# DISTRIBUTION SYSTEM MANAGEMENT PLAN UPDATE

FOR

THE BOARD OF WATER COMMISSIONERS
WAREHAM FIRE DISTRICT
WAREHAM, MASSACHUSETTS

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A Report Prepared for:

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**DISTRIBUTION SYSTEM MANAGEMENT PLAN UPDATE**
**WAREHAM FIRE DISTRICT**
**WAREHAM, MASSACHUSETTS**

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# 1 INTRODUCTION AND BACKGROUND

## 1.1 GENERAL DISCUSSION

The Wareham Fire District (WFD) retained Kleinfelder to update the relevant distribution system related aspects of the Water Supply and Distribution System Management Plan (Master Plan) that was originally developed in 2007. This report summarizes the work performed as part of the Master Plan update and presents key findings and recommendations.

## 1.2 SCOPE OF SERVICES

As requested by the WFD, the scope of services for the Master Plan update focused on the following major tasks:

<u>Task 1 – Water Demand Analysis</u>

A water demand analysis was performed to estimate current and future water demands within the District's system. Reassessing water demands approximately every ten years is appropriate to verify prior demand projections and to serve as a basis for assessing overall system infrastructure requirements, including water supply source development, water treatment capacity, pumping and transmission capacity, and water storage requirements, both under current conditions and future conditions.

The water demand analysis conducted for this report consisted of collecting recent water production and consumption data for the District to identify the existing average day and maximum day demands. Population projection data and information relative to future growth and development within the District was also obtained to estimate average day and maximum day demands in future years up to the year 2040.

This water demand analysis task was intended to serve as an update to Sections 3 and 4 of the 2007 Master Plan.

Task 2 - Distribution Storage Evaluation

In addition to the elevated composite Glen Charlie Tank that was constructed less than ten years ago, the District has two welded steel water storage tanks, the West Wareham Tank and the Bourne Hill Tank, that are scheduled for rehabilitation. Rehabilitation of the steel storage tanks, which typically includes sand-blasting to remove the old coating system, spot repairs, and repainting, represents a significant capital expenditure that the District is required to repeat approximately every 15 years. Before committing to a repeating cycle of major rehabilitation costs to maintain the aging tanks, the District elected to evaluate overall system storage infrastructure as part of this Master Plan update.

The aim of the distribution storage evaluation was three-fold: 1) determine if existing distribution system storage is adequate for current and future conditions, or whether providing additional storage capacity is necessary; 2) compare the life cycle costs of tank rehabilitation versus replacing the existing tanks with new elevated composite tanks; and 3) assess decommissioning the existing Bourne Hill Tank and constructing a new tank at the site of the new water purification plant (WPP) to provide the necessary levels of distribution storage and contact time (CT) requirements associated with secondary disinfection of WPP discharge.

The distribution storage evaluation task was intended to serve as an update to Section 6.2 of the 2007 Master Plan.

Task 3 – Hydraulic Model Updates

The hydraulic model of the District's water distribution system is a critical tool for assessing level of service issues (i.e. operating pressures, fire flows and pressures, etc.). Periodically updating the model to ensure it is accurately reflecting the physical infrastructure and the operation of that infrastructure is essential to maintaining the system, evaluating levels of service, and simulating the beneficial effects of new infrastructure improvements. The hydraulic model experienced a significant upgrade in 2007 and additional upgrades were performed in 2014, including improvements to the extended period simulation features.

The scope of services for the current Master Plan update included additional hydrant flow testing to further refine the calibration of the hydraulic model, with a particular emphasis on the northwestern area of the water distribution system, which is known to exhibit lower pressures and fire flows. Updates to the extended period simulation capability of the model to capture recent operational changes since 2014 was also addressed. For example, the future operation of the new water purification plant, which is scheduled to begin construction in summer of 2018, will affect the hydraulics in that area of the distribution system. Incorporating the operational characteristics of the new water purification plant into the model was another important goal of the Master Plan update.

The hydraulic model updates task was intended to serve as an update to relevant portions of Section 6.3 of the 2007 Master Plan.

Task 4 – Level of Service Analysis

Key objectives of this task included assessing level of service concerns, including developing a better understanding of operating pressures and available fire flows in the system, water age and quality issues, and the engineering refinement of the water main replacement schedule that was suggested in the District's most recent Asset Management & Fiscal Sustainability Plan (updated June 2017). Modeling scenarios to simulate the effects of various proposed improvements to improve level of service issues, such as new water storage tanks at the current or different locations, represented another aim of this task.

The distribution system issues identified in Tasks 1 through 4 are all inter-related and warranted attention at this time due to the pending need to address worsening deterioration in the coating systems for the West Wareham and Bourne Hill Tanks.

The level of service analysis task was intended to serve as an update to relevant portions of Sections 6.3 and 7.2 of the 2007 Master Plan.

# 2 EXISTING SYSTEM

This section provides an overview of the existing WFD water system, including a general description of the WFD, its water supply sources, water storage facilities, and its distribution system.

## 2.1 WAREHAM FIRE DISTRICT

The Wareham Fire District is located in the Town of Wareham. The WFD does not serve the entire town. Portions of the town rely on private wells or are served by the Onset Fire District (OFD), also located within Wareham.

The WFD serves the majority of the geographic area of the Town. The WFD provides water to more than 18,500 people through a total of 8,260 metered service connections, including 7,763 residential metered service connections, which were in place at the end of 2016. For all of 2016, demands within the WFD averaged 1.57 MGD. However, averaged day demands fluctuate significantly between the winter and summer months, averaging approximately 1.1 MGD during the winter to more than 2.5 MGD during summer periods, with demands on individual days reaching as high as 4.0 MGD.

Significant amounts of land remain available for potential development within the WFD, which suggests reasonable prospects for future growth in terms of new customers served and associated increases in overall water demand. This growth potential applies to properties located within existing WFD service areas and properties that could be served through reasonable extensions of the distribution system.

The WFD's water supply, water storage, and distribution system infrastructure is described in more detail in the following sections.

## 2.2 WATER SUPPLY

The WFD water distribution system is completely supplied by groundwater. There are a total of eight gravel-packed wells that are owned by the WFD, but not all of these are currently developed or active. The Maple Springs wellfield consists of Wells 1 through 4, the Seawood Springs wellfield consists of Wells 6 and 7, and Well 8 is referred to as the South Line Well. Well No. 4 is currently off-line due to high manganese concentrations. Well No. 5 was never developed to enable withdrawal due to interference of drawdown with Wells 1 - 4.

The Wareham Fire District has recently constructed a new well source to its system designated as the Maple Park Well (Well 9), which will be connected to the existing system as part of the new water purification plant project described below. Well No. 9 will supplement the existing sources described above.

The Maple Springs wells and the Seawood Springs wells each have an existing corrosion control facility (CCF) where lime is dosed for pH adjustment and chlorine is dosed for primary disinfection.

Prior investigations have indicated that manganese treatment will be required for Well No. 9, and for increasing iron and manganese levels at Wells No. 3 and No. 4. As a result, the WFD is moving forward with construction of the Maple Springs Water Purification Plant (MSWPP or WPP) to treat groundwater from these and the other Maple Springs wells. The new WPP will be located next to the existing Maple Springs CCF. As part of the WPP project, the WFD is also considering installation of a transmission main to route water from the Seawood Springs wellfield and Well 8 to the new WPP if additional treatment of those sources should become necessary in the future. The new WPP is being provided with the potential expansion capability to treat those well sources.

Summary data for the WFD wells is provided in Table 2-1.

## TABLE 2-1
## SUMMARY OF DATA FOR EXISTING PUMPING FACILITIES

| *Item Description* | *No. 1* | *No 2* | *No. 3* | *No. 4* | *No. 5* [8] | *No.6* | *No. 7* | *No. 8* | *No. 9* [4] |
|---|---|---|---|---|---|---|---|---|---|
| Well Name | Maple Springs | Maple Springs | Maple Springs | Maple Springs | Maple Springs | Seawood Springs | Seawood Springs | South Line | Maple Park |
| Date of Installation | 1946 | 1946 | 1950 | 1955 | 1955 | 1979 | 1989 | 2004 | 2014[4] |
| Type of Well | Gravel Packed | Gravel Packed | Gravel Packed | Gravel Packed | Gravel Packed | Gravel Packed | Gravel Packed | Gravel Packed | Gravel Packed |
| Depth, Feet | 52.5 | 52.4 | 54.4 | 54.1 | - | 61.5 | 84.3 | 115.0 | 96.0 |
| Screen diameter, inch | 14[10] | 12 [6] | 14[9] | 24 | 24 | 24 | 24 | 24 | 18 |
| Pumping Design Rate, gpm | 625 | 625 | 600 | 625 | N/A | 500 | 910 | 1,180 | 1,000 |
| Make of pump | Goulds [1] | Goulds [1] | Goulds [1] | Goulds [1] | N/A | Layne [3] | Byron-Jackson [7] | Goulds | TBD |
| Horsepower | 50 | 50 | 50 | 50 | N/A | 50 | 75 | 100 | TBD |
| Auxiliary Power | Yes [5] | Yes [2] | Yes | Yes | N/A | Yes | Yes | Yes | No |
| Flow Metering | Yes | Yes | Yes | Yes | N/A | Yes | Yes | Yes | Yes |

(1) Pump motor replaced 1987. Pumps replaced in 2017 (from Layne to Goulds).
(2) Auxiliary Engine Replaced 1987
(3) Pump Replaced 1998
(4) To become operational in 2019 as part of WPP Project
(5) Auxiliary Engine Replaced 1998
(6) Screen Replaced 1998
(7) Bowl Assembly Replaced 1998
(8) Well never constructed
(9) Well Re-lined in 2010
(10) 14-inch casing and screen installed in 2017

## 2.3 WATER STORAGE

Three water storage tanks currently provide finished water storage in the WFD. The Glen Charlie Tank was constructed in 2010 and is a composite glass-fused-to-steel bolted tank (AquaStore style) mounted on top of a concrete pedestal. The tank is furnished with an altitude valve vault to prevent overflowing due to system hydraulics. The Glen Charlie Tank is located at 281 Glen Charlie Road and is dedicating to serving the White Island Shores neighborhood located in the northeast area of the District.

The Bourne Hill standpipe was erected in 1956, is constructed of welded steel plates, and is located in the southeast portion of the WFD on Bourne Hill Road. The tank was originally constructed with a lower overflow elevation, but the entire tank was subsequently raised approximately 16 feet to its present overflow elevation.

The West Wareham standpipe was erected in 1967, is also of welded steel plate construction and is in the northwest corner of the WFD. The tank is located immediately south of I-495 near Judith Street.

The size and capacity of these storage tanks is summarized in Table 2-2.

**TABLE 2-2**
**WATER STORAGE SUMMARY**

| Tank Name | Height (ft) | Diameter (ft) | Current Overflow Elev. (ft)(1) | Total Storage (MG) |
|---|---|---|---|---|
| Glen Charlie | 38.5 | 30.8 | 199.31 | 0.22 |
| Bourne Hill | 84 | 50 | 198.71 | 1.23 |
| West Wareham | 85 | 56 | 198.67 | 1.57 |
| **TOTAL** | | | | **3.02 MG** |

(1) Based on NAVD 88 Datum.

All three tanks have essentially the same overflow elevation and are filled by the well pumps described in the preceding section.

Not all the storage volume listed in Table 2-2 is considered available or usable, particularly for the Bourne Hill and West Wareham standpipes. That is because minimum water surface elevations should be maintained in the tanks to achieve at least 35 psi of pressure at the highest customer served in the distribution system. If water levels drop too low in the tanks, pressures at buildings located at higher elevations within the system will fall below 35 psi. The useable, or available, storage volume for the WFD tanks is explored further in the storage capacity evaluation described in Section 4.0 of this report.

Passive hydraulic mixing systems were installed in the Bourne Hill and West Wareham tanks in 2011 to help address water stratification and water quality concerns. The mixing systems were manufactured by Tide-Flex and utilize flexible inlet and outlet valves to allow incoming water to enter near the tops of the tanks and outflowing water to exit near the floors of the tanks. The flexible valves open and close upon differential hydraulic pressure without the need for electrical power.

## 2.4 DISTRIBUTION SYSTEM

According to the WFD's 2017 Asset Management and Fiscal Sustainability Plan, the water distribution system includes the following:

- 170 miles of water main
- 1,290 public fire hydrants
- 3,239 valves
- 8,260 metered service connections

According to the WFD's 2016 Annual Statistical Report (ASR), the WFD serves a population of approximately 20,650 (assuming 2.5 persons per metered connection).

In general, the water distribution system consists of lined and unlined cast iron (C.I.), cement-lined ductile iron (D.I.), asbestos-cement piping (A.C.), high-density polyethylene (HDPE), and polyvinyl chloride (PVC) pipe. Unlined cast iron was used from the District's inception in 1907 until the late 1940's, at which time the District standardized on asbestos-cement piping. Cement-lined ductile iron pipe has been used in recent decades for new water main installation and represents the most common pipe material in the system (approximately 84 miles, or nearly 50%

of the total pipe length). Asbestos-cement pipe currently comprises approximately 58.2 miles of the distribution system (34.3% of total pipe length), and unlined cast-iron accounts for 16.3 miles (9.6% of total pipe length).

Cement-lined cast iron was used for special situations such as bridge and submarine pipe crossings. High Density Polyethylene (HDPE) pipe has been used for river crossings, the Parker Mill Bridge crossing and areas where 2-inch lines have been installed.

Over 70% of the total system length is comprised of water mains 8-inch or less in diameter. 10-inch and 12-inch piping accounts for 22% of total system length. And water mains 16-inch or larger in diameter, which may be considered major transmission mains, account for just over 7% of the total system length. A vast majority of the larger diameter transmission mains are asbestos-cement material.

Currently, there are no physical interconnections with other neighboring water systems. However, potential interconnections with the Town of Marion to the southwest and the Onset Fire District to the east have previously been discussed and may be implemented in the future. The WFD and the Onset Fire District are able to establish a temporary interconnection as conditions warrant by installing temporary hose between two existing hydrants located at the terminus of each water system in Route 6, near the D'Angelo sandwich shop. The two hydrants are located approximately 300 hundred feet apart and, when connected, the water is principally supplied from the WFD to the Onset Fire District, which typically has a lower operating hydraulic grade line.

Pressure in the water distribution system is maintained through control of the water level in the Bourne Hill and West Wareham standpipes. A pressure/level telemetering system at the standpipes transmits the water level in the tanks to the District SCADA system. Normal operation is for pre-selected pumps at the well fields to supply water into the distribution system in response to water level fluctuations in the standpipes.

# 3 WATER DEMAND ANALYSIS

A water demand analysis was performed to estimate current and future water demands within the District's system. Reassessing water demands approximately every 10 years is appropriate to verify prior demand projections and to serve as a basis for assessing overall system infrastructure requirements, including water supply source development, water treatment capacity, pumping and transmission capacity, and water storage requirements, both under current conditions and future conditions.

The water demand analysis consisted of collecting recent water production and consumption data for the District to identify the existing average day and maximum day demands. Population projection data and information relative to future growth and development within the District was also obtained to estimate average day and maximum day demands in future years up to the year 2040.

Existing information and data related to water production, consumption, and water demand was collected from the WFD, including:

- *Annual Statistical Reports (ASRs) and Water Management Act (WMA) Permits for the years 2010 to 2016* – These reports contain annual and monthly consumption and production data for all groundwater supply sources, including the maximum daily consumption for the annual period. Important information on the WFD's supply sources, treatment facilities, and storage facilities is also included in the ASRs.
- SCADA Data for 2016 and most of 2017 – The SCADA data provided by WFD included detailed recorded well pump data for all groundwater wells and tank levels for all three water storage tanks in one-minute increments.
- *Annual Water Consumption Data for 2016* – WFD provided an Excel spreadsheet containing the annual consumption for each metered account.
- *Obligated Demand Data* – WFD provided information regarding the number of existing curb stops that are inactive or not yet connected to buildings. Information on other approved or permitted connections not yet connected to the system were also provided.

## 3.1 EXISTING WATER DEMANDS

### 3.1.1 Existing Water Use and Consumption

The Annual Statistical Reports (ASRs) include annual totals of metered finished water use broken down into several categories according to the type of meter, including residential (i.e. domestic), commercial/business, agricultural, industrial, municipal/institutional, and other. Each year, the WFD is also required to develop a confidently estimated municipal use, which represents known water usage by the District that is not metered. Confidently estimated municipal use, or CEMU, includes water used for fire protection, hydrant flushing, flow testing, water main breaks, blow offs, and other known uses. Water that is not metered and not included in the CEMU estimates are considered unaccounted for water, or UAW. Additional information on the types of uses associated with the different water consumption classifications is included in Table 3-1.

**TABLE 3-1**
**WATER CONSUMPTION CLASSIFICATIONS**

| Class | Definition |
|---|---|
| Domestic | Metered water used in residential dwellings for drinking, bathing, sanitation and outdoor use (i.e. sprinkling, car washing). |
| Commercial / Industrial | Metered water used in retail business, restaurants, motels, etc., or used in manufacturing process plants or agriculture. |
| Municipal / Institutional | Metered water used by town buildings, playing fields, public and private schools, churches, etc. |
| Confidently Estimated Municipal Use (CEMU) | Other estimated municipal water use (fire protection, hydrant flushing, flow testing, blow-offs, known major water main breaks, etc.) |
| Unaccounted for Water | Unknown use not accounted for by the above categories (malfunctioning meters, water main leaks, etc.). |

A summary of annual water demand for WFD from 2010 to 2016 is presented in Table 3-2, broken down by water use category. Certain categories of water use from the ASR reports were consolidated into single columns in Table 3-2 for clarity. For example, commercial, industrial, and agricultural uses are shown in a single column. The total annual water demand shown in the far right hand column of the table also represents the total water production for the WFD, or the total amount of water pumped from all active wells each year. As shown in Table 3-2, total annual water demand has steadily dropped since 2010. Total annual water demand in 2016 was approximately 12.7% less than in 2010.

**TABLE 3-2**

**AVERAGE YEARLY WATER DEMAND**

| Year | Domestic Water Consumption (MG) | Commercial/ Industrial/ Agricultural Water Consumption (MG) | Municipal/ Institutional Consumption (MG) | CEMU and Other Accounted-For Water (MG) | Unaccounted For Water (MG) | Total Annual Water Demand (MG) |
|---|---|---|---|---|---|---|
| 2010 | 410.8 | 57.3 | 30.0 | 61.8 | 94.4 | 654.3 |
| 2011 | 379.2 | 54.3 | 28.1 | 73.2 | 63.7 | 598.5 |
| 2012 | 388.0 | 59.0 | 26.1 | 69.0 | 77.0 | 619.1 |
| 2013 | 384.6 | 59.4 | 21.1 | 71.7 | 63.7 | 600.5 |
| 2014 | 393.6 | 58.2 | 31.1 | 21.9 | 81.2 | 586.0 |
| 2015 | 409.6 | 60.4 | 32.4 | 26.0 | 53.6 | 582.0 |
| 2016 | 409.5 | 61.9 | 33.5 | 29.3 | 37.1 | 571.3 |

A review of the data in Table 3-2 indicates that the drop in overall demand over the last six years is most attributed to reductions in CEMU and Unaccounted-For Water categories. Domestic, commercial, industrial, and municipal categories have remained steady or experienced minor increases.

