaluate the Towns water supplies and future water supply alternatives. We have evaluated the ability of the water transmission system to transfer water around from the varying sources during current and future demand events for the differing water supply scenarios. As such, we have not focused on small diameter distribution system improvements in this study.

5.2 Water System GIS Pipe Network

It is not clear to us what year the Town's GIS pipe network was created, but it appears to have been created utilizing an earlier version of the Town's hydraulic model. The hydraulic model was updated from an EPA Net model during the 2003 Water Master Plan. At this point, it appears that the water system GIS pipe network and the hydraulic model pipe network have been edited separately with neither version appearing more accurate than the other.

The Town does not have a water system GIS with valves and hydrants. We recommend that the Town consider surveying / GPS locating the water structures so that an accurate water system GIS can be constructed. This would then allow the hydraulic model database to be merged with the GIS pipe network database to create one new accurate pipe network. The best way to manage a water distribution system model to avoid making edits to two different pipe networks, is to update the water system GIS as the master file and import into the modeling software regularly to keep the model updated. The first time this import is done, there is typically a lot of work that needs to be done to the water system GIS to make the import go more smoothly in the future. The cost to create a water system GIS with accurate GPS locations of valves and hydrants and water mains and generate up to date maps is approximately $100,000.

The current water system GIS and hydraulic model have Hazen Williams C-values populated, but do not have complete information on water main installation date or material of construction. Based on the C-values in the model, we have identified the unlined water main in Figure 5-1 (assuming that C-value less than 80 is unlined pipe).


Framingham
N
Southborough
Westborough
Ashland
Ashland
Interconnection
WTP
Fruit St.
Wells
#1, #2 & #3
Fruit St.
Well #6
Whitehall
Wells #4
& #5
Lake
Whitehall
Large &
Small
Grove St.
Tanks
Alprilla
Farm
Wells
#7 & #8
Lake
Maspenock
West Main
St. Tank
Echo
Lake
Holliston
Upton
Milford
Pipe Network Source:
Weston & Sampson Updated
Hydraulic Model (2013)
Hopedale
Medway
FIGURE 5-1
TOWN OF HOPKINTON, MASSACHUSETTS
LINED / UNLINED PIPE
Water Tanks
Wells
Water Treatment Plant
Pipe
Unlined
Lined
5,500
0
5,500
Scale in Feet
Weston&Sampson

5.3 Hydraulic Modeling

5.3.1 Future Water Demand Impacts

The hydraulic model was populated with future (2033) average day, maximum day and peak hour demand scenarios. The hydraulic model was then used to determine what impact the projected demands would have on the distribution system under different water supply scenarios.

When evaluating the adequacy of a water system to satisfy operating conditions, the ground water supplies must be capable of satisfying maximum day demands during low ground water operating conditions. We have assumed that the Town supplies will not be able to operate 24-hours a day at their safe yields during the summer conditions when a maximum day demand event would occur. For these scenarios, we have assumed that the well supplies will be pumping 16 of the 24 hours in a day or 67% of the safe yield. Peak hour demands which generally occur during the maximum day demand will be served through water storage.

There are two different water supply operating conditions for the water system that were analyzed in the hydraulic model, one with a WTP constructed to treat the Whitehall wells and one without a WTP. With the exception of the Fruit Street wells which can always pump their WMA allowed withdrawal of 0.75 MGD, we assumed that the well supply would be 67% of the safe yield. Table 5-1 demonstrates the operating conditions of the Town's wells during the two water supply scenarios.

TABLE 5-1
PUMPING CAPACITIES IN HYDRAULIC ANALYSIS

Source	w/ WTP (MGD)	w/o WTP (MGD)
Fruit Street Wells	0.75	0.75
Whitehall Wells	0.55	0
Alprilla Wells	0.28	0.28
Ashland*	0.5	0.5
TOTAL	**2.08**	**1.53**

* Hopkinton can take one day up to 0.8 MGD

5.3.2 Transmission Main Deficiencies

The existing distribution system was evaluated for transmission main deficiencies. The model was run under a peak hour demand scenario to locate transmission mains with high headloss and/or velocity. In general, velocities in the system are 2.5 ft/sec and lower during a future peak hour demand. Headlosses are mostly below 3.0 feet/1,000 feet of pipe during a future peak hour demand.

The model was run for a 24-hour extended period simulation under future maximum day demands (scenario 1) with a peak hour event occurring during the maximum demand day. The model indicates that all pressures are maintained at 35 psi and tanks re-fill by the end of the day.

5.3.3 High Service System Hydraulic Analysis

We utilized the hydraulic model to assess the impacts of removing the Small Grove Street tank from service with only the Large Grove Street and West Main Street tanks in operation (before the high service system is constructed). We ran an extended period simulation under future maximum day demands (2.27 MGD) with Whitehall pumping 67% of the safe yield and the largest source (Ashland) out of service. The Hopkinton sources that were online totaled 1.58 MGD during this event. During a 24-hour time period with the Large Grove Street and West Main Street tanks each drawn down approximately 10 feet from their overflow elevations at the start of the simulation period, the hydraulic gradeline in the system dropped approximately 20 feet during the simulation. This equates to an 8 – 9 psi drop in available static pressure in the system. It also reduces the available storage in the two tanks by over 700,000 gallons.

We ran the simulation again but turned on the Ashland source (0.8 MGD) thereby producing 2.38 MGD of available water supply during a future maximum day demand event. Both the Grove Street and the West Main Street tanks were again drawn down 10 feet from their overflow elevations at the start of the simulation. At the end of this simulation, the hydraulic gradeline had risen by 3 to 6 feet across the distribution system boosting static pressures slightly and replenishing storage volumes in the tanks by almost 175,000 gallons.

The model indicates that the water system can operate effectively with the small Grove Street Tank offline and prior to constructing a high service system with either the Whitehall Wells or Ashland connection pumping into the system.

Prior to creating the high service system in the model, we reviewed the impact that the high service system would have on providing water from the Grove Street tank(s) into the Main Service system. Currently there are three 12-inch diameter water mains which feed the Main Service system from the Grove Street tank(s). These mains are located in Grove Street, Pleasant Street and Hayden Rowe Street. When the high service system is constructed, the 12-inch main in Hayden Rowe Street will be split between the main service and the high service system. This will reduce the transmission mains that continually feed the Main Service system from the Grove Street tank from three to two. Approximately 50% of the flow from the Grove Street tank passes through the Hayden Rowe Street 12-inch main under current conditions (no high service system).

When the high service system is built, we recommend installing a 16-inch water main between the large Grove Street Tank and the 12-inch mains that meet at the intersection of Grove Street and Pleasant Street. We modeled the proposed 16-inch water main to evaluate whether this improvement would mitigate any reduction in hydraulic transmission from the Grove Street Tank to the Main Service system when the Hayden Rowe Street 12-inch main is divided into the two service systems. The model indicates that the 16-inch main will compensate for the loss of the Hayden Rowe Street 12-inch water main and provide similar flows into the main service system once the high service system is built. It should be noted that the 16-inch water main simulation was conducted with the Whitehall Wells and Ashland source online.

With the Ashland source offline and the new high service system in place, flows into the Main Service system from the Grove Street tank site differ from the results stated above. When the high service system is built, the proposed high service system tank will be fed from the Main Service system from a connection near the Grove Street Tank. During periods of filling the high service system tank, available flow into the main service system will be less. However, the two pressure reducing valves that are proposed as part of the high service system upgrade will enable supplemental flow to enter into the main service system during periods of high demand when pressures in the main service system drop to preset levels.

5.4 MWRA Connection

An MWRA connection was modeled under the two water supply alternatives. The first alternative included construction of a WTP to treat Wells No. 4 and No. 5. Under this alternative, the most that an MWRA connection would be needed (possibly would not be needed at all with Scenario 2 demand projections) is to supplement the maximum day demands during the summer starting in the year 2020 with 0.22 MGD (approximately 150 gpm). The water supply shortfalls for the different alternatives are discussed in detail in Chapter 8. The second water supply scenario assumes that no WTP is constructed for Whitehall Wells and that the MWRA is utilized for supplemental water supply. The MWRA is anticipated to be utilized to supplement the existing supplies for maximum day demand in the year 2033 by approximately 1.05 MGD (700 gpm). The MWRA connection would be utilized starting immediately in the summer (at a lesser flow) and will be used more frequently as the years progress. As the 1.05 MGD scenario has the largest impact on the water system, the MWRA modeling was performed with this scenario.

The MWRA connection was modeled through Southborough as well as Ashland. If Hopkinton purchases water from Southborough, a pump station will be required at the Town line as well as approximately 2.7 miles of pipeline down Route 85. If the connection to Southborough is made, the potential flows through this main would be approximately 700 gpm. The hydraulic analysis indicates that at these potential flows, a 12-inch water main would serve the Town most appropriately and would allow the Town a solid transmission main for future growth in this area of Town.

The Town of Hopkinton currently has an agreement with the Town of Ashland to purchase up to 1.0 MGD of water for a short duration of time. As such, the connection with Ashland is sized for 1.0 MGD. If the Town does not construct a WTP for Whitehall Wells, they will need to purchase water from the MWRA through the Ashland interconnection. The 1.0 MGD connection will serve them well during the majority of the year, but in the summer, the Town will need the MWRA at approximately 1.0 MGD and the Ashland WTP for their standard 0.5 MGD to meet 2033 projected (Scenario 1) water demands. This means that Hopkinton will need to pump 1.5 MGD of water through their existing interconnection with Ashland. The hydraulic model indicates that the headlosses through the 12-inch water main in Wilson Road are increased by 7 psi during this scenario which will have hydraulic impacts to the existing pumps located at the WTP.

We evaluated the static pressure variations across the entire water system during a peak hour event when the MWRA connection was online versus having Wells No. 4 and No. 5 online. The results indicate when Wells No. 4 and No. 5 are online, the water system provides marginally better static pressures (3 psi) for some of the lowest domestic pressures (below 35 psi) in the system. This is most likely attributed to the location of these high elevations in the West Main Street tank area.

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6.0 WATER SUPPLY EVALUATION

6.1 General

The purpose of this section is to evaluate if the Town's existing water supplies are adequate to meet the future water demands estimated in Chapter 2.0 of this report for the 20-year planning period. Each of the Town's existing water supplies was evaluated in terms of safe yield, Water Management Act (WMA) restrictions and water quality, including the water purchased from the Town of Ashland. In addition to the water quality at each source, the overall water quality in the distribution system was evaluated. We evaluated typical well capacity assuming that the average available water supply from a ground water source would be approximately 67% of the safe yield capacity which is representative of a 16 hour pumping day. There are many factors which affect the volume of water available from a well such as ground water levels, dry/drought conditions, well plugging, influences from other wells, etc. It can be expected that during drought conditions, or other unusual circumstances, the actual pumping capacity from the Town's wells could be reduced greater than 67% of the safe yield capacity.

6.2 Water Management Act and System Demands

The Town withdraws water from seven (7) active ground water wells and purchases water from the Town of Ashland, all within the Concord River Basin. Between 2009 and 2012, the Town's WMA permit included a registered volume of 0.56 MGD for the combined withdrawal from Wells No. 1 and No. 2 and a permitted volume of 0.42 MGD for the combined withdrawal from Wells No. 4, No. 5, No. 6, and the Ashland interconnection for a total of 0.98 MGD. The total authorized annual average withdrawal in the water management act includes the volume of water purchased from the Town of Ashland. Based on the water usage identified in Chapter 2.0, the Town averaged between 0.99 and 1.00 MGD each year from 2009 through 2012, just above the volume authorized by the MassDEP, for that time period, in the WMA permit but within the allowable ±10% tolerance of the permit.

The Water Management Act for the Town of Hopkinton was modified by the MassDEP at the Town's request in March 2013 to match the interim permit. The interim permit was determined by taking the amount of water the Town was using and increasing it for the proposed Legacy Farms development. The revised WMA permit modified the permitted volume from 0.42 MGD to 0.65 MGD and included the Alprilla wells No. 7 and No. 8, increasing the total authorized annual average withdrawal to 1.21 MGD. The WMA permit extends through August 31, 2015 although it is anticipated that the Town will need to increase the WMA permit within the next several years as that increase only theoretically included additional water needed for Legacy Farms.

Information on each well source, including the MassDEP established safe yield (also referred to as the Zone II maximum daily pump rate), was previously shown in Table 1-1. The safe yield for each well source and the actual maximum day volume pumped during each year between 2009 and 2012 are shown in Table 6-1. The combined safe yield when these individual safe yields are added together for the active well supplies is 2.6 MGD, including Well No. 7 and Well No. 8 which are new wells that were not active prior to 2013. However, the WMA permit restricts the total Fruit Street (Wells No. 1, No. 2, and No. 6) maximum daily pump rate to 0.75 MGD. With this restriction the actual combined safe yield for the active well supplies is 2.0 MGD. Well No. 3 has been reclassified from an active to an emergency source by the MassDEP and is not included in the Town's active water supplies. Well No. 6 has the largest individual safe yield of the active well sources with a safe yield of 0.72 MGD.

TABLE 6-1
MAXIMUM DAY WELL PUMPAGE

Year	Well 1 (MGD)	Well 2 (MGD)	Well 4 (MGD)	Well 5 (MGD)	Well 6 (MGD)	Well 7* (MGD)	Well 8* (MGD)	Total (MGD)
Safe Yield	***0.36***	***0.27***	***0.36***	***0.47***	***0.72***	***0.28***	***0.14***	***2.6****
WMA Yield	*	*	0.36	0.47	0.75*	0.28	0.14	**2.0**
2009	0.35	0.27	0.16	0.00	0.30	N/A	N/A	1.08
2010	0.36	0.26	0.16	0.21	0.39	N/A	N/A	1.38
2011	**0.44**	0.27	0.17	0.04	0.48	N/A	N/A	1.40
2012	**0.43**	0.27	0.23	0.13	0.46	N/A	N/A	1.52

*Actual combined safe yield is 2.0 MGD due to total Fruit Street (1,2,3 & 6) withdrawal limit of 0.75 MGD

Information provided in Table 6-1 shows that at times, Well No. 1 has been utilized beyond its MassDEP approved maximum daily withdrawal rates. Well No. 1 has been approved for a maximum daily rate of 0.36 MGD and exceeded this rate in 2011 and 2012, as indicated by the highlighted cells in the table. Based on the maximum day pumping rates shown in Table 6-1, the 0.56 MGD single day limit for the combined withdrawal from Wells No. 1 and No. 2 (registered portion of the WMA permit) may have been exceeded on some days throughout the summer months when demands were high. In addition, the maximum allowable daily pump rate of 0.75 MGD from all Fruit Street Wells (Wells No. 1, No. 2 and No. 6) may have been exceeded on some days throughout these same months. These exceedances are not in compliance with Hopkinton's WMA Permit and the MassDEP may take action against Hopkinton if these wells are pumped above their approved maximum daily rates in the future.

The agreement between Hopkinton and Ashland allows Hopkinton to purchase a maximum one-day volume of 1.0 MGD. However, Town water operators have indicated that demands in Ashland during the peak summer months make it difficult to pump 1.0 MGD and that a maximum pumping rate of up to 0.8 MGD (555 gpm) is representative of what is sustainable. The water supply from the Town of Ashland is Hopkinton's largest available source, 0.08 MGD greater than the safe yield of Well No. 6.

The future average day and maximum day water demands were estimated in Chapter 2.0 and shown in Table 2-11 and Figure 2-4. The average day demands projected for 2033 for Scenario 1 (65 RGPCD) and Scenario 2 (55.1 RGPCD) are approximately 1.36 and 1.15 MGD, respectively. The current WMA authorized annual average withdrawal is 1.21 MGD. Using the conservative demand projections from Scenario 1, the average day demand will exceed the WMA authorized withdrawal of 1.21 MGD starting in 2021. The Town will need to request and receive a WMA permit withdrawal increase of a minimum of 0.15 MGD to meet future average day demands in 2033. Under Scenario 2, the projected average day demand in 2033 is 1.15 MGD, below the current WMA authorized withdrawal, and an increase in the WMA withdrawal permit will not be necessary.

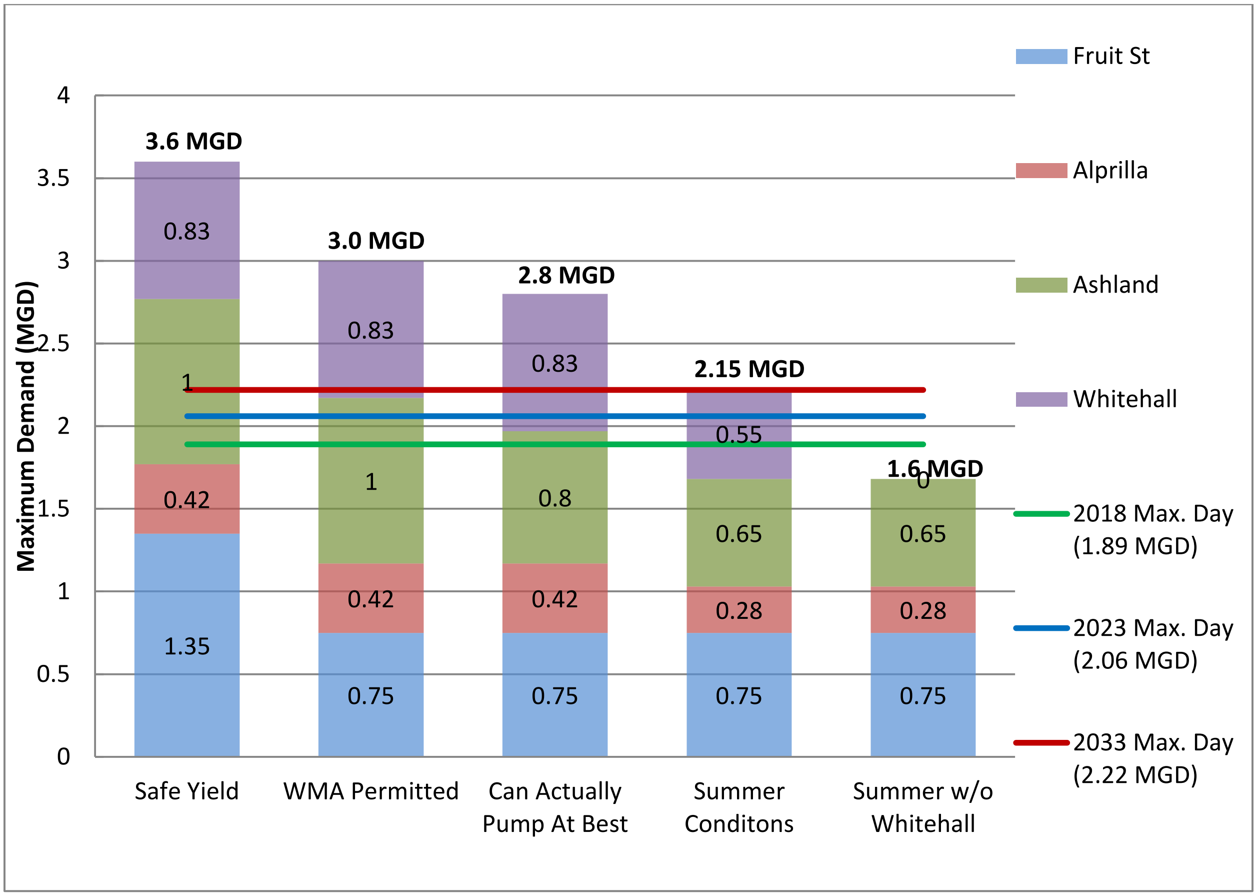
The maximum day demand projections for Hopkinton for the two Scenarios discussed in Chapter 2.0 are plotted in Figure 6-1 along with the total safe yield of Wells No. 1 through 8 (not including Well No. 3 which is classified as an emergency supply) and the maximum sustainable supply from Ashland. The maximum day demands projected for 2033 for Scenario 1 (65 RGPCD) and Scenario 2 (55.1 RGPCD) are approximately 2.22 MGD and 1.88 MGD respectively.

