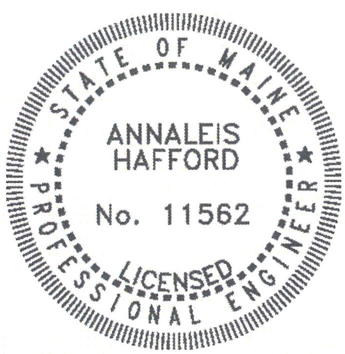
COMPREHENSIVE
MASTER PLAN UPDATE

TOWN OF BAR HARBOR, MAINE

OCTOBER, 2019
Revised MAY, 2020



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EXECUTIVE SUMMARY

The Town of Bar Harbor's Water Department provides water service to about 1,850 connected customers. The Town is in the process of embarking upon the next long-term program to improve water quality and infrastructure throughout the distribution system. The Town has successfully maintained its Filtration Avoidance Waiver by protecting the Eagle Lake watershed, upgrading its water treatment facility, and operating its infrastructure in compliance with the Drinking Water Standards. In 2018, the Town of Bar Harbor authorized Olver Associates Inc. to prepare an update to its Comprehensive Water Plan (Plan) which had last been completed in 2005. The Drinking Water Program suggests that these Plans are updated approximately every ten years. The previous plan and work completed since that time largely focused on water treatment compliance with the upgrade of the Duck Brook facility in 2013. The original facility was upgraded to comply with the Long Term 2 Enhanced Surface Water Treatment Rules and the Stage 2 Disinfection By-Products Rule. The Town commissioned a study in 2008 which determined that the upgrade including the addition of UV, chloramination, and updated chemicals for corrosion control would be the best method for achieving compliance with both of the Stage 2 Rules.

The Town's upgraded water treatment system has been evaluated and found to be producing acceptable water that meets both the primary and secondary EPA drinking water standards. Even though the Town's water system has been managed very well and is in compliance with all requirements, there is additional work suggested to be completed due to the lack of system storage especially in the Up-Island areas.

Distribution improvements need to be addressed to improve water quality and low pressure and to replace piping that is failing due to its age. The distribution system infrastructure is old with extensive amounts of undersized cast iron piping remaining in its system. The system also contains undersized galvanized steel dead-end piping in many of its downtown areas. Finally, there are multiple fire hydrants and system valves that need maintenance and/or replacement. These improvement areas are summarized in this report.

In order to evaluate decisions for updated storage and various other system responses to improvements, we completed a water model of the Town's water system. The water model was developed over the last six months with assistance from the Town for the data inputs. The model was calibrated and different system improvements were evaluated using this as a basis to better understand the impacts of storage, pipe size changes, system connectivity, and locations of booster pumps.

The model will need to be kept up-to-date and refined as further flow testing is conducted.

The Town's water system's layout causes issues with extreme variance in topography, resulting in some areas with lower than desired pressures and other areas with higher than desired pressures. Some newer development has been done in high elevation/low pressure areas with little planning for the impact to water system pressure.

All of these improvements will take time and may eventually require an increase in the Town's water rates. The Town of Bar Harbor has made significant progress in upgrading its water treatment facilities and the system is being operated at a high level of performance and efficiency. The improvements defined in this comprehensive water report will allow the Town to obtain a stable water system into the future and maintain customer satisfaction.

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1.0 INTRODUCTION

1.1 PURPOSE OF THE WATER SYSTEM COMPREHENSIVE MASTER PLAN UPDATE

The water system owned and operated by the Town of Bar Harbor contains an extensive amount of infrastructure, much of which is not well known by the customers which receive water from the system. The Water Department employs qualified personnel to manage, maintain, and operate the system which requires an ongoing capital improvement program to replace old components as necessary in order to provide reliable service and high-quality water. The Town of Bar Harbor complies with the requirements established by Federal and State of Maine regulations.

The Town's Water System Comprehensive Master Plan (Master Plan) serves as an important role in the management of the system by providing an updated evaluation of the Town's programs and system needs. It also summarizes the history and existing operations of the water system and provides a long-term plan for the next ten to twenty years with a basis for annual budget decisions and capital planning.

The objectives of the updated Master Plan are intended to:

- Summarize the history of the system and its current operations.
- Evaluate the existing water system and provide suggestions as to upgrades for the next ten to twenty years.
- Analyze the operations of the system and make suggestions for improvements to day to day operations.
- Prepare a schedule of improvements that meets the goals of the Town's financial program.
- Evaluate past water quality and identify water quality improvements, if necessary.
- Provide a summary of current regulatory requirements and emphasize importance of keeping the water systems' filtration waiver.
- Provide a summary of future regulatory requirements and what these requirements may have for an impact on the system.
- Evaluate suggested improvements and provide a cost estimation of what the Town would need to set aside for the recommended improvements.

- Address the long-term issue of water storage and make recommendations to improve the level of storage and location of tanks or reservoirs.

- Provides the Town with a working water model to assist with making future design decisions.

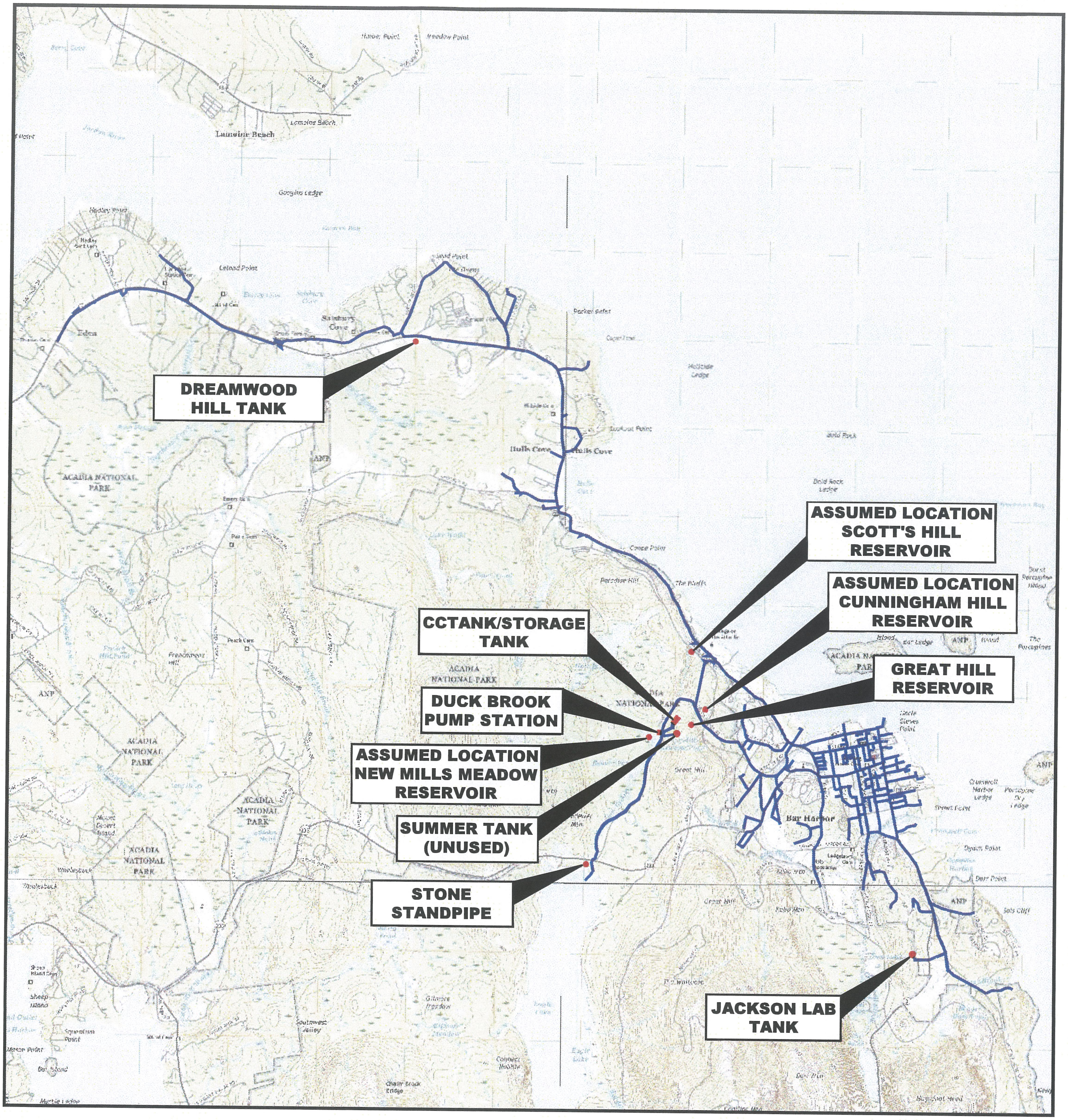
In 2018, the Town of Bar Harbor authorized Olver Associates Inc. to prepare a Comprehensive Water Plan as recommended by the State of Maine Drinking Water Program (DWP). The Maine DWP suggests updating the Master Plan approximately every ten years. The Town's last plan was completed back in 2005 and many of the suggested projects within that plan have since been completed. Following the 2005 Master Plan, an additional report was completed which evaluated long term treatment options for the Town's Surface Water in order to comply with the Surface Water Treatment Rules and the rules controlling the levels of chlorinated organics that can be formed during disinfection. This second report was the basis for the Town's major upgrade at the Duck Brook Pump Station, completed in 2013.

Included in this Master Plan is also a long-term planning strategy for the Town's water service area. The Master Plan evaluates the existing system and its ability to meet the anticipated requirements for water source, quality, transmission, storage, and distribution over a twenty-year planning period. Water system improvement projects have been developed to meet the potential changing demands of regulatory impacts and population growth as well as infrastructure repair and replacement. The Master Plan also identifies planning level costs (2020 dollars) of suggested improvement projects and provides a financial plan for funding the suggested projects.

1.2 HISTORY OF THE BAR HARBOR WATER SYSTEM

The Bar Harbor Water Company was officially established as a corporation in February of 1874. Figure 1 shows the existing water system with all historic tank locations from 1874 to present. For additional information on these historic tanks, see Table 14 on Page 48. The original system consisted of open wooden flumes that in its early years obtained water from New Mills Meadow and Duck Brook. These flumes extended to a 150,000-gallon reservoir on what was called "Scott's Hill." The original layout was a gravity feed system and the reservoir served as the intake location for only about 2 miles of 2-inch diameter to 4-inch diameter distribution lines. In 1880, the company began to install fire hydrants on these small water lines in downtown Bar Harbor. This early system was limited to a service area of between 50 to 75 customers.

In 1887, the Bar Harbor Water Company built a new stone standpipe just below Eagle Lake and laid the 16-inch diameter piping between Eagle Lake and New Mill's Meadow at the location that is now the Duck Brook Pump Station. At this point in the system's history, the pipe was connected to the old 12-inch Duck Brook line. In about 1888, the Water Company installed around 1,550 linear feet (LF) of 20-inch cast iron (CI) piping



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SOURCE:
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TOWN OF BAR HARBOR, MAINE

HISTORIC WATER SYSTEM TANK LOCATIONS

FIGURE 1

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from Eagle Lake alongside the 16-inch diameter piping. When the 24-inch and 16-inch diameter pipes were connected, the new Mills Meadow reservoir was removed from the water system and abandoned. Both the 16-inch diameter and 20-inch diameter CI pipes remain in service today.

As the Town expanded inland from the harbor, new hotel and home construction occurred at higher elevations. This construction overtook the elevation of the "Scott's Hill" reservoir, which was at or around 100 feet above sea level. The poor water pressure that resulted led to the Water Company's plans to build a new reservoir on what is called "Cunningham's Hill". In 1881, the elevation of this reservoir was at around 200 feet. "Scott's Hill" reservoir was then abandoned. In 1884, only three years after construction of the "Cunningham's Hill" reservoir, the new "Mill Meadow" complex was constructed which comprised of three dams, a new reservoir, and a bypass canal. At this point, the "Cunningham's Hill" reservoir was abandoned.

Even after these improvements, customers complained due to low or unreliable water pressure, interruptions to service, and sediment in the water. The water pressure problems were also from expansion to higher elevations approaching the hydraulic grade of the system. Supply was also an issue, primarily due to the high use of lawn irrigation systems.

Most all of the early work in the system was not engineered, but driven by the experience of prior Water Company Owners. Between 1874 and 1901, many downtown mains were upgraded with larger diameter pipe and fire hydrants were upgraded from 3-inch to 6-inch diameter. The system was expanded north to Hulls Cove with an 8-inch diameter pipe. During this period, much of the early 2-inch to 4-inch piping was upgraded to 6-inch to 10-inch sizes. However, based on our understanding of the system, there is still some original piping that remains.

Even back in 1900, the supply system was not adequate to meet the very large summer demand on the system. Construction of a 700,000-gallon reservoir was completed in 1901. This reservoir was located above the Duck Brook Road on Great Hill. This reservoir was concrete and was designed by Freeman Coffin. The reservoir was below the elevation of Eagle Lake and was intended to be filled by gravity at night when excess water was available.

In 1905, water users started to complain about the water's taste and odor. The source of the problem was determined to be Blue Green Algae or Uroglena. To address this issue, a filter system was designed and built near Eagle Lake, across from Route 233. Water was first brought in from Eagle Lake through 24-inch diameter piping to a primary dual-chambered sand filter, then to a single chambered sand filter, and finally was pumped through an aeration tank. These gravity sand filters were open to the environment. At the time, these significant treatment improvements stabilized the water quality issues.

Following this time, 2,700 LF of 20-inch diameter CI pipe was installed from the junction of the 16-inch diameter and 14-inch diameter pipe at New Mills Meadow to the new reservoir augmenting the supply that was already available from the 16-inch and 14-inch diameter pipes. The 12-inch diameter pipe was replaced with a new 20-inch diameter pipe between the reservoir and Bloomfield Road.

In 1921, the water distribution system was expanded from Hulls Cove to Salisbury Cove. This expansion was largely driven by the Mount Desert Island Biological Laboratory. At this time, a 50,000-gallon water tank was constructed on Dreamwood Hill near the Mount Desert Island Biological Laboratory.

The filtration system was operated for around 30 years and was abandoned around 1933 due to issues with the concrete structure heaving and ongoing plugging of the sand filters. Following the abandonment of the filtration system, the system started utilizing chlorine for disinfection to control taste and odors. A new gatehouse at Eagle Lake and chlorination building at Duck Brook were constructed. In and around 1936, a new steel tank was constructed on Great Hill above New Mills Meadow to aid supply during the high demand summers. The tank elevation was such that electric pumps were installed adjacent to the chlorination building. Raw water was historically pumped up to the summer tank in the summer and flowed gravity to the system during off season months. In a period of time around the 1950's, the water system started adding lime into the water for pH and corrosion control.

The extent of water treatment was the addition of chlorine and lime up until 1963 when fluoride was also added for dental protection considerations. In 1991, the water system had to expand its Eagle Lake gate house due to a requirement from the Drinking Water Program to move the chlorination system to the head of the water system to provide a sufficient contact time (CT) before it reached its first customers. The piping between Eagle Lake and the first customer was utilized to gain acceptable CT.

In 1968, a 500,000 gallon steel storage tank was installed on the southern end of the system with the main purpose of supplying domestic and fire protection to the Jackson Laboratory. This tank also provides some storage capacity to the downtown area during peak user demands in the summer months.

The 12-inch iron Duck Brook line was abandoned in 1997 and replaced with the 12-inch diameter welded HDPE pipe, which continues to serve as a summer line during higher production periods. The 12-inch cast iron line has leaded ball joints and was abandoned in-place within National Park property.

In 1998, the 20-inch intake pipe was also extended 284 feet further into Eagle Lake using 24-inch diameter HDPE pipe.

In 2000, after 126 years of operation as a privately-owned Water Company, the system was purchased by the Town of Bar Harbor, making it a Town owned and operated municipal water system.

Over the years, as regulations increased to include surface water treatment rules and control of chlorinated organics (trihalomethane and haloacetic acid), more recent additional upgrades to the Bar Harbor Water system have included the following:

- In 2001, a new 500,000-gallon concrete water storage reservoir was built below Great Hill to replace the "summer tank" and later partitioned to increase the contact time again between chlorine and the water in order to comply with requirements for unfiltered water systems.

- In 2009, the addition of ammonium sulfate (ammonia) along with chlorine was implemented in order to chloraminate for the control of chlorinated organics.

- In 2013, the water treatment building at Duck Brook was completely rebuilt including all new chemical feed systems. This project also included:

    - Installation of two new raw water pumps and one finish water pump to transport water into and out of the new concrete storage tank/chlorine contact tank.

    - Addition of Ultraviolet Light for the control of *Cryptosporidium*, compliance with the surface water treatment rules, and the allowance for reduction of chlorine for disinfection in order to control chlorinated organics.

    - New chemical feed systems for the addition of sodium hypochlorite for the control of *Giardia* and viruses.

    - Addition of carbon dioxide (CO_2) for pH control.

1.3 WATER SYSTEM OWNERSHIP, MANAGEMENT, AND STAFFING

The Town of Bar Harbor Water system is a municipal system that owns and operates the pubic water treatment and distribution system. The Town of Bar Harbor has owned the water system since 2000, when it was purchased from the Bar Harbor Water Company.

The Town of Bar Harbor's Water Department is structured under the Public Works Division of the Town's Government. The Water Department itself has a Utilities Superintendent, Office Manager, and four operational staff. These employees perform various operational functions, some of which are listed as follows:

- Quarterly Meter Readings
- Duck Brook Pump Station Operations and Testing
- Distribution System Maintenance
- System mapping
- Leak Repairs and Detection
- Meter Replacement and Repairs
- Response to multiple dig-safe requests (up to 300 per year)
- Seasonal meter and service turn on/turn off's
- Customer Relations and Complaint Response
- Hydrant Flushing and Maintenance
- Asset Management and Maintenance
- Supply Procurement
- Service Work Orders (Turn on, Turn off, Name Changes)
- Emergency/On-Call Services
- Drinking Water Program Sampling as Required
- System Freeze up Prevention as Required (Pumping Hydrants, Running Bleeders)
- New Service Installations
- Construction Supervision
- Recordkeeping/Administration/Training
- Drinking Water Program and PUC Reporting

These duties are disseminated daily by the Utilities Superintendent. Based on our understanding of the age of the system, extensive number of tasks, size of the system, complexity of the system and size of the seasonal population, we would recommend an additional staff member be hired to assist with distribution maintenance.

The Town's Water Treatment system is classified as a Grade III water system based on the Drinking Water Program classification recently updated during the recent Sanitary Survey. The distribution system is classified as a Grade II system.

1.4 OVERVIEW OF EXISTING WATER SYSTEM

The Town's water system is located in Hancock County, Maine and its service area encompasses an area of approximately 7.11 square miles serving only the Town of Bar Harbor. The Town has experienced significant development and population growth over the years since it was initially developed. A large percentage of the recent growth is due to seasonal customers and tourism. The population increased by approximately 5.8 percent during the period of 2010 to 2018. Based on past growth, the Town is expected to grow from a population of 5,535 in 2018 to 5,936 in 2028. In 2018, the Town provided water service to 1,850 customer connections, or a population of approximately 4,625 within the service area, which primarily serves the downtown portion of Bar Harbor.

Table 1 provides a summary of key water system information:

TABLE 1: KEY WATER SYSTEM INFORMATION

DESCRIPTION	TECHNICAL DATA
Water Service Area Population (2018)	4,625
Existing Water Service Area	7.11 Square Miles
Total Connections (2018)	1,850 Service Connections
Annual Supply (2018)	365,425,000 Gallons
Average Daily Demand (2018)	276,246,000 Gallons
Maximum Daily Demand (2018)	2,071,300 Gallons
Distribution System Leakage (2018)	24.4 percent or 89,179,000 Gallons
Number of Pressure Zones	Four
Source of Water Supply	Eagle Lake
Hydraulic Control/Maximum Elevation	Dam No. ME00397 at 276.5 feet
Eagle Lake Storage Area	Max. – 1620 acre-ft., Ave. – 1,350 acre-ft.
Capacity of Treatment System/Pumps	2,500 GPM (1 Finish Pump) (2 Raw Pumps)
Number of Booster Stations	3 - (Arata, Mountain Ave. and Straw. Hill)
Number of Active Reservoirs/CC Tank	2 – Jackson Lab and Great Hill
Capacity of Active Tanks/Reservoir's	1,000,000 Gallons (At full level)
Total Length of Water Main/Transmission	189,529 linear feet
Number of Public Fire Hydrants	110
Number of Private Fire Hydrants	25

1.5 COORDINATION WITH TOWN'S COMPREHENSIVE PLAN

The Town of Bar Harbor last updated its Comprehensive Plan in 2007. One of the Comprehensive Plan's major goals was to protect and manage the quality of freshwater resources in Bar Harbor. More specifically, the Town's Comprehensive Plan has policies which incorporate maintaining a high level of value on the Town's Filtration Avoidance Variance by maintaining high quality water, as well as incorporating improvements to the water system. Specifically, Policies 5G and 5H are noted below:

- 5G – To continue to ensure clean and efficient operation of the present water system and plan for improvements and/or expansion as specified in the Water System Master Plan or to support the Town's Future Land Use Plan.

- 5H – To continue to monitor the water quality of Eagle Lake, and Bubble Pond as needed, and limit any activity on Eagle Lake which could affect the rating of the water and trigger additional treatment requirements.

Each policy has a list of strategies to implement each of the desired goals. It is essential that the Water Department coordinate with the Town to continue to achieve these goals.

The Town's Comprehensive Plan also indicates that the goal would be to encourage some expansion in the Hull's Cove Village Areas:

- In the vicinity of Route 3 between the Bluffs and the northern side of the Crooked Road and,
- On either side of Crooked Road between the shore and the gravel pit.

The Town's Comprehensive Plan also encourages growth in the Salisbury Cove Areas of the system:

- Along and South of Route 3.

Expansion in the water system is limited due to Acadia National Park and the natural topography of any potential buildable properties. The Town encourages ongoing ten-year plans to identify the need to improve, replace and/or expand capital facilities and services necessary to support growth and development. The Town's Comprehensive Plan incorporates aspects of the previous Water System Master Plan which now will need to be updated to include the new recommendations included in Section 12.0. The Town plans to start the process to update its Comprehensive Plan in 2020.

1.6 WATER SERVICE TERMS AND CONDITIONS

The Town's Water Service Terms and Conditions were last updated and effective August 1, 2012. We would suggest an annual review of the Terms and Conditions and an update every five years if necessary.

These conditions cover a wide range of management activities and programs including customer service, fees for services, supply management, conservation, facility design and payment terms. A notable requirement related to system pressure is listed below:

- The Terms and Conditions provide that a limited service contract between the customer and the Water Department should be required where substantially uniform water system pressure may be expected to fall below 20 PSI except for periods of fire flow or system maintenance.

There may be areas within the system where there should be Limited Service Contracts based on our evaluation of the system.

2.0 EXISTING WATER SERVICE AREA AND USAGE EVALUATION

2.1 INTRODUCTION

This section of the Master Plan describes the Town of Bar Harbor's existing water service area, categories of water use, significant users in the system and water system losses.

2.2 WATER SERVICE AREA

Figure 2 shows the extent of the water system service area which extends out on either side of the downtown of Bar Harbor to its seasonal customers. Approximately 44 percent of taxable parcels have onsite private wells and about 56 percent are served by the Town's public water system.

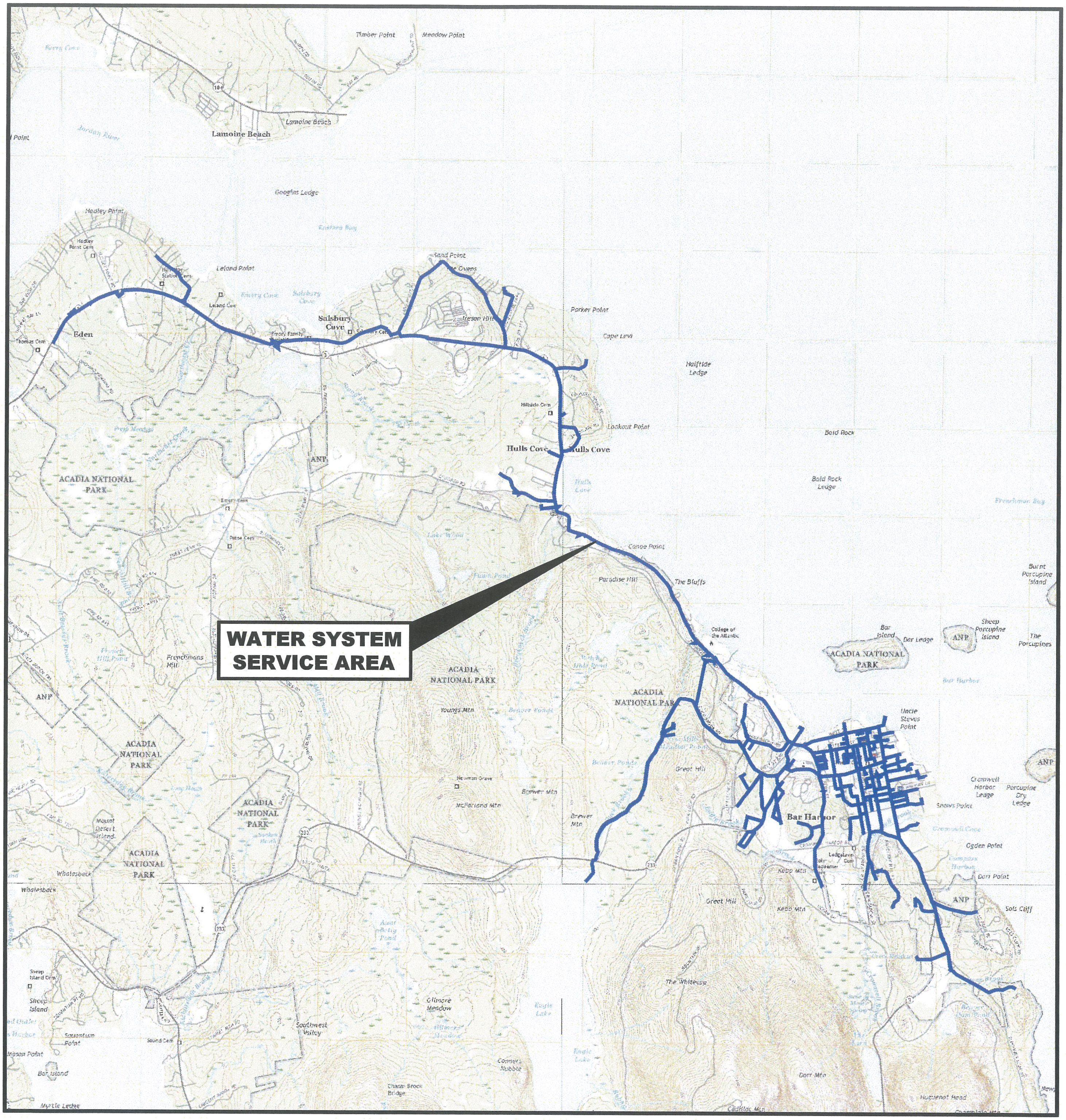
2.2.1 EXTENT OF SYSTEM

The Town consists of around 189,529 linear feet (LF) of potable water main with approximately 1,850 water service connections and a population of about 4,625 customers. The Town's water service area is about 7.11 square miles as approximately shown on Figure 2.

There are not any immediate plans to extend the service area beyond its current location. There are some locations within the existing service area, with the potential for development, that do not have access to public water supply and could be expanded to.

2.2.2 TOPOGRAPHY OF SYSTEM

The topography of the Town's water service area varies extensively in elevation. The lowest areas within the service area are downtown and in Hulls Cove where the elevation is approximately at sea level. These lower elevation customers have water system pressure values higher than 100 PSI, which is higher than recommended. Elevations range from sea level in this portion of the service area to a maximum elevation of over 230 feet as shown on Table 2. These customers have lower than the recommended water pressure. These areas are examples of how elevated topography impacts pressure. The data is based on the best-case conditions at average flows and maximum tank levels. These pressures are lower during the Summer and during hydrant operations.



TOWN OF BAR HARBOR, MAINE

WATER SYSTEM SERVICE AREA

FIGURE 2

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SOURCE:
USGS BAR HARBOR QUADRANGLE
USGS SALISBURY COVE QUADRANGLE
USGS SOUTHWEST HARBOR QUADRANGLE
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TABLE 2: KEY WATER SYSTEM ELEVATION AND PRESSURE DATA

LOCATION	ELEVATION, FEET	PRESSURE, PSI
Juliano House (Hamilton Hill)	231.90	19
Strawberry Hill	222.22	Boosted Pressure
Mountain Avenue	214.83	Boosted to 84
Arata Drive	211.60	Boosted to 90
Cleftstone Road	211.43	27
Route 3, Salisbury Cove	207.24	29
Kebo Ridge Development	207.00	28

Topography is a contributor to the challenges in system pressure and managing supply to the outskirts of the community near Jackson Laboratory and the MDI Biological Laboratory. The topography of the water system is also one of the most limiting or challenging factors of extending water supply. Any new connections in areas with expected low pressure should either be boosted and/or made with a Limited Service Contract.

2.3 EXISTING POPULATION AND FUTURE POPULATION PREDICTIONS

The Town of Bar Harbor has a significant downtown area which is the core of the community containing central services, shops and restaurants. Outlying areas include mixed residential, hotels, campgrounds and various institutional facilities. The water system extends out Route 3 to Salisbury Cove and has a significant seasonal population. In addition, the water system provides water to the College of the Atlantic, Jackson Laboratory and MDI Biological Laboratory.

During the summertime, the population in Bar Harbor increases from the census number of just over 5,000 residents to a peak of around 15,000 people. There is also a significant number of tourists visiting the Town during the summer months. This increase in population impacts the water system significantly. The Town's historical population data within the established Town limits since 2010 is shown below on Table 3.

TABLE 3: MOST RECENT POPULATION TRENDS

YEAR	TOWN POPULATION[1]	NO. OF METERS[2]	WATER SERVICE POPULATION[3]
2018	5,535	1881	4,703
2016	5,394	1868	4,670
2010	5,234	1854	4,635
Increase	301	27	68
Percent	5.8	1.5	1.5

[1]United States Census data for 2010, 2016 and 2018.
[2]PUC Report for listed year.
[3]PUC Report reported meters multiplied by 2.5 people/household.

Based on the data in Table 3, the overall Town's population has increased by 5.8 percent between 2010 and 2018. The number of water customers and population serviced in the water service area has only increased by about 1.5 percent over this same timeframe. This can be used to generalize that the majority of the recent growth is beyond or outside the water service area.

The actual number of people served by the Town's water system differs from the water service population presented in Table 3. The Town's Comprehensive Plan estimates that approximately 44 percent of the population does not have public water supply.

2.4 WATER USAGE: PAST AND PREDICTED FUTURE DEMANDS

The Town continually monitors water consumption or demand. Water consumption is the amount of water used by all customers in the system, as measured by the customers' water meters. The last four PUC reports are summarized in Table 4, which shows the seasonal demands on the system. The average supplied water over the last four years was 356.549 Million Gallons (MG) with a range between 343.470 MG and 371.224 MG. Non-revenue water levels averaged 107.122 MG, or 30 percent of the total water pumped. Unknown or unaccounted for water averaged 41.203 MG, or around 12 percent of the total water pumped.

The data presented in Table 4 clearly shows that the months of June through September have the highest flow demands with July and August being the highest two months. June averages 11.3 percent of the annual flow, July averages 14.2 percent of the annual flow, August averages 14.7 percent of the annual flow and September averages 11.7 percent of the annual flow. This corresponds to the noted population increase experienced during the summer months.

TABLE 4: DETAILED 2015 TO 2018 PUC WATER PRODUCTION AND CONSUMPTION DATA

Year	2015				2016				2017				2018				Trend Results	
Month	**Volume***	**% of Year**	**Quarter Total**	**% of Quarter**	**Volume***	**% of Year**	**Quarter Total**	**% of Quarter**	**Volume***	**% of Year**	**Quarter Total**	**% of Quarter**	**Volume***	**% of Year**	**Quarter Total**	**% of Quarter**	**Average Percentage of Year**	**Average Percentage of Quarter**
January	18,162	5.2%		34.0%	17,702	4.8%		32.5%	16,084	4.7%		33.1%	21,880	6.0%		38.9%	5.2%	34.6%
February	18,500	5.3%	53,361	34.7%	16,375	4.4%	54,531	30.0%	14,834	4.3%	48,523	30.6%	17,375	4.8%	56,205	30.9%	4.7%	31.5%
March	16,699	4.8%		31.3%	20,454	5.5%		37.5%	17,605	5.1%		36.3%	16,950	4.6%		30.2%	5.0%	33.8%
April	19,678	5.7%		23.0%	24,215	6.5%		24.9%	20,158	5.9%		23.0%	20,451	5.6%		21.4%	5.9%	23.1%
May	28,931	8.4%	85,618	33.8%	30,731	8.3%	97,217	31.6%	28,954	8.4%	87,817	33.0%	31,211	8.5%	95,524	32.7%	8.4%	32.8%
June	37,009	10.7%		43.2%	42,271	11.4%		43.5%	38,705	11.3%		44.1%	43,862	12.0%		45.9%	11.3%	44.2%
July	47,668	13.8%		34.0%	52,757	14.2%		35.2%	49,863	14.5%		35.2%	52,057	14.2%		35.4%	14.2%	34.9%
August	50,165	14.5%	140,165	35.8%	53,753	14.5%	149,894	35.9%	51,999	15.1%	141,510	36.7%	53,351	14.6%	147,239	36.2%	14.7%	36.2%
September	42,332	12.2%		30.2%	43,384	11.7%		28.9%	39,648	11.5%		28.0%	41,831	11.4%		28.4%	11.7%	28.9%
October	31,429	9.1%		47.0%	33,571	9.0%		48.2%	33,018	9.6%		50.3%	31,060	8.5%		46.7%	9.1%	48.1%
November	19,035	5.5%	66,934	28.4%	18,690	5.0%	69,582	26.9%	17,143	5.0%	65,620	26.1%	17,999	4.9%	66,457	27.1%	5.1%	27.1%
December	16,470	4.8%		24.6%	17,321	4.7%		24.9%	15,459	4.5%		23.6%	17,398	4.8%		26.2%	4.7%	24.8%
Total	346,078	100.0%	346,078		371,224	100.0%	371,224		343,470	100.0%	343,470		365,425	100.0%	365,425		100.0%	
Revenue Water	240,458				250,519				230,483				276,246					
Non-Revenue Water	105,620				120,705				112,987				89,179					
Unaccounted for Water	35,906				43,840				44,887				40,179					

*Volume data is in Thousand Gallons.

PUC Water Production and Consumption Data

Year	2015				2016				2017				2018				Trend Results	
May	28,931	8.4%	65,940	43.9%	30,731	8.3%	73,002	42.1%	28,954	8.4%	67,659	42.8%	31,211	8.5%	75,073	41.6%	8.4%	42.6%
June	37,009	10.7%		56.1%	42,271	11.4%		57.9%	38,705	11.3%		57.2%	43,862	12.0%		58.4%	11.3%	57.4%

The 2018 PUC report indicates that the average daily production was 1,002,200 GPD and the maximum daily demand was 2,071,300 GPD. The 2017 PUC report showed an average daily production of 940,000 GPD and a maximum daily demand of 2,236,000 GPD. The highest peak hourly flow rate was in 2017 at 3,550,000 GPD. The peak flow rate and average daily production data for 2017 and 2018 were used to determine the peaking factor (PF). The PF was determined to be around 3.5.

2.5 CLASSIFICATION OF WATER CONSUMPTION

Table 5 shows the water usage summary for 2018 based on each category of users. There are 1,206 total residential customers who consume an average of approximately 140 gallons per day per connection. This level of residential water use is in the range of what would be expected with typical values ranging from 100 to 175 GPD per household. There are 403 commercial customers who consume an average of approximately 790 gallons per day per connection. There are 41 governmental customers who consume an average of approximately 852 gallons per day per connection. There is only one industrial user (Jackson Laboratory); however, this user has 36 accounts. Jackson Laboratory accounts consume approximately 164,556 gallons per day. There are 55 institutional type users, who consume approximately 508 gallons per day per connection.

TABLE 5: TOWN OF BAR HARBOR – WATER USAGE BY CATEGORY

Customer Category	2018 Totals	Quarter 1 Total (Gallons)	Quarter 2 Total (Gallons)	Quarter 3 Total (Gallons)	Quarter 4 Total (Gallons)	Year Total (Gallons)
Commercial	**403**	**3,681,065**	**27,486,666**	**67,086,048**	**17,933,023**	**116,186,803**
Annual	253	3,680,654	17,046,920	37,382,594	11,769,892	69,880,060
Seasonal	150	411	10,439,746	29,703,454	6,163,131	46,306,743
Government	**41**	**1,243,535**	**2,642,632**	**5,965,300**	**2,900,617**	**12,752,084**
Annual	22	1,243,528	1,338,920	3,182,822	1,969,970	7,735,240
Seasonal	19	7	1,303,712	2,782,478	930,647	5,016,843
Industrial	**37**	**9,386,727**	**11,996,170**	**21,690,856**	**16,989,144**	**60,062,897**
Annual	36	9,386,727	11,853,810	21,132,720	16,981,515	59,354,772
Seasonal	1	0	142,359	558,135	7,630	708,124
Institution	**55**	**1,880,562**	**2,588,117**	**3,886,900**	**1,848,525**	**10,204,104**
Annual	46	1,880,562	2,347,059	3,624,868	1,803,600	9,656,089
Seasonal	9	0	241,058	262,032	44,925	548,015
Residential	**1,206**	**8,269,095**	**13,728,807**	**28,910,709**	**10,634,525**	**61,543,136**
Annual	974	8,257,755	11,307,142	21,393,346	8,998,709	49,956,953
Seasonal	232	11,340	2,421,665	7,517,363	1,635,816	11,586,183
Grand Total	**1,742**	**24,460,984**	**58,442,392**	**127,539,812**	**50,305,835**	**260,749,023**

The overall total use of water per year per customer category is shown below in Table 6.

TABLE 6: USAGE FOR EACH TYPE OF CUSTOMER

Customer Type	Percent of 2018 Water Used (Seasonal and Annual)
Residential	23.6
Commercial	44.6
Governmental	4.9
Industrial	23.0
Institutional	3.9
Total	100

Of all the water used in 2018, the seasonal customers utilized 64,165,908 gallons of water or 23.6 percent of all the water produced. This emphasizes the significance of the seasonal demand on the Bar Harbor water system since much of this use is between June and September.

Table 7 provides the relationship between annual and seasonal customers for each category of water used.

TABLE 7: USAGE PER CATEGORY FOR ANNUAL AND SEASONAL CUSTOMERS

Category	Annual, Percent	Seasonal, Percent	Total Percent
Residential	19.2	4.4	23.6
Commercial	26.8	17.8	44.6
Governmental	3.0	1.9	4.9
Industrial	22.7	0.3	23.0
Institutional	3.7	0.2	3.9
Total	75.4	24.6	100

Residential users utilize about 19.2 percent of the water on an annual basis, seasonal residential customers use only 4.4 percent of the water for a total used for the residential category of 23.6 percent. Commercial users utilize the most on an annual basis at 26.8 percent and are the significant highest user type at 17.8 percent on a seasonal basis for a total of 44.6 percent overall. Governmental users consume 3.0 percent on an annual basis and only 1.9 percent on a seasonal basis for a total use of only 4.9 percent of the water. Jackson Laboratory is the only user that is classified as Industrial and they utilize 22.7 percent on an annual basis and only 0.3 percent seasonally for a total of 23.0 percent of the water used. Institutional users include churches, schools, a library, a hospital, and a nursing home. This is the lowest water use category at only 3.7 percent annually and seasonally 0.2 percent for a total of 3.9 percent.

2.6 VOLUME OF WATER SUPPLY

Water supply is the total amount of water supplied to the system, as measured by the meter at the Duck Brook Pump Station. The measured amount of water supply in any system is typically more than the measured amount of water consumption due to water system leaks and other non-metered water users.

2.7 DISTRIBUTION SYSTEM LEAKAGE

The difference between the water supplied to the system and water consumed by customers is generally considered system leakage. There are many sources of distribution system leakage in a typical water system including actual water system leaks, inaccurate supply metering, inaccurate customer metering, use of water for treatment, illegal water system connections, water usage for flushing or other distribution water use such as firefighting activities. Firefighting activities, hydrant flushing and certain other water system uses can be tracked but are considered a loss or non-revenue water.

Table 8 shows the reported distribution system leakage for Bar Harbor between 2015 and 2018. This data was obtained from the Town's filed PUC reports.

TABLE 8: 2015-2018 REPORTED WATER USAGE AND LEAKS

Description	2015	2016	2017	2018	Average
	(Usage in millions of gallons)				
Metered Customer Use	240,458	250,519	230,483	276,246	246,426
Use at treatment plant	3,000	5,300	5,300	6,000	4,900
Use at flushing hydrants	8,000	8,000	8,000	6,000	7,500
Use at bleeders	10,400	11,000	11,000	8,000	10,100
Summer Mains (Filling/Flushing)	5,000	5,000	7,000	5,000	5,500
Fire Protection	250	250	800	0	325
Main Breaks	35,000	39,260	14,000	3,000	22,815
Service Line Breaks	8,065	8,052	22,000	10,000	24,058
Total Accounted for Non-revenue Water	69,715	76,865	68,100	49,000	65,920
Total Accounted for Water	310,177	327,384	298,583	325,246	315,348
Total Non-revenue Water	105,621	120,705	112,987	89,179	107,123
Percentage Non-revenue	30.52	32.52	32.90	24.4	30.08
Total supply (Finish Water)	346,078	371,224	343,470	365,425	356,549
Unknown Losses	35,906	43,840	44,887	40,179	41,202
Unaccounted for Water %	10.4	11.8	13.1	11.0	11.6

Metered customer use has averaged 246,426 million gallons per year between 2015 and 2018. Of significance is the 10.8% increase between the average use and metered use in 2018.

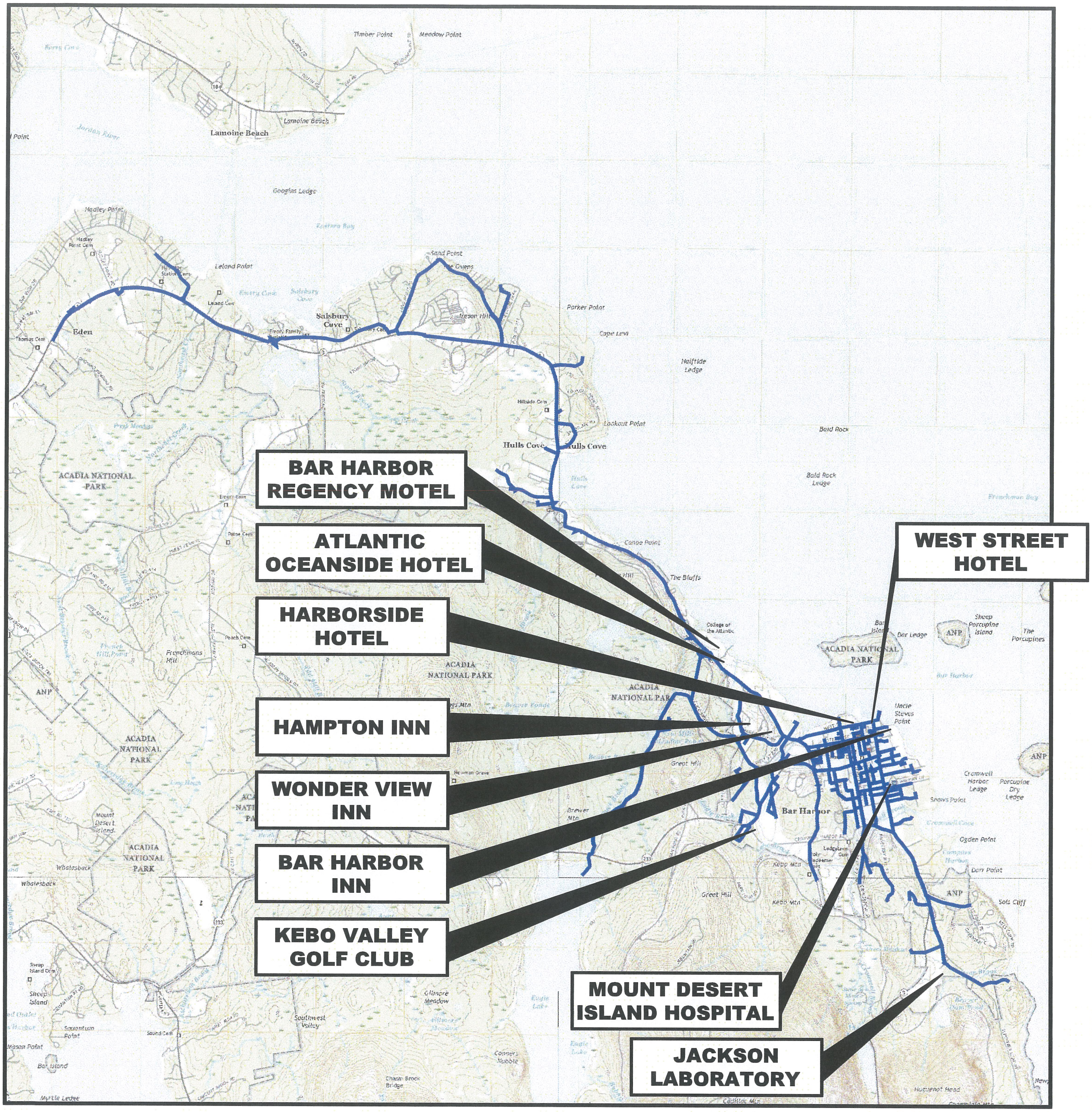
The data shown in Table 8 shows that Bar Harbor has a low unaccounted-for water percentage which ranges between 10.4 and 13.1 percent with an average of 11.6 percent over the last four years. This level of unidentified loss is not considered unacceptable with many systems experiencing losses far greater. The State of Maine does not currently have specific standards for water loss, but unknown losses with a total of less than 10 percent is considered to be the goal.

2.8 SIGNIFICANT WATER USERS

The Town of Bar Harbor has some significant water users who consume a large percentage of the system's water. The largest user is Jackson Laboratory at 23.3 percent of the total water utilized. Jackson Laboratory is located at the Southern end of the water system. Understanding the impact of these large users on the Town's water system is essential when looking at future system improvements. Table 9 presents the ten largest users of the Town's water system. Figure 3 shows where each user is located in the system.

TABLE 9: 2018 TEN LARGEST WATER USERS

Water User	Annual Amount, Gallons	Percent of Total Water
Jackson Laboratory	60,749,149	23.3
Kebo Valley Club	11,005,653	4.2
Bar Harbor Regency Motel	6,820,840	2.6
Bar Harbor Inn	5,818,542	2.2
Harborside Hotel	4,784,133	1.8
Atlantic Oceanside Hotel	4,660,055	1.8
Mount Desert Island Hospital	3,225,563	1.2
Hampton Inn	3,223,326	1.2
West Street Hotel	2,965,222	1.1
Wonder View Inn	2,505,456	0.96
	103,503,028	40%



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SOURCE:
USGS BAR HARBOR QUADRANGLE
USGS SALISBURY COVE QUADRANGLE
USGS SOUTHWEST HARBOR QUADRANGLE
USGS SEAL HARBOR QUADRANGLE
HANCOCK COUNTY, MAINE
7.5 MINUTE SERIES, 2018
SCALE: 1:62,500

TOWN OF BAR HARBOR, MAINE

LOCATION OF LARGEST WATER USERS

FIGURE 3

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Except for Jackson Laboratory and the Mount Desert Island Hospital, the remaining top water users are Motels or Inns related to the Town's significant tourist industry. The largest ten users of the system utilize about 40 percent of the water supplied, or 103.5 million gallons on an annual basis. The current top two users were also the most significant users back in 2002 (April, 2005 Master Plan) as shown on Table 10.

TABLE 10: COMPARISON OF TOP TWO USERS IN 2002 VERSUS 2018

Date	Jackson Laboratory	Kebo Valley Club
2002	44,148,538	12,463,026
2018	60,749,149	11,005,653
Comparison	37.6% more	13.2% less

Between 2002 and 2018, the Jackson Laboratory has increased its water use by 37.6 percent while Kebo Valley Club's water use has declined by about 13.2 percent. The increase in usage by Jackson Laboratory is significant.

2.9 WATER SYSTEM PREDICTED DESIGN FLOWS

Based on past design flow data, history of the water use since the Town's last Master Plan Update and the overall limits on system growth, we would summarize the system's design basis for the next twenty years as follows on Table 11:

TABLE 11: PREDICTED DESIGN FLOWS FOR NEXT 20 YEARS

Current Average Flow	Predicted Average Flow	20 Year Design - Ave
1,001,164 GPD	1,174,288 GPD	2.0 MGD
Peak Daily Flow	Predicted Daily Peak Flow	20 Year Design Daily Peak
2,236,000 GPD	2,623,000 GPD	3.0 MGD
Peak Hourly Flow	Pred. Hourly Peak Flow	20 Year Design Hourly Peak
3,550,000 GPD	4,200,000 GPD	5.0 MGD
(2,500 GPM)	(3,000 GPM)	(3,500 GPM)

In summary, the water system demands have increased about 0.86 percent per year over the last thirteen years. If this is projected over a twenty-year time-frame the expected average design flows would be 2.0 MGD, the peak daily design flows would be 3.0 MGD and the peak hourly flow would be 5.0 MGD or 3,500 GPM.

3.0 EXISTING WATER SUPPLY FACILITIES

This section of the plan provides a detailed description of the existing water system and the current operational practices of the facilities.

3.1 WATER SUPPLY

The Town obtains its water supply from Eagle Lake which is fed from Bubble Pond. This has been the supply for the Town since around 1880. Originally, the Water Company obtained water from New Mills Meadow through a wooden flume. Between 1906 and 1932, the Town operated a rapid sand filtration system with aeration which was installed in 1906 to deal with taste and odor issues due to Blue Green Algae. The Eagle Lake watershed is approximately 3.6 square miles or 436 acres with a storage capacity of about 2,130,920,000 gallons. The Town has indicated that the lake has an expected capacity of about 2,500 GPM of water by gravity based on the water surface elevation when at 276.5 feet.

The Eagle Lake watershed is extremely valuable to the Town and has sufficient capacity to supply water to its customers into the unforeseen future. In the past, the Water Company owned rights surrounding the lake. However, these rights were passed on to John D. Rockefeller Jr. and then to the Park Service which is now known as Acadia National Park.

The Surface Water Treatment Rule was published in the Federal Register by the Environmental Protection Agency on June 29, 1989. Significant revisions were ordered in August 1996, when the Safe Drinking Water Act was reauthorized by Congress. This rule contains provisions that require disinfection and filtration for all public water systems that use surface water or a source that is ground water under the direct influence of surface water.

Only those systems that were able to demonstrate compliance with the stringent source water quality criteria, meet the inactivation (contact time) requirements, and maintain an effective watershed control program obtained an avoidance to filtration. The State of Maine currently has eleven (11) community water systems that qualify for filtration avoidance. This has decreased by one system since the last Master Plan Update.

The Surface Water Treatment Rule established Treatment Technique (TT) Standards for Turbidity, Heterotrophic Plate Count (Bacteria), *Giardia lamblia* cysts, Legionella, and enteric viruses. The monitoring requirements are dependent on the type of filtration and/or disinfections treatment employed by the system.

3.2 SOURCE WATER PROTECTION PROGRAM

The Town of Bar Harbor has obtained its water source from Eagle Lake for many years. In the early years of the Bar Harbor Water Company, the watershed was owned by the system; however, the Town of Bar Harbor does not currently own any portion of the watershed at this time. The lake is about 436 acres, and is the largest fresh water lake in Acadia National Park on Mount Desert Island. It has a maximum depth of 110 feet and an average depth of about 50 feet.

The Bar Harbor Water Company supplies the Town with water from Eagle Lake, 2 miles from the center of the village and within the boundaries of Acadia National Park. The lake is fed by numerous streams flowing off the surrounding slopes and from Bubble Brook, that flows into the lake from Bubble Pond to the south. A dam owned by the Town currently holds the lake surface at a maximum level of 276.5 feet above sea level, 2 or 3 feet higher than it would be under natural conditions. The lake's natural outlet is Duck Brook at the north end of the lake.

The Maine Drinking Water Program (DWP) has evaluated all public water supplies as part of the Source Water Assessment Program (SWAP). The assessments included geology, hydrology, land uses, water testing information, and the extent of land ownership or protection by local ordinance to see how each drinking water source has the potential to being contaminated by human activities in the future. This information is displayed in Google Earth for use by all water systems. Samples have been collected from Eagle Lake since 1981 with very stable results as shown below in Table 12. Some years show a range of results due to increased sampling data points.

