Water Needs Assessment Final Report

Town of Rockport, Massachusetts

February 2015

SUBMITTED BY:

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SUBMITTED TO:

Town of Rockport, MA
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Dewberry

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February 4, 2015

Mr. Joseph Parisi
Director of Public Works
Town of Rockport
34 Broadway
Rockport, MA 01966

Subject: Final Report Submission
Water Needs Assessment

Dear Mr. Parisi:

In accordance with our agreement, Dewberry Engineers Inc. (Dewberry) has completed the evaluation of the Town's existing water system to determine its ability to meet present and future water demands over the next twenty years. Enclosed is the Final Report of the Water Needs Assessment summarizing the findings, system analysis and recommendations for your review and comment.

The analyses of the existing water system infrastructure and improvements are based on computer simulations utilizing the newly created WaterCAD model of the Town's water distribution system. A phased capital program of recommended improvements has been prepared to address initial, short-term and long-term system needs through the year 2034 so that the Town can continue to adequately serve its residents.

We appreciate the opportunity to assist the Town of Rockport in planning for future improvements to its water system for this project. At this time, we would like to express our appreciation to the Rockport Department of Public Works for their participation in completing this report, and for their help in collecting and providing data as requested. We would like to give special thanks to Tim Olson, Chris Martin and Matt Barrett for their invaluable contributions to this report.

If there are any questions, or if you need further clarification regarding this submission, please contact me at 617.531.0748, or pcalderazzo@dewberry.com.

Sincerely,

DEWBERRY

Peter A. Calderazzo, P.E.
Associate

Enclosure

Dewberry

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SECTION I - INTRODUCTION

In October 2013, the Town of Rockport contracted with Dewberry Engineers Inc. (Dewberry) to provide the Town with a Water Needs Assessment of its water system supply and infrastructure. Dewberry has completed this assessment and the results of this task are presented in this final report.

The Town has contracted several water needs studies and drought management reports in the past. The reports projected population growth, water demand increases, rainfall predictions, and made recommendations for potential new water sources to accommodate future water needs. The Town has used these studies and reports as an operational guideline and resource for planning and development of new water sources. The Town however has not reached the projected population growth or water usage demands predicted by these reports. Updating this information is necessary for current conditions, future operations, as well as continued planning for additional water sources.

1.1 SCOPE OF SERVICES

The purpose of this report is to evaluate the adequacy of the Town's water system including its supplies, water treatment and water distribution to meet projected domestic water demands as well as its ability to deliver required fire flows at adequate pressures. Dewberry created and utilized a computer model to evaluate the Town's existing water system under various scenarios and demand conditions. Options to alleviate system deficiencies identified from the analysis were evaluated to develop a prioritized plan of recommended improvements to address these system deficiences. Cost estimates for each recommended improvement were also prepared. The goal of this report is to provide the Town with the information necessary to improve its water system to meet current and future water supply needs over the next twenty years.

The scope of work to accomplish this goal is summarized as follows:

Task 1 – Water System Data Collection and Base Mapping

- Collect and review available data for the potable water system infrastructure including maintenance records, water production and consumption records, storage tank levels, hydrant flow tests, pump curves, water quality data and other pertinent documents. Obtain available as-built records, plans and electronic files of the existing water mains for use in producing the water system nodal base map.
- Configure a GIS-based layer of the water system infrastructure utilizing available MassGIS orthophotos, the Town's existing GIS and other collected data for use in constructing the computer model.

Task 2 – Establish Water System Demands and Modeling Scenarios

- Review available water demand and population data developed from the 2006 reports, and update data using historical water production and consumption data available from the Town. Determine current domestic water demands including average day, maximum day and peak hourly demands from the Town's water system.
- Estimate future water demands over 5-, 10- and 20-year planning periods to evaluate available supply and storage capacities within the Town's water system. Future projections will incorporate guidelines and projections by others such as the U.S. Census, Massachusetts Metropolitan Area Planning Council and the University of Massachusetts Donahue Institute (formerly MISER).