The total safe yield of the Town's active well sources is 2.6 MGD (includes water from Wells No. 4 and No. 5) which creates a theoretical pumping capacity of 3.6 MGD if we include Ashland. If we recalculate the maximum pumping capacity accounting for the WMA permit limitations at Fruit Street of 0.75 MGD, the permitted / allowable withdrawal with a theoretical Ashland capacity of 1.0 MGD is 3.0 MGD. When the maximum daily water supply from Ashland is reduced to what is actually sustainable (0.8 MGD), and without any other impacts (i.e. drought, source out of service) the Town of Hopkinton can achieve a maximum day water supply of approximately 2.8 MGD (assumes that Whitehall wells are active and pumping).

The amount of water that the Town can pump to meet their maximum day demands is further reduced during drought conditions. We have assumed that the actual supply of the Alprilla and Whitehall wells would be reduced by 67% to represent a reduced pumping day of 16 hours during summer conditions. The total available water supply during dry conditions if the Town were to pump the Whitehall wells would be 2.15 MGD. This is representative of an average worse case scenario and not necessarily the worst day/week of the year. This capacity will be reduced if we do not include the pumping capacity of the Whitehall wells. The iron and manganese in the Whitehall well water exceeds current secondary drinking water standards. The Town limits the volume pumped from this supply as it generates water quality complaints when utilized. The volume of water that the Town can pump to meet their maximum day demands without the Whitehall Wells during average summer conditions is 1.6 MGD. This supply is less than the Town's current maximum day demands. The Town utilizes the Whitehall wells to supplement supply when needed which demonstrates that the Town needs the water supply from the Whitehall wells to meet current and future water demands. Figure 6-1 shows the available water supply during the different conditions discussed above versus the projected maximum day demands with Scenario 1.

FIGURE 6-1

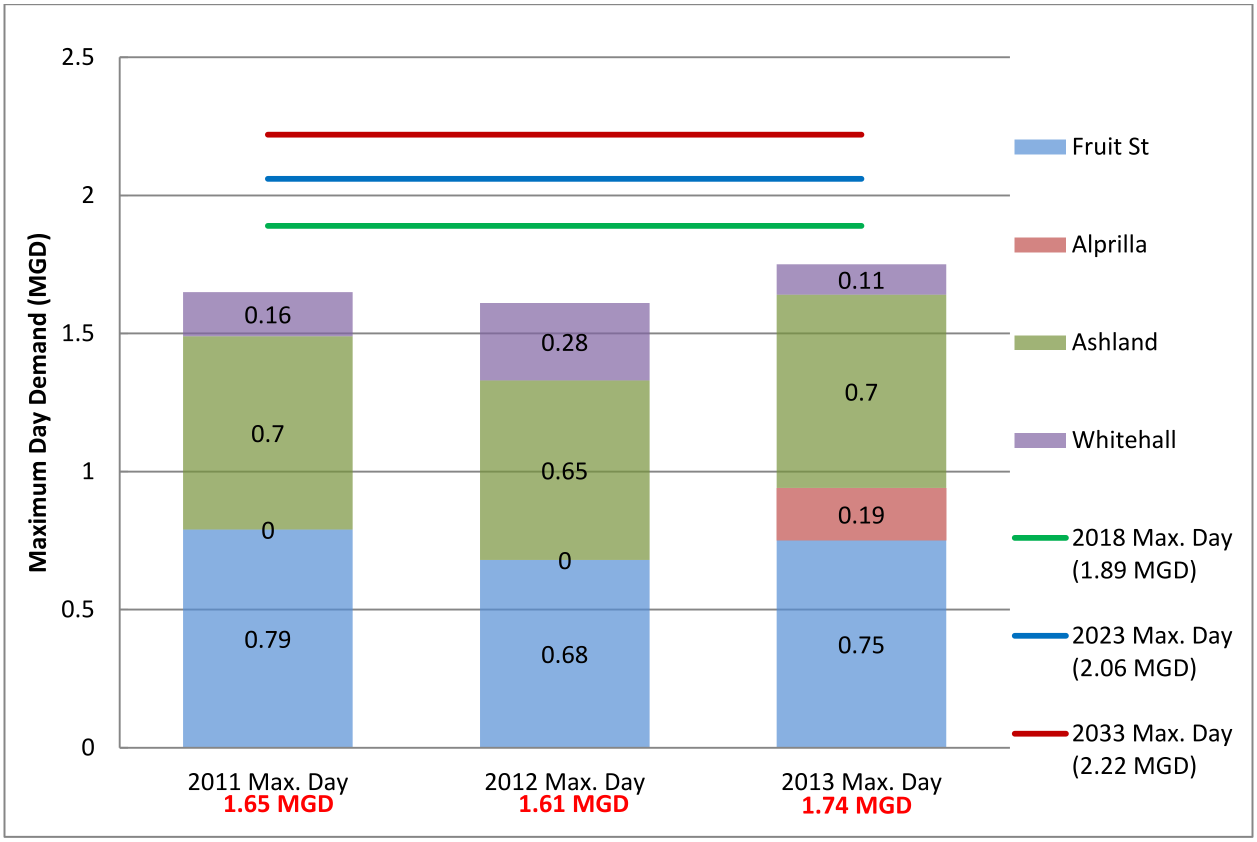
HOPKINTON WATER SUPPLY



It is necessary for Hopkinton to plan for worse case scenarios which include drought conditions and equipment failures. During normal operations, equipment can fail causing a source to be removed from service until repairs or replacement parts can be provided. To account for potential source disruptions, the future maximum day demand is compared to the Town's available supply with its largest source out of service. Figure 6-2 shows the historical available water supply during summer conditions with the Whitehall wells in service and with the largest source in service (Ashland).

FIGURE 6-2

WATER DEMAND PROJECTIONS AND AVAILABLE SUPPLY



With the Ashland water supply out of service, the Town of Hopkinton can supply up to 1.5 MGD during summer conditions resulting in a water supply deficit of 0.72 MGD at maximum day demands in 2033 under Scenario 1. Under Scenario 2, the Town is estimated to have a water supply deficit of 0.38 MGD during maximum day demands in 2033. Based on data presented in Figure 6-2 and using the more conservative demand projections in Scenario 1, the Town's existing water supplies will not produce a sufficient volume of water to meet the maximum day demand projections as of the year 2021. An additional, water supply will be necessary to supplement the Town's sources in the summer when maximum day conditions occur. However, using the demand projections from Scenario 2, the Town's existing sources (assuming that a treatment plant is constructed to treat Wells No. 4 and No. 5) can provide a sufficient volume of water to meet the Town's water needs during maximum day conditions in 2033 and beyond.

The data presented in Figure 6-2 demonstrates the important role that water conservation will play in the Town's ability to meet future water demands. In Scenario 1 at 65 RGPCD, the Town will need to request an increase in the WMA permit withdrawal to comply with future average day demands. The Town will need to find a new water source to supplement its current water supply to meet future maximum day demands when the Town's largest source is out of service.

In Scenario 2 at 55.1 RGPCD, the current authorized withdrawal in the WMA permit is sufficient for the Town to meet future average day demands and an increase in the WMA permitted withdrawal is not necessary. In Scenario 2, the Town has a surplus of water during maximum day demands when the largest source is out of service and a new water source is not needed.

6.3 Water Quality

In addition to the Town being able to provide a sufficient volume of water to its customers, the water supplied will have to meet all appropriate federal and state drinking water standards and regulations. The Town will have to provide the necessary treatment to meet these water quality requirements.

6.4 Drinking Water Regulations

Drinking water regulations have been established to protect the health of customers consuming the public water supply. Surface water supplies generally have to meet more regulations and follow more guidelines than ground water sources. The following list summarizes the major drinking water rules and the major components included in each rule.

6.4.1 Surface Water Treatment Rule (SWTR) and Interim Enhanced Surface Water Treatment Rule (IESWTR)

- Applies to public water systems supplied by surface water or ground water under the direct influence (GWUDI) of surface water.
- IESWTR is an amendment to the SWTR that applies to systems that serve at least 10,000 people.
- WTP must achieve a 99 percent (2-log) removal of Cryptosporidium, 99.9 percent (3-log) removal of Giardia cysts and 99.99 percent (4-log) removal of viruses.

- Disinfectant residuals entering the distribution system have to be monitored continuously and cannot be less than 0.2 mg/L for more than 4 hours.
- Combined filter effluent turbidity must be measured at least once every four hours, and turbidity levels must be less than or equal to 0.3 NTU for at least 95 percent of the measurements per month with no turbidity samples exceeding 1 NTU at any time.
- Established disinfection contact time (CT) requirements based on water temperature, pH, and inactivation requirements for various disinfectants including ozone, chlorine, chlorine dioxide, and chloramines.
- Requires that disinfection profiling be conducted by any system whose one year running annual average of TTHMs or HAA5 levels are greater than or equal to 80 percent of the MCLs. The 80 percent thresholds for TTHMs and HAA5 are 64 µg/L and 48 µg/L, respectively.

6.4.2 Long Term 2 Enhanced Surface Water Treatment Rule (LT2ESWTR)

- Applies to public water systems supplied by surface water or ground water under the direct influence (GWUDI) of surface water.
- Rule provided additional public health protection from Cryptosporidium requiring systems to monitor their source water to determine potential additional treatment requirements for Cryptosporidium.
- Systems serving greater than 10,000 people must conduct two years of sampling for Cryptosporidium, turbidity, and E. Coli. Sampling is used to classify water system into one of four different treatment categories called bins. Additional treatment may be required based on which bin a system is assigned.

6.4.3 Stage 1 Disinfection Byproduct Rule (Stage 1 DBPR)

- Applies to all public water systems.
- Set the MCL for TTHM at 80 µg/L and for HAA5 at 60 µg/L based on the running annual average (RAA) of quarterly samples.
- At least 25 percent of samples must be taken at locations with a maximum residence time within the distribution system; the remaining 75 percent of samples are collected at locations with an average residence time.

- Established requirements for Total Organic Carbon (TOC) removal from surface water and GWUDI systems using conventional treatment based on the RAA monthly raw water alkalinity and percent removals.

6.4.4 Stage 2 Disinfection Byproduct Rule (Stage 2 DBPR)

- Applies to all public water systems, but the number of required sampling locations is greater for surface water or GWUDI public water supplies.
- Requires water systems to meet "locational" running annual averages (LRAA) of 80 µg/L for TTHM and 60 µg/L for HAA5.
- Requires water system suppliers to conduct Initial Distribution System Evaluations (IDSE) to select new Stage 2 DBPR compliance monitoring locations that more accurately represent peak disinfection byproducts in the distribution system.

6.4.5 Total Coliform Rule (TCR)

- Applies to all public water systems.
- Established MCLs for the presence of total coliform in drinking water. Systems must not find coliform in more than five percent of the samples collected each month.
- The number of monthly samples collected is based on the population served.
- Each total coliform positive routine sample must be tested for the presence of fecal coliform or E. coli.
- If any routine sample is total coliform positive, at least three repeat samples must be collected and analyzed for total coliform. Repeat samples follow the same requirements of the initial routine samples.

6.4.6 Revised Total Coliform Rule (RTCR) (Effective April 1, 2016)

- Applies to all public water systems and replaces the previous Total Coliform Rule.
- Requires public water system to collect samples and measure for total coliform. The number of monthly samples collected is based on the population served.
- Each total coliform positive routine sample must be tested for the presence of E. coli.

- If any routine sample is total coliform positive, at least three repeat samples must be collected and analyzed for total coliform. Repeat samples follow the same requirements of the initial routine samples.
- Establishes Assessment and Corrective Action (A/CA) requirements when sampling indicates the presence of total coliform and/or E. Coli in drinking water.
- Systems that find coliform in more than five percent of the samples collected in a month or that fail to take every required repeat sample after any single routine total coliform positive sample are required to conduct a Level 1 Assessment. A Level 1 Assessment is a basic examination of the source water, treatment, distribution system and operational practices conducted by the Town.
- Establishes MCLs for the presence of E. Coli in drinking water based on positive routine and repeat sampling or the failure to take every required repeat sample after any single routine E. Coli positive sample. E. Coli MCL violations or two Level 1 Assessment Triggers in a 12-month period require the public water system to conduct a Level 2 Assessment. A Level 2 Assessment is a more detailed examination of the system and its monitoring and operation practices conducted by the State or party approved by the State.

6.4.7 Ground water Rule

- Applies to a public water system supplied by ground water or to a system that has both ground water and surface water sources if water from ground water sources is added to the distribution system directly without treatment.
- For a system that provides at least 99.99 percent (4-log) inactivation and/or removal of viruses from the ground water source(s), the system is required to conduct compliance monitoring to show the effectiveness of their treatment process. For systems using chemical disinfection, compliance monitoring consists of continuously monitoring disinfectant residual to maintain the minimum required concentration.
- For a system that does not provide 4-log inactivation of viruses, the system is required to sample each ground water source for E. coli if a routine TCR sample tests positive for total coliform. If the ground water source tests positive for E. coli, five additional samples from the same source shall be collected. If a repeat sample tests positive for E. coli, then the system must take corrective action.

- Corrective action can include correcting the deficiency if possible, eliminating the water source, providing an alternate source of water, or providing new treatment that achieves 4-log inactivation and/or removal of viruses.

6.4.8 Lead and Copper Rule

- Applies to all public water systems.
- Requires sampling from customer's faucets.
- Established action levels for lead of 15 ppb and copper of 1.3 ppm. If the action levels are exceeded in more than 10% of customer taps sampled, the system must undertake a number of additional actions to control corrosion.
- Requires water suppliers to optimize their treatment system to control corrosion in customer's plumbing;
- Requires sampling of sources to rule out the source water as a significant source of lead or copper.
- If lead action levels are exceeded, suppliers are required to educate their customers about lead and suggest actions they can take to reduce their exposure to lead through public notices and public education programs and may have to replace lead service lines under their control.

As detailed in the previous summary, there are several rules and regulations governing drinking water for public water supplies. However, due to the various contaminants typically present in surface water, the regulations are more extensive for a surface water or GWUDI supply compared to a ground water supply. The requirements of the rules and regulations were considered when evaluating the Town's future water supply alternatives as some regulations may make certain alternatives more difficult to implement.

6.4.9 Manganese Regulations

The United States Environmental Protection Agency (EPA) and MassDEP have established public health advisory levels for manganese. Drinking water may naturally contain manganese, and when concentrations are greater than 0.05 mg/L, the secondary maximum contaminant level (SMCL), the water may be discolored and taste bad. Over a lifetime, EPA recommends that people drink water with manganese levels less than 0.3 mg/L, and over the short term, EPA recommends that people limit their consumption of water with levels over 1.0 mg/L, primarily due to concerns about possible neurological effects. Children up to 1 year of age should not be given water with manganese concentrations over 0.3 mg/L, nor should formula for infants be made with that water for longer than 10 days.

The MassDEP recommends that Public Water Suppliers (PWS) test at least annually for secondary contaminants including manganese. A PWS shall conduct initial baseline sampling (consecutive quarters) for manganese at each individual source. If detected above the aesthetics-based SMCL of 0.05 mg/L, the MassDEP recommends the PWS routinely monitor and gather sufficient information to assess iron and manganese levels at affected sources that may account for fluctuations in levels above the SMCL, including pumping rates, blending patterns, periodic/seasonal use, and variations in seasonal water quality.

When combined iron and manganese levels are less than 1.0 mg/L, sequestering the iron and manganese is an approved means of treatment by the MassDEP. In addition, due to the implementation of the recent lifetime health advisory level of 0.3 mg/L and the potential to incorporate this level in future manganese regulations, we do not recommend planning for sequestering water with manganese levels greater than 0.3 mg/L. Sequestering does not remove the contaminant and the potential health risks associated with elevated levels of manganese are still present. When combined iron and manganese levels exceed 1.0 mg/L (or if manganese levels exceed 0.3 mg/L) at a specific source, a means other than sequestering such as blending with another source with low levels of manganese or treatment for removal of the manganese is required. If manganese concentrations exceed 1.0 mg/L at a source, treatment for removal of the manganese is required and blending with another source is no longer an option.

6.5 Water Quality of Existing Water Supplies

The water quality from the five active ground water sources was reviewed for the years 2009 through 2012. Well No. 7 and Well No. 8 were recently constructed and not active during this period. Many of the Town's water supplies have elevated levels of iron and manganese. The iron and manganese concentrations from samples collected from the wells are shown in Table 6-2. Although the MassDEP recommends annual sampling at a minimum, the Town is not required to sample for secondary contaminants every year, so iron and manganese data was not collected for every well each year.

We recommend that the Town begin a monthly sampling program to monitor the raw water at the wells for iron and manganese. This information will be important in the future should the Town want to consider constructing a filtration plant to remove iron and manganese from the wells. It is not prudent to make major filtration decisions with limited water quality information. We have discussed general iron and manganese water quality with Town personnel who report that generally the water quality of the Fruit Street Wells (Wells No. 1, 2, and 6) is better than the water quality of the Whitehall Wells (Wells No. 4 and 5)

The iron and manganese secondary maximum contaminant levels (SMCL) set by the MassDEP are 0.3 mg/L and 0.05 mg/L respectively. The MassDEP sets SMCLs not for health reasons, but for aesthetic reasons. However, the EPA and MassDEP have recently established health advisory levels for manganese including a lifetime exposure level of 0.3 mg/L and an acute exposure level of 1.0 mg/L. These health advisory levels may lead to the EPA and/or MassDEP setting primary maximum contaminant levels (MCL) for manganese in the future.

Iron and manganese levels at Well No. 1 and Well No. 6 are not a concern for the Town. All samples measured at Well No. 1 and Well No. 6 were below the detectable limits (ND) for iron and manganese with the exception of one manganese sample at Well No. 1 which was below the SMCL for manganese at 0.003 mg/L.