TABLE 12: HISTORICAL EAGLE LAKE WATER QUALITY DATA

Year	Epilimnetic*[1] Core (ug/L)	Phosphorus (ug/L)	Chlorophyll (ug/L)	Conductivity (uS/cm)	pH (s.u.)
1981	4	0.006	1.8	-	6.1 - 6.5
1986	4	0.004	1.4	40	7.0
1993	-	0.002	-	38	6.4 - 6.6
1995	4	0.004	1.1	35	6.37 - 7.07
1996	7	0.005	1.7	35	5.91 - 7.01
2001	3	0.003	9.9	38	-
2006	5	0.005	0.8	33	6.51 - 6.74
2007	2	0.002	1.4		5.36 - 6.95
2008	3	0.003	1.2	(33-89)	5.87 - 6.92
2009	3	0.003	1.5	(34-60)	5.80 - 6.96
2010	4	0.004	1.1	(34-82)	5.81 - 6.95
2011	5	0.004	1.3	(30-48)	5.82 - 7.09
2012	2	0.002	0.5	(32-82)	5.44 - 7.12
2013	3	0.003	1.7	32	6.49 - 7.07
2014	3	0.003	1.4	(31-49)	5.40 - 7.27
2015	3	0.003	1.6	31	6.72 - 6.82
2016	2	0.002	0.8	33-42	5.34 - 7.23
2017	3	0.003	1.2	33-39	5.61 - 7.24
2018	3	0.003	1.1	33-39	5.50 - 7.12

1. Epilimnetic Core is related to Total Phosphorus Levels.

The first column in Table 12 refers to Epilimnetic Core samples for Total Phosphorous. Total Phosphorus is the measure of all forms of phosphorus (organic and inorganic) in the water. It is one of the major nutrients needed for plant growth. Because its natural occurrence in lakes is very small, phosphorus "limits" the growth of algae in lake ecosystems. Small increases in phosphorus in lake water can cause substantial increases in algal growth. Phosphorus is measured in units of parts per billion or micrograms per liter (ug/l). Phosphorus concentrations may be based on samples taken from the surface of the lake, from discrete samples taken at specific depths, or from an integrated water column (Epilimnetic core) samples. The values since 1981 range from 2 to 7 ug/l for Total Phosphorus Core sampling. The data between 2012 and 2018 are actually improved from data taken in earlier years, indicating that the lake is not degrading in quality regarding phosphorus. Results between 2012 and 2018 are between 2 to 3 ug/l. The range of all lakes in Maine is from 1 to 137 ug/l with a mean of 12 ug/l.

Phosphorus as a measurement at the surface was also measured as shown in Table 12. The levels range from 0.002 ug/l to 0.006 ug/l. In the recent 2012 to 2018 years, the results are slightly lower with a range of 0.002 to 0.003 ug/l.

Another measurement that is performed in Eagle Lake is Chlorophyll, which is a pigment found in algae and other plants. It is used to estimate the biological productivity of lakes. By measuring the concentration of Chlorophyll in lake water, the algae population can be estimated. The results in Eagle Lake have ranged between 0.5 ug/l to 9.9 ug/l. The peak result was in 2001 and appears to be an outlier where all the other data ranges between 0.5 to 1.8 ug/l. To compare these results to other Maine lakes, the range of Chlorophyll is between 0.7 ug/l to 182 ug/l with a mean of 5.4 ug/l. Again, these results show that the water quality is excellent at Eagle Lake and recent results show no degradation in water quality as they are better now than results in earlier years.

Table 12 also shows results for specific conductance which is a measure of the level of dissolved ions in the water. In addition, conductivity levels will generally increase if there is an increase in pollutants in the lake water. Conductivity is measured in micro-siemens per centimeter (uS/cm). The results measured for Eagle Lake averaged consistently around 33 uS/cm but ranged up to 89 uS/cm due to storm events. Looking at the overall results from other Maine lakes, the conductivity ranges from 10 uS/cm to 2690 uS/cm with a mean result of 50 uS/cm. The mean level in Eagle Lake is less than the mean level found in other main lakes and overall very consistent with only short-term increases caused by storm events or seasonal variations.

Another measure of water quality is pH which is the relative measure of acid-base status of lake water. The pH scale is the inverse log of the hydrogen ion concentration. Water is increasingly acidic below 7 units and increasingly alkaline above 7. The water in Eagle Lake appears to average below 7 units, therefore it would be considered acidic.

The range of pH is from as low as 5.34 units to as high as 7.27 units. The range of pH found in Maine lake waters is from 4.23 to 9.51 units with a mean of around 6.83 units. Eagle Lake data for pH is similar to the mean of all lakes measured and would be considered slightly acidic.

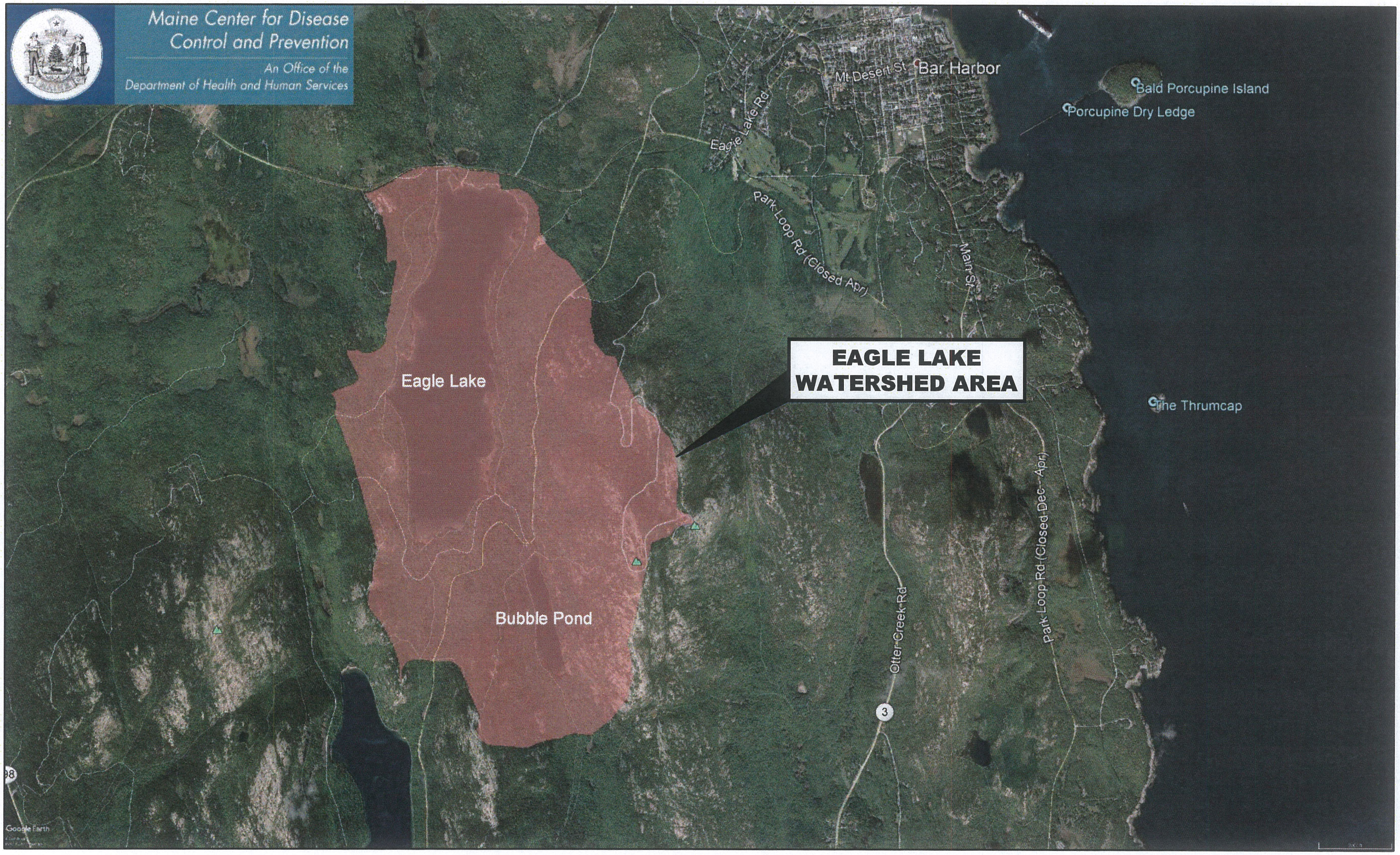
The quality of the lake is of highest importance to the Town of Bar Harbor's water system due to its filtration avoidance waiver. The watershed of the lake encompasses an area of about 3.6 square miles which exists within Acadia National Park. Figure 4 shows the watershed area that feeds the Town's water source.

As another level of protection, the Town has complete control over any potential development on the National Park property due to deed restrictions on the lands surrounding the lake. The Town regulates recreational and other uses of Eagle Lake. These controls prohibit swimming and windsurfing. The lake is posted with "No Swimming" signs which surround the lake. Inspections are done by the Town to control lake use. The Town provides an annual Watershed Report to summarize these activities.

Additional controls include a "Memorandum of Understanding" confirming cooperation between the Park and the Town. In addition, Chapter 14 of the Private and Special Laws of 1973 prohibit boating, fishing, or snowmobiling within 1,000 feet of the "Well House" or Eagle Lake Dam. The rules require all outboard motors to be 10 HP or less and no highway type vehicles are permitted on the ice. The Town also runs annual ads to notify residents of these requirements.

3.3 FILTRATION AVOIDANCE WAIVER

When the Surface Water Treatment Rule was initially enacted in 1989, public water systems using a surface water source became required to provide adequate filtration or switch to a groundwater source. A few select utilities that had protected watersheds and extremely high-quality raw water applied for and received a filtration avoidance waiver. This allowed for continued operation without filtration but included additional requirements detailed in Section 10. Fourteen Maine water systems received a waiver, which included the Town of Bar Harbor. Since then, three have lost this waiver. Given that the water supplies serving the cities of New York, Boston and Seattle also received and still have this waiver, there is support from within the industry for these waivers to be permissible into the future.



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SOURCE:
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OCTOBER 1, 2019
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TOWN OF BAR HARBOR, MAINE

EAGLE LAKE WATERSHED AREA

FIGURE 4

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The Eagle Lake source, the surrounding protected watershed and the filtration avoidance waiver itself are key assets of the Town. They aid in providing the customers with safe water that meets all federal drinking water standards without the additional costs and complexity of filtration. Substantial efforts should be made to continue to meet all requirements of this waiver. These should include continued interaction with the National Park Service regarding allowable uses in the watershed. Current permissible land use activities include hiking, bicycling, horseback riding (limited areas), snowshoeing, camping (limited areas), cross country skiing, and snowmobiling (limited areas). Dogs are also allowed provided they are on a leash and do not contact the water. Bodily contact is prohibited from all public water supplies located within Acadia National Park including Eagle Lake. Fishing is allowed as are boats provided the motor size does not exceed 10 horsepower and boats stay at least 1,000 feet away from the intake. Ice fishing shacks are allowed provided they have solid bases and are located at least 1,000 feet from the intake. ATV use is prohibited on trails but allowed on the lake when frozen. Winter use of snowmobiles on the lake is permissible outside of a distance of 1,000 feet from the intake.

One area of potential disagreement between the Town and the National Park Service is the presence of beavers in the watershed. The National Park Service is against removing them whereas it would be in the best interest of protecting the water supply if they were absent. Beavers are one of the primary host animals that can carry and spread *Giardia*, a pathogen present in freshwater that can cause illness.

Signs in areas surrounding Eagle Lake indicate the pond is a source of drinking water. This information is also included on maps and in the Park's Superintendent's compendium, the rules for activities within the Park.

While the watershed will essentially experience no new development, there will always be the competing needs between access and use by visitors versus the desire to keep activity as restricted as possible to prevent source contamination.

Should this waiver ever be rescinded, the Town would face the need to locate, design, construct and operate a filtration system that meets SWTR requirements. This would be a large capital investment with additional operating costs for chemicals, operation, maintenance and backwash disposal. The former filtration location utilized by the Water Company is now owned by the National Park.

As mentioned above, the Filtration Avoidance wavier is a key asset to the Town and the loss of this waiver would be very serious, costly and have a significant impact to the water system. The Town would have to establish a new location and complete extensive site development for a surface water treatment system which this point would be very difficult and expensive. The current site is surrounded by National Park land and would take serious negations with the park to obtain additional land. Once a location was sited, it would require the construction of a filtration plant designed specifically for treating surface water which would be millions of dollars.

The process of surface water treatment would require expensive utilities to deal with the backwash from the system and would have to covey the waste to the wastewater treatment facility which already has excessive water during I/I events. While operating the current system takes expertise, the filter plant would even be more difficult and time consuming to operate. This would result in more staff and added O&M costs.

3.4 EAGLE LAKE OUTLET DAM

Maine has over 1,000 dams registered with the Maine Emergency Management Agency (MEMA). Of these, 191 are classified as dams of significant and high hazard potential in which failure would result in considerable damages or loss of life. Most Maine Dams are low hazard, which is how the Bar Harbor Water Department's dam is currently classified.

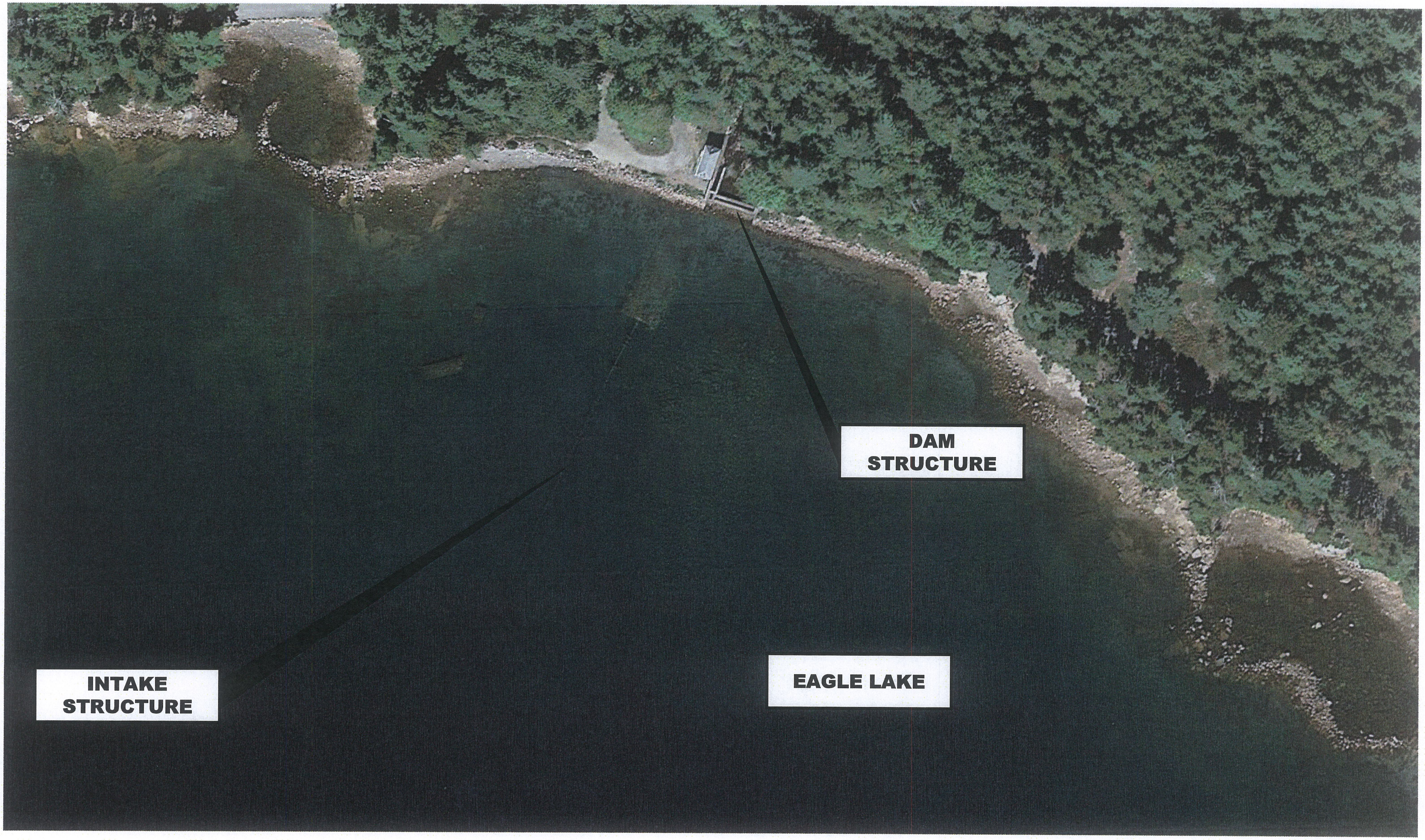
Back in or around 1880, the Water Company completed the initial construction of the earthen dam along the Northern Shore of Eagle Lake. Figure 5 shows the location of the dam. The original dam was approximately 430 feet in length. The dam elevation was raised in 1895. Now there is a concrete spillway and a flashboard structure that maintains the lake at its present normal elevation of 276.5 feet. Note that the level can be lower depending upon the upstream runoff conditions and due to ongoing water that leaks through the flashboards to Duck Brook. Our recent survey data indicated that the elevation was 275 feet at the time of our GPS survey which was conducted in the summer. To be conservative, the elevation of 275 feet was utilized to model system impacts.

The primary purpose of the Eagle Lake Dam is for water storage for the purpose of the Town's public water supply. The Town currently owns the dam at the outlet of Eagle Lake. Since its construction, there have been updates to the original earthen dam. In 1935, the concrete portion of the dam was constructed and was again modified in 1993. Since that time, there has been little to no work done to maintain the dam structure.

The Eagle Lake Dam has a longitude of -68.246 and a latitude of 44.379. The dam is State registered with an ID No. of ME00397. The dam at the outlet of Eagle Lake has a length of 430 feet, a 35-foot-long concrete spillway and a hydraulic height of eight feet. The maximum storage area is 1620 acre-feet and the typical average storage area is 1350 acre-feet. The total drainage area is about 3.6 square miles. The primary dam construction material is earthen dikes with a concrete and granite block dam and concrete spillway with wooden flashboards.

The last official dam inspection was completed in October 6, 2010 where it was determined to be classified as a "Low Hazard Dam" and therefore would not require an

Emergency Action Plan or (ERP). A "Low Hazard Dam" is one with a small storage capacity and which, when released, would be confined to the river channel (Duck Brook). In the event of a failure such a dam would not present danger to human life. Because the Town's dam is considered a "Low Hazard Dam", it is only required to be inspected at least every twelve years and is regulated by the Maine Emergency Management Agency (MEMA). Note that the new inspection only verifies the hazard potential during these inspections so the latest inspection completed in October 6, 2010 did not provide detailed data about the condition of the dam.



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SOURCE:
GOOGLE EARTH
OCTOBER 1, 2019
SCALE: 1:100

TOWN OF BAR HARBOR, MAINE

EAGLE LAKE INTAKE PIPE

FIGURE 5

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290 MAIN STREET WINTERPORT, MAINE

The condition of the dam is of concern since now the State's 494 NEMA-regulated low-hazard dams receive very little attention as can be seen in Table 13:

TABLE 13: CONDITION OF MAINE'S DAMS BY HAZARD POTENTIAL*[1]

Hazard Potential	Satisfactory	Fair	Unsatisfactory	Total
Low	Not Evaluated	Not Evaluated	Not Evaluated	Not Evaluated
Significant	32	25	28	85
High	41	17	6	65
Total	74	42	34	150

[1]ASCE Infrastructure Report Card, 2016

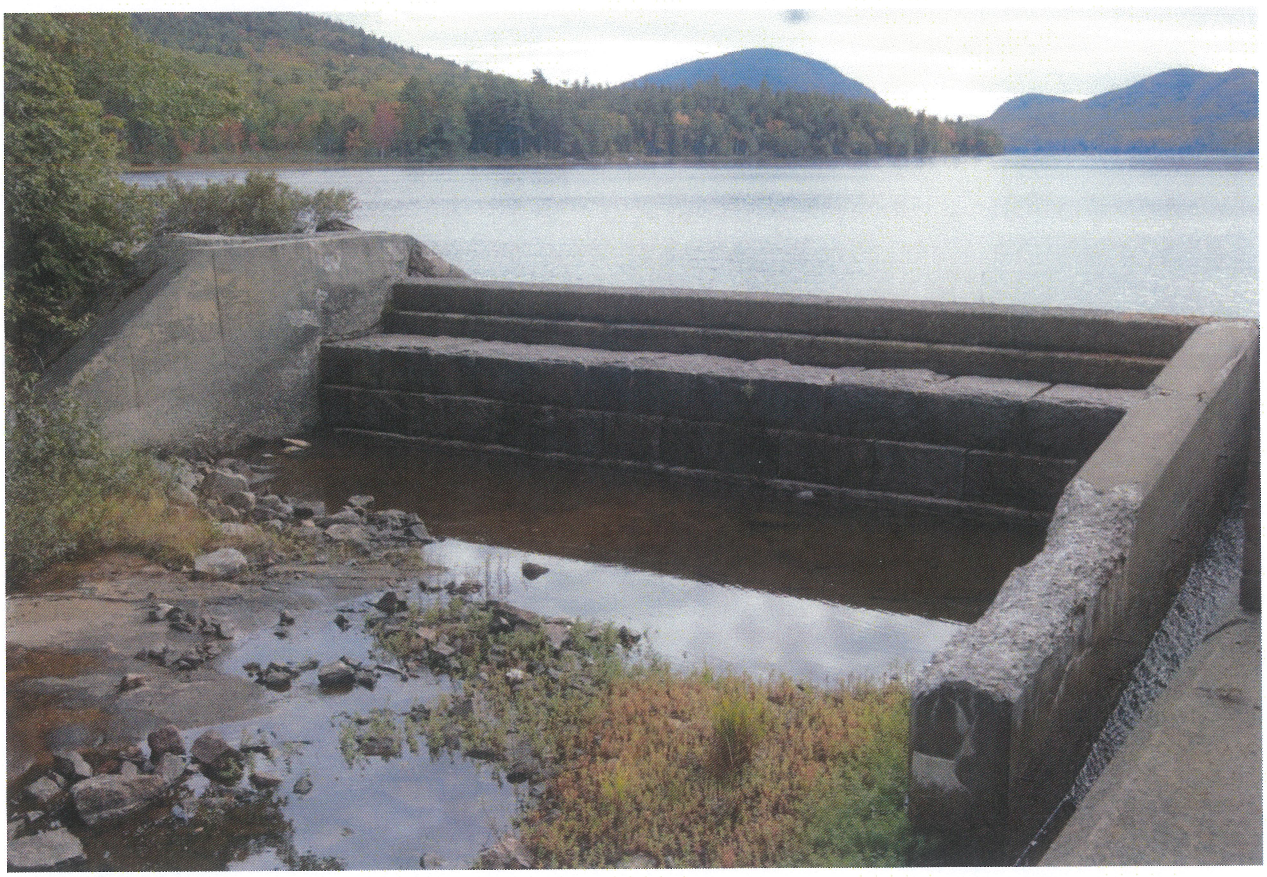
This is of concern to Bar Harbor since, based on our understanding, the dam's condition has not officially been inspected since 1997. In the previous Master Plan, the following observations were summarized by MEMA regarding the condition of the dam (This was excerpted from the 1997 inspection):

- The Eagle Lake Dam is in poor to fair condition.
- There is a reduction of spillway capacity due to the permanent attachment of wooden boards to the spillway crest.
- The sluiceway is not operational.
- The concrete masonry retaining walls are in poor to fair condition, allowing free entry of water.
- There is a concentrated seepage at the toe of the west dike masonry wall downstream from the old pump house.
- The East and West dikes are in poor condition. This includes undercutting, benching, displacement of riprap and overgrowth with large trees. The toe of the dikes contained large areas of wet soil and stagnant water.

The regular inspection and hazard rating of dams are the responsibility of regulatory agencies (MEMA in Bar Harbor's case), but the responsibility for operating, maintaining or repairing a dam is the responsibly of a dam's owner. Of significance to the Town, if the dam were to fail, this would have serious consequences to the water system. The Town would lose about 3.5 PSI and about eighty percent of the available storage at Duck Brook unless they were pumping. Because the conditions noted above have not been addressed to the best of our knowledge, the Town should consider planning for improvements at the dam to prevent such a failure. There are federal grants available for dam repair through FEMA as well as low interest loans through the Dam Repair and Reconstruction Fund.

Overall, most dams that are not inspected for condition in Maine are in poor condition and because of this, there is movement in the state by certain groups to require regular inspections of all dams. The Maine Chapter of the American Society of Civil Engineers gives Maine a grade of D+ considering the capacity, condition, safety, funding, operation, future need and innovation of dam infrastructure in Maine.

FIGURE 6: EAGLE LAKE OUTLET DAM STRUCTURE



3.5 EAGLE LAKE WATER INTAKE

The Town's water supply from Eagle Lake was originally through a 20-inch cast iron intake pipe. This pipe originally only extended about 80 feet out into the lake and was only about 4.5 feet from the surface of the lake. The previous intake had issues with total coliform counts and growth as well as wave action which impacted turbidity at its shallow inlet.

In 1998, the intake pipe was extended an additional 284 feet into Eagle Lake utilizing 24-inch diameter HDPE piping. The new intake extension was designed with a concrete vault structure at the end of the pipe. Figure 5 on Page 30 shows the location of the intake and concrete vault. This structure supports a "Johnson Intake Screen" which is a passive wedge wire screen with an opening of 0.25 inches. The tee with the screen has a normal water depth of about twenty feet which places the actual elevation of the screen at about 7.75 feet above the lake bottom. The screen is inspected annually by a diver and manually cleaned when necessary. Since these changes were made, coliform bacteria and high turbidity have not been an issue for Bar Harbor.

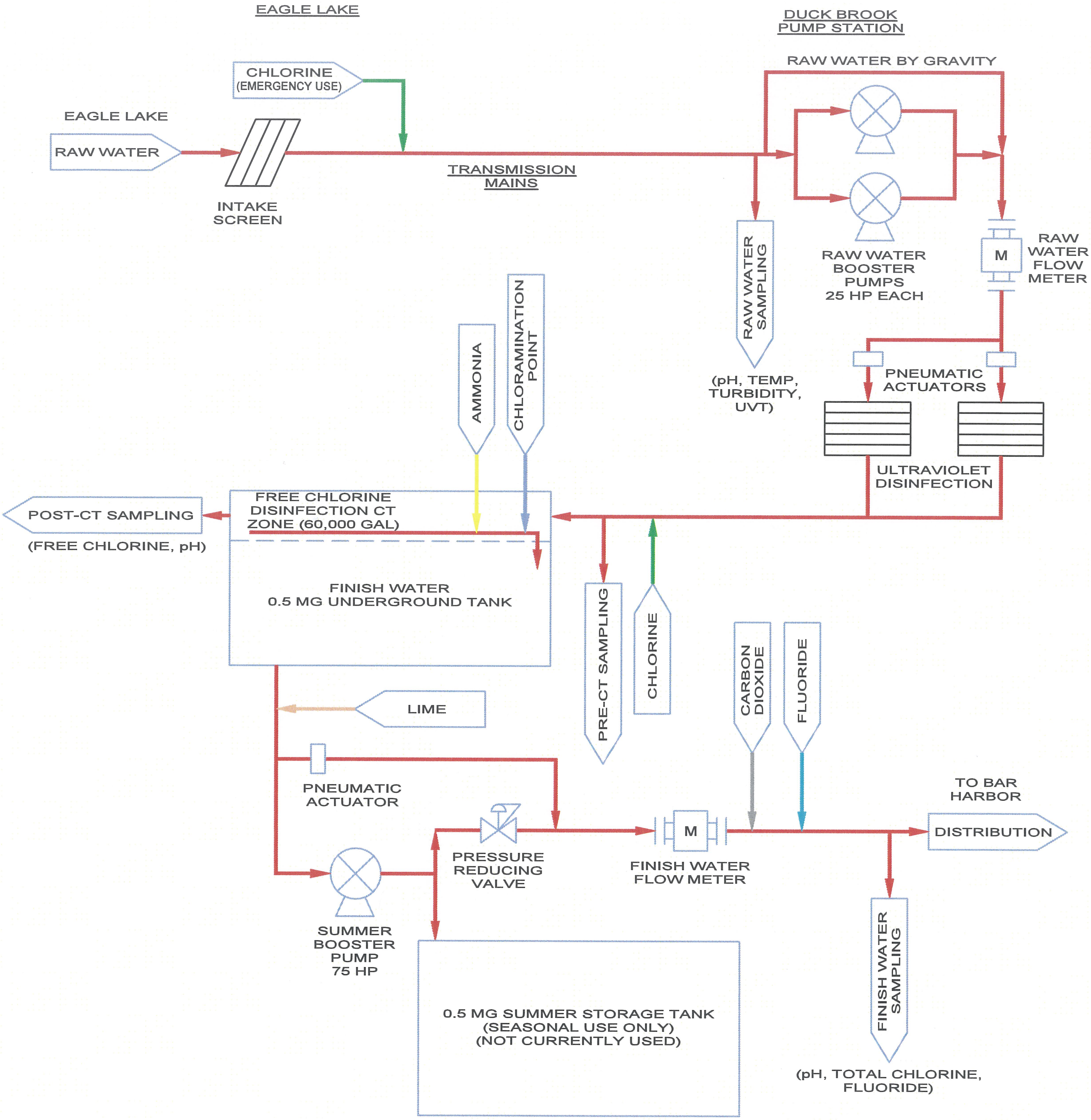
When the new inlet extension was installed, the drop-in screens were removed and the incoming water was piped directly through the screen vault beneath the chlorination building. There is no treatment of the water at the Eagle Lake site. From the intake, the water flows directly to the Duck Brook Pump Station.

4.0 EXISTING WATER TREATMENT FACILITIES

This section provides a brief summary of the Town's treatment process located at Duck Brook. In 2013, the Town constructed an upgrade to its Duck Brook Pump Station. The original facility was upgraded to comply with the Long Term 2 Enhanced Surface Water Treatment Rules and the Stage 2 Disinfection By-Products Rule. The surface water rule requires 2 to 3-log inactivation of *Cryptosporidium* depending upon the quality or test results of the source water. For unfiltered systems, this requirement can be achieved by using chlorine dioxide, ozone, or ultraviolet (UV) light. The Town commissioned a study which determined that the upgrade (addition of UV, and updated chemicals for corrosion control) would be the best method for achieving compliance with both of the Stage 2 Rules. Section 10 discusses the regulatory requirements that the Town of Bar Harbor must comply with.

4.1 DESCRIPTION OF EXISTING WATER TREATMENT AT DUCK BROOK

Figure 7 shows a simplified Process Flow Diagram of the Town's water treatment facility. The Stage 2 Rules Upgrade included installation of two new 25 HP centrifugal raw water pumps (2,500 GPM @ 16 TDH) and one new 75 HP centrifugal finish water pump (2,500 GPM @ 90 TDH). The raw water pumps are only utilized in the summer months and are intended to augment the gravity feed from Eagle Lake during high demand periods. The effluent pump is only used when the system pressure to the Jackson Lab tank cannot be kept at the designed settings by gravity feed from the outlet of Duck Brook. When it is used, the Town's general operating practice is to set the value of the effluent or finish water pump PSI set point to the current value of the Duck Brook effluent PSI. The system valves are opened to direct water through the pump directly into Town. The pump is activated in automatic at which point its output initially circulates. The valve that flows by gravity into Town is then closed to direct the flow from the pump into Town. The pump PSI set point is set to allow for the desired effluent flow/pressure that is between about 24.5 and 25.5 PSI. The Operator observes the Jackson Laboratory tank level to see how it reacts to these changes with the goal of it leveling off or increasing to its full level. In the initial years of the installation of the effluent pump it was operated 3 to 10 days each year. It was not operated in 2019, which the Town indicates is due to the new 16-inch diameter line installed on Route 3 between the South end of Highbrook Road and Eagle Lake Road. In the summer, the raw water pumps feed the Chlorine Contact tank to keep it full. During lower demand periods, the water flows from Eagle Lake by gravity through the 16-inch and 20-inch cast iron water lines which combine into one 16-inch diameter ductile iron line in the yard.



TOWN OF BAR HARBOR, MAINE

WATER SYSTEM SCHEMATIC

FIGURE 7

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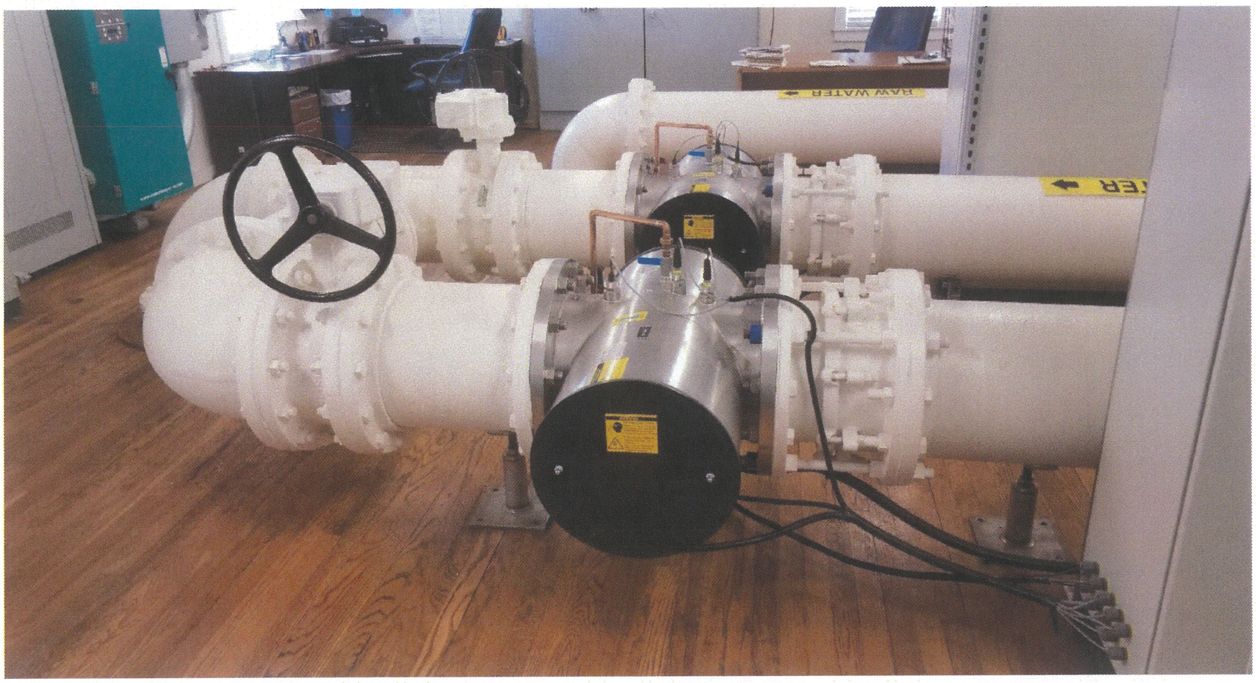
4.1.1 ULTRAVIOLET LIGHT SYSTEMS

The raw water is monitored by the Ultraviolet Transmittance (UVT) system located near the raw water pumps in order to control the UV system dose. During the period of time that the water is flowing by gravity or pumped, the flow is measured through flow meter FM-101. After flow measurement, the raw water flow then enters the Ultraviolet Treatment System (UV) as shown on Figure 8 where inactivation of C*ryptosporidium* occurs. The treatment system utilizes one of two UV systems (UV-201 or UV-202).

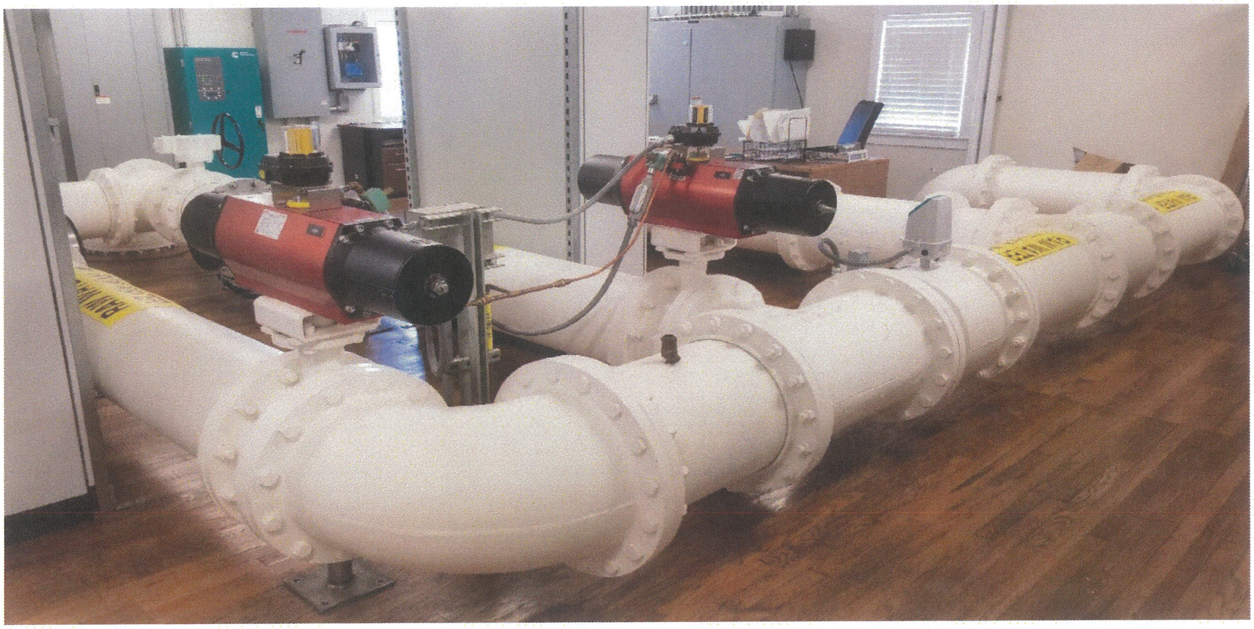
FIGURE 8: ULTRAVIOLET TREATMENT SYSTEMS (UV-201 AND UV-202)



The Town utilizes an UVT monitor for transmittance and the UV system controls to provide information pertaining to dose, lamp power, cleaning status and level data back to the SCADA system through ethernet connections. This information is monitored by the operators to ensure the UV system is operating correctly. If there is an issue with the system, the automated valves shown on Figure 9 will shut down the flow of water.

Typically, one UV system is utilized at a time providing a fully operational backup at all times. Following the UV system, the facility utilizes pneumatic actuators controlled through SCADA to open or close the outlet of the UV treatment systems.

FIGURE 9: ULTRAVIOLET TREATMENT SYSTEM AUTOMATED VALVES



4.1.2 CORROSION CONTROL

Corrosion control is very important in an older water distribution system, especially with the low alkalinity water experienced by Bar Harbor. Following the UV treatment system, the facility utilizes a lime solution to assist with corrosion control. The lime increases the pH and alkalinity of the water which assists with the control of lead and copper. Lime adds about 1.35 mg/l as $CaCO_3$ per mg/l of chemical that is added to the water. Note that the lime will not contribute to the overall carbonate in the system which is why carbon dioxide (CO_2) is added prior to distribution. The Town uses an Acrison Dry Feed system along with two lime solution feed pumps.

The pH level is monitored post CT Tank and also in the finish water. At the finish water location, a 12 GPM pump delivers additional corrosion control chemical feed based on the pH measured at this point. The target finish water residual is currently 9.5 standard units (s.u.) which is monitored by an online Rosemount pH monitor.

4.1.3 DISINFECTION AND ALKALINITY CONTROL

Sodium hypochlorite is added to the 16-inch diameter line heading to the CT Tank which treats *Giardia* and viruses. Two sodium hypochlorite pumps are available to supply disinfectant to the raw water following UV. The desired dosage at this location is 2.0 mg/l. An inline Rosemount monitor and sample point is available for monitoring the dose of chlorine prior to the CT Tank. The manual sample point is verified daily against the automated meter readings. The facility utilizes the SCADA system to calculate the required flow of chlorine to the system as well as monitor the actual flow from the pumps using a mass flow meter. The SCADA system controls the chemical pumps to meet the required chlorine flow. Based on these measurements, the facility will facilitate a shutdown should the chlorine pumps fail to meet the required chlorine flow within +/- 15% of the required flow.

FIGURE 10: SODIUM HYPOCHLORITE MAKEDOWN AND CHEMICAL FEED SYSTEMS

After sodium hypochlorite addition, the raw water flows to the underground storage tank where the required CT is obtained. This tank is baffled with a concrete wall to create two cells. The first cell has a minimum required volume of 60,000 gallons for CT. The baffle wall and a 16-inch ductile iron pipe used to connect the two cells of the tank creates a plugged flow regimen for adequate treatment. The entire volume when the tank is full is approximately 500,000 gallons.

Following the underground storage tank is where the Town applies sodium hypochlorite and then ammonium sulfate to achieve chloramination to control chlorinated byproducts. The amount of ammonia fed is based on the monitored chlorine residual.

Following this, the water flow is measured by the finish water flow meter. The pH of the water is monitored prior and after CO_2 addition. CO_2 is added to the water to provide additional corrosion control by maintaining the alkalinity of the water. Lime carrier water is also added directly following the CO_2.

4.1.4 FLUORIDE TREATMENT MONITORING AND CONTROL

The Town of Bar Harbor uses sodium fluoride to adjust the fluoride content of its water supply. The target residual is 0.7 mg/l with an alarm set point of 0.4 on the low end and 1.3 mg/l on the high side. The fluoride addition will shut down if elevated levels occur. (The levels monitored over the last year range from 0.54 mg/l to 0.75 mg/l.) Continuous fluoride residual monitoring is achieved using an ATI monitor with daily checks completed by the operators.

4.2 CONDITION OF TREATMENT SYSTEM

The condition of the Town's Duck Brook Treatment building and system is excellent and it appears to be operated as required to meet the regulatory standards described in Section 10. Other than how the operation controls the elevation of system storage tanks; we do not have any suggested operational or capital improvements.

5.0 EXISTING PRESSURE ZONES

5.1 PRESSURE ZONES

The Town currently serves its customers within an elevation range of about sea level in the Downtown area of the system to a maximum elevation of over 230 feet. The wide range of elevations is a challenge to the Town and is managed by operating four pressure zones.

5.1.1 PRESSURE ZONE 1

Most of the system is operated from the hydraulic elevation of Eagle Lake at 276.5 feet and the Duck Brook Pump Station and its storage reservoir with an overflow elevation of 276.5. This portion of the system has a pressure gradient at 276.5 feet.

5.1.2 PRESSURE ZONE 2

The tank at the Jackson Laboratory has a current maximum hydraulic elevation of 265 feet such that an altitude valve is needed to control the elevation at this lower point to avoid overflow. We have evaluated raising the elevation of the Jackson Laboratory tank to provide for additional storage. This is discussed in detail in Sections 9, 11, and 12.

5.1.3 PRESSURE ZONE 3

There are two booster stations operated in the system to maintain pressure at Mountain Avenue and at Arata Drive. These two stations are operated with pressure tanks that are connected to a pressure switch that turns on booster pumps when pressures in the pump discharge reach a specific set point. The Mountain Avenue Station is set at a maximum pressure of around 84 PSI and the Arata Drive station is set at a maximum pressure of around 90 PSI. The Town would like to combine these two stations into one above ground station located at the Mountain Avenue site. This will be discussed further under Section 6.

5.1.4 PRESSURE ZONE 4

The small development at Strawberry Hill utilizes a seasonal booster station to increase pressure due to the high elevation at this location. This booster station has one pump and a bladder tank.

There are not any current pressure reducing systems located in the distribution system.

6.0 WATER SYSTEM ACTIVE BOOSTER PUMP STATIONS

As mentioned under Section 5, there are several booster stations operated in the system to maintain pressure where there are high elevation locations which cannot maintain appropriate pressure without having the pressure boosted. This is accomplished utilizing pumps and bladder tanks. The location of the booster stations and bladder tanks are shown on Figure 11. The Town has three booster stations which are located as follows:

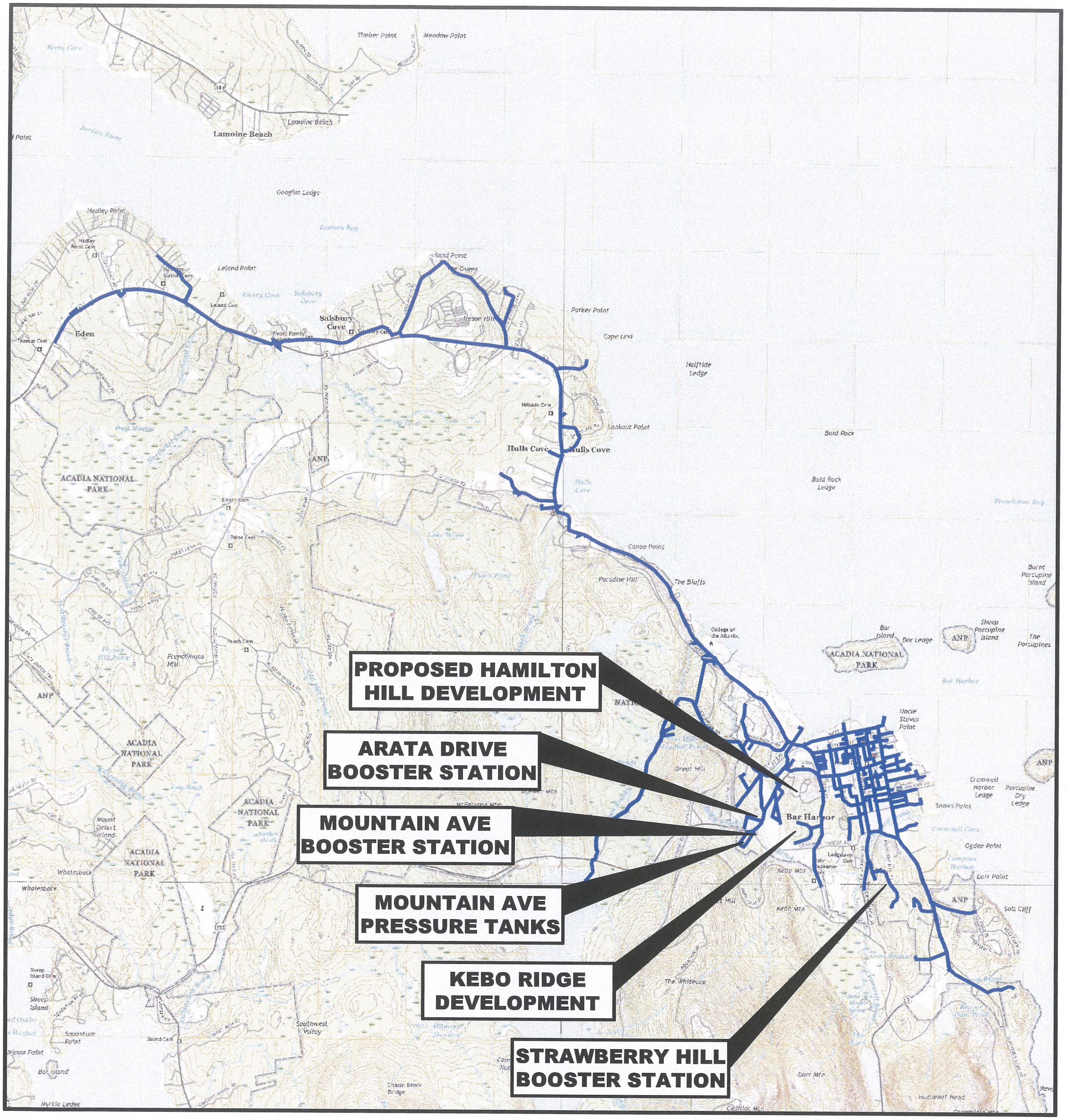
- Mountain Avenue Booster Station which pumps to the four bladder tanks located at Rockwood Avenue.
- Arata Drive Booster Station with one bladder tank.
- Strawberry Hill Booster Station with one bladder tank.

These booster stations are of the closed system design and operated with bladder type pressure tanks (higher pressure) that are connected to a pressure switch that turns on the pumps when pressures in the tank reach a specific set point.

6.1 MOUNTAIN AVENUE BOOSTER STATION

The Mountain Avenue Booster Station is located adjacent to the roadway in an underground building (concrete block vault) with unsafe entry and access. There is only one pump available in the booster pump station and no backup power supply. The current pump is a Gould's 7.5 HP pump and controls set the system pressure to shut off at around 84 PSI. The heating element for the building looks in poor condition as well and the overall insulation is substandard. In addition, the building is subject to groundwater entry which is a concern to the equipment and safety of personnel who need to enter the building. The station does not have any electronic output to the Town's SCADA system in the event of a pressure failure in this area. The Mountain Avenue Booster station provides water and pressure to the bladder tanks located at the top of Rockwood Avenue. The Mountain Avenue Booster station has approximately 28 residential users with 25 that are year-round and 3 customers that are seasonal. Based on the 2018 data, this booster system provided 1,462,879 gallons of water or 143 gallons per day per residential user. This booster station serves a small portion of Eagle Lake Road, Mountain Avenue, East Street, and Rockwood Avenue.

The condition of the tank building is poor with unpainted siding and blocks. The interior is in good condition, but should be have a ceiling installed over the insulation and be re-painted. The condition of the four bladder tanks at the Rockwood Avenue site are very good. If the system was redesigned, these tanks would likely be reutilized. Before this could be determined, actual sizing and the final location of these tanks should be evaluated as part of the upgrade design.



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SOURCE:
USGS BAR HARBOR QUADRANGLE
USGS SALISBURY COVE QUADRANGLE
USGS SOUTHWEST HARBOR QUADRANGLE
USGS SEAL HARBOR QUADRANGLE
HANCOCK COUNTY, MAINE
7.5 MINUTE SERIES, 2018
SCALE: 1:62,500

TOWN OF BAR HARBOR, MAINE

LOW PRESSURE AREAS AND BOOSTER STATIONS

FIGURE 11

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FIGURE 12 – MOUNTAIN AVENUE BOOSTER STATION BUILDING SITE

FIGURE 13 – MOUNTAIN AVENUE BOOSTER PUMP AND INTERIOR OF STATION

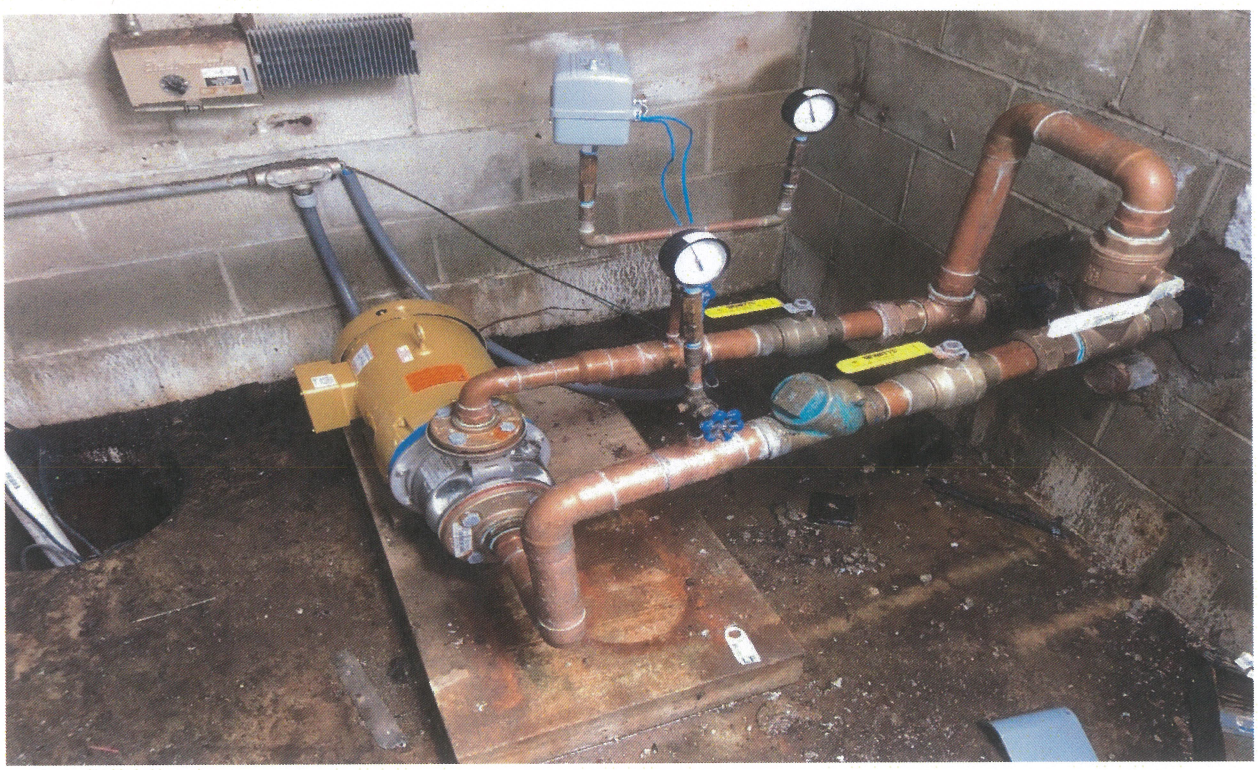


FIGURE 14 – ROCKWOOD AVENUE BLADDER TANK BUILDING

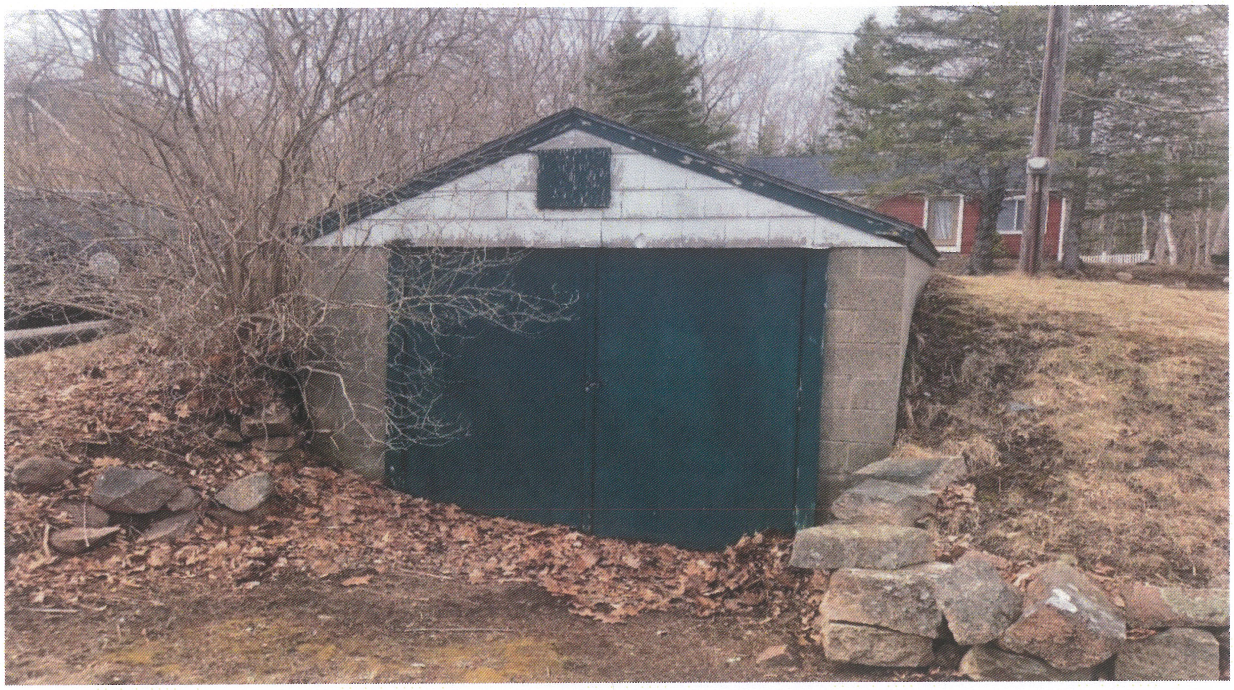


FIGURE 15 – ROCKWOOD AVENUE BLADDER TANKS IN INTERIOR OF BUILDING

If additional tanks were needed, they would be added to the existing. These tanks are 119 gallons with a maximum pressure of 125 PSI. The tanks are required to be pressure rated if any one unit is above 120 gallons. Since they are less than 120 gallons, they do not have to be pressure rated. Building improvements are discussed further under Section 12.