- Estimate fire flow needs based on potential development and zoning, historical Insurance Services Office data, industry standards for fire protection and through discussions with local and state officials such as the fire chief.
- Review existing and potential future system supply needs and operations, and establish various supply and operational scenarios to be modeled. These scenarios will include both steady-state and extended period simulations to evaluate water supply and storage system behavior, and hydraulic limitations of the system to meet present and future system needs.

Task 3 – Hydraulic Model Construction, Field Testing and Calibration

- Import the GIS-based water system layer into the WaterCAD/WaterGEMS software. The associated database for the water system components will be populated with pipe data, junction node elevations and demands sources of supply, and water storage tanks information.
- Develop a field testing program to conduct additional flow testing for the development and calibration of the computer model.
- Conduct hydraulic analyses under steady-state conditions and extended period simulations (EPS) to calibrate the newly created computer model using previous hydrant flow test data, available ISO flow data, and the data collected from the field testing program.

Task 4 – Assessment of Existing Water Storage Facilities

- Estimate required storage volumes for equalization, fire flows and emergency for each tank based on current and future water demands and operations. The available "usable storage" for each tank will be determined based on maintaining minimum hydraulic gradients to identify deficiencies with respect to capacity. Diurnal demand curves will be developed to further evaluate the adequacy of the existing storage tanks.
- Assess existing storage tanks to determine existing operating conditions and identify deficiencies with respect to tank cycling and normal fluctuation. Information on overflow elevation, operating range, volume and dimensions, tank control valves and level settings, and pump controls will be collected and reviewed.

Task 5 – Assessment of Existing Water Supplies

- Review storage volumes estimates from the 2006 Water Supply Operations and Drought Management Plans to determine the adequacy of the existing water supplies, and if excess supply capacity is available to meet system needs over 5-, 10- and 20-year planning periods.
- Perform an assessment of the Town's WTP and raw water supply pump stations to determine existing operating conditions and identify deficiencies with respect to supplying the future needs of Rockport. Evaluate alternatives for improving the ability of these facilities and associated infrastructure to meet the domestic and fire protection needs of the future water system.
- Evaluate alternative operational and supply strategies to meet future system demands and drought conditions.
- Evaluate the feasibility of a raw water interconnection with Gloucester as supplied by the Babson Reservoir, and the use of the existing interconnections with Gloucester for augmenting Rockport's supply during drought conditions.

- Assess potential water quality impacts from implementing alternative operational and supply strategies with respect to transferring and mixing different volumes of supply from source waters.

Task 6 – Assessment of Low Pressures and Air Introduction in the Great Hill Area

- Assess the potential cause of air within the Great Hill Area of the Town's water system. Identify possible sources of the air such as excessive turbulence within the system piping, presence of high points, and pumping connections that feed this area of the system.
- Evaluate options for eliminating air from the system or options for improving system pressures within the Great Hill Area.

Task 7 – Hydraulic Analysis of Water System Infrastructure

- Conduct computer simulations using the calibrated hydraulic model to evaluate the adequacy of the existing system infrastructure, and identify current and future deficiencies with respect to water supply, storage and transmission under normal day-time demands, peak-hour demands, night-time demands for refilling the storage tanks, and maximum-day demands with fire flows. Both steady-state and extended period simulations were conducted.
- Conduct computer simulations to evaluate the impact of imposing established operational and supply scenarios on the water system. Analyses will be performed to estimate the available supply and storage capacity that can be hydraulically conveyed through the existing system.
- Conduct computer simulations using the calibrated hydraulic model to evaluate options for improving the system infrastructure to alleviate deficiencies and limitations previously identified with respect to water supply, system storage, transmission and fire protection.