TABLE 6-2
IRON AND MANGANESE LEVELS, 2009-2012

DATE	Well No. 1		Well No. 2		Well No. 4		Well No. 5		Well No. 6	
	Fe (mg/L)	Mn (mg/L)	Fe (mg/L)	Mn (mg/L)	Fe (mg/L)	Mn (mg/L)	Fe (mg/L)	Mn (mg/L)	Fe (mg/L)	Mn (mg/L)
2/12/2009			ND	0.16	0.3	0.11				
4/30/2009	ND	ND	ND	0.22	ND	0.03	ND	0.02		
6/4/2009									ND	ND
11/12/2009				0.47		0.149				
2/4/2010				0.249		0.016				
5/6/2010	ND	0.003	ND	0.3	0.76	0.07	0.19	0.007	ND	ND
7/21/2010				0.34		0.094				
10/27/2010				0.62		0.2				
1/6/2011			ND	0.396	ND	0.098				
4/6/2011		ND		0.481		0.024		0.017		ND
4/28/2011	ND	ND	ND	0.18			4.7	0.14	ND	ND
5/11/2011					0.14	0.015				
8/2/2011				0.68						
10/12/2011				0.577						
2/7/2012				0.41						
4/19/2012	ND	ND	ND	0.446	ND	0.011	5.44	0.152	ND	ND
5/8/2012							6.44	0.182		
7/12/2012				0.43						
10/3/2012				0.276						
12/6/2012			ND						ND	
Average	**ND**	**0.001**	**ND**	**0.390**	**0.20**	**0.074**	**3.35**	**0.086**	**ND**	**ND**
Minimum	**ND**	**ND**	**ND**	**0.160**	**ND**	**0.011**	**ND**	**0.007**	**ND**	**ND**
Maximum	**ND**	**0.003**	**ND**	**0.680**	**0.76**	**0.200**	**6.44**	**0.182**	**ND**	**ND**

All iron samples measured at Well No. 2 were below the detectable limit, but the manganese levels have been consistently elevated above the SMCL. The average manganese level at Well No. 2 between 2009 and 2012 was 0.39 mg/L with a maximum manganese level of 0.68 mg/L measured in August 2011.

The average iron level at Well No. 4 between 2009 and 2012 was below the SMCL, but some samples exceeded the SMCL. The average iron level at Well No. 4 was 0.2 mg/L with a maximum iron level of 0.76 mg/L measured in May 2010. The average manganese level at Well No. 4 of 0.07 mg/L was greater than the SMCL, and a maximum manganese level of 0.2 mg/L was measured in October 2010.

The manganese levels at Well No. 5 are similar to those at Well No. 4 with the average manganese concentration of 0.09 mg/L exceeding the SMCL and a maximum manganese level of 0.18 mg/L measured in May 2012. However, the iron levels are significantly higher at Well No. 5 with an average level of 3.35 mg/L and a maximum level of 6.44 mg/L measured in May 2012. The average iron levels at Well No. 5 exceed the SMCL for iron.

In summary, iron levels are elevated at Well No. 4 and Well No. 5, but are a greater concern at Well No. 5 since the average level is more than ten times greater than the SMCL. Average manganese levels exceed the SMCL at Well No. 2, Well No. 4 and Well No. 5 with the highest levels of manganese observed at Well No. 2. The average manganese levels at Well No. 4 and Well No. 5 are just above the SMCL, while the average level at Well No. 2 is approximately eight times greater than the SMCL.

The Town currently sequesters at Well No. 4 and Well No. 5 with sodium silicate. At Well No. 5 where iron levels are the greatest and at Well No. 2 where manganese levels consistently exceed 0.3 mg/L, sequestering is not a sufficient form of treatment. High levels of iron and manganese are a major concern in the Town's drinking water sources. As a result of the high levels of iron and manganese at Well No. 4 and Well No. 5, these wells are not used during normal operations to meet average day demands. During the summer when demands are high, the Town may activate Well No. 4 as needed, but avoids activating Well No. 5 due to its poor water quality. The sodium silicate also serves to increase the pH for corrosion control. The plant Operators complain about the sodium silicate building up and clogging the lines, the high price and ineffectiveness. They have requested that the sodium silicate be removed from the station and converted to KOH or NaOH for pH adjustment. This conversion will require an engineering design, DEP approval, and construction compliance with the new Chapter 6 Chemical Feed requirements for a critical chemical. We have estimated the cost for this work to be between $35,000 and $50,000. This assumes that the bulk and day tanks can be installed within the existing buildings.

Color or total organic carbon (TOC) at each well source was not measured between 2009 and 2012. Color is often an indicator that organic substances are present in the water. We recommend that the Town begin a sampling program to monitor TOC and color every other month in the raw water at each well source.

Other contaminants have been detected in the five well sources, but the concentrations are below the maximum contaminant levels (MCL) for each substance and currently do not pose health risks to the public consuming the drinking water. Nitrate has been detected with an average concentration of 1.4 mg/L from the five well sources from 2010 through 2012. The maximum nitrate level measured during the three year period was 2.3 mg/L at Well No. 1 in 2012, below the MCL for nitrate of 10 mg/L.

Samples from each well source were collected and tested for perchlorate in 2011, and all samples measured below the detectable limit of 0.05 µg/L. The MCL for perchlorate is 2.0 µg/L.

Sodium has consistently been detected at all well sources with an average concentration of 45 mg/L measured from the five well sources in 2012. The maximum sodium level measured in 2012 was 65 mg/L at Well No. 2. Sodium is not regulated, but the Massachusetts Office of Research and Standards has set a guideline concentration (ORSG) of 20 mg/L for sodium. The ORSG for sodium was exceeded at all five well sources, but the levels measured from the Town's water supplies are not uncommon for ground water wells in New England.

VOCs detected in the Town's sources between 2010 and 2012 were bromoform, chloroform, bromodichloromethane, chlorodibromomethane, and methyl tert-butyl ether (MTBE). With the exception of MTBE, these VOCs are trihalomethanes that are not regulated as individual contaminants, but the combined concentration of these four contaminants make up the total trihalomethanes (TTHMs) concentration which is regulated in the distribution system. TTHMs are discussed in the next section with the water quality analysis for the distribution system.

The 3-year average between 2010 and 2012 for the VOCs detected in the Town's sources are shown in Table 3-3 by well source. Bromoform was detected at Well No. 5 with a maximum level of 1.7 µg/L measured in 2011. Chloroform was detected at Wells No. 2, No. 4, and No. 5 with a maximum level of 21 µg/L measured at Well No. 4 in 2011. Bromodichloromethane was detected at Wells No. 4 and No. 5 with a maximum level of 3.8 µg/L measured at Well No. 5 in 2011. Chlorodibromomethane was detected at Well No. 5 with a maximum level of 4.5 µg/L measured in 2011. MTBE was detected at Well No. 6 with a maximum level of 1.5 µg/L measured in 2010. The Massachusetts Office or Research and Standards has set a guideline concentration (ORSG) of 70 µg/L for MTBE. Based on the water quality sampling conducted in 2010 through 2012, low levels of multiple VOCs are consistently detected at Wells No. 4 and No. 5. However, the levels of VOCs measured from the Town's water supplies are not uncommon for ground water wells in New England.

TABLE 6-3
2010 – 2012 AVERAGE VOLATILE ORGANIC COMPOUND LEVELS

VOC	Well 1 (µg/L)	Well 2 (µg/L)	Well 4 (µg/L)	Well 5 (µg/L)	Well 6 (µg/L)
Bromoform	ND	ND	ND	1.2	ND
Chloroform	ND	1.4	7.4	3.9	ND
Bromodichloromethane	ND	ND	0.7	3.5	ND
Chlorodibromomethane	ND	ND	ND	3.8	ND
MTBE	ND	ND	ND	ND	1.3

We understand that the Town's active ground water sources are not considered to be ground water under the influence of surface water (GWUDI). In the 1990s and also for newer wells, the DEP requires that wells meet certain distance requirements from surface water or the DEP requires that a well be sampled for microscopic particulate analysis (MPA). An MPA test includes an analysis for certain surface water pathogens and substances. If a well pulls water from a nearby surface water supply and cannot essentially be filtered adequately through the aquifer material, it may contain surface water pathogens that are best removed with filtration. Because all surface waters are required to be filtered (with some rare exceptions), if a ground water is under the influence of surface water, then surface water filtration techniques must be employed. We recommend that the Town confirm that their wells are not GWUDI, by determining distances to surface water and potentially conducting MPA testing, if necessary. This is important because it would be necessary to employ a surface water treatment system rather than a ground water treatment system, if a well is GWUDI.

6.6 Water Quality in the Distribution System

Chlorination has made public water supplies safe from illness-producing bacteria, viruses, and parasites. However, using chlorine as a disinfectant introduces its own health risks because of the byproducts produced during the disinfection process. Disinfection byproducts (DBPs) form when chlorine reacts with naturally occurring organic matter (NOM) and naturally occurring inorganic compounds in water. It has been found that the resulting DBPs pose a health threat when consumed over long periods of time. DBPs are currently regulated in two groups: total trihalomethanes (TTHMs) and haloacetic acids (HAA5s).

A summary of the samples collected and measured for DBPs between 2010 and 2012 is shown in Table 3-4. The Town of Hopkinton was required to sample for DBPs once annually in the third quarter at three (3) sampling sites, including 229 Hayden Rowe Street, 57 Oakhurst Road, and 85 Wood Street (Department of Public Works). The table includes the highest quarterly average, highest single sample, and the number of single samples exceeding the MCL over the 3-year period.

TABLE 6-4
SUMMARY OF DISINFECTION BYPRODUCTS (2010 – 2012)

Substance	MCL (µg/L)	Highest Quarterly Average (µg/L)	Highest Single Sample (µg/L)	Number of Samples Exceeding MCL
Total Trihalomethanes	80	36.6	84	2[1]
Haloacetic Acids	60	6.0	7.8	0

1. The total trihalomethanes measured at the 229 Hayden Rowe site exceeded the MCL in 2010 and 2012.

HAA5s have not been a problem between 2010 and 2012 with the highest quarterly average of 6.0 µg/L, below the MCL of 60 µg/L. The highest single sample for HAA5 was also below the MCL at 7.8 µg/L measured in the third quarter of 2010.

The TTHM values have been consistently higher than the HAA5 values, although the MCL for TTHM does allow for a higher quarterly running annual average at 80 µg/L. Since the Town only samples and reports DBPs in the third quarter each year, a quarterly running annual average cannot be determined as information from the previous three quarters is not available. DBPs are highest in the late summer/early fall. Reporting the quarterly average for only the third quarter results in a value that is higher than what the quarterly running annual average would be if samples were collected every quarter.

The highest quarterly average for TTHM measured at the three sites between 2010 and 2012 was 36.6 µg/L. This occurred in the third quarter of 2012, the same quarter in which the highest single sample for TTHM was measured at 84 µg/L. The quarterly average of the three sites never exceeded the MCL during this 3-year period, but the highest single sample measured in the Town's distribution system exceeded the MCL by 4 µg/L. The TTHM level measured at 229 Hayden Rowe Street exceeded the MCL in 2010 (81 µg/L) and in 2012 (84 µg/L).

Although the quarterly average of all sample sites never exceeded the MCL and no violations occurred, exceeding the MCL at any single sampling point may be a potential water quality concern for the Town. As of October 1, 2013, the Town is required to comply with the Stage 2 DBP Rule meaning the MCLs for TTHMs and HAA5s are no longer based on the running annual averages for all sampling sites, but instead are based on locational running annual averages for each specific sampling site. Based on the high levels of TTHMs observed in the third quarter each year at the 229 Hayden Rowe Street site, complying with the locational running annual average at this individual site may be a concern for the Town. Since data has not been collected for quarters other than the third quarter, it is difficult to predict what the yearly location running annual average will be at this site, but the Town has observed TTHM levels that exceed the MCL in the small sample size of third quarter samples collected over the last three years. Based on the high levels of TTHMs observed, we recommend that the Town sample all of the wells every other month for total organic carbon (TOC) to help determine organic levels at each of its sources. The Town should also request TOC data from the Town of Ashland to determine organic levels from the interconnection.

The Town collects samples from 20 lead and copper sites in the distribution system. Lead and copper sampling was last conducted in 2011. The 90th percentile value for lead was 9.9 µg/L, below the MCL for lead of 15.0 µg/L. One lead sample collected in 2011 was greater than the

MCL with a lead concentration of 37 µg/L. The 90th percentile value for copper was 0.91 mg/L, below the MCL for copper of 1.3 mg/L. Similar to lead, one copper sample collected in 2011 was greater than the MCL with a copper concentration of 1.4 mg/L. The Town treats its well sources for corrosion control and has maintained compliance with the Lead and Copper Rule.

In summary, the Town's water quality meets the requirement of all the primary water quality standards (health related) at this time. However, three of the wells significantly exceed the SMCLs for both iron and/or manganese. With the new health advisory levels issued for manganese by the EPA and MassDEP and the potential for the MassDEP setting a primary MCL for manganese in the future, the wells with high manganese levels may be a particular concern for the Town. Due to the consistently high level of TTHMs measured at the 229 Hayden Rowe Street sampling site, the organic content in the Town's water supplies and disinfection practices may require further evaluation to comply with the Stage 2 DBP Rule implemented on October 1, 2013. Additional sampling is recommended for iron, manganese, total color and TOC to confirm these water quality concerns and develop a more complete and accurate summary of the Town's water quality.

7.0 WATER SUPPLY RECOMMENDATIONS – UPGRADES TO EXISTING WATER SOURCES

7.1 General

Based on the volume of water available and water quality concerns discussed in Chapter 6.0, we have identified the Town's options for making improvements to the water supply to meet the projected future water demands. The alternatives focus on the groups of wells where water quality issues exist with the goal of maximizing production from the Town's existing sources. The Fruit Street Wells consist of Well No. 1, Well No. 2 and Well No. 6 with the water quality concern being the elevated manganese levels at Well No. 2. The Whitehall Wells consist of Well No. 4 and Well No. 5, and both of these wells suffer from elevated levels of iron and manganese. The Alprilla Farm Wells constructed in 2012 consist of Well No. 7 and Well No. 8, and there are no current water quality concerns at these wells. The existing water quality from the Ashland WTP is not controlled or monitored by the Town of Hopkinton.

In addition to developing options for improving the Town's existing water supplies, we reviewed historical source water exploration reports and evaluated the potential for new water sources to supplement and/or replace some of the Town's existing sources. Potential new sources include new gravel packed ground water wells, bedrock wells, surface water supplies, and a connection to the Massachusetts Water Resources Authority (MWRA) water distribution system either through the Town of Southborough or Ashland.

7.2 Fruit Street Well Improvements

The active Fruit Street Wells consist of Well No. 1 and Well No. 2 located on the west side of Fruit Street and Well No. 6 located on the east side of Fruit Street. The historical water quality collected for Well No. 1 and Well No. 6 indicate iron and manganese levels are below the detectable limits. The sampling at Well No. 2 indicates iron levels are below the detectable limit, but the average manganese level of 0.4 mg/L between 2009 and 2012 exceeds the health advisory level of 0.3 mg/L. The maximum manganese level measured during this 4-year period was nearly 0.7 mg/L.

The Fruit Street wells are each individually pumped, treated, and discharged into the distribution system. Under current operations, water with high manganese levels from Well No. 2 is pumped into the distribution system without treatment for removal or sequestering of the manganese. With more stringent manganese regulations expected in the near future, the Town will be required to provide treatment for the removal of the manganese or blending of the water supply prior to discharging into the distribution system.

Blending the water from Well No. 2 will require the construction of a centralized facility on the Town owned Fruit Street property where the piping from Wells No. 1, No. 2, and No. 6 can be connected, treated and discharged to the distribution system in one combined pipeline. The new treatment facility could be located on the west side of Fruit Street adjacent to the existing Main Fruit Street Building and the Well No. 2 chemical feed station. Locating the facility on the west side of Fruit Street will minimize the piping modifications from Wells No. 1 and No. 2 and require piping modifications from only Well No. 6, which is currently located on the east side of Fruit Street. We are concerned that renovating the Main Fruit Street Building will be as or more costly than building a new building, so there are no cost savings in using the existing building.

As an alternative to constructing a new centralized blending facility on the west side of Fruit Street, the Town may be able to reuse the recently constructed Well No. 6 Pump Station as the new blending treatment facility. With Well No. 6 having a safe yield of 0.72 MGD, the existing sodium hypochlorite and potassium hydroxide chemical feed systems and/or chemical storage may be able to be reused for a blending facility with a capacity of approximately 1.1 MGD, which is the maximum allowable single day withdrawal from all Fruit Street wells as permitted under the Town's Water Management Act plus additional capacity for the wastewater plant recharge. Although there may be additional piping and pump modifications necessary to make use of the existing Well No. 6 Pump Station, the cost savings for reusing the building and equipment may result in a net savings from constructing a new facility on the west side of Fruit Street.

The water in the pump discharge at Well No. 2 will be treated immediately at the Well No. 2 pump with a polyphosphate chemical to sequester the manganese in solution. The water piping from the three wells will be connected at the new blending treatment facility and treated with sodium hypochlorite for disinfection. After being chlorinated, the water will be discharged through a series of looped water mains on the exterior of the treatment facility to provide the 4-

log inactivation of viruses required to comply with the Ground Water Rule (GWR). The water will return to the blending facility where it will be treated with sodium hydroxide for pH adjustment and corrosion control and discharged into the distribution system. The major improvements required to modify the existing well stations at Fruit Street and construct a new blending treatment facility on the west side of Fruit Street include the following:

- Demo the existing sodium hydroxide chemical feed system in the Main Fruit Street Building used for treatment of Well No. 1.
- Demo the existing sodium hypochlorite and sodium hydroxide chemical feed systems in the Well No. 2 Building.
- Demo the existing sodium hypochlorite and potassium hydroxide chemical feed systems in the Well No. 6 Building.
- Construct a new blending treatment facility on the west side of Fruit Street near the existing Main Fruit Street Building.
- Modify the piping from Wells No. 1, No. 2, and No. 6 so that each well pump discharge is connected in the blending treatment facility prior to the water entering the distribution system and serving any customers.
- Install a variable frequency drive (VFD) on the motor at Wells No. 1 and No. 2 to control the flow discharged from each well pump. Well No. 6 is currently equipped with a VFD for controlling the water flow discharged from the station. Controlling the flows from each well station will be critical to optimize manganese levels in the blended water.
- Construct a new polyphosphate chemical feed system at Well No. 2 to sequester the manganese in the water immediately upon being pumped from the well.
- Construct a new sodium hypochlorite and sodium hydroxide chemical feed system at the blending treatment facility.
- Construct the necessary looped piping on the exterior of the blending treatment facility to provide sufficient contact time for meeting the 4-log virus inactivation requirements of the GWR.