6.2 ARATA DRIVE BOOSTER STATION

The Arata Drive Booster Station is located adjacent to the roadway in an underground building (concrete block vault) with unsafe entry and access. There is only one pump available in the booster pump station and no backup power supply. The heating element for the building looks in poor condition as well. The station does not have any electronic output to the Town's SCADA system in the event of a pressure failure in this area.

The Arata Booster station provides water and pressure set at around 70 PSI to turn on and 90 PSI to shut off. The pump is a Bell and Gossett Model 31161. There is only one bladder tank located in the building. The building is extremely crowded as well with no room to work on equipment when needed. In addition, the building is subject to groundwater entry which is a concern to the equipment and safety of personnel who need to enter the building. The Arata Drive Booster station has approximately 14 users with 13 that are year-round and one customer that is seasonal. Based on the 2018 data, this booster system provided 583,462 gallons of water or 114 gallons per day per residential user. This booster station only serves Arata Drive.

The Town would like to combine the Mountain Avenue and Arata Drive Booster Stations into one above ground station located at the Mountain Avenue site. This improvement would include connecting the two areas together in a loop, and providing an above ground building, backup generator, dual centrifugal style pumps, VFD drives, backflow prevention and communication back to the Town's SCADA system. An easement would also need to be obtained to conduct this work. These improvements will be discussed further under Section 12.

6.3 STRAWBERRY HILL BOOSTER STATION

The Strawberry Hill Booster Station was installed recently and is in good working order. This Station serves just 6 residents with 2 year-round and 4 annual residents. This station was installed following the construction of the 3-inch diameter HDPE line that feeds up to elevation 222 feet. The booster station provided 242,367 gallons of water or 111 gallons per residential customer per day.

FIGURE 16 – ARATA DRIVE BOOSTER STATION BUILDING SITE

FIGURE 17 – ARATA DRIVE - INTERIOR OF BOOSTER STATION

7.0 WATER STORAGE FACILITIES

The Town of Bar Harbor historically and to this day has limited water storage capacity. Currently, the Town only has about a half day of water storage available. This limited storage capacity has been acceptable to the Town due to its gravity feed from Eagle Lake being considered "continuous feed". However, during peak season conditions, this concept of capacity would not be sufficient and further capacity should be considered. Another major concern would be a main break which could cut off flows along Route 3, Hulls Cove, and Salisbury Cove. A capacity evaluation was completed for the purpose of this Master Plan and is included in Section 9.

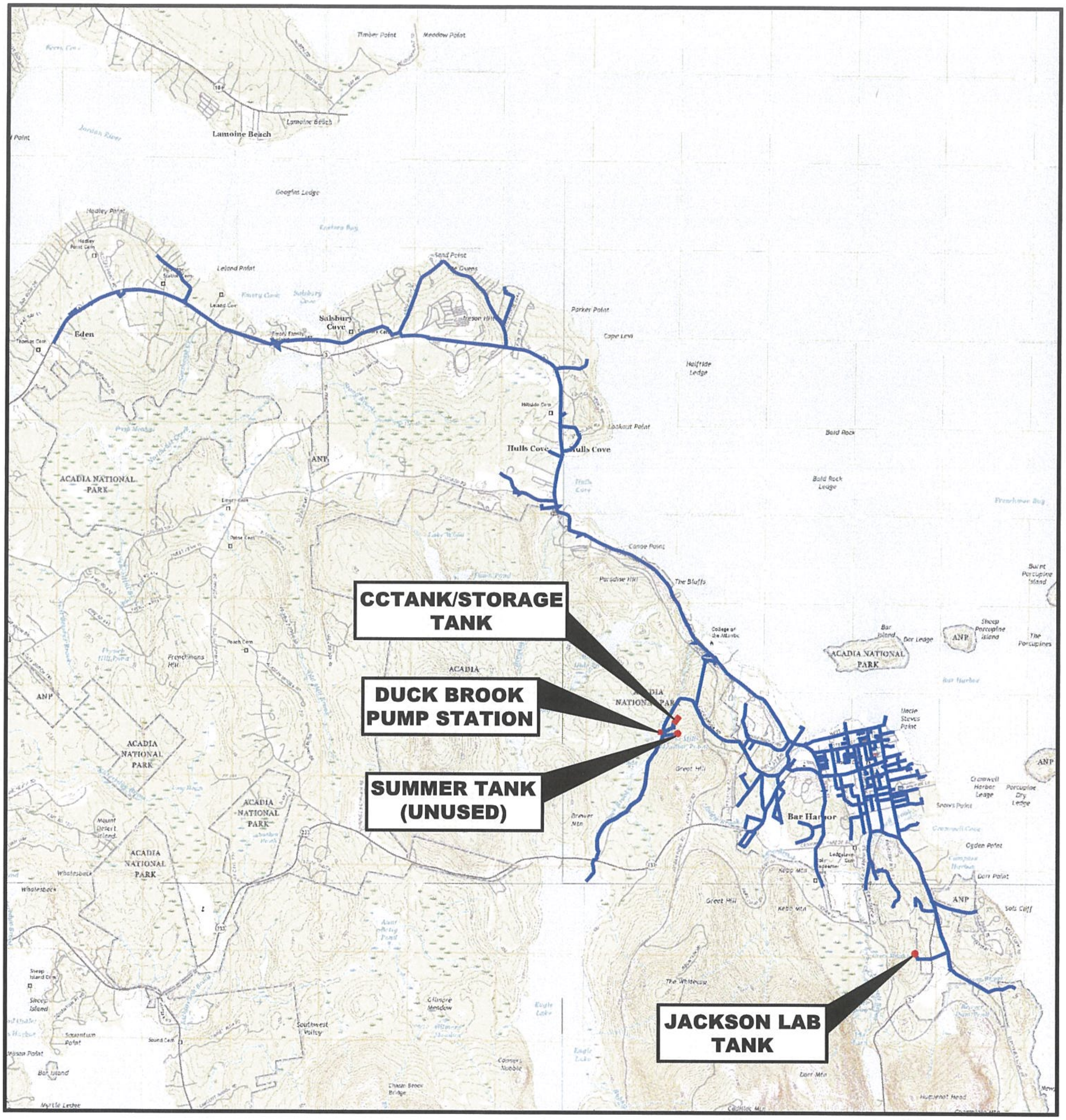
The maximum existing water storage capacity when tanks are full is only 1,000,000 gallons. Two 500,000-gallon storage tanks are presently in use. Figure 18 shows the location of the existing water storage tanks. One is located across from the Duck Brook Pump Station and the other tank is located on the far Southern end of the system on Route 3 across from the Jackson Laboratory. The tank at the Duck Brook Pump Station was installed initially for storage and subsequently baffled to provide the required chlorine contact time along with storage. Over the years, multiple water storage tanks have been installed and abandoned at different locations, pressure gradients, and volumes.

Table 14 provides a summary of the multiple reservoirs/tanks constructed over the life of the water system:

TABLE 14: WATER STORAGE HISTORY AND EXISTING DISTRIBUTION TANKS

Tank Name	Year Installed	Year Abandoned	Location	Tank Size Base El./ Overflow El.	Type
Scott's Hill Reservoir	1874	1881	Scott's Hill	150,000 Gal.	Stone
Cunningham Hill Reservoir	1881	1884	Cunningham Hill	Unknown	Unknown
New Mill's Meadow Reservoir	1884	1887	New Mill's Meadow	Unknown	Unknown
Stone Standpipe	1887	1895	Below Eagle Lake	Unknown	Stone
Great Hill Reservoir	1901	1928	Great Hill	700,000 Gal.	Open top Concrete
Dreamwood Hill	1921	Removed in 2019	Robbins Motel	50,000 Gal. 190 ft / 215 ft	Seasonal Riveted Steel Tank
Great Hill Standpipe	1936	Not in Use as of 2013	Across Duck Brook PS	528,000 Gal. 317 ft / 341 ft	Riveted Steel Standpipe
Jackson Lab Tank	**1968**	**In Use**	**Across Jackson Lab on Rt. 3**	**500,000 Gal. 235 ft / 265 ft**	**Welded Steel**
New Mills Meadow Tank	**2001**	**In Use**	**Great Hill, Across Duck Brook PS**	**500,000 Gal. 266 ft / 276.5 ft**	**Cast in place concrete**

*Bold indicates storage tanks presently on-line.



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SOURCE:
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TOWN OF BAR HARBOR, MAINE

EXISTING WATER SYSTEM TANK LOCATIONS

FIGURE 18

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A more detailed description of the reservoirs or tanks in service is provided below:

7.1 JACKSON LABORATORY STORAGE TANK

The tank at Jackson Laboratory provides 500,000 gallons of water storage for the southern end of the system along Route 3. This tank is primarily for the Jackson Laboratory which is the single largest user of the Bar Harbor Water System using 23.3 percent of the system's water demand or 166,436 gallons per day in 2018. The tank provides potable water and fire protection needs for Jackson Laboratory and also provides fire protection volume at this location in the system. The Jackson Laboratory tank is a 500,000-gallon welded steel tank constructed back in 1968. It was installed with a base elevation of 235 feet and an overflow elevation of 265 feet. The tank is controlled at a lower pressure gradient with an altitude valve. In the high demand portion of the day, this tank empties and fills at night when demand drops in the system.

This tank appears to be very well maintained and a single 10-inch diameter water main serves as the tank inlet and outlet pipe.

FIGURE 19 – JACKSON LABORATORY STORAGE TANK

7.2 NEW MILLS MEADOW RESERVOIR/CHLORINE CONTACT TANK

The reservoir on the hill across from the Duck Brook Treatment Plant was constructed in 2001 and upgraded in 2012. This reservoir is an insulated concrete cast-in-place tank with a base elevation of 266.0 feet with an overflow elevation of 276.5 feet. This reservoir contains 500,000 gallons of storage and replaced the steel Summer Tank adjacent to it that was constructed back in 1936. In 2012, the Town added a chlorine contact chamber (CC) sized at a minimum of 60,000 gallons. This tank was located part way up the hill adjacent to the off-line steel tank. Most of the year, the concrete tank receives gravity flow from Eagle Lake; however, in the busy summer months the treatment facility raw water pumps are designed to feed flow to the tank to maintain sufficient volume. The Town's treatment facility also has one finish water pump which supplies the distribution system from the tank in the summer months to maintain the required demand when necessary.

This tank appears to be in good condition. We would recommend a minimum of every five years cleaning and inspection of the concrete tank. Buried concrete tanks can, over time, have issues with roof seam leaks and growth through the roof, so regular inspections should be conducted.

FIGURE 20 – RESERVOIR/CHLORINE CONTACT TANK

7.3 GREAT HILL STANDPIPE

The Great Hill Standpipe was originally installed back in 1935 and is directly across from the Duck Brook Pump Station. The Standpipe is a Riveted Steel Tank and was designed to be a seasonal storage tank during high demand periods of time. The tank is on National Park property and at a higher-pressure gradient. The base elevation of the tank is at 317 feet and the overflow elevation is at 341 feet. The tank has approximately 528,000 gallons of storage when full. Since the inception of the new Duck Brook facility, the Town has been able to either flow gravity to Town or pump directly to Town during high demand situations and has avoided the use of this tank. The tank has not been utilized or inspected since 2013. If the tank was to be utilized, the water would go through the existing underground reservoir and have to be pumped up into the tank with the summer (effluent) pump. This tank would discharge back through the Duck Brook Pump Station through the pressure reducing valve and then to distribution.

FIGURE 21: OFF-LINE GREAT HILL STANDPIPE

At this point, the Town does not plan to use this tank as it is at a very high elevation which complicates the operation of the Duck Brook facility. We would recommend keeping the tank until final decisions are made about water storage such that the land would still be available for another use or if it is decided that this additional storage would be useful at Duck Brook the Town could decide if the tank was worth preserving. In our opinion, there are better locations for additional water storage than at this elevation or this location.

7.4 WATER STORAGE TANK MAINTENANCE

The Town inspects Jackson Laboratory Tank every year with a service contract. Every ten years the tank is painted. The Great Hill Standpipe is no longer inspected and remains empty at this point in time. The 500,000-gallon concrete reservoir is only visually inspected by the Town.

7.5 POTENTIAL STORAGE TANK IMPROVEMENTS

Back in 2005, the Town conducted another Water System Master Plan report which suggested that a new water tank be placed at Dreamwood Hill in the vicinity of the previous Steel tank near the Mount Desert Island Biological Laboratory. The Town owns a small parcel of land near this location. The past study provided that there had been discussions over the years about installing a 500,000-gallon tank at this location.

In addition, the Town has recently considered installation of a water tank in the elevated Hamilton Hill subdivision. Refer to Section 9 which evaluates the recommended additional sizing for water storage in Bar Harbor. We evaluated different locations for a new water tank and improvements to existing as listed below:

- Jackson Laboratory Tank - During high demand periods, the Jackson Laboratory tank can drop down to as low as 15 feet in elevation. This further reduces the available water storage by 250,000 gallons. In order to provide more water storage for the daytime for Jackson Laboratory, we evaluated raising the tank by about ten feet providing 167,000 gallons of additional volume. Refer to Section 11 for the results of this evaluation. This would make the maximum water surface of the Jackson Lab tank the same elevation as the rest of the system.

- Paradise Hill/Duck Brook Road – This is the location of a prior open top reservoir that was abandoned years ago. In order to locate a tank at this location, there should be a separate line into Town to duplicate the existing 20-inch cast iron line leading to Bloomfield Road. This would require about 2,000 LF of 16-inch diameter transmission line. It may be a possible location for the future but was not directly evaluated in this Master Plan. We recommend placing a tank on the northern end of the system prior to this location. In the future, this may be an optimal location to consider for additional volume to assist with vulnerabilities in the downtown area.

- Ireson Hill – The distribution system in the Up-Island area near Ireson Hill has only one line feeding approximately 147 seasonal and 143 year-round user accounts. There is not any storage in this region that spans from Eden Street North of the Ferry Terminal, including Salisbury Cove and Hulls Cove. If there was a line break anywhere along the 8-inch diameter Eden Street line, the users would have no access to water. This location was evaluated extensively in Section 11. The elevation of this Town owned property is optimal at 220 feet and it is of sufficient acreage (1.72 acres) to use as a proposed new storage tank location. It is also at the Up-Island end of the system along Route 3 where there is not any current storage, so would be an appropriate site for a new tank. We are recommending a new tank at this location.

- Dreamwood Hill – We have decided that this area is not as desirable of a location as the Ireson Hill location so for the purposes of the Master Plan we did not utilize this as a suggested site. The Dreamwood Hill site has a lower elevation (195 feet) and the size of the property is not sufficient for a tank volume that would be beneficial to the Town. The property is only 0.34 acres where we would suggest a minimum of 0.50 acres.

- Birch Bay off of Crooked Hill – This location is at the Birch Hill Retirement community. This location is at a minimum elevation of 215 which would be adequate for a new water tank. For the purpose of our evaluation, we stayed within the property of Birch Bay development. Higher elevations are available behind the site owned by Acadia National Park. A disadvantage of this location is that it is not owned by the Town. In addition, there would be a significant portion of transmission piping needed compared to if the tank was located at Ireson Hill. In addition, the tank at Birch Bay did not help to resolve pressure issues along Route 3 near Salisbury Cove. Therefore, we did not suggest that the tank be located at this location.

- Hamilton Hill – We evaluated the location of a new water storage tank at Hamilton Hill. This site has an elevation of approximately 230 feet and is of sufficient size for a storage tank. However, based on our evaluation, we have not recommended this site for storage to serve the Town. The location of the tank would be beneficial for the subdivision, especially for hydrant flows. While the model indicates that there would be an improvement in pressure along Kebo Street with the Hamilton Hill water tank, the resulting pressures at the end of Kebo Ridge Development are still simulated to be too low. Our evaluation determined that replacing the water line in Kebo Street was more beneficial than placement of a tank at this location.

8.0 DESCRIPTION OF WATER DISTRIBUTION SYSTEM

The Town has an extensive distribution system with vulnerabilities due to its layout, lack of storage, and age. Appendix C contains updated maps showing the extent of the distribution system.

8.1 WATER MAINS/TRANSMISSION MAINS

The Town's largest challenge to maintaining water quality is maintenance of its distribution system. Even if the quality is acceptable when it leaves the Duck Brook Pump Station, as the water moves through the distribution system, the water quality can begin to degrade. In addition, the Town's system has large pressure variations due to topography and high-volume users. The following distribution system issues can contribute to water quality degradation in Bar Harbor:

- Iron piping corrosion such as galvanized steel or unlined cast iron releasing iron and sediments.
- Potential contamination from leaky pipe joints or breaks; and,
- Inadequately sized small piping in some locations contributing to low pressure and pressure challenges during hydrant flushing.

Distribution system flushing is an important tool to keep the water system clean and free of sediment, remove stagnant water, and remove unwanted contaminates from the system. In addition, the water system has been previously expanded and updated over time without conducting a review of the entire system with regard for pressure and flow demand. Because of the complexity of the Town's water system, future upgrades should involve a system analysis using the modeling process as described in Section 11.

8.1.1 WATER MAIN SIZE INVENTORY

The Town's water service area consists of approximately 189,529 linear feet or 35.9 miles of water piping ranging in size from 1-inch up to 24-inches. The water distribution system includes 189,529 LF of distribution and transmission piping which either supplies water to one of the storage tanks or to the Town's customers from the Duck Brook Pump Station. As shown in Table 15, much of the water main (approximately 52.83 percent) within the water service area is smaller or equal to 6-inches in diameter and 47.17 is larger than 6-inches in diameter.

TABLE 15: WATER MAIN TYPE AND SIZE INVENTORY

Diameter (in)	Line Type	Annual/Seasonal	Length (LF)	% of Total	Length (LF)	% of Total
1	Distribution	Seasonal	1,441	0.76%	1,441	0.76%
2	Distribution	Annual	10,211	5.39%	16,175	8.53%
	Distribution	Seasonal	5,964	3.15%		
3	Distribution	Annual	13,107	6.92%	27,042	14.27%
	Distribution	Seasonal	13,935	7.35%		
4	Distribution	Annual	6,771	3.57%	15,101	7.97%
	Distribution	Seasonal	8,330	4.40%		
6	Distribution	Annual	40,362	21.30%	40,362	21.30%
8	Distribution	Annual	35,762	18.87%	35,762	18.87%
10	Distribution	Annual	21,008	11.08%	22,360	11.80%
	Transmission[1]	Annual	1,352	0.71%		
12	Distribution	Annual	4,899	2.58%	7,031	3.71%
	Transmission[2]	Seasonal	2,132	1.12%		
14	Transmission[2]	Annual	2,457	1.30%	2,457	1.30%
16	Distribution	Annual	4,013	2.12%	10,549	5.57%
	Transmission[2]	Annual	5,402	2.85%		
	Transmission[3]	Annual	1,134	0.60%		
20	Transmission[2]	Annual	10,052	5.30%	10,052	5.30%
24	Transmission[2]	Annual	1,197	0.63%	1,197	0.63%
		Total	189,529	100%	189,529	100%

1 - Transmission line is from Jackson Laboratory Tank.

2 - Transmission line to/from Duck Brook Pump Station.

3 - Transmission line to/from concrete reservoir at Duck Brook.

8.1.2 WATER MAIN MATERIAL INVENTORY

As shown in Table 16, the Town still has approximately 41.07 percent of unlined cast iron (CI), 11.19 percent lined CI water main, and only 8.09 percent galvanized steel (GS) piping. The Town has been replacing portions of this piping over the years and has an ongoing program to replace water mains that have a history of breaks or leaks. Approximately 11.02 percent of the existing distribution system is ductile iron (DI) with 25.69 percent high density polyethylene (HDPE). The system has two locations with transite or asbestos cement (AC) pipe which represents only 0.76 percent of the piping. The remainder of the Town's piping is varying types of plastic pipe (PVC) at 2.19 percent. All new water mains are either ductile iron, HDPE or PVC.

TABLE 16: WATER MAIN MATERIAL INVENTORY

Material	Line Type	Annual/Seasonal	Length (LF)	% of Total	Length (LF)	% of Total
Cast Iron	Distribution	Annual	57,590	30.39%	77,842	41.07%
	Transmission[1]	Annual	1,352	0.71%		
	Transmission[2]	Annual	18,824	9.93%		
	Transmission[2]	Seasonal	76	0.04%		
Lined Cast Iron	Distribution	Annual	21,202	11.19%	21,202	11.19%
Ductile Iron	Distribution	Annual	19,442	10.26%	20,892	11.02%
	Distribution	Seasonal	44	0.02%		
	Transmission[2]	Seasonal	272	0.14%		
	Transmission[3]	Annual	1,134	0.60%		
Galvanized	Distribution	Annual	12,502	6.60%	15,327	8.09%
	Distribution	Seasonal	2,825	1.49%		
HDPE (SDR 11)	Distribution	Annual	22,758	12.01%	48,682	25.69%
	Distribution	Seasonal	23,856	12.59%		
	Transmission[2]	Annual	284	0.15%		
	Transmission[2]	Seasonal	1,784	0.94%		
Transite	Distribution	Annual	1,438	0.76%	1,438	0.76%
SDR35 Plastic	Distribution	Annual	1,054	0.56%	1,054	0.56%
160# Plastic	Distribution	Seasonal	2,945	1.55%	2,945	1.55%
CTS Plastic	Distribution	Annual	147	0.08%	147	0.08%
		Total	189,529	100%	189,529	100%

1 - Transmission line is from Jackson Laboratory Tank.

2 - Transmission line to/from Duck Brook Pump Station.

3 - Transmission line to/from concrete reservoir at Duck Brook.

8.1.3 SEASONAL VERSUS YEAR-ROUND PIPING

The Town of Bar Harbor has an extensive seasonal population; therefore, a significant portion of the Town's water system (distribution/transmission) mains are seasonal. Of the total 189,529 LF of water main, 31,802 LF or 16.78 percent is seasonal. The remaining 157,727 LF or 83.22 percent is considered year-round or annual piping. Table 17 shows the relationship between the year-round and seasonal piping.

TABLE 17: SEASONAL VERSUS YEAR-ROUND PIPING

Annual/Seasonal	Line Type	Length (LF)	% of Total	Length (LF)	% of Total
Annual	Distribution	136,133	71.83%	157,727	83.22%
	Transmission[1]	1,352	0.71%		
	Transmission[2]	19,108	10.08%		
	Transmission[3]	1,134	0.60%		
Seasonal	Distribution	29,670	15.65%	31,802	16.78%
	Transmission[2]	2,132	1.12%		
		189,529	100%	189,529	100%

1 - Transmission line is from Jackson Laboratory Tank.

2 - Transmission line to/from Duck Brook Pump Station.

3 - Transmission line to/from concrete reservoir at Duck Brook.

8.1.4 WATER SYSTEM INVENTORY AND YEAR OF INSTALLATION

Table 18 over the next two pages provides a detailed summary of the overall water system. This Table includes data on the estimated year of installation. These dates are only an estimation since as-built plans for the entire system were not available for this study. About 42.36 LF or 22.35 percent of the piping was installed prior to the 1900's-based on the data provided. This older piping is either galvanized (8.09 percent) or unlined or lined cast iron (52.26 percent). To the best of our knowledge, about 8,523 LF of the piping installed prior to the 1900's is lined cast iron (11.19 percent) with the remainder 77,842 LF or 41.07 percent unlined.

We would recommend removing all the AC piping and GS piping from the water system and work towards upsizing the smaller 6-inch cast iron piping to 8-inch piping or sizing based on specific water model analysis prior to the specific design.

A priority list of pipe replacement projects for planning purposes is included in Section 12 with preliminary order-of-magnitude cost tables provided in Appendix A.

TABLE 18: OVERALL PIPE INVENTORY

Material	Diameter (in)	Line Type	Annual/ Seasonal	Year of Installation	Length (LF) Based on Year	Length (LF) Based on Diameter	Length (LF) Material Total
Cast Iron	4	Distribution	Annual	1970	357	701	77,842
				1897	344		
	6	Distribution	Annual	1957 - 1970	2,688	30,143	
				1920	15,329		
				1893 - 1897	12,126		
	8	Distribution	Annual	1970	3,595	8,189	
				1920	1,668		
				1893 - 1897	2,926		
	10	Distribution	Annual	1974	1,724	13,920	
				1920	5,375		
				1884 - 1897	5,469		
		Transmission - Jackson Laboratory Tank	Annual	1968	1,352		
	12	Distribution	Annual	1970	284	3,929	
				1897	3,569		
		Transmission	Seasonal	1884	76		
	14	Transmission - Duck Brook	Annual	1897	2,457	2,457	
	16	Distribution	Annual	1920	2,136	7,538	
		Transmission - Duck Brook	Annual	1887	5,402		
	20	Transmission - Duck Brook	Annual	1920	10,052	10,052	
	24	Transmission - Duck Brook	Annual	1887	913	913	
Lined Cast Iron	8	Distribution	Annual	1920	8,145	16,668	21,202
				1897	8,523		
	10	Distribution	Annual	1968	2,658	4,534	
				1920	1,876		
Galvanized	2	Distribution	Annual	1893 - 1897	563	12,010	15,327
				1920	1,248		
				1970	7,374		
		Distribution	Seasonal	Unknown	2,825		
	3	Distribution	Annual	1920	2,054	3,317	
				1970	1,263		

Material	Diameter (in)	Line Type	Annual/ Seasonal	Year of Installation	Length (LF)		
					Based on Year	Based on Diameter	Material Total
Ductile Iron	4	Distribution	Annual	Unknown	430	474	20,892
		Distribution	Seasonal	2017	44		
	6	Distribution	Annual	1970	3,533	6,394	
				1984	1,034		
				1996 - 2008	1,827		
	8	Distribution	Annual	1975	1,454	8,597	
				1997	577		
				2017	6,566		
	10	Distribution	Annual	1988	1,098	1,098	
	12	Distribution	Annual	2008 - 2017	1,046	1,318	
		Transmission - Duck Brook	Seasonal	2017	272		
	16	Distribution	Annual	2017	1,877	3,011	
		Transmission - Inlet Concrete Tank	Annual	2001	517		
		Transmission - Outlet Concrete Tank	Annual	2001	617		
HDPE (SDR 11)	2	Distribution	Annual	Unknown	765	2,514	48,682
				2011	114		
		Distribution	Seasonal	Unknown	1,635		
	3	Distribution	Annual	Unknown	5,662	22,671	
				2000s	3,074		
		Distribution	Seasonal	Unknown	13,935		
	4	Distribution	Annual	Unknown	619	13,926	
				2000s	5,021		
		Distribution	Seasonal	Unknown	8,286		
	6	Distribution	Annual	2000s	3,825	3,825	
	8	Distribution	Annual	2000s	2,308	2,308	
	10	Distribution	Annual	2015	1,370	1,370	
	12	Transmission - Duck Brook	Seasonal	1997 - 2007	1,784	1,784	
	24	Transmission - Duck Brook	Annual	1998	284	284	
Transite	10	Distribution	Annual	1957	1,438	1,438	1,438
SDR35 Plastic	3	Distribution	Annual	Unknown	1,054	1,054	1,054
160# Plastic	1	Distribution	Seasonal	Unknown	1,441	2,945	2,945
	2			Unknown	1,504		
CTS Plastic	2	Distribution	Annual	Unknown	147	147	147
						Total	189,529

8.2 WATER SYSTEM FIRE HYDRANTS

The Town's fire hydrants are one of the most visible components of the water distribution system. Keeping them well-maintained is essential but challenging. A hydrant that does not operate when needed could result in significant property damage. All critical hydrants should be inspected regularly. If a hydrant is found to be inoperable and cannot be repaired immediately, it should be placed out of service and the Fire Department should be notified. All issues that cannot immediately be corrected should be recorded and planned for subsequent repairs. Hydrants should also undergo flow testing to provide data on distribution system flows and the its flow capabilities. This data can also be used to further calibrate the Town's updated water model. The fire hydrants that are in good working order should be used to flush the system in the Spring and in the Fall based on a unidirectional program to improve water quality.

8.2.1 BAR HARBOR'S FIRE HYDRANTS

The Town has 110 public fire hydrants that they are responsible to maintain and flush. These hydrants range in installation year from prior to 1970 up to 2017. Many of the earlier hydrants appear to be in poor condition and have leaded joints. Our overview of the hydrants did not include an investigation or inspection of all hydrants in the system, but a small subset was flowed during our work to obtain additional data to calibrate the system water model which is detailed in Section 11.

The issues that were noted during the work to flow test select system hydrants include the following:

- Forest Street, Hydrant 40 – Hydrant could only flow partially during testing due to low pressure.
- Hancock Street, Hydrant 43 – Hydrant was leaking from the lead joint in the 4-inch nozzle.
- U.S. Route 3, Hydrant 51 – Hydrant had a broken breakaway flange.
- Main Street, Hydrant 71 – The hydrant's lead joint was twisting when moving the cap.
- Main Street, Hydrant 74 – Hydrant was leaking from Lead joint in nozzle.
- First South Street, Hydrant 87 – Hydrant was leaking from 4-inch cap, leaded.
- Schooner Head Road, Hydrant 91 – Hydrant had a bad nozzle joint, would break loose.

Most of the hydrants that we saw problems with are over fifty years old and installed in the 1960's - 1970's or earlier timeframe. The Town's database appears to have defaulted to 1970 installation year, which would include about sixty-two system hydrants. These hydrants should be either replaced individually or replaced as projects are completed. From our evaluation of the condition of the hydrants, we would recommend a more rigorous hydrant replacement program to remove these older hydrants from the system.

Table 19 provides a summary of the relative year of the water system hydrants which provides a sense as to when hydrants have been replaced:

TABLE 19: NUMBER OF HYDRANTS REPLACED ANNUALLY

No. of Hydrants Replaced	Year of Replacement	Comments
15	2017	Rt. 3 Project/Eden St.
3	2010 - 2015	-
4	2005 - 2009	-
8	2000 - 2004	-
7	1995 - 1999	-
3	1990 - 1994	-
4	1985 - 1989	-
5	1980 - 1984	-
4	1975 - 1979	-
54	1970 - 1974	1970's may be much older
3	1960 - 1966	May be much older

Any hydrants that are not safe to operate should be bagged to notify the fire department and staff that they should not be utilized.

8.2.2 BAR HARBOR'S HYDRANT FLUSHING PLAN

The Town has a very well written hydrant flushing plan that details the valves that should be opened, closed, and the order of hydrants that need to be flushed. The Town flushes hydrants in the Spring and in the Fall. From our observations, the plan is acceptable but not completely followed due to limited staffing, the difficulty of flushing some of the older hydrants, difficulty of access to flow some of the hydrants due to their locations and the challenge of opening/closing the valves during this process. Table 20 provides information on each of the Water Department's hydrants including location, installation year, elevation, main size and material, model predicted flows, and locations of low pressures observed in the model while simulating the hydrant flowing. This flow data was determined using the water model developed, assuming average system demand and maximum tank levels.

The hydrants were flow tested in 2004 and in 2011. Due to the variability of the data, changes in the operation of the system since this work was done, and changes in how flow testing is recommended to be done, we would recommend a completed update on the hydrant flow testing.

TABLE 20: DETAILED HYDRANT INFORMATION

Hydrant ID	Location Description	Zone	Installation Year	Ground Elevation	Nozzle Elevation	Main Size (inches) and Material	Predicted Model Flow[3] (GPM)		Low Pressure Locations during Model Simulated Hydrant Flow		
							12" HDPE Line Active	12" HDPE Line Inactive	< 0	< 20 psi	< 35 psi
1	Albert Meadows	Downtown	1977	45.35	46.60	6 CI	835	830		A	B, C, D, E, F
2	Armory Lane	Downtown	1970[1]	66.56	67.57[2]	6 CI	1,180	1,175		A	B, C, D, E, F
3	Ash Street	Downtown	1992	32.37	33.95	6 CI	945	940		A	B, C, D, E, F
4	Atlantic Ave & Seamist Lane	Downtown	1970[1]	31.78	33.16	6 CI	790	790		A	B, C, D, E, F, G
5	Lenox Place Parking Lot	Downtown	1966	39.44	41.55	10 Lined CI	1,135	1,130		A	B, C, D, E, F
6	Cleftstone Road & Devon Road	Kebo	1970[1]	171.70	172.91	6 CI	430	425		A, B, H	C, D, E, F
7	Highbrook Road & Wonderview Road	Eden	1970[1]	151.70	153.44	16 CI	1,100	1,090		A	B, C, D, E, F
8	West Street Extension & Cleftstone Road	Kebo	1970[1]	155.30	157.17	12 CI	1,080	1,075		A	B, C, D, E
9	Eden Street & Cottage Street	Downtown	1981	46.50	48.40	16 DI	1,520	1,510		A	B, C, D, E, F
10	Cottage Street & Bridge Street	Downtown	1999	51.72	53.46	8 CI	1,435	1,420		A	B, C, D, E, F
11	Cottage Street & Holland Ave	Downtown	1970[1]	44.23	46.18	6 CI	1,390	1,380		A	B, C, D, E, F
12	Cottage Street & Federal Street	Downtown	1970[1]	52.06	53.57	10 Lined CI	1,395	1,385		A	B, C, D, E, F
13	Main Street & Cottage Street	Downtown	2004	52.61	54.24	10 Lined CI	1,450	1,435		A	B, C, D, E, F
14	Cottage Street & Roberts Ave	Downtown	2000	53.83	55.49	6 DI	1,440	1,425		A	B, C, D, E, F
15	Cottage Street & Rodick Street	Downtown	1970[1]	46.30	48.25	10 Lined CI	1,430	1,420		A	B, C, D, E, F
16	Derby Lane & Elbow Lane	Downtown	1997	42.04	44.22	8 DI	875	870		A	B, C, D, E, F, G
17	West Street Extension & Woodbury Road	Kebo	1975	125.90	127.12	8 DI	1,185	1,180		A, B	C, D, E, F
18	Devon Road	Kebo	1970[1]	141.29	143.37	6 CI	570	570	B	A, H	C, D, E
19	Eagle Lake Road & Cromwell Harbor Road	Kebo	1981	198.77	200.56	6 CI	390	390		A, E	B, C, D
20	Eagle Lake Rd & Cross St	Kebo	1970[1]	115.22	116.86	12 CI	1,210	1,200		A	B, C, D, E, F
21	Eagle Lake Road & Woodbury Road	Kebo	1997	133.57	135.23	8 DI	1,070	1,060		A, E	B, C, D, F
22	60 Eagle Lake Road	Kebo	2004	132.34	133.83	6 CI	930	930		A	B, C, D, E,
23	100 Eden Street	Eden	2017	73.94	75.48	10 CI	1,325	960	D[5]	A	B, C, D[4], E, F[5], I, J[5]
24	119 Eden Street	Eden	1970[1]	63.10	64.80	10 CI	1,365	910	D[5]	A, I[5]	B, C, D[4], E, F[5], J[5], K[5]
25	Eden Street & Highbrook Road	Eden	1970[1]	69.48	70.61	10 CI	1,285	1,040	D[5]	A	B, C, D[4], E, F, I[5]
26	Mount Desert St, Eagle Lake Rd, & Eden St	Downtown	1970[1]	82.11	83.33	12 DI	1,380	1,365		A	B, C, D, E, F
27	Eden Street & Myrtle Ave	Downtown	1970[1]	60.33	62.09	16 DI	1,485	1,470		A	B, C, D, E, F
28	West Street & Eden Street	Downtown	2017	35.21	38.21	16 DI	1,540	1,530		A	B, C, D, E, F
29	149 Eden Street	Eden	2017	58.84	60.68	8 Lined CI	1,310	870	D[5]	A, D[4], I[5]	B, C, E, F[5], J[5], K[5],
30	ABANDONED										
31	ABANDONED										
32	83 Eden Street	Eden	2017	66.04	67.72	10 CI	1,390	1,300		A, D[5]	B, C, D[4], E, F,
33	455 Eden Street	Hulls Cove	2017	-	62.81[2]	8 DI	1,100	785	D, I[5]	A, I[5], J[5]	B, C, E, I[4], K[5]
34	123 Eden Street	Eden	2017	47.99	49.48	8 Lined CI	1,480	940	D[5]	A, D[4], I[5], J[5]	B, C, E, F[5], K[5]
35	131 Eden Street	Eden	1970[1]	59.85	61.53	8 DI	1,335	880	D[5]	A, D[4], I[4]	B, C, E, F[5]
36	Eden Street & Highbrook Road & Wonderview Road	Eden	2017	-	65.44[2]	16 DI	1,475	1,460		A	B, C, D, E, F
37	Eden Street; Allied Whale	Eden	2017	57.20	59.00	10 CI	1,385	1,180	D[5]	A, I[5]	B, C, D[4], E, F
38	Kennebec Pl & Rodick St	Downtown	1970[1]	70.95	72.62	6 CI	1,365	1,350		A	B, C, D, E, F
39	Pine Street & Forest Street	Kebo	1993	133.36	136.03	6 HDPE	815	810		A	B, C, D, E, L
40	Pine Street & Forest Street	Kebo	1970[1]	140.92	142.54	6 HDPE	710	710		A	B, C, D, E, L
41	Glen Mary Rd & Park Street	Downtown	1970[1]	43.05	44.83	6 CI	1,140	1,135		A	B, C, D, E, F, M
42	Greeley Ave	Downtown	1970[1]	65.28	66.94	6 HDPE	1,125	1,120		A	B, C, D, E, F
43	Hancock Street & Reef Pt	Downtown	1970[1]	31.58	33.30	6 CI	660	660		A	B, C, D, E, N
44	Hancock Street; Mount Desert Hospital	Downtown	1970[1]	39.07	40.45	6 CI	860	860		A	B, C, D, E, F, N
45	Harbor Lane	Eden	1970[1]	33.17	34.88	6 CI	975	970		A	B, C, D, E, F[5]
46	Highbrook Road & Norman Road	Eden	1984	134.29	135.65	6 DI	880	875		A	B, C, D, E
47	Champlain Road & Highbrook Road	Eden	1970[1]	155.58	157.23	16 CI	1,030	1,025		A	B, C, D, E
48	Crooked Road & Wilcomb Lane	Hulls Cove	2006	48.23	49.91	8 HDPE	965	736	D	A, I[5], J[5],	B, C, E, I[4], J[4], K[5]
49	Route 3 & Clover Farm Road	Hulls Cove	2017	18.21	19.63	8 Lined CI	1,135	840	D, I[5]	A, I[4], J[5], K[5]	B, C, E, J[4]

TABLE 20: DETAILED HYDRANT INFORMATION

Hydrant ID	Location Description	Zone	Installation Year	Ground Elevation	Nozzle Elevation	Main Size (inches) and Material	Predicted Model Flow[3] (GPM)		Low Pressure Locations during Model Simulated Hydrant Flow		
							12" HDPE Line Active	12" HDPE Line Inactive	< 0	< 20 psi	< 35 psi
50	Eden Street & Ocean Ave (North)	Hulls Cove	1990	19.28	21.24	8 Lined CI	1,135	840	D, I[5]	A, I[4], J[5], K[5]	B, C, E, J[4]
51	Route 3 & Wildwood Way	Salisbury Cove	1970[1]	113.91	115.81	8 Lined CI	850	625	D	A, I[5]	B, C, E, I[4], J[5]
52	Route 3 & Hamor Lane	Hulls Cove	2017	32.04	33.46	8 Lined CI	1,090	810	D, I[5]	A, I[4], J[5], K[5]	B, C, E, J[4]
53	302 Route 3	Salisbury Cove	2017	165.10	167.09	8 Lined CI	655	480	D	A	B, C, E
54	Route 3 & Hutchins Lane	Salisbury Cove	2017	122.16	123.78	8 Lined CI	795	590	D	A	B, C, E, I[5]
55	Route 3 & Fire Road 314	Salisbury Cove	2017	108.17	110.17	8 Lined CI	845	630	D	A	B, C, E, I, J[5]
56	Eden Street & Ocean Ave (South)	Hulls Cove	1984	59.43	61.10	8 Lined CI	1,075	780	D	A, I[5], J[5]	B, C, E, I[4], K[4]
57	Route 3 & Fire Road 316	Salisbury Cove	2000	102.29	104.67	8 Lined CI	850	635	D	A, I[5]	B, C, E, I[4], J[5]
58	Route 3 & East Hillside Drive	Salisbury Cove	1970[1]	110.86	112.28	8 Lined CI	860	635	D	A, I[5]	B, C, E, I[4], J[5]
59	Lookout Point Road and Syndicate Road	Hulls Cove	1970[1]	66.09	68.20	8 Lined CI	975	730	D	A, I[5], J[5]	B, C, E, I[4], K
60	Route 3 & Lookout Point Road (North)	Salisbury Cove	2017	92.26	95.57	8 Lined CI	895	665	D	A	B, C, E, I
61	Kebo Street	Kebo	1970[1]	132.67	134.89	8 CI	705	700	C	A	B, D, E, O
62	Kebo Street; Quality Inn	Kebo	1970[1]	94.58	96.11	8 CI	1,045	1,035	C	A	B, D, E, F, O
63	Cromwell Harbor Road & Ledgelawn Ave	Downtown	1988	38.88	40.69	10 DI	1,505	1,495		A	B, C, D, E, F
64	36 Ledgelawn Ave	Downtown	1978	50.63	52.25	8 Lined CI	1,440	1,430		A	B, C, D, E, F
65	Ledgelawn Ave & Park Street	Downtown	2006	36.40	38.08	8 Lined CI	1,515	1,505		A	B, C, D, E, F
66	Ledgelawn Ave & Pleasant Street	Downtown	1970[1]	42.23	43.94	8 Lined CI	1,470	1,460		A	B, C, D, E, F
67	Barberry Lane & Livingston Road	Downtown	1970[1]	41.90	43.62	6 CI	695	690		A	B, C, D, E, F, P
68	Main St & Newport Drive	Downtown	1970[1]	43.42	45.54	10 CI	1,425	1,415		A	B, C, D, E, F
69	Main Street & Atlantic Ave	Downtown	1984	46.90	49.79	8 CI	1,360	1,350		A	B, C, D, E, F
70	Main Street & Cromwell Harbor Road	Downtown	2004	33.46	36.01	6 CI	1,425	1,420		A	B, C, D, E, F
71	Main Street & Oliver Street	Downtown	1970[1]	33.79	35.59	6 CI	1,265	1,260		A	B, C, D, E, F
72	Main Street & 1st South Street	Downtown	1970[1]	43.11	44.99	6 CI	1,310	1,305		A	B, C, D, E, F
73	Mount Desert St & Main St	Downtown	1970[1]	56.67	58.74	10 CI	1,390	1,380		A	B, C, D, E, F
74	Main St & Rodick Pl	Downtown	1970[1]	63.38	65.02	10 CI	1,380	1,370		A	B, C, D, E, F
75	Main Street & Schooner Head Road	Jackson Laboratory	1987	99.55	101.50	6 CI	930	925		A, F	B, C, D, E
76	Main Street & Peach Street	Jackson Laboratory	1970[1]	44.84	46.56	10 CI	1,385	1,380		A	B, C, D, E, F
77	Jackson Laboratory Parking Lot	Jackson Laboratory	1970[1]	127.02	129.08	10 CI	1,085	1,085		A	B, C, D, E, F
78	382 Main Street	Jackson Laboratory	1970[1]	39.21	40.91	6 CI	1,010	1,010		A	B, C, D, E, F
79	Mount Desert St & Amory Lane	Downtown	2008	77.91	79.93	12 DI	1,405	1,390		A	B, C, D, E, F
80	Mount Desert St & High St	Downtown	1970[1]	70.52	72.06	8 HDPE	1,365	1,355		A	B, C, D, E, F
81	Mount Desert St & Kennebec Pl	Downtown	1974	63.04	65.08	10 CI	1,385	1,375		A	B, C, D, E, F
82	Mount Desert St & Shannon Way	Downtown	1974	71.23	72.77	10 CI	1,390	1,380		A	B, C, D, E, F
83	Old Farm Road	Jackson Laboratory	1970[1]	85.05	87.87	6 CI	585	580		A, Q	B, C, D, E, F
84	Old Farm Road & Graff Road	Jackson Laboratory	1996	-	116.06[2]	6 DI	510	510		A, Q	B, C, D, E, F
85	Everard Ct	Downtown	1970[1]	54.74	56.42	6 CI	1,135	1,130		A	B, C, D, E, F
86	Sand Point Road	Salisbury Cove	2000	105.88	108.58	6 DI	790	605	D	A	B, C, E
87	School Street & 1st South Street	Downtown	1970[1]	44.66	45.91	6 CI	1,315	1,305		A	B, C, D, E, F
88	Heterodox View Ave; Baseball Field	Downtown	1960	31.43	32.91	6 CI	1,220	1,215		A	B, C, D, E, F
89	School Street & Edgewood Street	Downtown	1970[1]	35.01	37.10	6 CI	1,345	1,340		A	B, C, D, E, F
90	School Street & Newton Way	Downtown	1979	48.72	50.33	6 CI	1,250	1,245		A	B, C, D, E, F
91	Schooner Head Road & Seely Road	Jackson Laboratory	1970[1]	96.58	98.06[2]	6 CI	445	440		A, R	B, C, D, E, F
92	Eden Street & Crooked Road	Hulls Cove	2006	17.40	19.15	8 Lined CI	1,130	840	D, I[5]	A, I[4], J[5], K[5]	B, C, E, J[4]
93	Seely Road	Jackson Laboratory	1996	56.84	59.64	6 CI	420	420		A, R	B, C, D, E, F
94	Shannon Road	Downtown	2000	47.49	48.97	10 AC	1,490	1,480		A	B, C, D, E, F
95	Spring Street & Shannon Road	Downtown	1986	60.74	62.45	10 AC	1,440	1,430		A	B, C, D, E, F
96	Spring Street & Norris Ave	Downtown	1970[1]	60.19	61.67	6 CI	770	770		A	B, C, D, E, F, M
97	Ells Pier	Downtown	1999	10.28	12.14	6 DI	1,310	1,300		A	B, C, D, E, F
98	Rodick St & Rodick Pl	Downtown	1997	55.63	57.89	6 CI	1,280	1,270		A	B, C, D, E, F

TABLE 20: DETAILED HYDRANT INFORMATION

Hydrant ID	Location Description	Zone	Installation Year	Ground Elevation	Nozzle Elevation	Main Size (inches) and Material	Predicted Model Flow[3] (GPM)		Low Pressure Locations during Model Simulated Hydrant Flow		
							12" HDPE Line Active	12" HDPE Line Inactive	< 0	< 20 psi	< 35 psi
99	Waldron Road	Downtown	2003	47.57	49.50	6 DI	1,425	1,415		A	B, C, D, E, F
100	Wayman Lane & Hideaway Lane	Downtown	1960	35.51	37.66	6 CI	680	680		A	B, C, D, E, S
101	Wayman Lane; Mount Desert Island Hospital	Downtown	1970[1]	40.69	42.95	6 CI	925	920		A	B, C, D, E, F
102	West Street & Bridge Street	Downtown	1987	46.90	49.23	10 CI	1,450	1,440		A	B, C, D, E, F
103	West Street & Holland Ave	Downtown	1970[1]	38.07	39.89	10 CI	1,475	1,465		A	B, C, D, E, F
104	West Street & Main Street	Downtown	1970[1]	20.86	22.58	8 CI	1,490	1,480		A	B, C, D, E, F
105	West Street & Rodick Street	Downtown	1970[1]	20.75	22.31	8 CI	1,500	1,490		A	B, C, D, E, F
106	Ledgelawn Ave; Bus Garage	Jackson Laboratory	1970[1]	42.08	44.43	6 HDPE	1,165	1,160		A	B, C, D, E, F, T
107	White Spruce Road & Shortcake Way	Jackson Laboratory	2010	93.84	95.08	6 HDPE	770	770		A	B, C, D, E, F, T
108	Kebo Street & Harden Farm Road	Kebo Street	1970	104.12	106.54	8 CI	635	630	C	A, O	B, D, E
109	Eden Street & Bouge Chito Way	Hulls Cove	2015	70.35	72.72	8 Lined CI	1,050	560	D	A, I[5], J[5]	B, C, E, I[4], K[5]
110	Jackson Laboratory Tank	Jackson Laboratory	1970[1]	231.47	233.46	10 CI	580	580		A	B, C, D, E
111	Treatment Plant	Treatment Facility	2013	220.49	222.84	16 DI	750	750		A	B, C, D, E
112	Route 3; Near old Dreamwood Hill tank	Salisbury Cove	2017	198.60	201.17	8 DI	485	360		A, D	B, C, E

Notes:
1 – Hydrant installation date could be prior to 1970.
2 – Nozzle elevation could not be measured using GPS survey equipment. Elevation is from other source.
3 – Simulated hydrant flow is based on gravity flow from the hydrant's 2-½ inch nozzle with an assumed emitter coefficient equal to 165 gpm/psi^n. Actual flow from hydrants will depend on system demands, tank levels, and pump and valve operation status. Specifically, since the 12-inch HDPE line from Duck Brook to the Ferry Terminal is deactivated every year, two scenarios were evaluated and are shown in this table.
4 – Pressure result only occurs when the seasonal 12-inch HDPE line is inactive.
5 – Pressure result occurs when the seasonal 12-inch HDPE line is active.

Low Pressure Location Codes:
A – Hamilton Hill
B – Cleftstone Road
C – Kebo Ridge Development
D – Ireson Hill
E – Mountain Avenue Booster Station Inlet
F – East Strawberry Hill Road
G – Atlantic Avenue
H – Devon Road
I – US Route 3 North of Lookout Point Road
J – Wilcomb Lane
K – Eden Street near Bogue Chitto Lane
L – Pine Street & Forrest Street
M – End of Spring Street
N – End of Hancock Street
O – Kebo Street
P – Livingston Road
Q – Old Farm Road
R – Schooner Head Road & Seely Road
S – Wayman Lane
T – White Spruce Road

Each hydrant in the system was modeled to be flowing by gravity from the 2-1/2" nozzle with an assumed emitter coefficient of 165 gpm/psi^n. The model was also set to be at average demand conditions and full storage tank levels and all pumps inactive. Hydrant flows were also simulated with the seasonal 12-inch HDPE line from Duck Brook to the Ferry terminal as open and closed. The resulting Town-wide system pressures were then observed and specific locations were noted if pressures were less than 35-psi, 20-psi, and 0-psi, the latter of which indicating a vacuum in the system was being created. Note that actual hydrant flows will depend on local and system wide demand conditions, tank levels, and other system conditions such as local valve status and pipe conditions.

8.3 WATER SYSTEM VALVES

Water system valves are important components of a water distribution system. The average estimated life span of a water distribution valve is between 40 to 60 years but they can last longer if properly maintained, cleaned and exercised. Each section of the Town's water system should be evaluated for existing system valves and data should be recorded including condition, location, type, use, size, opening direction, and other information. All programs involving underutilized valves should consider having replacement valves in close proximity due to problems with existing valves that have not been consistently utilized in an old system.

8.3.1 BAR HARBOR'S VALVES

The Town has approximately 273 valves located throughout its water system which are shown and discretely identified on their GIS system mapping. The valves are of varying ages depending upon the year of the water lines they are associated with. The Town's data base has documentation of what they know about the valves but based on the age of many of them, there is not extensive documentation. It appears that some of the valves are operated clockwise and others counterclockwise. With future valve and water main replacement work, the Town should choose a direction of either clockwise or counterclockwise for open/close.

We would suggest fixing all known valves that are broken. During our water model study, we were informed of one valve that is believed to be broken closed. That is Valve WSV-120 on Atlantic Avenue. There are also several valves noted broken open on the water valve inventory schedule. These should also be repaired. We understand that the Town does operate many of the valves during hydrant flushing but there is not a specific valve exercise and cleaning program due to limited staffing.

8.4 WATER SYSTEM BLOW-OFFS

The water system has thirteen blow-offs that are used to maintain water quality at dead-end water main locations. Blow-offs are generally installed when water lines are not large enough to have a typical fire hydrant. The Town's flushing program includes operation of the blow-offs as part of the flushing process. Table 21 shows the location of the known blow-offs that are still utilized in the system.

TABLE 21: LOCATION OF WATER SYSTEM BLOW-OFFS

Blow-off Number	Location	Description
1	Great Meadow Drive	Above Ground
2	Atlantic Avenue	Below Ground
3	Billings Avenue	Below Ground
4	Barberry Lane	Below Ground
5	Hulls Cove WWTP	Above Ground
6	Public Works Building	Below Ground
7	Shannon Way	Below Ground
8	Forest Street	Below Ground
9	Davis Place	Below Ground
10	Kavanaugh Place	Below Ground
11	Bogue Chitto	Above Ground
12	Kebo Ridge	Below Ground
13	Kebo Ridge	Below Ground

8.5 WATER SYSTEM BLEEDERS

The Town utilizes water system bleeders to maintain water quality and to also keep shallow lines from freezing. As old distribution lines are replaced, the goal should be to remove system bleeders. The known bleeders are listed as follows:

- Old Farm Road, which is on a 6-inch diameter cast iron water main with about three customers, runs annually at or less than 1 GPM. This is operated to control water quality on a year-round basis.