### 3.1.2 Existing Average Day, Maximum Day and Peak Hour Demands

Table 3-3 includes average day, maximum month, and maximum day demands over the same six-year period. Given the consistent decline in average day demand values over that time frame, the average day demand for the most recent year (2016), which is equal to 1.56 MGD, therefore represents an appropriate value for the existing average day demand for the WFD distribution system. Averaging the annual average day demand values since 2010 as a means of developing the current ADD is not recommended given the established trend in lowering demands.

The recorded annual maximum day demands over the same period has not followed an observable trend. Values have ranged from a low of 3.10 MGD in 2011 to a peak of 4.03 MGD in 2016. These values represent a range of approximately 9% below and 18% above the average maximum day demand value of 3.41 MGD over the six year period. Given the degree of fluctuation in the maximum day demands over the last six years, Kleinfelder recommends assuming the 3.41 MGD for the existing maximum day demand.

**TABLE 3-3**

**EXISTING AVERAGE DAY AND MAXIMUM DAY DEMANDS**

| Year | Total Annual Water Consumption (MG) | Average Day Demand (MGD) | Maximum Month Demand (MGD) | Maximum Day Demand (MGD) | Ratio of Max Day to Average Day Demand |
|---|---|---|---|---|---|
| 2010 | 654.3 | 1.79 | 2.88 | 3.94 | 2.20 |
| 2011 | 598.5 | 1.64 | 2.43 | 3.10 | 1.89 |
| 2012 | 619.1 | 1.70 | 2.50 | 3.18 | 1.87 |
| 2013 | 600.5 | 1.65 | 2.38 | 3.45 | 2.09 |
| 2014 | 586.0 | 1.61 | 2.29 | 3.15 | 1.96 |
| 2015 | 582.0 | 1.59 | 2.33 | 3.05 | 1.92 |
| 2016 | 571.3 | 1.56 | 2.79 | 4.03 | 2.57 |
| **AVG** | **601.7** | **1.65** | **2.51** | **3.41** | **2.07** |

The ratio of maximum day demand to average day demand for the period 2010 to 2016 is 2.07, as shown in Table 3-3. Similar to the methodology employed in the 2007 Master Plan, this same peaking factor will be assumed to estimate the maximum day demand for 2040 (refer to Section 3.2 for the development of the estimated future demands).

The 2007 Master Plan utilized an assumed peak-hour demand to maximum day demand ratio of 1.5 to estimate the peak hour demand. This ratio was selected based on typical published values and by relying on peaking factors in similar New England communities. As part of this current study, Kleinfelder obtained SCADA data that included well pump flows and storage tank levels in 1-minute increments for 2016 and most of 2017 (through November). Kleinfelder utilized the 2017 SCADA data to develop estimates of actual demand in the system in 1-minute increments, which considered the water delivered to the distribution system by the well pumps, as well as the water storage tank inflows/outflows. This data was then used to develop diurnal demand curves for various scenarios, including annual data, monthly data, and curves based on percentile exceedances.

The results of the analysis suggest that during the higher demand periods, including July and August, the peak hour to maximum day peaking factor slightly exceeds 1.5. For example, the maximum day in 2017 occurred on July 5th with a daily demand of 3.11 MGD. The peak hour demand on that day was approximately 4.98 MGD, resulting in a peak hour to maximum day peaking factor of approximately 1.60. Similar peaking factors were also observed based on a more comprehensive statistical evaluation of all demand data. As a result, a peak hour to maximum day peaking factor of 1.60 has been assumed for this study. Applying the 1.60 peaking factor to the current maximum day demand (3.41 MGD) results in an existing peak hour demand equal to 5.46 MGD.

Table 3-4 summarizes the existing average day, maximum day, and peak hour demands for the WFD.

**TABLE 3-4**

**EXISTING DEMAND SUMMARY**

| Condition | Average Day Demand (MGD) | Maximum Day Demand(MGD) | Peak Hour Demand (MGD) |
|---|---|---|---|
| Existing | 1.56[1] | 3.41[2] | 5.46[3] |

(1) Average Day Demand for 2016
(2) Average of the Annual Maximum Day Demand values from 2010 to 2016
(3) Estimated by multiplying Maximum Day Demand times 1.60 peaking factor

Section 3.2 describes the methodology for developing the future (2040) average day, maximum day and peak hour demands.

### 3.1.3 Per Capita Demands

Understanding per capita demands is an important factor in estimating future demands, particularly demand growth associated with various residential developments. To effectively estimate per capita demands in the WFD, it is first necessary to understand the population served by the WFD. The Town of Wareham's permanent population was 22,601 in 2016 according to available U.S. Census Bureau estimates (estimates using 2010 U.S. Census data as a baseline). The entire Town's population except for the area served by the Onset Fire District, resides in the Wareham Fire District service boundaries. Actual population data for the District is not readily available.

Similar to the methodology employed for the 2007 Management Plan, the total population of the District was estimated by multiplying the total number of residential service connections (i.e. number of residential water meters) by typical per capita values. The U.S. Census Bureau estimated the average Wareham household size to be 2.40 people in 2016. Since there are 7,763 residential service connections according to the District's 2016 Annual Statistical Report, the total population for the District is therefore estimated to be 18,631.

The total District population may also be broken down into permanent residents and summer residents. Again, a similar methodology to that performed in 2007 was followed. The American Water Works Research Foundation (AWWARF) found indoor per capita water use (showers, drinking, cooking) to be 58.6 gallons per capita per day (*Residential End Uses of Water*, *Version 2*, 2016). The original 1999 version of the AWWARF report, which was cited in the 2007 Master Plan, referenced 69.3 gallons per capita per day (gpcpd). The downward revision to the per capita per day estimate supports what is now a well understood trend – water usage in homes has steadily declined since 1999. Assuming both the 58.6 gpcpd value from the recent AWWARF report and the 2.40 people per household value from the U.S. Census Bureau results in a typical daily water use of 141 gallons for the typical home in Wareham. Consistent with the approach taken in 2007, metered residential accounts with usage less than or equal to ¼ of 141 gallons/day were assumed to be seasonal properties. The rationale is that seasonal properties are typically occupied approximately 3 months per year during the summer months and are vacant during other times of the year. Utilizing these values, the updated threshold for defining seasonal property status is approximately 1,750 cubic feet per year.

A review of WFD's water meter record data for 2016 indicates that 19.7% of the District's residential service connections recorded less than 1,750 cubic feet of water use in a 12-month period. The water meter record received represented total usage in 2016 for each account. Individual meter readings conducted throughout the year were not provided, so water use for the entire 12-month period for each account was assessed.

The District's permanent population and summer population were calculated based on the assumption that 19.7% of the service connections apply to summer-only residents and 80.3% of the service connections apply to permanent year-round residents. Applying these percentages to the total District population of 18,631 calculated previously results in permanent and summer populations of 14,961 and 3,670, respectively.

**TABLE 3-5**

**RESIDENTIAL PER CAPITA WATER USE**

| Year | Permanent Town Population[1] | Number of Residential Metered Connections[2] | Estimated Total District (Summer) Population[3] | Estimated District Permanent (Winter) Population[4] | Weighted Average District Population[5] | Annual Domestic Water Consumption (MG) | Residential Per Capita Water Use (gpcd) |
|---|---|---|---|---|---|---|---|
| 2010 | 21,822 | 7,206 | 17,294 | 13,887 | 14,739 | 410.8 | 76.4 |
| 2011 | 22,222 | 7,650 | 18,360 | 14,743 | 15,647 | 379.2 | 66.4 |
| 2012 | 22,343 | 7,691 | 18,458 | 14,822 | 15,731 | 388.0 | 67.6 |
| 2013 | 22,420 | 7,711 | 18,506 | 14,861 | 15,772 | 384.6 | 66.8 |
| 2014 | 22,480 | 7,685 | 18,444 | 14,811 | 15,719 | 393.6 | 68.6 |
| 2015 | 22,519 | 7,721 | 18,530 | 14,880 | 15,793 | 409.6 | 71.1 |
| 2016 | 22,601 | 7,763 | 18,631 | 14,961 | 15,878 | 409.5 | 70.7 |
| **AVG** | | | | | | | **69.6** |

(1) Based on U.S. Census Bureau data and estimates for 2010 to 2016 as described in Section 3.1.3
(2) Number of residential metered connections taken from ASR reports
(3) Total District population estimated by multiplying number of WFD residential meters x assumed 2.4 persons per household, in accordance with U.S. Census Data estimates. Total District population is assumed equal to District Summer population (i.e. permanent plus summer residents).
(4) Permanent (Winter) population assumed equal to 80.3% of total District population. Summer-only population assumed equal to 19.7% of total District Population.
(5) Weighted Average District population = [(Total District/Summer population x 3) + (Permanent Winter population x 9) ]÷12

## 3.2 FUTURE WATER DEMANDS

The 2007 Master Plan adopted a hybrid approach to estimate future water demands within the WFD, which considered both anticipated development and population projections over the 20+ year planning period. Anticipated residential and commercial development was primarily relied upon to estimate water demands at the 10-year milestone of the planning period, and population projections were used to estimate demand growth from the 10-year milestone to the end of the planning period. A similar method was utilized during this study to develop water demand projections for 2040.

### 3.2.1 Anticipated Development

Anticipated development can be divided into two components: obligated demand, or approved development; and potential, or unapproved, development.

Obligated demand represents water that the District has already agreed to supply, or is required to supply, which is not reflected in the current water use. This includes previously approved permits associated with new residential or commercial development projects that have not fully or partially connected to WFD's distribution system. Obligated demand also includes existing water services that have been extended from the water main in the street to a curb stop near the property line, but which has not been connected to the building on the property. In some cases, curb stops have been extended to the property line of vacant lots, which are also considered obligated demand. And most properties located on a street with an existing water main are usually considered obligated demand, whether a curb stop has been installed to-date or not, as the WFD would typically approve such hook-up requests (provided it is consistent with WFD rules and regulations).

Potential, or unapproved, development, is different from obligated demand in that it includes development projects that have not yet received formal approval or been granted a permit to connect to the existing water system. For example, a large proposed subdivision or significant commercial development that is in the early planning stages or still requires approval by the Planning Board would represent potential development. For the purposes of this study, potential development projects are those that have a very high probability of undergoing construction and hooking up to the water system within the next 10 years.

Based on data provided by WFD, there are 243 obligated residential service connections associated with active development projects. Approximately 78% of those connections are attributable to the five largest existing residential projects, as follows:

- Rosebrook Place (65 remaining connections)
- Plymouth Ave, Plymouth (45 remaining connections)
- Pond at Fearing Hill (44 remaining connections)
- Tihonet North, Cranes Landing (21 remaining connections)
- Hathaway Estates (14 remaining connections)

There are also 26 obligated commercial service connections, including eight (8) associated with the Wareham Market Place Mall, six (6) associated with Charlotte Furnace LLC, and three (3) associated with Robertson's Corner.

The District also includes 777 existing, unconnected residential curb stops. This total includes 181 curb stops that were previously active, but are now inactive, as well as 596 curb stops that have been installed but not yet connected to buildings.

The obligated demands described above will result in additional water demands for the WFD water system. The extent of the demand increase will be dependent upon how many of these connections are actually completed. Comparing the projected number of new service connections from the 2007 Master Plan to the actual number of service connections completed over the ensuing 10 years can lend important insight to aid in developing the projections as part of this Master Plan update.

For the ten-year period from 2006 through 2016, the WFD added 419 residential service connections (7,344 to 7,763) and 65 commercial service connections (220 to 285). However, the 2007 Master Plan identified a total of 1,417 obligated residential service connections, 51 obligated commercial service connections, and additional commercial growth associated with several major potential (unapproved) development projects (i.e. projects in the application stage at that time).

Therefore, the actual number of new residential service connections during the prior ten years was equivalent to approximately 30% (419 ÷1,417) of the obligated residential service connections that were identified in 2006. The actual number of new commercial service connections during the prior ten years was equivalent to 127% (65/51) of the obligated commercial service connections identified in 2006. It is important to note that during the 2007 Master Plan, additional commercial growth above and beyond the 51 obligated commercial service connections was envisioned, which was associated with potential (unapproved) development anticipated from 2006 to 2016.

To be conservative, it is assumed that 75% of the known obligated residential service connections will be connected over the next ten years. It is further assumed that 100% of the known obligated commercial service connections will become connected during the planning period. From 2006 to 2016 commercial water use increased by approximately 10 MG per year. To achieve at least the same level of commercial growth over the next 10 years that has occurred over the last 10 years, it is further assumed that an additional 33 commercial service connections will be connected in addition to 100% of the current obligated commercial service connections. The total number of new commercial service connections assumed over the next 10 years is therefore equal to 59, as shown in Table 3-6, which equates to 27,700 gpd, or 10 MG per year. A summary of estimated additional demand from anticipated development is provided in Table 3-6.

**TABLE 3-6**
**ESTIMATED ADDITIONAL DEMAND FROM ANTICIPATED DEVELOPMENT**
**(2016 – 2026)**

| Demand Category | Potential New Service Connections | Additional Demand Per Connection (gpd)[1,2] | Assumed 10-Year Connection Rate (%) | Estimated Additional Average Day Demand (gpd) |
|---|---|---|---|---|
| Obligated Service Connections (Residential) | 243 | 167 | 75% | 30,400 |
| Unconnected Curb Stops (Residential) | 777 | 167 | 75% | 97,300 |
| Obligated Service Connections (Commercial) | 26 | 471 | 100% | 12,200 |
| Additional Commercial Growth | 33 | 471 | 100% | 15,500 |
| **TOTAL** | **1,106** | | | **168,200** |

(1) Additional demand per new residential service connection equal to 69.6 gpcd (Table 3-5) x 2.4 residents per household.
(2) Additional demand per new commercial service connection equal to total commercial demand for 2016 (49.0 MGY) ÷ total number of commercial service connections in 2016 (285), as described in the 2016 ASR.

As shown in Table 3-6, the total estimated increase in the average day demand from 2016 to 2026 is approximately 168,200 gpd (0.17 MGD). Anticipated development was not considered in estimating future demand increases after 2026 because of the inherent uncertainty involved with longer-term projections. Therefore, demand increases from 2026 to 2040 were based solely on population projections, as described in Section 3.2.2.

### 3.2.2 Population Projections

Consistent with the methodology used in 2007, population projections were relied upon for estimating future water demand increases in the WFD system through the end of the planning period (Year 2040).

The population of the Town of Wareham is provided by the U.S. Census Bureau in 10-year intervals. For the 10-year period between the completion of each full census, the U.S. Census Bureau also provides an estimated annual census based on the population estimates program (PEP). Table 3-7 below shows the Town's U.S. Census Bureau population data since 1980. Populations for 1980, 1990, 2000, and 2010 are based on actual census data, and the population data for 2011 through 2016 are based on the U.S. Census Bureau PEP data estimates.

**TABLE 3-7**

**TOWN OF WAREHAM, MA PERMANENT POPULATION DATA (U.S. CENSUS BUREAU)**

| Year | Population[1] | Annualized Growth Rate |
|---|---|---|
| 1980 | 18457 | - |
| 1990 | 19,232 | 0.41% |
| 2000 | 20,335 | 0.56% |
| 2010 | 21,822 | 0.71% |
| 2011 | 22,222 | 1.83% |
| 2012 | 22,343 | 0.54% |
| 2013 | 22,420 | 0.34% |
| 2014 | 22,480 | 0.27% |
| 2015 | 22,519 | 0.17% |
| 2016 | 22,601 | 0.36% |

(1) Populations for 1980, 1990, 2000, and 2010 are based on actual U.S. Census data, and the population data for 2011 through 2016 are based on the U.S. Census Bureau PEP data estimates.

The U.S. Census Bureau population data served as a baseline for the population projections from 2016 to 2040, the end of the planning period for this study. To estimate population growth in future years, data provided by the Massachusetts Institute for Social and Economic Research (MISER) was relied upon. MISER prepares population projections that employ a component-of-change model in which fertility, mortality and migration are projected independently. Since the 2007 Master Plan, MISER has transitioned away from the "Low, Medium, High" projection methodology and has simplified their model to include only the “medium” value. The new method is referred to as the UMDI series. The current UMDI method provides projections in 5-year time intervals vs. the prior “low, medium, high” method that provided projections in 10-year time intervals. A summary of MISER, UMDI series projections are provided in Table 3-8. The projections are based upon the most recent full census completed by the U.S. Census Bureau (2010).

**TABLE 3-8**

**TOWN OF WAREHAM, MA (MISER) POPULATION PROJECTIONS**

| Year | Population | Annualized Growth Rate |
|---|---|---|
| 2010 | 21822 | - |
| 2020 | 24089 | 0.99% |
| 2025 | 24981 | 0.73% |
| 2030 | 25584 | 0.48% |
| 2035 | 26004 | 0.33% |

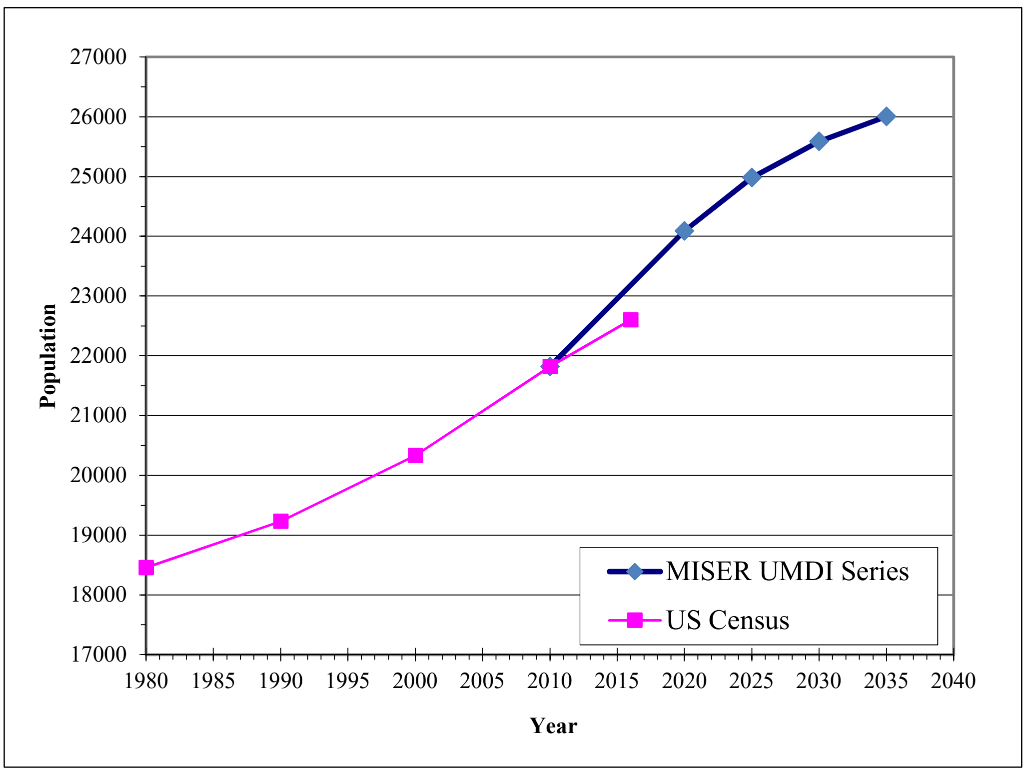


**FIGURE 3-1**

**WAREHAM, MA POPULATION PROJECTION COMPARISON**

The population data from the U.S. Census Bureau up to 2016 and the MISER projections from 2010 through 2035 were plotted to compare the two data sources. This comparison is shown in Figure 3-1.