Prior to design or construction, we recommend the Town complete pilot testing to confirm adding the sequestering agent and blending the manganese achieves the intended water quality and prevents manganese precipitation in customers' hot water tanks. The estimated capital cost for the design and construction of a new blending treatment facility for the Fruit Street Wells located on the west side of Fruit Street is approximately $1.6 million. The cost includes the major improvements listed above and engineering fees and contingency for design and construction oversight. As previously discussed, the Town may be able to reuse the existing Well No. 6 Pump Station, along with the existing chemical feed equipment and storage tanks, as the centralized blending facility depending on the final design requirements. If the Well No. 6 Pump Station can be reused, the Town may be able to reduce the cost for the new centralized blending treatment facility significantly.

7.3 Whitehall Well Improvements

7.3.1 Pilot Study History

The historical water quality observed at Well No. 4 and Well No. 5 shows elevated levels of iron and manganese above the SMCLs for iron and manganese of 0.3 mg/L and 0.05 mg/L respectively. Between 2009 and 2012, the maximum iron and manganese levels recorded at Well No. 4 were 0.76 mg/L and 0.2 mg/L respectively. During the same time period, the maximum iron and manganese levels measured at Well No. 5 were 6.44 mg/L and 0.18 mg/L respectively. Due to the elevated levels of iron and manganese, The Town does not typically use Well No. 5 and only uses Well No. 4 to supplement its water supply during the summer when water demands are high.

In 2004, SEA Consultants conducted a pilot study on the water supply from Wells No. 4 and No. 5. For most of the testing, the water supply from Well No. 5 was used to simulate a worst case scenario as the iron levels in Well No. 5 were historically greater than the levels in Well No. 4. The final run of the pilot study included a blend of water from Well No. 4 and Well No. 5 to more accurately simulate the expected typical operations of the wells.

During the 2004 pilot study, treatment units were provided by Roberts Filter Group and included a contact clarifier, greensand pressure filter, and conventional tri-media filter. Using the equipment provided by Roberts Filter Group, four (4) treatment methods were tested including the following:

- Contact Clarification
- Greensand Pressure Filtration
- Contact Clarification Followed by Greensand Pressure Filtration
- Contact Clarification Followed by Conventional Tri-Media Filtration

Results collected during the pilot study showed direct greensand pressure filtration was not a viable option for treatment. Due to the high levels of iron in the raw water, short filter run times of approximately two hours were observed with rapid breakthrough of iron levels in the filter effluent. The use of direct greensand pressure filtration was an inefficient treatment process where the filter required frequent backwashes and the net water production was lower than the other treatment alternatives. The finished water quality for direct greensand filtration was not discussed in the pilot study report since it was not a viable treatment solution.

Contact clarification alone did not provide sufficient iron and manganese removal to be considered a viable treatment option. Although the contact clarifiers provided some removal of iron and manganese, the iron and manganese levels in the finished water remained above the SMCLs.

The remaining two treatment techniques included contact clarification followed by greensand filtration and contact clarification followed by conventional tri-media filtration. Both treatment alternatives showed effective removal of iron and manganese to levels below the SMCLs. Historical iron and manganese sampling has shown increasing levels of iron and manganese in the raw water with increased pumping / use. If iron and manganese levels continue to rise, the levels of iron and manganese in the clarified water entering the filters may be similar to the raw water quality observed during the pilot. Based on the short filter run times observed during the pilot testing conducted using direct greensand filtration, the resulting filter run times through the greensand filters may be reduced to 2 hours or less making contact clarification followed by greensand filtration no longer a viable solution.

Based on the results of the pilot study, SEA Consultants recommended a full scale 0.75 MGD water treatment plant (WTP) that included contact clarification followed by conventional tri-media filtration. The contact clarification followed by greensand filtration and contact clarification followed by conventional tri-media filtration produced a similar finished water quality. SEA may have recommended the contact clarification followed by conventional tri-media filtration due to concern of increasing levels of iron and manganese in the raw water as the wells are pumped more vigorously which would cause significantly shorter filter run times through the greensand filters. The media costs for greensand filtration are greater than those for conventional tri-media filtration so the tri-media filtration process produces a similar finished water quality at a lower cost.

In February 2005, SEA estimated the cost to design and construct the facility with complete redundancy at $3.0 million. The cost included the design and construction of a 3,000 square-foot building, treatment equipment based on the results of the pilot study, the associated chemical feed systems, site work including residuals handling structures, and electrical work.

The additional yearly operation and maintenance costs were estimated in the pilot study at approximately $50,000. The additional costs included chemicals not previously used at Wells No. 4 and No. 5, residuals disposal, and power costs. The labor and maintenance costs to operate the new WTP were considered similar to the Town's costs associated with operating the two individual well stations and were not included in the additional yearly costs.

7.3.2 Future Treatment Options

Included below is a summary of the treatment processes that may be options to treat water in the future from the existing ground water supplies. If new sources are developed, ground water or surface water, the WTP options may need to be re-evaluated if the raw water quality changes. These treatment options should allow the Town to meet the required primary and secondary water quality standards and regulations, but some options will be more viable than others based on costs, treatment effectiveness, and net water production. Any treatment option will need to be pilot tested to confirm that it provides the necessary treatment for the particular water source.

7.3.3 Conventional Treatment

Conventional treatment consists of chemical addition followed by rapid mix, flocculation, sedimentation, and filtration. Typical chemicals added for conventional treatment include, but are not limited to, aluminum sulfate (alum), ferric chloride, polyaluminum chloride, powdered activated carbon and sodium or potassium hydroxide. The alum, ferric chloride or polyaluminum chloride will coagulate suspended particles into a settleable floc. The powdered activated carbon will reduce dissolved organic material such as taste, odor and color compounds, and will be removed during the sedimentation stage. Hydroxide is added to adjust pH for coagulation.

Rapid mixing after chemical addition provides an initial interaction period between the chemicals and the targeted constituents. Typically, rapid mixing consists of a short detention time (approximately 30 seconds) and a mechanical process that adds a significant amount of energy to the water.

After rapid mixing, flocculation allows for continued interaction between the chemicals and the targeted constituents. Detention times are greater than rapid mixing but less than sedimentation (approximately 15 to 30 minutes). Mechanical processes used during this stage add a predetermined amount of energy to the water that promotes the formation of floc. Floc formation is essential to achieve settling during the sedimentation stage.

The sedimentation process allows the floc to settle. Sedimentation basins are often long, rectangular basins that are designed based on the settling velocity of a targeted particle size. The maximum flowrate entering the basin is determined and the critical settling velocity is calculated. Once these two values are known, the area of the basins can be sized with typical detention times being two to four hours. To enhance the settling efficiency, settling devices such as tube settlers or plate settlers can be installed to maintain a comparable water quality entering the filtration stage while also allowing for smaller sedimentation basins. The use of dissolved air flotation (DAF) is another treatment process the Town can implement instead of gravity settling to produce the necessary water quality prior to filtration in a smaller overall footprint than a standard sedimentation basin.

Residuals that are effectively settled in the sedimentation basins or floating on top in the DAF basins need to be removed from the bottom or top of the basin periodically. The method that is used needs to ensure that disturbances are minimized so that particles do not become re-suspended. A flight and chain system or a suction system is a common method used for residuals disposal from a sedimentation basin. For a flight and chain system, the operation is most efficient if the direction of travel of the flights is in the opposite direction of the flow through the basin. The residuals are transported away from the outlet zone thereby reducing the solids loading on the filters. The sumps should be located in the inlet zone for this form of residuals management. As an alternate to a flight and chain system, a residuals suction system can be used. This operation employs a roving vacuum installed on tracks on the bottom of the basin for the removal of residuals. The sludge is located on top of the water in a DAF basin, and a skimmer system is typically used to force the sludge layer into a waste trough where it can be collected and disposed of properly.

After settling or DAF, a filtration stage is necessary to physically remove particles that have not been removed prior to this step. Different forms of media exist to achieve the desired level of filtration. A combination of one or more different granular media sizes, known as mono-, dual-, or multi-media filtration may be used. Anthracite coal, garnet, and sand are often used. The media is placed over an underdrain system. A gravel bed may be used to disperse the water before being collected by underdrain piping. However, an alternative underdrain technology may be considered that eliminates the need for a gravel bed as part of the collection system. During backwashing, an air-wash system should be used. The addition of air aids in the scouring of filter media granules and the removal of particulates trapped in the filter bed.

Conventional treatment is a proven water treatment technology that has been used with New England surface and ground waters. It provides good water treatment capabilities with variable raw water quality. Generally, conventional treatment provides more flexibility than other treatment technologies. However, the footprint for conventional treatment is the largest of the alternatives resulting in a higher capital cost. Due to its treatment flexibility and ability to treat water with high levels of iron and manganese, conventional treatment will likely be an effective process for the treatment of water from Well No. 4 and Well No. 5 in Hopkinton.

7.3.4 Pressure Filtration

Pressure filtration is commonly used in New England to treat ground water with elevated levels of iron and manganese. Generally, pressure filtration is not used to treat surface water. The media used in pressure filtration typically includes an upper layer of anthracite for rough particulate removal followed by a layer of catalytic media specifically designed for iron and manganese removal. The lower level of the filter is usually equipped with a gravel support media above the underdrain system. Three viable media options include Greensand Plus, LayneOx, and Pureflow. The filter vessels are either vertical or horizontal cylindrical steel tanks.

The influent water supply is pre-chlorinated for oxidation of the dissolved iron and manganese in preparation for removal through the pressure filters. In some applications, the media also requires the addition of potassium permanganate for the oxidation of manganese and continuous media regeneration. The typical filter loading rate ranges from 3 to 10 gpm/ft2, depending on water quality. The filters are equipped with a backwash system. Some manufacturers also employ an air scour system. Replacement of filter media is typically required every 10 to 15 years.

If there are elevated color and organic levels, the addition of a small amount of coagulant prior to filtration, such as alum, may help to remove the organics during filtration. The use of alum is not necessary for most ground water sources with low levels of color and organics, but for certain ground waters which can have elevated levels of color and organics depending on the season, a coagulant may be necessary. The potential need for a coagulant can be determined by sampling for and evaluating color and total organic carbon levels and verified during pilot testing.

Pressure filtration is a proven technology for treating ground water with elevated levels of iron and manganese. Pressure filtration can be used for treating water with elevated levels of TOC and color, but a coagulant may be needed to help remove the additional organics during filtration. The additional organics and coagulant added will increase residuals and potentially shorten filter run times. For waters known to have higher levels of organics, a clarification step prior to pressure filtration may provide an advantage over direct pressure filtration.

For the specific application for treatment of water from Well No. 4 and Well No. 5, the pilot study conducted by SEA Consultants showed that direct pressure filtration was not a viable option due to the high levels of iron and short filter run times. Direct pressure filtration is effective for removal of iron and manganese, but is typically not used for treating water with iron levels as high as those observed in the raw water from Well No. 5 in Hopkinton.

7.3.5 <u>Membrane Filtration</u>

Membranes separate particles from water using a simple sieving process. The pore openings of the various available membrane media determine which application the membranes will be used for. In drinking water treatment in New England, ultrafiltration and microfiltration membranes are common.

Membrane filtration can either be a pressure driven or vacuum driven process. The way in which a membrane separates solids will also define the type of membrane process. Raw water can flow inside the membrane media and be forced out via pressure for what is known as an inside-out process. Target particles are trapped on the inside of the membrane. Raw water can also flow on the outside of the membrane media and be forced into the hollow structure. This is known as the outside-in process and causes particles to be trapped on the outside of the membrane. Pressure membranes can employ either method. Vacuum membrane filtration uses the outside-in process.

Membrane designs are based on a parameter known as the flux rate. The flux rate is the rate of finished water produced per square foot of membrane area. Once the flux rate is determined, the required membrane area can be calculated. If the quality of the water is poor, a lower flux rate is used, thereby increasing the required membrane area and increasing the footprint of the process. The same membrane area footprint is typically required for either the pressure or the vacuum membrane alternatives.

One of the major differences between the vacuum and pressure alternatives is the housing. Pressure membranes are housed in a pressure vessel (canister). Vacuum membranes are housed in an open tank. Pressure membrane housing is generally long and tubular in structure. The system arrangement includes piping and valving connecting a number of individual membrane bundles within canisters. Vacuum membranes are setup on a rack with no piping or valving separating the membrane filters, and are immersed in a tank.

The energy requirements of the two systems may vary. A closed pressure vessel is needed in pressure membrane filtration to support the higher pressure conditions during operation. In vacuum membrane filtration, the membranes are in a tank open to the atmosphere and operate under suction. The difference in operating conditions may be seen in power costs. Operating a system with suction may require less power than operating a system with pressure. It should be noted that a vacuum membrane system also employs an air-blower system during normal operation. The power costs associated with the blowers may make the overall power costs of the two alternatives comparable.

Backwashing of the membrane alternatives differ also. Both alternatives utilize a back-pulsing technique that reverses the flow of water to clean the surface of the membrane. The transport of the residual produced is what differs between the two systems. In a pressure membrane system, the residual flow exits the membrane canister through a separate residual line and is pumped to the residuals management system. In a vacuum membrane system, the residuals are pulsed off the membrane surface and fall to the bottom of the water tank that houses the membrane units. Once the concentration of residuals at the bottom of the tank reaches a certain level, the residuals are pumped out.

For water sources with elevated levels of iron and manganese and possibly color/TOC, an initial clarification step may be needed prior to membrane filtration for best removal results. Coagulation and flocculation may occur in the same basin as the membranes if used in conjunction with vacuum membrane filtration. The coagulation and flocculation processes would be similar to those detailed for conventional treatment. Chemicals are added just prior to a rapid mix stage and allowed to coagulate within a designated flocculation basin. A pin floc need only be formed due to the small pore size of the membrane media. As long as the pin floc is larger than the pore opening (0.1 micron), removal will occur through the membrane, eliminating the need for a settling stage. This is a form of direct filtration (without settling) using membrane filter technology.

Membrane filtration is a proven water treatment technology that provides high level water treatment capabilities for surface and ground water with variable raw water quality. Due to the high capital and operational costs, a membrane WTP was not considered a likely option for the Town of Hopkinton to treat water from Well No. 4 and Well No. 5.

7.3.6 Upflow Contact Clarification Followed by Filtration

Upflow contact clarification through a naturally buoyant media followed by filtration is a package plant technology that does not rely on the formation of a settleable floc. As a result, the footprint can be reduced to as low as 20 percent of the footprint of a conventional treatment system. The clarification stage uses a buoyant media to remove small floc formations from the water. Chemical addition similar to conventional treatment occurs prior to the package unit and contact time occurs in the area of the tank beneath the buoyant media. Water is forced upward through the clarifier media bed and then discharged to the downstream filters.

Pressure filters or multi-media filters are typically used for the filtration step following the upflow clarification process. Since the upflow clarifiers are operated under pressure, there is no need for repumping before the filters. Removing small floc through the upflow clarifiers reduces the strain on the filters and improves treatment and extends filter run time. Less coagulant and polymer are used than would be needed for conventional treatment.

When pressure filtration or multi-media filtration was used following contact clarification, the pilot study showed sufficient iron and manganese removal and longer filter run times than direct filtration. However, due to the potential for raw water and clarified water iron and manganese levels to increase in the future and cause shorter filter run times through the pressure filters and the higher media costs for pressure filtration compared to multi-media filtration, contact clarification followed by multi-media filtration was recommended for full scale treatment over contact clarification followed by pressure filtration.

Upflow contact clarification is most applicable to sources with consistent raw water quality. When raw water quality fluctuates or iron and manganese levels become too elevated, contact clarifiers are no longer viable options for efficient treatment as breakthrough is observed at short clarifier and filter run times. High levels of iron consistent with those observed in the raw water at Well No. 5 can be treated with the upflow clarification followed by filtration packaged units, but an increase in the recent raw water iron and manganese levels will likely require a more robust, conventional treatment with a larger footprint.

7.3.7 Biological Filtration

Biological filtration works by utilizing naturally occurring bacteria to catalyze the oxidation of iron and manganese. The sand media used in the filters acts as a support for bacteria and the development of an efficient biofilm that will react with the iron and manganese resulting in their oxidation and precipitation. Air is injected into the raw water in order to maintain the proper dissolved oxygen (DO) concentration in the water during treatment. The pH of the raw water often requires adjustment with sodium hydroxide or similar chemical for the proper operation of the biological process.

The biological filters can be pressurized or open to atmosphere. For a water supply that requires removal of both iron and manganese, one filter for iron removal and one filter for manganese removal may be required. The biological media has been shown to have a higher metals retention that allows for longer filter runs than other media used in non-biological pressure filtration that rely strictly on physical and chemical reactions. Due to rapid biological oxidation rates, filtration rates can be higher than with some of the other treatment options, allowing for smaller filters and smaller overall footprints.

The biological filtration process has been shown to treat water with high levels of iron and manganese, up to 20 mg/L of iron and 3.0 mg/L of manganese, and remove the iron and manganese to levels below the SMCLs. With the high levels of iron and manganese observed at Well No. 4 and Well No. 5, a treatment technology with a high metals retention will be needed to prevent breakthrough issues and short filter run times in the future if iron and manganese levels continue to rise. The biological filtration may be an effective treatment solution for the Town due to its ability to adapt to varying raw water quality.

7.4 Future Pilot Study Recommendations

The five treatment options listed in this report may provide the necessary treatment required to comply with primary and secondary water quality standards and regulations, but some options are less advantageous based on costs, treatment effectiveness, and net water production. Due to the high levels or iron present in Well No. 5, direct pressure filtration will result in short filter run times and is not a viable treatment solution. Membrane filtration is effective for removal of iron and manganese, but due to the high capital and operational costs, a membrane filtration WTP is not considered a viable treatment alternative.

Upflow contact clarification followed by filtration is applicable to sources with consistent raw water quality and lower levels of iron and manganese. The high levels of iron in the raw water at Well No. 5 can be treated with the upflow clarification followed by filtration, but any future increase in the iron and manganese levels will likely require a more robust, conventional treatment with a larger footprint. In a similar situation, a community near Hopkinton with high levels of iron and manganese piloted upflow contact clarification followed by filtration and achieved sufficient iron and manganese removal. After the full-scale units were constructed and the WTP was in operation, iron levels continued to rise to a point where the upflow contact clarification followed by filtration treatment process was no longer sufficient due to quick breakthrough and short run times. The community has since conducted additional pilot testing and is converting to a DAF conventional treatment process to better treat the changing water quality.