- At the intersection of Norman Road and Highbrook Road, which is on a 3-4-inch diameter line, runs year-round to control water quality.

- Hydrant 96 located at Norris Avenue/Spring Street operates year-round for water quality at less than 1 GPM.

- Harbor Lane, runs year-round and is located at Hydrant 46 and runs at about 2 GPM. This bleeder is utilized for water quality.

- The only seasonal bleeder is at the Pier or Harbor Master and at Hydrant 97.

Each of the above bleeders, except at the Pier, would be eventually removed by completing distribution system projects suggested in this Master Plan. This would reduce system water losses by about 2,628,000 gallons per year.

9.0 WATER STORAGE VOLUME EVALUATION

As was previously summarized, the Town currently has a maximum possible volume of 1,000,000 gallons for water storage split between two separate 500,000-gallon tanks. One storage location is the underground reservoir/chlorine contact tank near the treatment facility at Great Hill and the other is located across from Jackson Laboratory on Route 3. Both of these tanks fluctuate in volume so that the actual storage at any one time is not at the maximum. The underground reservoir is not operated at full volume due to the gravity operation of the system and the local demands at the Jackson Laboratory tank combined with the manual operation of the gravity system results in less than the maximum volume on a daily basis.

This plan provides a preliminary analysis of the storage tank volume needed but the actual volume would depend on a final design analysis and the specific location for additional storage. Figure 18 showed locations of all existing storage tanks.

There are different methods used for determining adequate storage volume capacity. Ten States Standards indicates that the minimum storage capacity, **not including** fire protection shall be equal to the average daily consumption. This provides a total volume of 1,001,164 gallons if you utilize the volume of water produced in 2018. This volume would be need to be **effective** volume, that could actually be utilized.

The design of a storage volume or "Effective Storage" involves several different components, some of which can be omitted under certain considerations. Additionally, this evaluation did not consider the need to reduce system storage volume due to water quality concerns.

The sizing components are listed below for reference:

- Operational Storage (OS)
- Equalizing Storage (ES)
- Standby Storage (SB)
- Fire Suppression Storage (FSS)
- Dead Storage (DS), if any

For this analysis, we used a Maximum Daily Demand (MDD) of 2.07 MGD (2018 PUC Report), a Total Annual Flow of 365 MG (2018 PUC Report, includes leaks, and unknown losses) and a peak hourly flow of 2,900 GPM (2018 PUC Report). We assume that the system Equivalent Dwelling Unit (EDU) equals a number of meters of around 1,819 (2018 PUC Report). Interestingly, we calculate the peak hourly demand using the MDD and EDU assumptions to be very near the actual demand of 2,900 GPM). We utilized a treated water flow of 2,500 GPM which would feed the system during a pumped condition.

Total tank volume, as measured between the overflow and the tank outlet elevations, may not necessarily equal the effective volume available to the water system. Effective storage volume is equal to the total volume less any dead storage built into the tank or reservoir. For many storage tanks, part of a standpipe's capacity is designed as dead storage. Dead storage is the water that is below a certain surface elevation within the tank such that the pressure delivered to some customers falls below a minimum pressure requirement for the water system or falls below 20 psi.

9.1 OPERATIONAL STORAGE

Operational Storage (OS) is the volume devoted to supplying the water system while, under normal operating conditions, the sources of supply are in an off status. This volume could vary based on the sensitivity of the water level sensors controlling the pumps or the configuration of the tank designed to provide the volume required to prevent excessive cycling. In Bar Harbor's situation, the system is largely gravity fed and the source is generally on-line with the pump only used to supplement pressure during high demand periods.

OS is in addition to other storage components. When the tank or reservoir is full, this provides a safety factor beyond that provided by the other components when it is applicable to be considered. Simply stated or generalized, some standards require that this volume be about 2.5 times the capacity of what the largest pump could provide. Additionally, some designers calculate this volume to be what is needed for the level measurement sensors.

Sometimes OS is ignored when a water system operates under a continuous pumping or flowing mode. In Bar Harbor's case, the system does operate with continuous flow because the system is generally gravity feeding to maintain pressure at all times. We suggest that the OS is disregarded.

9.2 EQUALIZING STORAGE

In a system where the source pumping capacity cannot meet the periodic daily or long peak demands placed on the water system, the water system must provide Equalizing Storage (ES) as a part of the total storage volume. The ES must be available at a minimum of 20 psi to all service connections. Several considerations influence the recommended volume including peak variations in water system demand, source production capacity and the specific mode of operation.

Based on our preliminary assessment, the recommended ES volume is calculated to be around 60,000 gallons. This value is very low since the design of the Duck Brook pumps are near the peak hourly flow.

9.3 STANDBY STORAGE

Standby Storage (SS) provides a measure of reliability in case all sources fail, there is some kind of operational issue causing a plant upset, a major main break, or there are higher demands than anticipated. The SS can be different depending upon how many sources are provided and the actual number of ERU's. For the purpose of this preliminary evaluation we assumed that the ERU's were equal to the number of meters. The general design for water storage from a single source provides that the tank should have a volume of at least twice the water system's average daily demand for the design year. Just based on this general sizing, the system should have standby storage of at least 2.0 million gallons.

Standby volumes are intended to satisfy the requirements imposed by a water system for usual situations and the use of two days has been general practice. There are some design methods permitted that utilize only one day of storage which would be 1.0 million gallons. Depending on which method is utilized, the additional standby volume could be as low as 1.0 million gallons to as high as 2.0 million gallons.

9.4 FIRE SUPPRESSION STORAGE

Fire Suppression Storage (FSS) needs to either be satisfied by the standby storage volume or needs to be an additional volume in addition to the amount calculated for SS. In other words, water systems can exclude the SB or FSS component, whichever is smaller, from a water system's total storage requirement unless such practice is prohibited by local ordinance or the local fire protection authority. A typical value used for FSS would be 1,500 GPM for 2.5 hours of time. This would require about 225,000 gallons of water to achieve this rate. Preliminarily, based on the 1.0 to 2.0 million gallons of SS discussed above, an additional volume for FSS of 225,000 gallons would not be needed.

9.5 DEAD STORAGE VOLUME

Dead Storage (DS) volume is the water that exists below the minimum design pressure of 20 PSI. This volume would vary depending on the elevation of the tank, reservoir or the location of the tank or reservoir. The dead storage for any new tank would need to be determined during design. This has been evaluated preliminarily for both the Jackson Laboratory tank and for the underground storage tank at Duck Brook.

9.5.1 JACKSON LABORATORY

Based on evaluating the distribution system at different elevations utilizing average and peak flow conditions, we have observed that at around 15 feet of water elevation, the pressures reduce significantly in this end of the system. The most significant area of concern in this location is East Strawberry Hill which does not have a booster station. This area has the potential to drop below 20 PSI. Fifteen feet is the elevation which the Town already uses as a minimum level to start the finish water pump at Duck Brook. The Jackson Laboratory tank is around 30 feet tall with a capacity of 500,000 gallons; therefore, the tank has a dead storage volume of about 250,000 gallons when at an elevation of 15 feet.

9.5.2 DUCK BROOK RESERVOIR/CC TANK

Based on evaluating the Duck Brook facility, the only Dead Storage would be the CC tank which needs to be at the volume of 60,000 gallons for proper CT.

9.6 TOTAL SUGGESTED EFFECTIVE STORAGE VOLUME

- Operational Storage (OS) – Omit due to gravity feed system
- Equalizing Storage (ES) – 60,000 gallons
- Standby Storage (SB) – 2.0 million gallons
- Fire Suppression Storage (FSS) – Included in SB
- Dead Storage would need to be computed for new tank during design.

The Existing Dead Storage (DS) - (310,000 gallons)

MAXIMUM SUGGESTED TOTAL EFFECTIVE STORAGE VOLUME, GALLONS

60,000 + 2,000,000 = 2,060,000

USEABLE VOLUME IF ALL CURRENT TANKS ARE FULL

1,000,000 – 310,000 = 690,000

TOTAL SUGGESTED ADDED VOLUME, MAXIMUM

2,060,000 – 690,000 = 1,370,000 added gallons

The total effective storage volume excludes the dead storage (310,000 gallons) which would also need to be determined during final design of any new tank. The maximum suggested added volume would be around 1,370,000 gallons. We would suggest that new storage be added over time with first installing one new storage tank and with augmentation of the Jackson Laboratory tank.

10.0 WATER SYSTEM REGULATORY CONSIDERATIONS

10.1 INTRODUCTION

The Bar Harbor Water Department is a public water system that serves approximately 1,850 connections and a population of about 4,625 customers. The Water Department is regulated under the Safe Drinking Water Act (SDWA) as a community water system. The following sections describe specific SDWA rules, how each applies to Bar Harbor and what the Water Department does to remain in compliance.

10.2 TOTAL COLIFORM RULE

10.2.1 PURPOSE

The purpose of the Total Coliform Rule (TCR) is to increase public health protection through routine monitoring of bacteria in the distribution system. Whenever there is a confirmed positive sample, an investigation with follow-up corrective activities is required. This reduces the potential pathways of entry for fecal contamination into the distribution system.

10.2.2 GENERAL REQUIREMENTS

The TCR and its revision require public water systems to collect routine bacteria samples at designated locations in the distribution system. The frequency of sampling and number of samples are determined by the number of people served. These samples are analyzed for total coliform bacteria and if present, also either fecal coliform bacteria or *E. coli*. Samples absent of coliform bacteria help confirm that the water in the piping and storage tanks is also absent of fecal pathogens.

Should any initial sample test positive for total coliform, in addition for testing for *E. coli*, three recheck samples must be collected: one at the original location, one within five service connections upstream and one within five service connections downstream. Whenever any two samples test positive for total coliform, Bar Harbor must perform a sanitary assessment to identify the cause. Any deficiencies identified in the assessment must be corrected. Failing to do so may result in a treatment technique (TT) violation.

Whenever a sample tests positive for fecal coliform or *E. coli*, this is a maximum contaminant level (MCL) violation. The utility must contact the DWP within 24 hours for additional guidance. In addition to performing a sanitary assessment to attempt to identify and then correct the cause, follow-up activities may also include issuing a boil water order.

10.2.3 BAR HARBOR WATER DEPARTMENT SPECIFIC INFORMATION

The Town is required to collect five bacteria samples per month at locations documented in its approved sampling plan. The designated sample locations are spread through the distribution system to ensure the water that all customers consume is being monitored. Any changes to the sampling plan must first be approved by the DWP. A certified lab analyzes the samples for total coliform bacteria, and if present, must also analyze each positive sample for either fecal coliform bacteria or *E. coli*.

The Town disinfects the raw water from Eagle Lake using UV radiation then free chlorine. Both of these treatment processes inactivate harmful microorganisms. After adequate disinfection is achieved, ammonium sulfate is added to the water to convert hypochlorous acid and hypochlorite ions (free chlorine) into chloramines (monochloramine). This allows for maintaining a disinfection residual throughout the distribution system while minimizing the potential for disinfection byproduct formation.

10.2.4 REPORTING REQUIREMENTS

All laboratories certified to analyze bacteria samples for Maine public water systems automatically submit the results to the DWP. It is recommended that the Town verify the samples have been reported by checking the DWP's website before the end of each monitoring period. Because of this, it is recommended to collect samples early in the monitoring period.

The daily production and chemical usage monthly operating report form includes a section at the bottom to report the number of routine TCR samples collected in that monitoring period, the number of repeat samples and the number of positive samples. The average chlorine residual taken at the time and location of the TCR samples is also recorded here.

10.3 RADIONUCLIDES RULE

10.3.1 PURPOSE

The purpose of the Radionuclides Rule is to reduce the risk of exposure to radionuclides in potable water, which will reduce the risk of certain cancers. This is accomplished through routine monitoring and establishing MCLs.

10.3.2 GENERAL REQUIREMENTS

The Radionuclides Rule requires public water systems to monitor for specific naturally-occurring radioactive elements and particles at the entry point to the distribution system. Sampling results are compared against MCLs and the Maine

Exposure Guideline for radon. Monitoring frequencies depend on the concentration of each contaminant in the water and whether treatment is present. Depending on specific results, the frequency can range from quarterly sampling to one sample every nine years.

The MCLs for each regulated radionuclide are:

Maximum Contaminant Levels (MCLs)
Net Alpha Radiation: 15 pCi/L
Combined Radium: 5 pCi/L
Uranium: 30 µg/L
Beta/photon Emitters: 4 millirems/year

Maine Maximum Exposure Guideline (MEG)
Radon: 4,000 pCi/L

10.3.3 BAR HARBOR WATER DEPARTMENT SPECIFIC INFORMATION

These radionuclides have either not been detected or were detected at low levels in the Town's water. As a result, the DWP requires the Town to monitor only once every nine years for net alpha particles, radium, uranium and radon.

The Town does not provide any treatment to remove radionuclides due to their already low levels in the Eagle Lake source water.

10.3.4 REPORTING REQUIREMENTS

The laboratories analyzing the radionuclide samples submit these results to the DWP. The Town may want to verify the DWP has received each result prior to the end of the monitoring period.

10.4 LEAD AND COPPER RULE

10.4.1 PURPOSE

The purpose of the Lead and Copper Rule (LCR) is to minimize lead and copper levels in the drinking water. This is primarily accomplished by reducing the corrosivity of the water and/or adding a corrosion inhibitor. Exposure to lead can cause damage to the brain, red blood cells and kidneys especially for young children and infants. Copper exposure can cause stomach and intestinal distress as well as kidney and liver damage. People with Wilson's disease may experience complications as a result of consuming water with elevated copper levels.

10.4.2 GENERAL REQUIREMENTS

Community water systems are required to collect lead and copper samples either every six months, annually during the summer or every three years during the summer. This depends on the results of previous monitoring or if treatment changes were made. The number of samples ranges between five and one hundred, depending on the number of people served, as well as, whether a system is required to be on six month or summer monitoring.

Lead and copper element concentrations are typically absent in source waters. These metals dissolve into the water from water distribution mains, customer piping and plumbing fixtures that contain these materials. The longer the same water is in contact with these plumbing materials, the higher the concentrations of lead and copper will be. Due to this, compliance samples are required to be taken directly from customer fixtures. Sample locations are identified through a tier designation that attempts to identify the highest risk sites. The philosophy behind this is that because not every faucet used for human consumption can reasonably be tested, some of the worst-case fixtures are selected. If these locations test below the Action Levels (AL), then it is assumed that lower risk sites will also test below.

Each sample kit is left with a customer, who is instructed to collect a "first draw" sample from a tap normally used for consumption. This is typically a kitchen or bathroom faucet. The water is left to sit for at least six hours unused before the sample is collected. Only the cold water is to be tested. This method replicates normal use from a household fixture and tests the interaction of the utility water with the plumbing materials used by the customers.

The sampling results are compared against the AL. For lead, the AL is 0.015 mg/L and for copper the AL is 1.3 mg/L. Provided at least ninety percent of the samples do not exceed these values, the system is considered to be in compliance. The only follow-up activity is to provide each participating customer with their individual lead sample result along with lead heath information.

For systems that exceed either AL with greater than ten percent of the samples, several follow-up activities are required. The exceedance itself is not a violation but failing to conduct any of the follow-up activities within the prescribed timeframes would be. These activities include returning to standard six-month monitoring if on summer monitoring, collecting entry point and distribution system water quality parameters, developing and implementing a lead education program (for lead AL exceedance only), conducting source water lead and copper monitoring, identifying and replacing any existing lead service lines and developing a corrosion control treatment plan based on existing water quality. The recommendations in the plan must be implemented if future AL exceedances occur.

An additional provision of the LCR is that any proposed changes to treatment must be reviewed and approved by the DWP prior to implementing. This is to help ensure that treatment changes do not create corrosive water, which could lead to excessive levels of lead or copper.

10.4.3 BAR HARBOR WATER DEPARTMENT SPECIFIC INFORMATION

The Town is required to collect twenty lead and copper samples every three years. The samples are to be collected from June 1 through September 30 of the sampling year. This reduced monitoring is the result of multiple rounds of testing below the ALs.

The Town adds lime and carbon dioxide to adjust the carbonate-bicarbonate alkalinity while maintaining a finished water pH of approximately 9.5 standard units. Both of these chemicals are added for corrosion control, which helps reduce lead and copper dissolving from the customers' plumbing into the water. They are also used to optimize the water chemistry for the chloramine disinfectant.

10.4.4 REPORTING REQUIREMENTS

The certified laboratory submits the lead and copper sampling results to the DWP. Each water system must also complete the 141-A form certifying the samples were collected properly (first draw). This is then submitted to the DWP.

Each public water system is required to provide each customer that samples with their individual lead result. This notice includes standard public health language about lead and how to reduce exposure. Following distribution, each water system sends a certification to the DWP indicating that the lead customer notification requirement has been met.

10.5 DISINFECTANTS AND DISINFECTION BYPRODUCTS RULE

10.5.1 PURPOSE

The purpose of the Disinfectants and Disinfection Byproducts Rule (D/DBP) is to improve public health protection by reducing exposure to disinfection byproducts. Disinfection byproducts are suspected to cause bladder cancer and cause negative reproductive effects.

Disinfection byproducts are formed when a chemical disinfectant such as chlorine is used in a water that contains certain organic compounds such as humics and tannins. These disinfection byproduct precursors are the result of decaying organic matter such as leaves, wood and other plant debris.

10.5.2 GENERAL REQUIREMENETS

For community water systems that use a chemical disinfectant, the D/DBP Rule requires routine monitoring of total trihalomethanes (TTHMs) and haloacetic acids (HAA5) as well as disinfectant residual in the distribution system. TTHMs include four compounds: chloroform, bromodichloromethane, dibromochloromethane and bromoform. HAA5 are five compounds: monochloroacetic acid, dichloroacetic acid, trichloroacetic acid, monobromoacetic acid and dibromoacetic acid. Compliance with TTHMs and HAA5 is based on a locational running annual average of the samples. The MCL for TTHMs is 80 μg/L and for HAA5 is 60 μg/L.

Each unfiltered surface water system is required to collect at least one TTHM and one HAA5 sample each quarter at a designated location in the distribution system. The sampling locations were determined through previous monitoring aimed at identifying the highest levels of each contaminant within the distribution system. Because these two families of contaminants form under different conditions, the sampling locations may also be different. Samples must be collected in a specific month each quarter to help ensure representative monitoring is occurring.

Systems must also analyze and record the disinfectant residual at the time and location of when TCR samples are collected. The maximum residual disinfectant level (MRDL) for chlorine is 4 mg/L.

10.5.3 BAR HARBOR WATER DEPARTMENT SPECIFIC INFORMATION

The Town collects two TTHM sample and two HAA5 sample per quarter. The TTHM samples are collected at Jackson Laboratory with the HAA5 samples being collected at the Paradis True Value hardware store. These locations were selected based on previous monitoring results as well through the Initial Distribution System Evaluation. The Town is required to collect these samples during the middle of the second month of each quarter.

In 2018, the TTHM results ranged from 27 to 44 μg/L with a year-end running annual average of 34 μg/L. The HAA5 results ranged from 19 to 33 μg/L with a year-end running annual average of 26 μg/L. Both are below half the respective MCLs.

The Town adds sodium hypochlorite to inactivate microbial contaminants such as viruses and *Giardia*. The chlorine is injected just prior to the 60,000-gallon contact tank. After adequate disinfection has occurred, ammonium sulfate is added to convert the free chlorine to chloramines (monochloramine). This conversion allows for maintaining a chlorine residual in the distribution system while reducing the potential for TTHM and HAA5 formation. Unlike with filtered surface water plants, the Duck Brook Pump Station has no treatment methods to remove disinfection byproduct precursors. Source water protection and effective use of chloramines are the primary means for controlling DBPs. Should issues arise in the future, optimizing storage turnover such as tank mixers as well as targeted flushing could be implemented.

10.5.4 REPORTING REQUIREMENTS

The laboratory analyzing the DBP samples submits the TTHM and HAA5 results to the DWP. In addition, the Town must submit its quarterly DBP results and includes its chlorine residual MRDL data.

10.6 SURFACE WATER TREATMENT RULE

10.6.1 PURPOSE

The purpose of the Surface Water Treatment Rule (SWTR) and its amendments is to improve public health protection through the control of microbial contaminants. To accomplish this, all public water systems using surface water sources as well as groundwater sources under the direct influence of surface water must implement a multiple barrier approach that includes source protection, disinfection, routine monitoring and may also include filtration. The microbial contaminants of concern include viruses, *Giardia* and *Cryptosporidium* that can cause illness and even lead to death. It is imperative that all requirements are met continuously to ensure public health protection.

10.6.2 GENERAL REQUIREMENTS

All unfiltered surface water systems must disinfect to inactivate viruses, *Giardia* and *Cryptosporidium*. Disinfection requirements include inactivating at least 99.99% of viruses, 99.9% of *Giardia* (unfiltered systems) and either 99% or 99.9% of *Cryptosporidium*, depending on source water characterization results. A minimum of two disinfectants must be used to meet these requirements. Each contaminant must have all of its inactivation requirements met by a single disinfectant. In addition, a disinfectant residual of at least 0.2 mg/L must be present at the entry point into the system and should be detectable throughout the distribution system. The distribution residual is monitored at the time and location of TCR sampling.

Each unfiltered surface water system must have an active Watershed Control Program to minimize source water contamination. An annual watershed and source protection inspection reviews the protection efforts.

All surface water systems, including unfiltered ones, must conduct routine source water monitoring to confirm treatment requirements for *Cryptosporidium* are appropriate based on monitoring results. Additional *Cryptosporidium* inactivation may be necessary should the results show higher concentrations of oocysts present.

A disinfection profile must have been previously conducted unless waived by the DWP and a benchmark must be calculated each time there are changes to any disinfection process.

To maintain its filtration avoidance waiver, all unfiltered surface systems also meet the following additional requirements:

A. Routine coliform density monitoring must be conducted prior to the first disinfection application location. The fecal coliform density must be no greater than 20 colony forming units (CFU) per 100 milliliters or the total coliform density of no greater than 100 CFU per 100 milliliters. Sampling must occur between once and five times per week, depending on system size, as well as whenever the source water turbidity exceeds 1 nephelometric turbidity units (NTU).

B. Turbidity must be monitored prior to the first disinfection stage no less often than once every four hours. The levels cannot exceed 5 NTU.

C. The total inactivation must be calculated daily. The system must achieve at least 3 log *Giardia* and 4 log virus inactivation daily except any one day each month in eleven of the twelve previous months. This determination also requires daily pH, temperature, peak hourly flow and disinfectant residual measurements. The system must also achieve at least 2 log inactivation of *Cryptosporidium*. A minimum of two disinfectants must be used. Each target pathogen must have its minimum inactivation achieved by a single disinfectant. The same disinfectant can be used to achieve minimum inactivation for multiple pathogens such as chlorine for viruses and *Giardia*.

D. Each unfiltered system must maintain at least 0.2 mg/L disinfectant residual at the entry point into the distribution system continuously with an allowance for less than this for no greater than four hours.

E. The system must meet all D/DBP Rule requirements continuously.

F. The treatment facility must have redundant disinfection that can be placed online or have an automatic shutoff whenever the entry point disinfection residual falls below 0.2 mg/L.

G. Each system must have an active watershed control program to minimize potential for contamination by *Giardia* and *Cryptosporidium*.

H. Each unfiltered surface water source system must prepare an annual source water report and submit this to the DWP by October 10 of each year. The DWP reviews the report and conducts an annual watershed inspection.

I. A sanitary survey must be conducted by the DWP no less frequently than once every three years. Any sanitary deficiencies identified during the survey must be addressed within a prescribed time.

J. No sources for an unfiltered surface water system can have been identified as a source of a waterborne disease outbreak.

K. The disinfectant residual cannot be undetectable in greater than five percent of the TCR monthly samples for any two consecutive months.

10.6.3 BAR HARBOR WATER DEPARTMENT SPECIFIC INFORMATION

As Eagle Lake water enters the Duck Brook Pump Station, the flow is measured using a magnetic flow meter. This flow meter is located after the raw water booster pumps that are either bypassed with raw water flowing by gravity or used to pump raw water from Eagle Lake through the UV treatment system to the underground storage tank. Following the flow meter, several raw water parameters are measured continuously including the UV transmittance, pH, temperature and turbidity. Whether gravity flow or pumped, the water is then disinfected through one of the two parallel Aquionics UV reactors. This treatment inactivates at least 99% of any *Cryptosporidium* oocysts that are present in the source water.

Sodium hypochlorite is injected to inactivate *Giardia* and viruses. This disinfected water flows into a 60,000-gallon contact chamber that is located inside the 500,000-gallon underground storage tank. After adequate inactivation has occurred, the water flows through a pipe into the underground storage tank. The free chlorine residual and pH are measured as the water enters the contact tank and again as it leaves using continuous online analyzers.

Ammonium sulfate is added into the pipe connecting the contact chamber with the underground storage tank. This chemical converts the chlorine into chloramines, which reduces the formation of disinfection byproducts but allows for maintaining a disinfectant residual throughout the system. The finished water pH and total chlorine are monitored using a continuous online analyzer.

The Town meets the additional unfiltered surface water system requirements identified in the previous section with the following details:

A. The Town completes raw water coliform sampling as required by the unfiltered surface water waiver. This is done three times per week in the summer and twice each week in the wintertime. The results of this testing are less than 5 CFU's.

B. In 2018, the highest raw water turbidity reading was 1.06 NTU, which is below the 5 NTU trigger.

C. The BHWD uses UV to achieve its *Cryptosporidium* inactivation requirements and free chlorine for *Giardia* and viruses.

D. The BHWD maintains at least a 0.2 mg/L total chlorine residual at the entry point into the distribution system.

E. In 2018, the running annual average for TTHMs was 34 µg/L (MCL = 80 µg/L) and for HAA5 was 24 µg/L (MCL = 60 µg/L). Each is below half its respective MCL.

F. The Duck Brook Pump Station has redundant chlorine injection pumps and continuously monitors free chlorine residual flowing into and out of the contact chamber as well as entering the distribution system. The low level alarm set points are much higher than 0.2 mg/L, so the plant staff can respond well before the regulatory limit would ever be reached.

G. Eagle Lake is a protected source with an active watershed control program. See Section 3.0 for details.

H. Each year, the BHWD submits a source protection summary report to the DWP who conducts a follow-up inspection.

I. The most recent sanitary survey was conducted by the DWP in April, 2019.

J. Eagle Lake has not been identified as a source of a waterborne disease outbreak.

K. A chlorine residual has been present in all TCR bacteria samples collected.

10.6.4 REPORTING REQUIREMENTS

Each unfiltered system must report monthly operational data to the DWP by the tenth day of the following month. Information specific to the SWTR is reported on two separate forms. Other operational data is also reported to the DWP at the end of each month.

In addition to monthly reporting, all unfiltered surface water systems must report to the DWP within twenty-four hours whenever the turbidity exceeds 5 NTU or should there be a waterborne disease outbreak.

Should the entry point chlorine residual drop below 0.2 mg/L, the Town must notify the DWP as soon as possible but no later than the end of the next business day.

10.7 ARSENIC RULE

10.7.1 PURPOSE

The purpose of the Arsenic Rule is to improve public health by reducing exposure of arsenic through drinking water.

Arsenic is a naturally occurring element that when ingested at higher concentrations over a long period of time can lead to bladder and lung cancers.

10.7.2 GENERAL REQUIREMENTS

All community water systems are required to regularly test for arsenic at the point of entry to the distribution system. The maximum contaminant level for arsenic is 0.010 mg/L. Surface water systems must monitor for arsenic between quarterly and annually or once every three years if it has received a waiver based on previous low results.

10.7.3 BAR HARBOR WATER DEPARTMENT SPECIFIC INFORMATION

Eagle Lake has below detectable levels of arsenic in the water. As a result, the Duck Brook Pump Station does not have arsenic removal treatment. Arsenic is analyzed with other annually required metals included in the same test method so the Town is monitoring for this contaminant annually. The 2019 compliance sample collected at the entry point to the distribution system was again below detectable levels.

10.7.4 REPORTING REQUIREMENTS

The laboratory analyzing the arsenic sample submits the result to the DWP. The Town may want to verify the DWP has received the results prior to the end of each annual monitoring period.

10.8 PHASE II/V RULE

10.8.1 PURPOSE

The purpose of the Phase II/V Rule is to protect public health by reducing exposure to inorganic and organic contaminants. Exposure to these chemicals above certain levels can result in cancer, organ damage, circulatory system disorders, nervous system disorders and reproductive system disorders. Contaminants that create aesthetic problems if present at certain levels in the water but have no known adverse health effects such as iron are also monitored under this rule. These contaminants have secondary standards that are not enforceable.

10.8.2 GENERAL REQUIREMENTS

The Phase II/V Rule also known as the Chemical Contaminants Rule applies to approximately one hundred inorganic elements and compounds, volatile organic compounds and semi-volatile organic compounds. Community water systems must monitor for these at frequencies ranging from quarterly to every three years with an opportunity to receive a waiver for the semi-volatile (synthetic) compounds if the water system can document that they are not used in the watershed. Arsenic and fluoride are discussed in separate sections.

10.8.3 BAR HARBOR WATER DEPARTMENT SPECIFIC INFORMATION

The Town uses a protected surface water source with little human and no industrial activity occurring within the watershed. The Duck Brook Pump Station does not have treatment for any of these contaminants because they have either not been detected in sampling or found at concentrations far below the regulatory limits. The only Phase II/V contaminant detected from 2016 through 2018 was barium at 0.002 mg/L. The MCL for barium is 2 mg/L. The maximum contaminant level goal for barium is also 2 mg/L.

The following list is the Phase II/V contaminant monitoring schedules:

- Asbestos: one sample per nine years in the distribution system.
- Cyanide: one sample per year at the entry point to the distribution system.
- Inorganic compounds: one sample per year at the entry point to the distribution system.
- Nitrate: one sample per year at the entry point to the distribution system.
- Nitrite: one sample per nine years at the entry point to the distribution system.
- Volatile organic compounds: one sample per three years at the entry point to the distribution system.
- Semi-volatile organic compounds: one sample per three years per family of contaminants (there are four families). In 2017, the Town applied for and received a three-year waiver from monitoring for these contaminants.

10.8.4 REPORTING REQUIREMENTS

The laboratories analyzing these samples submit the results to the DWP. The Town may want to verify the DWP has received the results prior to the end of each monitoring period.

In 2020, the Town will need to apply and receive a new waiver for the semi-volatile organic compounds or conduct the testing.

10.9 FLUORIDE

10.9.1 PURPOSE

The purpose of regulating fluoride in non-fluoridated utilities is to prevent exposure to water that may result in bone disease and mottling of children's teeth. For water systems that add fluoride for the purpose of dental protection, regulation helps ensure the appropriate amount is present in the water.

10.9.2 GENERAL REQUIREMENTS

All public water systems must regularly test for fluoride either through the Phase II/V Rule discussed above or as described in this section if it actively fluoridates. The sixty-five water systems in Maine that fluoridate must follow the State of Maine Rules Related to Drinking Water requirements for this chemical. The optimal fluoride range is 0.5 mg/L to 1.2 mg/L with a target concentration of 0.7 mg/L.

Each water system that fluoridates must monitor the daily fluoride concentration using an acceptable analytical method. Theoretical calculations are encouraged for quality control and can be used as a substitute for up to ten days per month.

Water systems that fluoridate must also collect a monthly distribution sample.

10.9.3 BAR HARBOR WATER DEPARTMENT SPECIFIC INFORMATION

Sodium fluoride is added at the Duck Brook Pump Station for the purposes of dental protection. It is injected as a saturated solution after all other treatment just prior to the distribution system. An online fluoride analyzer is present that monitors the concentration. In 2018, the average fluoride concentration in the water was 0.73 mg/L.

10.9.4 REPORTING REQUIREMENTS

The Town is required to report daily fluoride results to the DWP by the tenth of the following month.

The laboratory analyzing the monthly distribution sample submits this to the DWP.

10.10 CONSUMER CONFIDENCE REPORT RULE

10.10.1 PURPOSE

The purpose of the Consumer Confidence Report (CCR) Rule is to provide the customers with water quality data from the previous year so that they can make health decisions about the water they consume.

10.10.2 GENERAL REQUIREMENTS

Every community water system is required to prepare and distribute a CCR. The CCR includes water quality data from the previous year, federal limits for reported contaminants, system information, health effects language and any violations that were incurred. The deadline for distribution is July 1 of the year following the year for the report. A copy of the report along with a distribution certification are required to be submitted to the DWP by October 1 or within ninety days, whichever comes first.

10.10.3 BAR HARBOR WATER DEPARTMENT SPECIFIC INFORMATION

The Town publishes and distributes its annual CCR to its customers. Copies of the CCR are also available at the Public Works Department building and on the Town's website.

10.10.4 REPORTING REQUIREMENTS

Following distribution of the CCR, the Town must submit a copy to the DWP along with a certification form by October 1 or within ninety days, whichever comes first.

10.11 UNREGULATED CONTAMINANTS

10.11.1 GENERAL INFORMATION

There are many compounds found in drinking water that are not regulated. They may be safe, extremely rare or have unknown health effects. Periodically, the EPA investigates regulating additional compounds. This is conducted through the Unregulated Contaminant Monitoring Rule (UCMR). Under the UCMR, select water systems collect samples for compounds that are not regulated. The EPA then determines whether a drinking water standard is necessary for each based on their occurrence and health risk factors.

10.11.2 PER- & POLYFLUOROALKYL SUBSTANCES

The family of contaminants that is currently being considered for regulation are per- and polyfluoroalkyl substances (PFAS). These PFAS are man-made chemicals that have been manufactured since the 1940s. PFAS uses have included food packaging, household products such as for stain prevention and non-stick cookware. They have also been used in firefighting foams. Their presence in drinking water is typically associated with nearby manufacture, use or disposal. There are no known PFAS sources within the Eagle Lake watershed.

10.11.3 CYANOTOXINS

A family of contaminants included in the current 2017 through 2021 UCMR cycle that may be a concern are cyanotoxins. Blue-green algae or cyanobacteria is commonly found in freshwater bodies. Some species within this family can release cyanotoxins either during their life cycle or when cells rupture. This becomes a concern during algal blooms when a large amount cyanobacteria is present.

11.0 WATER DISTRIBUTION SYSTEM MODEL

11.1 SYSTEM MODELING BACKGROUND

To assist in the evaluation of the Bar Harbor water system, a water distribution model was developed using the Bentley WaterGEMS CONNECT Edition 2 Version 10 modeling program. Computer models are formed using elements that represent the various components within a water system, including reservoirs, tanks, pipes, nodes, hydrants, pumps, and valves. Attributes and system characteristics gathered from available system maps, plans, reports, and survey are applied to each system element to create a fully representative model that can be used to simulate situational conditions of various flow, tank elevation, and pressure. These simulations may be evaluated for use in making planning, operational, and design decisions.

11.2 WATER MODEL NODES

System demands are applied to nodes to simulate water leaving the system at a specific location in the network, such as from the Town's residential, commercial, or industrial users. Historic usage from the Town's user account database was analyzed and base user demands were applied to the model with nodes in the approximate locations of the users in the network. Hydrants are a specific type of node in which demands may be applied to simulate high hydrant flows. Elevation data obtained from survey and topographical maps is applied to both nodes and hydrants to allow for the model to calculate pressure based on the model simulated hydraulic grade. Tanks and reservoirs are considered to be storage nodes and are used in the model to simulate the source of the system's water supply. The main attribute for tanks and reservoirs is the water surface elevation and, in the case of Bar Harbor, the maximum hydraulic grade for the majority of the system. All elements of the Town's water system are incorporated into the water model including the Eagle Lake reservoir, Duck Brook and Jackson Laboratory storage tanks, water system valves, and the various pumps throughout the system.

The pipe segments created by nodes link all elements within the system network. These have multiple attributes that may be inputted including pipe length, size, material, age, minor losses, and Hazen-Williams roughness coefficient. The Hazen-Williams roughness coefficient, or C-value, is a measurement of friction that occurs within a flowing pipe and is based on pipe condition and material. A high C-value is associated with pipes of low friction such as smooth HDPE or PVC, while a low C-value is associated with pipes of high friction such as old, tuberculated cast iron. High friction pipes result in greater headloss and are often the bottlenecks in any water system; however, bottlenecks could also be caused by any combination of pipe material, small pipe size, and lengthy pipe runs.

11.3 MODEL CALIBRATION

Once a representative model is created, it must be evaluated and calibrated such that the simulated model flow and pressure results match that of observed water system data obtained from field testing and system records. Calibration of models is completed by adjusting specific system characteristics such as the Hazen-Williams roughness coefficient in pipe segments, system demand applied at the nodes, and element status such as the open or closed orientation of a valve. These characteristics tend to be relatively unknown and may need to be assumed in any system. Automatic computer model calibration allows for multiple combinations of the varying characteristics to be attempted using systematic algorithms such that the best combination for the system can be found. For automatic calibration to occur, field data obtained from field testing including known hydrant flows, approximate system demands, system pressures, and boundary conditions such as tank elevations and pump status must be input into the model program. The program then automatically adjusts the roughness coefficient, system demand, and/or valve status in an attempt to find the system characteristics that produce simulated results that provide the best fit with the observed field data.

Once model calibration is accepted, the model can be manipulated to simulate various system conditions, both existing and hypothetical, including the introduction of new storage tanks at various hydraulic grades, heavy user demands in specific areas of the system, impacts of hydrant flows, etc. The following are specific modeled scenarios that were completed:

- Evaluation of Hamilton Hill storage tank.
- Evaluation of Up-Island storage tank.
- New pump station at the Town's Ferry Terminal for filling of the Up-Island storage tank.
- Addition of height to the existing Jackson Laboratory Tank.
- Replacement of 4,277 LF of 8-inch diameter cast iron pipe on Kebo Road with 12-inch diameter ductile iron pipe.
- Addition of 740 LF of 16" ductile iron pipe on Eagle Lake Road between Prospect Avenue and Cross Street connecting the 6" and 12"Ø cast iron lines on Eagle Lake Road.
- Replacement of 6-inch diameter cast iron pipe on Main Street to 12" Ø ductile iron pipe.
- Replacement and looping of 6-inch cast iron line on Devon Street and Cleftstone Road with 8-inch ductile iron.
- Looping on Holland Avenue between Cottage Street and West Street.
- Replacement of various small lines within the system.

11.4 BAR HARBOR MODEL STATUS AND ACCURACY

The accuracy of any model is dependent on the accuracy of the data used to develop it. Therefore, it should be noted that the model and its results should be taken as it is: a model that will always have room for improvement. System data inputs to the model program are as accurate to the extent possible using the references available including water system maps, plans, historical reports, and survey. The model developed for the Bar Harbor water system is as up to date as possible, and although efforts were made to keep these minimal, there are still many assumptions and potential sources of error. The many possible sources of error include: (1) specific system characteristics such as pipe size, material, length, age, fittings, elevations, location of nodes and system connectivity, etc.; (2) operational information including status of valves in the field, condition of pipe which will affect the roughness coefficients and consequently the headloss across the system, flow demand data such as locations of individual user demands and locations of leaks that may be unknown; and (3) testing data including errors due to the accuracy of the instruments, quality of the data recorded, and unknown or assumed operational conditions of the system during field testing and data gathering. Further development of a system model will provide increased accuracy as unknowns or assumptions are tested and/or discovered. This can be done through an iterative process of re-testing and re-calibrating the model with additional in-depth system testing and evaluation during varying system conditions.

11.5 SPECIFIC BAR HARBOR MODEL ISSUES

11.5.1 FLOW TESTING NEEDS FOR FUTURE MODEL CALIBRATION

Due to system limitations such as hydrant conditions, hydrant locations, and limited system flows and pressures, some areas of the distribution system were not able to be flow tested, such as the southeast section of the system near the Jackson Laboratory tank where the condition of many hydrants were such that they were not able to be flowed or tested. These areas do not have a representative test to use during model calibration and verification. Therefore, it was assumed that the C-values of similarly sized and aged pipes of the same material were of similar condition, and therefore would have similar roughness coefficients. However, there is the possibility that pipes of the same material, size, and age are not of the same condition, which could be determined through system testing.

11.5.2 UP-ISLAND UNCALIBRATED MODEL ISSUES

Additionally, the uncalibrated model produced results indicating that the hydrant flows observed during field testing in the Up-Island portion of the distribution system near Hulls Cove and Salisbury Cove were not possible and that hydrant flow should be much lower than what is experienced in this region. Typically, the uncalibrated model should provide reasonable results such that adjustments to the C-values and system demands are reasonable; however, when model simulated results are drastically different this indicates that data inputs in the model, such as pipe diameter, pipe length, hydrant elevation, or system connectivity, are incorrect. After extensive examination of the model and available system maps, plans, and site survey, the model inputs are accurate to the best of our knowledge. As discussed above, it is possible there are system unknowns that are not in the system records that limit the accuracy of the model in this section. To remediate this issue, the model calibration has adjusted the C-values of this region such that the model data matches that of the observed data, however please note that the model should not be taken as exact fact but as a representation of the system.

Therefore, as mentioned previously, further ongoing testing and system evaluation should be completed to improve the accuracy of the model and its inputs into the future.

11.6 WATER MODEL SCENARIO ANALYSIS

The developed water model for the Town's water distribution system was used to evaluate impacts of multiple scenarios in the system including the inclusion of new tanks, a new pump station at the Town's Ferry Terminal, replacement of system bottlenecks, and more. Each scenario considered was evaluated at varying system conditions including average versus peak demands, maximum versus minimum tank levels, and the consideration of high flow demands including hydrants and specific large users. The general data inputs used for analysis are included in Table 22. A few specific items should be noted:

- The maximum water surface elevation of the concrete tank at the Duck Brook Treatment Plant is based on a recent surveyed elevation. While the overflow elevation of the concrete tank is 276.5 feet to match the maximum elevation of the Eagle Lake reservoir, survey indicates that the actual water surface of the reservoir during the 2019 hydrant testing was 275 feet. Therefore, this was considered the maximum elevation of the concrete tank during modeled simulations.

- The maximum water surface elevation of the Jackson Laboratory Tank is the overflow elevation of 265 feet.

- The minimum water surface elevation of all existing tanks is based on the minimum observed water surface elevation during the time of the 2019 hydrant testing. There is a possibility the tanks would drop below this water surface elevation; however, this would depend on system demand and operation.

- Peak system demand is based on a peaking factor of 3.5.

- To analyze and compare the system scenarios during conditions of additional flow demands from hydrants, the model demands are based on the flow observed from each hydrant during the 2019 hydrant testing. During this time, the system demand conditions were slightly above average. To provide a baseline, the observed hydrant flow was applied to scenarios during both average and peak demand conditions unless stated otherwise. In reality, the actual hydrant flow during peak demand conditions would not be equal to that of the observed flow during hydrant testing and would likely be lower due to the reduction in system pressure from additional headloss. Therefore, when reviewing results from the scenarios including additional hydrant flows during peak demand, it is possible that pressures are lower than actual. In these situations, recognize that actual flows will likely be less than the model unless water is being pumped from the system at these hydrants.

TABLE 22: VARIABLE DATA INPUTS FOR MODEL ANALYSIS

Basis of Existing Tanks

Water Surface (Level)	Concrete Tank Inlet	Concrete Tank Outlet	Jackson Lab Tank
Minimum (Observed)	EL 272.95 (6.95 ft)	EL 272.95 (6.95 ft)	EL 261.00 (26 ft)
Maximum	EL 275.00 (9 ft)	EL 275.00 (9 ft)	EL 265.00 (30 ft)

System Demand

Average	725 GPM	
Peak	2,550 GPM	Peaking factor of 3.5

Specific Hydrant or Significant Additional Demands

Hydrant ID	Zone	Observed Flow (2019 Testing)
H-108	Kebo	700 GPM
H-22	Kebo	540 GPM
Golf Course	Kebo	600 GPM
H-16	Downtown	825 GPM
H-104	Downtown	1,300 GPM
H-35	Up-Island (Ferry Terminal)	1,225 GPM
H-92	Up-Island (Hulls Cove)	1,015 GPM
H-60	Up-Island (Salisbury Cove)	850 GPM

- Seasonal 12-inch HDPE line is active during model simulations unless stated otherwise.
- The existing Summer Pump is not active during model simulations unless stated otherwise.
- The system was simulated as a gravity system as it is typically run.

11.7 WATER MODEL RESULTS

11.7.1 ANALYSIS OF ADDITION OF STORAGE TANK AT HAMILTON HILL

One of the Bar Harbor water distribution system limitations is the total storage volume available. While storage volume is important for backup water supply, fire flows, and peak instantaneous system demands, a water tank can also provide improved system pressures and, therefore, improved water flow capacity of specific regions depending on the location and resulting working water surface elevations of the tank.

One of the higher locations in the distribution system area is Hamilton Hill located between Eagle Lake Road and Kebo Street with a peak elevation of around 237 feet. In 2018, the 40-acre parcel of land on the Hill owned previously by the Juliano family was purchased by developers with the intent of developing the area into a subdivision. An evaluation of the subdivision water line was completed for the developer using the water model. In general, the high elevation of the proposed development lots as well as the relative distance from both existing storage tanks results in relatively low waterline pressures for a majority of the sixteen (16) proposed subdivision lots, especially during peak demand and additional hydrant fire flow conditions. The model evaluation determined that the subdivision would greatly benefit from a storage tank located directly on Hamilton Hill, which would allow for reduced impacts in the system during peak demand and fire flow conditions as well as improved local hydrant flows.

Meanwhile, the Town's existing system around Hamilton Hill including Eagle Lake Road and more specifically Kebo Street experiences high headloss during peak demand and fire flow conditions due to the older, small, and tuberculated cast iron waterlines. The Water Department's efforts to loop the system on the west side of Hamilton Hill on Woodbury Road to Eagle Lake Road has reduced the overall pressure impacts of peak demands and fire flow conditions on this side of the Hill. Conversely, Kebo Street contains approximately 4,220 LF of 8-inch cast iron line with no looping back into the system. The Kebo Valley Golf Club pulls water from the system approximately 3,300 LF down Kebo Street and irrigates its course during the nighttime hours of the summer months when the Town's user demand is at its highest. The facility has a pump with a maximum capacity of 600 GPM which has been recently upgraded to be controlled with a variable frequency drive. The high demand through the high friction cast iron line results in significant head losses which create pressure issues for users along this line.

The addition of a storage tank on Hamilton Hill was evaluated to determine the benefits for the Town in this area. The evaluated tank was modeled to have a maximum hydraulic grade equal to that of the concrete Duck Brook Tank with an overflow at 276.5 feet, a tank height of 60 feet, and a separate 16-inch diameter ductile iron inlet and outlet connections to the system. Table 23 summarizes tank model inputs used for the evaluation and Table 24 provides model pressure results during the various system conditions.

As shown in Table 24, the waterlines along Eagle Lake Road improve by 8-psi maximum while more significant pressure improvements occur on Kebo Street, especially during periods of additional demands in that area. One important item to note is the impacts of high flow on the Kebo Ridge Development, which was constructed around 2015. The peak waterline elevation in this development is around 207 feet near the end of Kebo Ridge Road. Here, the pressure is maintained above the standard 20 psi minimum during all conditions without additional demands in the system. However, even during best case conditions of average system demand and maximum tank levels, the model indicates that the waterline in this area with the existing system begins to pull a vacuum when additional high flows are applied to the end of Kebo Street. Therefore, if the golf course is pumping near maximum capacity, it is likely low pressures are resulting in this area of the Development. It appears there are currently three (3) existing residences on Kebo Ridge Road that are at risk of pressures 20 psi or worse. It is recommended that Water Department establish a Limited Service Contract with the existing residences and future residences to be built in this area.

TABLE 23: HAMLTON HILL TANK INPUTS FOR MODEL ANALYSIS

Hamilton Hill Tank Basis	
Minimum Water Surface Level	EL 272.95 (56.45 ft)
Maximum Water Surface Level	EL 275 (58.50 ft)
Ground Elevation at Lot 21	EL 230
Tank Height	60 ft
Base Elevation	EL 216.50
Maximum Overall Elevation	EL 276.50
Separate inlet & outlet with check valves	
16” diameter ductile iron, C = 130	
Tank fills from bottom	

As expected, the greatest improvements on the system are in the immediate area of the tank around Hamilton Hill, particularly when there are high additional demands in this area. However, improvements were also seen throughout the system as shown in Table 25, which focuses mainly on locations within the Town with high elevations and low system pressures. The highest service location in the distribution system is at the old Juliano residence on Hamilton Hill. As seen in Table 25, under no circumstance does the addition of the tank on Hamilton Hill allow for the pressure of the residence to be above the 20-psi minimum; however, the tank does mitigate the pressure drop during additional hydrant demands in the system. This residence will likely require a local booster pump if one is not already existing. Meanwhile, it is recommended that the Water Department negotiate a Limited Service Contract with this and any other lot that may be developed in the future Hamilton Hill Development.

Other locations such as near the end of the 2" Ø GS Cleftstone Road line and Hydrant #6 on the corner of Cleftstone Road and Devon Street experiences the greatest improvement of 8 psi maximum during high additional flows from the Downtown areas during peak demands. This is also true for the location of Hydrant #104 and the corner of Eden Street, Eagle Lake Road, and Mount Desert Street. The areas near the North and South ends of the distribution system near Salisbury Cove and Jackson Laboratory, specifically the end of East Strawberry Hill and the top of Ireson Hill, experience lower improvements compared to areas close to the proposed tank. One item to note is the negative values simulated at the top of Ireson Hill. The model indicates that when hydrants are flowing in the Salisbury Cove line, downstream lines at higher elevations tend to pull a vacuum. The proposed Hamilton Hill tank does not resolve this issue.

TABLE 24: HAMILTON HILL TANK SCENARIO
LOCAL SYSTEM PRESURE (PSI) COMPARISON

		Location	Hydrant #19 Eagle Lake Road and Cromwell Harbor Road			Hydrant #20 Eagle Lake Road and Cross Street			Hydrant #62 Kebo Street			Hydrant #61 Kebo Street			Kebo Ridge Development Kebo Ridge Road			Kebo Golf Club Kebo Street			Hydrant #108 Kebo Street		
		Waterline Elevation	194			110.1			89.9			127.25			207			95.85			99.45		
		Scenario	Existing	New Tank	Δ	Existing	New Tank	Δ	Existing	New Tank	Δ	Existing	New Tank	Δ	Existing	New Tank	Δ	Existing	New Tank	Δ	Existing	New Tank	Δ
Maximum Tank Level	Average Demand 725 GPM	No Additional Demand	34	35	1	71	71	0	79	80	1	63	64	1	28	29	1	77	78	1	75	76	1
		Hydrant #22 at 540 GPM	30	32	2	69	71	2	78	80	2	61	64	3	27	29	2	75	77	2	74	76	2
		Hydrant #108 at 700 GPM	33	35	2	68	71	3	63	80	17	25	45	20	-6	14	20	21	41	20	8	28	20
		Golf Course at 600 GPM	33	35	2	69	71	2	67	80	13	34	49	15	3	18	15	34	50	16	33	48	15
	Peak Demand 2,550 GPM	No Additional Demand	31	35	4	67	71	4	75	80	5	58	64	6	24	29	5	72	77	5	70	76	6
		Hydrant #22 at 540 GPM	25	32	7	63	71	8	72	79	7	56	64	8	21	29	8	69	77	8	68	76	8
		Hydrant #108 at 700 GPM	28	34	6	63	71	8	56	79	23	17	44	27	-13	14	27	13	40	27	0	27	27
		Golf Course at 600 GPM	29	34	5	63	71	8	60	79	19	27	49	22	-4	18	22	27	49	22	26	48	22
Minimum Tank Level	Average Demand 725 GPM	No Additional Demand	33	34	1	70	70	0	78	79	1	62	63	1	27	29	2	76	77	1	74	75	1
		Hydrant #22 at 540 GPM	29	32	3	68	70	2	77	79	2	60	63	3	26	28	2	74	77	3	72	75	3
		Hydrant #108 at 700 GPM	32	34	2	67	70	3	62	79	17	24	44	20	-7	13	20	20	40	20	7	27	20
		Golf Course at 600 GPM	32	34	2	68	70	2	66	79	13	33	49	16	1	17	16	33	49	16	32	47	15
	Peak Demand 2,550 GPM	No Additional Demand	30	34	4	66	70	4	74	79	5	57	63	6	23	28	5	71	77	6	69	75	6
		Hydrant #22 at 540 GPM	24	31	7	62	70	8	71	79	8	55	63	8	20	28	8	68	76	8	67	75	8
		Hydrant #108 at 700 GPM	27	34	7	62	70	8	55	78	23	16	43	27	-14	13	27	12	39	27	-1	26	27
		Golf Course at 600 GPM	28	34	6	62	70	8	59	78	19	26	48	22	-5	17	22	26	49	23	25	47	22

TABLE 25: HAMILTON HILL TANK SCENARIO
TOWN-WIDE SYSTEM PRESSURE (PSI) COMPARISON

		Location	Juliano Residence Hamilton Hill			Cleftstone Road			Hydrant #6 Cleftstone Road and Devon Road			Hydrant #104 Ells Pier			Corner of Eden Street, Eagle Lake Road & Mount Desert Street			East Strawberry Hill Jackson Laboratory Area			Top of Ireson Hill Route 3		
		Waterline Elevation	231.9			211.45			166.2			15.4			78.75			182			207.25		
		Scenario	Existing	New Tank	Δ	Existing	New Tank	Δ	Existing	New Tank	Δ	Existing	New Tank	Δ	Existing	New Tank	Δ	Existing	New Tank	Δ	Existing	New Tank	Δ
Maximum Tank Level	Average Demand 725 GPM	No Additional Demand	18	19	1	27	27	0	46	47	1	111	111	0	84	85	1	36	36	0	29	29	0
		Hydrant #16 at 825 GPM	16	19	3	25	27	2	45	47	2	106	108	2	81	83	2	35	36	1	28	29	1
		Hydrant #104 at 1,300 GPM	14	19	5	24	27	3	43	47	4	96	98	2	79	82	3	33	35	2	27	28	1
		Hydrant #60 at 850 GPM	16	19	3	26	27	1	45	47	2	110	111	1	83	84	1	36	36	0	-13	-12	1
	Peak Demand 2,550 GPM	No Additional Demand	14	19	5	24	27	3	43	47	4	104	108	4	79	83	4	29	30	1	25	27	2
		Hydrant #16 at 825 GPM	10	18	8	20	26	6	40	46	6	96	101	5	74	81	7	24	28	4	23	26	3
		Hydrant #104 at 1,300 GPM	7	18	11	18	26	8	38	46	8	82	90	8	71	79	8	21	26	5	22	26	4
		Hydrant #60 at 850 GPM	11	18	7	21	26	5	41	46	5	102	107	5	77	82	5	27	30	3	-25	-22	3
Minimum Tank Level	Average Demand 725 GPM	No Additional Demand	17	18	1	26	27	1	45	46	1	110	111	1	83	84	1	35	35	0	28	28	0
		Hydrant #16 at 825 GPM	15	18	3	24	26	2	44	46	2	105	107	2	80	82	2	33	34	1	27	28	1
		Hydrant #104 at 1,300 GPM	13	18	5	23	26	3	42	46	4	95	97	2	78	81	3	32	33	1	26	28	2
		Hydrant #60 at 850 GPM	15	18	3	25	26	1	44	46	2	109	110	1	82	83	1	34	35	1	-14	-13	1
	Peak Demand 2,550 GPM	No Additional Demand	13	18	5	23	26	3	42	46	4	103	107	4	78	82	4	27	29	2	24	26	2
		Hydrant #16 at 825 GPM	9	18	9	19	26	7	39	45	6	94	100	6	73	80	7	23	26	3	22	25	3
		Hydrant #104 at 1,300 GPM	6	17	11	17	25	8	37	45	8	81	89	8	70	78	8	20	25	5	21	25	4
		Hydrant #60 at 850 GPM	10	18	8	20	26	6	40	45	5	101	106	5	76	81	5	26	28	2	-26	-23	3

11.7.2 ANALYSIS OF IMPACTS OF UPGRADING EXISTING 8" Ø CAST IRON MAIN ON KEBO STREET

While the model indicates that there would be an improvement in pressure along Kebo Street with the addition of a Hamilton Hill water tank, the resulting pressures at the end of the Kebo Ridge Development during additional flow demands are not simulated to be above 20 psi (See Table 24). Additionally, since the old 8-inch cast iron line on Kebo Street is the cause for the major headloss in this portion of the system, a potential cost-effective alternative to the proposed tank addition on Hamilton Hill would be to replace the existing 4,277-LF line altogether with a larger, lower friction pipe.