Task 8 – Recommendations and Capital Improvement Program

- Develop recommendations to improve the system infrastructure including distribution, transmission, storage and supply systems to alleviate deficiencies and limitations identified during the completion of the hydraulic analyses to meet present and future water demands through the 5-, 10- and 20-year periods.
- Develop recommendations to improve the capacity and reliability of the existing water supplies to meet future water demands for the operational and supply scenarios modeled including drought conditions.
- Prepare cost estimates associated with the recommended infrastructure improvements to the Town's water supply, storage, transmission, pumping and distribution systems.
- Prepare a prioritized capital improvement program for the Town to implement the recommended infrastructure improvements to its transmission, supply, storage, pumping and distribution systems. The program will be phased over 5-, 10- and 20-year periods and will be coordinated with the Town's fiscal budget.

SECTION II – EXISTING WATER SYSTEM

2.1 GENERAL

The Town of Rockport's water system currently includes three (3) primary surface water supplies, one (1) primary groundwater supply, three (3) storage tanks, two (2) water treatment plants and approximately 40 miles of distribution piping. The system includes approximately 3,550 service connections, and serves residential, commercial, industrial, and agricultural users. For 2013, the average daily water consumption for the system was reported to be approximately 0.57 million gallons per day (MGD), with approximately 65% attributed to residential usage, 6% attributed to commercial, 8% attributed to municipal usage, 17% attributed to un-accounted for usage, and the remaining 4% attributed to industrial, agricultural, recreational and miscellaneous usages. The Town historically experiences a seasonal increase of approximately 30 to 35 percent above its average daily water usage during the summer months due to it being a popular summer destination. In addition to its primary water supplies, the Town also has one (1) potential emergency supply and two interconnections with the City of Gloucester that are available if needed to supplement its primary supplies under extreme water shortage conditions.

The distribution system consists of a single pressure zone, referred to herein as the Main Service system. Raw water from each primary supply is treated at the Town's water treatment plants located off Main Street prior to being pumped into the distribution system. Service elevations in the Main Service system range from as low as 4.5 feet at the south end of Town off Old County Road to as high as 191 feet towards the north section of Town off Landmark Lane. Approximately 4.14 million gallons (MG) of storage are provided within the Main Service system by the three storage tanks, which all operate at an overflow elevation of 234 feet.

The major components and layout of the Town's current water distribution system are shown on the Existing Water Distribution System Nodal Map included as Plate No. 1 in Appendix C. The information presented on this plan is a compilation of data obtained from the Town's existing GIS maps and available records.

2.2 WATER SUPPLY, TREATMENT & PUMPING

Rockport's current water system is comprised of three registered water supplies and a single permitted supply, all located within the North Coastal River Basin. Cape Pond Reservoir, Carlson's Quarry Reservoir and the Mill Brook Well Field are registered supplies, having been in operation prior to the 1950's and before the Water Management Act (WMA) came into effect in 1986. New sources of withdrawals approved after 1986 require a WMA permit. Flat Ledge Quarry was added to the Town's Water Management Act (WMA) Permit as a permitted supply back in 2002. The Town's WMA permit was recently amended in 2010 to include the new bedrock well that is currently being constructed as a permitted supply. This new source is expected to be active by 2015.

The Town's current WMA Permit allows a maximum registered withdrawal of 0.72 MGD, or 262.8 MG in a year, from all four sources combined, with no more than 0.20 MGD withdrawn from the Mill Brook Well Field, and no more than 0.29 MGD withdrawn from Flat Ledge. For 2013, the total raw water pumped from these four supplies was reported to be approximately 214.5 MG, which equates to an average daily withdrawal of 0.588 MGD. The Town commissioned a "Water Supply Operations Plan" in 2006 that included the completion of a hydrographic survey for Cape Pond, Carlson's Quarry and Flat

Ledge. Surveyed and usable water volumes noted in the following paragraphs for these supplies are as reported in this study.