Based on the recent water quality observed at Well No. 4 and Well No. 5, the limitations and costs of direct pressure filtration, membrane filtration, and upflow clarification followed by filtration make these treatment options not viable solutions for the Town. Conventional treatment and biological filtration may be the two best treatment alternatives based on the Town's existing raw water quality and the ability for these treatment processes to remove even higher levels of iron and manganese in the future. We recommend the Town conduct side by side pilot testing using a conventional treatment process and biological filtration process to compare the results of the two treatment alternatives. If the pilot study shows biological filtration provides similar finished water quality to conventional treatment, a smaller footprint biological filtration WTP may cost less to construct than a larger, conventional WTP. The cost to conduct the pilot study, including engineering fees to oversee the pilot and complete the final report submitted to the MassDEP, is approximately $125,000. Given the significant potential for cost savings between the two alternatives, the Town may want to just pilot the biological filtration process to identify if it is a viable treatment for Well #4 and #5 water which would allow the costs to build a treatment plant to be further refined.

TABLE 7-1

WTP PILOTING COSTS

Description	2013 Cost
Conventional (DAF)	$75,000
Biological Filtration	$50,000
Total	$125,000

7.5 Future Water Treatment Plant

The water from Well No. 4 and Well No. 5 is only pumped during periods of high demand in the summer to supplement the Town's other sources due to the high levels of iron and manganese. If the Town constructed a WTP for treating this water, it would be able to use the water on a daily basis and potentially not have water supply shortages over the 20-year planning period depending on how much Hopkinton grows. Depending on the results of the future pilot study, a conventional or biological filtration WTP can be designed and constructed to handle the potential increasing levels of iron and manganese in the raw water as previously discussed.

The site plan for the Whitehall Wells is shown in Figure 7-1. The Town owns a small parcel, approximately 3.7 acres, on the south side of the Wells No. 4 and No. 5. The remaining land around the wells and within the 400-foot protective radius is owned by the Massachusetts Department of Conservation. Due to the limited Town owned property around the Whitehall Wells site, building a new WTP and the necessary residuals handling structures would be difficult. We would anticipate that 4 to 5 acres would be required to construct a treatment plant (final size depends on which type of plant is being constructed). Our experience in constructing treatment plants in state conservation land is that it is very difficult to get approval for the land, would require local conservation commission and state legislature approval and would likely require a land transfer. The type of water treatment facility (biological filtration vs. conventional/DAF plant) will also have significant impacts on the type and volume of residuals generated and how they are handled. DEP will not allow lagoons to be located within the Zone 1.

All residuals handling for a treatment plant located on the Whitehall site would need to be managed outside of the Zone 1 which would mean that if a treatment plant is constructed on this site, that the Town would need to obtain more land. The site plan for the Fruit Street Wells is shown in Figure 7-2. The Town owns 85.75 acres of land on the Fruit Street Wells site and owns all land within the 400-foot protective radius of each well. Based on the surplus of Town owned property, The Fruit Street site may be a better location for a new WTP with residuals handling facilities than the Whitehall site. If the WTP is located on the Fruit Street site, we recommend the Town consider treating water from Well No. 2, which contains elevated levels of

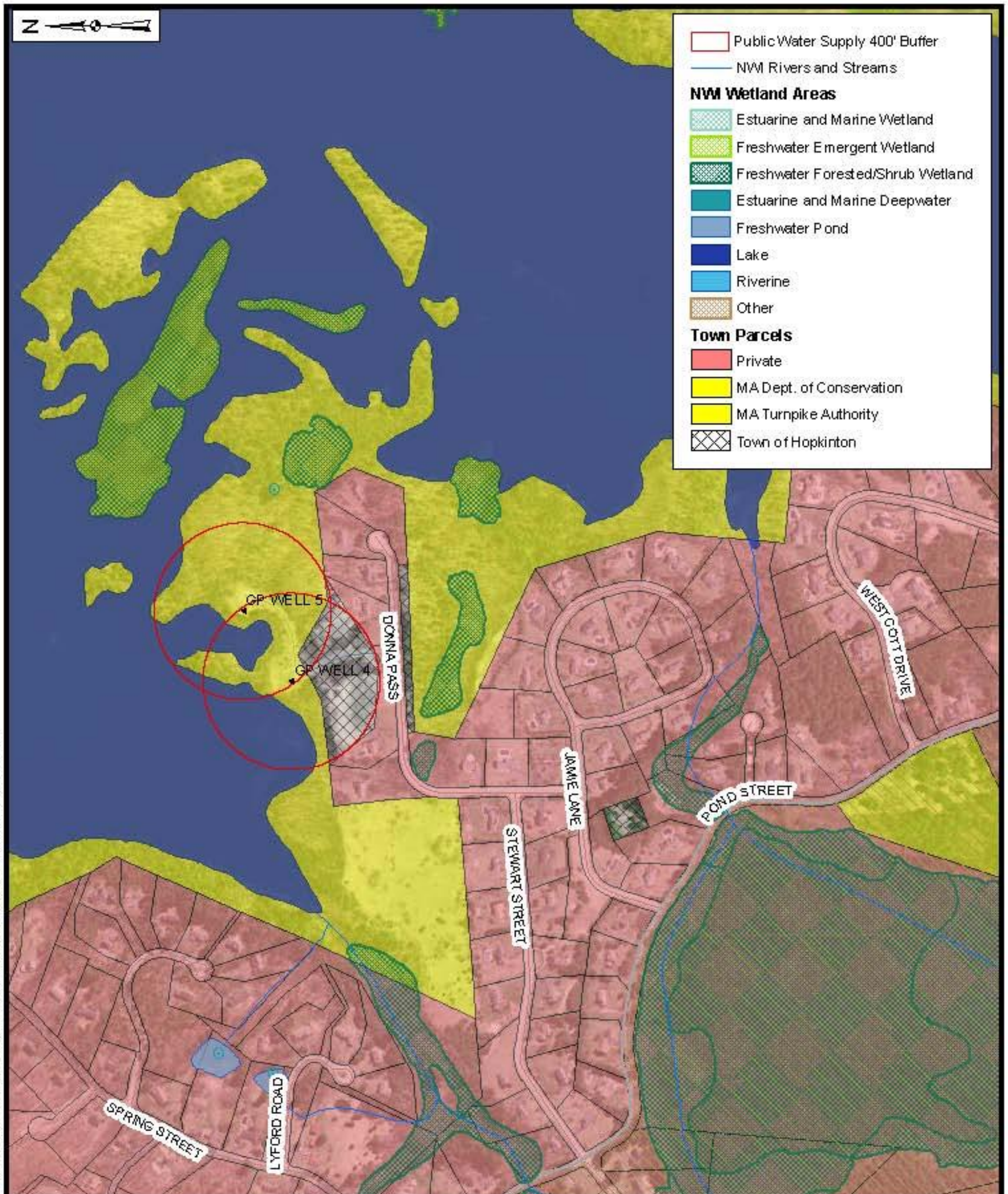


FIGURE 7-1
TOWN OF HOPKINTON, MA

WHITEHALL WELLFIELD SITE MAP

1,000 0 1,000
Scale In Feet

Weston&Sampson

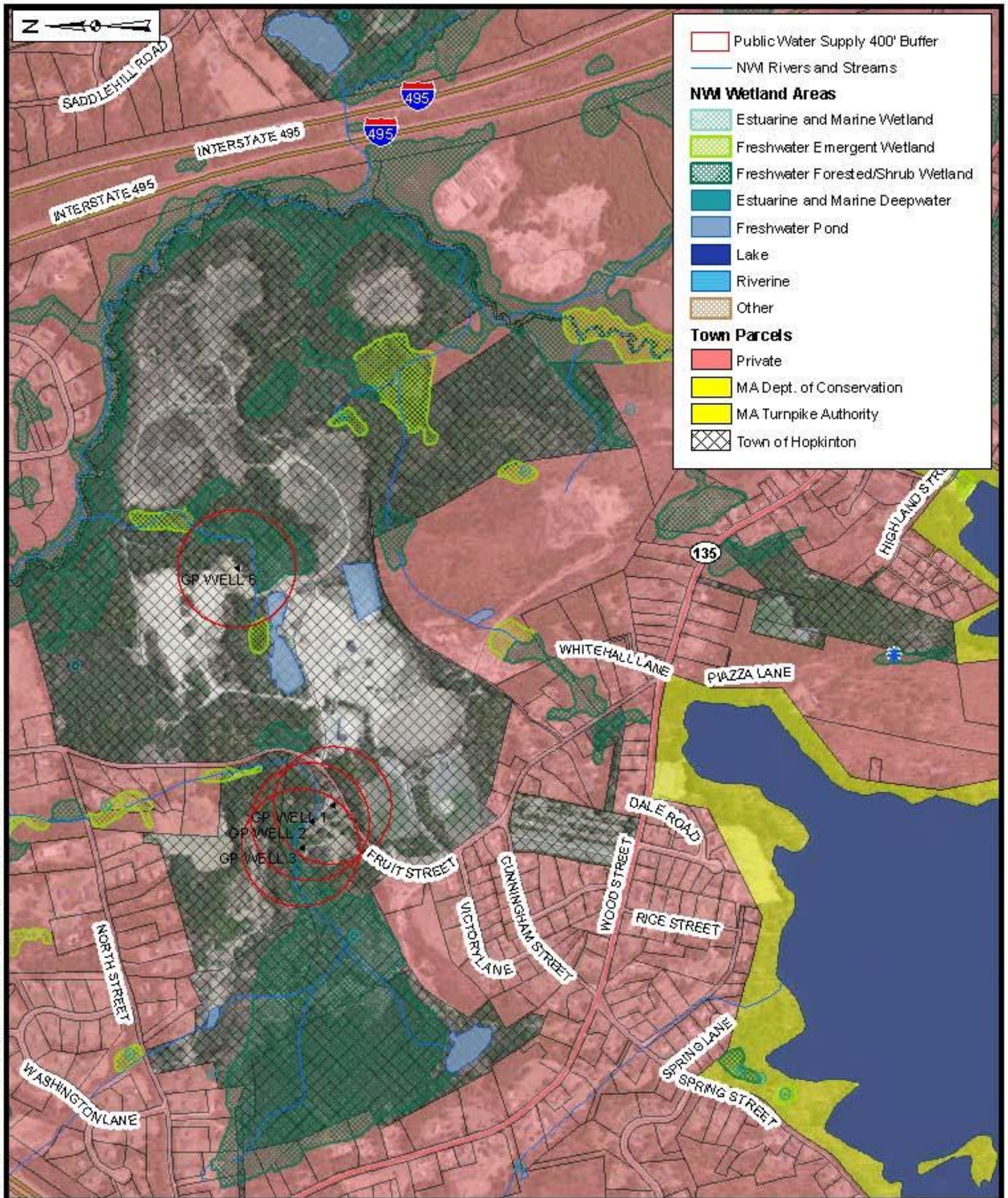


FIGURE 7-2
TOWN OF HOPKINTON, MA

FRUIT STREET WELLFIELD SITE MAP

manganese, through the WTP instead of constructing a centralized blending facility for the Fruit Street Wells as previously discussed. The blending facility would combine water from Wells No. 1, No. 2, and No. 6 in such a way to reduce the manganese concentration in the combined effluent to below 0.3 mg/L, but would not remove the manganese from the water. With a WTP constructed on the Fruit Street site, removal of the manganese from the water in Well No. 2 will provide operating flexibility in which wells are pumped, provide reduced operating costs because only one WTP will need to be operated, and will save approximately $1.5 million in constructing a separate blending facility on the Fruit Street site. Pumping the Whitehall wells to the Fruit Street site for treatment, versus on-site treatment, will provide significant operational flexibility as the wells can be rested without having to shutdown the WTP.

In addition to the surplus of Town owned property making the Fruit Street site a better location than the Whitehall Well site, residuals discharges of supernatant into the Fruit Street aquifer may be utilized through SWMI to obtain an increase in the permitted withdrawals set in the Town's WMA permit. The total safe yield for the three active Fruit Street wells is 1.35 MGD, but the current WMA permit limits the maximum daily withdrawal to 0.75 MGD. With supernatant residuals discharges back into the Fruit Street aquifer, the Town may be able to obtain a new WMA permit with a higher maximum daily withdrawal limit. The increase in the maximum daily withdrawal limit will allow the Town to utilize more water from its Fruit Street Wells instead of potentially relying on sources outside of Town to meet future demands.

Based on the safe yields for Wells No. 2, No. 4, and No. 5, the new WTP should be designed with a minimum capacity of 1.1 MGD. The estimated project cost for constructing a 1.1 MGD WTP, including engineering, construction and 25% contingencies, is between $9.2 and $14 million depending on what type of plant and treatment technology is designed and constructed. This cost is representative of a basic brick and block building with a simple roof system, a small laboratory, and minimal administrative and storage space. If the treatment plant is located at the Whitehall site, additional land would need to be purchased for the residuals handling facilities outside the Zone 1. For the purposes of comparing water supply alternatives, we have assumed that the biological filtration would be suitable for the Whitehall wells and that we could construct the water treatment plant for $9.2 million.

If a new WTP is constructed on the Fruit Street site, a new pipeline will need to be constructed for transmission of the water from Wells No. 4 and No. 5. The estimated pipeline distance is approximately 15,300 linear feet. In addition, if the new WTP is located on the east side of Fruit Street where more land is available, two new pipelines between Well No. 2 and the WTP and Well No. 1 and 3 may need to be constructed. The estimated cost for constructing the pipelines including engineering, construction and 20% contingency is approximately $3.77 million. A summary of the estimated capital costs for the pilot study, design, and construction of a new 1.1 MGD WTP on the Fruit Street site, including the necessary pipelines between the wells and the WTP, is shown in Table 7-2.

TABLE 7-2
WATER TREATMENT PLANT CAPITAL COSTS

Task	2013 Cost Biological Filtration Plant	2013 Cost Conventional Plant
Pilot Testing	$50,000	$125,000
WTP Construction & Engineering	$9,200,000	$14,000,000
#4 & #5 Raw Water Pipeline	$3,300,000	$3,300,000
Fruit St Site Piping	$470,000	$470,000
Total	**$13,020,000**	**$17,895,000**

The Town can obtain a 20-year State Revolving Fund (SRF) loan with a 2-percent interest rate for payment over a 20-year period for the capital costs. The additional yearly operations cost to treat water through a new WTP is estimated at approximately $130,000 in present worth including electricity, heating, chemicals, and one additional full-time operator. ***Based on a 2-percent increase in yearly operations costs and a present worth interest rate of 2-percent, the 20-year life cycle cost in present worth for a new biological filtration WTP ($13 million) located on the Fruit Street site, the financing and operating costs add another $5 million to the capital cost bringing the total to approximately $18 million.***

The cost to construct a WTP at the Whitehall site (Table 7-2 without pipeline costs) would be between $9 million and $14 million depending on which treatment alternative is determined viable. The biological filtration looks like a promising technology for treatment of Well No. 4 and No. 5 but cannot be firmly considered viable without piloting. This cost assumes that the Town can obtain additional land from the Department of Conservation for no cost for construction of the residuals handling facilities and plant building as needed.

7.6 Cost Estimates

Estimated costs are present day costs including design, bidding, construction administration and construction with a contingency. Weston & Sampson follows the American Association of Cost Engineer's guidelines for a cost estimating classification system. Typically there are three cost estimating stages of water engineering projects, including the conceptual/evaluation estimate, the pre-design estimate and the final design estimate. The table below summarizes the expected range of accuracy for each of the three typical phases of a project.

TABLE 7-3

COST ESTIMATION CLASSIFICATIONS

Project Maturity Level	Low Range of Expected Accuracy	High Range of Expected Accuracy
Conceptual	-15% to -30%	+20% to +50%
Pre-Design	-10% to -20%	+10% to +30%
Final Design	-3% to -10%	+3% to +15%

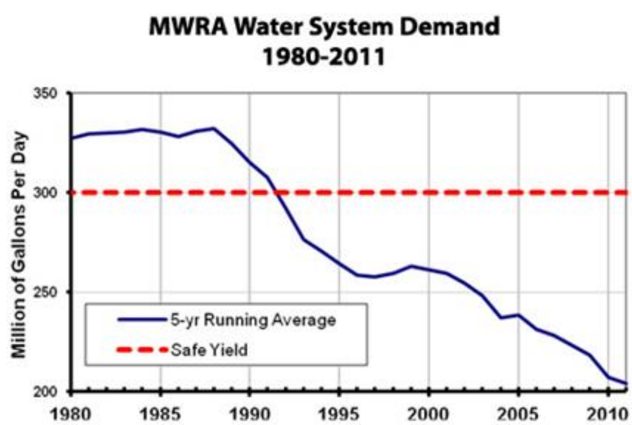
We have kept all costs in 2013 (present day) costs for the purposes of comparing water supply alternatives. The earliest a WTP would be constructed would be 2016. It will be necessary for the Town to increase these project costs based on inflation for the year of anticipated construction when / if budgeting these costs for Town Meeting.

8.0 WATER SUPPLY RECOMMENDATIONS - POTENTIAL FUTURE WATER SOURCES

8.1 MWRA Connection Evaluation

MWRA is a Massachusetts public authority established by an act of the Legislature in 1984 to provide wholesale water and sewer services to 2.5 million people and more than 5,500 large industrial users in 61 metropolitan Boston communities. The MWRA outlines the process for connecting to the MWRA in Policy #OP-10 for Admission of a New Community to the MWRA Water System. The Quabbin/Wachusett reservoir system has an estimated safe yield of 300 MGD. The projected water demands of MWRA supplied communities is estimated to remain well below the safe yield of the reservoir system, implying that the MWRA has a current surplus of water. In fact the current MWRA administration is looking to identify potential communities that will connect and believe that this as a potential solution to reduce or stabilize their wholesale water rates.

FIGURE 8-1



8.2 MWRA Admission Criteria

The admission of a new community to the MWRA system requires that a community demonstrate that the following criteria have been met:

- The safe yield of the watershed system, on the advice of the MDC, is sufficient to meet the new community's demand.
- No existing or potential water supply source for the community has been abandoned, unless the DEP has declared that the source is unfit for drinking and cannot be economically restored for drinking purposes.

- A water management plan has been adopted by the community and approved by the Water Resources Commission.

- Effective demand management measures have been developed by the community and approved by the Water Resources Commission.
- Effective demand management measures have been developed by the community, including the establishment of leak detection and other appropriate system rehabilitation programs.
- A local water supply source feasible for development has not been identified by the community or DEP.
- A water use survey has been completed which identifies all users within the community that consume in excess of twenty million gallons a year.
- Admission of the applicant community into the MWRA has received approval from the MWRA Advisory Board, the General Court, and the Governor.
- An applicant community has accepted the extension of the MWRA's water system to the community by majority vote of the Town meeting.
- Any expansion of the MWRA water service system shall strive for no negative impact on the interests of the current MWRA water communities, water quality, hydraulic performance of the MWRA water system, the environment, or on the interest of the watershed communities; shall attempt to achieve economic benefit for existing user communities; and shall reserve the rights of the existing member communities. Any evaluation of the impacts of new communities shall clearly evaluate all changes to system reliability.
- MEPA filing (Secretary of Environmental Affairs)
- Interbasin Transfer Act permit (Water Resources Commission)

8.3 MWRA Water Service Connection

If the Town of Hopkinton continues with moderate growth balanced with good water conservation efforts, they will be able to continue meeting their average and maximum day water supply needs with their current sources (providing that they construct a water treatment plant to treat wells with poor water quality).