Table 26 provides results of model simulations with 10-inch and 12-inch diameter ductile iron lines on Kebo Street. While there is no improvement on system pressures with only typical system demands, the model indicates that a larger diameter ductile iron line provides between 9 and 63-psi improvements on the Kebo Street waterline when additional demands are applied. Larger improvements can be seen further along the waterline where compounding headloss of the existing 8-inch cast iron would have the most effect. The model simulations also indicate that the end of the Kebo Ridge Development is able to be maintained above 20-psi during average demand conditions; however, this location will still drop below 20-psi during peak demand conditions. In this scenario, it should be noted that the line size increase and the subsequent reduction in friction will also likely result in an increased flow capacity of the Kebo Street hydrants. Therefore, the flow used in the simulation of Hydrant #108 is likely low, and the actual pressures with this hydrant flowing will be lower than simulated. Meanwhile, the maximum flow from the golf course is accurate and will not change unless the pump is upsized. Due to the combination of the peak demand condition results and the likelihood that the hydrant flow will be greater than the original field observed 700 GPM, Limited Service Contracts are still recommended for the Kebo Ridge Development. Compared to the Hamilton Hill tank scenario, a Kebo Street waterline upgrade shows a greater improvement to Kebo Street pressures during additional system demands conditions.

Comparing the two different line size scenarios, the larger 12-inch line provides a greater improvement to the area pressures as expected; however, the significance of the improvement between the 10-inch to 12-inch diameter pipe is relatively small. There are approximately seventeen (17) user accounts that are supplied from the Kebo Street distribution line including the three (3) existing accounts on Kebo Ridge. Six (6) of these accounts are considered seasonal. Based on historic user account data, it is estimated that the peak demand on this line excluding the golf course flow is around 25 GPM. Therefore, aside from the seasonal flushing of this line from the golf course, it is possible a larger line could risk stagnation of the water during the winter season. Meanwhile, it is also possible there could be some further development of this area requiring the larger line.

TABLE 26: KEBO STREET UPGRADE (10" and 12" DI) SCENARIO

LOCAL SYSTEM PRESSURE (PSI) COMPARISON

Tank Level	Demand	Location	Hydrant #62 Kebo Street					Hydrant #61 Kebo Street					Kebo Ridge Development Kebo Ridge Road					Kebo Golf Club Kebo Street					Hydrant #108 Kebo Street				
		Waterline Elevation	89.9					127.25					207					95.85					99.45				
		Scenario	Existing	10" DI	Δ	12" DI	Δ	Existing	10" DI	Δ	12" DI	Δ	Existing	10" DI	Δ	12" DI	Δ	Existing	10" DI	Δ	12" DI	Δ	Existing	10" DI	Δ	12" DI	Δ
Maximum Tank Level	Average Demand 725 GPM	No Additional Demand	79	79	0	79	0	63	63	0	63	0	28	28	0	28	0	77	77	0	77	0	75	75	0	75	0
		Hydrant #108 at 700 GPM	63	76	13	76	13	25	57	32	59	34	-6	23	29	25	31	21	69	48	72	51	8	66	58	70	62
		Golf Course at 600 GPM	67	76	9	77	10	34	58	24	60	26	3	24	21	25	22	34	71	37	73	39	33	69	36	71	38
	Peak Demand 2,550 GPM	No Additional Demand	75	75	0	75	0	58	58	0	58	0	24	24	0	24	0	72	72	0	72	0	70	70	0	70	0
		Hydrant #108 at 700 GPM	56	69	13	69	13	17	50	33	52	35	-13	16	29	18	31	13	62	49	65	52	0	59	59	63	63
		Golf Course at 600 GPM	60	70	10	70	10	27	52	25	53	26	-4	18	22	19	23	27	64	37	66	39	26	63	37	65	39
Minimum Tank Level	Average Demand 725 GPM	No Additional Demand	78	78	0	78	0	62	62	0	62	0	27	27	0	27	0	76	76	0	76	0	74	74	0	74	0
		Hydrant #108 at 700 GPM	62	74	12	75	13	24	56	32	58	34	-7	22	29	24	31	20	68	48	71	51	7	65	58	69	62
		Golf Course at 600 GPM	66	75	9	76	10	33	57	24	59	26	1	23	22	24	23	33	70	37	72	39	32	68	36	70	38
	Peak Demand 2,550 GPM	No Additional Demand	74	74	0	74	0	57	57	0	57	0	23	23	0	23	0	71	71	0	71	0	69	69	0	69	0
		Hydrant #108 at 700 GPM	55	68	13	68	13	16	49	33	51	35	-14	15	29	17	31	12	61	49	64	52	-1	58	59	62	63
		Golf Course at 600 GPM	59	69	10	69	10	26	51	25	52	26	-5	17	22	18	23	26	63	37	65	39	25	61	36	64	39

11.7.3 EVALUATE LOOPING ON EAGLE LAKE ROAD (BETWEEN PROSPECT AVENUE AND CROSS STREET

The Town has done well with looping various parts of the distribution system, which helps reduce headloss and improve system pressures during high demand periods, as well as prevent water stagnation on dead-end lines. A potential location for increased looping that has been proposed by the Water Department is on Eagle Lake Road between Prospect Avenue and Cross Street. A model simulation of this was performed involving 740-LF of 16-inch ductile iron pipe to connect the existing 6-inch cast iron line on Prospect Avenue and Eagle Lake Road to the existing 12-inch cast iron line on Cross Street and Eagle Lake Road. As shown in Table 27 and Table 28, looping of the system in this location does not provide any improvements to pressures in the local Kebo area including the problem area of Kebo Street, the area near Arata Drive and Mountain Avenue, and in many of the worst-case pressure locations observed in the model simulations. While there is little improvement in terms of system pressures with the addition of a 16-inch diameter line on Eagle Lake Road, the looping would improve water quality and system reliability and could still be considered.

TABLE 27: EAGLE LAKE ROAD LOOP SCENARIO
LOCAL SYSTEM PRESSURE (PSI) COMPARISON

		Location	Hydrant #19 Eagle Lake Road and Cromwell Harbor Road			Hydrant #20 Eagle Lake Road and Cross Street			Hydrant #62 Kebo Street			Hydrant #61 Kebo Street			Kebo Ridge Development Kebo Ridge Road			Kebo Golf Club Kebo Street			Hydrant #108 Kebo Street		
		Waterline Elevation	194			110.1			89.9			127.25			207			95.85			99.45		
		Scenario	Existing	Loop	Δ	Existing	Loop	Δ	Existing	Loop	Δ	Existing	Loop	Δ	Existing	Loop	Δ	Existing	Loop	Δ	Existing	Loop	Δ
Maximum Tank Level	Average Demand 725 GPM	No Additional Demand	34	34	0	71	71	0	63	63	0	79	79	0	77	77	0	28	28	0	28	28	0
		Hydrant #22 at 540 GPM	30	31	1	69	69	0	61	61	0	78	78	0	75	75	0	27	27	0	27	27	0
		Hydrant #108 at 700 GPM	33	33	0	68	68	0	25	25	0	63	63	0	21	21	0	-6	-6	0	-6	-6	0
		Golf Course at 600 GPM	33	33	0	69	69	0	34	34	0	67	67	0	34	34	0	3	3	0	3	3	0
	Peak Demand 2,550 GPM	No Additional Demand	31	31	0	67	67	0	58	58	0	75	75	0	72	72	0	24	24	0	24	24	0
		Hydrant #22 at 540 GPM	25	26	1	63	63	0	56	56	0	72	72	0	69	69	0	21	21	0	21	21	0
		Hydrant #108 at 700 GPM	28	28	0	63	63	0	17	17	0	56	56	0	13	13	0	-13	-13	0	-13	-13	0
		Golf Course at 600 GPM	29	29	0	63	63	0	27	27	0	60	60	0	27	27	0	-4	-4	0	-4	-4	0
Minimum Tank Level	Average Demand 725 GPM	No Additional Demand	33	33	0	70	70	0	62	62	0	78	78	0	76	76	0	27	27	0	27	27	0
		Hydrant #22 at 540 GPM	29	30	1	68	68	0	60	60	0	77	77	0	74	74	0	26	26	0	26	26	0
		Hydrant #108 at 700 GPM	32	32	0	67	67	0	24	24	0	62	62	0	20	20	0	-7	-7	0	-7	-7	0
		Golf Course at 600 GPM	32	32	0	68	68	0	33	33	0	66	66	0	33	33	0	1	1	0	1	1	0
	Peak Demand 2,550 GPM	No Additional Demand	30	30	0	66	66	0	57	57	0	74	74	0	71	71	0	23	23	0	23	23	0
		Hydrant #22 at 540 GPM	24	25	1	62	62	0	55	55	0	71	71	0	68	68	0	20	20	0	20	20	0
		Hydrant #108 at 700 GPM	27	27	0	62	62	0	16	16	0	55	55	0	12	12	0	-14	-14	0	-14	-14	0
		Golf Course at 600 GPM	28	28	0	62	62	0	26	26	0	59	59	0	26	26	0	-5	-5	0	-5	-5	0

TABLE 28: EAGLE LAKE ROAD LOOP SCENARIO
TOWN-WIDE SYSTEM PRESSURE (PSI) COMPARISON

		Location	Juliano Residence Hamilton Hill			Cleftstone Road			Hydrant #6 Cleftstone Road and Devon Road			Hydrant #104 Ells Pier			Corner of Eden Street, Eagle Lake Road & Mount Desert Street			East Strawberry Hill Jackson Laboratory Area			Top of Ireson Hill Route 3		
		Waterline Elevation	231.9			211.45			166.2			15.4			78.75			182			207.25		
		Scenario	Existing	Loop	Δ	Existing	Loop	Δ	Existing	Loop	Δ	Existing	Loop	Δ	Existing	Loop	Δ	Existing	Loop	Δ	Existing	Loop	Δ
Maximum Tank Level	Average Demand 725 GPM	No Additional Demand	18	18	0	27	27	0	46	46	0	111	111	0	84	84	0	36	36	0	29	29	0
		Hydrant #16 at 825 GPM	16	16	0	25	25	0	45	45	0	106	106	0	81	81	0	35	35	0	28	28	0
		Hydrant #104 at 1,300 GPM	14	14	0	24	24	0	43	43	0	96	96	0	79	79	0	33	33	0	27	27	0
		Hydrant #60 at 850 GPM	16	17	1	26	26	0	45	45	0	110	110	0	83	83	0	36	36	0	-13	-13	0
	Peak Demand 2,550 GPM	No Additional Demand	14	14	0	24	24	0	43	43	0	104	105	1	79	79	0	29	29	0	25	25	0
		Hydrant #16 at 825 GPM	10	10	0	20	20	0	40	40	0	96	96	0	74	74	0	24	25	1	23	23	0
		Hydrant #104 at 1,300 GPM	7	7	0	18	18	0	38	38	0	82	82	0	71	71	0	21	22	1	22	22	0
		Hydrant #60 at 850 GPM	11	11	0	21	21	0	41	41	0	102	102	0	77	77	0	27	27	0	-25	-25	0
Minimum Tank Level	Average Demand 725 GPM	No Additional Demand	17	17	0	26	26	0	45	45	0	110	110	0	83	83	0	35	35	0	28	28	0
		Hydrant #16 at 825 GPM	15	15	0	24	24	0	44	44	0	105	105	0	80	80	0	33	33	0	27	27	0
		Hydrant #104 at 1,300 GPM	13	13	0	23	23	0	42	42	0	95	95	0	78	78	0	32	32	0	26	26	0
		Hydrant #60 at 850 GPM	15	16	1	25	25	0	44	44	0	109	109	0	82	82	0	34	34	0	-14	-14	0
	Peak Demand 2,550 GPM	No Additional Demand	13	13	0	23	23	0	42	42	0	103	104	1	78	79	1	27	27	0	24	24	0
		Hydrant #16 at 825 GPM	9	9	0	19	19	0	39	39	0	94	95	1	73	73	0	23	23	0	22	22	0
		Hydrant #104 at 1,300 GPM	6	6	0	17	17	0	37	37	0	81	81	0	70	70	0	20	20	0	21	21	0
		Hydrant #60 at 850 GPM	10	10	0	20	20	0	40	40	0	101	101	0	76	76	0	26	26	0	-26	-26	0

11.7.4 EVALUATE PROVIDING AN UP-ISLAND STORAGE TANK (EVALUATE AS PART OF EXISTING PRESSURE ZONE)

The distribution system in the Up-Island area of Bar Harbor supplies roughly 147 seasonal and 143 year-round user accounts. This region spans from the section of Eden Street north of the Ferry Terminal to the seasonal lines past Salisbury Cove, including the year-round area of Hulls Cove. This area has major vulnerabilities. The Water Department had a steel tank called the Dreamwood Hill tank at the location of current Hydrant #112. This tank was demolished in 2019. As can be seen on the system map, if a line break occurred anywhere along the 8-inch diameter Eden Street line, the users downstream would have no access to a standby supply of water. Such an occurrence with the Water Department's current infrastructure could have catastrophic impacts to the Town's water users. Secondly, the single and lengthy distribution line to the area provides significant headloss during hydrant flows that limit hydrant flow capacity and result in low pressures below the standard 20-psi minimum during high flow events, especially in locations at high elevations. The model also indicates that these high elevations pull a vacuum during existing hydrant flows.

To remove these system vulnerabilities, it is recommended that a storage tank be located in the general vicinity of the Hulls Cove and Salisbury Cove region. Two (2) potential tank locations were modeled. The first location is on Town-owned land at the top of Ireson Hill in Salisbury Cove situated between the High Seas Motel and the Acadia Ocean View Motel. The lot is 1.72 acres with an approximate ground elevation of 220 feet and appears to have adequate room and necessary ground elevation for a tank of 45 feet in diameter and 60 feet in height. An alternative location was considered at the original location of the old steel Dreamwood Hill tank; however, the Ireson Hill location was the selected location of a new tank for multiple reasons. First, it appears the original tank was situated on an abutting property owned by others, and the Town owned property at this location is only 0.36 acres. Approximately 0.50 acres is required for locating an adequately sized water tank. Additionally, the ground elevation of this lot is only around 198 feet which is 22 feet lower in elevation than the proposed Ireson Hill location. The proposed Ireson Hill tank would require approximately 540-LF of 16-inch distribution piping to connect to the newly constructed 8-inch ductile iron line on US Route 3.

The second potential storage tank location on this portion of the distribution system is near the Birch Bay retirement facility located off Crooked Road prior to the entrance to the Bar Harbor Public Works Garage and the Water Department Office. This storage tank location was also evaluated. The retirement facility is situated on a hill overlooking Hulls Cove and has a cleared, but undeveloped area of land with elevation around 215 feet. This land may not have adequate space for a tank; however, further up the hill on the same property is more potential tree covered land with an elevation of around 230 feet that could potentially be utilized. The abutter to this property that owns the highest point of the hill is Acadia National Park. For the purpose of this plan, the tank was evaluated as if it was on the retirement facility property. The lower elevation would require a taller tank than what is required for the Ireson Hill location in order to meet the same hydraulic grade as the rest of the system. Therefore, to meet this elevation, a tank height of 66.5 feet was evaluated. This location also would require replacement of approximately 1,355-LF of existing 8-inch HDPE and 280-LF of 6-inch ductile iron line on Crooked Road in addition to approximately 3,000-LF of new 16-inch distribution piping to connect to the existing 8-inch lined cast iron pipe on Eden Street/US Route 3.

The proposed tanks in Table 29 for the Up-Island region were first evaluated as if they were still a part of the original pressure zone. This evaluation considered whether they could be filled by gravity or with the existing summer pump. To be filled by gravity such that no altitude valve or overflow would be required in the design of the tank, the proposed maximum water surface elevation in the tanks was evaluated at 276.5 feet. The heights of the tanks were then determined based on the approximate ground elevation of each proposed tank location. Table 29 below summarizes tank model inputs used for the evaluation for both of the proposed tank locations.

<u>TABLE 29: UP-ISLAND TANK INPUTS FOR MODEL ANALYSIS</u>

<u>Ireson Hill Tank Basis</u>	
Minimum Water Surface Level	EL 272.95 (56.45 ft)
Maximum Water Surface Level	EL 275 (58.5 ft)
Ground Elevation (Town Lot 209-015-000)	EL 220
Tank Height	60 ft
Base Elevation	EL 216.5
Maximum Overall Elevation	EL 276.5
Separate inlet & outlet with check valves	
16” diameter ductile iron, C = 130	540 LF
Tank fills from bottom	
<u>Birch Bay Tank Basis</u>	
Minimum Water Surface Level	EL 272.95 (62.95 ft)
Maximum Water Surface Level	EL 275 (65 ft)
Ground Elevation (Lot 216-060-000)	EL 215
Tank Height	66.5 ft
Base Elevation	EL2 210
Maximum Overall Elevation	EL 276.5
Separate inlet & outlet with check valves	
16” diameter ductile iron, C = 130	3,040 LF
Tank fills from bottom	

As mentioned, the first evaluation for the proposed Up-Island tanks include them as a part of the original pressure zone. Regardless of the proposed tank location, Table 30 and Table 31 demonstrate that the Up-Island area of the system will greatly benefit in the addition of a local storage tank. However, the level of improvement is dependent on the location in the system in relation to the location of the proposed tank as well as the specific hydrant that was modeled to be flowing. For example, the further away a point in the system is from the tank in combination with a relatively high elevation would produce reduced pressure improvements. This can be seen when observing the north end of the Salisbury Cove area which would be north of the Ireson Hill tank as shown in Table 30. These locations show the greatest improvements with a local Ireson Hill tank. In addition, when the modeling the Birch Bay tank with Salisbury and Hulls Cove hydrants flowing, the top of Ireson Hill appears to remain at risk of dropping below 20-psi. The Hulls Cove locations shown in Table 31 indicate that there is not much difference in the improvements between the two proposed tank locations. Table 32 shows that minor improvements would occur in some of the Down-Island locations of the distribution system. This is favorable, since the pressure in the Downtown locations is higher than recommended.

During model simulation with the tank as part of the original zone, it appears that when either of the proposed tanks are full and there is no additional flow demand in the system, the direction of flow from the Up-Island area is towards downtown, meaning that the storage volume of the Up-Island tank is contributing to the rest of the Town when included in the original pressure zone. The model indicates that the seasonal Duck Brook line is then primarily used to send flow downtown as well. Note that this volume is small and the model indicates up to 100 GPM is flowing from the Up-Island area towards Downtown depending on the system demand.

TABLE 30: PROPOSED UP-ISLAND TANK SCENARIOS

ORIGINAL ZONE
12" HDPE SEASONAL LINE OPEN
SALISBURY COVE SYSTEM PRESSURE (PSI) COMPARISON

		Location	US Route 3 and Lindsay Way Near End of Seasonal Line					Sand Point Road and Bishops Way US Route 3					Hydrant #112 US Route 3					Top of Ireson Hill US Route 3				
		Waterline Elevation	167.9					50.3					200					207.25				
		Scenario	Existing	Ireson Hill	Δ	Birch Bay	Δ	Existing	Ireson Hill	Δ	Birch Bay	Δ	Existing	Ireson Hill	Δ	Birch Bay	Δ	Existing	Ireson Hill	Δ	Birch Bay	Δ
Maximum Tank Level	Average Demand 725 GPM	No Additional Demand	45	46	1	46	1	97	97	0	97	0	32	32	0	32	0	29	29	0	29	0
		Hydrant #35 at 1,225 GPM	27	46	19	46	19	78	96	18	97	19	13	32	19	32	19	10	29	19	29	19
		Hydrant #92 at 1,015 GPM	-1	45	46	45	46	51	96	45	96	45	-14	32	46	31	45	-17	29	46	28	45
		Hydrant #60 at 850 GPM	4	45	41	37	33	55	96	41	88	33	-10	32	42	24	34	-13	29	42	20	33
	Peak Demand 2,550 GPM	No Additional Demand	39	43	4	43	4	93	97	4	96	3	28	32	4	32	4	25	29	4	29	4
		Hydrant #35 at 1,225 GPM	15	42	27	42	27	69	96	27	96	27	5	32	27	31	26	2	29	27	28	26
		Hydrant #92 at 1,015 GPM	-17	41	58	41	58	37	95	58	95	58	-27	32	59	30	57	-30	29	59	27	57
		Hydrant #60 at 850 GPM	-12	41	53	32	44	42	95	53	86	44	-22	32	54	22	44	-25	29	54	19	44
Minimum Tank Level	Average Demand 725 GPM	No Additional Demand	45	45	0	45	0	96	96	0	96	0	31	32	1	31	0	28	28	0	28	0
		Hydrant #35 at 1,225 GPM	26	45	19	45	19	77	96	19	96	19	12	31	19	31	19	9	28	19	28	19
		Hydrant #92 at 1,015 GPM	-2	44	46	44	46	50	95	45	95	45	-15	31	46	30	45	-18	28	46	27	45
		Hydrant #60 at 850 GPM	3	44	41	36	33	54	95	41	87	33	-11	31	42	23	34	-14	28	42	20	34
	Peak Demand 2,550 GPM	No Additional Demand	38	42	4	42	4	92	96	4	96	4	27	31	4	31	4	24	28	4	28	4
		Hydrant #35 at 1,225 GPM	14	41	27	41	27	68	95	27	95	27	4	31	27	31	27	1	28	27	28	27
		Hydrant #92 at 1,015 GPM	-18	41	59	40	58	36	94	58	94	58	-28	31	59	29	57	-31	28	59	26	57
		Hydrant #60 at 850 GPM	-13	41	54	31	44	41	94	53	85	44	-23	31	54	21	44	-26	28	54	18	44

TABLE 31: PROPOSED UP-ISLAND TANK SCENARIOS

ORIGINAL ZONE
12" HDPE SEASONAL LINE OPEN
HULLS COVE SYSTEM PRESSURE (PSI) COMPARISON

		Location	Hydrant #60 US Route 3					The Colony Cottages US Route 3					Hydrant #48 Crooked Road				
		Waterline Elevation	88.9					7.5					42.5				
		Scenario	Existing	Ireson Hill	Δ	Birch Bay	Δ	Existing	Ireson Hill	Δ	Birch Bay	Δ	Existing	Ireson Hill	Δ	Birch Bay	Δ
Maximum Tank Level	Average Demand 725 GPM	No Additional Demand	80	80	0	80	0	115	116	1	116	1	100	100	0	101	1
		Hydrant #35 at 1,225 GPM	61	78	17	80	19	96	112	16	115	19	81	96	15	100	19
		Hydrant #92 at 1,015 GPM	34	76	42	79	45	69	107	38	114	45	54	92	38	100	46
		Hydrant #60 at 850 GPM	38	76	38	72	34	81	112	31	114	33	66	97	31	100	34
	Peak Demand 2,550 GPM	No Additional Demand	76	80	4	80	4	112	115	3	116	4	97	100	3	100	3
		Hydrant #35 at 1,225 GPM	53	77	24	80	27	88	110	22	115	27	73	95	22	100	27
		Hydrant #92 at 1,015 GPM	21	75	54	78	57	56	105	49	114	58	41	90	49	99	58
		Hydrant #60 at 850 GPM	26	74	48	70	44	69	110	41	113	44	55	95	40	99	44
Minimum Tank Level	Average Demand 725 GPM	No Additional Demand	79	80	1	80	1	114	115	1	115	1	99	100	1	100	1
		Hydrant #35 at 1,225 GPM	60	77	17	79	19	95	111	16	114	19	80	96	16	99	19
		Hydrant #92 at 1,015 GPM	33	75	42	78	45	68	106	38	114	46	53	91	38	99	46
		Hydrant #60 at 850 GPM	37	75	38	71	34	80	111	31	113	33	65	96	31	99	34
	Peak Demand 2,550 GPM	No Additional Demand	76	79	3	79	3	111	114	3	115	4	96	99	3	100	4
		Hydrant #35 at 1,225 GPM	52	76	24	79	27	87	109	22	114	27	72	94	22	99	27
		Hydrant #92 at 1,015 GPM	20	74	54	78	58	56	104	48	113	57	40	89	49	98	58
		Hydrant #60 at 850 GPM	25	74	49	69	44	68	109	41	112	44	54	94	40	99	45

TABLE 32: PROPOSED UP-ISLAND TANK SCENARIOS

ORIGINAL ZONE
12" HDPE SEASONAL LINE OPEN
DOWN-ISLAND SYSTEM PRESSURE (PSI) COMPARISON

		Location	Hydrant #34 Eden Street					Ferry Terminal Eden Street					Hydrant #104 Ells Pier				
		Waterline Elevation	42.9					48.8					15.4				
		Scenario	Existing	Ireson Hill	Δ	Birch Bay	Δ	Existing	Ireson Hill	Δ	Birch Bay	Δ	Existing	Ireson Hill	Δ	Birch Bay	Δ
Maximum Tank Level	Average Demand 725 GPM	No Additional Demand	100	100	0	100	0	97	98	1	98	1	111	111	0	111	0
		Hydrant #35 at 1,225 GPM	91	96	5	96	5	91	94	3	95	4	109	110	1	110	1
		Hydrant #92 at 1,015 GPM	94	99	5	100	6	93	96	3	97	4	109	110	1	111	2
		Hydrant #60 at 850 GPM	95	99	4	100	5	94	97	3	97	3	110	111	1	111	1
	Peak Demand 2,550 GPM	No Additional Demand	97	99	2	99	2	95	96	1	96	1	104	105	1	105	1
		Hydrant #35 at 1,225 GPM	86	92	6	93	7	85	91	6	91	6	101	103	2	103	2
		Hydrant #92 at 1,015 GPM	88	96	8	98	10	88	93	5	96	8	101	104	3	105	4
		Hydrant #60 at 850 GPM	90	97	7	99	9	89	94	5	96	7	102	104	2	105	3
Minimum Tank Level	Average Demand 725 GPM	No Additional Demand	99	99	0	99	0	96	97	1	97	1	110	110	0	110	0
		Hydrant #35 at 1,225 GPM	90	95	5	95	5	90	93	3	94	4	108	109	1	109	1
		Hydrant #92 at 1,015 GPM	93	98	5	99	6	92	95	3	96	4	108	109	1	110	2
		Hydrant #60 at 850 GPM	94	98	4	99	5	93	96	3	96	3	109	110	1	110	1
	Peak Demand 2,550 GPM	No Additional Demand	97	98	1	98	1	94	95	1	95	1	103	104	1	104	1
		Hydrant #35 at 1,225 GPM	85	91	6	92	7	85	90	5	90	5	99	102	3	102	3
		Hydrant #92 at 1,015 GPM	87	95	8	97	10	87	92	5	95	8	100	103	3	104	4
		Hydrant #60 at 850 GPM	89	96	7	98	9	88	94	6	95	7	101	103	2	104	3

11.7.5 EVALUATE PROVIDING A NEW STORAGE TANK UP-ISLAND (EVALUATE AS A SEPARATE ZONE WITH PUMP STATION)

An alternate option for the inclusion of an Up-Island tank is to separate this portion of the distribution system and create a separate zone with its own pump station to fill the tank. The proposed location of pump station is on the corner of the Town-owned Ferry Terminal property (Lot 231-004-000) at the end of the newly installed 8-inch ductile iron line on Eden Street/US Route 3. Piping through the station is assumed to be 8-inch ductile iron with a pump bypass including a check valve to allow for gravity feed to the Up-Island portion of system. The bypass would then be used for peak demands when the tank discharge is insufficient or during additional hydrant flows to maintain hydrant capacity. The pump at the station would then be used to fill the tank under extreme circumstances when filling by gravity is not sufficient.

With a separate zone, the ferry pump off, and the bypass closed, the water source would be solely dependent on the proposed Up-Island tank. This would be fine for regular system demands; however, hydrant flows, especially for hydrants located a distance away from the proposed tanks, would have reduced capacities due to the long distance and high headloss. As discussed, it is assumed the proposed ferry pump for the proposed Up-Island storage tank is to be used for the purpose of providing boosted pressure and/or filling the tank under peak demand conditions, and not for the purpose of providing hydrant flows. A fire flow pump would drive the pump size up significantly. Therefore, it will be assumed that the pump bypass line is open during regular operation. During hydrant flows, it would be expected for some water to flow through the pump bypass and the pressures and hydrant capacities would not be significantly impacted with the exception of the additional headloss through the proposed pump station lines and fittings. The level of impact on the zone during these high flow periods would again depend on the location of the tank within the zone, which hydrants are flowing, and also if the seasonal 12-inch HDPE line located near the Ferry Terminal is open.

First, the system was evaluated without the proposed pump running to determine the effects of separating this portion of the system from the original pressure zone. The pressure results comparing the two zone scenarios are shown in Tables 33 through 38. Pressure results comparing the two tank options in a new zone are shown in Tables 39 through 41.

There is not much difference in the improvements to the Salisbury Cove area when comparing the proposed Ireson Hill tank in the original zone with that of the tank in a new zone (See Table 33). This is due to the tank being located in the Salisbury area. The improvements in either case are significant. Even though there are still pressure improvements with the tank, when comparing the zone separation with the Birch Bay tank, the improvements seen are greatly reduced (See Table 36). The critical area at the top of Ireson Hill still remains at risk of dropping below 20-psi when Salisbury or Hulls Cove hydrants are flowed with the Birch Bay tank option. The Hulls Cove area also sees little difference in the improvements with the Ireson Hill tank in either zone scenario (See Table 34). Again, the Birch Bay tank in a new zone sees less improvement in this area; however, compared to Salisbury Hill, it is not nearly as significant and the final resulting pressures are still acceptable (See Table 37). Tables 35 and 38 respectively show minor reductions in improvement just downstream of the proposed pump station at Hydrant #34 with the Ireson Hill tank in a separate zone and a slightly higher reduction in improvement with the Birch Bay tank in a separate zone.

In conclusion, the proposed Ireson Hill location will provide the most benefit to the Up-Island area where the greatest pressure improvements are needed in the Salisbury Cove area. While the introduction of the Birch Bay tank would provide improvements for the area, it does not provide enough of an improvement for the Salisbury Cove area, particularly during hydrant flows.

Note that the above analysis is with the seasonal 12-inch HDPE line open. With the seasonal line inactive, the flow of water needs to go to the intersection of Eden Street and Highbrook Street before it is able to travel North to the Up-Island area of the system. The longer route and the relatively small diameter piping creates a higher headloss and therefore reduces the flow able to make it to the Up-Island area under the same hydraulic grade. This will therefore reduce hydrant flow capacities as well as reduce pressures under equivalent system demands. Tables 42 through 44 provide the pressure results for the Ireson Hill tank in both zone scenarios with the seasonal line inactive. Again, the same flows were applied to each hydrant to compare the resulting pressures even though the hydrant flows in reality would be much less. This can be seen in the existing scenario pressure results, which are negative during the additional hydrant flow conditions. Even with the seasonal line closed, major improvements are seen in pressure during both typical system demands and during additional hydrant flows. The relatively low drop in pressure when applying the hydrant flows with the proposed Ireson Tank active also indicates that the hydrant capacities will be improved with the tank even with the seasonal HDPE line inactive. These capacities may even be in the ballpark of the existing capacities with the seasonal HDPE line active.

TABLE 33: IRESON HILL TANK ZONE SCENARIOS

12" HDPE SEASONAL LINE OPEN
SALISBURY COVE SYSTEM PRESSURE (PSI) COMPARISON

		Location	US Route 3 and Lindsay Way Near End of Seasonal Line					Sand Point Road and Bishops Way US Route 3					Hydrant #112 US Route 3					Top of Ireson Hill US Route 3				
		Waterline Elevation	167.9					50.3					200					207.25				
		Scenario	Existing	Original Zone	Δ	New Zone	Δ	Existing	Original Zone	Δ	New Zone	Δ	Existing	Original Zone	Δ	New Zone	Δ	Existing	Original Zone	Δ	New Zone	Δ
Maximum Tank Level	Average Demand 725 GPM	No Additional Demand	45	46	1	46	1	97	97	0	97	0	32	32	0	32	0	29	29	0	29	0
		Hydrant #35 at 1,225 GPM	27	46	19	46	19	78	96	18	96	18	13	32	19	32	19	10	29	19	29	19
		Hydrant #92 at 1,015 GPM	-1	45	46	45	46	51	96	45	96	45	-14	32	46	32	46	-17	29	46	29	46
		Hydrant #60 at 850 GPM	4	45	41	45	41	55	96	41	96	41	-10	32	42	32	42	-13	29	42	29	42
	Peak Demand 2,550 GPM	No Additional Demand	39	43	4	43	4	93	97	4	97	4	28	32	4	32	4	25	29	4	29	4
		Hydrant #35 at 1,225 GPM	15	42	27	42	27	69	96	27	95	26	5	32	27	32	27	2	29	27	29	27
		Hydrant #92 at 1,015 GPM	-17	41	58	41	58	37	95	58	95	58	-27	32	59	32	59	-30	29	59	29	59
		Hydrant #60 at 850 GPM	-12	41	53	41	53	42	95	53	95	53	-22	32	54	32	54	-25	29	54	29	54
Minimum Tank Level	Average Demand 725 GPM	No Additional Demand	45	45	0	45	0	96	96	0	96	0	31	32	1	32	1	28	28	0	28	0
		Hydrant #35 at 1,225 GPM	26	45	19	45	19	77	96	19	95	18	12	31	19	31	19	9	28	19	28	19
		Hydrant #92 at 1,015 GPM	-2	44	46	44	46	50	95	45	95	45	-15	31	46	31	46	-18	28	46	28	46
		Hydrant #60 at 850 GPM	3	44	41	44	41	54	95	41	95	41	-11	31	42	31	42	-14	28	42	28	42
	Peak Demand 2,550 GPM	No Additional Demand	38	42	4	42	4	92	96	4	96	4	27	31	4	31	4	24	28	4	28	4
		Hydrant #35 at 1,225 GPM	14	41	27	41	27	68	95	27	94	26	4	31	27	31	27	1	28	27	28	27
		Hydrant #92 at 1,015 GPM	-18	41	59	41	59	36	94	58	94	58	-28	31	59	31	59	-31	28	59	28	59
		Hydrant #60 at 850 GPM	-13	41	54	41	54	41	94	53	94	53	-23	31	54	31	54	-26	28	54	28	54

TABLE 34: IRESON HILL TANK ZONE SCENARIOS

12" HDPE SEASONAL LINE OPEN

HULLS COVE SYSTEM PRESSURE (PSI) COMPARISON

		Location	Hydrant #60 US Route 3					The Colony Cottages US Route 3					Hydrant #48 Crooked Road				
		Waterline Elevation	88.9					7.5					42.5				
		Scenario	Existing	Original Zone	Δ	New Zone	Δ	Existing	Original Zone	Δ	New Zone	Δ	Existing	Original Zone	Δ	New Zone	Δ
Maximum Tank Level	Average Demand 725 GPM	No Additional Demand	80	80	0	80	0	115	116	1	116	1	100	100	0	101	1
		Hydrant #35 at 1,225 GPM	61	78	17	78	17	96	112	16	111	15	81	96	15	96	15
		Hydrant #92 at 1,015 GPM	34	76	42	76	42	69	107	38	107	38	54	92	38	91	37
		Hydrant #60 at 850 GPM	38	76	38	76	38	81	112	31	112	31	66	97	31	96	30
	Peak Demand 2,550 GPM	No Additional Demand	76	80	4	80	4	112	115	3	115	3	97	100	3	100	3
		Hydrant #35 at 1,225 GPM	53	77	24	77	24	88	110	22	109	21	73	95	22	94	21
		Hydrant #92 at 1,015 GPM	21	75	54	74	53	56	105	49	105	49	41	90	49	89	48
		Hydrant #60 at 850 GPM	26	74	48	74	48	69	110	41	110	41	55	95	40	95	40
Minimum Tank Level	Average Demand 725 GPM	No Additional Demand	79	80	1	80	1	114	115	1	115	1	99	100	1	100	1
		Hydrant #35 at 1,225 GPM	60	77	17	77	17	95	111	16	110	15	80	96	16	95	15
		Hydrant #92 at 1,015 GPM	33	75	42	75	42	68	106	38	106	38	53	91	38	91	38
		Hydrant #60 at 850 GPM	37	75	38	75	38	80	111	31	111	31	65	96	31	96	31
	Peak Demand 2,550 GPM	No Additional Demand	76	79	3	79	3	111	114	3	114	3	96	99	3	99	3
		Hydrant #35 at 1,225 GPM	52	76	24	76	24	87	109	22	108	21	72	94	22	93	21
		Hydrant #92 at 1,015 GPM	20	74	54	73	53	56	104	48	104	48	40	89	49	88	48
		Hydrant #60 at 850 GPM	25	74	49	73	48	68	109	41	109	41	54	94	40	94	40

TABLE 35: IRESON HILL TANK ZONE SCENARIOS

12" HDPE SEASONAL LINE OPEN
DOWN-ISLAND SYSTEM PRESSURE (PSI) COMPARISON

		Location	Hydrant #34 Eden Street					Ferry Terminal Eden Street					Hydrant #104 Ells Pier				
		Waterline Elevation	42.9					48.8					15.4				
		Scenario	Existing	Original Zone	Δ	New Zone	Δ	Existing	Original Zone	Δ	New Zone	Δ	Existing	Original Zone	Δ	New Zone	Δ
Maximum Tank Level	Average Demand 725 GPM	No Additional Demand	100	100	0	100	0	97	98	1	98	1	111	111	0	111	0
		Hydrant #35 at 1,225 GPM	91	96	5	94	3	91	94	3	94	3	109	110	1	110	1
		Hydrant #92 at 1,015 GPM	94	99	5	98	4	93	96	3	96	3	109	110	1	110	1
		Hydrant #60 at 850 GPM	95	99	4	99	4	94	97	3	97	3	110	111	1	111	1
	Peak Demand 2,550 GPM	No Additional Demand	97	99	2	100	3	95	96	1	96	1	104	105	1	105	1
		Hydrant #35 at 1,225 GPM	86	92	6	90	4	85	91	6	91	6	101	103	2	103	2
		Hydrant #92 at 1,015 GPM	88	96	8	95	7	88	93	5	93	5	101	104	3	104	3
		Hydrant #60 at 850 GPM	90	97	7	97	7	89	94	5	95	6	102	104	2	104	2
Minimum Tank Level	Average Demand 725 GPM	No Additional Demand	99	99	0	99	0	96	97	1	97	1	110	110	0	110	0
		Hydrant #35 at 1,225 GPM	90	95	5	93	3	90	93	3	93	3	108	109	1	109	1
		Hydrant #92 at 1,015 GPM	93	98	5	97	4	92	95	3	95	3	108	109	1	109	1
		Hydrant #60 at 850 GPM	94	98	4	98	4	93	96	3	96	3	109	110	1	110	1
	Peak Demand 2,550 GPM	No Additional Demand	97	98	1	99	2	94	95	1	95	1	103	104	1	104	1
		Hydrant #35 at 1,225 GPM	85	91	6	89	4	85	90	5	90	5	99	102	3	102	3
		Hydrant #92 at 1,015 GPM	87	95	8	94	7	87	92	5	93	6	100	103	3	103	3
		Hydrant #60 at 850 GPM	89	96	7	96	7	88	94	6	94	6	101	103	2	103	2

TABLE 36: BIRCH BAY TANK ZONE SCENARIOS

12" HDPE SEASONAL LINE OPEN
SALISBURY COVE SYSTEM PRESSURE (PSI) COMPARISON

		Location	US Route 3 and Lindsay Way Near End of Seasonal Line					Sand Point Road and Bishops Way US Route 3					Hydrant #112 US Route 3					Top of Ireson Hill US Route 3				
		Waterline Elevation	167.9					50.3					200					207.25				
		Scenario	Existing	Original Zone	Δ	New Zone	Δ	Existing	Original Zone	Δ	New Zone	Δ	Existing	Original Zone	Δ	New Zone	Δ	Existing	Original Zone	Δ	New Zone	Δ
Maximum Tank Level	Average Demand 725 GPM	No Additional Demand	45	46	1	46	1	97	97	0	97	0	32	32	0	32	0	29	29	0	29	0
		Hydrant #35 at 1,225 GPM	27	46	19	40	13	78	97	19	91	13	13	32	19	26	13	10	29	19	23	13
		Hydrant #92 at 1,015 GPM	-1	45	46	35	36	51	96	45	86	35	-14	31	45	21	35	-17	28	45	18	35
		Hydrant #60 at 850 GPM	4	37	33	30	26	55	88	33	81	26	-10	24	34	16	26	-13	20	33	13	26
	Peak Demand 2,550 GPM	No Additional Demand	39	43	4	41	2	93	96	3	95	2	28	32	4	31	3	25	29	4	28	3
		Hydrant #35 at 1,225 GPM	15	42	27	33	18	69	96	27	87	18	5	31	26	22	17	2	28	26	19	17
		Hydrant #92 at 1,015 GPM	-17	41	58	28	45	37	95	58	82	45	-27	30	57	17	44	-30	27	57	14	44
		Hydrant #60 at 850 GPM	-12	32	44	22	34	42	86	44	76	34	-22	22	44	12	34	-25	19	44	9	34
Minimum Tank Level	Average Demand 725 GPM	No Additional Demand	45	45	0	45	0	96	96	0	96	0	31	31	0	31	0	28	28	0	28	0
		Hydrant #35 at 1,225 GPM	26	45	19	39	13	77	96	19	90	13	12	31	19	25	13	9	28	19	22	13
		Hydrant #92 at 1,015 GPM	-2	44	46	34	36	50	95	45	85	35	-15	30	45	20	35	-18	27	45	17	35
		Hydrant #60 at 850 GPM	3	36	33	29	26	54	87	33	80	26	-11	23	34	15	26	-14	20	34	12	26
	Peak Demand 2,550 GPM	No Additional Demand	38	42	4	40	2	92	96	4	94	2	27	31	4	30	3	24	28	4	27	3
		Hydrant #35 at 1,225 GPM	14	41	27	32	18	68	95	27	86	18	4	31	27	21	17	1	28	27	18	17
		Hydrant #92 at 1,015 GPM	-18	40	58	27	45	36	94	58	81	45	-28	29	57	16	44	-31	26	57	13	44
		Hydrant #60 at 850 GPM	-13	31	44	21	34	41	85	44	75	34	-23	21	44	11	34	-26	18	44	8	34

TABLE 37: BIRCH BAY TANK ZONE SCENARIOS

12" HDPE SEASONAL LINE OPEN

SALISBURY COVE SYSTEM PRESSURE (PSI) COMPARISON

		Location	Hydrant #60 US Route 3					The Colony Cottages US Route 3					Hydrant #48 Crooked Road				
		Waterline Elevation	88.9					7.5					42.5				
		Scenario	Existing	Original Zone	Δ	New Zone	Δ	Existing	Original Zone	Δ	New Zone	Δ	Existing	Original Zone	Δ	New Zone	Δ
Maximum Tank Level	Average Demand 725 GPM	No Additional Demand	80	80	0	80	0	115	116	1	116	1	100	101	1	101	1
		Hydrant #35 at 1,225 GPM	61	80	19	74	13	96	115	19	109	13	81	100	19	98	17
		Hydrant #92 at 1,015 GPM	34	79	45	69	35	69	114	45	104	35	54	100	46	97	43
		Hydrant #60 at 850 GPM	38	72	34	64	26	81	114	33	107	26	66	100	34	98	32
	Peak Demand 2,550 GPM	No Additional Demand	76	80	4	79	3	112	116	4	114	2	97	100	3	100	3
		Hydrant #35 at 1,225 GPM	53	80	27	71	18	88	115	27	106	18	73	100	27	97	24
		Hydrant #92 at 1,015 GPM	21	78	57	66	45	56	114	58	101	45	41	99	58	95	54
		Hydrant #60 at 850 GPM	26	70	44	60	34	69	113	44	103	34	55	99	44	96	41
Minimum Tank Level	Average Demand 725 GPM	No Additional Demand	79	80	1	79	0	114	115	1	115	1	99	100	1	100	1
		Hydrant #35 at 1,225 GPM	60	79	19	73	13	95	114	19	108	13	80	99	19	97	17
		Hydrant #92 at 1,015 GPM	33	78	45	68	35	68	114	46	104	36	53	99	46	96	43
		Hydrant #60 at 850 GPM	37	71	34	63	26	80	113	33	106	26	65	99	34	97	32
	Peak Demand 2,550 GPM	No Additional Demand	76	79	3	78	2	111	115	4	113	2	96	100	4	99	3
		Hydrant #35 at 1,225 GPM	52	79	27	70	18	87	114	27	105	18	72	99	27	96	24
		Hydrant #92 at 1,015 GPM	20	78	58	65	45	56	113	57	100	44	40	98	58	94	54
		Hydrant #60 at 850 GPM	25	69	44	59	34	68	112	44	102	34	54	99	45	96	42

TABLE 38: BIRCH BAY TANK ZONE SCENARIOS

12" HDPE SEASONAL LINE OPEN
SALISBURY COVE SYSTEM PRESSURE (PSI) COMPARISON

		Location	Hydrant #34 Eden Street					Ferry Terminal Eden Street					Hydrant #104 Ells Pier				
		Waterline Elevation	42.9					48.8					15.4				
		Scenario	Existing	Original Zone	Δ	New Zone	Δ	Existing	Original Zone	Δ	New Zone	Δ	Existing	Original Zone	Δ	New Zone	Δ
Maximum Tank Level	Average Demand 725 GPM	No Additional Demand	100	100	0	100	0	97	98	1	98	1	111	111	0	111	0
		Hydrant #35 at 1,225 GPM	91	96	5	93	2	91	95	4	94	3	109	110	1	110	1
		Hydrant #92 at 1,015 GPM	94	100	6	98	4	93	97	4	96	3	109	111	2	110	1
		Hydrant #60 at 850 GPM	95	100	5	98	3	94	97	3	96	2	110	111	1	110	0
	Peak Demand 2,550 GPM	No Additional Demand	97	99	2	99	2	95	96	1	96	1	104	105	1	105	1
		Hydrant #35 at 1,225 GPM	86	93	7	89	3	85	91	6	91	6	101	103	2	103	2
		Hydrant #92 at 1,015 GPM	88	98	10	94	6	88	96	8	93	5	101	105	4	104	3
		Hydrant #60 at 850 GPM	90	99	9	95	5	89	96	7	93	4	102	105	3	104	2
Minimum Tank Level	Average Demand 725 GPM	No Additional Demand	99	99	0	99	0	96	97	1	97	1	110	110	0	110	0
		Hydrant #35 at 1,225 GPM	90	95	5	92	2	90	94	4	93	3	108	109	1	109	1
		Hydrant #92 at 1,015 GPM	93	99	6	97	4	92	96	4	95	3	108	110	2	109	1
		Hydrant #60 at 850 GPM	94	99	5	97	3	93	96	3	95	2	109	110	1	109	0
	Peak Demand 2,550 GPM	No Additional Demand	97	98	1	98	1	94	95	1	95	1	103	104	1	104	1
		Hydrant #35 at 1,225 GPM	85	92	7	88	3	85	90	5	90	5	99	102	3	101	2
		Hydrant #92 at 1,015 GPM	87	97	10	93	6	87	95	8	92	5	100	104	4	103	3
		Hydrant #60 at 850 GPM	89	98	9	94	5	88	95	7	92	4	101	104	3	103	2

TABLE 39: PROPOSED UP-ISLAND TANK SCENARIOS

NEW ZONE
12” HDPE SEASONAL LINE OPEN
SALISBURY COVE SYSTEM PRESSURE (PSI) COMPARISON

		Location	US Route 3 and Lindsay Way Near End of Seasonal Line					Sand Point Road and Bishops Way US Route 3					Hydrant #112 US Route 3					Top of Ireson Hill US Route 3				
		Waterline Elevation	167.9					50.3					200					207.25				
		Scenario	Existing	Ireson Hill	Δ	Birch Bay	Δ	Existing	Ireson Hill	Δ	Birch Bay	Δ	Existing	Ireson Hill	Δ	Birch Bay	Δ	Existing	Ireson Hill	Δ	Birch Bay	Δ
Maximum Tank Level	Average Demand 725 GPM	No Additional Demand	45	46	1	46	1	97	97	0	97	0	32	32	0	32	0	29	29	0	29	0
		Hydrant #35 at 1,225 GPM	27	46	19	40	13	78	96	18	91	13	13	32	19	26	13	10	29	19	23	13
		Hydrant #92 at 1,015 GPM	-1	45	46	35	36	51	96	45	86	35	-14	32	46	21	35	-17	29	46	18	35
		Hydrant #60 at 850 GPM	4	45	41	30	26	55	96	41	81	26	-10	32	42	16	26	-13	29	42	13	26
	Peak Demand 2,550 GPM	No Additional Demand	39	43	4	41	2	93	97	4	95	2	28	32	4	31	3	25	29	4	28	3
		Hydrant #35 at 1,225 GPM	15	42	27	33	18	69	95	26	87	18	5	32	27	22	17	2	29	27	19	17
		Hydrant #92 at 1,015 GPM	-17	41	58	28	45	37	95	58	82	45	-27	32	59	17	44	-30	29	59	14	44
		Hydrant #60 at 850 GPM	-12	41	53	22	34	42	95	53	76	34	-22	32	54	12	34	-25	29	54	9	34
Minimum Tank Level	Average Demand 725 GPM	No Additional Demand	45	45	0	45	0	96	96	0	96	0	31	32	1	31	0	28	28	0	28	0
		Hydrant #35 at 1,225 GPM	26	45	19	39	13	77	95	18	90	13	12	31	19	25	13	9	28	19	22	13
		Hydrant #92 at 1,015 GPM	-2	44	46	34	36	50	95	45	85	35	-15	31	46	20	35	-18	28	46	17	35
		Hydrant #60 at 850 GPM	3	44	41	29	26	54	95	41	80	26	-11	31	42	15	26	-14	28	42	12	26
	Peak Demand 2,550 GPM	No Additional Demand	38	42	4	40	2	92	96	4	94	2	27	31	4	30	3	24	28	4	27	3
		Hydrant #35 at 1,225 GPM	14	41	27	32	18	68	94	26	86	18	4	31	27	21	17	1	28	27	18	17
		Hydrant #92 at 1,015 GPM	-18	41	59	27	45	36	94	58	81	45	-28	31	59	16	44	-31	28	59	13	44
		Hydrant #60 at 850 GPM	-13	41	54	21	34	41	94	53	75	34	-23	31	54	11	34	-26	28	54	8	34