2.2.1 Water Treatment

Treatment of the Town's water supplies is provided by two water treatment plants that are located at the DPW Facility off Main Street behind the Police Station just across from Hill Top Lane (see Plate 1 in Appendix C). The two plants include the following:

- Rapid Sand Filtration (RSF) facility constructed in 1939 that predominately treats water from Cape Pond Reservoir
- Dissolved Air Flotation (DAF) facility constructed in 1998 to treat water from the combined sources of Carlson's Quarry and Flat Ledge along with water from the Mill Brook Well Field

The Town also has the ability to divert raw water from Carlson's Quarry and the Mill Brook Well Field directly into Cape Pond to treat all of its sources through the RSF facility if needed. The two treatment facilities operate in parallel with each having a maximum capacity of approximately 700 gallons per minute (gpm), or 1 MGD, for a total treated water capacity of 1,400 gpm, or 2.0 MGD. Finished water from each treatment facility is combined into a dual 60,000 gallon and 300,000 gallon common clearwell system where it is then pumped into the distribution system via two 40 HP horizontal split-case finished water pumps located in the basement level of the RSF facility. Each pump has rated capacity of 1,185 gpm at a total dynamic head (TDH) of 105 feet.

Both treatment facilities include pre- and post pH adjustment, coagulation, post disinfection and fluoridation. The RSF facility has the ability to add potassium permanganate as an enhancement for treating high iron and manganese concentrations that are experienced within Cape Pond when water levels become low enough. Both the Mill Brook Well Field and Carlson's Quarry have also exhibited elevated iron and manganese when pumped down below certain levels. During these instances, the Town can pump Carlson's Quarry and the Mill Brook Well Field directly into Cape Pond to take advantage of the more effective treatment for iron and manganese removal as provided by the RSF facility. The benefit of this operational feature with respect to optimizing the Town's existing water supplies to meet future water demands is discussed in Section 5.

2.2.2 Surface Water Supplies and Pump Stations

Cape Pond

Cape Pond Reservoir is Rockport's oldest source of water supply and the Town's primary water supply source. This natural water body has a reported watershed area of approximately 0.350 square miles, and contains approximately 175 MG of storage when full based on a total depth of 16 feet. The Town currently utilizes Cape Pond based on a maximum operating range of 8 feet to maintain at least 6.75 feet of water above the intake, which based on the hydrographic study completed, correlates to an estimated "useable" supply volume of approximately 106 MG.

The safe yield for this supply as reported in the 2003 "Comprehensive Water Resources Management Plan" (CWRMP) is approximately 113 MG per year, or 0.310 MGD, based on an estimated "usable" supply capacity of 108 MG. For 2013, the Town pumped approximately 100.9 MG of raw water from Cape Pond (0.277 MGD), which represents 47% of the total supply pumped.

Raw water from Cape Pond is pumped up to the original RSF facility for treatment via the low lift pump station which includes three 25 HP vertical turbine can pumps that provide a maximum capacity of 700 gpm, or 1 MGD, with two pumps running. The intake consists of approximately 800 feet of 12" cast iron pipe that runs along the bottom of the reservoir and terminates with a 24" diameter well screen connected to a 24" 90-degree turned up bend. The screen and fitting is secured to a timber pad base that sits on the reservoir bottom and is supported on piles. The top of the screen is set to be 14.75 feet below the full reservoir level. On the pump end, the 12" intake pipe enters the lower level of the pump station and connects to the pump manifold at an elevation of approximately 79.5 feet, which is just below the reported full water level elevation of 80.0 feet. The pumps sit about five to six feet below the centerline of the intake pipe connection within the cans. The low lift pump station also includes a vacuum-priming system, which is required to operate the pumps due to the elevation of the station as compared to the water level within Cape Pond. The limitation of this pump station with respect to accessing additional "useable" volume from this source is discussed in Section 5.