If the Town of Hopkinton does not build a WTP to treat the Whitehall Wells, they will almost immediately find themselves in a water shortage during the summer. This will depend on many characteristics, such as how quickly Legacy Farms builds out and seasonal variations in the ground water table. If a WTP is not constructed within the next two years, the Town will likely need another water source.

It is understandable why connection to the MWRA would be of interest for the Town of Hopkinton. The MWRA has a water supply that appears to be unaffected by drought conditions. In short – they never appear to run out of water, a situation that Hopkinton has not found themselves in for the last several decades. However, this surplus of water supply does not come without cost. MWRA communities pay significantly higher water rates than non-MWRA communities. *Water rates in Hopkinton are currently approximately 60% of the average MWRA community rates.* The Town of Reading is the most recent community to abandon their ground water sources and WMA permit and connect to the MWRA for 100% of their water supply, and their water rates are over three times the Town of Hopkinton water rates. Table 8-1 is an attempt to show a representative cross section of water rates across the Commonwealth. The intent of the table isn't to demonstrate that Hopkinton has low rates, although based on the Tighe and Bond 2012 Water Rate Study, Hopkinton's annual water bill is approximately 60% of the average Massachusetts water bill, it is to demonstrate that MWRA communities pay more than non-MWRA communities as it is typically less expensive to treat the water you have in the long-term rather than buy from the MWRA. The exception to this can occur when a community needs to build a water treatment plant and carry the debt for such an expense in their water rate. Typically, communities that are constructing new treatment plants to meet water quality regulations are experiencing significant water rate increases.

TABLE 8-1
MASSACHUSETTS WATER RATES

Community	Water Rate $ per HCF	Comment
Hopkinton	$2.65	
Reading	$8.96	Recently connected to the MWRA
Gloucester	$7.18	Recently spent a lot of money on their WTPs
Belmont	$5.46	MWRA community no WTPs or tanks
Dedham-Westwood Water District	$4.17	Ground water supply, aged WTPs, MWRA connection for summer supply
Pembroke	$3.76	WTP built in 1990s, good quality ground water supply
WaterTown	$3.80	Representative of average MWRA rate
Southborough	$3.50	MWRA community
Milford	$2.69	Ground water supply currently Going through WTP upgrades

We have attempted to outline the capital and 20-year operating costs the Town of Hopkinton would need to incur to make a connection to the MWRA so that the water supply and treatment alternatives can be compared.

The closest current MWRA community to Hopkinton is Southborough Massachusetts. Ashland is currently listed as an MWRA community but is only utilizing water on an emergency basis. Ashland utilized a temporary hydrant to hydrant pumped connection to Southborough at Thomas Road in 2007 to receive water under emergency. The Town of Ashland is actively working towards making this a permanent connection. Ashland intends to utilize this connection under emergencies to supplement their water supply when needed. It may be possible for Hopkinton to purchase MWRA water through their existing interconnection with Ashland at a significant cost savings compared with constructing an interconnection directly with Southborough. If MWRA water is purchased through Ashland rather than Southborough, it is possible that Hopkinton could utilize the pumping equipment and piping infrastructure that are already in place for the treatment plant connection.

If Ashland is not successful in making a permanent installation to purchase water from the MWRA through Southborough, or if a three-way municipal agreement cannot be reached, the Town of Hopkinton would need to extend their water system to connect to the Southborough system in order to make connection with the MWRA. The Southborough hydraulic grade line is lower than the Town of Hopkinton's, which means that a pump station would need to be constructed at the Town line. The Towns would also need an inter-municipal agreement with Southborough to purchase and wheel water.

8.4 Water Supply Requirements

The Town has differing water supply scenarios of how much water they would need to purchase from the MWRA depending on water demand growth and whether the Town constructs a water treatment plant for Wells No. 4 and No. 5. Table 8-2 outlines the different water supply scenarios and the varying water supply shortages for the different scenarios with the Town's largest (Ashland) source out of service. We typically look at supplying the maximum day demand with the largest well out of service as a conservative approach to making sure the Town has enough water to meet their maximum demands. We typically consider this in lieu of calculating the anticipated reduced pumping at each facility due to low ground water tables and drought conditions.

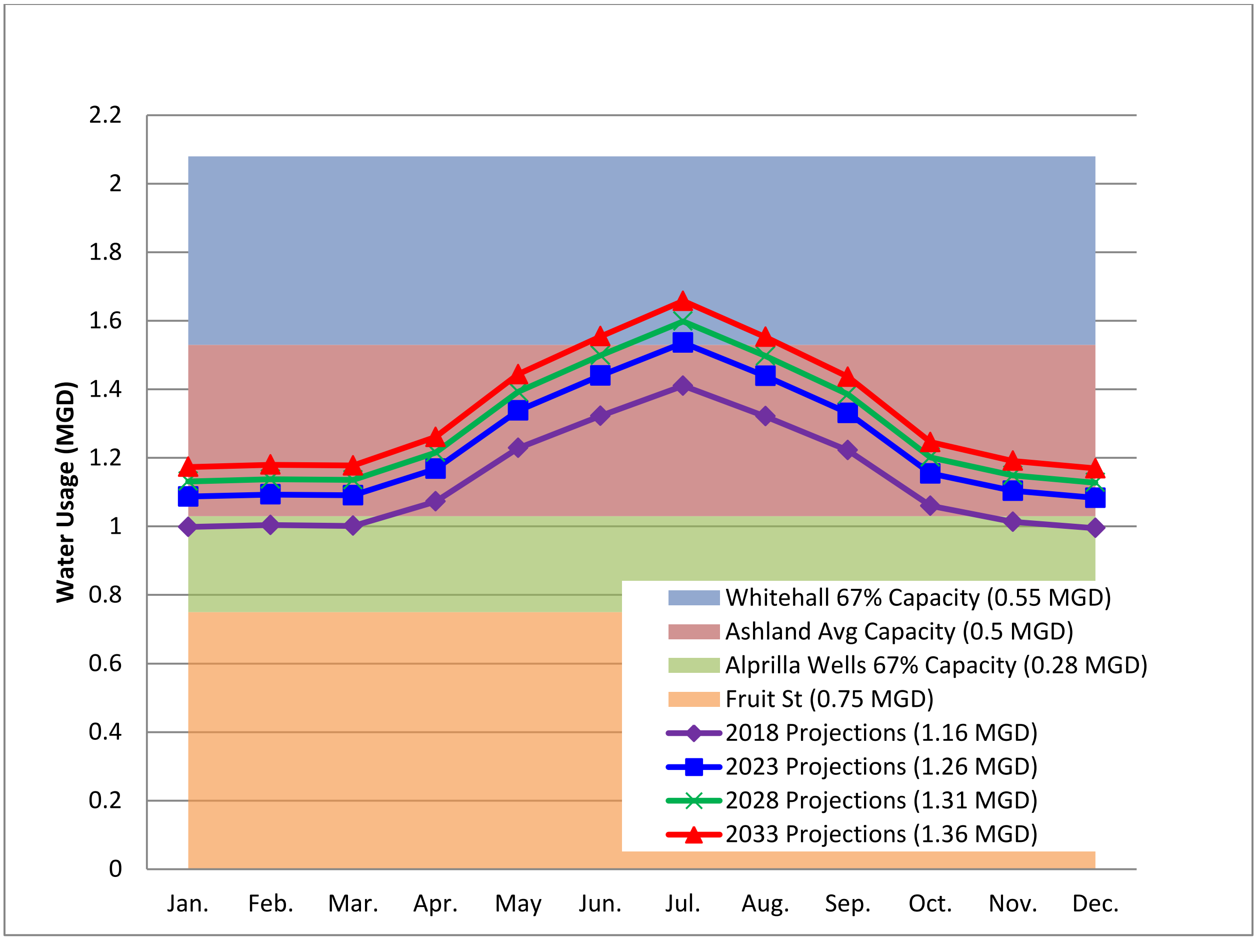
TABLE 8-2
WATER SUPPLY SCENARIOS

Well Source	Typical Supply w/ WTP (MGD)	Typical Supply w/o WTP (MGD)
Whitehall No. 4*	0.24	0
Whitehall No. 5*	0.31	0
Fruit Street**	0.75	0.75
Alprilla*	0.28	0.28
Ashland	0.5	0.5
Available Pumping from Sources	**2.08**	**1.53**
Scenario 1 Projected 2033 Max. Day	2.22	2.22
Scenario 2 Projected 2033 Max. Day	1.88	1.88
Scenario 1 Max. Day Water Supply Shortage (2033)	-0.14	-0.69
Scenario 2 Max. Day Water Supply Shortage (2033)	0.20	-0.35
Scenario 1 Projected 2033 Avg Day	1.36	1.36
Scenario 2 Projected 2033 Avg Day	1.15	1.15
Scenario 1 Avg. Day Water Supply Shortage (2033)	0.72	0.17
Scenario 2 Avg. Day Water Supply Shortage (2033)	0.93	0.38

* Assumes that wells operate 16 hours a day
** Fruit St capacity limited to 0.75 MGD by WMA permit (Well No. 1,2&6)

Figure 8-2 demonstrates the average daily projected water demands for Scenario 1 assumptions and the available typical pumping capacity from Town sources. The figure demonstrates that the Town will be able to satisfy 2033 average projected water demands if a WTP is constructed and the Whitehall well water can be utilized. If a WTP is not constructed, the Town will need to supplement their sources by purchasing water or constructing another source. The volume of water that the Town will need is the area under the projected demand curve and above the supply of the sources. As the Figure shows, as demands continue to increase, the volume under the curve continues to get larger and the Town will need to increase the number of months that they purchase water.

FIGURE 8-2
AVERAGE DAILY WATER DEMAND PROJECTIONS (SCENARIO 1) W/ AVAILABLE TYPICAL TOWN WATER SUPPLY



The MWRA Entrance Fee will be based on the annual volume of water that the Town anticipates needing. The MWRA Entrance Fee can be made in blocks over the 20-year planning period as Hopkinton needs the water. The Town would not need to purchase the rights to all the 2033 water supply shortage immediately, however, if we compare life-cycle costs of alternatives over a 20-year period, all the cost will be realized over the planning period.

Table 8-2 demonstrates that under Scenario 1 water demand projections, if a WTP is constructed, Hopkinton will need to supplement their well sources in the summer with approximately 0.14 MGD (100 gpm) of water for the summer peak days in the year 2033. If we assume that Hopkinton will use this connection for a maximum period of 30 days, this totals a very small volume of water (4.2 MGD) that the Town needs to purchase the rights to, with the purchase not being required until 2028. If the Town does not construct a WTP, they will need to purchase approximately 0.69 MGD (480 gpm) of water during the summer peak days. If we assume that this will be required for 90 days over the summer, the total volume of water required to be purchased from the MWRA would be 62 MGY. The Entrance Fee for this volume of water will be based on purchasing 0.17 MGD (62MGY/365 days). We have assumed a linear projection of water use for the purposes of estimating the cost to purchase water.

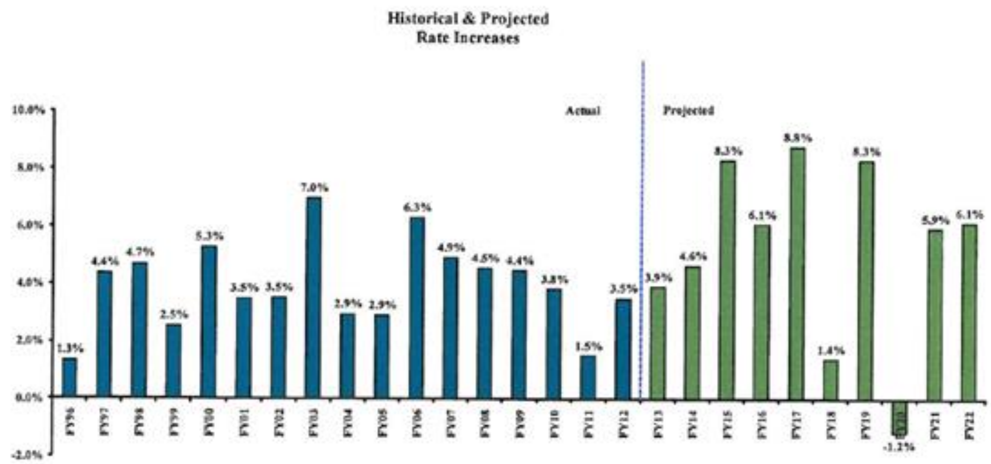
8.5 Costs of an MWRA Connection

All costs associated with an MWRA connection, including meters, piping, potential improvements to the Southborough, Ashland or MWRA systems would be paid by Hopkinton. The MWRA Entrance Fee is the up-front cost associated with the rights to purchase the volume of water needed and is currently $5.3 million / MGD. The volume of water Hopkinton would need is based on the annual amount of water the Town plans to purchase. If the Town constructs a water treatment plant and is able to utilize all their supplies, they would only need the MWRA water in the summertime during high demand and/or drought periods and only after the year 2020 (even further out depending on the demand projection scenario used) which significantly reduces the Entrance Fee as well as the costs to purchase water.

In addition to the Entrance Fee, and the capital cost to construct facilities and improvements in other systems, the Town will pay more for water purchased from the MWRA. The current unit price of purchasing wholesale water from the MWRA is $3,000 per MG of water purchased ($2.24/HCF). We have assumed that Hopkinton would pay a 50% markup on MWRA water wheeled through Southborough and Ashland and that MWRA water rates would continue to increase 5% per year. MWRA water rate increases have been significantly higher than 5% historically, in fact, double digit increases have been observed with some frequency (see chart below showing historical and projected MWRA rate increases). The MWRA capital debt commitment is significant and is blamed for much of the high water rate increases. Projections of MWRA rates show that the water rate increases will begin to level off in 7 to 10 years providing that the MWRA does not continue accruing new debt.

FIGURE 8-3

HISTORICAL & PROJECTED RATE INCREASES



We have outlined the capital costs associated with a connection to the MWRA through Ashland or directly with Southborough. These costs do not include Entrance Fees or the cost to purchase water, as the Entrance Fee is based on how much water the Town needs to purchase and is dependent on the various demand scenarios and whether the Town builds a WTP. The hydraulic gradeline of the Southborough system is less than Hopkinton's system so a pump station would need to be constructed to serve water to Hopkinton from Southborough. A water main (~15,000 ft) would be constructed in Route 85 that would connect the Hopkinton and Southborough systems. Based on recent discussions with the Ashland DPW, they are currently budgeting $1.75 million in improvements to their own and Southborough's water system to construct an interconnection. We have assumed that the Town of Hopkinton would participate in these improvements. We have estimated the capital costs to connect to the MWRA in Table 8-3.

TABLE 8-3
MWRA CONNECTION CAPITAL COSTS SOUTHBOROUGH VS ASHLAND

Description	Southborough Connection 2013 Estimated Cost	Ashland Connection 2013 Estimated Cost
Meter Installation & Pump Station	$1,000,000	$500,000*
Engineering & Permitting	$500,000	$250,000*
Pipeline Improvements	$4,000,000	$900,000*
Improvements in Southborough System	$1,500,000	$500,000
Subtotal	**$7,000,000**	**$2,150,000**
20% Contingency	$1,400,000	$430,000
TOTAL	**$8,400,000**	**$2,580,000**

* Assumes that Hopkinton will share costs with Ashland

The cost of an interconnection with the MWRA through Ashland has the potential to be significantly less than directly connecting with Southborough. Table 8-3 demonstrates that if Hopkinton cannot wheel MWRA water through Ashland and they need to construct a dedicated pipeline, that it will rival the cost of a WTP for Well Nos. 4 and 5 and will make the MWRA alternative less desirable than constructing a WTP.

The Town of Hopkinton currently has an interconnection with Ashland designed for 1.0 MGD. If the Town of Hopkinton does not construct a WTP at the Whitehall wells, they will need more than 1.0 MGD from the MWRA through the Ashland interconnection to supply future (2033) summer demands. We have estimated that Hopkinton would need to spend approximately $2,000,000 on infrastructure improvements to increase the size of the interconnection with Ashland if the Whitehall WTP is not constructed. The existing infrastructure is not capable of taking water from the Ashland system and pumping it to Hopkinton as the pumping equipment is located in the WTP clearwell. Therefore, Hopkinton will need to get Ashland to agree to send more water from their WTP and supplement their own water supply with MWRA water. This may have financial impacts on Ashland rate payers as the cost to purchase MWRA water may be greater than the cost to treat their own water. Hopkinton may need to renegotiate a water purchase price with Ashland to make this equitable for Ashland customers. We recommend that the Town of Hopkinton begin working with Ashland immediately to try and participate in the costs associated with the interconnection with Southborough and negotiate an agreement for water supply purchase.

In order to estimate the full costs of connecting to the MWRA, so that it can be compared with constructing a WTP for Wells No 4 and 5, we need to include the Entrance Fee as well as the cost to purchase water as these are typically the highest costs associated with the MWRA. As these costs are associated with the volume of water the Town needs to purchase, they are different depending on whether the Town constructs a WTP to treat water from the Whitehall wells. For the purposes of comparing these alternatives, we have utilized the more conservative (Scenario 1) water demand projections based on 65 gpcd and we have assumed that the Town would construct a connection with the MWRA through Ashland.

A simple approach to determining whether it is more cost effective for the Town to construct a WTP for the Whitehall wells or purchase water from the MWRA is to calculate the costs associated with the volume or safe yield of the Whitehall wells. The safe yield of the Whitehall wells is 0.83 MGD (0.36 and 0.47 MGD). The MWRA Entrance Fee alone for the rights to purchase 0.83 MGD is $4.40 million. The actual (present worth) cost to purchase 0.83 MGD over a 20-year period when wheeled through Southborough, assuming 50% markup on the MWRA wholesale rate and 5% annual inflation is over $35 million. Table 8-4 outlines the 20-year life-cycle costs of purchasing 0.83 MGD of water from the MWRA.

TABLE 8-4
20-YEAR LIFE CYCLE COST TO PURCHASE 0.83 MGD FROM MWRA

	MWRA Costs
Capital Costs*	$2,580,000
Additional Costs in Hopkinton & Ashland systems	$2,000,000**
Entrance Fee	$4,400,000
Cost to purchase water (present worth)	$35,600,000
TOTAL 20-year costs	**$44,280,000**

*Assumes connection through Ashland
** Assumes significant additional costs would be required to make this a permanent MWRA connection, as Ashland is only going after an Emergency Connection with the MWRA

Table 8-4 demonstrates that it would be more cost effective to construct a WTP for the Whitehall Wells No. 4 and No. 5 if the Town needed all the water from these wells over the entire year. However, the Town can handle the majority of the demands in the system without the Whitehall wells except in the summertime. If the Town constructs a WTP they will need to construct it for all the capacity in Wells No. 4 and No. 5, where if the Town connects to the MWRA, they may only need to purchase the water they need and the amount of water they need will increase over the 20-year planning period.