TABLE 40: PROPOSED UP-ISLAND TANK SCENARIOS

NEW ZONE
12" HDPE SEASONAL LINE OPEN
HULLS COVE SYSTEM PRESSURE (PSI) COMPARISON

		Location	Hydrant #60 US Route 3					The Colony Cottages US Route 3					Hydrant #48 Crooked Road				
		Waterline Elevation	88.9					7.5					42.5				
		Scenario	Existing	Ireson Hill	Δ	Birch Bay	Δ	Existing	Ireson Hill	Δ	Birch Bay	Δ	Existing	Ireson Hill	Δ	Birch Bay	Δ
Maximum Tank Level	Average Demand 725 GPM	No Additional Demand	80	80	0	80	0	115	116	1	116	1	100	101	1	101	1
		Hydrant #35 at 1,225 GPM	61	78	17	74	13	96	111	15	109	13	81	96	15	98	17
		Hydrant #92 at 1,015 GPM	34	76	42	69	35	69	107	38	104	35	54	91	37	97	43
		Hydrant #60 at 850 GPM	38	76	38	64	26	81	112	31	107	26	66	96	30	98	32
	Peak Demand 2,550 GPM	No Additional Demand	76	80	4	79	3	112	115	3	114	2	97	100	3	100	3
		Hydrant #35 at 1,225 GPM	53	77	24	71	18	88	109	21	106	18	73	94	21	97	24
		Hydrant #92 at 1,015 GPM	21	74	53	66	45	56	105	49	101	45	41	89	48	95	54
		Hydrant #60 at 850 GPM	26	74	48	60	34	69	110	41	103	34	55	95	40	96	41
Minimum Tank Level	Average Demand 725 GPM	No Additional Demand	79	80	1	79	0	114	115	1	115	1	99	100	1	100	1
		Hydrant #35 at 1,225 GPM	60	77	17	73	13	95	110	15	108	13	80	95	15	97	17
		Hydrant #92 at 1,015 GPM	33	75	42	68	35	68	106	38	104	36	53	91	38	96	43
		Hydrant #60 at 850 GPM	37	75	38	63	26	80	111	31	106	26	65	96	31	97	32
	Peak Demand 2,550 GPM	No Additional Demand	76	79	3	78	2	111	114	3	113	2	96	99	3	99	3
		Hydrant #35 at 1,225 GPM	52	76	24	70	18	87	108	21	105	18	72	93	21	96	24
		Hydrant #92 at 1,015 GPM	20	73	53	65	45	56	104	48	100	44	40	88	48	94	54
		Hydrant #60 at 850 GPM	25	73	48	59	34	68	109	41	102	34	54	94	40	96	42

<u>TABLE 41: PROPOSED UP-ISLAND TANK SCENARIOS</u>

<u>NEW ZONE</u>
<u>12" HDPE SEASONAL LINE OPEN</u>
<u>DOWN-ISLAND SYSTEM PRESSURE (PSI) COMPARISON</u>

		Location	Hydrant #34 Eden Street					Ferry Terminal Eden Street					Hydrant #104 Ells Pier				
		Waterline Elevation	42.9					48.8					15.4				
		Scenario	Existing	Ireson Hill	Δ	Birch Bay	Δ	Existing	Ireson Hill	Δ	Birch Bay	Δ	Existing	Ireson Hill	Δ	Birch Bay	Δ
Maximum Tank Level	Average Demand 725 GPM	No Additional Demand	100	100	0	100	0	97	98	1	98	1	111	111	0	111	0
		Hydrant #35 at 1,225 GPM	91	94	3	93	2	91	94	3	94	3	109	110	1	110	1
		Hydrant #92 at 1,015 GPM	94	98	4	98	4	93	96	3	96	3	109	110	1	110	1
		Hydrant #60 at 850 GPM	95	99	4	98	3	94	97	3	96	2	110	111	1	110	0
	Peak Demand 2,550 GPM	No Additional Demand	97	100	3	99	2	95	96	1	96	1	104	105	1	105	1
		Hydrant #35 at 1,225 GPM	86	90	4	89	3	85	91	6	91	6	101	103	2	103	2
		Hydrant #92 at 1,015 GPM	88	95	7	94	6	88	93	5	93	5	101	104	3	104	3
		Hydrant #60 at 850 GPM	90	97	7	95	5	89	95	6	93	4	102	104	2	104	2
Minimum Tank Level	Average Demand 725 GPM	No Additional Demand	99	99	0	99	0	96	97	1	97	1	110	110	0	110	0
		Hydrant #35 at 1,225 GPM	90	93	3	92	2	90	93	3	93	3	108	109	1	109	1
		Hydrant #92 at 1,015 GPM	93	97	4	97	4	92	95	3	95	3	108	109	1	109	1
		Hydrant #60 at 850 GPM	94	98	4	97	3	93	96	3	95	2	109	110	1	109	0
	Peak Demand 2,550 GPM	No Additional Demand	97	99	2	98	1	94	95	1	95	1	103	104	1	104	1
		Hydrant #35 at 1,225 GPM	85	89	4	88	3	85	90	5	90	5	99	102	3	101	2
		Hydrant #92 at 1,015 GPM	87	94	7	93	6	87	93	6	92	5	100	103	3	103	3
		Hydrant #60 at 850 GPM	89	96	7	94	5	88	94	6	92	4	101	103	2	103	2

TABLE 42: IRESON HILL TANK SCENARIOS

12" HDPE SEASONAL LINE CLOSED
SALISBURY COVE SYSTEM PRESSURE (PSI) COMPARISON

		Location	US Route 3 and Lindsay Way Near End of Seasonal Line					Sand Point Road and Bishops Way US Route 3					Hydrant #112 US Route 3					Top of Ireson Hill US Route 3				
		Waterline Elevation	167.9					50.3					200					207.25				
		Scenario	Existing	Original Zone	Δ	New Zone	Δ	Existing	Original Zone	Δ	New Zone	Δ	Existing	Original Zone	Δ	New Zone	Δ	Existing	Original Zone	Δ	New Zone	Δ
Maximum Tank Level	Average Demand 725 GPM	No Additional Demand	44	46	2	46	2	95	97	2	97	2	30	32	2	32	2	27	29	2	29	2
		Hydrant #35 at 1,225 GPM	-62	45	107	45	107	-10	95	105	95	105	-75	32	107	32	107	-78	29	107	29	107
		Hydrant #92 at 1,015 GPM	-65	45	110	45	110	-14	95	109	95	109	-79	32	111	32	111	-82	29	111	29	111
		Hydrant #60 at 850 GPM	-44	45	89	45	89	7	95	88	95	88	-58	32	90	32	90	-61	29	90	29	90
	Peak Demand 2,550 GPM	No Additional Demand	28	43	15	43	15	82	96	14	97	15	18	32	14	32	14	15	29	14	29	14
		Hydrant #35 at 1,225 GPM	-108	41	149	41	149	-54	94	148	94	148	-119	32	151	32	151	-122	29	151	29	151
		Hydrant #92 at 1,015 GPM	-112	41	153	41	153	-58	94	152	93	151	-123	32	155	32	155	-126	29	155	29	155
		Hydrant #60 at 850 GPM	-88	41	129	41	129	-34	94	128	94	128	-98	32	130	32	130	-101	29	130	29	130
Minimum Tank Level	Average Demand 725 GPM	No Additional Demand	43	45	2	45	2	94	96	2	96	2	29	32	3	32	3	26	28	2	28	2
		Hydrant #35 at 1,225 GPM	-63	44	107	44	107	-12	94	106	94	106	-76	31	107	31	107	-80	28	108	28	108
		Hydrant #92 at 1,015 GPM	-66	44	110	44	110	-15	94	109	94	109	-80	31	111	31	111	-83	28	111	28	111
		Hydrant #60 at 850 GPM	-46	44	90	44	90	6	94	88	94	88	-59	31	90	31	90	-62	28	90	28	90
	Peak Demand 2,550 GPM	No Additional Demand	27	42	15	42	15	81	96	15	96	15	17	31	14	31	14	14	28	14	28	14
		Hydrant #35 at 1,225 GPM	-109	40	149	40	149	-55	93	148	93	148	-120	31	151	31	151	-123	28	151	28	151
		Hydrant #92 at 1,015 GPM	-113	40	153	40	153	-59	93	152	93	152	-124	31	155	31	155	-127	28	155	28	155
		Hydrant #60 at 850 GPM	-89	40	129	40	129	-35	93	128	93	128	-99	31	130	31	130	-102	28	130	28	130

TABLE 43: IRESON HILL TANK SCENARIOS
12" HDPE SEASONAL LINE CLOSED
HULLS COVE SYSTEM PRESSURE (PSI) COMPARISON

		Location	Hydrant #60 US Route 3					The Colony Cottages US Route 3					Hydrant #48 Crooked Road				
		Waterline Elevation	88.9					7.5					42.5				
		Scenario	Existing	Original Zone	Δ	New Zone	Δ	Existing	Original Zone	Δ	New Zone	Δ	Existing	Original Zone	Δ	New Zone	Δ
Maximum Tank Level	Average Demand 725 GPM	No Additional Demand	79	80	1	80	1	114	115	1	116	2	99	100	1	101	2
		Hydrant #35 at 1,225 GPM	-27	74	101	74	101	8	105	97	105	97	-7	89	96	89	96
		Hydrant #92 at 1,015 GPM	-31	74	105	74	105	5	104	99	104	99	-11	88	99	88	99
		Hydrant #60 at 850 GPM	-10	74	84	74	84	32	110	78	110	78	18	95	77	95	77
	Peak Demand 2,550 GPM	No Additional Demand	66	79	13	80	14	101	114	13	115	14	86	98	12	100	14
		Hydrant #35 at 1,225 GPM	-70	72	142	72	142	-35	101	136	101	136	-50	85	135	85	135
		Hydrant #92 at 1,015 GPM	-74	71	145	71	145	-39	99	138	99	138	-54	83	137	83	137
		Hydrant #60 at 850 GPM	-50	71	121	71	121	-7	106	113	106	113	-21	91	112	91	112
Minimum Tank Level	Average Demand 725 GPM	No Additional Demand	78	79	1	80	2	113	115	2	115	2	98	99	1	100	2
		Hydrant #35 at 1,225 GPM	-28	74	102	73	101	7	104	97	104	97	-8	88	96	88	96
		Hydrant #92 at 1,015 GPM	-32	73	105	73	105	3	103	100	103	100	-12	87	99	87	99
		Hydrant #60 at 850 GPM	-11	73	84	73	84	31	109	78	109	78	17	94	77	94	77
	Peak Demand 2,550 GPM	No Additional Demand	65	78	13	79	14	100	113	13	114	14	85	97	12	99	14
		Hydrant #35 at 1,225 GPM	-72	71	143	71	143	-36	100	136	100	136	-51	84	135	84	135
		Hydrant #92 at 1,015 GPM	-75	70	145	70	145	-40	98	138	98	138	-55	82	137	82	137
		Hydrant #60 at 850 GPM	-51	70	121	70	121	-8	105	113	105	113	-22	90	112	90	112

TABLE 44: IRESON HILL TANK SCENARIOS

12" HDPE SEASONAL LINE CLOSED
DOWN-ISLAND SYSTEM PRESSURE (PSI) COMPARISON

Location			Hydrant #34 Eden Street					Ferry Terminal Eden Street					Hydrant #104 Ells Pier				
Waterline Elevation			42.9					48.8					15.4				
Scenario			Existing	Original Zone	Δ	New Zone	Δ	Existing	Original Zone	Δ	New Zone	Δ	Existing	Original Zone	Δ	New Zone	Δ
Maximum Tank Level	Average Demand 725 GPM	No Additional Demand	99	100	1	100	1	96	97	1	97	1	111	111	0	111	0
		Hydrant #35 at 1,225 GPM	3	77	74	76	73	3	75	72	75	72	106	109	3	109	3
		Hydrant #92 at 1,015 GPM	29	91	62	91	62	28	89	61	89	61	107	110	3	110	3
		Hydrant #60 at 850 GPM	47	96	49	96	49	46	93	47	93	47	107	110	3	110	3
	Peak Demand 2,550 GPM	No Additional Demand	87	96	9	100	13	84	93	9	91	7	102	105	3	104	2
		Hydrant #35 at 1,225 GPM	-38	68	106	68	106	-38	66	104	66	104	93	100	7	100	7
		Hydrant #92 at 1,015 GPM	-7	85	92	84	91	-8	82	90	82	90	95	102	7	102	7
		Hydrant #60 at 850 GPM	15	91	76	91	76	14	89	75	89	75	96	103	7	103	7
Minimum Tank Level	Average Demand 725 GPM	No Additional Demand	98	99	1	99	1	95	96	1	96	1	110	110	0	110	0
		Hydrant #35 at 1,225 GPM	2	76	74	75	73	1	74	73	74	73	104	108	4	108	4
		Hydrant #92 at 1,015 GPM	28	90	62	90	62	27	88	61	88	61	105	109	4	109	4
		Hydrant #60 at 850 GPM	46	95	49	95	49	44	92	48	92	48	106	109	3	109	3
	Peak Demand 2,550 GPM	No Additional Demand	86	95	9	99	13	83	92	9	90	7	101	104	3	103	2
		Hydrant #35 at 1,225 GPM	-39	67	106	67	106	-39	65	104	65	104	92	99	7	99	7
		Hydrant #92 at 1,015 GPM	-8	84	92	83	91	-9	81	90	81	90	94	101	7	101	7
		Hydrant #60 at 850 GPM	13	90	77	90	77	12	88	76	88	76	95	102	7	102	7

11.7.6 CONSIDERATIONS OF GRAVITY SUMMER PUMP OR TERMINAL FERRY PUMP FILLING UP-ISLAND TANK AND JACKSON LABORATORY

The inclusion of the pump bypass in the proposed Ferry Terminal pump station would allow for gravity filling of the Ireson Hill tank; therefore, the height of the Ireson Hill tank would need to be such that the maximum hydraulic grade is equal to that of the Eagle Lake reservoir (276.5 feet) to prevent the need for an altitude valve. Even with this elevation, the static pressure at the top of Ireson Hill would be 29-psi maximum, such that it would not be recommended to lower the tank height. With the Ferry Pump Station it would be possible to have the tank height be higher than what is currently proposed, however there are existing high pressures in the system in locations with low elevations. Specifically, the lowest elevation in this part of the system is near Hulls Cove at 7.5 feet and a modeled pressure of 115-psi with the existing system and 116-psi with the proposed Ireson Hill tank. Raising the proposed tank height would further increase this pressure which would not be recommended as the pressure is already higher than recommended.

11.7.6.1 GRAVITY FILLING

The existing system typically operates by gravity to fill the Jackson Laboratory tank as the maximum water surface elevation of the tank is 11.5 feet lower than that of the Duck Brook concrete tank and the Eagle Lake reservoir. During the day, the tank is drawn down primarily by Jackson Laboratory, and during the night the system demand is such that the tank is able to fill. Since the proposed Up-Island tank has the same maximum water surface as the Duck Brook tank, the proposed tank will only ever reach its maximum level by gravity if the Duck Brook tank was also full. The tank also will only fill by gravity if the hydraulic grade at the Duck Brook tank level is greater than that of the storage tanks such that it can overcome the system headloss.

Assuming the Duck Brook tank is 1 foot below its maximum and assuming the system storage tanks drop to a level 4 feet below their maximum, the model indicates the Ireson Hill tank will fill by gravity when the overall system demand is at 1,000 GPM or less. <u>This is with the seasonal 12-inch line active</u>. During the winter months when this seasonal line is <u>inactive</u>, the model indicates that under the same tank level conditions the overall system demand must be much lower at around 550 GPM or less to fill the proposed Ireson Hill tank by gravity. During this latter simulation, the Jackson Laboratory tank is already filling.

11.7.6.2 USE OF EXISTING SUMMER EFFLUENT PUMP

Historically during periods of high system demand, such as during the tourist season, the Jackson Laboratory Tank is not always able to be filled by gravity and the Summer Pump can be used to boost system pressure to help fill the tank. Following the replacement of a section of 10-inch cast iron line on Eden Street/US Route 3 between Highbrook Road and Eagle Lake Road with 16-inch ductile iron in 2017, the Water Department has not needed to run the pump for the past few years. However, the model was used to simulate scenarios to determine if the existing pump could be used to fill both the existing Jackson Laboratory tank and the proposed Up-Island tank without over-pressurizing the low points in the system. (Please note that the demand allocations during the model simulation may not be exactly equal to what is seen every day as the model is an approximation and system demands can vary depending on time of day and time of year). It is assumed that the summer pump would be required to fill the system storage tanks during peak demand days when the tank levels are dropping. The peak hourly flow seen in the system is around 2,500 GPM; however, the overall system demand could be greater than that depending on the flow out of the Jackson Laboratory tank. When the Summer Pump is operated, it is typically turned down to less than 50 percent capacity such that the output pressure observed is raised from 23 psi to between 24 and 26 psi. The flow from the pump is therefore less than the original design point of 2,500 GPM and would never meet a peak instantaneous flow of 2,500 GPM. The Summer Pump would fill the tanks during the maximum daily flows which are reported to be around 1,450 GPM.

For the purpose of evaluating the advantages and disadvantages of the existing and the proposed pumps, a system demand of 1,200 GPM was applied to the model while assuming that the Jackson Laboratory demand is at 250 GPM such that the Jackson Laboratory tank was modeled to be filling when the pump was set to 43 percent with the existing system layout. With the Jackson Laboratory tank set at a level of 15 feet (half full and the point at which the Water Department says they use the pump), the flow from the pump at 43 percent speed was simulated to be 1,415 GPM. This is around the flow output from the pump when it is used. Under this condition, the highest pressure simulated in the system is in Hulls Cove at 117-psi. The highest pressure in the downtown area is 112-psi. It should be noted that the Summer Pump should not be operated much higher than how it is currently as it would cause very high pressures in the system, particularly in the downtown area and in the low elevation areas of Hulls Cove where pressures are already high. If the pump was set to 50 percent at the same conditions in the model, the pressure in Hulls Cove is simulated to be 120-psi and downtown is 115-psi.

The Jackson Laboratory tank was evaluated with the minimum level of the tank at 4 feet below its maximum versus the 15-foot level mentioned above. With the same 43 percent pump speed at this higher level, the increase in static head at the tank reduces the modeled pump flow to 1,320 GPM. This also results in a Hulls Cove pressure of 118-psi and a downtown pressure of 113-psi. Since the pump VFD is controlled by pressure at the pump discharge, the speed of the pump, and therefore the pump flow, will change depending on the system demand. A higher system demand would result in a greater headloss and lower system pressure requiring a higher pump speed to achieve the pressure setpoint. The pump flow will then be dependent on both system pressure and pump speed. However, for comparison purposes, the same pump speed was used to evaluate the different model scenarios.

Under the same demand conditions at 43 percent pump speed, if the Jackson Laboratory tank and the proposed Ireson Hill tank levels are set to be 4 feet below their maximum levels, the Jackson Laboratory tank is going to fill slower, as some of the pumped flow would go to the Ireson Hill tank. In addition, the model indicates that more flow goes to the Ireson Hill tank; however, this will be dependent on the local flow demand at Jackson Laboratory. If Jackson Laboratory has a lower demand, this flow will be directed to the local tank. Under these conditions, the Hulls Cove pressure is now 115-psi compared to 117-psi with no Ireson Hill tank as there is a higher flow rate through the area creating some pressure drop. Meanwhile, the Downtown pressure remains at 112-psi. The pump flow at 43 percent in this scenario is modeled to be 1,420 GPM compared to 1,320 GPM, where the additional flow is going to the Ireson Hill tank. In general, if the tanks are to be drained down, the pump may need to be run more for a longer period of time to fill the tanks. Alternatively, the pump speed could be increased; however, the model indicates that if the pump speed was increased to 50 percent at the simulated system demand, the maximum pressure in the downtown area would increase to 115-psi. A further increase in speed results in more and more areas of the downtown portion of the system going above 100-psi with the lower elevation locations above 110-psi.

The above analysis was in the event the seasonal 12-inch HDPE line from Duck Brook to the Ferry Terminal was active, which is when the Summer Pump is typically utilized. During the winter months when this line is closed, if the system demands force the tanks to drain to 4 feet below the maximum levels, it appears that the summer pump would not fill the Ireson Hill tank at the modeled 43 percent speed, and the flow would only go to the Jackson Laboratory tank. To fill Ireson Hill, the model shows that the pump speed would need to be increased to at least

48 percent under these conditions to overcome the added friction in the Eden Street/US Route 3 line to the Up-Island area. During the winter months when the system demand is much lower than that of the summer months, the system demand may drop enough such that the tanks would be able to fill by gravity, as was discussed previously.

With the proposed Ireson Hill tank in the same zone as the Jackson Laboratory tank, there is no way to direct the flow pumped from the Summer Pump to one tank or the other. Hydraulic headloss, tank levels, and system demands will dictate the direction of the flow. By separating the tanks with a pump station for the Up-Island tank including the gravity pump bypass, the Summer Pump will still be able to fill the Ireson Hill tank through the Ferry pump bypass and the flows to each tank under the same conditions as previously evaluated are essentially the same. <u>This is with the seasonal line active</u>. Should the Jackson Laboratory tank need to be filled, the proposed pump station could include a valve such that the flow from the Summer Tank could be prevented from going to the Ireson Hill tank. The Summer Pump flow would then only go to the Jackson Laboratory tank. This could only be done with the proposed Ferry Terminal Pump Station. The valve in the pump station could be automatic and controlled through SCADA such that the Water Department could open or close the valve as needed when the Summer Pump is active.

11.7.6.3 Using Ferry Terminal Pump to Fill Ireson Hill Tank

A preliminary pump was selected to model the proposed Ferry Terminal Pump Station and evaluate the Up-Island pressures when filling the proposed Ireson Hill tank. The model was first used to determine the effects of a pump on the region at varying flows. This essentially created a system curve for the Up-Island area. From this, a maximum flow of the pump was determined based on the maximum pressures seen in the area when the pump was "active". As discussed, some areas of the Up-Island region already have high pressures due to low elevations. Therefore, boosted pressure in these areas should be limited. The model shows that a pump at around 580 GPM at 50-feet TDH under peak conditions would result in a maximum pressure of 120-psi in the low elevation area of Hulls Cove. Since the Up-Island area already has a high pressure of around 115-psi, a pump flow was selected such that the pressure was not drastically increased. Based on this, the preliminary pump design flow is 530 GPM at 41-feet TDH such that the maximum pressure in the Up-Island area is 119-psi. The peak demand in the Up-Island area of the distribution system is assumed to be around 210 GPM, therefore at a flow of 530 GPM, the pump would still be able to fill the proposed Ireson Hill tank under peak demands. This is not the case with the Summer Pump as the peak system demand is around 2,500 GPM and the pump's design flow is 2,500 GPM.

The preliminary pump used in the model is a 15 HP Goulds 3656 M Series 6x8 Pump with a 10-inch trimmed impeller. Under the same demand conditions evaluated with the Summer Pump and with the Ireson Hill tank 4 feet below the system's maximum hydraulic grade, the flow to the tank with the Ferry pump is 3.25 times that from the Summer Pump. In addition, with the Ferry Pump on, the pressures just upstream of the pump station drop only a few psi. The low elevation of this area allows the pressure to remain reasonable. This analysis was with the seasonal 12-inch Duck Brook HDPE line active as is the case during the summer months.

If this line is inactive, the suction pressure observed at the pump is reduced due to the increased friction loss through the Eden Street/US Route 3 line. This causes the overall boosted pressure from the pump to be reduced, and therefore the flow to the tank is also reduced. Under the same conditions as above with the exception of the seasonal line operation, the pump flow is modeled to be 330 GPM versus 530 GPM with the bypass line open and the maximum Up-Island pressure is 115-psi versus 119-psi. The tank is still filling but the rate is only 1.6 times that from the Summer Pump versus 3.25 times with the seasonal line open. While there was little impact on the upstream pressures with the pump on with the seasonal line open (the pressure drop in the immediate area was only around 2-psi), the model indicates that with the seasonal line inactive, the pressure drop is around 10-psi. While the latter is greater, the pressure remains above 75-psi. This is during the normal flows with no hydrants active.

With the addition of the Ireson Hill tank, it would be expected that the pressures in the area of Eden Street/US Route 3 south of the Ferry Terminal would improve at least during additional hydrant flow conditions as the hydrants would pull not just from the Duck Brook concrete tank, but also from the Ireson Hill tank. For this analysis, three hydrants along Eden Street/US Route 3 were simulated to be opened from the 2-½-inch nozzle with the existing system layout. The same flow was then applied as a demand for the subsequent scenarios to compare the effects on the resulting pressure with the proposed Ireson Hill tank as part of the original system and with the tank separated by the pump station and bypass check valve. Note that the actual flow from the hydrant will vary depending on tank levels, tank zone scenario, and system demands. As shown in Table 45, there were very minor improvements to the Eden Street area with the inclusion of the proposed Ireson Hill tank with the largest pressure improvement of 5-psi seen in the vicinity of the Ferry Terminal. The area pressures are already between 70 and 98-psi, therefore the pressure improvements are not significant. These improvements also reduce in magnitude as the distance from the Ferry Terminal increases. If the Up-Island region is separated by a pump station and gravity bypass line with a check valve, the improvements seen are reduced with the largest improvement of around 3-psi near the Ferry Terminal. The minor improvements in this case are seen due to the seasonal 12-inch HDPE line that now contributes to Eden Street rather than contributing to the Up-Island area.

11.7.6.4 IMPACT OF SHUTTING OFF THE SEASONAL 12" HDPE LINE

When the seasonal 12-inch HDPE line is inactive, the hydrant capacities in general will decrease significantly with the existing distribution system. Based on the resulting pressures in the model, it is unlikely the applied flows would be achieved by gravity flow at the hydrants. This is particularly true for the Hydrant #23 that is closest to the location of the seasonal line on Eden Street/US Route 3. However, for analysis purposes, the same flows were applied to the hydrants in the model to compare pressures. As shown in Table 46, the improvements with the tank as part of the original zone are significant, especially for the hydrants closest to the Ferry Terminal. These improvements are greatly reduced when the Up-Island is split by the pump station, as they would no longer draw from the proposed Ireson Hill tank due to the check valve at the pump station. There are still some improvements to the area, likely due to the fact that the Up-Island demands are supplied by the proposed Ireson Hill tank versus the Duck Brook tank. We therefore determined that the Town should consider looping Highbrook Road to minimize the impact to hydrants when the 12" Ø line is shut down for the season as evaluated in Section 11.7.7.

TABLE 45: IRESON HILL TANK SCENARIO

12" HDPE SEASONAL LINE OPEN
LOWER EDEN STREET SYSTEM PRESSURE (PSI) COMPARISON

		Location	Ferry Terminal					Hydrant #24					Hydrant #23					Hydrant #37					Hydrant #32					Hydrant #36				
		Waterline Elevation	48					56.8					69.3					52					61.5					58.2				
		Scenario	Existing	Original Zone	Δ	New Zone	Δ	Existing	Original Zone	Δ	New Zone	Δ	Existing	Original Zone	Δ	New Zone	Δ	Existing	Original Zone	Δ	New Zone	Δ	Existing	Original Zone	Δ	New Zone	Δ	Existing	Original Zone	Δ	New Zone	Δ
Maximum Tank Level	Average Demand 725 GPM	No Additional Demand	98	98	0	98	0	94	94	0	94	0	88	89	1	88	0	96	96	0	96	0	92	92	0	92	0	93	93	0	93	0
		Hydrant #23 at 1,330 GPM	92	94	2	93	1	86	88	2	86	0	73	75	2	74	1	86	87	1	87	1	86	87	1	86	0	90	90	0	90	0
		Hydrant #37 at 1,385 GPM	94	95	1	94	0	89	90	1	89	0	79	80	1	79	0	79	79	0	79	0	82	83	1	82	0	89	89	0	89	0
		Hydrant #36 at 1,475 GPM	96	97	1	96	0	92	92	0	92	0	85	86	1	86	1	91	92	1	92	1	86	87	1	87	1	87	88	1	87	0
	Peak Demand 2,550 GPM	No Additional Demand	95	97	2	96	1	91	92	1	92	1	85	86	1	86	1	92	93	1	93	1	88	88	0	88	0	89	89	0	89	0
		Hydrant #23 at 1,330 GPM	87	92	5	90	3	80	84	4	83	3	67	70	3	68	1	79	82	3	80	1	79	80	1	80	1	83	84	1	83	0
		Hydrant #37 at 1,385 GPM	89	93	4	91	2	83	87	4	85	2	73	76	3	74	1	71	73	2	72	1	74	76	2	75	1	81	82	1	82	1
		Hydrant #36 at 1,475 GPM	91	94	3	93	2	87	89	2	88	1	79	81	2	80	1	84	85	1	85	1	78	79	1	79	1	78	79	1	79	1
Minimum Tank Level	Average Demand 725 GPM	No Additional Demand	97	97	0	97	0	93	93	0	93	0	87	88	1	88	1	95	95	0	95	0	91	91	0	91	0	92	92	0	92	0
		Hydrant #23 at 1,330 GPM	91	94	3	92	1	85	87	2	85	0	72	74	2	73	1	85	86	1	86	1	85	85	0	85	0	89	89	0	89	0
		Hydrant #37 at 1,385 GPM	93	94	1	93	0	88	89	1	88	0	78	79	1	78	0	77	78	1	78	1	81	81	0	81	0	88	88	0	88	0
		Hydrant #36 at 1,475 GPM	95	96	1	95	0	91	91	0	91	0	84	85	1	85	1	90	91	1	90	0	85	86	1	85	0	86	86	0	86	0
	Peak Demand 2,550 GPM	No Additional Demand	95	96	1	95	0	91	91	0	91	0	85	85	0	85	0	92	92	0	92	0	88	87	-1	87	-1	89	88	-1	88	-1
		Hydrant #23 at 1,330 GPM	86	91	5	89	3	80	84	4	82	2	66	69	3	67	1	78	81	3	79	1	78	79	1	78	0	82	83	1	82	0
		Hydrant #37 at 1,385 GPM	88	92	4	90	2	82	86	4	84	2	72	75	3	73	1	70	72	2	71	1	73	75	2	74	1	80	81	1	81	1
		Hydrant #36 at 1,475 GPM	90	93	3	92	2	86	88	2	87	1	78	80	2	79	1	83	84	1	84	1	77	78	1	78	1	77	78	1	78	1

TABLE 46: IRESON HILL TANK SCENARIO

12" HDPE SEASONAL LINE CLOSED
LOWER EDEN STREET SYSTEM PRESSURE (PSI) COMPARISON

		Location	Ferry Terminal					Hydrant #24					Hydrant #23					Hydrant #37					Hydrant #32					Hydrant #36				
		Waterline Elevation	48					56.8					69.3					52					61.5					58.2				
		Scenario	Existing	Original Zone	Δ	New Zone	Δ	Existing	Original Zone	Δ	New Zone	Δ	Existing	Original Zone	Δ	New Zone	Δ	Existing	Original Zone	Δ	New Zone	Δ	Existing	Original Zone	Δ	New Zone	Δ	Existing	Original Zone	Δ	New Zone	Δ
Maximum Tank Level	Average Demand 725 GPM	No Additional Demand	96	97	1	97	1	93	94	1	93	0	87	88	1	88	1	95	96	1	95	0	91	92	1	91	0	93	93	0	93	0
		Hydrant #23 at 1,330 GPM	14	74	60	20	6	10	69	59	16	6	5	60	55	11	6	49	80	31	53	4	68	83	15	70	2	87	90	3	87	0
		Hydrant #37 at 1,385 GPM	48	82	34	51	3	44	77	33	48	4	39	70	31	42	3	46	73	27	50	4	67	80	13	68	1	86	89	3	87	1
		Hydrant #36 at 1,475 GPM	90	94	4	90	0	86	90	4	87	1	80	84	4	81	1	88	91	3	89	1	84	86	2	85	1	86	87	1	86	0
	Peak Demand 2,550 GPM	No Additional Demand	85	94	9	91	6	81	90	9	87	6	76	84	8	82	6	86	91	5	90	4	84	87	3	86	2	86	89	3	88	2
		Hydrant #23 at 1,330 GPM	-24	65	89	1	25	-27	60	87	-2	25	-32	51	83	-8	24	24	71	47	38	14	51	75	24	58	7	75	83	8	77	2
		Hydrant #37 at 1,385 GPM	19	74	55	36	17	16	69	53	32	16	11	61	50	27	16	21	64	43	34	13	49	71	22	56	7	75	81	6	77	2
		Hydrant #36 at 1,475 GPM	72	87	15	79	7	69	82	13	75	6	64	76	12	70	6	74	82	8	78	4	71	78	7	74	3	74	79	5	76	2
Minimum Tank Level	Average Demand 725 GPM	No Additional Demand	95	97	2	96	1	92	93	1	92	0	86	87	1	87	1	94	95	1	94	0	90	91	1	90	0	92	92	0	82	-10
		Hydrant #23 at 1,330 GPM	13	73	60	19	6	9	68	59	15	6	3	59	56	10	7	48	78	30	52	4	67	82	15	69	2	85	89	4	86	1
		Hydrant #37 at 1,385 GPM	47	81	34	50	3	43	76	33	46	3	37	69	32	41	4	45	72	27	49	4	66	78	12	67	1	85	88	3	85	0
		Hydrant #36 at 1,475 GPM	88	93	5	89	1	85	89	4	85	0	79	83	4	80	1	87	90	3	88	1	83	85	2	84	1	85	86	1	85	0
	Peak Demand 2,550 GPM	No Additional Demand	84	93	9	90	6	80	89	9	86	6	75	83	8	81	6	85	90	5	89	4	83	86	3	85	2	85	88	3	87	2
		Hydrant #23 at 1,330 GPM	-25	64	89	0	25	-28	59	87	-4	24	-33	50	83	-9	24	23	70	47	37	14	50	74	24	57	7	74	82	8	76	2
		Hydrant #37 at 1,385 GPM	18	73	55	35	17	14	68	54	31	17	10	60	50	26	16	20	63	43	33	13	48	70	22	55	7	74	80	6	75	1
		Hydrant #36 at 1,475 GPM	71	86	15	78	7	67	81	14	74	7	63	75	12	69	6	73	81	8	77	4	70	77	7	73	3	73	78	5	75	2

11.7.7 CONSIDER LOOPING HIGHBROOK ROAD TO MINIMIZE IMPACT OF SHUTTING OFF 12" Ø HDPE LINE

A way to remediate the reduction in hydrant capacity for the lower Eden Street hydrants with a separated Up-Island zone is to make the seasonal HDPE line a year-round line. As this line is approximately 2,125-LF of cross country line, this endeavor might not be feasible. A second option would be to complete the loop on the north end of Highbrook Road near Hydrant #46 to the 10-inch cast iron line on Eden Street/US Route 3. The existing line on this end of Highbrook Road is mostly 6-inch ductile iron that was installed around 1984, however 381-LF at the end of the line is 6-inch cast iron. It would require 845-LF of 8-inch ductile iron line to complete the loop on this street, including the replacement of the 6-inch cast iron portion. Table 47 provides resulting pressures of the lower Eden Street line with the Highbrook loop. Comparing the results from Tables 46 and 47, the Highbrook loop will provide significant improvements to the lower Eden Street area when the seasonal 12" Ø HDPE line is inactive and if the Up-Island tank is separated from the original zone. While the hydrant capacities of the more isolated hydrants in this area such as Hydrant #23 may not be the same as with the seasonal line active, the capacities are going to be greatly improved from that of those without the Highbrook Road loop. Additional benefits would also be seen by removing the existing 6-inch cast iron line such as water quality and the prevention of water stagnation at the end of the existing dead-end line. Hydrant #46 located on this end of Highbrook Road may also see improved flow capacity.

TABLE 47: IRESON HILL TANK SCENARIO

HIGHBROOK ROAD LOOP
12" HDPE SEASONAL LINE CLOSED
LOWER EDEN STREET SYSTEM PRESSURE (PSI) COMPARISON

		Location	Ferry Terminal					Hydrant #24					Hydrant #23				
		Waterline Elevation	48					56.8					69.3				
		Scenario	Existing	Original Zone	Δ	New Zone	Δ	Existing	Original Zone	Δ	New Zone	Δ	Existing	Original Zone	Δ	New Zone	Δ
Maximum Tank Level	Average Demand 725 GPM	No Additional Demand	96	98	2	97	1	93	94	1	93	0	87	88	1	88	1
		Hydrant #23 at 1,330 GPM	14	81	67	55	41	10	77	67	51	41	5	69	64	46	41
		Hydrant #37 at 1,385 GPM	48	90	42	79	31	44	86	42	75	31	39	79	40	70	31
		Hydrant #36 at 1,475 GPM	90	95	5	92	2	86	91	5	88	2	80	85	5	83	3
	Peak Demand 2,550 GPM	No Additional Demand	85	94	9	92	7	81	90	9	89	8	76	85	9	83	7
		Hydrant #23 at 1,330 GPM	-24	74	98	43	67	-27	69	96	39	66	-32	61	93	34	66
		Hydrant #37 at 1,385 GPM	19	83	64	69	50	16	79	63	65	49	11	72	61	59	48
		Hydrant #36 at 1,475 GPM	72	89	17	82	10	69	85	16	78	9	64	78	14	73	9
Minimum Tank Level	Average Demand 725 GPM	No Additional Demand	95	97	2	96	1	92	93	1	92	0	86	87	1	87	1
		Hydrant #23 at 1,330 GPM	13	80	67	54	41	9	76	67	50	41	3	68	65	45	42
		Hydrant #37 at 1,385 GPM	47	89	42	78	31	43	85	42	74	31	37	78	41	69	32
		Hydrant #36 at 1,475 GPM	88	94	6	91	3	85	90	5	87	2	79	84	5	82	3
	Peak Demand 2,550 GPM	No Additional Demand	84	93	9	91	7	80	89	9	88	8	75	84	9	82	7
		Hydrant #23 at 1,330 GPM	-25	73	98	42	67	-28	68	96	38	66	-33	60	93	33	66
		Hydrant #37 at 1,385 GPM	18	82	64	68	50	14	78	64	64	50	10	71	61	58	48
		Hydrant #36 at 1,475 GPM	71	88	17	81	10	67	84	17	77	10	63	77	14	72	9

		Location	Hydrant #37					Hydrant #32					Hydrant #36				
		Waterline Elevation	52					61.5					58.2				
		Scenario	Existing	Original Zone	Δ	New Zone	Δ	Existing	Original Zone	Δ	New Zone	Δ	Existing	Original Zone	Δ	New Zone	Δ
Maximum Tank Level	Average Demand 725 GPM	No Additional Demand	95	96	1	95	0	91	92	1	91	0	93	93	0	93	0
		Hydrant #23 at 1,330 GPM	49	87	38	79	30	68	86	18	82	14	87	90	3	88	1
		Hydrant #37 at 1,385 GPM	46	81	35	75	29	67	83	16	80	13	86	89	3	88	2
		Hydrant #36 at 1,475 GPM	88	92	4	90	2	84	87	3	85	1	86	87	1	87	1
	Peak Demand 2,550 GPM	No Additional Demand	86	92	6	91	5	84	88	4	87	3	86	89	3	88	2
		Hydrant #23 at 1,330 GPM	24	80	56	69	45	51	79	28	73	22	75	83	8	80	5
		Hydrant #37 at 1,385 GPM	21	74	53	65	44	49	76	27	70	21	75	82	7	79	4
		Hydrant #36 at 1,475 GPM	74	84	10	80	6	71	79	8	76	5	74	79	5	77	3
Minimum Tank Level	Average Demand 725 GPM	No Additional Demand	94	95	1	95	1	90	91	1	90	0	92	92	0	92	0
		Hydrant #23 at 1,330 GPM	48	86	38	78	30	67	85	18	81	14	85	89	4	87	2
		Hydrant #37 at 1,385 GPM	45	80	35	74	29	66	82	16	79	13	85	88	3	87	2
		Hydrant #36 at 1,475 GPM	87	90	3	89	2	83	85	2	84	1	85	86	1	85	0
	Peak Demand 2,550 GPM	No Additional Demand	85	91	6	90	5	83	87	4	86	3	85	88	3	87	2
		Hydrant #23 at 1,330 GPM	23	79	56	68	45	50	78	28	71	21	74	82	8	78	4
		Hydrant #37 at 1,385 GPM	20	73	53	64	44	48	75	27	69	21	74	81	7	78	4
		Hydrant #36 at 1,475 GPM	73	83	10	79	6	70	78	8	75	5	73	78	5	76	3

11.7.8 CONSIDER RAISING THE ELEVATION OF THE JACKSON LABORATORY TANK

As discussed, the overflow elevation of the Jackson Laboratory tank is 265 feet, while the overflow elevation of the Duck Brook concrete tank and the maximum water surface elevation of the Eagle Lake reservoir is 276.5 feet. This elevation difference requires the Jackson Laboratory tank to include an altitude valve to prevent it from overflowing. A failure in this valve would result in loss of water when the tank is full, which occurs often during the winter season. As the tank typically fills by gravity, the Water Department has a missed opportunity for increased tank volume, as well as increased local hydraulic grade, that may help with local pressures and hydrant flow capacity by keeping the tank at the existing maximum level. The developed model was used to determine the pressure benefits of raising the existing Jackson Laboratory tank height to meet that of the Duck Brook tank and the Eagle Lake reservoir. Note that the maximum elevation of the latter is 276.5 feet as mentioned, however recent survey found the maximum surface elevation of the reservoir to be 275 feet which was the elevation used in the model simulation. If the tank height was to be raised, it should be raised such that the overflow elevation is equal to 276.5 feet such that no altitude valve would be required. The tank could then fill by gravity without overflowing. Note that to meet this maximum level, the Duck Brook concrete tank would need to be full. This would likely cause more operation of the Summer Pump.

Also note that the area of Jackson Laboratory was calibrated assuming the local pipes were similar in condition and roughness to those in the rest of the system. Flow testing in this area was limited as many of the hydrants were deemed as unsafe to flow. Therefore, results in this area should be seen as approximate and the model could be improved/verified, especially in this area. Similar to the lower Eden Street/US Route 3 hydrant flow analysis, three hydrants in the Jackson Laboratory area were simulated to be opened from the 2-½-inch nozzle with the existing system layout. The same flow was then applied as a demand for the subsequent scenarios to compare the effects on the resulting pressure with the raised Jackson Laboratory tank height. Again, please note that the actual flow from the hydrant will vary depending on tank levels and system demands. In this area specifically, the major system demand will be the operation of the Jackson Laboratory.

Tables 48 and 49 provide pressure results with the increased height of the Jackson Laboratory tank. The model indicates the local Jackson Laboratory area pressure increases by 2 to 4 psi while downtown areas and beyond occasionally increase by 1 to 2 psi if at all. Looking at the downtown area, these pressures increase slightly but are fairly close to the same pressure as it was before. It appears raising the tank will not impact system pressures greatly but would be helpful to provide about 167,000 gallons of additional water volume. This would reduce the impact on the rest of the system and keep the tank from dropping too low during high-consumption periods.

TABLE 48: JACKSON LABORATORY TANK HEIGHT INCREASE SCENARIO

JACKSON LABORATORY AREA HYDRANTS SYSTEM PRESSURES (PSI) COMPARISON

		Location	End of Seely Road			Hydrant #91 Schooner Head Road			Jackson Laboratory			East Strawberry Hill			Hydrant #84 Old Farm Road			End of Spring Street			Hydrant #104 Ells Pier			Kebo Ridge Road			End of Cleftstone Road		
		Waterline Elevation	28			91.45			136.25			182			111.25			95			15.4			207			211.45		
		Scenario	Existing Height	New Height	Δ	Existing Height	New Height	Δ	Existing Height	New Height	Δ	Existing Height	New Height	Δ	Existing Height	New Height	Δ	Existing Height	New Height	Δ	Existing Height	New Height	Δ	Existing Height	New Height	Δ	Existing Height	New Height	Δ
Maximum Tank Level	Average Demand 725 GPM	No Additional Demand	102	106	4	75	78	3	55	59	4	36	39	3	67	70	3	76	77	1	111	112	1	28	29	1	27	27	0
		Hydrant #70 at 1,420 GPM	96	99	3	69	72	3	49	52	3	29	32	3	60	62	2	69	70	1	106	107	1	25	25	0	24	25	1
		Hydrant #84 at 510 GPM	100	103	3	72	75	3	53	56	3	33	36	3	13	15	2	76	76	0	110	111	1	28	28	0	27	27	0
		Hydrant #93 at 420 GPM	21	24	3	17	20	3	53	56	3	34	37	3	65	68	3	76	77	1	111	111	0	28	29	1	27	27	0
	Peak Demand 2,550 GPM	No Additional Demand	95	97	2	67	70	3	45	47	2	29	31	2	59	62	3	70	71	1	104	105	1	24	24	0	24	24	0
		Hydrant #70 at 1,420 GPM	81	84	3	54	56	2	32	34	2	15	17	2	45	47	2	58	59	1	95	95	0	17	17	0	19	19	0
		Hydrant #84 at 510 GPM	88	91	3	60	63	3	38	41	3	21	24	3	0	3	3	68	69	1	103	103	0	22	23	1	23	23	0
		Hydrant #93 at 420 GPM	8	11	3	3	6	3	39	42	3	23	26	3	54	57	3	69	70	1	103	104	1	23	23	0	23	23	0
Minimum Tank Level	Average Demand 725 GPM	No Additional Demand	101	104	3	73	77	4	54	57	3	35	38	3	65	69	4	75	76	1	110	111	1	27	28	1	26	26	0
		Hydrant #70 at 1,420 GPM	95	98	3	68	71	3	48	51	3	28	30	2	58	61	3	68	69	1	105	106	1	24	24	0	24	24	0
		Hydrant #84 at 510 GPM	98	101	3	71	74	3	51	54	3	32	35	3	11	14	3	75	75	0	109	110	1	27	27	0	26	26	0
		Hydrant #93 at 420 GPM	20	23	3	15	18	3	52	55	3	33	35	2	64	66	2	75	76	1	110	110	0	27	28	1	26	26	0
	Peak Demand 2,550 GPM	No Additional Demand	93	96	3	66	68	2	43	46	3	27	30	3	58	60	2	69	70	1	103	104	1	23	23	0	23	23	0
		Hydrant #70 at 1,420 GPM	80	82	2	53	55	2	30	33	3	13	15	2	44	46	2	57	58	1	94	94	0	16	16	0	18	18	0
		Hydrant #84 at 510 GPM	86	89	3	59	62	3	37	39	2	20	23	3	-1	1	2	67	68	1	102	102	0	21	22	1	22	22	0
		Hydrant #93 at 420 GPM	7	9	2	2	5	3	37	40	3	22	24	2	53	55	2	68	69	1	102	103	1	22	22	0	22	22	0

TABLE 49: JACKSON LABORATORY TANK HEIGHT INCREASE SCENARIO

TOWN-WIDE HYDRANTS
SYSTEM PRESSURES (PSI) COMPARISON

		Location	End of Seely Road			Hydrant #91 Schooner Head Road			Jackson Laboratory			East Strawberry Hill			Hydrant #84 Old Farm Road			End of Spring Street			Hydrant #104 Ells Pier			Kebo Ridge Road			End of Cleftstone Road		
		Waterline Elevation	28			91.45			136.25			182			111.25			95			15.4			207			211.45		
		Scenario	Existing Height	New Height	Δ	Existing Height	New Height	Δ	Existing Height	New Height	Δ	Existing Height	New Height	Δ	Existing Height	New Height	Δ	Existing Height	New Height	Δ	Existing Height	New Height	Δ	Existing Height	New Height	Δ	Existing Height	New Height	Δ
Maximum Tank Level	Average Demand 725 GPM	No Additional Demand	55	59	4	75	78	3	55	59	4	36	39	3	67	70	3	76	77	1	111	112	1	28	29	1	27	27	0
		Hydrant #16 at 825 GPM	54	57	3	74	77	3	54	57	3	35	37	2	66	68	2	73	74	1	106	107	1	26	27	1	25	26	1
		Hydrant #104 at 1,300 GPM	53	56	3	73	75	2	53	56	3	33	36	3	64	67	3	71	72	1	96	97	1	24	25	1	24	24	0
		Hydrant #108 at 700 GPM	55	58	3	75	77	2	55	58	3	36	38	2	66	69	3	74	75	1	109	110	1	-6	-5	1	25	26	1
	Peak Demand 2,550 GPM	No Additional Demand	45	47	2	67	70	3	45	47	2	29	31	2	59	62	3	70	71	1	104	105	1	24	24	0	24	24	0
		Hydrant #16 at 825 GPM	41	43	2	63	66	3	41	43	2	24	27	3	55	57	2	64	65	1	96	97	1	19	20	1	20	21	1
		Hydrant #104 at 1,300 GPM	38	40	2	61	62	1	38	40	2	21	23	2	52	54	2	59	61	2	82	84	2	15	16	1	18	19	1
		Hydrant #108 at 700 GPM	43	45	2	65	67	2	43	45	2	26	28	2	57	59	2	66	67	1	100	101	1	-13	-12	1	21	21	0
Minimum Tank Level	Average Demand 725 GPM	No Additional Demand	54	57	3	73	77	4	54	57	3	35	38	3	65	69	4	75	76	1	110	111	1	27	28	1	26	26	0
		Hydrant #16 at 825 GPM	53	56	3	73	75	2	53	56	3	33	36	3	64	67	3	72	73	1	105	106	1	25	26	1	24	25	1
		Hydrant #104 at 1,300 GPM	52	54	2	71	74	3	52	54	2	32	35	3	63	65	2	69	71	2	95	96	1	23	24	1	23	23	0
		Hydrant #108 at 700 GPM	53	56	3	73	76	3	53	56	3	34	37	3	65	67	2	73	74	1	107	109	2	-7	-6	1	24	25	1
	Peak Demand 2,550 GPM	No Additional Demand	43	46	3	66	68	2	43	46	3	27	30	3	58	60	2	69	70	1	103	104	1	23	23	0	23	23	0
		Hydrant #16 at 825 GPM	40	42	2	62	64	2	40	42	2	23	25	2	54	56	2	62	64	2	94	96	2	18	19	1	19	20	1
		Hydrant #104 at 1,300 GPM	37	39	2	59	61	2	37	39	2	20	22	2	51	53	2	58	60	2	81	82	1	14	15	1	17	18	1
		Hydrant #108 at 700 GPM	41	44	3	64	66	2	41	44	3	25	27	2	56	58	2	65	66	1	99	100	1	-14	-13	1	20	20	0

11.7.9 EVALUATION OF UPSIZING SMALL DIAMETER PIPING IN DOWNTOWN AREA

Main Street in the southern portion of downtown has approximately 2,360-LF of 6-inch cast iron piping from the intersection of Cromwell Harbor Road and Main Street almost to Newton Way where it transitions to 8-inch cast iron. Historical reports indicate that this line was installed between the late 1890s to the early 1920s. The model was used to evaluate the benefits of increasing this line to either 8-inch or 12-inch ductile iron. Tables 50 and 51 show that Town-wide pressures are not greatly impacted by the increase in pipe size. Upgrading obsolete piping should be completed over time; however, not all areas result in notable improvements when modeled.

Tables 52 and 53 show pressure results in specific locations where old water lines on dead-end roads will be suggested to be upgraded in addition to the Main Street upgrade. Cottage Way is currently 2-inch HDPE and galvanized steel. With the recommended increase to 4-inch ductile iron, there is minimal pressure improvements seen during additional hydrant flow conditions. The same is true for when the local Hydrant #13 is simulated to be flowing open versus the other three hydrants indicated in the tables. The same can be said for Stephens Lane which is recommended to be updated from 2-inch galvanized to 8-inch ductile iron. The pressure improvements seen on Albert Meadows and Atlantic Avenue are significant when the local Hydrant #16 is flowed; however, this could also be attributed to fixing the broken valve on Atlantic Avenue and connecting Derby Lane and Grason Lane to the existing 8-inch ductile iron line on Elbow Lane. The existing configuration of this portion of the system in combination with the broken valve that is broken closed is limiting the flow through only Albert Meadows. The upgrade on Hancock Street shows at least a 10-psi improvement when the local Hydrant #72 is flowed. The upgrade of Stanwood Place from 3-inch galvanized to 4-inch ductile iron sees at least 5-psi improvement with Hydrant #71 flowing and at least 9-psi improvement with Hydrant #72 flowing. Center Street, Snow Street, and Oliver Street see a significant improvement of at least 21-psi when Hydrant #71 is flowed following the upgrade of Center Street and Oliver Street. Finally, Scotts Lane sees only 5-psi improvement after upgrading from 2-inch galvanized to 8-inch ductile iron.