Carlson's Quarry

Carlson's Quarry is a man-made quarry that has been in use as a raw water supply since 1953. This supply has a reported watershed area of approximately 0.330 square miles, and contains approximately 92 MG of storage when full based on a total available depth of 90 feet. The Town however, maintains a minimum of 18 feet of water above the pump intake that is set 80 feet below the full water level, which based on the hydrographic study completed, correlates to an estimated "useable" supply capacity of approximately 81 MG. As shown on the hydrographic survey completed in 2006, there is a definitive ridge approximately 20 feet below the full water level that divides the quarry into two bowls. The 2006 "Water Supply Operations Plan" indicated that as this source approaches an elevation of 70 feet, or 20 feet below high water level, water quality begins to degrade as iron and manganese levels begin to increase, and that only the top 20 feet should be considered "useable" volume for supply. However, as previously noted, the Town has the flexibility to divert raw water from Carlson's Quarry into Cape Pond for treatment through the RSF Plant, which is better equipped to handle elevated iron and manganese. Therefore, the "useable" volume from Carlson's Quarry should not be limited by water quality, but by the capacity of the pump station and intake. Another potential limiting factor is the noted ridge that would isolate a portion of the useable volume for Carlson's Quarry when water levels drop below the ridge elevation. The impact of this potential limitation is discussed in Section 5.

The safe yield for Carlson's Quarry as reported in the 2003 "CWRMP" is approximately 81 MG per year, or 0.222 MGD, based on an estimated "usable" supply capacity of 99 MG. For 2013, the Town pumped approximately 83.34 MG of raw water from Carlson's Quarry (0.228 MGD), which represents 39% of the total supply pumped.

Under normal operations, raw water from Carlson's Quarry is pumped up to the newer DAF facility for treatment via a raw water pump station which includes one 50 HP submersible pump rated at a nominal capacity of 700 gpm, or 1 MGD. The pump sits approximately 80 feet below the full water level elevation within a 12-inch pipe that runs down along the side of the quarry, terminating with a screen.

Flat Ledge Quarry

Flat Ledge Quarry, added to the Town's WMA permit as a permitted supply in 2002, has a reported watershed area of approximately 0.144 square miles, and according to the 2006 "Water Supply Operations Plan", contains approximately 62 MG of storage when full, based on a total available depth of approximately 100 feet. The 2006 report referred to a reading of "zero" on the Town's gauge as an indication of full capacity, but according to Town staff, there is no gauge currently in use at this supply. The 2006 plan also noted that based on maintaining a targeted minimum water level of 30 feet above the pump intake for operational purposes, the total estimated "useable" capacity would be 59 MG.

However, no reference to a specific intake elevation is noted, and there is currently no permanent intake structure within the Flat Ledge Quarry.

The safe yield for Flat Ledge as stated in the DEP-approved WMA Permit dated 2002 is 0.29 MGD, which is equivalent to its maximum daily permitted withdrawal. For 2013, the Town pumped approximately 17.65 MG of raw water from Flat Ledge (0.0484 MGD), which represents 8% of the total supply pumped. Raw water from Flat Ledge is pumped directly into the adjacent Carlson's Quarry via a 10 HP submersible pump that is rated at a capacity of 90 gpm, or 0.13 MGD. The pump hangs about 40 feet below the full water level off a 4-inch pipe that extends out of the water and continues along an access road up and over the Carlson's Quarry Dam. The Town has been discussing the option of constructing a more permanent larger pump station and intake, similar to the one at Carlson's Quarry, to access the full available supply capacity of Flat Ledge.

2.2.3 Groundwater Supply

The Mill Brook Well Field is a registered ground water source that includes three 36-foot deep, 12-inch x 8-inch gravel-pack wells equipped with submersible pumps that provide a maximum capacity of 140 gpm, or 0.2 MGD. Water from these wells is pumped up to the Town's water treatment plants where flow can either be combined with raw water from Carlson's Quarry prior to entering the DAF facility, or can be diverted into Cape Pond for treatment through the RSF facility. The Mill Brook Well Field has a DEP-approved pumping rate, or safe yield, of 0.2 MGD. For 2013, the Town pumped approximately 12.60 MG of raw water from the Mill Brook Well Field (0.0345 MGD), which represents 6% of the total supply pumped.