To reconcile the slight differences between the two data sources, the MISER projected annualized growth rate(s) shown in Table 3-8 were applied to the 2016 U.S. Census Bureau PEP data estimate, the most recent available from the U.S. Census Bureau that served as a baseline. For example, to project population from 2016 through 2020, an annualized growth rate of 0.94% (i.e. MISER projected annualized growth from 2010 to 2020) was applied to the U.S. Census Bureau population estimate for 2016. Estimated population for future years was then determined by applying the MISER annualized growth rates for the successive 5-year periods shown in Table 3-8.

**TABLE 3-9**
**TOWN AND WFD POPULATION PROJECTIONS**

| Year | Estimated Town Population | Annualized Growth Rate | Estimated Total District (Summer) Population[1] |
|---|---|---|---|
| 2010 (Actual) | 21,822 | 0.71% | 17,294 |
| 2011 | 22,222 | 1.83% | 18,360 |
| 2012 | 22,343 | 0.54% | 18,458 |
| 2013 | 22,420 | 0.34% | 18,506 |
| 2014 | 22,480 | 0.27% | 18,444 |
| 2015 | 22,519 | 0.17% | 18,530 |
| 2016 | 22,601 | 0.36% | 18,631 |
| 2020 | 23,512 | 0.99% | 19,374 |
| 2025 | 24,383 | 0.73% | 20,092 |
| 2026 | 24,500 | 0.48% | 20,188 |
| 2030 | 24,972 | 0.48% | 20,577 |
| 2035 | 25,382 | 0.33% | 20,915 |
| 2040 | 25,798 | 0.33% | 21,258 |

(1) Estimated Total District (Summer) Population for 2010 to 2016 is taken from Table 3-5 and is based on the actual number of metered residential connections. For 2020 to 2040, the Estimated Total District (Summer) Population was estimated by multiplying the Estimated Town Population by a factor equal to the Estimated Total District Population for 2016 ÷ the Estimated Town Population for 2016 (18,631÷22,601 = 0.824).

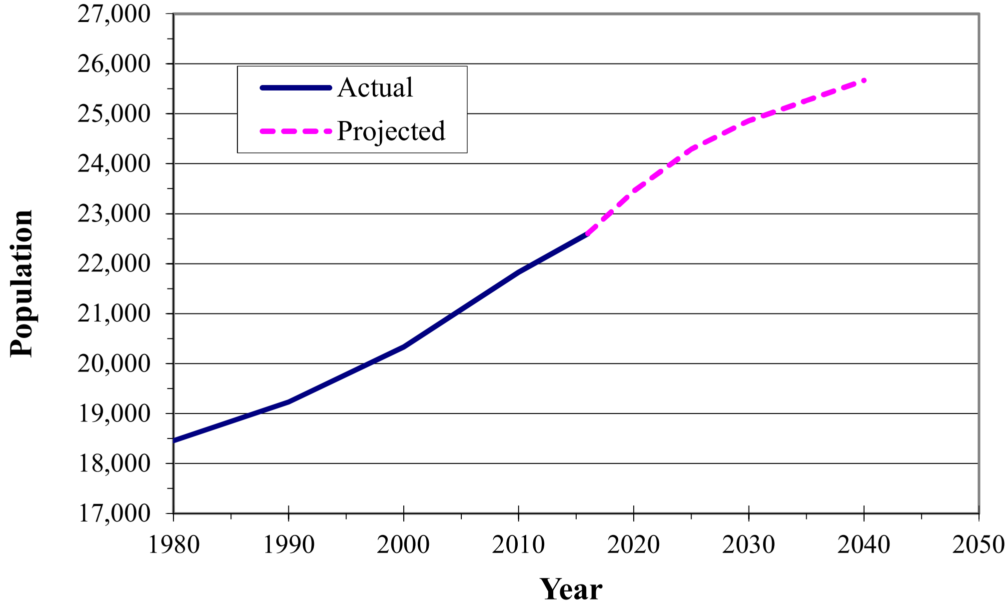


**FIGURE 3-2**

**TOWN PERMANENT POPULATION PROJECTIONS**

Table 3-9 summarizes the population projections for the Town of Wareham and for the District (summer). Figure 3-2 illustrates the estimated Town of Wareham population projections graphically. The estimated additional average day demand in the WFD associated with the population projections from 2016 to 2040 is included in Table 3-10.

**TABLE 3-10**

**SUMMARY OF ADDITIONAL DEMAND FROM POPULATION GROWTH (2016 – 2040)**

| Year | Estimated Total District (Summer) Population | District Population Growth | Estimated Additional Average Day Demand (gpd)[1] |
|---|---|---|---|
| 2016 | 18,631 | - | |
| 2040 | 21,258 | 2,627 | **182,839** |

(1) Additional demand based on 69.6 gpcd (from Table 3-5).

### 3.2.3 Future Average Day, Maximum Day, and Peak Hour Demands

Future average day, maximum day and peak hour demands are summarized in Table 3-11. Future average day demands are equal to the existing average day demand plus the additional demands from anticipated development (Table 3-6) and future population growth (Table 3-10).

**TABLE 3-11**
**EXISTING AND FUTURE (2040) DEMAND SUMMARY**

| Condition | Average Day Demand (MGD) | Maximum Day Demand(MGD) | Peak Hour Demand (MGD) |
|---|---|---|---|
| Existing | 1.56[1] | 3.41[2] | 5.46[3] |
| 2040 | 1.91[4] | 3.95[5] | 6.32[3] |

(1) Average Day Demand for 2016
(2) Average of the Annual Maximum Day Demand values from 2010 to 2016
(3) Estimated by multiplying Maximum Day Demand times 1.60 peaking factor
(4) Existing Average Day Demand plus the additional demands from Table 3-6 and Table 3-10
(5) Estimated by multiplying Average Day Demand times 2.07 peaking factor

Based on the analysis, the average day demand is estimated to increase by approximately 22% between 2016 and 2040. Maximum day demands and peak hour demands are estimated to increase by approximately 16%.

# 4 DISTRIBUTION STORAGE EVALUATION

A distribution storage evaluation was conducted to 1) determine if existing distribution system storage is adequate for current and future demand conditions and to assess the need for additional storage capacity, and 2) compare the life cycle costs of rehabilitating the existing steel tanks versus replacing the tanks with new elevated composite tanks. Additional tank-related tasks requiring hydraulic modeling simulations are described in Section 6 of this report (Level of Service Analysis). This includes an assessment of decommissioning the existing Bourne Hill Tank and replacing it with a new elevated composite tank at the site of the new water purification plant (WPP) to provide both distribution storage and contact time (CT) requirements associated with secondary disinfection of WPP discharge.

## 4.1 STORAGE CAPACITY EVALUATION

### 4.1.1 Storage Components

Table 4-1 summarizes the key characteristics of the three water storage tanks located in the WFD water system. Data is provided on each tank including: tank height, tank diameter, overflow elevation, total storage volume, minimum allowable tank level, and available usable storage volume.

In accordance with the American Water Works Association (AWWA) standards, there are three major components of tank storage to consider: equalization storage; fire storage; and emergency storage. Equalization storage represents the top layer of storage in the tank and includes the volume needed to make up the difference between peak daily flows and the water produced by the water supply source, or the groundwater wells in the case of the WFD. From a design perspective, equalization storage should equal the volume necessary to make up the difference between peak hourly demands and the maximum day demand, provided the water supply sources can meet the maximum day demand. From an operational perspective, equalization storage may include the volume of storage between the overflow elevation and the level in the tanks at which the pumps supplying the tanks (i.e. the well pumps in the WFD) turn on to refill the tanks (i.e. the normal operating range of the tank.) Different methodologies are available to estimate the amount of equalization storage. Often, equalization storage is estimated by assuming it is equal to some percentage (15% - 25%) of the maximum day demand or, alternatively, equal to the peak hour demand on the maximum day, or some multiple of the peak hour demand. The selection of which method to use is ultimately based on the specifics of the water system and engineering judgement. Prior WFD planning documents, including the 2007 Master Plan, have assumed the required equalization storage is equal to the future peak hourly demand on the maximum day.

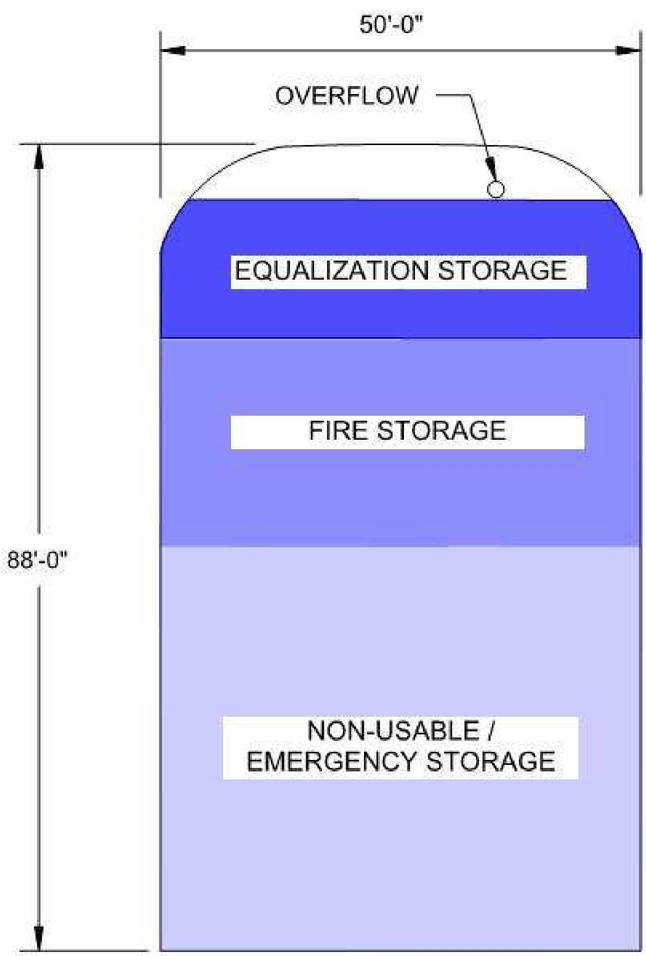


**FIGURE 4 – 1**
**BOURNE HILL TANK STORAGE COMPONENTS**

Fire storage includes the volume necessary to provide some maximum flow rate for a set period of time in accordance with ISO requirements. Minimum recommended fire flows based on ISO requirements vary by water system, and even smaller geographic areas within each water system, according to building density, land use, zoning and other factors.

The 2007 Master Plan identified a WFD maximum fire flow requirement of 3,500 gpm for 3 hours, or 630,000 gallons total. This fire flow was also subsequently utilized in an assessment of tank sizing and replacement options conducted by Kleinfelder for the WFD, which was summarized in a Technical Letter to the WFD dated May 22, 2012. This same fire flow requirement and storage volume was also utilized for this storage capacity evaluation.

Emergency storage includes the lowest layer of storage in the tank and is reserved for unforeseen emergencies that might occur when the equalization and fire storage volumes have been depleted. Since the use of emergency storage applies to very rare events that are not well defined (i.e. extended loss of supply for instance), there are typically no specific level of service requirements associated with sizing this component of storage. Incorporating emergency storage is system specific and needs to be balanced against several factors, including the overall supply capacity of the system, water quality associated with larger storage volumes, redundancy within the system, and other factors. Emergency storage was not specifically addressed in prior WFD planning analyses, due in part to the abundant available supply capacity in the WFD, which as noted in Section 3 well exceeds the maximum day (existing and future) and is very nearly equal to the future peak hour demand, depending on which of the wells are assumed to be simultaneously available at any given time. The water stored below the minimum allowable water surface elevation in the tanks, and therefore considered not 'useable', may be considered emergency storage by default.

### 4.1.2 Available Useable Storage

Existing available useable storage is summarized in Table 4-1.

**TABLE 4-1**
**EXISTING AVAILABLE USEABLE STORAGE**

| Tank Name | Height (ft) | Dia. (ft) | Overflow Elev. (NAVD 88)[1] | Total Storage (MG) | Min. Allowable Tank Level (MSL) | Available Useable Storage (MG) |
|---|---|---|---|---|---|---|
| Glen Charlie | 38.5 | 30.8 | 199.31 | 0.22 | 163.5 | 0.20 |
| Bourne Hill | 84 | 50 | 198.71 | 1.23 | 163.5 | 0.52 |
| West Wareham | 85 | 56 | 198.67 | 1.57 | 163.5 | 0.65 |
| **TOTAL** | | | | **3.02** | | **1.37** |

(1) Based on 2016 survey conducted by WFD.

Typical industry practice (and MassDEP guidance) is to maintain at least 35 psi at the highest building elevation served by the storage tanks when the equalization storage volume has been depleted. Prior WFD planning efforts, including the 2007 Master Plan and the aforementioned

May 22, 2012 Technical Letter, have assumed a more stringent approach of maintaining at least 35 psi at the highest elevation served when *both* the equalization storage *and* fire storage have been depleted. The intersection of Main Street and Hathaway Road has traditionally been observed as having the highest building elevation served in the WFD distribution system at 83 feet (MSL). The minimum allowable tank level shown in Table 4-1 (163.5 feet MSL) is based on maintaining at least 35 psi at this location (83 feet + (35 psi x 2.3 feet/psi)). Based on these assumptions, the total available useable storage in all three WFD storage tanks equals 1.37 MG.

As noted in Section 5, one of the model updates performed by Kleinfelder as part of this study was incorporating more accurate MA State LiDAR survey data into the model. Node elevations in the model have been updated to reflect the LiDAR data, which is considered more accurate than prior topographic informational references (e.g. USGS maps, etc.). Based on the updated nodal elevations in the model, the highest elevation served is located on Judith Street near the West Wareham Tank, which has an approximate building elevation equal to 105 feet MSL. Pressures of 35 psi can only be maintained at this location if levels in the West Wareham Tank stay above 185 feet MSL (105 feet + (35 psi x 2.3 feet/psi)). Currently, the West Wareham and Bourne Hill tanks drop to approximately 175 feet during a typical drain/fill cycle, meaning that pressures near Judith Street regularly fall to approximately 30 psi just prior to when the well pumps kick on and start filling the tanks. Fire events that cause the water surface elevations in the tanks to drop below the 175 feet level result in even further lowering of available pressures on Judith Street. For example, if water surface levels in the tanks drops to the minimum allowable tank level during a fire event, or 163.5 feet MSL, then pressures on Judith Street would fall to approximately 25 psi. It is important to note that hydraulic modeling, discussed in Section 6.0, indicates that adequate fire protection can be provided at this location due to proximity to the West Wareham Tank.

It is therefore recommended that the long-term storage solution for the WFD, whether it be retaining the existing welded steel tanks or replacing those tanks with new elevated storage tanks, consider the preference to increase operating pressures on Judith Street, particularly when only equalization storage has been depleted, to be more consistent with industry practice. For example, depending on the storage solution that is implemented, it may be desirable to alter the typical operating range of the tanks so that the well pumps turn on at a higher elevation. If the well pumps were to turn on when the water surface elevation in the tanks reaches 185 feet MSL instead of approximately 175 feet MSL as is the current practice, then minimum pressures under normal operating conditions would increase from approximately 30 psi to nearly 35 psi on Judith Street, which is more consistent with industry practice and MassDEP guidance.

It is recognized that changing the operating range of the existing tanks represents a challenge due to water age and water quality issues, but such a change may be more feasible if the existing tanks are replaced with elevated composite tanks, similar to the Glen Charlie Tank. Generally speaking, the total volume of the operating band of all tanks should not exceed the required equalization storage volume. Otherwise, as the tanks draw down during a normal drain cycle it will begin to deplete fire storage. If the Bourne Hill and West Wareham tanks were replaced with elevated composite tanks having similar bottom of bowl elevations as that of the Glen Charlie Tank (i.e. 161.5 MSL), a 10-foot operating band (188 feet to 198 feet) would constitute more than 25% of the overall storage volume, providing ample draw down volume that is more aligned with the required equalization storage volume (refer to Section 4.1.3). It would also serve to raise minimum operating pressures throughout the system, including Judith Street, by approximately 5 psi.

### 4.1.3 Required Useable Storage

Required useable storage is the sum of required equalization storage, fire storage, and emergency storage. As indicated previously, prior WFD planning efforts have considered the peak hourly demand on the maximum day as a reasonable estimate for the required equalization storage. The demand analysis described in Section 3 of this report estimated the future (2040) peak hourly demand for the WFD system to be 6.32 MGD (refer to Table 3-11), which equates to approximately 0.27 MG over the course of one hour. And as described in Section 4.1.1, required fire storage is equal to 0.63 MG. Therefore, total required useable storage in the WFD is equal to at least 0.9 MG.

### 4.1.4 Summary of Storage Capacity Evaluation

The storage capacity evaluation suggests that the existing *available* useable storage volume (1.37 MG from Table 4-1) exceeds the *required* amount of useable storage (0.9 MG) by approximately 0.47 MG, and that existing levels of useable storage are sufficient to meet the demands of the WFD system through the end of the planning period. The surplus useable storage capacity of 0.47 MG is based on the geometry of the existing storage tanks and provides a level of insurance against any unexpected increases in demand beyond that anticipated according to the water demand analysis summarized in Section 3.