If the Town constructs a WTP for the Whitehall wells and is able to utilize the water from these sources, the Town may not need to purchase any water from the MWRA and if they do, it would be fairly minor and will not be for another 10 to 15 years. Table 8-2 demonstrates that if a WTP is constructed, the Town will have a water supply deficit only in the summer maximum day demands of 0.14 MGD in the year 2033. If we assume that the Town will need 0.14 MGD for 30 days, the Town would need to purchase the rights to 4.2 MG per year. Based on the MWRA entrance fee of $5.3 million per million gallons per day, the entrance fee to purchase 4.2 MGY would be less than $100,000. The Town would be responsible for paying the cost of all capital improvements needed to connect and wheel water. The purchase cost for this water is insignificant.

If the Town does not construct a WTP for the Whitehall wells, the volume of water that the Town needs to purchase over a year is significantly higher. Based on Figure 8-2 we estimate that the Town will need to purchase 62 MGY or 0.17 MGD in 2033. The MWRA entrance fee for this volume of water is approximately $1 million. The cost to purchase this water from the MWRA is estimated in Table 8-5 which summarizes the 20-year life-cycle costs associated with purchasing water from the MWRA if the Town does not build a WTP for the Whitehall wells.

TABLE 8-5
20-YEAR LIFE CYCLE MWRA COSTS

	MWRA Cost w/ Whitehall WTP	MWRA Costs w/o Whitehall WTP
Capital Costs*	$2,580,000**	$2,580,000**
Additional Costs in Hopkinton & Ashland systems	0	$2,000,000***
Entrance Fee	$100,000	$1,000,000
Cost to purchase water (present worth)	$365,000	$6,200,000
Construct blending facility at Fruit Street	N/A	$1,600,000
TOTAL 20-year costs	**$3,045,000**	**$13,380,000**

* Assumes connection through Ashland
**Cost carried from Table 8-3
*** Assumes significant additional costs would be required to make this a permanent MWRA connection, as Ashland is only going after an Emergency Connection with the MWRA

The largest portion of the costs in Table 8-5 are the costs to purchase water. *It is important for the Town to remember when comparing the costs of an MWRA connection with constructing a WTP, that history shows that the MWRA costs to purchase water will continue to increase. In all reality, if we looked out longer than 20-years for the life-cycle costs, the cost to build a WTP would likely be significantly less than the cost to connect to the MWRA primarily due to the costs to purchase water.*

There are some very significant unknowns if the Town of Hopkinton wants to utilize the MWRA through Ashland in lieu of constructing a WTP. The Town of Ashland is only pursuing an emergency connection with the MWRA which is significantly different than a permanent connection. The wheeling of water through two different Towns also poses some unique challenges to Hopkinton both politically and financially. There are also some unique political and financial challenges associated with wheeling water through Ashland. The easiest way to do this, would be for Ashland to send Hopkinton more water through their combined WTP which would cause Ashland to purchase MWRA water.

If Hopkinton does not construct a WTP to treat water from the Whitehall wells on the Fruit Street site, a combined blending treatment facility to help blend the water from Well No. 2 will be required due to the anticipated changes in manganese regulations. We have evaluated and compared the financial impacts of the water supply alternatives for the Town in Chapter 9.

8.6 MWRA Water Quality

The MWRA utilizes chloramines as a secondary disinfectant to maintain a residual in the water system. As chloramines are not as strong as chlorine, they do not react with organics as quickly to form disinfection-by-products, and therefore they last longer to maintain a residual in the far reaches of the distribution system. Based on the available literature, water systems that utilize both free chlorine (Hopkinton) and chloramine disinfection may experience water quality degradation in the distribution system. It is important to note that although the literature indicates that this can be a problem, there are many communities that purchase supply water from the MWRA (who utilize free chlorine) without observing any issues.

The easiest way for the Town to get MWRA water through Ashland may result in never actually seeing MWRA water in Hopkinton. Ideally, Ashland would increase the volume of water they send to Hopkinton through the WTP, and supplement their sources with purchased MWRA water. If Hopkinton is able to work this out, chloraminated MWRA water would rarely mix with the Town's water.

8.7 Historical Ground water Exploration

A Water Alternatives Study was conducted in 1996 to identify the best potential sites for future gravel packed wells in Hopkinton. The evaluation reviewed information associated with the more than 100 test borings and wells drilled throughout the Town of Hopkinton since 1970 looking for potential water sources. Ultimately 24 different sites were identified for a detailed review. Most of the 24 sites were quickly eliminated based on low ground water transmissivity and the lack of availability of a 400-foot protective radius. Other characteristics, such as the close proximity of bedrock to the ground surface, poor site access, and water quality hazards, were also considered. The 1996 evaluation concluded that there were four potential sites to investigate further to determine the validity of each site as a future water source for the Town: WH-2, WH-3, WH-4, and the site of the former Pyne & New England Gravel Pits. Since the evaluation, the Town has constructed new wells at two of those four potential sites (Figure 8-4).

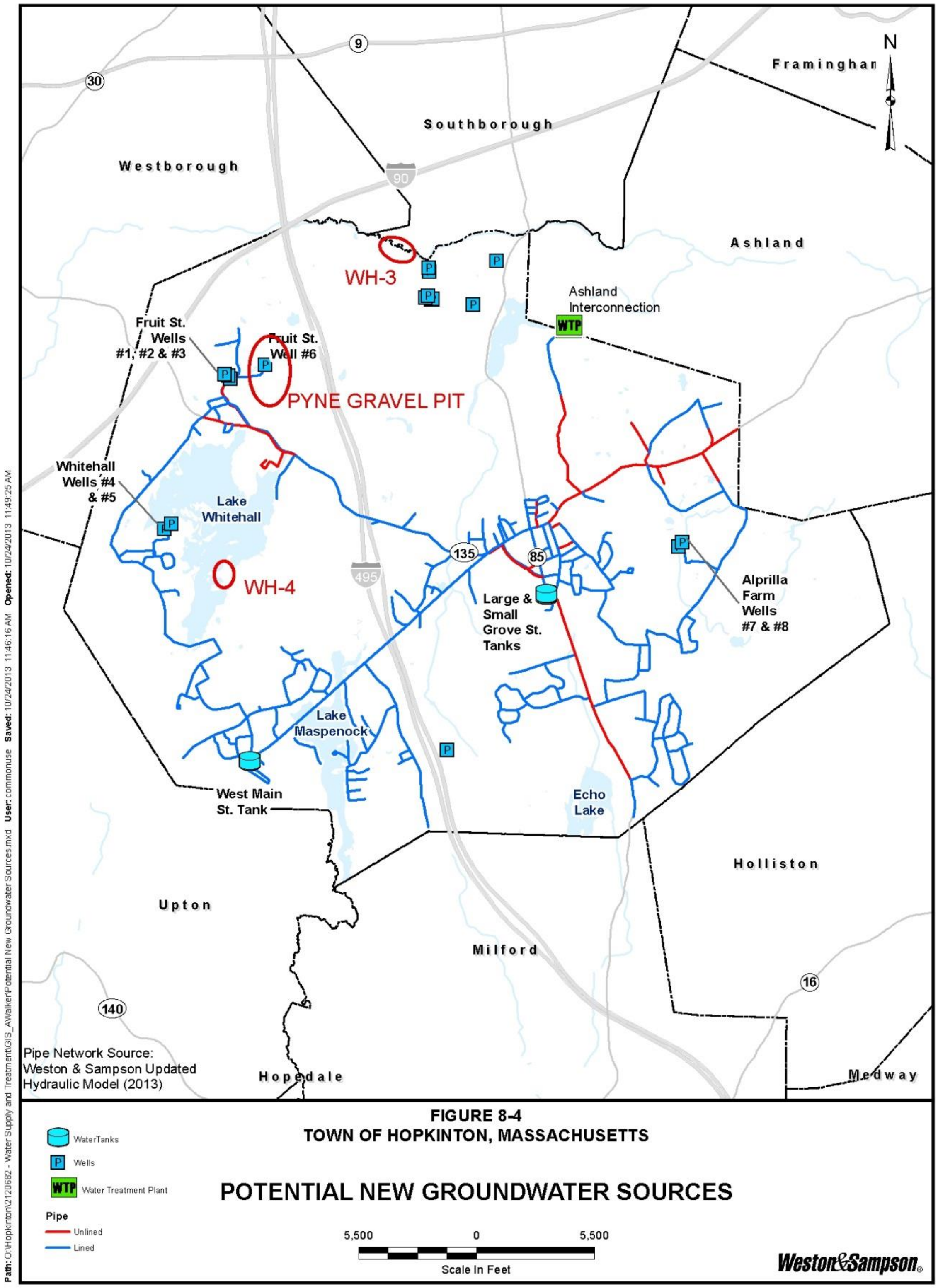


FIGURE 8-4
TOWN OF HOPKINTON, MASSACHUSETTS

POTENTIAL NEW GROUNDWATER SOURCES

The Pyne Gravel Pit site, located on the east side of Fruit Street, is the location of Fruit Street Well No. 6. The 1996 study actually identified three separate sites within the former Pyne Gravel Pit area that appeared to have modest potential: H-7, H-2, and an existing “Gravel Wash Well.” Well No. 6, was constructed in 2009 on the H-2 site, with a safe yield of 0.72 million gallons per day (MGD), approximately 500 gallons per minute (gpm). It is possible that additional withdrawals could be made from the wellfield with the construction of an additional well, perhaps in the vicinity of H-7 or the Gravel Wash Well. Prior to the construction of any new wells or withdrawals from the wellfield, the safe yield of the Fruit Street aquifer should be determined.

Site WH-2 is the Alprilla Farm site where Well No. 7 and Well No. 8 were constructed in 2012. Well No. 7 was constructed with a safe yield of 0.14 MGD, approximately 100 gpm. Well No. 8 was constructed with a safe yield of 0.28 MGD, approximately 200 gpm. The total combined safe yield from these two well supplies is 0.42 MGD, approximately 300 gpm.

Site WH-3 has not been utilized as a new well site. The site is located in the northern part of Hopkinton near the Sudbury River on land owned by the Southborough Rod & Gun Club. The site is located in an area identified by the United States Geological Survey (USGS) as having low transmissivity. However, according to the 1996 study, subsurface investigations were not conducted to identify the actual conditions in the area. While the Water Alternatives Study acknowledged that the aquifer in that area is relatively small, the report did note that there is a potential for induced infiltration from the Sudbury River. It is possible that if a moderate thickness of sand and gravel were identified at this site such induced infiltration may provide water supply potential. However, at this time, the full potential of the site remains unclear. A preliminary subsurface investigation, including the rate testing of up to 8 test wells and a pumping test of the most favorable, must be conducted at the site before its suitability can be accurately assessed. While it is possible that the site may be capable of producing a useful withdrawal rate, there are several technical factors to consider, such as the distance to the existing distribution system and the testing and permitting requirements associated with sites associated with induced infiltration. In addition, as noted in the Water Alternatives Study, the site may be crossed by an MWRA easement, adding another level of complexity to the construction of a new water supply source at WH-3.

Also, the WH-3 site is located within sub-basin 12028, identified by the Sustainable Water Management Initiative (SWMI) as a Category 4 sub-basin under the Ground water Withdrawal Category. Ranked from 1 to 5, a Category 4 rating indicates that ground water withdrawals are already impacting August streamflow in the sub-basin by 25-55%. Permitting increased or additional ground water withdrawals above the MWA baseline for this sub-basin will likely face Tier 2 MWA/SWMI permitting requirements, including the mitigation of any impacts associated with the increased withdrawal. It is estimated that an additional withdrawal of 400 gpm may cause the sub-basin to be upgraded to a Category 5 sub-basin, triggering the Tier 3 MWA/SWMI permitting and mitigation requirements.

Site WH-4 has also not been utilized as a new well site. According to the 2003 Water Master Plan, WH-4 is located on the eastern shore of the Whitehall Reservoir within the Whitehall State Park. Due to its location on state forest land, construction of a new well in this area would require dealing with several political obstacles. The geological conditions in the area appear favorable for a well with induced infiltration from the Whitehall Reservoir. A new well in this area may be similar to the existing Whitehall Well No. 4 and Well No 5. Based on the poor water quality observed at Well No. 4 and Well No. 5, a new well similar to the existing Whitehall Wells may not be a sustainable solution unless the new well could be treated along with Wells No. 4 and No. 5. In addition, at this time, the full hydrogeological potential of the site remains unclear. A preliminary subsurface investigation, including the rate testing of up to 8 test wells and a pumping test of the most favorable, must be conducted at the site before its suitability can be accurately assessed.

Also, the WH-4 site is located within sub-basin 12025, identified by the Sustainable Water Management Initiative as a Category 5 sub-basin under the Ground water Withdrawal Category. Ranked from 1 to 5, a Category 5 rating indicates that ground water withdrawals are already impacting August streamflow in the sub-basin by more than 55%. Permitting increased or additional ground water withdrawals above the MWA baseline for this sub-basin will likely face MWA/SWMI Tier 2 permitting requirements, including the mitigation of any impacts associated with the increased withdrawal.

8.8 Future Gravel Pack Wells Summary

In conclusion, the 1996 Water Alternatives Study identified four potential sites for additional ground water withdrawals: WH-2, WH-3, WH-4, and the site of the former Pyne & New England Gravel Pits. A review of the most recent hydrogeologic data concurs with the conclusions of the Water Alternatives Study. Since that 1996 evaluation, Well No. 6 was constructed in 2009 at the Pyne Gravel Pit site and Wells No. 7 and No. 8 were constructed in 2012 at the Alprilla Farms site, WH-2. The remaining potentially suitable locations for future gravel pack wells include additional sites within the Pyne Gravel Pit, WH-3, and WH-4. Little is known about the subsurface conditions at WH-3 or WH-4; preliminary sub-surface investigations on the order of $35,000 each would be required at each site to determine its potential suitability. In contrast, the Pyne Gravel Pit area has been studied extensively and presents greater potential at this time. However, the Fruit Street aquifer beneath the gravel pit is already tapped by four of Hopkinton's existing wells, Well Nos. 1, 2, 3, and 6. While the safe yield of each individual well has been evaluated, the safe yield of the aquifer as a whole has not yet been determined. If the Town wishes to pursue additional withdrawals from existing wells or construct additional wells within the Fruit Street aquifer, such a study should be conducted. It is likely that the numerical ground water model developed in support of the permitting of Well No. 6 could be adapted to that end, but determination of the safe aquifer yield would also involve a pumping test of the wellfield and monitoring of streams, wetlands, and other nearby surficial waterbodies to identify potential environmental impacts. Such a study would cost on the order of $100,000-150,000. If the Town wishes to pursue additional ground water withdrawals, it is recommended that the Town focus its attention on the Fruit Street aquifer. Past water alternative studies confirm that the aquifer represents the most reliable, most understood source of ground water within the Town's boundaries. It is likely that the aquifer can support additional withdrawals above the current WMA permitted limit of 0.75 MGD, although the safe yield of the aquifer should be studied before withdrawals are increased. The Town may also want to pursue preliminary subsurface investigations of the WH-3 and WH-4 sites to better gauge their future potential. However, both sites face several permitting and technical hurdles that would be avoided by focusing on the Fruit Street aquifer.

8.9 Historical Surface Water Exploration

There are four (4) major surface water bodies within the Town of Hopkinton that could be considered for use as surface water supply to supplement the Town's water supply:

- Whitehall Reservoir
- Hopkinton Reservoir
- Lake Maspenock (North Pond)
- Echo Lake

8.9.1 Whitehall Reservoir

The Whitehall Reservoir is operated by the Department of Conservation and Recreation (DCR). The reservoir was once used as a drinking water supply for the City of Boston. After the Quabbin and Wachusett reservoirs were constructed, the Whitehall Reservoir use as a drinking water supply was discontinued due to deteriorating water quality. The reservoir was transferred to Department of Environmental Management (DEM) in 1947. The reservoir is currently used for recreational purposes including, but not limited to, boating, canoeing, and fishing. There is a history of Massachusetts' agencies not giving permission to local municipalities to utilize these types of surface water bodies for local drinking water supplies. Furthermore, construction and operation of a surface water treatment plant could approach twice the cost of constructing and operating a ground water treatment plant. Surface water treatment plants are more heavily regulated than ground water plants and the operator license requirements are significantly higher. It is not possible that the cost of constructing a surface water supply on the Whitehall Reservoir or any other surface water body for that matter, would be more cost effective and feasible than constructing a ground water source for the Whitehall wells.

8.9.2 Hopkinton Reservoir

The Hopkinton Reservoir is owned and operated by the Department of Conservation and Recreation (DCR). The reservoir is used for recreational purposes including, but not limited to, boating, canoeing, and fishing. The Hopkinton Reservoir also has an accessible beach with a designated swimming area and life guards for public safety. The Town of Ashland wells influence the water level in the Hopkinton Reservoir significantly. Their water management act permit has pumping restrictions on the Ashland wells based on water levels in the Hopkinton Reservoir. The Hopkinton Reservoir is not a viable surface water source for water supply to Hopkinton nor would it be cost effective compared to constructing a ground water plant.

8.9.3 Lake Maspenock (North Pond)

North Pond also known as Lake Maspenock, is a raised Great Pond with a surface area that occupies approximately 234 acres. The Lake Maspenock Lake Preservation Association maintains a website, www.lmpa.org regarding this surface water body which has a very informative history of the lake. The major use of the lake is for water-based recreation which is generally in conflict with utilizing this lake as a drinking water reservoir.

8.9.4 Echo Lake

Echo Lake is the headwaters of the Charles River. Hopkinton is the highest point in the region and is at the headwaters of three watersheds: the Charles, the Blackstone, and the "SuAsCo" (Sudbury, Assabet, and Concord) tributary of the Merrimack River. The Town of Milford utilized water from Echo Lake and the Charles River through their newly renovated Dilla Street WTP. The Milford Water Company is currently permitted to withdraw 1.57 mgd from this source. It is not likely that the Town of Hopkinton would be successful in permitting a surface water source in Echo Lake as any withdrawals would impact the Milford Water Company's withdrawals and would not likely be a source that could be permitted.

9.0 WATER SUPPLY SUMMARY OF ALTERNATIVES

9.1 General

We have outlined many alternatives that the Town of Hopkinton can undertake to solve their water supply shortages such as; water conservation and reducing the UAW, develop potential new sources, treat existing sources, or connection to the MWRA. It is a matter of which alternatives are the least costly and most effective or more likely which combination of alternatives yields the lowest cost and most effective approach. We have attempted to compare the alternatives outlined in previous Chapters in this section of the report and provide a streamlined course of action for the Town of Hopkinton to follow to solve their current and future water supply shortages.