TABLE 50: MAIN STREET 6-INCH CAST IRON UPGRADE SCENARIO DOWNTOWN LOCAL SYSTEM PRESSURE (PSI) COMPARISON

		Location	Atlantic Avenue, Newton Way, and Main Street					Kebo Ridge Road					Hydrant #104 Ellis Pier					Corner of Eden Street, Eagle Lake Road, and Mount Desert Street					End of Cleftstone Road				
		Waterline Elevation	44.3					207					15.4					78.75					211.45				
		Scenario	Existing	8" DI	Δ	12" DI	Δ	Existing	8" DI	Δ	12" DI	Δ	Existing	8" DI	Δ	12" DI	Δ	Existing	8" DI	Δ	12" DI	Δ	Existing	8" DI	Δ	12" DI	Δ
Maximum Tank Level	Average Demand 725 GPM	No Additional Demand	98	98	0	98	0	28	28	0	28	0	111	111	0	111	0	84	84	0	84	0	27	27	0	27	0
		Hydrant #70 at 1,420 GPM	93	91	-2	90	-3	25	24	-1	24	-1	106	105	-1	105	-1	80	80	0	80	0	24	24	0	24	0
		Hydrant #16 at 825 GPM	93	94	1	94	1	26	26	0	26	0	106	106	0	106	0	81	81	0	81	0	25	25	0	25	0
		Hydrant #104 at 1,300 GPM	90	91	1	92	2	24	24	0	24	0	96	96	0	96	0	79	79	0	79	0	24	24	0	24	0
	Peak Demand 2,550 GPM	No Additional Demand	92	92	0	92	0	24	24	0	24	0	104	104	0	104	0	79	79	0	79	0	24	24	0	24	0
		Hydrant #70 at 1,420 GPM	81	79	-2	78	-3	17	16	-1	16	-1	95	93	-2	93	-2	72	72	0	71	-1	19	19	0	18	-1
		Hydrant #16 at 825 GPM	83	84	1	84	1	19	19	0	19	0	96	96	0	96	0	74	74	0	74	0	20	20	0	20	0
		Hydrant #104 at 1,300 GPM	78	79	1	80	2	15	16	1	16	1	82	83	1	83	1	71	71	0	71	0	18	18	0	18	0
Minimum Tank Level	Average Demand 725 GPM	No Additional Demand	97	97	0	97	0	27	27	0	27	0	110	110	0	110	0	83	83	0	83	0	26	26	0	26	0
		Hydrant #70 at 1,420 GPM	92	90	-2	89	-3	24	23	-1	23	-1	105	104	-1	104	-1	79	79	0	79	0	24	23	-1	23	-1
		Hydrant #16 at 825 GPM	92	93	1	93	1	25	25	0	25	0	105	105	0	105	0	80	80	0	80	0	24	24	0	24	0
		Hydrant #104 at 1,300 GPM	89	90	1	91	2	23	23	0	23	0	95	95	0	95	0	78	78	0	78	0	23	23	0	23	0
	Peak Demand 2,550 GPM	No Additional Demand	91	91	0	91	0	23	23	0	23	0	103	103	0	103	0	78	78	0	78	0	23	23	0	23	0
		Hydrant #70 at 1,420 GPM	80	78	-2	77	-3	16	15	-1	15	-1	94	92	-2	92	-2	71	71	0	70	-1	18	18	0	17	-1
		Hydrant #16 at 825 GPM	82	83	1	83	1	18	18	0	18	0	94	95	1	95	1	73	73	0	73	0	19	19	0	19	0
		Hydrant #104 at 1,300 GPM	77	78	1	79	2	14	14	0	15	1	81	82	1	82	1	70	70	0	70	0	17	17	0	17	0

TABLE 51: MAIN STREET 6-INCH CAST IRON UPGRADE SCENARIO
JACKSON LABORATORY LOCAL SYSTEM PRESSURE (PSI) COMPARISON

		Location	End of Seely Road					Jackson Laboratory					East Strawberry Hill					Cromwell Harbor Road and Main Street					End of Spring Street				
		Waterline Elevation	28					136.25					182					30					95				
		Scenario	Existing	8" DI	Δ	12" DI	Δ	Existing	8" DI	Δ	12" DI	Δ	Existing	8" DI	Δ	12" DI	Δ	Existing	8" DI	Δ	12" DI	Δ	Existing	8" DI	Δ	12" DI	Δ
Maximum Tank Level	Average Demand 725 GPM	No Additional Demand	102	102	0	102	0	55	55	0	55	0	36	36	0	36	0	104	104	0	104	0	76	76	0	76	0
		Hydrant #70 at 1,420 GPM	96	97	1	98	2	49	50	1	50	1	29	30	1	30	1	93	95	2	95	2	69	70	1	70	1
		Hydrant #16 at 825 GPM	102	101	-1	101	-1	54	54	0	54	0	35	35	0	35	0	101	101	0	101	0	73	73	0	73	0
		Hydrant #104 at 1,300 GPM	100	100	0	100	0	53	53	0	53	0	33	33	0	33	0	99	99	0	99	0	71	71	0	71	0
	Peak Demand 2,550 GPM	No Additional Demand	95	95	0	95	0	45	45	0	45	0	29	29	0	29	0	98	98	0	98	0	70	70	0	70	0
		Hydrant #70 at 1,420 GPM	81	83	2	83	2	32	33	1	34	2	15	16	1	17	2	80	82	2	83	3	58	58	0	58	0
		Hydrant #16 at 825 GPM	91	91	0	91	0	41	41	0	41	0	24	24	0	24	0	91	91	0	91	0	64	64	0	64	0
		Hydrant #104 at 1,300 GPM	88	88	0	87	-1	38	38	0	38	0	21	21	0	21	0	87	87	0	87	0	59	59	0	59	0
Minimum Tank Level	Average Demand 725 GPM	No Additional Demand	101	101	0	101	0	54	54	0	54	0	35	35	0	35	0	103	103	0	103	0	75	75	0	75	0
		Hydrant #70 at 1,420 GPM	95	96	1	96	1	48	49	1	49	1	28	29	1	29	1	92	94	2	94	2	68	68	0	68	0
		Hydrant #16 at 825 GPM	100	100	0	100	0	53	53	0	53	0	33	33	0	33	0	100	99	-1	99	-1	72	72	0	72	0
		Hydrant #104 at 1,300 GPM	99	99	0	98	-1	52	51	-1	51	-1	32	32	0	32	0	98	97	-1	97	-1	69	69	0	69	0
	Peak Demand 2,550 GPM	No Additional Demand	93	93	0	93	0	43	43	0	43	0	27	27	0	27	0	97	97	0	97	0	69	69	0	69	0
		Hydrant #70 at 1,420 GPM	80	82	2	82	2	30	32	2	32	2	13	15	2	15	2	79	81	2	81	2	57	57	0	57	0
		Hydrant #16 at 825 GPM	90	89	-1	89	-1	40	40	0	39	-1	23	23	0	23	0	90	90	0	90	0	62	62	0	62	0
		Hydrant #104 at 1,300 GPM	87	86	-1	86	-1	37	37	0	36	-1	20	20	0	20	0	86	86	0	86	0	58	58	0	58	0

TABLE 52: MAIN STREET 6-INCH CAST IRON UPGRADE SCENARIO
DOWNTOWN LOCAL SYSTEM PRESSURE (PSI) COMPARISON

		Location	End of Cottage Way					End of Stephens Lane					End of Grason Lane					End of Albert Meadows					Atlantic Ave and Elbow Lane				
		Waterline Elevation	37.9					39.1					28.1					37.4					40.8				
		Scenario	Existing	8" DI	Δ	12" DI	Δ	Existing	8" DI	Δ	12" DI	Δ	Existing	8" DI	Δ	12" DI	Δ	Existing	8" DI	Δ	12" DI	Δ	Existing	8" DI	Δ	12" DI	Δ
Maximum Tank Level	Average Demand 725 GPM	No Additional Demand	101	101	0	101	0	101	101	0	101	0	105	105	0	105	0	101	101	0	101	0	100	100	0	100	0
		Hydrant #16 at 825 GPM	96	97	1	97	1	96	96	0	96	0	99	100	1	100	1	79	96	17	96	17	77	94	17	94	17
		Hydrant #71 at 1,265 GPM	95	95	0	95	0	95	94	-1	95	0	99	99	0	99	0	95	95	0	95	0	93	93	0	93	0
		Hydrant #72 at 1,315 GPM	94	94	0	95	1	93	94	1	94	1	96	97	1	98	2	92	94	2	94	2	90	92	2	93	3
	Peak Demand 2,550 GPM	No Additional Demand	95	95	0	95	0	94	94	0	94	0	99	99	0	99	0	95	95	0	95	0	94	94	0	93	-1
		Hydrant #16 at 825 GPM	86	87	1	87	1	85	86	1	86	1	89	89	0	90	1	67	86	19	86	19	66	84	18	84	18
		Hydrant #71 at 1,265 GPM	83	83	0	84	1	83	83	0	83	0	87	87	0	87	0	83	83	0	83	0	81	81	0	82	1
		Hydrant #72 at 1,315 GPM	82	82	0	83	1	81	82	1	82	1	84	85	1	86	2	79	82	3	83	4	78	80	2	81	3
Minimum Tank Level	Average Demand 725 GPM	No Additional Demand	100	100	0	100	0	100	100	0	100	0	104	104	0	104	0	100	100	0	100	0	99	99	0	99	0
		Hydrant #16 at 825 GPM	95	96	1	96	1	94	95	1	95	1	98	98	0	99	1	78	95	17	95	17	76	93	17	93	17
		Hydrant #71 at 1,265 GPM	94	94	0	94	0	93	93	0	94	1	98	97	-1	98	0	94	94	0	94	0	92	92	0	92	0
		Hydrant #72 at 1,315 GPM	93	93	0	94	1	92	93	1	93	1	95	96	1	97	2	91	93	2	93	2	89	91	2	92	3
	Peak Demand 2,550 GPM	No Additional Demand	94	94	0	94	0	93	93	0	93	0	98	98	0	98	0	94	94	0	94	0	93	92	-1	92	-1
		Hydrant #16 at 825 GPM	85	86	1	86	1	84	85	1	85	1	88	88	0	89	1	66	85	19	85	19	65	83	18	83	18
		Hydrant #71 at 1,265 GPM	82	82	0	83	1	82	82	0	82	0	86	86	0	86	0	81	82	1	82	1	80	80	0	80	0
		Hydrant #72 at 1,315 GPM	80	81	1	82	2	80	80	0	81	1	82	84	2	85	3	78	80	2	81	3	77	78	1	80	3

TABLE 53: MAIN STREET 6-INCH CAST IRON UPGRADE SCENARIO DOWNTOWN LOCAL SYSTEM PRESSURE (PSI) COMPARISON

		Location	Hydrant #44 Hancock Street					End of Stanwood Place					Center Street and Snow Street					Oliver Street and Snow Street					End of Scotts Lane				
		Waterline Elevation	34.1					36.4					37.6					36.7					15.6				
		Scenario	Existing	8" DI	Δ	12" DI	Δ	Existing	8" DI	Δ	12" DI	Δ	Existing	8" DI	Δ	12" DI	Δ	Existing	8" DI	Δ	12" DI	Δ	Existing	8" DI	Δ	12" DI	Δ
Maximum Tank Level	Average Demand 725 GPM	No Additional Demand	103	103	0	103	0	102	102	0	102	0	101	101	0	101	0	101	101	0	101	0	110	111	1	111	1
		Hydrant #16 at 825 GPM	97	98	1	99	2	97	97	0	98	1	97	97	0	97	0	97	97	0	97	0	107	107	0	107	0
		Hydrant #71 at 1,265 GPM	93	94	1	95	2	87	92	5	94	7	70	91	21	93	23	70	91	21	93	23	98	103	5	103	5
		Hydrant #72 at 1,315 GPM	82	92	10	94	12	82	91	9	93	11	88	92	4	93	5	89	92	3	93	4	103	103	0	103	0
	Peak Demand 2,550 GPM	No Additional Demand	96	96	0	96	0	95	95	0	95	0	95	95	0	95	0	95	95	0	95	0	104	104	0	104	0
		Hydrant #16 at 825 GPM	87	88	1	89	2	87	87	0	88	1	87	87	0	87	0	87	87	0	87	0	97	97	0	97	0
		Hydrant #71 at 1,265 GPM	80	82	2	83	3	75	81	6	82	7	57	79	22	81	24	57	79	22	82	25	86	91	5	91	5
		Hydrant #72 at 1,315 GPM	69	80	11	82	13	69	79	10	81	12	76	79	3	81	5	76	80	4	81	5	91	91	0	91	0
Minimum Tank Level	Average Demand 725 GPM	No Additional Demand	102	102	0	102	0	101	101	0	101	0	100	100	0	100	0	100	100	0	100	0	109	109	0	110	1
		Hydrant #16 at 825 GPM	96	97	1	97	1	96	96	0	96	0	96	96	0	96	0	96	96	0	96	0	106	105	-1	105	-1
		Hydrant #71 at 1,265 GPM	92	93	1	94	2	86	91	5	93	7	69	90	21	92	23	69	90	21	92	23	97	102	5	102	5
		Hydrant #72 at 1,315 GPM	81	91	10	93	12	80	90	10	92	12	87	90	3	92	5	88	91	3	92	4	102	102	0	102	0
	Peak Demand 2,550 GPM	No Additional Demand	95	95	0	95	0	94	94	0	94	0	94	94	0	94	0	94	94	0	94	0	103	103	0	103	0
		Hydrant #16 at 825 GPM	86	87	1	87	1	86	86	0	86	0	86	86	0	86	0	86	86	0	86	0	96	96	0	96	0
		Hydrant #71 at 1,265 GPM	79	81	2	82	3	74	79	5	81	7	56	78	22	80	24	56	78	22	80	24	85	90	5	90	5
		Hydrant #72 at 1,315 GPM	68	78	10	81	13	68	78	10	80	12	75	78	3	79	4	75	79	4	80	5	90	90	0	90	0

11.7.10 CONSIDER LOOPING CLEFTSTONE/ DEVON ROAD PIPING BACK TO WEST STREET EXTENSION

The Cleftstone Road and Devon Road portion of the distribution system located South of the West Street Extension and West of Eagle Lake Road is currently old 6-inch cast iron and 2-inch galvanized pipe. This section is also a dead-end line which could benefit in looping back to West Street Extension with 480-LF of 8-inch ductile iron pipe. Additionally, the remaining line on Devon Road and Cleftstone Road should be increased to 8-inch ductile iron as well. Model simulation showed the end of the original 2-inch galvanized line going out towards a small neighborhood on Cleftstone Road also has a high elevation of 211.4 feet that results in a maximum existing water pressure of 26-psi. This pressure drops with additional system demands as well as additional hydrant flows. This section of waterline was replaced with 4" HDPE in 2019 and was not included in the water model simulation, however it can be assumed that this adjustment would result in improvements to final water pressure at the end of the line. By completing the loop to Eagle Lake Road, water quality would also improve.

As shown in Tables 54 and 55, the greatest improvements will be seen locally on Cleftstone and Devon Road, especially when local hydrants are flowing. Again, note that the model simulation was with the original 2" galvanized line leading to the Cleftstone Road neighborhood and does not reflect results with the newly install 2019 4" HDPE waterline for this area. Regardless, the improvements are significant, especially for the high elevation location in the Cleftstone Road. It should be noted, however that while there are improvements for this location, the pressure is still below 30-psi and the Water Department should negotiate a Limited Service Contract with the residences fed by this line. Alternatively, the area could have a booster station.

TABLE 54: CLEFTSTONE/ DEVON ROAD UPGRADE AND LOOPING SCENARIO
LOCAL HYDRANTS SYSTEM PRESSURE (PSI) COMPARISON

		Location	Hydrant #8 Cleftstone Road & West Street Extension			Hydrant #6 Cleftstone Road & Devon Road			Devon Road Corner			End of Cleftstone Road XC Line			West Street Ext, Devon Road, and Woodbury Road			Woodbury Road and Eagle Lake Road		
		Waterline Elevation	150.3			166.2			134.2			211.4			122.4			129.2		
		Scenario	Existing	Loop	Δ	Existing	Loop	Δ	Existing	Loop	Δ	Existing	Loop	Δ	Existing	Loop	Δ	Existing	Loop	Δ
Maximum Tank Level	Average Demand 725 GPM	No Additional Demand	53	53	0	46	46	0	60	60	0	27	27	0	65	65	0	62	62	0
		Hydrant #6 at 430 GPM	52	52	0	10	45	35	39	59	20	5	25	20	64	64	0	61	61	0
		Hydrant #17 at 1,100 GPM	49	49	0	40	41	1	54	55	1	20	22	2	59	60	1	55	56	1
		Hydrant #22 at 540 GPM	52	52	0	44	45	1	58	58	0	25	25	0	63	63	0	59	59	0
	Peak Demand 2,550 GPM	No Additional Demand	51	51	0	43	44	1	57	57	0	24	24	0	62	62	0	59	59	0
		Hydrant #6 at 430 GPM	48	48	0	5	40	35	34	54	20	0	21	21	59	59	0	56	57	1
		Hydrant #17 at 1,100 GPM	44	44	0	34	36	2	48	49	1	15	16	1	53	54	1	49	50	1
		Hydrant #22 at 540 GPM	48	48	0	39	40	1	53	54	1	20	20	0	58	59	1	53	54	1
Minimum Tank Level	Average Demand 725 GPM	No Additional Demand	52	52	0	45	46	1	59	59	0	26	26	0	64	64	0	61	62	1
		Hydrant #6 at 430 GPM	51	51	0	9	44	35	38	58	20	4	24	20	63	63	0	60	60	0
		Hydrant #17 at 1,100 GPM	48	48	0	39	40	1	53	54	1	19	21	2	58	59	1	54	55	1
		Hydrant #22 at 540 GPM	51	51	0	43	44	1	57	57	0	24	24	0	62	62	0	58	58	0
	Peak Demand 2,550 GPM	No Additional Demand	50	50	0	42	43	1	56	56	0	23	23	0	61	61	0	58	58	0
		Hydrant #6 at 430 GPM	47	47	0	4	39	35	33	53	20	-1	20	21	58	59	1	55	56	1
		Hydrant #17 at 1,100 GPM	43	43	0	33	35	2	47	48	1	14	15	1	52	53	1	48	49	1
		Hydrant #22 at 540 GPM	47	47	0	38	39	1	52	53	1	19	19	0	57	58	1	52	53	1

TABLE 55: CLEFTSTONE/ DEVON ROAD UPGRADE AND LOOPING SCENARIO TOWN-WIDE HYDRANTS SYSTEM PRESSURE (PSI) COMPARISON

		Location	Hydrant #8 Cleftstone Road & West Street Extension			Hydrant #6 Cleftstone Road & Devon Road			Devon Road Corner			End of Cleftstone Road XC Line			West Street Ext, Devon Road, and Woodbury Road			Woodbury Road and Eagle Lake Road		
		Waterline Elevation	150.3			166.2			134.2			211.4			122.4			129.2		
		Scenario	Existing	Loop	Δ	Existing	Loop	Δ	Existing	Loop	Δ	Existing	Loop	Δ	Existing	Loop	Δ	Existing	Loop	Δ
Maximum Tank Level	Average Demand 725 GPM	No Additional Demand	53	53	0	46	46	0	60	60	0	27	27	0	65	65	0	62	62	0
		Hydrant #16 at 825 GPM	52	52	0	45	45	0	59	59	0	25	25	0	64	64	0	61	61	0
		Hydrant #104 at 1,300 GPM	51	51	0	43	44	1	57	58	1	24	24	0	62	63	1	59	60	1
		Hydrant #108 at 700 GPM	52	52	0	45	45	0	59	59	0	25	25	0	64	64	0	61	61	0
	Peak Demand 2,550 GPM	No Additional Demand	51	51	0	43	44	1	57	57	0	24	24	0	62	62	0	59	59	0
		Hydrant #16 at 825 GPM	48	48	0	40	41	1	54	54	0	20	21	1	59	59	0	56	56	0
		Hydrant #104 at 1,300 GPM	46	46	0	38	39	1	52	52	0	18	19	1	57	57	0	54	54	0
		Hydrant #108 at 700 GPM	48	48	0	40	41	1	54	55	1	21	21	0	59	59	0	56	56	0
Minimum Tank Level	Average Demand 725 GPM	No Additional Demand	52	52	0	45	46	1	59	59	0	26	26	0	64	64	0	61	62	1
		Hydrant #16 at 825 GPM	51	51	0	44	44	0	58	58	0	24	24	0	63	63	0	60	60	0
		Hydrant #104 at 1,300 GPM	50	50	0	42	43	1	56	57	1	23	23	0	61	62	1	58	59	1
		Hydrant #108 at 700 GPM	51	51	0	44	44	0	58	58	0	24	24	0	63	63	0	60	60	0
	Peak Demand 2,550 GPM	No Additional Demand	50	50	0	42	43	1	56	56	0	23	23	0	61	61	0	58	58	0
		Hydrant #16 at 825 GPM	47	47	0	39	40	1	53	53	0	19	20	1	58	58	0	55	55	0
		Hydrant #104 at 1,300 GPM	45	45	0	37	38	1	51	51	0	17	18	1	56	56	0	53	53	0
		Hydrant #108 at 700 GPM	47	47	0	39	40	1	53	54	1	20	20	0	58	59	1	55	56	1

12.0 RECOMMENDED INFRASTRUCTURE AND WATER SYSTEM IMPROVEMENT PLAN

12.1 SUMMMARY OF CRITICAL INFRASTRUCTURE ISSUES

This Comprehensive Water Plan Update report has evaluated the components of the Town of Bar Harbor's water system. The Town has an aging distribution system and additional infrastructure which needs to be addressed in order to improve water quality and maintain the integrity of the existing system.

The water source from Eagle Lake is adequate for the Town's current and future water needs. Improvements will eventually be necessary regarding the Lake's outlet dam which is owned by the Town. The recently upgraded Duck Brook Pumping Station facility appears to be operating as intended and is in excellent condition.

The Town's distribution system is in poor condition in certain areas with noted leaks, galvanized piping, asbestos cement piping, unlined cast iron piping and an extensive amount of inadequately sized piping. Some of the piping was installed in the late 1800's. In addition, some valves are inoperable and many hydrants are also in need of upgrade and maintenance. Because of these issues, over the next twenty years, there is a significant amount of distribution piping that should be improved.

The Town has inadequate water storage and challenges within its system regarding pressure due mainly to significant elevation changes throughout its water system. This issue is compounded due to the extensive amount of undersized water distribution piping. Additional Booster Stations and repairs to those existing are suggested to optimize the Town's water system.

This Comprehensive Plan Report has evaluated the Town's water system operating and infrastructure deficiencies which need to be addressed in order to properly maintain and improve the water quality and integrity of the existing system. The recommended items for implementation include the following:

12.1.1 EAGLE LAKE OUTLET DAM IMPROVEMENTS

Deficiency: The Town owns the dam at the outlet of Eagle Lake. The last time the dam's condition was officially inspected was in 1997 by MEMA where the dam was found to have several deficiencies. To the best of our knowledge, there has not been any major work conducted on the outlet dam since this inspection. However, back in October of 2010, MEMA classified the dam as a "Low Hazard Dam" which means that the failure would not present a danger to human life. Because it is classified as a "Low Hazard Dam", it is only inspected by MEMA every 12 years which would put the next inspection date into 2022. As part of this Master Plan Update, we visited the site to review the condition of the outlet dam. We did not complete an inspection of the structural integrity of the dam or its berms. However, we did note that the wooden boards which block the original sluiceway were degrading and would recommend that these are replaced with a gate or other device that could control the outlet of the water versus the water seeping through the damaged boards. If the boards were to fail, the impact to the water system would be significant.

FIGURE 22: LEAKING WOODEN SLUICEWAY BOARDS

Suggested Dam Improvements:

- Structural Inspection - Although, the site is due for a MEMA inspection in 2022, we would suggest having a local structural firm evaluate the dam for recommendations for its ongoing maintenance.

- Tree Growth - We would recommend controlling tree growth along the dikes.

- Sluiceway Boards – We understand that the water flowing though the boards is desirable to permit the downstream brook to flow. We would recommend replacement of the boards with a more permanent gate structure to better control this flow.

12.1.2 WATER STORAGE IMPROVEMENTS

Deficiency: As was discussed in Sections 7 and 9, the Town has insufficient storage capacity in its distribution system. The existing storage capacity including the concrete tank and Jackson Laboratory tank, in which both are not typically operated full, is 1,000,000 gallons. If you decrease this amount by what has been estimated as Dead Storage, the Town actually only has 690,000 gallons of usable storage assuming that each tank is operated full, which they are not.

In addition, based on modeling and on-site preliminary flow testing, the water system is predicted to drop below the required 20 PSI pressure in certain elevation areas during certain hydrant flow conditions whether it be at average demand or peak demand. Adding additional storage will assist with improving this condition.

Jackson Laboratory – The Jackson Laboratory tank operates at a lower elevation than the Duck Brook tank and the maximum water surface elevation of Eagle Lake. This difference requires the use of an altitude valve to prevent the tank from overflowing. Jackson Laboratory pulls the level of the storage tank located at the lab down during the daytime to levels lower than what would be recommended. These levels can drop as low as to 15 feet in the summertime. This significantly reduces the volume of water available in the event of a system emergency or causes a concern to the Jackson Laboratory in protecting their operation. In addition, this problem drops the hydraulic grade in this area which reduces pressures in some areas by around 8 PSI.

Additional Storage Up-Island – The distribution system in the Up-Island area between Eden Street north of the Ferry Terminal to the seasonal lines in Salisbury Cove, including the Hulls Cove area, has major system vulnerabilities. If a break occurred along the 8-inch diameter Eden Street line, the users downstream would have no access to a standby supply of water. In addition, the single 8-inch diameter distribution line that supplies water to this area provides significant head losses during hydrant flows that limit the flow capacities and are predicted to cause pressure issues below 20 PSI in higher elevation areas along this route. Because of this issue, we are recommending that a new storage tank be located in the Up-Island area.

Suggested Water Storage Improvements:

- Jackson Laboratory – Increase the height of the Jackson Laboratory tank by ten feet to match the hydraulic gradient of the remainder of the system in order to provide additional storage volume for Jackson Laboratory. Adding ten feet of additional elevation would provide 167,000 additional gallons of storage. (The exact elevation of the tank would be finalized during design.) The disadvantage of raising the elevation of this tank is that the elevation of the Duck Brook reservoir would need to be maintained at a higher elevation for the tank to fill by gravity to its full capacity. However, this would also eliminate the need for an altitude valve. Note that raising the tank would increase the pressure near the tank by around 2 to 4 psi while the downtown areas are fairly close to the same pressure as before.

Note that there are higher risks in raising the tank versus providing a new tank adjacent to the existing. These factors are listed below:

- There is more risk to the operation of Jackson Laboratory due to needing to provide temporary water volume adjacent to the existing tank. The cost of providing this temporary water was included in Section 12.2.2 as an estimate.

- Prior to any final decision on raising the Jackson Laboratory tank, the original construction drawings of the existing tank and its foundation will need to be reviewed. This is to determine the original design of the foundation to determine if it is sufficient to carry the load of the additional weight of water. One option for design consideration in the future is to raise the elevation by providing a new bottom to the tank such that the existing tank bottom would be raised 10 feet. This allows for proper load design of this portion of the tank.

- We would want to update the water model with additional fire hydrant results from the Jackson Laboratory area to ensure that the tank height would not cause any issues to the existing area. Based on our initial results, this does not look to be an issue.

- <u>Proposed New Water Storage Tank Up-Island</u> – An extensive amount of work was done to evaluate the best location for a new water storage tank. This is discussed in detail in Section 11.7.4. We evaluated two potential tank locations in this area. From this evaluation, we suggest placing a new water storage tank at Ireson Hill. The advantages of this site are that the Town already owns the property, it would be feasible to connect it to the water system and the site is elevated at around 220 feet. The exact size of the proposed new water storage tank would be determined upon final system design but we would suggest a minimum volume of 500,000 gallons. Note that the exact elevation of the tank would also need to be determined during final design.

- <u>Existing Duck Brook Operations</u> – We would suggest if these tank improvements are made, that the Duck Brook raw water pumps be operated with the goal of keeping the Duck Brook tank at a higher level.

- <u>Existing Storage Tank Operational Levels</u> – We would suggest running the levels higher at the Jackson Lab to maximize the storage capacity of the system.

Based on our evaluation, the proposed new tank would also benefit the Jackson Laboratory tank in that demands from the Route 3 area would be satisfied by the volume in the new tank. We suggest phasing the work such that the Ireson Hill tank be installed prior to the proposed Ferry Terminal Booster Pump to be discussed under Section 12.1.3.

With the improvements to the Jackson Laboratory tank and the proposed new tank, the Town would gain a minimum of 667,000 gallons of storage. We would suggest completing full system updated fire flow analysis, evaluating the impact that this added storage has on water quality and operations prior to planning additional water storage.

12.1.3 BOOSTER PUMPING STATION IMPROVEMENTS

Deficiency: Based on the poor condition of the Arata Drive and Mountain Avenue Pump stations, we suggest that they be upgraded. Both stations only have one pump, no backup power and poor access conditions. Both buildings that house the pump equipment are also in very poor condition.

Suggested Improvements:

Arata Drive and Mountain Avenue Booster Station Upgrade - The Town would like to upgrade these two booster pump stations by looping Arata Drive and using one new booster station to supply both Mountain Avenue and Arata Drive. Because of their proximity, we would suggest upgrading them with one above grade station located at the Mountain Avenue site. In addition, we suggest replacing the waterline along Arata Drive before this work is done since it is 3-inch diameter galvanized steel and considered undersized, poor condition piping. This should be done prior to the station upgrade or along with it to ensure that the booster station design is done to consider the new piping versus the existing.

Deficiency: If the Town installed the proposed Ireson Hill tank, it should be noted that it is in the same pressure zone as the Jackson Laboratory tank and there is no present way to direct the flow pumped from the Summer Pump or the gravity system to one tank or the other. System hydraulics, specific tank levels and system demands will dictate the direction of the flow. This could be remedied by separating the tanks with a pump station located at the Town owned property near the Ferry Terminal.

Suggested Improvements:

Install a Booster Station at the Town's Property near the Ferry Terminal - The purpose of the booster station would be to assist with filling the Ireson Hill storage tank and allow the Summer Pump at Duck Brook to separately fill Jackson Laboratory tank when needed. By separating the tanks with a pump station for the proposed Ireson

Hill tank, the system could include a bypass line to fill the new tank by gravity, have a pump to direct flows to the new tank when high system demands require it, and also be set up to isolate this area from the Duck Brook system with an automated valve when necessary to fill the Jackson Laboratory tank discretely.

12.1.4 DISTRIBUTION SYSTEM PIPING IMPROVEMENTS

Deficiency: The Town has 189,529 LF of water piping and about 52.83 percent is smaller than 6-inches in diameter. These lines are likely even smaller than the stated sizes due to tubercles that form over time in old water system. In addition, the water system has about 47.07 percent cast iron piping and about 8.09 percent galvanized pipe. Although, some of the piping has been replaced over the years, there is still a significant amount of older, small piping left in the system. There will be a significant challenge to replacing the piping since much of the older piping is in the downtown areas of the distribution system. These older pipes contribute to poor water quality, low hydrant flow rates, lower than desired pressure and overall water losses.

Suggested Improvements:

Table 56 presents the highest priority distribution system areas that should be improved due to issues with water quality, pressure, maintenance, and those considered to be bottlenecks. Some areas are also related to other suggested projects such as Arata Drive piping. The Town should consider removing all of its GS, AC and over time, its CI lines. These projects should be coordinated with other Town projects as applicable.

TABLE 56: SUMMARY OF RECOMMENDATIONS FOR LONG TERM WATER MAIN REPLACEMENT PROGRAM

STREET/LOCATION	RECOMMENDATION	BENEFITS
Devon Road[1]	Increase line size from 6-inch CI to 8-inch DI. Loop piping to West Street Extension	Improved water quality, pressure, aging infrastructure, and fire flow.
Old Farm Road	Increase the line size from 6-inch CI to 8-inch DI.	Improved water quality, fire flow, aging infrastructure, and eliminates a bleeder.
Harbor Lane	Increase line size from 6-inch CI to 8-inch DI.	Improved water quality, fire flow, aging infrastructure, and eliminates a bleeder.
Hamor Lane	Increase the line size from 2-inch GS to 4-inch DI.	Improved water quality and aging infrastructure.
Loren Street	Increase the line size from 2-inch GS to 4-inch DI.	Improved water quality and aging infrastructure.
Kebo Street[2]	Increase the line size from 8-inch CI to 12-inch DI.	Remove significant bottleneck and improve pressure reliability.

TABLE 56: CONTINUED

STREET/LOCATION	RECOMMENDATION	BENEFITS
Arata Drive[2]	Increase the line size from 3-inch GS to 6-inch DI.	Improved water quality, increased reliability and reduced pressure losses for combined booster station.
Hancock Street	Increase the line size from 6-inch CI and 2-inch GS to 8-inch DI and 4-inch DI.	Improved water quality, aging infrastructure, and fire flow.
Atlantic Street	Increase the line size from 6-inch CI and 2-inch GS to 8-inch DI and 4-inch DI.	Improved water quality, aging infrastructure, and fire flow.
Albert Meadows and Derby Lane	Increase the line size from 6-inch CI and 3-inch GS to 8-inch DI.	Improved water quality, aging infrastructure, and fire flow.
Stephen's Lane and Cottage Way	Increase the line size from 2-inch GS to 4-inch DI.	Improved water quality and aging infrastructure.
Newton Way and Des Isle Avenue	Increase the line size from 2-inch and 3-inch GS to 8-inch DI.	Improved water quality and aging infrastructure.
Eagle Lake Road	Increase the line size from 2-inch to 8-inch DI. Loop to Cross Street.	Improved water quality, aging infrastructure, extend fire protection service area, extend service through to Cross Street.
Spring Street (Includes southern end)	Remove 10-inch CI pipe and replace with 10-inch DI, upsize 2-inch GS and 4-inch CI to 8-inch DI.	Improve water quality, aging infrastructure, and fire protection. Upper Spring in location of future sewer project.
Shannon Road	Remove last remaining 10-inch AC pipe and replace with 10-inch DI.	Improved water quality, aging infrastructure in location of future sewer project.
Glen Mary Road	Upgrade 3-inch GS and replace with 8-inch DI.	Improved water quality, aging infrastructure, and fire protection.
Holland Avenue	Increase 6-inch CI and 2-inch GS to 8-inch DI.	Improve water quality, aging infrastructure, and fire protection. Future location of sewer project.
Main Street	Increase 6-inch CI to 12-inch DI.	Improve water quality and bottleneck through downtown.
Scott's Lane	Increase 2-inch GS to 8-inch DI.	Improve water quality, aging infrastructure, and fire protection.
Stanwood Place	Increase 3-inch GS to 4-inch DI.	Improve water quality and aging infrastructure.
Kavanaugh Place	Increase 2-inch GS to 4-inch DI.	Improve water quality and aging infrastructure.

TABLE 56: CONTINUED

STREET/LOCATION	RECOMMENDATION	BENEFITS
Oak Street	Increase 2-inch GS to 8-inch DI.	Improve water quality, aging infrastructure, and fire protection.
Highbrook Road[2]	Replace 6-inch CI with 8-inch DI and loop to West Street Extension.	Improve water quality, aging infrastructure, and fire flow. This needs to be done with new storage tank, and Ferry Terminal Booster Pump.
Eden Street[3]	Replace 8-inch CI with 8-inch DI. (This was not upsized to 12-inches since the downstream piping was recently replaced to 8-inch DI.)	Needs to be done if Ferry Terminal Booster Pump is installed. This also is older CI piping and may need updating at some point due to its age.

1 - Two cost estimates were done (A.1a without looping and A.1b includes looping).
2 - See below for further discussion.
3 - As part of an ongoing cast iron replacement.

A few areas are notable due to modeled bottle necks. These areas are summarized in Table 56 but also discussed in more detail below:

- Old Farm Road and Harbor Lane – Both of these areas are known to have poor water quality and the Town operates year-round bleeders at these two locations. The bleeders are operated to control or improve water quality. Water systems in general should achieve to eliminate bleeders since they are considered non-revenue water. The Old Farm Road bleeder runs about 525,600 gallons per year of water and the Harbor Lane bleeder runs about 1,051,200 gallons per year of water to improve local water quality.

- Kebo Street – During the completion of the water model analysis, we evaluated installation of a new water tank at Hamilton Hill. The initial thoughts of picking this location were to improve pressures and flows along Kebo Street. While this tank indicates that there would be an improvement in pressure along Kebo Street, the resulting pressure at the end of Kebo Street and specifically at the Kebo Ridge Development are not simulated by the model to all be above 20 PSI. In Section 11.7.2 of the water model analysis, we determined that it may make more sense to install a larger line along Kebo Street. This is because the piping along Kebo Street is 8-inch CI and appears to be the cause of the major head loss for the system. If the Kebo Street line was increased from 8-inch CI to 12-inch DI, it would have better results than a new storage tank located at Hamilton Hill.

- Arata Drive – The Town wishes to combine the Arata Drive Booster Station with the Mountain Avenue Booster station as discussed under 12.1.2. Prior to making this upgrade, we would recommend upsizing the 3-inch GS piping to 6-inch DI. While the area has been boosted though the 3-inch GS piping in the past, we believe that once these two areas are combined and looped together, the system would work better with the same size piping. We also would recommend this on the basis of improving water quality and updating old obsolete system infrastructure.

- Highbrook Road – During the summer months, the Town shuts off the 12-inch HDPE summer line between the Duck Brook Road and Eden Street. This cuts off significant water supply to the hydrants along lower Eden Street below the Ferry Terminal. Based on our water model evaluation, there are several hydrants that are severely impacted when this seasonal line is shut down. If the Town decides to install the Ferry Terminal Pump to better feed a new water tank proposed at Ireson Hill, during the winter months when the 12-inch HDPE line is shut off, the hydrants upstream of the station along Eden Street will not have the benefit of the new tank or the waterline heading Up-Island along Route 3. Assuming that the Town would not want to consider replacing the entire 2,125 LF of cross-country seasonal HDPE line with permanent piping, we evaluated replacing the water line on Highbrook Road by upgrading the 6-inch CI with 8-inch DI pipe and looping it to Eden Street. (The portion of newer 6-inch DI line would remain.) The loop to Eden Street would improve water quality and fire flow significantly during the wintertime.

- Eden Street - If the Town decides to install the new Ireson Hill Tank and Ferry Terminal Booster Pump Station, we suggest replacing the old CI piping between the booster station and the 8-inch DI piping recently replaced in 2017. Pumping into the line will increase the pressure through this location and the older line runs the risk of damage, leaks, or breaks with the sudden increase in pressure.

12.1.5 FIRE HYDRANT MAINTENANCE AND REPLACEMENT

Deficiency: As discussed in Section 8.2, the Town's fire hydrants are not well maintained due to lack of staffing. What the customer sees in general are the fire hydrants so they should be in good working order and painted. We estimate that there are about 58 hydrants that were installed prior to 1970 and should be replaced. Additionally, if there is a fire, there is some concern that some of the observed hydrants would not be available due to poor conditions. This would be a considerable liability for the Town.

Suggested Improvements:

• The Town should notify the Fire Department, and bag all hydrants that are not safe to operate.

• Hydrants should all be fire flow tested in a uniform manner. Once this is done, the system model should be updated. This information should also be placed with the specific hydrant record keeping system.

• Once tested, hydrants should be identified to indicate their expected flow range.

• The hydrants that do not work should be replaced with the street piping if there are pending distribution improvements in the area, or the Town should initiate a hydrant upgrade program to include at least 5-hydrants each year until they are all in working order.

• All working hydrants should be fully maintained including annual inspections, all repairs as needed and full flushing. When the hydrants are flushed, the Town should inspect all components and prepare a working log of maintenance to be done to each hydrant.

• All hydrants should have working isolation valves to ensure that they can be shut off in the event of damage or when planned maintenance is scheduled.

12.1.6 VALVE MAINTENANCE, REPLACEMENT, AND REPAIRS

Deficiency: As discussed in Section 8.3, the Town does not have a formal valve cleaning and exercising program. Again, from our observations this is due to the limited distribution system staffing. This type of program would involve cleaning each valve on every street, identifying buried valves and bringing them up to grade, exercising each valve every year and recording the condition of all valves. The Town has been exercising valves that are part of the hydrant flushing program, but not all system valves including hydrant street valves.

Suggested Improvements:

• Initiate a valve maintenance program that schedules street by street valve cleaning followed by exercising once per year.

• Data from this program should be recorded including direction of valve (open/close), number of turns, condition of valve.

• All broken valves should be replaced.

12.1.7 POLICY IMPROVEMENTS AND UPDATES

• Terms and Conditions – The Town's current Terms and Conditions are dated August 2012. We would suggest updating them every five years to make sure the newest PUC regulations are included. Also, water system specific changes and practices should be evaluated at this time to make sure that they reflect the current costs for services.

• Construction Materials – Due to the extent of development and need to standardize the water system, we suggest completing a construction practices and materials specification document for the water system.

• Formal Water Use Ordinance - A formal Water Use Ordinance would be suggested to protect Eagle Lake and the Town's filtration avoidance waiver. This will be a lengthy process but is suggested. We understand that there are ongoing concerns or discussions with the Park with the control of beavers and other types of activities. This would be an opportunity to formally regulate all activities that are of concern to protect the excellent water quality of this very important source water. The ordinance would establish short and long-term management strategies to effectively manage the lake to support both the Park and the Town.

• Review of Developments – Due to the extent of development and areas that new development is being constructed, we would suggest having these development projects be reviewed by a professional engineer to ensure that they are complying with overall Town Ordinances.

• Low Pressure Limited Service Contracts – Because much of the recent development and some already constructed development has been permitted in areas that do not provide 20 PSI of pressure, we suggest Low Pressure Limited Service Contracts be required in advance for all development and be recorded with the deed for each property owner. Specific areas in which these Contracts may be required include the following:

- Hamilton Hill and future development subdivision lots
- Kebo Ridge Development
- East Strawberry Hill
- Cleftstone Road

Low Pressure Limited Service Contracts may not be required if a booster station is installed in these areas.

12.1.8 STAFFING FOR DISTRIBUTION SYSTEM WORK

Deficiency: The Town has a highly educated staff and completes an extensive amount of system operational and maintenance activities. Due to the size, age and complexity of the system, we would suggest adding additional staff for distribution system maintenance.

Suggested Improvements:

Add to the present staff, another full-time distribution system operator. This operator would be responsible for upgrading the level of valve and hydrant maintenance which would also be the recordkeeping of these efforts.

12.2 ESTIMATED COSTS OF INITIAL INFRASTRUCTURE IMPROVEMENTS

As discussed in this report, the Town of Bar Harbor water supply system has several components where capital improvements are required. Many of these improvements are related to the Town's aged water distribution system where many feet of old, deteriorated water lines should be replaced over time. It is likely that these repairs will take many years to complete due to the extent of the distribution system and the high-density population in the areas that need improvement.

As an initial goal, this plan considers the highest priority projects that should be addressed over the next ten to twenty-year period subject to the Town's ability to secure funding sources or to gradually raise water rates to make system improvements.

In order to compare the relative economic merits of the required improvements, order-of-magnitude costs have been developed for each. These costs do not represent final estimates of the detailed expenses which may be required to construct proposed facilities. The purpose of conducting this economic evaluation is to provide a comparative analysis of costs. Cost quotations from manufacturers, as well as contractor's prices for similar recent installations, were utilized as a data source to develop these cost estimates. More detailed and accurate cost estimates cannot be made during the planning phase of this project because finalized project sizing and design data are not yet available. More detailed cost estimates should be updated upon the completion of final design plans and specifications.

The estimates include an allowance for construction cost categories, such as demolition, electrical, instrumentation, site development, piping and contractor overhead and markup, as well as engineering design, inspection and project contingency costs. Ledge excavation, piping costs, estimated customer curb stop costs, fire hydrants, easement and land acquisitions, road trench pavement, and surface restoration are also included.

Since the costs of these projects are subject to inflation, the 2020 dollars will likely be subject to increases of perhaps five percent per year for each year beyond 2020 in which all or part of the construction is delayed. The current costs presented herein should be adjusted for inflation once a final implementation plan is approved and an exact construction schedule is known. It is likely that the project's schedule will be driven by the Town's prioritization of certain water main improvements and internal finances to allow specific projects to proceed.

12.2.1 EAGLE LAKE OUTLET DAM EVALUATION

Before conducting any improvements at the Outlet Dam, we would suggest that the Town have an evaluation completed to prioritize the improvements and make sure that they are in the best interest of the Town's water system. We indicated in Section 12.1.1 that we would suggest a structural inspection, tree growth removal around the berm and a better control method for water flow than the failing old sluiceway boards. Suggested Order of Magnitude Costs are listed below:

- Structural Inspection and Report - $10,000
- Tree Growth Removal - $5,000
- Sluiceway Board Replacement with controlled release valve - $50,000

12.2.2 WATER STORAGE TANK IMPROVEMENTS

The cost estimate for the suggested water tank improvements in 12.1.2 are provided below. The detailed cost estimate for the Ireson Hill tank includes the transmission line to connect into Route 3. This cost also includes all site work, geotechnical work and engineering to complete the project. The Ireson Hill tank cost estimate is located in Appendix B.2.

The cost of increasing the elevation of the Jackson Laboratory tank of $581,000 includes preliminary budgetary costs provided by Pittsburg Tank and Tower. This work would include a site visit to assist with final estimation of the work, mobilization, tree clearing and staging area for the temporary water, temporary water systems to provide volume and keep the system pressurized during the work, cutting of the tank and removing the dome, extending the tank ten feet, welding back the dome, cleaning and painting the tank and disinfection. The Jackson Laboratory tank cost estimate is located in Appendix B.5.

TABLE 57: SUMMARY OF WATER STORAGE TANK IMPROVEMENT COSTS

Location	Estimated Costs
New Tank at Ireson Hill (B.2)	$1,929,000
Increase Elevation of Jackson Laboratory Tank (B.5)	$581,000
Total Suggested Water Tank Improvements	$2,510,000

The total currently suggested storage tank improvement projects would be about $2,510,000 in 2020 dollars. Alternatively, the Town could consider replacing the Jackson Laboratory Tank which would be expected to cost about $1,000,000 more than the estimate in Appendix B.5.

12.2.3 BOOSTER PUMPING STATION IMPROVEMENTS

As discussed in 12.1.3, the Arata Drive and Mountain Avenue Booster Stations have needed upgraded for quite some time. The Town would like to combine these two stations into one at the Mountain Avenue site. The cost estimate in B.1 would include a new building, foundation, duplicate pumps, a generator, power, piping to loop Arata Drive, site work, legal, engineering services. This project should be completed along with replacement of the Arata Drive waterline which is included in Table 58, Cost Estimate A.7. The estimated cost for the Arata Drive/Mountain Avenue booster pump upgrade is $528,000. The estimated cost for increasing the piping from 3-inch GS to 6-inch DI is $356,000.

As discussed in detail in the water model Section 11.7.6.3 as well as above in 12.1.3, a possible method to fill the proposed Ireson Hill storage tank during high demand periods would be with a separate booster station. Note that this station would only be required if filling the proposed Ireson Hill tank was found to be difficult during the summer and is not recommended until after the new tank is installed. The cost estimate in B.3 and as shown in Table 58 below would include a building, foundation, duplicate pumps, a generator, power, controls, piping to connect to Eden Street, site work, legal and engineering services. The estimated cost for installing a booster station for a proposed tank Up-Island at Ireson Hill would be about $743,000. We also suggest coordinating the proposed booster station with looping the waterline along Highbrook Road for approximately $375,000 as shown on cost Table A.23 and the replacement of the old CI line on Eden Street for approximately $469,000 as shown on cost Table A.24.

The bladder tanks for the Mountain Avenue Booster Stations are planned to be re-utilized and are located at Rockwood Avenue. Table B.4 provides an estimate to upgrade the building that houses these bladder tanks. The estimated cost is around $20,000 including a new roof, paining, new doors and updating the exterior trim. Table 58 summarizes these costs:

TABLE 58: SUMMARY OF BOOSTER PUMP IMPROVEMENT COSTS

Location	Estimated Costs
Arata Drive/Mountain Avenue Booster Station (B.1)	$528,000
Ferry Terminal Pump Station (B.3)	$743,000
Building Improvements at Rockwood Avenue (B.4)	$20,000
Total Suggested Booster Pump Station Improvements	$548,000 to $1,291,000

We recognize that there are several other low-pressure areas at high elevations in the water system that could benefit from having a booster station. Estimates were not provided for these areas since the pressure at the main is expected to be 20 PSI. However, in the future as fire flow testing is finalized along with other updates to the water model, small booster stations may want to be considered for the end of Cleftstone Road and East Strawberry Hill. The total currently suggested booster pump improvement projects would be about $548,000 to $1,291,000 in 2020 dollars depending on the necessity of installing the Ferry Terminal Pump Station.

The schedule of a new water storage tank should be coordinated with looping the water line on Highbrook Road, installing a booster station at the Ferry Terminal and replacing the remainder of the old 8-inch cast iron piping on Eden Street on the downstream side of the Ferry Terminal. We would suggest that the Town complete the storage tank first, then the project to loop Highbrook water main, and finally complete the Ferry Terminal Booster station along with the replacement of the Eden Street piping.

12.2.4 DISTRIBUTION SYSTEM PIPING IMPROVEMENTS

As was discussed in Section 8 and 12.1.4 as well as in the model evaluation in Section 11, the Town has a large quantity of remaining CI piping as well as a concentration of GS piping in its Downtown area. Unfortunately, replacement of this piping will be difficult due to the population density in the location of the work. Many of the waterline projects have additional benefits as they upsize unsuitably small piping, eliminate water quality bleeders due to poor quality piping or dead-ends, loop lines where possible and coincide with a suggested project. The benefits of each project were summarized back on Table 56. The preliminary order-of-magnitude cost estimates for each of the projects included below on Table 59 are also located in Appendix A.

TABLE 59: SUMMARY OF SUGGESTED WATER MAIN DEFICIENCY REPLACEMENT PROGRAM AND COSTS

WATER MAIN DEFICIENCY REPLACMENT PROGRAM INVENTORY							
Cost Table No.	Length (LF)	Exist. Material	Exist. Diam., inches	Prop. Diam., inches	Proposed Distribution Improvement Location	Estimated Cost	Priority
A.1a	1,350	CI	6	8	Devon Road	$618,000	H
A.1b	2,000	CI	6	8	Loop Above to West Extension	$803,000	H
A.2	1,150	CI	6	8	Old Farm Road	$359,000	H
A.3	624	CI	6	8	Harbor Lane	$229,000	H
A.4	500	GS	2	4	Hamor Lane	$170,000	H
A.5	730	GS	2	4	Loren Street	$218,000	H
A.6	4,310	CI	8	12	Kebo Street	$1,440,000	H
A.7	1,053	GS	3	6	Arata Drive	$356,000	H
A.8	2,120	CI/GS	6/2	8/4	Hancock Street	$749,000	M
A.9	1,100	CI/GS	6/2	8/4	Atlantic Street	$489,000	M
A.10	952	CI/GS	6/3	8	Albert Meadows and Derby Lane	$531,000	H
A.11	750	GS	2	4	Stephen's Lane and Cottage Way	$323,000	M
A.12	1,100	GS	2/3	8	Newton Way and Des Isle Avenue	$463,000	M
A.13	740	GS	2	8	Eagle Lake Road	$319,000	M
A.14	1,250	CI/GS	10/4/2	10/8	Spring Street	$502,000	M
A.15	1,500	AC	10	10	Shannon Road	$718,000	M
A.16	1,200	GS	3	8	Glen Mary Road	$531,000	M
A.17	1,450	CI/GS	6/2	8	Holland Avenue	$577,000	M
A.18	2,400	CI	6	12	Main Street	$1,274,000	M
A.19	850	GS	2	8	Scott's Lane	$289,000	M
A.20	200	GS	3	4	Stanwood Place	$146,000	M
A.21	350	GS	2	4	Kavanaugh Place	$167,000	M
A.22	440	GS	2	8	Oak Street	$206,000	M
A.23	850	CI	6	8	Highbrook Road	$375,000	H
A.24	960	CI	8	8	Eden Street	$469,000	H/L[1]
Total Estimated Water Line Costs (2020)						$11,518,000 to $11,703,000	

1 – Depending on if the Ferry Booster Station is ever needed.

12.2.5 FIRE HYDRANT IMPROVEMENTS

The cost of hydrant replacement depends on whether the work is done in-house, by a contractor or along with a pipeline project. Table 60 below provides an estimate of what hydrant replacement costs would be under these different scenarios:

TABLE 60: ESTIMATED COST OF FIRE HYDRANT REPLACEMENTS

Replacement Method	Estimated Cost, Each Hydrant
By a Contractor, replacing hydrants only	$10,000 - $15,000 (with a new valve)
With a pipeline project	$6,000

The cost of replacing a fire hydrant would depend upon if there is an isolation valve that can be used to isolate the hydrant or if the entire line needs to be shut down and a valve installed along with the hydrant. Newer hydrants that can be maintained or rebuilt can be done at a fraction of the above costs which could easily be included in the Town's Operation and Maintenance budget. Assuming five hydrants are replaced each year, the cost for a replacement program could be between $50,000 to $75,000 each year.

The Town is in the planning phases to complete fire hydrant flow testing to update the testing that was done in the past. We estimate this work to cost around $15,000 to complete.

12.2.6 WATER SYSTEM VALVE IMPROVEMENTS

The cost of valve replacement also depends on whether the work is done in-house, by a contractor or along with a pipeline project. Table 61 below provides an estimate of what individual valve replacement costs would be under these different scenarios:

TABLE 61: ESTIMATED COST OF VALVE REPLACEMENTS

Replacement Method	Estimated Cost, Each Valve
By a Contractor, replacing valves only, including excavation.	$10,000 - $15,000
Insertion Valves including excavation by contractor.	$10,000 - $20,000
With a pipeline project – 8-inch valve.	$2,500

The cost of replacing a valve would depend upon if there is a way to shut down the water in order to isolate the line where the valve is located. Sometimes, if this cannot be done, an insertion valve may be needed. The problem with insertion valves on older lines is that they sometimes do not close properly because the line can be ovalized or full of tubercles. Newer valves which can be exercised and cleaned out as well as repaired can be done at a fraction of the above costs which could easily be included in the Town's Operation and Maintenance budget. Assuming five valves are replaced each year, the cost for a replacement program could be between $50,000 to $100,000 each year.

12.2.7 POLICY OR OTHER SUGGESTED PROGRAMS

Most of the suggested policy improvements under 12.1.7 could be completed in-house or with minor engineering assistance that would fit within the Town's Water Department budget. The most expensive suggested improvement would be developing a Water Use Ordinance. Table 62 provides an order of magnitude preliminary estimate of each of these activities if the Town were to want assistance completing them:

TABLE 62: UPDATED WATER SYSTEM POLICIES OR PROGRAMS

Suggested Improvements	Order of Magnitude Cost
Terms and Conditions	$5,000
Construction Specifications for the Water System	$1,500
Water Shed Protection Plan	$10,000 - $15,000
Development Review	$5,000/year
Limited Service Contracts	Not Applicable – Complete by Water System Staff

12.2.8 ADDITIONAL STAFFING FOR DISTRIBUTION SYSTEM MAINTENANCE

Additional staffing within the Water Department would depend upon the position advertised. We would expect that this would as a minimum add between $45,000 in salary and depending upon the specific benefits an additional $20,000 for insurances with a minimum impact to the budget of around $65,000.

12.3 CAPITAL IMPROVEMENT PROGRAM SUMMARY

Scheduling of capital improvement projects is paramount to budget development and fiscal solvency. The current suggested Capital Improvements Program (CIP) will take the Town along time to complete due to the extent of old distribution system piping. The planning for this work will likely be done in large in coordination with the overall Town when roadway or other improvements projects are completed. The current estimated program should be reevaluated by the Town as roadway work is completed by the Town since there are many remaining old distribution piping not identified in this CIP since it was either not GS, small CI, undersized, a bottleneck, or in an area that we suggested additional looping.