**TABLE 4-2**
**STORAGE CAPACITY EVALUATION SUMMARY**

| Storage Component | Volume (MG) |
|---|---|
| *Required Useable Storage:* | |
| Equalization Storage | 0.27 |
| Fire Storage | 0.63 |
| Total | 0.90 |
| *Total Available Useable Storage:* | 1.37 |
| ***Total Surplus Useable Storage:*** | **0.47** |

As shown in the Table 4-1, the total volume of the three (3) existing water storage tanks is 3.02 MG. The difference between the total storage volume (3.02 MG) and the total required useable storage volume (0.90 MG) may be considered the volume of emergency storage that is available for other unforeseen events and is equal to approximately 2.12 MG (3.02 – 0.90), based on the existing tanks. The total *surplus* useable storage volume in Table 4-2 (0.47 MG) may be considered part of the emergency storage volume. In other words, it represents the portion of emergency storage that, if depleted, would still meet the minimum service level requirements in the system (i.e. 35 psi at the traditional high point of Main Street and Hathaway Road).

It is also important to note that the available supply capacity in the WFD system equals approximately 3,885 gpm, or 5.6 MGD (assuming Well Nos. 8 and 9 are out of service) and approximately 4,885 gpm, or 7.0 MGD (assuming only Well No. 8, the largest supply source, is out of service). This supply capacity significantly exceeds the future (2040) maximum day demand of the system (3.95 MGD) and nearly meets or exceeds the future (2040) peak hourly demand of the system (6.32 MGD) depending on the assumption of which of the larger wells are out of service.

The amount of *surplus* useable storage (0.47 MG) is important because if new storage tanks are eventually constructed to replace the existing West Wareham and Bourne Hill steel storage tanks, the amount of useable storage constructed for the new tanks could be significantly less compared to the useable storage in the existing tanks. Constructing smaller tanks would reduce capital costs associated with the new tanks and potentially improve water quality through enhanced water turnover compared to the existing tanks. The potential replacement of the existing steel tanks with elevated composite tanks is discussed in Section 4.2

## 4.2 TANK IMPROVEMENT ALTERNATIVES EVALUATION

### 4.2.1 General

Rehabilitation of the steel storage tanks, which typically includes sand-blasting to remove the old coating system, spot repairs, and repainting, represents a significant capital expenditure that the District is required to repeat approximately every 15 years. Before committing to a repeating cycle of major rehabilitation costs to maintain the aging tanks, the WFD elected to evaluate the potential replacement of the existing tanks with new tanks. Since a significant portion of the water stored in the lower portion of the existing steel tanks is not even considered useable, the total storage volume of new tanks could be significantly reduced by utilizing an elevated composite style tank.

The last tank assessment, conducted by Kleinfelder and summarized in a May 22, 2012 Technical Letter, looked specifically at replacing the existing tanks with elevated bolted glass-fused-to-steel storage tanks mounted on a concrete pedestal. This style of tank is similar to the Glen Charlie Tank that was constructed in 2010, and the WFD has expressed interest in standardizing to this type of tank to the extent possible and if replacement of the tanks was proven justified. Key benefits of the bolted glass-fused-to-steel tank include the following:

- Exterior glass-fused-to-steel surfaces do not require re-painting
- The concrete pedestal on which the tank is mounted does not require re-painting
- Significantly lower annual maintenance costs compared to welded steel tanks
- The interior of the concrete pedestal can be used for other purposes, such as vehicle and equipment storage
- This style of tank can be more easily modified for future expansion by unbolting the base of the tank and installing new bolted panels to increase the height of the tank
- Improved water age by eliminating the storage of non-useable water in the lower portion of the tanks

If new elevated bolted glass-fused-to-steel tanks were to replace the existing Bourne Hill and West Wareham tanks, the size of the new tanks could be significantly reduced, as suggested by the storage capacity evaluation that was performed and summarized in Section 4.1. Since the existing Glen Charlie Tank provides approximately 0.2 MG of useable storage (0.22 MG total), the combined useable storage volume of the new Bourne Hill and West Wareham tanks could be reduced to approximately 0.7 MG to achieve the 0.9 MG of required useable storage volume (refer to Table 4-2).

However, the overall volume of the storage tanks would already be significantly reduced by installing elevated style tanks, which would effectively remove the non-useable portion of water storage that currently exists in the lower portion of the existing tanks. Therefore, it is assumed that the new tanks would be sized to preserve some amount of the existing surplus useable storage that is estimated to exist in year 2040 as a contingency against unknown factors, and to allow for modest growth occurring in the system beyond 2040. For this analysis, it is assumed that surplus useable storage equal to roughly 50% of the existing surplus useable storage (0.47 MG) will be available at the end of the planning period, which equates to an additional 0.25 MG (50% of 0.47 MG). Therefore, it is assumed the total combined volume of the new Bourne Hill and West Wareham replacement tanks would be at least 0.95 MG (0.7 MG needed to provide minimum amount of useable storage in 2040 + 0.25 MG contingency). Total system storage would then equal approximately 1.17 MG with the Glen Charlie Tank, with nearly all of that volume considered useable storage.

The nominal volume of each new tank would therefore equal approximately 475,000 gallons, with the bottom of bowl elevations roughly corresponding to the minimum allowable water surface elevation shown in Table 4-1 (i.e. 163.5 feet MSL). Overflow elevations would remain at approximately 200 feet MSL so that that all three tanks, including the Glen Charlie Tank, could continue to operate in a single pressure zone at approximately the same HGL.

### 4.2.2 Life Cycle Cost Analysis - Tank Rehabilitation versus Tank Replacement

A life cycle cost analysis was conducted for both the tank rehabilitation option and the tank replacement option to provide an objective basis for comparison. Since new storage tanks are generally considered to have a useful life of 75 years, that duration was selected for the analysis. Furthermore, costs are based on rehabilitation or replacement of both the Bourne Hill Tank and the West Wareham Tank, as both tanks are approximately equal in overall age and follow similar rehabilitation schedules.

For the rehabilitation option, costs include miscellaneous repairs and re-painting the full interior and exterior of each tank at 15-year intervals for the next 75 years, in keeping with current WFD practice. A 15-year painting schedule is considered typical for steel water storage tanks in New England. Costs for the rehabilitation option are based on an assumed tank painting cost equal to

$23/square foot, and a 15% contingency for engineering and other costs. For the rehabilitation option, it is assumed that the existing tanks will not require replacement for at least another 75 years.

For the tank replacement option, costs are based on demolishing the existing tanks and replacing with new elevated composite tanks (bolted glass-fused-to-steel bowl mounted on top of concrete pedestal). Costs for the new tanks include demolition of the existing steel storage tanks (excluding any salvage value), construction of the elevated composite tanks, site work, and a 25% contingency for engineering and other costs. New tank costs are based on recent estimates provided by a leading U.S. manufacturer and installer of bolted glass-fused-to-steel tanks. Costs for the new tanks also assume inspections of the cathodic protection systems and replacement of the galvanic anodes every five years ($4,500 per tank in $2018). Economy of scale savings associated with constructing two tanks simultaneously were not factored into the analysis.

To conduct the life cycle cost analysis, it was necessary to develop assumptions related to inflation costs over the 75-year life cycle period. Review of Engineering News Record's Construction Cost Index (ENR CCI) data for the last 75 years indicates an annualized inflation rate of approximately 5% for the construction industry. The last 30 years of ENR CCI data reveals a lower annualized inflation rate of approximately 3%. Since there is no assurance that the lower inflation rate realized over the last 30 years will continue for the next 75 years, a slightly higher inflation rate of 3.25% was assumed for the life cycle cost analysis. The actual dollar amounts expended in future years are determined by applying this inflation to the current costs (2018 dollars) associated with those items.

Once specific costs are inflated into future years using the inflation rate, their net present worth is then determined by bringing those actual costs back to present day using what is referred to as a capital investment or discount rate. The capital investment discount rate determines what amount of money would need to be set aside today to pay for the expense at some point in the future. For this analysis, 4% was assumed for the capital investment discount rate (capital discount rate), which is intended to represent current yields associated with relatively safe fixed-income type of investments, such as U.S. Treasury Bonds (30-year) and other investment-grade bonds.

As an example, an improvement that costs $1.00 today would equal $1.38 in actual dollars 10 years from now, assuming an annualized inflation rate of 3.25%. And approximately $0.93 would need to be set aside in an account today earning 4% interest to pay for the $1.38 improvement 10 years from now. Therefore, the improvement would have a net present value of $0.93.

Generally speaking, higher inflation rates tend to make options with more up-front capital costs more attractive, while higher capital discount rates tend to make options that are more maintenance intensive more attractive from a financial perspective.

In reality, the full amount of the net present value of known future expenditures would *not* be set aside as an investment in Year 1 (i.e. for the tank rehabilitation option). More likely, lesser amounts might be set aside annually in a capital reserve fund over the course of many years and withdrawn as the expenditures occur to help defray costs. That would have the effect of increasing the net present value, provided the capital discount rate exceeds the inflation rate. Likewise, the full amount of the up-front capital costs associated with new elevated storage tanks would not be completely paid in full by the District in Year 1 (i.e. for the tank replacement option). More likely, the up-front capital costs would be bonded, resulting in principle and interest payments for the District for 15 – 30 years. That may have the effect of increasing the net present value as well, provided the amortization rate on the loan exceeds the inflation rate. But if a Drinking Water SRF loan were obtained with an amortization rate lower than the inflation rate, the net present value could decrease.

The results of the life cycle cost analysis are summarized in Table 4-3.

**TABLE 4-3**
**LIFE CYCLE COST ANALYSIS – TANK IMPROVEMENT OPTIONS**

| Cost Component | Tank Improvement Options (Bourne Hill and West Wareham Tanks) | |
|---|---|---|
| | Tank Rehabilitation (Painting) | Tank Replacement |
| Initial Capital Costs ($2018) | $1,711,000 | $5,597,000 |
| Net Present Value of Future Capital Upgrades and O&M ($2018) | $6,249,000 | $101,000 |
| **Total Net Present Value ($2018)** | **$7,960,000** | **$5,698,000** |

(1) Assumes 75-year life cycle, with Year 1 beginning in 2019; 3.25% inflation rate; 4.0% capital discount rate.
(2) Future capital upgrades for tank rehabilitation option includes complete repainting of both tanks every 15 years.
(3) Costs represent Engineer's Preliminary Opinion of Probable Costs.

The results of the life cycle cost analysis suggests that the net present value of costs for the tank rehabilitation option (i.e. repainting every 15 years) is higher than the net present value of costs for the tank replacement option (i.e. replacement with new elevated composite tanks). The net present value of costs for the tank rehabilitation option is $7,960,000, compared to $5,698,000 for the tank replacement option. It is important to note that adjustments in the assumed inflation rate and the capital discount rate can significantly impact the net present value calculations. For example, decreasing the assumed inflation rate from 3.25% to 2.5% would lower the net present value of the tank rehabilitation option to $6,327,000.

A life cycle cost analysis was also performed for an option involving replacement of the Bourne Hill and West Wareham tanks with a single new elevated composite tank. This option assumes that a new elevated composite tank with a total volume equal to approximately 950,000 gallons would be constructed at the site of one of the existing water storage tanks. Since the West Wareham Tank is located more centrally within the existing distribution system, it is assumed that the new water storage tank would be located at that site. Construction of a single new elevated composite water storage tank with a volume of 950,000 gallons would allow for the demolition of the existing Bourne Hill and West Wareham Tanks. The net present value of costs for this option, which is not shown in Table 4-3, is approximately $5,425,000, which includes demolition of both existing storage tanks. This net present value of cost for this option is very comparable, but slightly less than, the option involving replacement with two new storage tanks that is shown in Table 4-3.

An important factor to consider is that the life cycle cost analysis assumes that, for the tank rehabilitation (repainting) option, the existing tanks will not require replacement prior to the end of the 75-year life cycle period. Although this option assumes the existing tanks will be repainted every 15 years to minimize corrosion, tank repainting by itself does not usually prevent all forms of future deterioration. The Bourne Hill Tank and the West Wareham Tank will be 137 years old and 126 years old, respectively, at the end of the 75-year life cycle period. Assuming a useful life of 75 years, commonly considered by the water works industry as a reasonable estimation of the useful life of a new water storage tank, the existing tanks will be 62 and 51 years *beyond* their useful life at the end of the life cycle period. **In other words, there is a reasonable chance that at least one of the tanks may require replacement during the 75-year life cycle**. If the capital replacement cost for the Bourne Hill Tank was introduced in the latter half of the life cycle period for the tank rehabilitation option, it would further increase the net present value of that option.

Another key consideration is the fact that the tank replacement option offers important water quality benefits that are more difficult to quantify than the purely financial-based life cycle cost analysis that is summarized in Table 4-3. As described earlier, replacement of the existing steel tanks with new elevated composite tanks will significantly reduce the amount of excess storage that is deemed unessential from a level of service standpoint. The total amount of storage would be reduced from approximately 3.02 MG to 1.17 MG, with nearly all the new storage volume considered useable for equalization and fire storage. The smaller overall tank volumes will lead to enhanced water turnover and reduce average water age throughout the WFD distribution system (refer to Section 6 for a more detailed modeling assessment of water age related to the tank replacement option). Water quality issues often translate to financial expenditures, either in the form of capital or O&M improvements to address public health concerns and/or associated regulatory requirements, including improvements or processes to address new regulations that might become enacted during the 75-year life cycle period. Therefore, it is likely that the tank replacement option will result in water quality improvements and further financial savings over the course of the life cycle period, which are not reflected in Table 4-3.

# 5 MODEL UPDATES

The Wareham Fire District owns a distribution system model developed by Kleinfelder in 2007 and last updated in 2014. The model utilizes WaterGEMS CONNECT hydraulic modeling software as produced by Bentley Systems, Inc. The original model was developed to run both steady state and EPS simulations to observe pressure, flow, fire flow availability, and other hydraulic parameters in the water system.

Prior to updating and calibrating the hydraulic model, Kleinfelder performed fire flow tests on November 9, 2017 to determine flow characteristics of the existing pipe network. Kleinfelder performed three fire flow tests each at 15 different locations throughout the water distribution system. An emphasis was placed on locations of the system with known pressure or fire flow concerns. Additional flow test data obtained from the District was reviewed and used to address the needs of this update.

Kleinfelder first updated the elevations within the hydraulic model using a combination of the latest MassGIS LiDAR data. Kleinfelder then used individual customer-billing data provided by the WFD for 2016 to more accurately allocate the water usage throughout the model. Billing accounts were imported into WaterGEMS based on their addresses. The LoadBuilder tool within WaterGEMS then assigns each billing account to the closest pipe. Well pumping SCADA data combined with tank elevation SCADA data was used to update the 2014 diurnal use pattern to improve simulated changes in demand throughout a typical day. The pumping data and tank inflow/outflow SCADA data was used to calculate the actual net demand for each time interval, which was then used to refine the diurnal demand pattern used in the model. The diurnal demand pattern is used for extended period simulations to account for changes in domestic demand that occur throughout a typical 24-hour period.

Kleinfelder updated the model to incorporate the current (2016) and future (2040) average day and maximum day demands developed under Section 3 of this report. Refer to Section 3, Table 3-11 for the current average day, maximum day, and peak hour demands.

Based on the available flow test data, Kleinfelder calibrated the model through adjustments to pipe C-Factors and node elevations. The calibration effort focused on areas of the system where known or suspected pressure or fire flow deficiencies may exist. The updated model provides simulated operating pressure and available fire flows at all system nodes.

Kleinfelder updated the extended period simulation (EPS) operational controls to reflect current tank level ranges based on recorded tank level data. Pump curve definitions were updated for all wells based on information provided by the WFD at the outset of this study, including the 2016 Well and Pump Inspection Report (Weston & Sampson CMR, Inc.).

Finally, Kleinfelder updated the model to reflect the proposed operation conditions following the construction of the new water purification plant (WPP) at the Maple Springs Wells. A fixed HGL elevation of 208.00 feet MSL from the WPP design plans was modeled immediately downstream of the WPP. It was assumed that the pressure immediately downstream of the WPP is relatively constant under various WPP flow ranges, which is consistent with the proposed operation of the new WPP. Figure 5-1 below shows the Wareham distribution system.

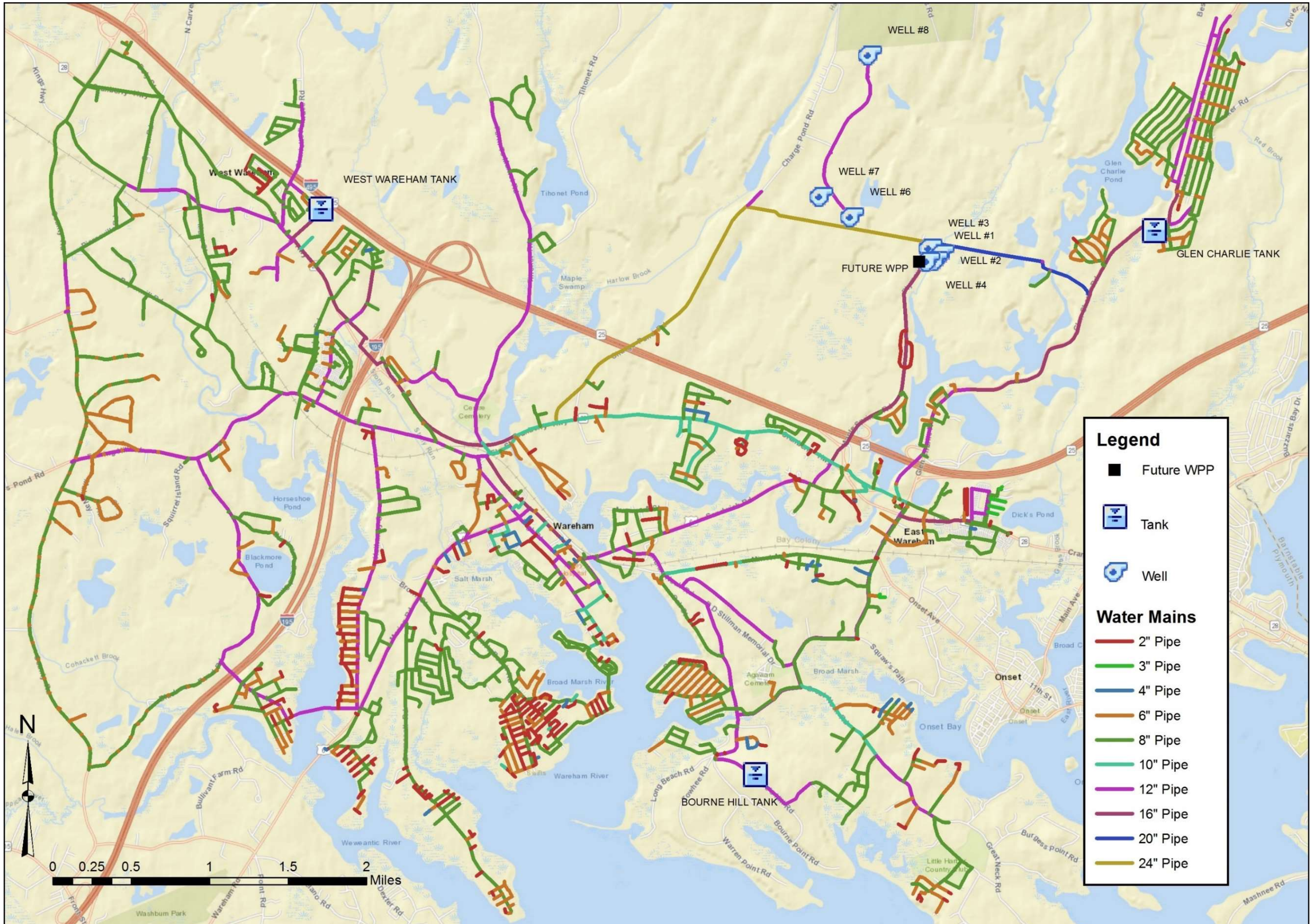


**Figure 5.1**
**Map of Wareham Distribution System**

# 6 LEVEL OF SERVICE ANALYSIS

## 6.1 OPERATING PRESSURES AND FIRE FLOWS

### 6.1.1 Normal Operating Conditions

Kleinfelder performed steady state simulations of the WFD water distribution system under existing conditions for 2016 and under projected conditions for 2040 average day and max day demands to assess pressures under normal operating conditions. The Massachusetts Department of Environmental Protection recommends that no node in the water distribution system drop below 35 pounds per square inch (psi) under normal operating conditions. The following table lists locations with pressures less than or approximately equal to 35 psi.