9.2 Water Management Act (WMA) Permitting

The Town's WMA permit was revised in 2012 to increase the allowed withdrawals to 1.21 MGD. The primary reason for this increase was because of the Legacy Farms development and the installation of Alprilla Wells No. 7 and No. 8 and an onsite wastewater treatment plant that recharges back into the basin. The 1.21 MGD was provided by the DCR as an interim permit which was based on the amount of water that Hopkinton is currently utilizing plus the amount needed for Legacy Farms.

The WMA permit limits the withdrawals of the Fruit Street wells to 0.75 MGD. The wells that comprise the Fruit Street aquifer (No. 1, No. 2, & No. 6) have a combined safe yield of 1.35 MGD (Well No. 3 is in emergency status). Well No. 2 has high manganese and without improvements to this wellfield to incorporate blending or filtration of Well No. 2, the Town will be limited in how much they can pump Well No. 2 in the future as manganese in raw water becomes further regulated. Treatment of Well No. 2 does not get the Town more water to meet their maximum day demands, unless the Fruit Street wellfield pumping limit is increased within their WMA permit. We suggest that the Town work with DEP through the future SWMI process to update their WMA permit to increase the amount allowed to be pumped out of the Fruit Street basin using the recharge on the Fruit Street site from the wastewater treatment plant. Although the wastewater treatment plant is permitted for 350,000 gpd, it is only discharging 110,000 gpd currently. This increase in allowable withdrawal may help the Town to supply water through the

2014 summer. Even if the regulatory restrictions on pumping were lifted at the Fruit Street well site, the Town is limited in how much water they can physically get out of the ground due to reduced water table and well drawdown cones of influence. It is possible that the Town will need to perform additional ground water modeling to further understand the ground water balance issues and return flows in the Fruit Street aquifer and how the well hydraulics will be affected. The additional volume that the Town can pull from the Fruit Street wells above 0.75 MGD may be limited during the summer dry periods due to interferences from other wells. We estimate the cost to perform this modeling to be approximately $30,000.

9.3 Develop New Sources in Hopkinton

The sources outlined in Chapter 8 that have not been developed for water sources are WH-2 (on the Fruit Street property), WH-3 and WH-4. There is very little information known about these potential well sites. The Town will need to spend money to determine if these are viable for future well sites. WH-3 and WH-4 are unlikely to have water quality that does not require treatment, so the likelihood of these being better supplies than what the Town already has is not good. Developing a ground water source from either of these sites, does not seem like a viable water supply option for Hopkinton at this time, given their other water supply alternatives.

9.4 Fruit Street Wellfield Improvements

The high manganese in Well No.2 raw water can be blended with water from Wells No. 1 and No. 6 to lower the manganese below the future regulatory limit of 0.3 ppm as discussed in Chapter 7. In order for this to be done, the water from Wells No. 1 and No. 2 would need to be piped across Fruit Street to a central location (either at Well No. 6 building or a new WTP located at Fruit Street). This improvement would allow the Town to pump Well No.2 more freely without as significant an impact to the water quality in the distribution system. If a water treatment plant is constructed on the Fruit Street property to treat Wells No. 4 and No. 5, the water from Well No.2 can be treated for manganese removal in that facility.

9.4.1 Construct Treatment Plant for Whitehall

The Whitehall (No. 4 and No. 5) well water is only utilized during extreme emergencies due to the high iron concentrations. If the Town constructed a water treatment plant for this water, they would be able to utilize the water and will potentially not have water supply shortages over the 20-year planning period depending on how much Hopkinton grows (and the role water conservation plays). Scenario 1 demand projections paint the worst case scenario which is that the Town would only need to supplement their maximum day demands with 100 gpm of water in 2033. If a water treatment plant is constructed for the Whitehall wells, we suggest that the Town utilize the Fruit Street site. The Whitehall site will be difficult to construct a WTP due to land ownership issues. Constructing a WTP at Fruit Street would also allow the Town to treat Well No. 2 water through the WTP and provide one central location for a blending/treatment facility for the water in the Fruit Street aquifer and would allow the Town to pump the Whitehall wells with more flexibility as they would not need to shut the plant down to rest the wells. Lastly, discharges of residuals in the Fruit Street aquifer could be utilized through SWMI for future WMA permit increases. The 20-year life-cycle cost of a new biological filtration treatment facility located on the Fruit Street property, including raw water piping, as discussed thoroughly in Chapter 7 is estimated to be $19.1 million.

If the Town chooses to not build a WTP to treat Whitehall Wells No. 4 and No. 5, they will still need to construct a blending water treatment plant on the Fruit Street site to deal with the water quality from Well No.2. We have estimated the cost to construct a new facility on the west side of Fruit Street to be $1.6 million. It is possible that the existing Well #6 building could be utilized as the blending facility and that Well No. 1 and No. 2 water could be piped to that building which would result in significant cost savings over constructing a new building. We recommend that the Town perform a preliminary evaluation to determine if this is viable and evaluate cost savings.

9.4.2 Purchase MWRA Water

The Town has an opportunity to work with Ashland to share in the costs of an MWRA connection. This connection could be utilized by Hopkinton to supplement water supply in the summer when their water supplies are diminished due to low water tables. If the Town could work out the details of this connection with Ashland and the costs, this has the potential to be a relatively inexpensive way for Hopkinton to meet summer peak demands for the next five to ten years (or maybe longer). However, as the base water demands in Hopkinton continue to increase and MWRA rates increase, purchasing water from the MWRA (and wheeling fees associated with it) may become costly. This report compared the 20-year life-cycle costs of each alternative and the two water supply scenarios are fairly close. *If the study projected out past 20-years, the Fruit Street WTP would be paid for and the Town's water rate would likely be significantly lower than if the Town depends on the MWRA for water supply instead of building a WTP.*

The present day cost to purchase the same volume of water that the Whitehall Wells can provide (0.83 MGD) through the MWRA (does not include wheeling fees) is over $44 million, demonstrating that it is typically less expensive to treat your own water than to purchase from the MWRA. Hopkinton however, is in a situation where it doesn't currently need all 0.83 MGD every day of the year. The Town could potentially peak off the MWRA system and only utilize and pay for what it needs above what the Town supplies can deliver. Supply of MWRA water through Ashland has potential to be relatively inexpensive because the infrastructure is already in place and provides significant freedom from the WMA permit for the Ashland plant. If the Town does not build a WTP to treat the Whitehall wells, they will need to begin purchasing water from the MWRA almost immediately. This may be possible if Hopkinton partners with Ashland and works on a deal where Ashland sends more of their WTP water to Hopkinton and they buy from the MWRA (Hopkinton may need to discuss rate relief with Ashland for this scenario). We have estimated the 20-year life-cycle costs for this scenario to be approximately $13.3 million. Table 9-1 demonstrates the comparison of the 20-year life-cycle costs for the WTP versus the MWRA connection. As the majority of the MWRA life-cycle costs are in the cost to purchase water, this could result in a relatively low capital cost source for Hopkinton with significant permit fee operating flexibility, that can be utilized for meeting their peak demands.

TABLE 9-1

20-YEAR LIFE-CYCLE COSTS MWRA VS. WTP

Description	Costs to Construct WTP	Cost to Connect to MWRA
WTP Capital[1]	$13,020,000[1]	
Water Costs, Financing, Addtl. Operating Costs	$5,000,000[2]	$6,200,000[3]
Cost to Construct Interconnections w/ Ashland		$2,580,000[4]
Improvements to Hopkinton's system		$2,000,000[5]
MWRA Entrance Fee		$1,000,000[6]
Construct Fruit St Blending Facility		$1,600,000
TOTAL	$18,020,000	$13,380,000

1. Assumes a biological filtration suitable for well water Table 7-2
2. Assumes borrowing of $13 million, 2% interest for 20 years, additional operation expense $130,000 annually, 2% inflation
3. Does not include any debt financing costs
4. Table 8-3
5. May not be realized in the first 10 years of the program
6. May not need to be paid all up front

We recommend that Hopkinton continue discussions with Ashland to determine if these assumptions are viable and if a negotiated volume and rate can be determined. If Hopkinton cannot reach an agreement with Ashland for long-term water supply in excess of their 1.0 MGD agreement utilizing a combination of MWRA and the WTP, the additional cost of connecting to Southborough will change the recommendation away from the MWRA and back to an onsite WTP.

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10.0 CAPITAL IMPROVEMENT PROGRAM

10.1 General

In this Section, we have attempted to prioritize the recommendations made in previous chapters. The improvements to the water system are significant. As discussed previously, Hopkinton's largest concern facing them in the 20-year planning period is implementation of a new source to serve future water demands.

10.2 Estimated Construction Costs

The estimated costs specified for each group of improvements include construction costs, engineering costs, and contingencies. The estimated costs were developed in part by using recent construction costs for Towns with similar development and geographic location to Hopkinton. These costs were updated to an Engineering News Record (ENR) 20 City index for July 2013 of 9551. Other sources include the Means "Building Construction Cost Data" and manufacturers' quotations. It should be noted that the estimated costs listed in this report reflect 2013 construction costs and engineering costs only, and no steps were taken to inflate the costs to reflect future construction costs. We recommend an average annual construction cost increase of 3 to 5% for budgeting purposes. The per foot construction costs used in this section are shown in Table 9-1.

TABLE 10-1
PIPELINE UNIT PRICE COSTS

Recommendation	Cost (per foot)
Replace with 8-inch Ductile Iron Water Main	$165
Replace with 12-inch Ductile Iron Water Main	$165

The pipeline costs used in this study are for year 2013 construction and engineering costs and include the following:

- Design, construction administration, and resident oversight engineering
- Hydrant spacing at 500 feet and gate valve spacing at 1,000 foot maximum
- 2-inches of temporary trench binder course pavement

- Trench-width top-course pavement 3½-inches thick.
- Traffic control officers

10.3 Phased Capital Improvement Plan

We have attempted to summarize the capital expenses outlined in this Water Supply and Treatment Evaluation in Table 10-2. Many of these costs will be bonded and financed through rates. Based on discussions with Ashland, the MWRA capital costs (i.e. meter vault and pipeline construction, and access fee) may be financed through the MWRA and repaid through the annual MWRA assessment.

TABLE 10-2

CAPITAL IMPROVEMENT PLAN

FY	Project Description	Year 2013 Project Cost
2015	Amend WMA Permit for Fruit St WW flows	$15,000
	Evaluation to identify costs to convert silica at #4 and #5 to KOH	$3,000
	Finance Water Audit	$30,000
2016	Design Fruit St WTP Improvements	$100,000
	Ashland / MWRA interconnection	$2,580,000
	Design 1.0 MG Grove Street Standpipe	$75,000
	Preliminary evaluation of utilizing #6 BLDG for centralized treatment facility	$15,000
	Pilot Well No. 4 and No. 5 Water for biological filtration	$50,000
	Convert silica to KOH at No. 4 and 5	$50,000
	Construct 1.0 MG Grove Street standpipe	$1,275,000
2017	Fruit Street Wellfield improvements	$1,500,000
	MWRA Entrance Fee (first half)	$500,000
	Fruit Street Ground water Modeling to determine pumping dynamics	$30,000
	Update water system GIS	$100,000
2018	Additional pipeline improvements in Hopkinton (1st half)	$1,000,000
	Construct High Service System	$2,495,000
2019	MWRA Entrance Fee (second half)	$500,000
2020 to 2025	Additional pipeline improvements in Hopkinton (2nd half)	$1,000,000
2026 to 2030	**TOTAL**	**$11,318,000**

APPENDIX A

Existing Water Distribution System

APPENDIX B

Water Demand Projections

INTERNAL CENSUS TREND (1st 10 years) & MAPC METRO TREND (2nd 10 years) POPULATION (With 2012 ASR)
SCENARIO 1: 65 GPCD, 15% UAW 2016 FORW.

	Current Population						Base Water Use: Residential				Base Water Use: Non-Residential		Base Water Use: Treatment Loss		Base Water Use: Unaccounted Use	
	(A)	(B)	(C)	(D)	(E)	(F)	(G)	(H)	(I)	(J)	(K)	(L)	(M)	(N)	(O)	(P)
Projection Date	Community	Average Year-Round Town Pop.	Pop. Serv. %	2011 Out-of-Town Pop.	2011 Annualized Add'l Seas. Pop.	Base Service Pop.	Base System ADD	Res. ADD	Res. % of Base ADD	Res.	Non- Res. ADD	Non-Res % of Base ADD	Treatment Plant Processing Loss	% Treatment Plant Processing Loss	Base UAW ADD	UAW % of Base ADD (minus TPL)
							(MGD)	(MGD)	(%)	(gpcd)	(MGD)	(%)	(MGD)	(%)	(MGD)	(%)
Baseline 2012	Hopkinton	15,527	55.51%	0	0	8,619	0.996	0.475	47.71%	55.1	0.300	30.15%	0.000	0.00%	0.221	22.1%
2013	Hopkinton															
2015	Hopkinton															
2018	Hopkinton															
2020	Hopkinton															
2023	Hopkinton															
2025	Hopkinton															
2028	Hopkinton															
2030	Hopkinton															
2033	Hopkinton															
		Town Census	DEP ASR		DEP ASR	DEP ASR							2006 DEP ASR		(O=G-(H+K+M)	DEP ASR

	POPULATION						RESIDENTIAL		NON-RES	UAW		TREATMENT LOSS	
	(Q)	(R)	(S)	(T)	(U)	(V)	(W)	(X)	(Y)	(Z)	(AA)	(AB)	**(AC)**
Projection Date	Future Year Round Pop.	Future % Service Pop.	Future Out-of-Town Pop.	Future Annualized Add'l Seas. Pop. Served	Future Pop. Served	Pop. Change, Past-Future	Future Res. Consumption Rate	Future Res ADD	Future Non-Res. ADD	Future 15% UAW ADD[1]	Future Signif. Change ADD	Future Treatment Plant Processing Loss	**Future Total ADD**
							(gpcd)	(MGD)	(MGD)	(MGD)	(MGD)	(MGD)	**(MGD)**
Baseline 2012									**28.904**				
2013	16,019	57%	0	0	9,152	625	65.0	0.595	0.304	0.225	0.000	0	**1.123**
2015	16,413	59%	0	0	9,626	1,099	65.0	0.626	0.308	0.233	0.000	0	**1.167**
2018	17,004	61%	0	0	10,337	1,810	65.0	0.672	0.314	0.174	0.000	0	**1.160**
2020	17,398	62%	0	0	10,811	2,284	65.0	0.703	0.318	0.180	0.000	0	**1.201**
2023	17,989	64%	0	0	11,522	2,995	65.0	0.749	0.323	0.189	0.000	0	**1.261**
2025	18,122	65%	0	0	11,735	3,209	65.0	0.763	0.327	0.192	0.000	0	**1.282**
2028	18,322	66%	0	0	12,055	3,528	65.0	0.784	0.332	0.197	0.000	0	**1.312**
2030	18,455	66%	0	0	12,268	3,742	65.0	0.797	0.335	0.200	0.000	0	**1.332**
2033	18,599	67%	0	0	12,532	4,006	65.0	0.815	0.341	0.204	0.000	0	**1.360**
	WSE Projection												

1. UAW for 2015 is assumed to be 20%

INTERNAL CENSUS TREND (1st 10 years) & MAPC METRO TREND (2nd 10 years) POPULATION (With 2012 ASR)
SCENARIO 2: 55.1 GPCD, 15% UAW 2016 FORW.

	Current Population						Base Water Use: Residential				Base Water Use: Non-Residential		Base Water Use: Treatment Loss		Base Water Use: Unaccounted Use	
	(A)	(B)	(C)	(D)	(E)	(F)	(G)	(H)	(I)	(J)	(K)	(L)	(M)	(N)	(O)	(P)
Projection Date	Community	Average Year-Round Town Pop.	Pop. Serv. %	2011 Out-of-Town Pop.	2011 Annualized Add'l Seas. Pop.	Base Service Pop.	Base System ADD	Res. ADD	Res. % of Base ADD	Res.	Non- Res. ADD	Non-Res % of Base ADD	Treatment Plant Processing Loss	% Treatment Plant Processing Loss	Base UAW ADD	UAW % of Base ADD (minus TPL)
							(MGD)	(MGD)	(%)	(gpcd)	(MGD)	(%)	(MGD)	(%)	(MGD)	(%)
Baseline 2012	Hopkinton	15,527	55.51%	0	0	8,619	0.996	0.475	47.71%	55.1	0.300	30.15%	0.000	0.00%	0.221	22.1%
2013	Hopkinton															
2015	Hopkinton															
2018	Hopkinton															
2020	Hopkinton															
2023	Hopkinton															
2025	Hopkinton															
2028	Hopkinton															
2030	Hopkinton															
2033	Hopkinton															
		Town Census	DEP ASR		DEP ASR	DEP ASR							2006 DEP ASR		(O=G-(H+K+M)	DEP ASR

	POPULATION						RESIDENTIAL		NON-RES	UAW		TREATMENT LOSS	
	(Q)	(R)	(S)	(T)	(U)	(V)	(W)	(X)	(Y)	(Z)	(AA)	(AB)	**(AC)**
Projection Date	Future Year Round Pop.	Future % Service Pop.	Future Out-of-Town Pop.	Future Annualized Add'l Seas. Pop. Served	Future Pop. Served	Pop. Change, Past-Future	Future Res. Consumption Rate	Future Res ADD	Future Non-Res. ADD	Future 15% UAW ADD[1]	Future Signif. Change ADD	Future Treatment Plant Processing Loss	**Future Total ADD**
							(gpcd)	(MGD)	(MGD)	(MGD)	(MGD)	(MGD)	**(MGD)**
Baseline 2012									**28.904**				
2013	16,019	57%	0	0	9,152	625	51.1	0.468	0.304	0.193	0.000	0	**0.964**
2015	16,413	59%	0	0	9,626	1,099	51.1	0.492	0.308	0.200	0.000	0	**1.000**
2018	17,004	61%	0	0	10,337	1,810	51.1	0.528	0.314	0.149	0.000	0	**0.991**
2020	17,398	62%	0	0	10,811	2,284	51.1	0.552	0.318	0.154	0.000	0	**1.024**
2023	17,989	64%	0	0	11,522	2,995	51.1	0.589	0.323	0.161	0.000	0	**1.073**
2025	18,122	65%	0	0	11,735	3,209	51.1	0.600	0.327	0.163	0.000	0	**1.090**
2028	18,322	66%	0	0	12,055	3,528	51.1	0.616	0.332	0.167	0.000	0	**1.115**
2030	18,455	66%	0	0	12,268	3,742	51.1	0.627	0.335	0.170	0.000	0	**1.132**
2033	18,599	67%	0	0	12,532	4,006	51.1	0.640	0.341	0.173	0.000	0	**1.155**
	WSE Projection												

1. UAW for 2015 is assumed to be 20%