In addition, we did not provide any estimations for the future cost of filtration if the Town ever loses its filtration avoidance waiver. This would be an extensive expense for the Town and should be avoided at all cost. The 2005 Master Plan indicated that the cost of filtration could be as high as 7.2 million dollars. At this point, this number would be closer to 15 million dollars if you assume the costs increase five percent each construction year.

All cost estimates provided in this report are presented in 2020 dollars as just order-of-magnitude planning estimates. Therefore, it is recommended that all future costs be adjusted to account for the effects of inflation and the changing construction market conditions at the time of project implementation. Future costs can be estimated using the Engineering News Record (ENR) Construction cost Index or by applying an estimated rate of inflation that reflects the current and anticipated future market conditions.

APPENDIX A

PRELIMINARY ORDER-OF-MAGNITUDE DISTRIBUTION PIPING COST ESTIMATES

A.1a – UPGRADE DEVON ROAD WATER LINE
FROM 6” Ø CI TO 8” Ø DI
PRELIMINARY ORDER-OF-MAGNITUDE COST ESTIMATE

QUANTITY	DESCRIPTION/COST	ESTIMATE
LS	Traffic control @ $50,000/LS	$ 50,000
1350 LF	8” Ø DI water main @ $125/LF	168,750
2 EA	12” Ø wedge valves @ $3,500/EA	7,000
3 EA	8” Ø wedge valves @ $2,500/EA	7,500
1 EA	4” Ø wedge valve @ $1,500/EA	1,500
18 EA	1” Ø curb stops and boxes @ $500/EA	9,000
18 EA	1” Ø corporation stops @ $500/EA	9,000
500 LF	1” Ø CU water service @ $70/LF	35,000
2 EA	Fire hydrants and valves @ $6,000/EA	12,000
300 Tons	4” Trench pavement @ $200/Ton	60,000
300 CY	Ledge removal @ $200/CY	60,000
LS	Erosion control @ $5,000/LS	5,000
LS	Loam and seed @ $5,000/LS	5,000
LS	Owner’s testing allowance @ $5,000/LS	5,000
LS	General conditions @ $43,250/LS	43,250
	Subtotal	$ 478,000
	Ledge probes	4,000
	Design allowance	38,000
	Inspection allowance	40,000
	Administration allowance	10,000
	Contingency allowance	48,000
	Estimate	$ 618,000

* Adds a hydrant at the end of the southern extension of Devon Road. (Does not include cost to loop to West Street Extension)

A.1b – UPGRADE DEVON ROAD WATER LINE FROM 6" Ø CI TO 8" Ø DI AND LOOP TO WEST STREET EXT. PRELIMINARY ORDER-OF-MAGNITUDE COST ESTIMATE

QUANTITY	DESCRIPTION/COST	ESTIMATE
LS	Traffic control @ $50,000/LS	$ 50,000
2000 LF	8" Ø DI water main @ $125/LF	250,000
2 EA	12" Ø wedge valves @ $3,500/EA	7,000
3 EA	8" Ø wedge valves @ $2,500/EA	7,500
1 EA	4" Ø wedge valve @ $1,500/EA	1,500
20 EA	1" Ø curb stops and boxes @ $500/EA	10,000
20 EA	1" Ø corporation stops @ $500/EA	10,000
560 LF	1" Ø CU water service @ $70/LF	39,200
3 EA	Fire hydrants and valves @ $6,000/EA	18,000
400 Tons	4" Trench pavement @ $200/Ton	80,000
400 CY	Ledge removal @ $200/CY	80,000
LS	Erosion control @ $5,000/LS	5,000
LS	Loam and seed @ $5,000/LS	5,000
LS	Owner's testing allowance @ $5,000/LS	5,000
LS	General conditions @ $56,800/LS	56,800
	Subtotal	$ 625,000
	Ledge probes	5,000
	Design allowance	50,000
	Inspection allowance	50,000
	Administration allowance	10,000
	Contingency allowance	63,000
	Estimate	$ 803,000

* Adds a hydrant at the end of the southern extension of Devon Road.
(Includes cost to loop to West Street Ext. Road)
-8"Ø DI along Devon Road and loop to West Street Ext.

A.2 – UPGRADE OLD FARM ROAD 6” Ø CI PIPING TO 8” Ø DI
PRELIMINARY ORDER-OF-MAGNITUDE COST ESTIMATE

QUANTITY	DESCRIPTION/COST	ESTIMATE
LS	Traffic control @ $10,000/LS	$ 10,000
1150 LF	8” Ø DI water main @ $125/LF	143,750
150 LF	1” Ø CU water service @ $70/LF	10,500
3 EA	8” Ø wedge valves @ $2,500/EA	7,500
3 EA	1” Ø Curb stops and boxes @ $500/EA	1,500
3 EA	1” Ø Corporation stops @ $500/EA	1,500
1 EA	Fire hydrant and valve @$6,000/EA	6,000
200 Tons	4” Trench pavement @ $200/Ton	40,000
100 CY	Ledge removal @ $200/CY	20,000
LS	Erosion control @ $5,000/LS	5,000
LS	Loam and seed @ $5,000/LS	5,000
LS	Owner’s testing allowance @ $2,000/LS	2,000
LS	General conditions @ $25,250/LS	25,250
	Subtotal	$ 278,000
	Ledge probes	4,000
	Design allowance	22,000
	Inspection allowance	22,000
	Administration allowance	5,000
	Contingency allowance	28,000
	Estimate	$ 359,000

* Would connect to existing 608 LF of 6” Ø DI line.

A.3 – UPGRADE HARBOR LANE WATER LINE
FROM 6" Ø CI TO 8" Ø DI
PRELIMINARY ORDER-OF-MAGNITUDE COST ESTIMATE

QUANTITY	DESCRIPTION/COST	ESTIMATE
LS	Traffic control @ $10,000/LS	$ 10,000
624 LF	8" Ø DI water main @ $125/LF	78,000
1 EA	8" Ø wedge valve @ $2,500/EA	2,500
6 EA	1" Ø curb stops and boxes @ $500/EA	3,000
6 EA	1" Ø corporation stops @ $500/EA	3,000
300 LF	1" Ø CU water service @ $70/LF	21,000
1 EA	Fire hydrant and valve @$6,000/EA	6,000
150 Tons	4" Trench pavement @ $200/Ton	30,000
200 CY	Ledge removal @ $200/CY	40,000
LS	Erosion control @ $5,000/LS	5,000
LS	Loam and seed @ $5,000/LS	5,000
LS	Owner's testing allowance @ $5,000/LS	5,000
LS	General conditions @ $21,500/LS	21,500
	Subtotal	$ 230,000
	Ledge probes	3,000
	Design allowance	19,000
	Inspection allowance	19,000
	Administration allowance	5,000
	Contingency allowance	23,000
	Estimate	$ 229,000

* Before design verify fire protection flows at hydrant. May reduce pipe diameter back to 6" Ø if fire protection flows are adequate.

A.4 – UPGRADE HAMOR LANE WATER MAIN
FROM 2" Ø GS TO 4" Ø DI
PRELIMINARY ORDER-OF-MAGNITUDE COST ESTIMATE

QUANTITY	DESCRIPTION/COST	ESTIMATE
LS	Traffic control @ $5,000/LS	$ 5,000
500 LF	4" Ø DI water main/service @ $90/LF	45,000
1 EA	4" Ø wedge valve @ $1,500/EA	1,500
4 EA	1" Ø curb stops and boxes @ $500/EA	2,000
4 EA	1" Ø corporation stops @ $500/EA	2,000
100 LF	1" Ø CU water service @ $70/LF	7,000
1 EA	Blow-off hydrant and valve @$3,500/EA	3,500
90 Tons	4" Trench pavement @ $200/Ton	18,000
100 CY	Ledge removal @ $200/CY	20,000
LS	Erosion control @ $3,000/LS	3,000
LS	Loam and seed @ $3,000/LS	3,000
LS	Owner's testing allowance @ $2,500/LS	2,500
LS	General conditions @ $12,500/LS	12,500
	Subtotal	$ 125,000
	Ledge probes	3,000
	Design allowance	12,000
	Inspection allowance	12,000
	Administration allowance	5,000
	Contingency allowance	13,000
	Estimate	$ 170,000

A.5 – UPGRADE LOREN STREET WATER MAIN
FROM 2" Ø GS TO 4" Ø DI
PRELIMINARY ORDER-OF-MAGNITUDE COST ESTIMATE

QUANTITY	DESCRIPTION/COST	ESTIMATE
LS	Traffic control @ $5,000/LS	$ 5,000
730 LF	4" Ø DI water main/service @ $90/LF	65,700
1 EA	4" Ø wedge valve @ $1,500/EA	1,500
6 EA	1" Ø curb stops and boxes @ $500/EA	3,000
6 EA	1" Ø corporation stops @ $500/EA	3,000
150 LF	1" Ø CU water service @ $70/LF	10,500
1 EA	Blow-off hydrant and valve @$3,500/EA	3,500
130 Tons	4" Trench pavement @ $200/Ton	26,000
100 CY	Ledge removal @ $200/CY	20,000
LS	Erosion control @ $3,000/LS	3,000
LS	Loam and seed @ $3,000/LS	3,000
LS	Owner's testing allowance @ $2,500/LS	2,500
LS	General conditions @ $15,300/LS	15,300
	Subtotal	$ 162,000
	Ledge probes	3,000
	Design allowance	16,000
	Inspection allowance	16,000
	Administration allowance	5,000
	Contingency allowance	16,000
	Estimate	$ 218,000

A.6 – UPGRADE KEBO STREET WATER LINE FROM 8" Ø CI TO 12" Ø DI
PRELIMINARY ORDER-OF-MAGNITUDE COST ESTIMATE

QUANTITY	DESCRIPTION/COST	ESTIMATE
LS	Traffic control @ $75,000/LS	$ 75,000
4310 LF	12" Ø DI water main @ $150/LF	646,500
400 LF	1" Ø CU water service @ $70/LF	28,000
100 LF	2" Ø CU water service @ $85/LF	8,500
7 EA	12" Ø wedge valves @ $3,500/EA	24,500
15 EA	1" Ø Curb stops and boxes @ $500/EA	7,500
15 EA	1" Ø Corporation stops @ $500/EA	7,500
2 EA	2" Ø Curb stops and boxes @ $1,000/EA	2,000
2 EA	2" Ø Corporation stops @ $1,000/EA	2,000
4 EA	Fire hydrants and valves @$6,000/EA	24,000
720 Tons	4" Trench pavement @ $200/Ton	144,000
300 CY	Ledge removal @ $200/CY	60,000
LS	Erosion control @ $5,000/LS	5,000
LS	Loam and seed @ $5,000/LS	5,000
LS	Owner's testing allowance @ $5,000/LS	5,000
LS	General conditions @ $100,500/LS	100,500
	Subtotal	$ 1,145,000
	Ledge probes	5,000
	Design allowance	90,000
	Inspection allowance	90,000
	Administration allowance	10,000
	Contingency allowance	100,000
	Estimate	$ 1,440,000

A.7 – UPGRADE ARATA DRIVE WATER LINE FROM 3" Ø GS TO 6" Ø DI PRELIMINARY ORDER-OF-MAGNITUDE COST ESTIMATE

QUANTITY	DESCRIPTION/COST	ESTIMATE
LS	Traffic control @ $10,000/LS	$ 10,000
1053 LF	6" Ø DI water main @ $100/LF	105,300
3 EA	6" Ø wedge valves @ $2,000/EA	6,000
12 EA	1" Ø curb stops and boxes @ $500/EA	6,000
12 EA	1" Ø corporation stops @ $500/EA	6,000
300 LF	1" Ø CU water service @ $70/LF	21,000
200 Tons	4" Trench pavement @ $200/Ton	40,000
200 CY	Ledge removal @ $200/CY	40,000
LS	Erosion control @ $5,000/LS	5,000
LS	Loam and seed @ $5,000/LS	5,000
LS	Owner's testing allowance @ $5,000/LS	5,000
LS	General conditions @ $25,700/LS	25,700
	Subtotal	$ 275,000
	Ledge probes	4,000
	Design allowance	22,000
	Inspection allowance	22,000
	Administration allowance	5,000
	Contingency allowance	28,000
	Estimate	$ 356,000

* Portion of pipe to create loop to Mountain Avenue in Booster Station Cost Estimate. Assumes line will be looped with Booster Station so no hydrant or blow-off is included.

A.8 – HANCOCK STREET 6" Ø CI / 2" Ø GS WATER MAIN REPLACEMENT / LOOPING WITH 8" Ø DI / 4" Ø DI
PRELIMINARY ORDER-OF-MAGNITUDE COST ESTIMATE

QUANTITY	DESCRIPTION/COST	ESTIMATE
LS	Traffic control @ $40,000/LS	$ 40,000
1500 LF	8" Ø DI water main @ $125/LF	187,500
620 LF	4" Ø DI water main @ $90/LF	55,800
4 EA	8" Ø wedge valves @ $2,500/EA	10,000
30 EA	1" Ø Curb stops and boxes @ $500/EA	15,000
30 EA	1" Ø Corporation stops @ $500/EA	15,000
1 EA	2" Ø Curb stops and boxes @ $1,000 EA	1,000
1 EA	2" Ø Corporation stops @ $1,000/EA	1,000
750 LF	1" Ø CU water service @ $70/LF	52,500
50 LF	2" Ø CU water service @ $85/LF	4,250
2 EA	Fire hydrants and valves @ $6,000/EA	12,000
450 Tons	4" Trench pavement @ $200/Ton	90,000
150 CY	Ledge removal @ $200/CY	30,000
LS	Erosion control @ $5,000/LS	5,000
LS	Loam and seed @ $5,000/LS	5,000
LS	Owner's testing allowance @ $5,000/LS	5,000
LS	General conditions @ $53,950/LS	53,950
	Subtotal	$ 583,000
	Ledge probes	4,000
	Design allowance	47,000
	Inspection allowance	47,000
	Administration allowance	10,000
	Contingency allowance	58,000
	Estimate	$ 749,000

* Assumes cost for looping through Hancock Lane to Atlantic Avenue.

A.9 –ATLANTIC AVENUE 6” Ø CI / 2” Ø GS WATER MAIN REPLACEMENT WITH 8” Ø DI / 4” Ø DI
PRELIMINARY ORDER-OF-MAGNITUDE COST ESTIMATE

QUANTITY	DESCRIPTION/COST	ESTIMATE
LS	Traffic control @ $40,000/LS	$ 40,000
800 LF	8” Ø DI water main @ $125/LF	100,000
300 LF	4” Ø DI water main @ $90/LF	27,000
4 EA	8” Ø wedge valves @ $2,500/EA	10,000
19 EA	1” Ø Curb stops and boxes @ $500/EA	9,500
19 EA	1” Ø Corporation stops @ $500/EA	9,500
1 EA	2” Ø Curb stops and boxes @ $1,000 EA	1,000
1 EA	2” Ø Corporation stops @ $1,000/EA	1,000
480 LF	1” Ø CU water service @ $70/LF	33,600
50 LF	2” Ø CU water service @ $85/LF	4,250
2 EA	Fire hydrants and valves @ $6,000/EA	12,000
250 Tons	4” Trench pavement @ $200/Ton	50,000
150 CY	Ledge removal @ $200/CY	30,000
LS	Erosion control @ $5,000/LS	5,000
LS	Loam and seed @ $5,000/LS	5,000
LS	Owner’s testing allowance @ $5,000/LS	5,000
LS	General conditions @ $34,150/LS	34,150
	Subtotal	$ 377,000
	Ledge probes	4,000
	Design allowance	30,000
	Inspection allowance	30,000
	Administration allowance	10,000
	Contingency allowance	38,000
	Estimate	$ 489,000

* Assumes looping through Hancock Lane done with Hancock Street.

A.10 – UPGRADE ALBERT MEADOW/DERBY LANE WATER LINE FROM 6" Ø CI / 3" Ø GS TO 8" Ø DI PRELIMINARY ORDER-OF-MAGNITUDE COST ESTIMATE

QUANTITY	DESCRIPTION/COST	ESTIMATE
LS	Traffic control @ $40,000/LS	$ 40,000
952 LF	8" Ø DI water main @ $125/LF	119,000
6 EA	8" Ø wedge valves @ $2,500/EA	15,000
33 EA	1" Ø curb stops and boxes @ $500/EA	16,500
33 EA	1" Ø corporation stops @ $500/EA	16,500
2 EA	2" Ø curb stops and boxes @ $1,000 EA	2,000
2 EA	2" Ø corporation stops @ $1,000/EA	2,000
830 LF	1" Ø CU water service @ $70/LF	58,100
100/LF	2" Ø CU water service @ $85/LF	8,500
1 EA	Fire hydrant and valve @ $6,000/EA	6,000
280 Tons	4" Trench pavement @ $200/Ton	56,000
100 CY	Ledge removal @ $200/CY	20,000
LS	Erosion control @ $5,000/LS	5,000
LS	Loam and seed @ $5,000/LS	5,000
LS	Owner's testing allowance @ $5,000/LS	5,000
LS	General conditions @ $38,400/LS	38,400
	Subtotal	$ 413,000
	Ledge probes	3,000
	Design allowance	33,000
	Inspection allowance	33,000
	Administration allowance	8,000
	Contingency allowance	41,000
	Estimate	$ 531,000

* Connect upgraded lines to existing 8" Ø DI line.

A.11 – UPGRADE STEPHENS LANE / COTTAGE WAY WATER LINE FROM 2" Ø GS / 2" Ø HDPE TO 4" Ø DI PRELIMINARY ORDER-OF-MAGNITUDE COST ESTIMATE

QUANTITY	DESCRIPTION/COST	ESTIMATE
LS	Traffic control @ $10,000/LS	$ 10,000
750 LF	4" Ø DI water main @ $90/LF	67,500
4 EA	10" Ø wedge valves @ $3,000/EA	12,000
2 EA	4" Ø wedge valve @ $2,000/EA	4,000
20 EA	1" Ø curb stops and boxes @ $500/EA	10,000
20 EA	1" Ø corporation stops @ $500/EA	10,000
500 LF	1" Ø CU water service @ $70/LF	35,000
2 EA	Blow-off hydrants and valves @ $3,500/EA	7,000
180 Tons	4" Trench pavement @ $200/Ton	36,000
100 CY	Ledge removal @ $200/CY	20,000
LS	Erosion control @ $5,000/LS	5,000
LS	Loam and seed @ $5,000/LS	5,000
LS	Owner's testing allowance @ $5,000/LS	5,000
LS	General conditions @ $23,500/LS	23,500
	Subtotal	$ 250,000
	Ledge probes	3,000
	Design allowance	20,000
	Inspection allowance	20,000
	Administration allowance	5,000
	Contingency allowance	25,000
	Estimate	$ 323,000

A.12 – UPGRADE NEWTON WAY / DES ISLE AVENUE
FROM 2" Ø / 3" Ø GS TO 8" Ø DI
PRELIMINARY ORDER-OF-MAGNITUDE COST ESTIMATE

QUANTITY	DESCRIPTION/COST	ESTIMATE
LS	Traffic control @ $25,000/LS	$ 25,000
1100 LF	8" Ø DI water main @ $125/LF	137,500
6 EA	8" Ø wedge valves @ $2,500/EA	15,000
4 EA	6" Ø wedge valves @ $2,000/EA	8,000
1 EA	4" Ø wedge valve @ $1,500/EA	1,500
22 EA	1" Ø curb stops and boxes @ $500/EA	11,000
22 EA	1" Ø corporation stops @ $500/EA	11,000
550 LF	1" Ø CU water service @ $70/LF	38,500
50 LF	4" Ø DI water service @ $90/LF	4,500
1 EA	Fire hydrant and valve @ $6,000/EA	6,000
160 Tons	4" Trench pavement @ $200/Ton	32,000
100 CY	Ledge removal @ $200/CY	20,000
LS	Erosion control @ $5,000/LS	5,000
LS	Loam and seed @ $5,000/LS	5,000
LS	Owner's testing allowance @ $5,000/LS	5,000
LS	General conditions @ $32,000/LS	32,000
	Subtotal	$ 357,000
	Ledge probes	3,000
	Design allowance	28,000
	Inspection allowance	28,000
	Administration allowance	5,000
	Contingency allowance	42,000
	Estimate	$ 463,000

A.13 – UPGRADE EAGLE LAKE ROAD WATER MAIN FROM 2" Ø GS TO 8" Ø DI AND LOOP TO CROSS STREET
PRELIMINARY ORDER-OF-MAGNITUDE COST ESTIMATE

QUANTITY	DESCRIPTION/COST	ESTIMATE
LS	Traffic control @ $10,000/LS	$ 10,000
740 LF	8" Ø DI water main @ $125/LF	92,500
2 EA	8" Ø wedge valves @ $2,500/EA	5,000
8 EA	1" Ø curb stops and boxes @ $500/EA	4,000
8 EA	1" Ø corporation stops @ $500/EA	4,000
250 LF	1" Ø CU water service @ $70/LF	17,500
1 EA	Fire hydrant and valve @ $6,000/EA	6,000
150 Tons	4" Trench pavement @ $200/Ton	30,000
200 CY	Ledge removal @ $200/CY	40,000
LS	Erosion control @ $5,000/LS	5,000
LS	Loam and seed @ $5,000/LS	5,000
LS	Owner's testing allowance @ $5,000/LS	5,000
LS	General conditions @ $22,000/LS	22,000
	Subtotal	$ 246,000
	Ledge probes	3,000
	Design allowance	20,000
	Inspection allowance	20,000
	Administration allowance	5,000
	Contingency allowance	25,000
	Estimate	$ 319,000

* Includes looping section of Eagle Lake Road to Cross Street to improve water quality.

A.14 – UPGRADE SPRING STREET (SOUTH) 10” Ø / 4” CI/ 2” GS LINE
PRELIMINARY ORDER-OF-MAGNITUDE COST ESTIMATE

QUANTITY	DESCRIPTION/COST	ESTIMATE
LS	Traffic control @ $40,000/LS	$ 40,000
450 LF	10” Ø DI water main @ $140/LF	63,000
800 LF	8” Ø DI water main @ $125/LF (Spring South)	100,000
2 EA	10” Ø wedge valves @ $3,000/EA	6,000
1 EA	8” Ø wedge valve @ $2,500/EA	2,500
16 EA	1” Ø curb stops and boxes @ $500/EA	8,000
16 EA	1” Ø corporation stops @ $500/EA	8,000
400 LF	1” Ø CU water service @ $70/LF	28,000
2 EA	Fire hydrants and valves @ $6,000/EA	12,000
250 Tons	4” Trench pavement @ $200/Ton	50,000
100 CY	Ledge removal @ $200/CY	20,000
LS	Erosion control @ $5,000/LS	5,000
LS	Loam and seed @ $5,000/LS	5,000
LS	Owner’s testing allowance @ $5,000/LS	5,000
LS	General conditions @ $35,500/LS	35,500
	Subtotal	$ 388,000
	Ledge probes	4,000
	Design allowance	31,000
	Inspection allowance	31,000
	Administration allowance	10,000
	Contingency allowance	38,000
	Estimate	$ 502,000

* Adds hydrants at southern end of Spring Street.

A.15 – UPGRADE SHANNON ROAD 10" Ø AC LINE TO 10" Ø DI
PRELIMINARY ORDER-OF-MAGNITUDE COST ESTIMATE

QUANTITY	DESCRIPTION/COST	ESTIMATE
LS	Traffic control @ $50,000/LS	$ 50,000
1500 LF	10" Ø DI water main @ $140/LF	210,000
6 EA	10" Ø wedge valves @ $3,000/EA	18,000
2 EA	8" Ø wedge valves @ $2,500/EA	5,000
24 EA	1" Ø curb stops and boxes @ $500/EA	12,000
24 EA	1" Ø corporation stops @ $500/EA	12,000
600 LF	1" Ø CU water service @ $70/LF	42,000
2 EA	Fire hydrants and valves @ $6,000/EA	12,000
350 Tons	4" Trench pavement @ $200/Ton	70,000
300 CY	Ledge removal @ $200/CY	60,000
LS	Erosion control @ $5,000/LS	5,000
LS	Loam and seed @ $5,000/LS	5,000
LS	Owner's testing allowance @ $5,000/LS	5,000
LS	General conditions @ $50,000/LS	50,000
	Subtotal	$ 556,000
	Ledge probes	6,000
	Design allowance	45,000
	Inspection allowance	45,000
	Administration allowance	10,000
	Contingency allowance	56,000
	Estimate	$ 718,000

* Includes Spring Street (South)

A.16 – UPGRADE GLEN MARY ROAD 3" Ø GS TO 8" Ø DI
PRELIMINARY ORDER-OF-MAGNITUDE COST ESTIMATE

QUANTITY	DESCRIPTION/COST	ESTIMATE
LS	Traffic control @ $30,000/LS	$ 30,000
1200 LF	8" Ø DI water main @ $125/LF	150,000
2 EA	8" Ø wedge valves @ $2,500/EA	5,000
25 EA	1" Ø curb stops and boxes @ $500/EA	12,500
25 EA	1" Ø corporation stops @ $500/EA	12,500
650 LF	1" Ø CU water service @ $70/LF	45,500
1 EA	Fire hydrant and valve @ $6,000/EA	6,000
280 Tons	4" Trench pavement @ $200/Ton	56,000
200 CY	Ledge removal @ $200/CY	40,000
LS	Erosion control @ $5,000/LS	5,000
LS	Loam and seed @ $5,000/LS	5,000
LS	Owner's testing allowance @ $5,000/LS	5,000
LS	General conditions @ $37,500/LS	37,500
	Subtotal	$ 410,000
	Ledge probes	4,000
	Design allowance	33,000
	Inspection allowance	33,000
	Administration allowance	10,000
	Contingency allowance	41,000
	Estimate	$ 531,000

* Adds hydrants at southern end of Glen Mary Road.

A.17 – UPGRADE HOLLAND AVENUE (NORTH)
FROM 6" Ø CI / 2" Ø GS TO 8" Ø DI
PRELIMINARY ORDER-OF-MAGNITUDE COST ESTIMATE

QUANTITY	DESCRIPTION/COST	ESTIMATE
LS	Traffic control @ $40,000/LS	$ 40,000
1450 LF	8" Ø DI water main @ $125/LF	181,250
4 EA	8" Ø wedge valves @ $2,500/EA	10,000
16 EA	1" Ø curb stops and boxes @ $500/EA	8,000
16 EA	1" Ø corporation stops @ $500/EA	8,000
400 LF	1" Ø CU water service @ $70/LF	28,000
300 Tons	4" Trench pavement @ $200/Ton	60,000
200 CY	Ledge removal @ $200/CY	40,000
LS	Erosion control @ $10,000/LS	10,000
LS	Loam and seed @ $10,000/LS	10,000
LS	Owner's testing allowance @ $10,000/LS	10,000
LS	General conditions @ $40,750/LS	40,750
	Subtotal	$ 446,000
	Ledge probes	4,000
	Design allowance	36,000
	Inspection allowance	36,000
	Administration allowance	10,000
	Contingency allowance	45,000
	Estimate	$ 577,000

* Includes looping Holland Avenue from Cottage Street to West Street.

A.18 – UPGRADE MAIN STREET WATER LINE FROM 6” Ø CI TO 12” Ø DI
PRELIMINARY ORDER-OF-MAGNITUDE COST ESTIMATE

QUANTITY	DESCRIPTION/COST	ESTIMATE
LS	Traffic control @ $150,000/LS	$ 150,000
2400 LF	12” Ø DI water main @ $150/LF	360,000
16 EA	12” Ø wedge valves @ $3,500/EA	56,000
22 EA	1” Ø curb stops and boxes @ $500/EA	11,000
22 EA	1” Ø corporation stops @ $500/EA	11,000
2 EA	2” Ø curb stops and boxes @ $1,000/EA	2,000
2 EA	2” Ø corporation stops @ $1,000/EA	2,000
600 LF	1” Ø CU water service @ $70/LF	42,000
100 LF	2” Ø CU water service @ $85/LF	8,500
4 EA	Fire hydrants and valves @ $6,000/EA	24,000
460 Tons	4” Trench pavement @ $200/Ton	92,000
300 CY	Ledge removal @ $200/CY	60,000
LS	Erosion control @ $5,000/LS	5,000
LS	Loam and seed @ $5,000/LS	5,000
LS	Owner’s testing allowance @ $10,000/LS	10,000
LS	General conditions @ $84,500/LS	84,500
	Subtotal	$ 923,000
	Ledge probes	6,000
	Design allowance	90,000
	Inspection allowance	90,000
	Administration allowance	15,000
	Contingency allowance	150,000
	Estimate	$ 1,274,000

A.19 – UPGRADE SCOTT'S LANE 2" Ø GS LINE TO 8" Ø DI
PRELIMINARY ORDER-OF-MAGNITUDE COST ESTIMATE

QUANTITY	DESCRIPTION/COST	ESTIMATE
LS	Traffic control @ $8,000/LS	$ 8,000
850 LF	8" Ø DI water main @ $125/LF	106,250
3 EA	8" Ø wedge valves @ $2,500/EA	7,500
4 EA	1" Ø curb stops and boxes @ $500/EA	2,000
4 EA	1" Ø corporation stops @ $500/EA	2,000
200 LF	1" Ø CU water service @ $70/LF	14,000
1 EA	Fire hydrant and valve @ $6,000/EA	6,000
160 Tons	4" Trench pavement @ $200/Ton	32,000
100 CY	Ledge removal @ $200/CY	20,000
LS	Erosion control @ $2,000/LS	2,000
LS	Loam and seed @ $2,000/LS	2,000
LS	Owner's testing allowance @ $2,000/LS	2,000
LS	General conditions @ $20,250/LS	20,250
	Subtotal	$ 224,000
	Ledge probes	3,000
	Design allowance	18,000
	Inspection allowance	18,000
	Administration allowance	4,000
	Contingency allowance	22,000
	Estimate	$ 289,000

* Added fire hydrant due to length of street. Upsized line to 8" Ø due to recommended upsizing of Main Street to 12" Ø.

A.20 – UPGRADE GALVANIZED DEAD-END AT STANWOOD PLACE FROM 3" Ø GS TO 4" Ø DI PRELIMINARY ORDER-OF-MAGNITUDE COST ESTIMATE

QUANTITY	DESCRIPTION/COST	ESTIMATE
LS	Traffic control @ $10,000/LS	$ 10,000
200 LF	4" Ø DI water main @ $90/LF	18,000
1 EA	4" Ø wedge valve @ $1,500/EA	1,500
3 EA	1" Ø curb stops and boxes @ $500/EA	1,500
3 EA	1" Ø corporation stops @ $500/EA	1,500
100 LF	1" Ø CU water service @ $70/LF	7,000
1 EA	Blow-off and valve @ $3,500/EA	3,500
100 Tons	4" Trench pavement @ $200/Ton	20,000
100 CY	Ledge removal @ $200/CY	20,000
LS	Erosion control @ $5,000/LS	5,000
LS	Loam and seed @ $5,000/LS	5,000
LS	Owner's testing allowance @ $5,000/LS	5,000
LS	General conditions @ $10,000/LS	10,000
	Subtotal	$ 108,000
	Ledge probes	2,000
	Design allowance	10,000
	Inspection allowance	10,000
	Administration allowance	5,000
	Contingency allowance	11,000
	Estimate	$ 146,000

A.21 – UPGRADE GALVANIZED DEAD-END AT
KAVANAUGH PLACE FROM 2" Ø GS TO 4" Ø DI
PRELIMINARY ORDER-OF-MAGNITUDE COST ESTIMATE

QUANTITY	DESCRIPTION/COST	ESTIMATE
LS	Traffic control @ $10,000/LS	$ 10,000
350 LF	4" Ø DI water main @ $90/LF	31,500
1 EA	4" Ø wedge valve @ $1,500/EA	1,500
4 EA	1" Ø curb stops and boxes @ $500/EA	2,000
4 EA	1" Ø corporation stops @ $500/EA	2,000
100 LF	1" Ø CU water service @ $70/LF	7,000
1 EA	Blow-off and valve @ $3,500/EA	3,500
100 Tons	4" Trench pavement @ $200/Ton	20,000
100 CY	Ledge removal @ $200/CY	20,000
LS	Erosion control @ $5,000/LS	5,000
LS	Loam and seed @ $5,000/LS	5,000
LS	Owner's testing allowance @ $5,000/LS	5,000
LS	General conditions @ $11,500/LS	11,500
	Subtotal	$ 124,000
	Ledge probes	2,000
	Design allowance	12,000
	Inspection allowance	12,000
	Administration allowance	5,000
	Contingency allowance	12,000
	Estimate	$ 167,000

A.22 – UPGRADE OAK STREET WATER MAIN FROM 2” Ø GS TO 8” Ø DI
PRELIMINARY ORDER-OF-MAGNITUDE COST ESTIMATE

QUANTITY	DESCRIPTION/COST	ESTIMATE
LS	Traffic control @ $5,000/LS	$ 5,000
440 LF	8” Ø DI water main @ $125/LF	55,000
1 EA	8” Ø wedge valve @ $2,500/EA	2,500
5 EA	1” Ø curb stops and boxes @ $500/EA	2,500
5 EA	1” Ø corporation stops @ $500/EA	2,500
200 LF	1” Ø CU water service @ $70/LF	14,000
1 EA	Fire hydrant and valve @ $6,000/EA	6,000
100 Tons	4” Trench pavement @ $200/Ton	20,000
100 CY	Ledge removal @ $200/CY	20,000
LS	Erosion control @ $5,000/LS	5,000
LS	Loam and seed @ $5,000/LS	5,000
LS	Owner’s testing allowance @ $5,000/LS	5,000
LS	General conditions @ $14,500/LS	14,500
	Subtotal	$ 157,000
	Ledge probes	2,000
	Design allowance	13,000
	Inspection allowance	13,000
	Administration allowance	5,000
	Contingency allowance	16,000
	Estimate	$ 206,000

* Includes hydrant at end of street due to lack of hydrants in area.

A.23 – UPGRADE HIGHBROOK ROAD 6” Ø CI TO 8” Ø DI AND EXTEND TO LOOP TO EDEN STREET PRELIMINARY ORDER-OF-MAGNITUDE COST ESTIMATE

QUANTITY	DESCRIPTION/COST	ESTIMATE
LS	Traffic control @ $50,000/LS	$ 50,000
850 LF	8” Ø DI water main @ $125/LF	106,250
4 EA	8” Ø wedge valves @ $2,500/EA	10,000
4 EA	1” Ø curb stops and boxes @ $500/EA	2,000
4 EA	1” Ø corporation stops @ $500/EA	2,000
200 LF	1” Ø CU water service @ $70/LF	14,000
1 EA	2” Ø curb stop and box @ $1,000/EA	1,000
1 EA	2” Ø corporation stop @ $1,000/EA	1,000
50 LF	2” Ø CU water service @ $85/LF	4,250
1 EA	Fire hydrant and valve @ $6,000/EA	6,000
165 Tons	4” Trench pavement @ $200/Ton	33,000
100 CY	Ledge removal @ $200/CY	20,000
LS	Erosion control @ $5,000/LS	5,000
LS	Loam and seed @ $5,000/LS	5,000
LS	Owner’s testing allowance @ $5,000/LS	5,000
LS	General conditions @ $26,500/LS	26,500
	Subtotal	$ 291,000
	Ledge probes	3,000
	Design allowance	23,000
	Inspection allowance	23,000
	Administration allowance	5,000
	Contingency allowance	30,000
	Estimate	$ 375,000

* Loops Highbrook to Eden Street to improve fire flow when summer line is shut off.

A.24 – UPGRADE EDEN STREET 8" Ø CI TO 8" Ø DI DOWNSTREAM FROM PROPOSED FERRY TERMINAL BOOSTER STATION PRELIMINARY ORDER-OF-MAGNITUDE COST ESTIMATE

QUANTITY	DESCRIPTION/COST	ESTIMATE
LS	Traffic control @ $75,000/LS	$ 75,000
960 LF	8" Ø DI water main @ $125/LF	120,000
4 EA	8" Ø wedge valves @ $2,500/EA	10,000
1 EA	4" Ø wedge valve @ $1,500/EA	1,500
50 LF	4" Ø DI water main @ $90/LF	4,500
1 EA	1" Ø curb stop and box @ $500/EA	500
1 EA	1" Ø corporation stop @ $500/EA	500
50 LF	1" Ø CU water service @ $70/LF	3,500
1 EA	2" Ø curb stop and box @ $1,000/EA	1,000
1 EA	2" Ø corporation stop @ $1,000/EA	1,000
50 LF	2" Ø CU water service @ $85/LF	4,250
2 EA	Fire hydrants and valves @ $6,000/EA	12,000
220 Tons	4" Trench pavement @ $200/Ton	44,000
200 CY	Ledge removal @ $200/CY	40,000
LS	Erosion control @ $5,000/LS	5,000
LS	Loam and seed @ $5,000/LS	5,000
LS	Owner's testing allowance @ $5,000/LS	5,000
LS	General conditions @ $33,250/LS	33,250
	Subtotal	$ 366,000
	Ledge probes	3,000
	Design allowance	29,000
	Inspection allowance	29,000
	Administration allowance	5,000
	Contingency allowance	37,000
	Estimate	$ 469,000

* We did not upsize this to 12" Ø DI since much of the downstream piping was just upgraded to 8" Ø.

APPENDIX B

PRELIMINARY ORDER-OF-MAGNITUDE PUMPING AND STORAGE COST ESTIMATES

B.1 – ARATA DRIVE AND MOUNTAIN AVENUE BOOSTER STATION UPGRADE
PRELIMINARY ORDER-OF-MAGNITUDE COST ESTIMATE

QUANTITY	DESCRIPTION/COST	ESTIMATE
LS	Traffic control @ $25,000/LS	$ 25,000
LS	Building site work / demolition @ $41,000/LS	41,000
LS	Pump station building @ $50,000/LS	50,000
LS	Pumps and controls @ $60,000/LS	60,000
LS	Generator and conduits @ $80,000/LS	80,000
LS	Internal piping @ $20,000/LS	20,000
LS	Internal electrical @ $20,000/LS	20,000
LS	Internal valves @ $5,000/LS	5,000
300 LF	6" Ø DI water main @ $100/LF	30,000
3 EA	6" Ø wedge valves @ $2,000/EA	6,000
100 CY	Ledge removal @ $200/CY	20,000
LS	Erosion control @ $5,000/LS	5,000
LS	Loam and seed @ $5,000/LS	5,000
LS	Owner's testing allowance @ $5,000/LS	5,000
LS	General conditions @ $37,000/LS	37,000
	Subtotal	$ 409,000
	Ledge probes	4,000
	Design allowance	33,000
	Inspection allowance	33,000
	Administration allowance	8,000
	Contingency allowance	41,000
	Estimate	$ 528,000

* Assumes reuse of the existing four Bladder Tanks at Rockwood Avenue.

* Cost of looping waterline to complete this project is included in estimate.

* See Appendix A, Cost A.6 for Arata Drive waterline upgrade cost estimate.

B.2 – PROPOSED NEW WATER STORAGE TANK AT IRESON HILL
PRELIMINARY ORDER-OF-MAGNITUDE COST ESTIMATE

QUANTITY	DESCRIPTION/COST	ESTIMATE
LS	Tank/foundation (concrete) @ $900,000/LS	$ 900,000
LS	Civil construction @ $250,000/LS	250,000
LS	Piping (Internal to Tank) @ $20,000/LS	20,000
LS	Controls/electrical @ $30,000/LS	30,000
540 LF	Transmission piping @ $200/LF	108,000
2 EA	16" Ø wedge valves @ 5,000/EA	10,000
2 EA	8" Ø wedge valves @ 2,500/EA	5,000
1 EA	Fire hydrant and valve @ $6,000/EA	6,000
LS	Erosion control @ $5,000/LS	5,000
LS	Loam and seed @ $5,000/LS	5,000
LS	Owner's testing allowance @ $10,000/LS	10,000
LS	General conditions @ $135,000/LS	135,000
	Subtotal	$ 1,484,000
	Ledge removal	100,000
	Geotechnical	25,000
	Engineering	90,000
	Inspection	90,000
	Contingency allowance	140,000
	Total Estimate	$ 1,929,000

* Estimate includes transmission piping and valves.

B.3 – FERRY TERMINAL PUMP STATION
PRELIMINARY ORDER-OF-MAGNITUDE COST ESTIMATE

QUANTITY	DESCRIPTION/COST	ESTIMATE
LS	Traffic control @ $20,000/LS	$ 20,000
LS	Building site work @ $60,000/LS	60,000
LS	Pump station building @ $100,000/LS	100,000
LS	Pumps and controls @ $60,000/LS	60,000
LS	Generator and controls @ $80,000/LS	80,000
80 LF	8" Ø DI water main @ $125/LF	10,000
8 EA	8" Ø wedge valves @ $2,500/EA	20,000
50 Tons	Trench pavement @ $200/Ton	10,000
LS	Internal piping @ $40,000/LS	40,000
LS	Internal electrical/controls @ $40,000/LS	40,000
LS	Internal valves @ $40,000/LS	40,000
100 CY	Ledge removal @ $200/CY	20,000
LS	Erosion control @ $5,000/LS	5,000
LS	Loam and seed @ $5,000/LS	5,000
LS	Owner's testing allowance @ $5,000/LS	5,000
LS	General conditions @ $51,000/LS	51,000
	Subtotal	$ 566,000
	Ledge probes	5,000
	Legal allowance	5,000
	Design allowance	50,000
	Inspection allowance	50,000
	Administrative allowance	10,000
	Contingency allowance	57,000
	Total Estimate	$ 743,000

* Cost assumes connecting to existing 8" Ø CI piping on Eden Street. See A.20 for estimate to replace 8" Ø CI piping.

B.4 – ROCKWOOD AVENUE BUILDING IMPROVEMENTS
PRELIMINARY ORDER-OF-MAGNITUDE COST ESTIMATE

QUANTITY	DESCRIPTION/COST		ESTIMATE
LS	Interior painting @ $2,500/LS	$	2,500
LS	Exterior painting @ $3,500/LS		3,500
LS	Repairs to trim work @ $2,000/LS		2,000
LS	Repairs to ceiling @ $2,000/LS		2,000
LS	Replace entry doors @ $2,000/LS		2,000
LS	Reshingle roof @ $4,000/LS		4,000
LS	General conditions @ $2,000/LS		2,000
	Subtotal	$	18,000
	Contingency allowance		2,000
	Total Estimate	$	20,000

B.5 – JACKSON LABORATORY TANK IMPROVEMENTS
PRELIMINARY ORDER-OF-MAGNITUDE COST ESTIMATE

QUANTITY	DESCRIPTION/COST	ESTIMATE
LS*1	Temporary water @ $190,000/LS	190,000
LS*1	Tank modifications @ $340,400/LS	340,400
LS	Site preparation @ $50,600/LS	50,600
	Total Estimate	$ 581,000

1. See following pages for estimate.

PITTSBURG TANK & TOWER GROUP
GROUND DIVISION Since 1919

1 Watertank Place
PO Box 517
Henderson, KY 42419
P: (270) 826-9000
F: (270) 215-5722
www.pttg.com

OCTOBER 22, 2019 PROPOSAL#DD19164

Olver Associates Inc.
290 Main St.
Winterport, ME 04496

Attention: Annaleis Hafford, P.E., Vice President

Reference: Raise 10' by jacking
(1) 53.5' x 30' Existing Welded Steel Ground Storage Tank
Jackson Lab Tank
Bar Harbor, Maine

This is in response to your above referenced inquiry. We are pleased to offer the following preliminary pricing information for your consideration:

SCOPE OF WORK

1.) Furnish all engineering, labor, materials, equipment, and insurance necessary to jack the above referenced storage tank and add a 10' bottom shell ring to increase the storage tank to a 53.5' x 40' storage tank. The new shell ring will be 7/16" A36 carbon steel. The new shell ring will be spot radiographed in accordance with AWWA D100.
2.) We include removal and patching of the existing shell manways and the furnishing and installation of (2) new 30" shell manways in the new bottom ring.
3.) We include extending the fittings and accessories as required due to the tank raising.
4.) We include coating of the new tank materials and any areas damaged by our tank jacking and welding operations. The new interior surfaces and damaged areas will be coated with (2) coats of NSF 61 Approved, Hi Solids Epoxy to a total minimum dry film thickness of 8 mils. The tank interior surfaces will be shop primed. The new exterior surfaces and damaged areas will be coated with (1) coat of Hi Solids Epoxy and (1) coat of Urethane to a dry film thickness of 6.5 mils. The tank exterior surfaces will be shop primed.
5.) We are quoting alternate pricing to sandblast the existing tank interior rusted and abraded areas to an SSPC SP#10 and the remaining areas to a SSPC #7 and then to apply 8 mils of Hi Solids Epoxy.
6.) We are quoting alternate price to clean the existing exterior surfaces by power tool cleaning and to coat the exterior surfaces with (1) full coat of epoxy primer and (1) coat of urethane.

CONDITIONS/EXCEPTIONS

- Our price is based on State of Maine Prevailing Wage Labor.
- We do not include any modification or repair to the tank foundation.
- We do not include any fittings, piping, or accessories unless specifically listed above.

- Our price is based on the site being accessible by tractor-trailer with no overhead or other obstructions and adequate room directly adjacent to the tank foundations for storage of materials and equipment.
- We do not include any hydrostatic testing, disinfection, or bacteriological testing in our proposal.
- Our price is based on standard OSHA safety requirements. We have not provided for fulltime non-productive safety, rescue team, fire watch or other dedicated personnel. Any special safety training and/or site requirements will be at an additional cost.
- Our price is based on standards as set forth by the applicable codes. We have not included for AISC, ISO, QP or any other certifications unless otherwise stated in this proposal. We have not included for dedicated onsite QC personnel such as NACE, CWI or other.
- We have not included any special licenses, fees, permits or bonds. **No tax included.** If any are required, the cost will be added to the quoted price.
- We do not include the handling or disposal of any hazardous materials including but not limited to lead paint or asbestos.
- All prices and schedules offered are subject to prior sale. Quotes on steel materials (both Carbon and Stainless) and consumables are based on the mill price at time of quote. Due to the volatility in the market at this time, material pricing will be revised upon placement of the steel order. Schedule and pricing are based on material availability.

TANK PRICING

Raise (1) 53.5' x 30' Steel Storage Tank x 10'	**$340,350.00**

ALTERNATE PRICING

Paint Existing Interior Surfaces	**$79,880.00**
Paint Existing Exterior Surfaces	**$45,690.00**

TERMS

25% with the order
25% upon delivery of materials and mobilization
25% upon completion of the installation of the 7' shell ring
25% upon completion of tank painting

SCHEDULE

Shop drawings will be submitted for approval 3-4 weeks after receipt of an order.
The tank materials can be delivered 6-12 weeks after receipt of approved drawings.
Field erection will take 3-4 weeks.
Field painting will take 2-6 weeks depending on the option chosen.

Thank you for the opportunity of offering this proposal. If you have any questions, or require any additional information please feel free to call the undersigned at 270-826-9000 ext 2603.

Respectfully submitted,
Pittsburg Tank and Tower Co., Inc.

PTTG Since 1919
GROUND DIVISION

Dennis Davis
Regional Sales Manager
270-826-9000 ext. 2603
270-860-9645 cell
270-831-6963 direct line/fax line
ddavis@pttg.com

cc: Rick DiZinno, Vice President
Vicky Caudill, Special Project Manager

Gretel Breton

From:	Vicky Caudill <vcaudill@pttg.com>
Sent:	Tuesday, October 22, 2019 3:37 PM
To:	Annaleis Hafford
Cc:	Gretel Breton
Subject:	Bar Harbor ME pricing - PORTABLE WATER TANKS

Annaleis,

Below is the portable water pricing for you to work direct with the portable water company to save you money.

210,000 gallons of non-pressurized storage delivered to the testing lab area would be $155,000 for the first 30 days and $1762.00 a day there after
252,000 gallons of non-pressurized storage delivered to the testing lab area would be $190,000 for the first 30 days and $ 2,352.00 a day there after

Thanks,

"100 years and still climbing"

Vicky Caudill
Special Project Manager
Pittsburg Tank & Tower Group
Maintenance Division
PO Box 1849 Henderson, KY 42419

P: 270-826-9000 Ext: 4609 |F: 270-873-8296
www.PTTG.com Facebook LinkedIn Twitter Instagram YouTube


MATCH — MAP 3
MATCH — MAP 2
Eagle Lake
EL. 275.00
Eagle Lake Rd
Park Loop Rd
Cadillac Summit
7
6
5
3
4
1
2
LEGEND
Annual Water Mains
1"ø
2"ø
3"ø
4"ø
6"ø
8"ø
10"ø
12"ø
14"ø
16"ø
18"ø
20"ø
24"ø
Seasonal Water Mains
2"ø
3"ø
4"ø
12"ø
HYDRANT
N
SCALE 1" = 300'
0
150
300
600
MAP 1 0F 7
EXISTING WATER SYSTEM MAP
TOWN OF BAR HARBOR, MAINE
OLVER ASSOCIATES INC.
ENVIRONMENTAL ENGINEERS
290 MAIN STREET
WINTERPORT, MAINE


MATCH – MAP 4
MATCH – MAP 1
Arata Drive
Booster Station
Mountain Ave
Booster Station
Mountain Ave
Pressure Tanks
Strawberry Hill
Booster Station
Jackson Lab
Tank
H-21
H-40
H-19
H-61
H-108
H-96
H-41
H-65
H-63
H-88
H-70
H-106
H-107
H-78
H-76
H-83
H-84
H-75
H-110
H-77
H-93
H-91
7
6
5
3
4
1
2
SCALE 1" = 300'
0
150
300
600
LEGEND
Annual Water Mains
1"ø
2"ø
3"ø
4"ø
6"ø
8"ø
10"ø
12"ø
14"ø
16"ø
18"ø
20"ø
24"ø
Seasonal Water Mains
2"ø
3"ø
4"ø
12"ø
HYDRANT
EXISTING WATER SYSTEM MAP
TOWN OF BAR HARBOR, MAINE
OLVER ASSOCIATES INC.
ENVIRONMENTAL ENGINEERS
290 MAIN STREET
WINTERPORT, MAINE
MAP 2 0F 7


MATCH – MAP 5
MATCH – MAP 1
MATCH – MAP 4
N
H-35
H-34
H-24
H-23
H-46
H-6
H-111
Duck Brook
Pump Station
CCTank/Storage Tank
Summer Tank
(Unused)
7
6
5
3
4
1
2
LEGEND
Annual Water Mains
1"ø
2"ø
3"ø
4"ø
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8"ø
10"ø
12"ø
14"ø
16"ø
18"ø
20"ø
24"ø
Seasonal Water Mains
2"ø
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4"ø
12"ø
HYDRANT
SCALE 1" = 300'
0 150 300 600
MAP 3 0F 7
EXISTING WATER SYSTEM MAP
TOWN OF BAR HARBOR, MAINE
OLVER ASSOCIATES INC.
ENVIRONMENTAL ENGINEERS
290 MAIN STREET
WINTERPORT, MAINE


MAP 4 0F 7
BM: USGS DISC NO. 1
EL. 10.75
MATCH – MAP 3
MATCH – MAP 2
7
6
5
3
4
1
2
LEGEND
Annual Water Mains
1"ø
2"ø
3"ø
4"ø
6"ø
8"ø
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12"ø
14"ø
16"ø
18"ø
20"ø
24"ø
Seasonal Water Mains
2"ø
3"ø
4"ø
12"ø
HYDRANT
EXISTING WATER SYSTEM MAP
TOWN OF BAR HARBOR, MAINE
OLVER ASSOCIATES INC.
ENVIRONMENTAL ENGINEERS
290 MAIN STREET
WINTERPORT, MAINE
SCALE 1" = 300'
0
150
300
600


MAP 5 0F 7
MATCH – MAP 6
MATCH – MAP 3
N
H-52
H-49
H-48
H-92
H-50
H-56
H-109
H-33
H-29
Crooked Rd
7
6
5
3
4
2
1
LEGEND
Annual Water Mains
1"ø
2"ø
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20"ø
24"ø
Seasonal Water Mains
2"ø
3"ø
4"ø
12"ø
HYDRANT
EXISTING WATER SYSTEM MAP
TOWN OF BAR HARBOR, MAINE
OLVER ASSOCIATES INC.
ENVIRONMENTAL ENGINEERS
290 MAIN STREET
WINTERPORT, MAINE
SCALE 1" = 300'
0
150
300
600


SCALE 1" = 300'
0 150 300 600
LEGEND
Annual Water Mains
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16"ø
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20"ø
24"ø
Seasonal Water Mains
2"ø
3"ø
4"ø
12"ø
HYDRANT
N
7
6
5
3
4
2
1
MATCH – MAP 7
MATCH – MAP 5
H-112
H-53
H-54
H-86
H-57
H-55
H-60
H-51
H-58
H-59
Old Bar Harbor Rd
Sand Point Rd
State Highway 3
Stony Brook Way
Owls Nest Lane
Baymeath Rd
Lookout Point Rd
Hillside
EXISTING WATER SYSTEM MAP
TOWN OF BAR HARBOR, MAINE
OLVER ASSOCIATES INC.
ENVIRONMENTAL ENGINEERS
290 MAIN STREET WINTERPORT, MAINE
MAP 6 0F 7

MAP 7 0F 7
SCALE 1" = 300'
0 150 300 600
LEGEND
Annual Water Mains
1"ø
2"ø
3"ø
4"ø
6"ø
8"ø
10"ø
12"ø
14"ø
16"ø
18"ø
20"ø
24"ø
Seasonal Water Mains
2"ø
3"ø
4"ø
12"ø
HYDRANT
7
6
5
3
4
2
1
N
MATCH — MAP 6
State Highway 3
Old Bar Harbor Rd
Ledgewood Rd
EXISTING WATER SYSTEM MAP
TOWN OF BAR HARBOR, MAINE
OLVER ASSOCIATES INC.
ENVIRONMENTAL ENGINEERS
290 MAIN STREET
WINTERPORT, MAINE