**TABLE 6-1**

**MODELED LOW PRESSURE LOCATIONS (NORMAL OPERATING CONDITIONS[1])**

| Location | Nodes with Lowest Pressure | Node Elevation | Existing Max Day Demand Pressure (psi) | Existing Avg Day Demand Pressure (psi) | Future (2040) Max Day Demand Pressure (psi) | Future (2040) Avg Day Demand Pressure (psi) |
|---|---|---|---|---|---|---|
| Judith Street | J-5817 | 105.63 | 32.1 | 32.6 | 32.0 | 32.5 |
| Intersection of Scott Lane and Judith Street | J-6110 | 98.47 | 35.2 | 35.7 | 35.1 | 35.6 |
| Intersection of Timber Lane and Windswept Road | J-5839 | 98.30 | 35.9 | 36.7 | 35.8 | 36.6 |

(1) Pumps 2, 3, 6, and 7, which provide a good representation of normal operating conditions, were turned on and tank levels were set at the low end of their typical operating range (based on recorded tank level SCADA data). The WPP was not included in these simulations.

The locations listed in the above table are the nodes in the hydraulic model with the highest elevations, other than the tank nodes. Therefore, any pipe rehab or replacement would make negligible difference in improving pressure at these locations. Additionally, all nodes were located in close proximity to the West Wareham Tank. Thus, head loss plays nearly no role in contributing to these low pressures. Booster pumps, either individual pumps for each residence or a District-owned booster pump station, could be considered for these areas, which are shown in the figure below.


**FIGURE 6 - 1**

**MODELED LOW PRESSURE AREAS**

### 6.1.2 Fire Flow Simulations

For the purpose of this analysis, the available fire flow at a particular location is the maximum flow rate than can be pulled from the node while still retaining a minimum 20 psi residual. Available fire flows are in addition to domestic flows. For fire flow simulations, all pumps were assumed to be off and tank levels were set at the lower end of their typical operating range. In this manner, the available fire flows assume the storage tanks are draining and above the level that would signal the well pumps to turn on. This is a realistic scenario, as the tanks are often draining without any pump operation for significant periods of time during a typical day, particularly during overnight hours. If several pumps are operating, such as during a fill cycle, it would have the effect of increasing available fire flows (due to the higher HGL created by the pumps).

#### 6.1.2.1 Residential Fire Flows

Available fire flow (AFF) results were compared to the needed fire flow for one and two-family dwellings according to the American Water Works Association (AWWA). The image below, taken from Table1-6 in the AWWA "Distribution System Requirements for Fire Protection, Fourth Edition M31", shows the residential needed fire flow for various building spacings.

Table 1-6 Needed fire flow for one- and two-family dwellings§

| Distance Between Buildings | | Needed Fire Flow | |
|---|---|---|---|
| *ft* | *(m)* | *gpm* | *(L/sec)* |
| More than 100 | (more than 30.5) | 500 | (31.5) |
| 31–100 | (9.5–30.5) | 750 | (47.3) |
| 11–30 | (3.4–9.2) | 1,000 | (63.1) |
| Less than 11 | (Less than 3.4) | 1,400 | (94.6) |

§Dwellings not to exceed two stories in height.

The approximate spacing of residential buildings was estimated using aerial images. Typical residential building spacings in the Town of Wareham were observed to be over 30 feet and therefore require available fire flows of at least 750 gpm at 20 psi residual at the flow hydrant. If individual houses exist that are spaced closer than 30 feet, higher available fire flows would be required and should be investigated on a case-by-case basis. The following table lists locations with AFFs below or approximately equal to 750 gpm based on the modeling simulations (assuming future maximum day demands).

**TABLE 6-2**

**RESIDENTIAL LOCATIONS WITH AVAILABLE FIRE FLOWS LESS THAN 750 GPM**

| Location | Nodes with Lowest AFF | Future (2040) Max Day AFF (gpm) |
|---|---|---|
| Heather Hill Road | J-6038 | 528 |
| Mayflower Ridge Drive | J-6070 | 614 |
| Cromesett Road from Birenback Way to Cromesett Road Cul-de-sac | J-5927 | 650 |
| | J-6208 | 836 |
| Bethel Way | J-6513 | 670 |
| Donna Road Cul-de-sac | J-6595 | 724 |
| Fillmore Street | J-6357 | 686 |
| Helen Street | J-4826 | 803 |
| John Street | J-5878 | 792 |
| Farmers Lane | J-6076 | 811 |
| Jupiter Circle | J-5806 | 840 |
| Emma Lane | J-5945 | 890 |
| Bodfish Avenue | J-5987 | 870 |
| White Pine Avenue | J-5776 | 900 |

Some houses were observed to have spacings greater than 100 feet, however those do not account for any of the locations listed in Table 6-2. Therefore, the locations listed in Table 6-2 may be considered deficient in terms of AFF based on the hydraulic modeling.

Simulations were performed to identify various improvements to increase AFFs to above 750 gpm in the following most deficient residential areas:

- Mayflower Ridge Drive and Heather Hill Road
- Cromesett Road
- Bethel Way

**<u>Improvement 1 (Mayflower Ridge Rd & Heather Hill Rd Area)</u>:** Modeling suggests that a significant length of 6-inch distribution piping is responsible for the low AFFs at Heather Hill Road and Mayflower Ridge Drive. According to simulations, replacing approximately 3,900 linear feet of 6-inch pipe with new 8-inch ductile iron pipe as shown in the tables and figure below would result in sufficient AFF in this area. The following tables and figure list pipes for replacement, the change in AFFs, and shows the extent of the replacement.

**TABLE 6-3**
**PIPE REPLACEMENT FOR IMPROVEMENT 1**

| Location | Pipe Number | Existing Size | Length | Proposed Size |
|---|---|---|---|---|
| Mayflower Ridge Drive | P-798 | 6” | 730 feet | 8” |
| | P-799 | 6” | 440 feet | 8” |
| | P-328 | 6” | 1,110 feet | 8” |
| | 32 | 6” | 1,190 feet | 8” |
| Heather Hill Road | P-666 | 6” | 150 feet | 8” |
| | P-667 | 6” | 290 feet | 8” |
| Total Length of 6” to 8” Pipe | | | 3,910 feet | |

**TABLE 6-4**
**NEW AFF’S AFTER IMPROVEMENT 1**

| Location | Node | Future Max Day AFF Under Existing Conditions (gpm) | Future Max Day AFF Under Proposed Conditions (gpm) |
|---|---|---|---|
| Mayflower Ridge Road | J-5755 | 894 | 3,014 |
| | J-6070 | 614 | 2,139 |
| Heather Hill Road | J-6038 | 528 | 1,034 |


**FIGURE 6-2**
**LOCATION OF IMPROVEMENT 1**

The opinion of probable construction cost for Improvement 1 is approximately $750,000.

According to the model simulations, the improvements summarized in Table 6-3 would provide available fire flow of approximately 1,000 gpm at Node 6038. However, substantial improvement in available fire flows at the eastern end of Heather Hill Road (Node 6070) and Node 6038 could still be realized by replacing only the existing 6-inch pipe in Mayflower Ridge Road from Route 28 to the road fork in the vicinity of 25 Mayflower Ridge Road, a distance of approximately 1,170 linear feet. According to the model, replacing this section of existing 6-inch pipe with new 8-inch pipe would provide available fire flows equal to approximately 750 gpm at Node 6038. This option would reduce the opinion of probable construction cost for Improvement 1 to approximately $225,000.

**Improvement 2 (Cromesett Rd Area):** Modeling suggests that a significant length of existing 8-inch distribution piping that dead-ends at the southern end of Cromesett Road contributes to relatively low AFFs in this area of the WFD system. According to simulations, replacing approximately 6,100 linear feet of existing 8-inch pipe with new 12-inch ductile iron pipe from the intersection of Route 6 to the intersection of Connehasset Road would result in increasing AFFs above 750 gpm. The following tables and figure show the extent of the pipe replacement that would result in available fire flows above 750 gpm at all locations on Cromesett Road.

**TABLE 6-5**

**PIPE REPLACEMENT FOR IMPROVEMENT 2**

| Location | Pipe Number | Existing Size | Length | Proposed Size |
|---|---|---|---|---|
| Cromesett Road | P-241 | 8” | 210 feet | 12” |
| | P-1197 | 8” | 430 feet | 12” |
| | P-1196 | 8” | 980 feet | 12” |
| | P-346 | 8” | 20 feet | 12” |
| | P-958 | 8” | 900 feet | 12” |
| | P-959 | 8” | 2,390 feet | 12” |
| | P-104 | 8” | 250 feet | 12” |
| | P-105 | 8” | 420 feet | 12” |
| | P-1008 | 8” | 180 feet | 12” |
| | P-1506 | 8” | 200 feet | 12” |
| | P-1632 | 8” | 150 feet | 12” |
| | P-1633 | 8” | 30 feet | 12” |
| | P-1686 | 8” | 40 feet | 12” |
| | P-1702 | 8” | 40 feet | 12” |
| | P-1703 | 8” | 190 feet | 12” |
| Total Length of 8” to 12” Pipe | | | 6,130 feet | |

**TABLE 6.6**

**NEW AFF'S AFTER IMPROVEMENT 2**

| Location | Node | Future Max Day AFF Under Existing Conditions (gpm) | Future Max Day AFF Under Proposed Conditions (gpm) |
|---|---|---|---|
| Cromesett Road | J-6208 | 836 | 2,594 |
| | J-5927 | 650 | 965 |


**FIGURE 6-3**

**LOCATION OF IMPROVEMENT 2**

The opinion of probable construction cost for Improvement 2 is approximately $1,400,000.

It is important to note that the assumed C-factors for the existing pipe in Cromesett Road has a major influence on modeled fire flows due to the length of this dead-end main. Moreover, there is some uncertainty regarding the age and condition of portions of the water main in Cromesett Road. Therefore, additional field tests are recommended to verify the C-factor ratings, approximate age and condition of the existing pipe in Cromesett Road prior to undertaking any major pipe replacement. The area represents an ideal candidate for C-factor testing given the potential for high velocity, unidirectional flow, with minimal branched connections. Conducting fire flow tests during a period when the well pumps are turned off is also recommended to provide an additional calibration point for this area of the system. Once the additional field data is collected, the simulation should be re-run to assess any changes in AFF and the resulting recommendation with respect to the extent of the replacement.

It is also important to note that several of the pipe segments in Table 6-5 correspond to pipes on the Priority List of Assets that were recommended for replacement in the 2017 Asset Management Plan Report.

**Improvement 3:** The hydraulic modeling indicates that the dead end 6-inch distribution piping is responsible for the low AFFs on Bethel Way. According to simulations, adding approximately 500 linear feet of 6-inch pipe from the existing terminus of the water main in Bethel Way to County Road to create a loop would result in sufficient AFF at this location. The following tables and figure list pipes for replacement, the change in AFFs, and shows the extent of the replacement.

**TABLE 6-7**
**PIPE REPLACEMENT FOR IMPROVEMENT 3**

| Location | Proposed Length | Proposed Size |
|---|---|---|
| Bethel Way | 500 feet | 6” |
| Total Length of 6” | | 500 feet |

**TABLE 6-8**

**NEW AFF'S AFTER IMPROVEMENT 3**

| Location | Node | Future (2040) Max Day AFF Under Existing Conditions (gpm) | Future (2040) Max Day AFF Under Proposed Conditions (gpm) |
|---|---|---|---|
| Bethel Way | J-6513 | 670 | 1,450 |

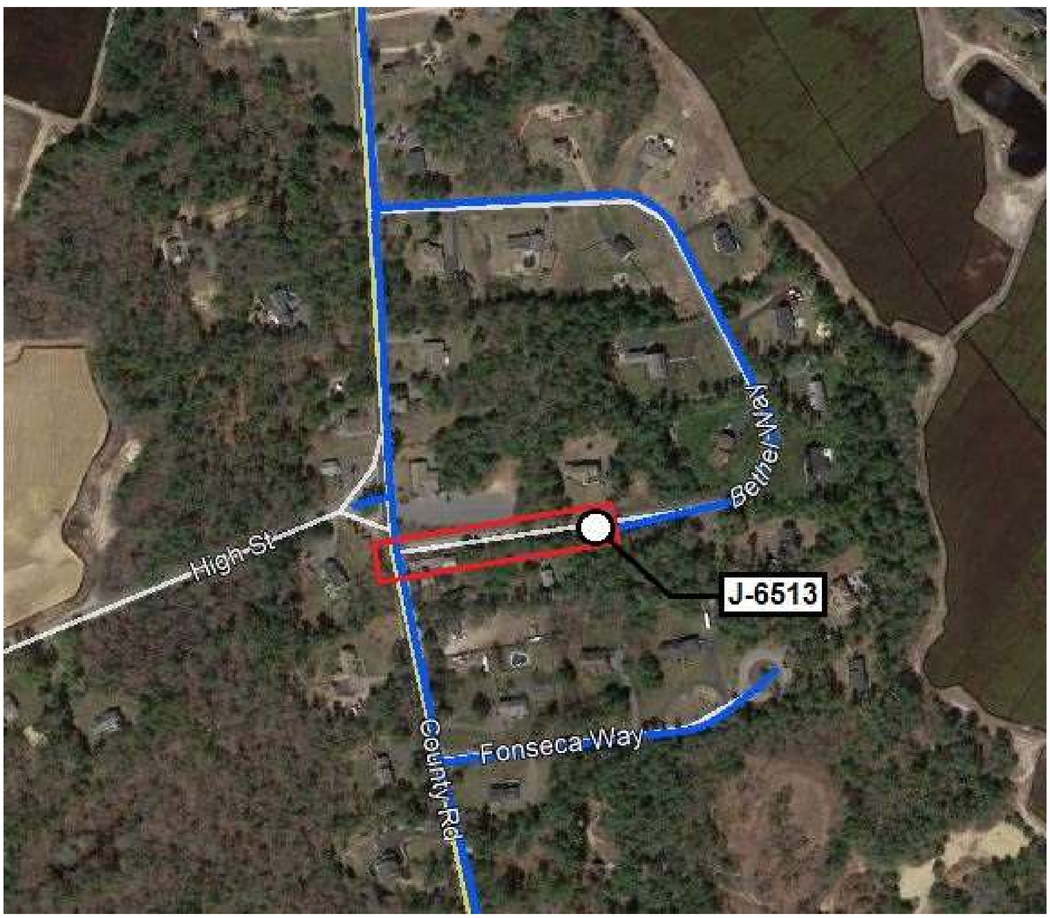


**FIGURE 6-4**

**LOCATION OF IMPROVEMENT 3**

The opinion of probable construction cost for Improvement 3 is approximately $85,000.

#### 6.1.2.2 Commercial Fire Flows

Needed fire flows for commercial and industrial areas vary according to several factors, including the presence and extent of on-site sprinkler systems, building materials, building use, building size, etc. During the 2007 Master Plan, an average required needed fire flow of 2,000 gpm was utilized in the fire flow analysis for commercial and industrial properties. It is important to note, however, that some specific locations have needed fire flows in excess of that amount. For example, as noted in the 2007 Master Plan, minimum required fire flows for the Cape Cod Shipbuilding at Narrows Road is 3,500 gpm according to ISO.

Recently, Kleinfelder developed average needed fire flows for the WFD distribution system as part of the Fiscal Year 2017 Asset Management Plan. For the Asset Management Plan, needed fire flows were calculated for each building in the WFD system according to the simplified National Fire Academy Method, which is one of the suggested methods in AWWA Manual M31. Kleinfelder utilized GIS data to determine the area of each building throughout the distribution system, which is the primary variable used to calculate needed fire flow according to the National Fire Academy Method. Figure 30 from the Asset Management Plan Report, included here as Figure 6-5 below, summarizes the needed fire flows throughout WFD's system utilizing this particular method.

As shown in Figure 6-5, utilizing the National Fire Academy Method results in needed fire flows equal to or greater than 6,000 gpm in several major commercial/industrial areas within the distribution system, which is significantly higher than needed fire flows according to ISO requirements. However, fire flows of 6,000 gpm and higher do not typically apply to the types of buildings located in many of the commercial areas shown on Figure 6-5. For example, Wareham Crossing, located within the commercial/industrial area along Route 28 (Cranberry Highway) west of I-195, includes newer commercial stores, which are constructed according to the latest building codes and that are equipped with modern sprinkler systems. The method used to develop the needed fire flows for the WFD's Asset Management Plan are useful as a means of comparison in a broader risk assessment, but caution should be exercised in applying those NFF values for evaluating the need and effectiveness of specific distribution system improvements.

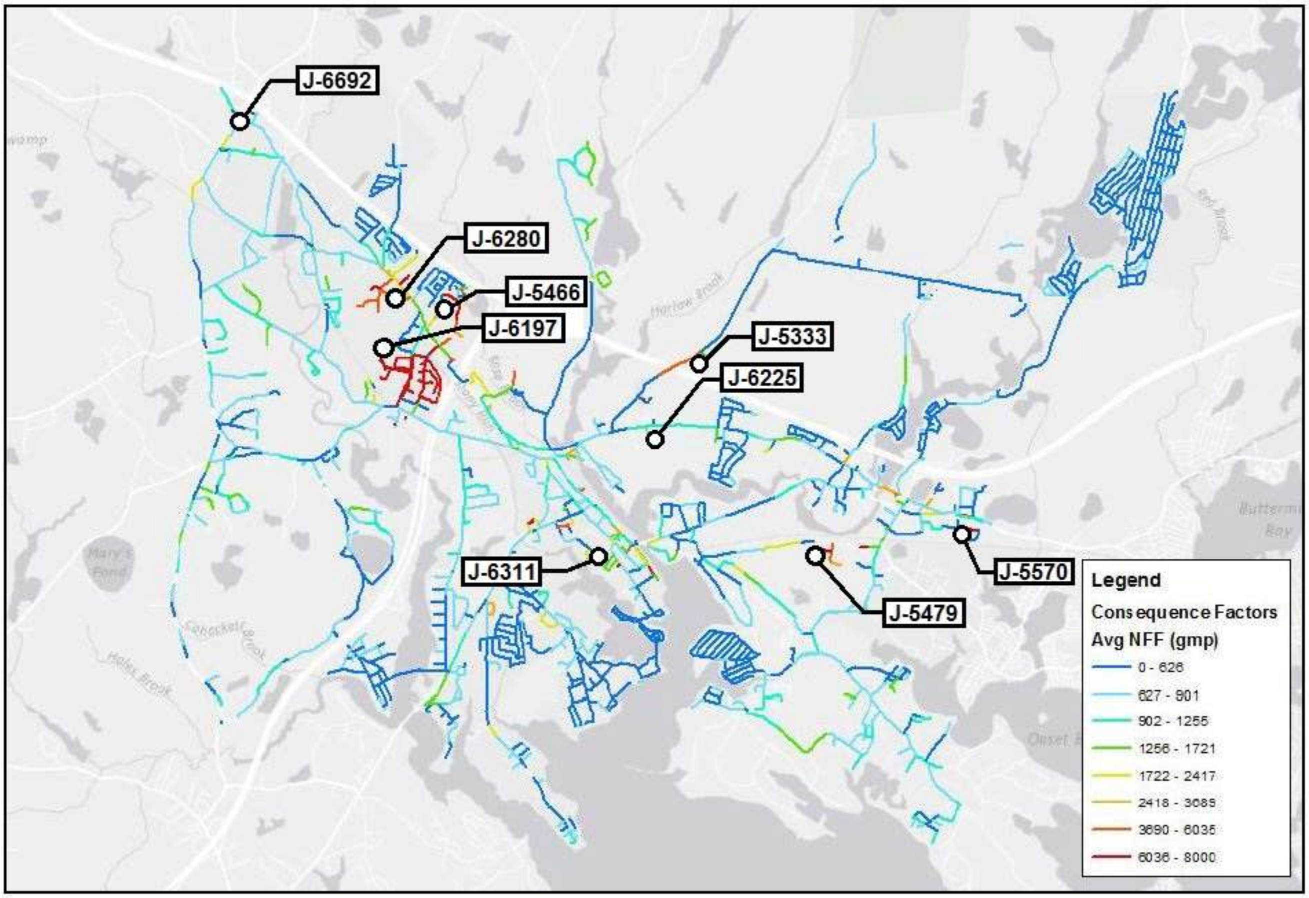


**Figure 30 – Average NFF (gpm)**

**FIGURE 6-5**

**FIGURE 30 FROM FISCAL YEAR 2017 ASSET MANAGEMENT PLAN, AVERAGE NFF (gpm)**

Therefore, for this analysis, an average needed fire flow of 2,000 gpm was assumed for the initial fire flow simulations, which is consistent with the approach utilized during the 2007 Master Plan. According to the hydraulic modeling, a total of eight general commercial locations had available fire flows less than 2,000 gpm, which are listed in Table 6-9. The table lists the node with the lowest AFF from each general area. These nodes were superimposed on to Figure 6-5 as well for reference. The NFF based on the method used in the Fiscal Year 2017 Asset Management Plan is also listed as a point of comparison.

Upon further examination, it was noted that Node 5382 at Tobey Hospital corresponds to a site hydrant on the hospital campus that is connected to a 4-inch main that is connected to a 6-inch main. There are other hydrants located off the 12-inch main in High Street providing ample fire protection to the hospital.

**TABLE 6-9**

**COMMERCIAL LOCATIONS WITH AVAILABLE FIRE FLOWS LESS THAN 2,000 GPM**

| Location | Node with Lowest AFF | Future Max Day AFF (gpm) | Deficit Below 2,000 gpm | FY 2017 Asset Management Plan NFF |
|---|---|---|---|---|
| Tobey Hospital | J-5382 | 788 | (1,212) | 3,890 to 6,035 |
| Express Drive | J-6197 | 996 | (1,004) | 6,036 to 8,000 |
| Intersection of County Road and Doty Street | J-6692 | 1,208 | (792) | 0 to 626 |
| Patterson Brook Road | J-6280 | 1,034 | (966) | 6,036 to 8,000 |
| Cranberry Highway | J-5570 | 1,164 | (836) | 6,036 to 8,000 |
| Cranberry Highway | J-6225 | 1,275 | (725) | 902 to 1,255 |
| State Street | J-5479 | 1,504 | (496) | 6,036 to 8,000 |
| Charge Pond Road | J-5333 | 1,614 | (386) | 3,890 to 6,035 |
| End of Tow Road | J-5466 | 2,039 | 39 | 6,036 to 8,000 |

The Express Drive location refers to a hydrant located at the rear northern end of the Cape Cod Express, Inc. site. According to the model, the hydrant is supplied by 8-inch mains in Tobey Road and Express Drive up to the entrance of the warehouse facility, and then by more than 600 feet of 6-inch main on private property terminating at the hydrant. The available fire flow at the hydrant located at the end of Express Drive (i.e. end of 8-inch main) is also well below the 2,000 gpm threshold established for the analysis. There are a number of interconnected water mains serving the Wareham Crossing shopping mall that is located immediately east of Tobey Road, including 12-inch and 8-inch water mains that circumvent the PETCO/Staples building. This existing network of mains could be used to augment flow to the head end of Express Drive.

For example, if a short section of new main were installed to connect the existing 8-inch main that runs along the west of the PETCO/Staples building to the existing 8-inch main in Tobey Road at Express Drive, it would provide another point of supply to Express Drive and increase available fire flow. Further investigation and discussions are required however to verify the ownership of the water mains within and around Wareham Crossing and to determine the feasibility of connecting these mains with the WFD's mains in Tobey Road. Potential impacts on the ability to meet pressure and flow requirements for the sprinkler systems within Wareham Crossing would need to be explored. If utilization of these mains is not permitted, then replacing sections of the existing 8-inch main in Tobey Road with new 12-inch main is another alternative that could increase available fire flows. Several of these water mains are included in the Priority List of Assets contained in the 2017 Asset Management Plan Report.

The intersection of County Road and Doty Street is located at the northwest corner of the WFD distribution system. This area is served by 8-inch dimeter water mains that are located approximately one mile away from the nearest connection to a 12-inch main, resulting in available fire flows well below 2,000 gpm. The higher elevations in this area contribute to lower static pressures and also to lower available fire flows. This area of the system continues to experience commercial growth, however. Major commercial customers in the area include Zero Waste Solutions, Flagship Cinemas, Concord Nurseries, and the District Court building.

Replacing approximately one mile of existing 8-inch main with new 12-inch main would aid in providing greater fire flow, but it would not increase pressures significantly, which average in the low 40s in terms of psi, and at times drop below 40 psi for short durations. Furthermore, increasing the size of the water mains supplying this area will increase average water age – the area already possesses some of the oldest water in the WFD system. Development of a separate pressure zone to serve the area approximately bounded by Route 28, County Road, and Doty Street is another alternative that would require construction of a new booster pumping station, but retain the existing piping and result in less water quality disadvantages. It is recommended that the WFD continue to monitor potential commercial growth in this area and simultaneously explore options for creating a separate pressure zone. The potential interconnection of the WFD system with the Carver water system could be included as part of that effort.

At Patterson Brook Road, the low available fire flow value is associated with a hydrant located at the end of a 6-inch water main stub with a low C-factor, which serves a large warehouse facility. However, there is an existing 12-inch water main in Patterson Brook Road with several hydrants nearby the facility and other industrial buildings in the area that is able to provide ample fire protection. Upon further examination, system improvements to enhance fire flow are not recommended for this area.

The relatively low fire flows at the two nodes on Cranberry Highway that are shown in Table 6-9 correspond to smaller private water mains, such as on the Stop and Shop grocery store property, which are connected off the existing 16-inch water main in Cranberry Highway. There are hydrants located off the 16-inch water main in Cranberry Highway that are able to supply at least 2,500 gpm of available fire flow to these locations. Therefore, there are no fire flow issues associated with this area.

The low fire flow at State Street corresponds to a location near the Minot Forest School off of Minot Avenue. 12-inch water mains are available at the intersection of Minot Avenue and Indian Neck Road to the west and Great Neck Road approximately ½ mile to the east. However, the water main in Minot Avenue is limited primarily to 8-inch diameter (there is some 10-inch water main in a section of Minot Avenue west of the school), which restricts the ability of the system to convey large flows. The area to the east is supplied by several 8-inch interconnected water mains, including those passing through the Brandy Hill residential development. The current connectivity of this general area should be verified to confirm the available fire flow at this location. If the fire flow is in fact limited to approximately 1,500 gpm at the school, and improvements to connectivity are not possible, then pipe replacement may be warranted. There are several options for pipe replacement, including the replacement of approximately ½ mile of the existing 8-inch pipe in Minot Avenue with new 12-inch water main.

In summary, we recommend further investigation and discussions with the WFD regarding the following commercial locations to confirm the need for additional pipe replacement to improve available fire flow:

- Express Drive
- County Road and Doty Road, including assessment of a separate pressure zone
- Minot Avenue (Minot Forest School)

## 6.2 FACILITY IMPROVEMENT SIMULATIONS

### 6.2.1 New Water Purification Plant (WPP) On-Line

Kleinfelder conducted simulations to assess the level of service in the system following construction and start-up of the new WPP. A fixed HGL elevation of 208.00 feet MSL from the WPP was modeled immediately downstream of the WPP, regardless of the number of well pumps on and the flow rate through the WPP. This is consistent with the assumed design conditions of the WPP. Once the new facility is on-line, the pressure immediately downstream of the WPP will be relatively constant under various WPP flow ranges. The model assumes that the Maple Springs wells (Well Nos. 1 – 4) and the Maple Park Well (Well No. 9) are the only wells being directed to the new WPP for treatment prior to entering the distribution system. All other active wells were assumed to discharge directly to the distribution system without passing through the new WPP, which is consistent with existing operation and proposed initial operation of the WPP.

The hydraulic modeling suggests that the new WPP will have little to no effect on operating pressures and fire flows within the District's distribution system. Model results showed that operating pressures at a sampling of locations under current and future maximum day demand conditions would vary 1 percent or less with the new WPP on line, compared to the WPP off-line. Similarly, there would be little to no effect on available fire flows. And since system water age is more dependent on system demands and the physical configuration of distribution storage facilities, water age would also not be significantly impacted by operation of the new WPP.

### 6.2.2 Storage Tank Improvements

Hydraulic modeling simulations were conducted for the following distribution storage tank improvement alternatives:

- Replacing the Bourne Hill Tank and West Wareham Tank with new elevated composite tanks, similar to the Glen Charlie Tank
- Demolishing the existing Bourne Hill Tank and adding a new elevated composite tank similar to the Glen Charlie Tank adjacent to the WPP

- Replacing the Bourne Hill and West Wareham tanks with new elevated composite tanks with overflow elevations raised approximately 10 feet to 209.00 feet MSL
- Placing the existing 20-inch water main located between the Maple Springs wellfield and Glen Charlie Road back into service
- Demolishing the existing Bourne Hill Tank and replacing the existing West Wareham Tank with a single new elevated composite tank similar to the Glen Charlie Tank

It is important to note that the rehabilitation (repainting) of the existing West Wareham and Bourne Hill Tanks are not expected to result in consequential changes to distribution system hydraulics, provided there are no changes to tank geometry, elevation, or operating range. The four improvement alternatives identified above were simulated and evaluated based on pressure, available fire flow, and water age. All simulations assume that the new WPP is in operation.

#### 6.2.2.1 Replacing the Bourne Hill Tank and West Wareham Tank With New Elevated Composite Tanks Similar to the Glen Charlie Tank:

A description of this alternative, including a life-cycle cost analysis, was presented in Section 4.2.2. of this report. The result of the life-cycle cost analysis was that this tank replacement option is more favorable than the tank rehabilitation option (i.e. tank repainting) from a financial perspective. It was further noted that this alternative provides other advantages, including water quality improvements associated with reduced average water age in the system.

A steady-state simulation performed for this alternative confirmed that, in terms of pressure and available fire flow, there is not a significant level of service change compared with rehabilitating the existing tanks. That is expected, given that pressure and available fire flows are dependent upon water surface elevation in the tanks and not tank geometry and overall size. Pressures and fire flows will not appreciably change if overflow and operating ranges stay relatively the same.

A series of extended period simulations were performed to evaluate the effects of this alternative on water quality. An initial simulation was performed to assess average water age in the system under existing conditions. Figure 6-6 below shows the results of the existing conditions 10-day water age simulation (dark red areas include water age up to 12 days old and black areas represent water age up to 18 days old). As shown in the figure, locations on the west side of town, specifically the northwest and southwest corners, generally have higher water ages. Water age is also higher near the Bourne Hill Storage Tank where there are a large number of dead end mains being partially fed with older water from the tank.

Figure 6-7 shows the results of the water age simulation with the new WPP on-line and replacement of the Bourne Hill Tank and West Wareham Tank with new elevated composite tanks. The figure illustrates significant improvement in water age in all areas of the system, including the areas near the tanks. The reduction in overall storage volume, from 3.02 MG with the existing tanks to 1.17 MG with the new elevated composite tanks (i.e. volume of all three tanks), has a major positive impact on system water age.

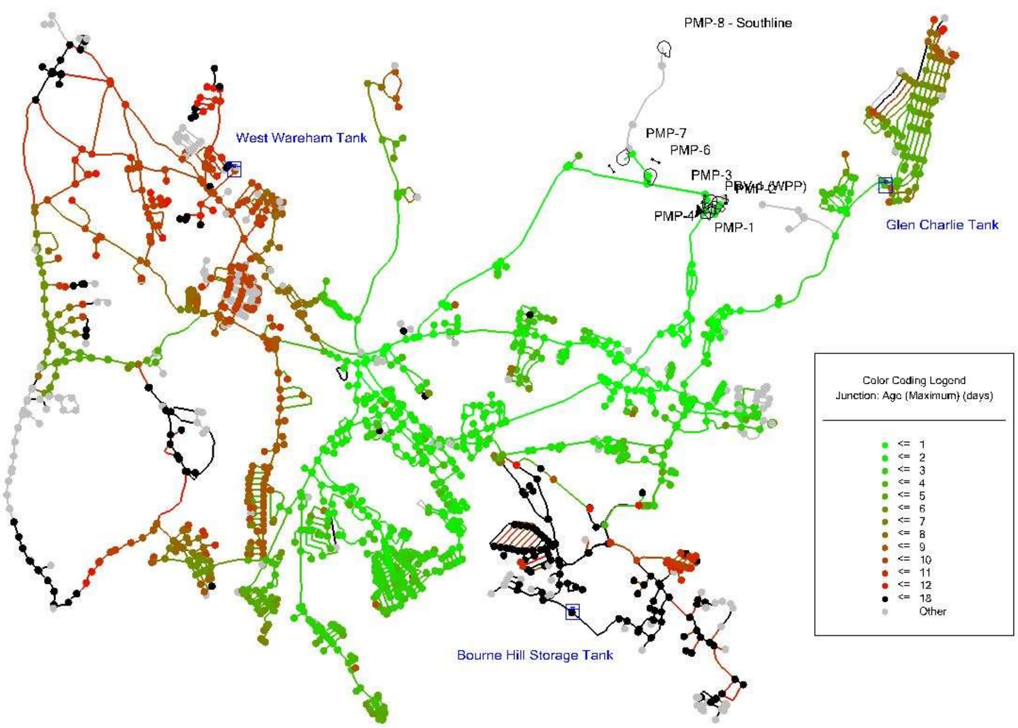


**FIGURE 6-6**

**EXISTING AVERAGE DAY DEMAND WATER AGE**

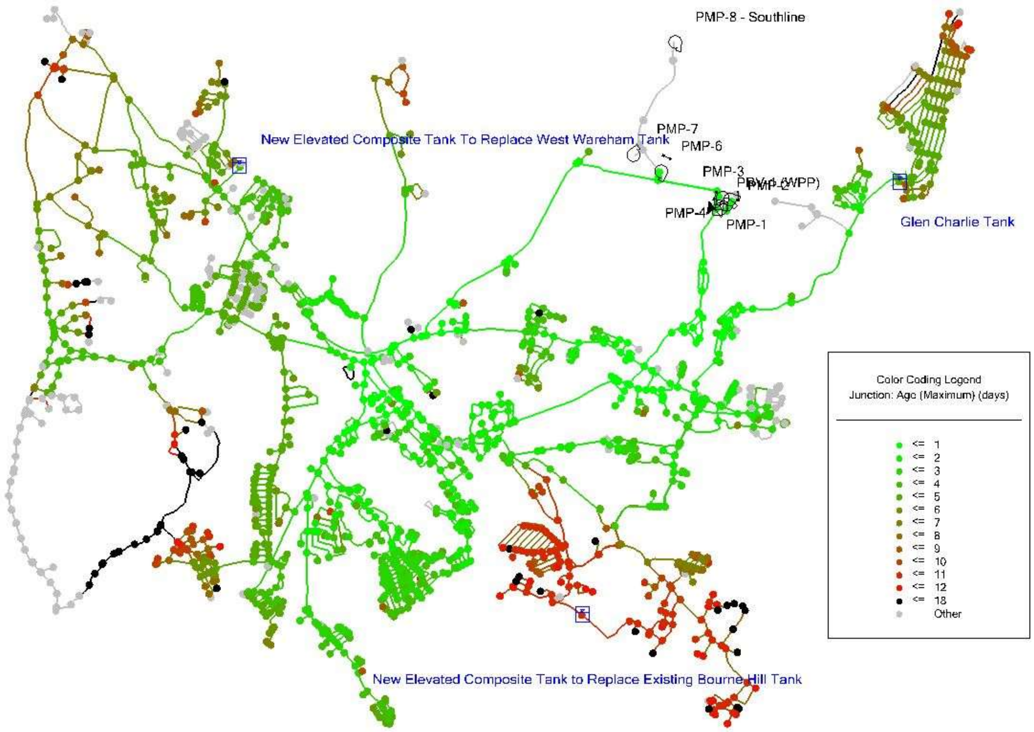


**FIGURE 6-7**

**EXISTING AVERAGE DAY DEMAND WATER AGE WITH REPLACEMENT OF BOURNE HILL TANK AND WEST WAREHAM TANK WITH NEW ELEVATED COMPOSITE TANKS**

6.2.2.2 Demolishing the Existing Bourne Hill Tank and Adding a New Elevated Composite Tank at the WPP:

This alternative assumes that the existing Bourne Hill Tank would be demolished and replaced by a new elevated bolted composite tank next to the new WPP that is under construction. A new tank at that location could provide the dual benefit of meeting distribution storage requirements as well as disinfectant contact time (CT) requirements for treated water produced by the new WPP. A new storage tank constructed immediately downstream of the new WPP could theoretically eliminate the need for the serpentine contactor that is currently included as part of the design of the new WPP.

Since all of the water produced by the new WPP would need to pass through the new tank to satisfy CT requirements, the new tank would need to be baffled or configured to introduce incoming water in one area of the tank while exiting at the opposing end of the tank.

Forcing all water produced by the new WPP through a new tank (to meet CT requirements) would introduce hydraulic and operational challenges. The new tank would always have to operate at a higher HGL than the existing West Wareham Tank in order to fill the West Wareham Tank. If the water surface level in the new tank were set as the control for the status of the finish water pumps at the new WPP, the pump shut-off level in the new tank would likely be reached before the existing West Wareham Tank had a chance to completely fill, assuming the new tank has a similar overflow elevation as the existing West Wareham Tank. Conversely, if the water surface elevation in the existing West Wareham Tank were set as the control for the status of the finish water pumps, it would likely be necessary to utilize one or more altitude and control valves to bypass the new tank at the WPP in order to completely fill the West Wareham Tank (to prevent regular overflowing of the new tank at the WPP). Since the water elevations in the two tanks will tend to converge once the finish water pumps at the new WPP are turned off, simply raising the overflow elevation of the new tank (i.e. above 200 feet MSL) to address this issue would still potentially result in regular overflowing at the West Wareham Tank.

Given the challenges cited above, and the fact that the District has already committed to constructing the serpentine contactor as part of the WPP construction, the probability of constructing a new tank at the WPP is unlikely. Regardless, simulations were performed with the updated model to assess level of service impacts associated with this alternative, particularly average water age using an extended period simulation.

The following table shows the change in operating pressure for the low-pressure nodes from Table 6-1 as well as several other nodes from representative locations throughout the distribution system.

**TABLE 6-10**

**PRESSURE COMPARISON FOR TAKING THE BOURNE HILL TANK OFFLINE AND ADDING A NEW ELEVATED COMPOSITE TANK AT THE WPP**

| Location | Node | Current 2040 Max Day Pressure (psi) | New 2040 Max Day Pressure (psi) |
|---|---|---|---|
| Intersection of Judith Street and Longbow Way | J-5817 | 32.0 | 31.1 |
| Intersection of Scott Lane and Longbow Way | J-6110 | 35.1 | 34.2 |
| Intersection of Timber Lane and Windswept Road | J-5839 | 35.8 | 34.1 |
| Blue Star Memorial Highway | J-6693 | 37.2 | 36.1 |
| Doty Street | J-5790 | 41.7 | 40.6 |
| Intersection of Main Street and Mill Street | J-4915 | 51.4 | 50.4 |
| Windy Hill Drive Cul-de-sac | J-4893 | 52.8 | 50.9 |
| Kendrick Road | J-5830 | 58.8 | 56.5 |
| Intersection of Blackmore Pond Road and Barlow Avenue | J-5809 | 64.2 | 61.6 |
| Intersection of Pilgrim Avenue and Broadmarsh Avenue | J-5729 | 75.3 | 72.5 |
| Johnson Street | J-6045 | 76.6 | 72.7 |
| Charge Pond Road | J-5197 | 62.1 | 53.4 |
| Intersection of Plymouth Avenue and Scheffler Drive | J-6516 | 38.5 | 38.4 |
| Mogan Way | J-5933 | 68.8 | 66.3 |
| Intersection of Little Harbor Road and Look Out Lane | J-5853 | 71.2 | 69.3 |

The modeling suggests operating pressures will decrease slightly for this alternative compared to existing conditions. The cause for the decrease is most attributable to the assumed reduction in HGL in the area of the new tank next to the new WPP. For the proposed condition simulation, the HGL in this area was set equal to the lower end of the operating range of the new tank (the higher HGL at the discharge side of the new WPP does not control since all WPP flow is routed through the new tank), whereas for existing conditions the HGL is determined by the operation of the existing well pumps, and is thus higher.

The following table shows the change in AFF for the four lowest AFF nodes, the four lowest AFF commercial nodes, as well as seven other nodes from throughout the distribution system.

**TABLE 6-11**

**AFF COMPARISON FOR TAKING THE BOURNE HILL TANK OFFLINE AND ADDING A NEW ELEVATED COMPOSITE TANK AT THE WPP**

| Location | Node | Current 2040 Max Day AFF (gpm) | New 2040 Max Day AFF (gpm) |
|---|---|---|---|
| Heather Hill Road | J-6038 | 528 | 497 |
| Cromesett Road from Birenback Way to Cromesett Road Cul-de-sac | J-5927 | 650 | 622 |
| Mayflower Ridge Drive | J-6070 | 614 | 579 |
| Bethel Way | J-6513 | 670 | 638 |
| Tobey Road | J-6197 | 996 | 950 |
| Intersection of County Road and Doty Street | J-6692 | 1,208 | 1,103 |
| Patterson Brook Road | J-6280 | 1,034 | 1,000 |
| Cranberry Highway | J-5570 | 1,164 | 1,049 |
| Doty Street | J-5790 | 1,123 | 1,048 |
| Intersection of Main Street and Mill Street | J-4915 | 2,886 | 2,576 |
| Windy Hill Drive Cul-de-sac | J-4893 | 1,071 | 1,012 |
| Kendrick Road | J-5830 | 2,351 | 2,222 |
| Intersection of Pilgrim Avenue and Broadmarsh Avenue | J-5729 | 1,830 | 1,718 |
| Charge Pond Road | J-5197 | >5,000 | < 5,000 |
| Intersection of Plymouth Avenue and Scheffler Drive | J-6516 | 4,338 | 4,308 |
| Intersection of Little Harbor Road and Look Out Lane | J-5853 | 1,042 | 860 |

Generally speaking, there are slight decreases in AFF due to taking the Bourne Hill Tank offline and adding a new elevated composite tank similar to the Glen Charlie Tank at the WPP.

Figure 6-8 shows the results of a water age simulation for this scenario. There is an overall improvement in water age near the location of the existing Bourne Hill Tank as a result of its removal. Water quality elsewhere in the system is also improved due to the reduction in overall storage volume – a new elevated tank at the WPP would have a smaller overall volume compared to the existing Bourne Hill Tank.

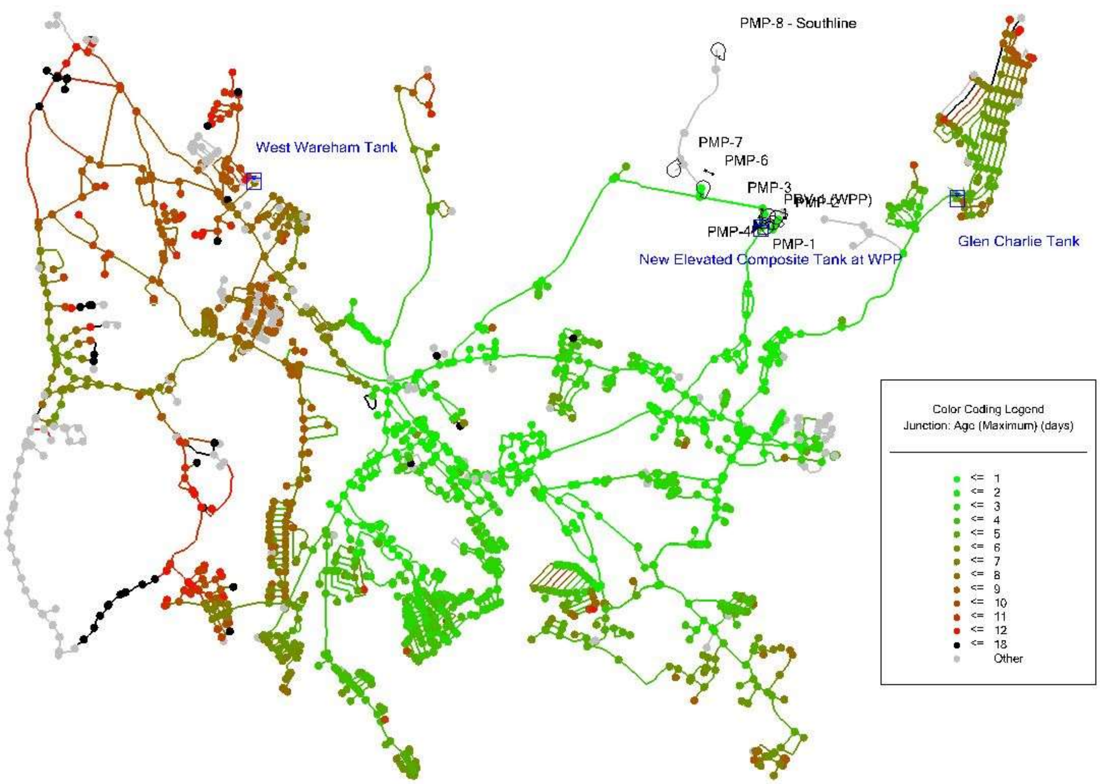


**FIGURE 6-8**

**2016 AVERAGE DAY DEMAND WATER AGE WITH TAKING THE BOURNE HILL TANK OFFLINE AND ADDING A NEW ELEVATED COMPOSITETANK SIMILAR TO THE GLEN CHARLIE TANK AT THE WPP**

Due to the hydraulic issues and operational challenges associated with constructing a new elevated tank adjacent to the new WPP to help achieve CT requirements, we do not recommend pursuing this option further.

#### 6.2.2.3 Replacing the Bourne Hill and West Wareham tanks to elevated composite tanks with tank overflow elevations raised to 209.00 feet MSL

This alternative is very similar to the alternative discussed in Section 6.2.2.1, except the overflow elevation of the new elevated composite tanks would be set approximately 10 feet higher. The purpose of this alternative is to assess the effect of higher HGLs on levels of service in the system, in the event the additional head is considered necessary either when new tanks are originally constructed or at some point in the future after the tanks are constructed. A benefit of the bolted glass-fused-to-steel style of elevated tanks is that the panels can be unbolted and new panels added to raise the height of the tank in the future, provided the concrete pedestal design accounts for this potential change during the up-front construction.

Table 6-12 shows the change in pressure for the low-pressure nodes from Table 6-1 as well as several other nodes from throughout the distribution system. As expected, pressures throughout the distribution system increase as a result of raising the overflow elevation.

Table 6-13 shows the change in AFF for the four lowest residential nodes, the four lowest commercial nodes, as well as seven other nodes from throughout the distribution system. Average available fire flows generally increase at all nodes, as expected given the higher HGLs within the system associated with this option.

## TABLE 6-12

## PRESSURE COMPARISON FOR CHANGING ALL TANKS TO ELEVATED COMPOSITE TANKS WITH OVERFLOW ELEVATIONS OF 209.00 FEET MSL

| Location | Node | Current 2040 Max Day Pressure (psi) | New 2040 Max Day Pressure (psi) |
|---|---|---|---|
| Intersection of Judith Street and Longbow Way | J-5817 | 32.0 | 34.5 |
| Intersection of Scott Lane and Longbow Way | J-6110 | 35.1 | 37.6 |
| Intersection of Timber Lane and Windswept Road | J-5839 | 35.8 | 38.1 |
| Blue Star Memorial Highway | J-6693 | 37.2 | 39.6 |
| Doty Street | J-5790 | 41.7 | 44.1 |
| Intersection of Main Street and Mill Street | J-4915 | 51.4 | 53.8 |
| Windy Hill Drive Cul-de-sac | J-4893 | 52.8 | 55.1 |
| Kendrick Road | J-5830 | 58.8 | 61.0 |
| Intersection of Blackmore Pond Road and Barlow Avenue | J-5809 | 64.2 | 66.3 |
| Intersection of Pilgrim Avenue and Broadmarsh Avenue | J-5729 | 75.3 | 77.3 |
| Johnson Street | J-6045 | 76.6 | 78.5 |
| Charge Pond Road | J-5197 | 62.1 | 62.9 |
| Intersection of Plymouth Avenue and Scheffler Drive | J-6516 | 38.5 | 42.8 |
| Mogan Way | J-5933 | 68.8 | 71.3 |
| Intersection of Little Harbor Road and Look Out Lane | J-5853 | 71.2 | 73.2 |

**TABLE 6-13**

**AFF COMPARISON FOR CHANGING ALL TANKS TO ELEVATED COMPOSITE TANKS WITH OVERFLOW ELEVATIONS OF 209.00 FEET MSL**

| Location | Node | Current 2040 Max Day AFF (gpm) | New 2040 Max Day AFF (gpm) |
|---|---|---|---|
| Heather Hill Road | J-6038 | 528 | 548 |
| Cromesett Road from Birenback Way to Cromesett Road Cul-de-sac | J-5927 | 650 | 669 |
| Mayflower Ridge Drive | J-6070 | 614 | 637 |
| Bethel Way | J-6513 | 670 | 701 |
| Tobey Road | J-6197 | 996 | 1,032 |
| Intersection of County Road and Doty Street | J-6692 | 1,208 | 1,318 |
| Patterson Brook Road | J-6280 | 1,034 | 1,079 |
| Cranberry Highway | J-5570 | 1,164 | 1,201 |
| Doty Street | J-5790 | 1,123 | 1,203 |
| Intersection of Main Street and Mill Street | J-4915 | 2,886 | 3,167 |
| Windy Hill Drive Cul-de-sac | J-4893 | 1,071 | 1,121 |
| Kendrick Road | J-5830 | 2,351 | 2,448 |
| Intersection of Pilgrim Avenue and Broadmarsh Avenue | J-5729 | 1,830 | 1,895 |
| Charge Pond Road | J-5197 | >5,000 | >5,000 |
| Intersection of Plymouth Avenue and Scheffler Drive | J-6516 | 4,338 | 4,367 |
| Intersection of Little Harbor Road and Look Out Lane | J-5853 | 1,042 | 1,074 |

Figure 6-9 shows the results of the water age simulation for the alternative involving new elevated composite tanks at the existing tank sites with overflow elevations raised to 209.00 feet MSL. The figure shows improvements in average water age compared to the existing conditions simulation that is shown in Figure 6-6. The level of improvement is similar to, but slightly less than, that exhibited in Figure 6-7 for the alternative involving replacement of the existing tanks with new elevated composite tanks (assuming existing overflow elevations equal to 200 feet MSL). This alternative assumed higher overflow elevations of 209 feet MSL, but assumed the same base of tank (i.e. bottom of bowl) elevations. Therefore, the overall volume of the elevated composite tanks for this alternative are slightly larger than those assumed for the tank replacement option based on existing overflow elevations, resulting in longer detention times within the tank and higher average water age.

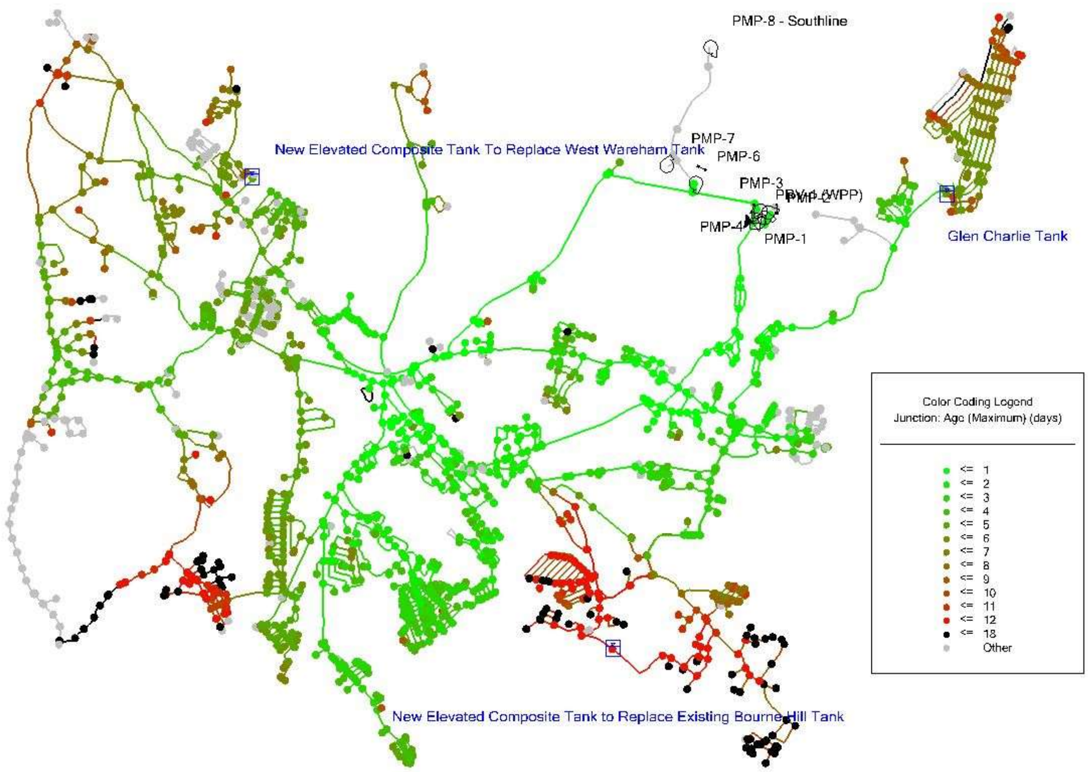


**FIGURE 6-9**

**2016 AVERAGE DAY DEMAND WATER AGE FROM CHANGING ALL TANKS TO ELEVATED COMPOSITE TANKS WITH OVERFLOW ELEVATIONS OF 209.00 FEET**

6.2.2.4 Placing the Existing 20-inch Water Main Located Between the Maple Springs Wellfield and Glen Charlie Road Back Into Service:

Hydraulic modeling suggests that by placing the existing 20-inch water main back into service, the Glen Charlie Tank would completely fill and activate closure of the altitude valve for approximately 3 – 4 hours while the WPP and wells are operating and filling the Bourne Hill and West Wareham tanks. The following figure shows the tank levels at all three tanks with the 20-inch main back in service, with the red line depicting water levels in the Glen Charlie Tank.

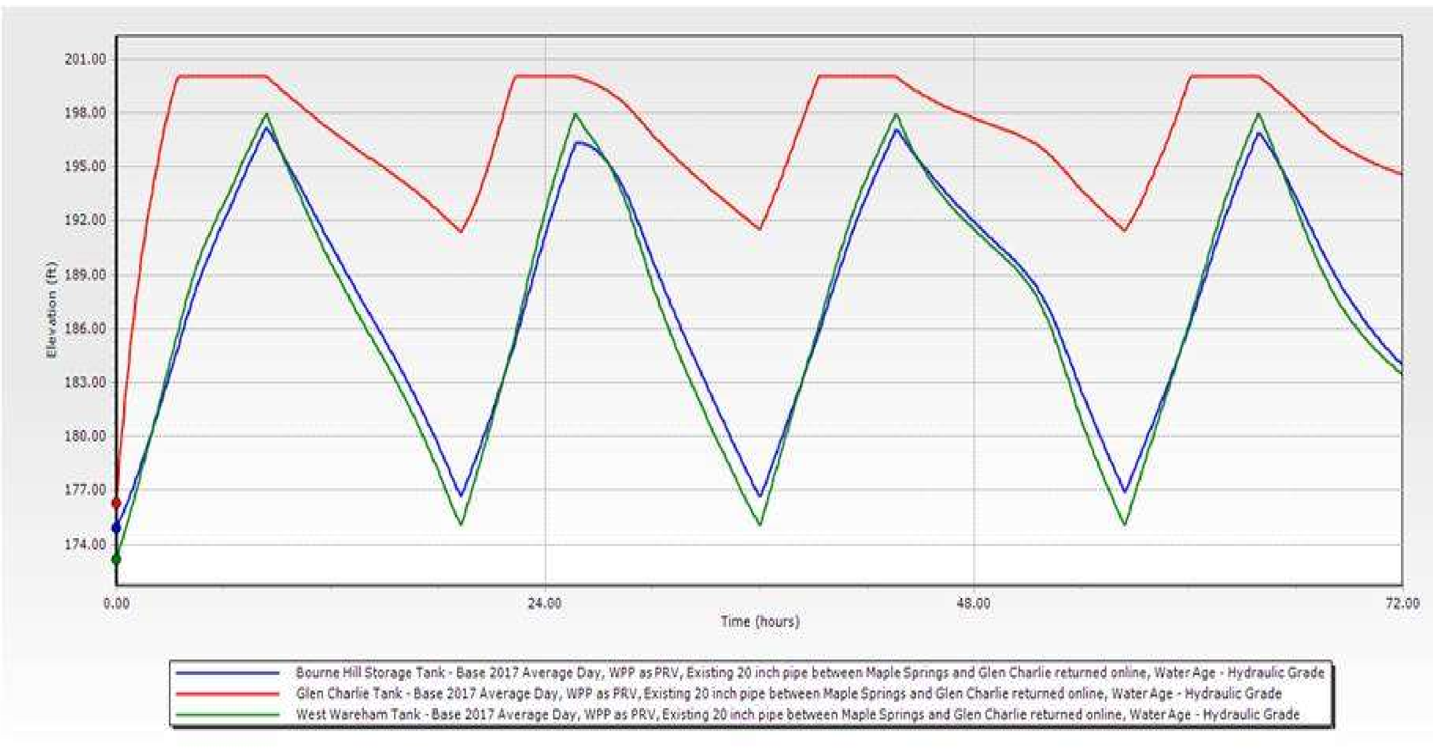


**FIGURE 6-10**

**TANK LEVELS WITH EXISTING 20-INCH MAIN RETURNED TO SERVICE**

Under existing conditions, with the 20-inch water main inactive, water levels in the Glen Charlie Tank follow a similar pattern as the two other tanks, typically ranging from approximately 187 feet to 196 feet MSL without activation of the altitude valve, which also represents a slightly larger operating range with more drawdown than with the 20-inch water main in service. Therefore, it is not recommended that the 20-inch water main be returned to service at this time.

#### 6.2.2.5 Demolishing the Existing Bourne Hill Tank and Replacing the Existing West Wareham Tank With a Single New Elevated Composite Tank:

This alternative includes constructing a single new elevated composite tank at the site of the existing West Wareham Tank, which would serve to replace both the existing Bourne Hill and West Wareham Tanks. Both existing tanks would be permanently removed from service. To provide the necessary amount of useable storage, the single new tank would have a volume of approximately 950,000 gallons.

A key benefit of this alternative is the economy of scale associated with constructing a single tank instead of two separate tanks. As described in Section 4.2.2, the net present worth of costs of constructing a single 950,000 gallon elevated composite tank is approximately $275,000 less than constructing two 475,000 gallon tanks. A disadvantage of operating a fewer number of storage tanks in the distribution system is the potential reduction in pressure equalization effectiveness. Operating more tanks at a similar HGL dispersed throughout the distribution system tends to stabilize pressures and minimize pressure fluctuations during tank drain/fill cycles.

Table 6-14 shows the change in pressure for the low-pressure nodes from Table 6-1 as well as several other nodes from throughout the distribution system. The modeling indicates that pressures throughout the distribution system would generally decrease, albeit slightly, due to replacing the two existing tanks with a single new storage tank operating at similar elevations.

Table 6-15 shows the change in AFF for the four lowest residential nodes, the four lowest commercial nodes, as well as seven other nodes from throughout the distribution system. Average available fire flows decrease slightly.

**TABLE 6-14**

**PRESSURE COMPARISON FOR TAKING THE BOURNE HILL TANK OFFLINE AND REPLACING THE WEST WAREHAM TANK WITH A NEW ELEVATED GLASS WELDED COMPOSITE TANK**

| Location | Node | Current 2040 Max Day Pressure (psi) | New 2040 Max Day Pressure (psi) |
|---|---|---|---|
| Intersection of Judith Street and Longbow Way | J-5817 | 32.0 | 30.6 |
| Intersection of Scott Lane and Longbow Way | J-6110 | 35.1 | 33.7 |
| Intersection of Timber Lane and Windswept Road | J-5839 | 35.8 | 34.6 |
| Blue Star Memorial Highway | J-6693 | 37.2 | 35.9 |
| Doty Street | J-5790 | 41.7 | 40.3 |
| Intersection of Main Street and Mill Street | J-4915 | 51.4 | 50.1 |
| Windy Hill Drive Cul-de-sac | J-4893 | 52.8 | 51.8 |
| Kendrick Road | J-5830 | 58.8 | 57.9 |
| Intersection of Blackmore Pond Road and Barlow Avenue | J-5809 | 64.2 | 63.5 |
| Intersection of Pilgrim Avenue and Broadmarsh Avenue | J-5729 | 75.3 | 74.9 |
| Johnson Street | J-6045 | 76.6 | 76.6 |
| Charge Pond Road | J-5197 | 62.1 | 61.5 |
| Intersection of Plymouth Avenue and Scheffler Drive | J-6516 | 38.5 | 38.5 |
| Mogan Way | J-5933 | 68.8 | 69.4 |
| Intersection of Little Harbor Road and Look Out Lane | J-5853 | 71.2 | 72.4 |

**TABLE 6-15**

**AFF COMPARISON FOR TAKING THE BOURNE HILL TANK OFFLINE AND REPLACING THE WEST WAREHAM TANK WITH A NEW ELEVATED GLASS WELDED COMPOSITE TANK**

| Location | Node | Current 2040 Max Day AFF (gpm) | New 2040 Max Day AFF (gpm) |
|---|---|---|---|
| Heather Hill Road | J-6038 | 528 | 516 |
| Cromesett Road from Birenback Way to Cromesett Road Cul-de-sac | J-5927 | 650 | 637 |
| Mayflower Ridge Drive | J-6070 | 614 | 600 |
| Bethel Way | J-6513 | 670 | 650 |
| Tobey Road | J-6197 | 996 | 971 |
| Intersection of County Road and Doty Street | J-6692 | 1,208 | 1,135 |
| Patterson Brook Road | J-6280 | 1,034 | 1,006 |
| Cranberry Highway | J-5570 | 1,164 | 1,086 |
| Doty Street | J-5790 | 1,123 | 1,070 |
| Intersection of Main Street and Mill Street | J-4915 | 2,886 | 2,650 |
| Windy Hill Drive Cul-de-sac | J-4893 | 1,071 | 1,037 |
| Kendrick Road | J-5830 | 2,351 | 2,267 |
| Intersection of Pilgrim Avenue and Broadmarsh Avenue | J-5729 | 1,830 | 1,757 |
| Charge Pond Road | J-5197 | > 5,000 | 4,952 |
| Intersection of Plymouth Avenue and Scheffler Drive | J-6516 | 4,338 | 3,882 |
| Intersection of Little Harbor Road and Look Out Lane | J-5853 | 1,042 | 903 |

Figure 6-11 shows the results of a water age simulation for this scenario. There is an overall improvement in water age near the location of the existing Bourne Hill Tank as a result of its removal. Water quality elsewhere in the system is also improved due to the reduction in overall storage volume. This scenario is most optimal in terms of reducing water age in the distribution system.

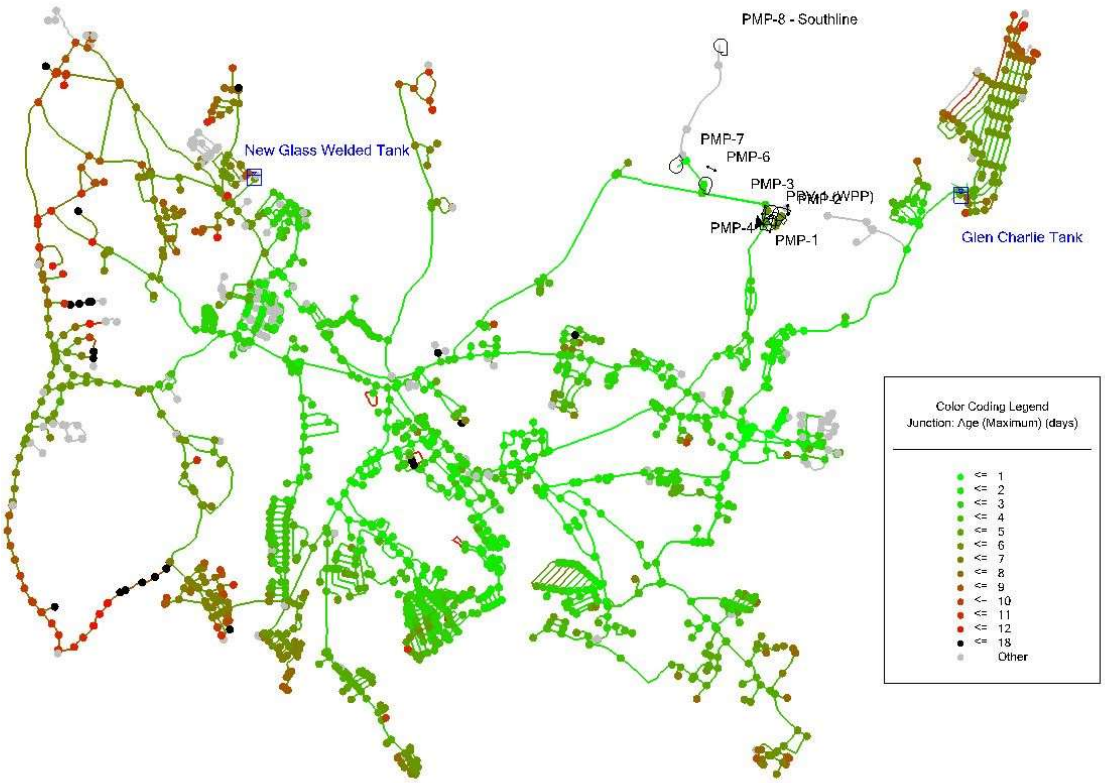


**FIGURE 6-11**

**2016 AVERAGE DAY DEMAND WATER AGE FROM REPLACING THE EXISTING BOURNE HILL AND WEST WAREHAM TANKS WITH A SINGLE NEW ELEVATED COMPOSITE TANK**

# 7 SUMMARY OF FINDINGS AND CONCLUSIONS

*Water Demand Analysis*

The results of the water demand analysis indicate that the District will experience modest growth in residential and commercial demand through the end of the planning period (Year 2040). Average day demand is estimated to increase approximately 22%, from 1.56 MGD under existing conditions to 1.91 MGD in 2040. The maximum day demand is estimated to increase approximately 16%, from 3.41 MGD under existing conditions to 3.95 MGD in 2040. And the peak hour demand is estimated to increase from 5.46 MGD to 6.32 MGD.

The existing maximum available supply capacity for the WFD may be considered as high as 5.6 MGD (assuming both Well Nos. 8 and 9 are out of service) to 7.0 MGD (assuming only Well No. 8 is out of service). Typically, the maximum available supply capacity of a water system should at least equal the maximum day demand. The maximum available supply capacity for the District far exceeds the maximum day demand under both existing and future conditions, and is greater than the existing peak hour demand as well.

*Storage Capacity Evaluation*

The storage capacity evaluation shows that the total available useable storage in the existing WFD distribution system is equal to 1.37 MG, which is equal to the volume of storage above elevation 163.5 feet (MSL) in the Bourne Hill, West Wareham, and Glen Charlie Tanks. Water stored in the three tanks below that elevation is not considered useable because if the water surface elevation drops below that level, there will be insufficient operating pressures in the system. The total amount of required useable storage for equalization and fire-fighting purposes is equal to 0.9 MG. The difference between the total available useable storage (1.37 MG) and the total required useable storage (0.9 MG) is equal to 0.47 MG and is considered surplus useable storage.

The existing Glen Charlie Tank provides 0.2 MG of available useable storage. Therefore, if the existing Bourne Hill and West Wareham tanks are eventually replaced, the new tanks will need to provide a minimum of 0.7 MG of useable storage, regardless of the type of tank constructed. To provide a contingency for system growth beyond the end of the planning period (i.e. beyond

2040) and other unknown factors, Kleinfelder recommends that the WFD build-in approximately 0.25 MG of additional useable storage into any new Bourne Hill and West Wareham tanks. Therefore, any new tanks should have a total useable storage volume of at least 0.95 MG total between the two tanks.

*Level of Service Analysis*

The hydraulic computer model was updated to incorporate the new demand projections, an improved method for allocating demand across the system, recent system improvements, updated pump curves, operational changes since the model was last updated in 2014, and the new water purification plant (WPP). The updated model was then used to conduct a level of service evaluation to assess system operating pressures and fire flows under various scenarios. Based on the level of service evaluation, the following system improvements are recommended to enhance system hydraulics, particularly with respect to improving available fire flows to better meet industry standards:

- 1,200 linear feet of new 8-inch water main on Mayflower Ridge Drive (opinion of probable construction cost = $225,000)
- 6,100 linear feet of new 12-inch water main in Cromesett Road, pending additional field testing to further verify C-factors of existing water mains (opinion of probable construction cost = $1,400,000)
- 500 linear feet of new 6-inch water main in Bethel Way (opinion of probable construction cost = $85,000)

There are also several other commercial locations with inadequate available fire flow that may require capital improvements, pending further verification of existing system connectivity and the prospects for future commercial development in those areas. These areas include Express Drive, County Road and Doty Road, and Minot Avenue (Minot Forest School).

We recommend that the WFD undertake a more detailed evaluation of the County Road and Doty Road area to address inadequate fire flows in that commercial area, including the potential development of a separate pressure zone.

The hydraulic modeling suggested that the new WPP will have little to no effect on operating pressures and fire flows within the District's distribution system. Model simulation results showed that operating pressures at a group of representative locations under current and future maximum day demand conditions would vary 1 percent or less with the new WPP on-line, compared to the WPP off-line. Similarly, the impact on fire flows would be negligible.

*Storage Tank Improvement Scenarios*

Several tank improvement alternatives were evaluated, including the following:

- Maintaining and regularly rehabilitating (i.e. repainting) the existing welded steel Bourne Hill and West Wareham tanks
- Replacing the existing welded steel Bourne Hill and West Wareham tanks with new elevated bolted composite tanks (i.e. AquaStore style tanks) at existing overflow elevation 200 feet MSL
- Demolishing the existing welded steel Bourne Hill Tank and adding a new elevated bolted composite tank at the new WPP
- Replacing the existing welded steel Bourne Hill and West Wareham tanks with new elevated bolted composite tanks with overflow elevations raised to 209 feet MSL (to simulate increasing the height of elevated tanks in the future)
- Demolishing the Existing Bourne Hill Tank and Replacing the Existing West Wareham Tank with a Single New Elevated Composite Tank

A life cycle cost analysis was conducted to compare the net present worth of costs of maintaining and regularly rehabilitating (painting) the Bourne Hill and West Wareham tanks for the next 75 years versus replacing both tanks with new elevated bolted composite tanks (i.e. AquaStore). The life cycle cost analysis concluded that the net present worth of costs for the tank rehabilitation option exceeds the net present worth of costs for the tank replacement option, although the tank rehabilitation option requires a lower initial capital investment at the beginning of the life cycle period. The analysis also indicates that the net present worth of costs associated with constructing a single new elevated bolted composite tank (with a volume of 950,000 gallons) is less than the net present worth of costs associated with constructing two new elevated bolted composite tanks (475,000 gallons each).

The tank rehabilitation option assumes that neither of the existing tanks will require replacement over the next 75 years, which may not be realistic given that the tanks will be 137 and 126 years old, respectively, at the end of the life cycle period. If either of the existing storage tanks requires replacement during the later portion of the life cycle period, then the tank replacement option (i.e. construct new elevated bolted composite tanks at the beginning of the life cycle period) becomes even more advantageous from a long-term financial perspective.

Furthermore, replacing the existing steel storage tanks with new elevated bolted composite tanks will significantly reduce overall storage volume and eliminate much of the excess water storage that is deemed unessential from a level of service standpoint. The total storage volume of all three tanks combined will decrease from 3.02 MG currently to 1.17 MG, with nearly the full amount of the new storage volume considered useable for equalization and fire storage.

Extended period simulations verified that the smaller overall tank volumes will lead to enhanced water turnover and significantly reduce average water age throughout the WFD distribution system, which will improve disinfectant residual concentrations and water quality. When accounting for these other factors, the tank replacement option emerges as the preferred alternative. Kleinfelder's opinion of initial capital costs for the construction of new elevated bolted (glass-fused-to-steel) composite tanks is approximately $5,597,000, which includes demolition of the existing tanks, new tanks, site work, and a 25% contingency. The cost conservatively assumes that no economy-of-scale will be realized if the two tanks are constructed together as part of a single construction contract.

Due to the hydraulic issues and operational challenges associated with constructing a new elevated tank adjacent to the new WPP to help achieve CT requirements, we do not recommend pursuing this option further.