2.2.4 Summary of Existing Water Supplies

Table 2-1 presents a summary of the Town's primary water supplies including estimated safe yields obtained from previous reports completed for the Town and as approved in the WMA permits issued by the MassDEP. The difference between the total volume and usable volume for Cape Pond, Carlson's Quarry and Flat Ledge reflect adjustments made to address water quality issues experienced when withdrawing these supplies below certain levels, and to maintain adequate intake levels.

TABLE 2-1
EXISTING WATER SUPPLIES

Supply	Total Surveyed Volume (MG)(3)	Total Usable Volume (MG)	Reservoir & Watershed Areas (sq. mi)(1)	Estimated Annual Safe Yield (MGD)(2)	Estimated Annual Safe Yield (MG)(2)
Cape Pond	175	106(4)	0.350	0.31	113
Carlson's Quarry	92	81(5)	0.330	0.22	81
Flat Ledge Quarry	62	59(6)	0.144	0.29(7)	106(7)
Mill Brook Well Field	n/a	n/a	0.552	0.20	73
Totals	**329**	**246**	**1.376**	**1.02**	**373**

1. Taken from the 1997 "Water Supply Management Plan" prepared by Metcalf & Eddy.
2. Taken from the 2003 "Comprehensive Water Resources Management Plan" prepared by SEA.
3. These values represent the hydrographic surveyed volumes presented in the 2006 "Water Supply Operations Plan" prepared by SEA.
4. Based on pumping Cape Pond over an operating range of 8 feet.

5. Based on maintaining an operating range of 18 feet over the pump intake.
6. Based on maintaining 30 feet of water over pump intake for operational purposes.
7. Based on the 2002 WMA Permit Application.

As shown in Table 2-1, the total combined estimated safe yield for the Town's existing water supplies is 1.02 MGD which exceeds the maximum registered withdrawal of 0.72 MGD, and provides a surplus of approximately 0.3 MGD. However, it should be noted that the estimated safe yields are based on having the total volume of storage available from each source. From the 1997 "Water Supply Management Plan" prepared by Metcalf & Eddy, the estimated safe yields for Cape Pond and Carlson's Quarry were based on usable storage volumes of 108 MG and 99 MG, respectively.

For Cape Pond, the estimated useable volume of 106 MG presented in Table 2-1 is almost equivalent to its safe yield estimate of 108 MG noted in the 1997 report, and therefore most of its estimated safe yield can be considered available. For Carlson's Quarry, the estimated useable volume of 81 MG presented in Table 2-1 is approximately 82-percent of its safe yeild estimate of 99 MG noted in the 1997 report. As such, not all of its estimated safe yield may be available, which would reduce the total combined safe yield of the Town's existing water supplies noted in Table 2-1. The impact of this limitation to the overall adequacy of the Town's water supplies is further discussed in detail in Section 5 of this report.

2.2.5 New Bedrock Well Supply

In addition to the four primary water supplies discussed above, Dewberry has recently completed the design for a new Bedrock Well and Pump Station located within the upland wooded area approximately 600 feet east of Cape Pond *(see Plate 1 in Appendix C)*. This new station will pump raw water from the existing 400-foot deep, 8-inch bedrock well directly into Cape Pond to supplement its water level, similar to the diversion of the Mill Brook Well Field into Cape Pond. The Town's WMA permit has been amended to include this new well source as a permitted supply with a Mass-DEP approved maximum withdrawal rate of 204 gpm, or 0.29 MGD. The impact of this new well source in improving the adequacy of the Town's water supplies to meet future water needs is further discussed in detail in Section 5 of this report.

2.3 WATER STORAGE

As prevously noted, there are three (3) water storage tanks located within the Town's existing water distribution system including: the Pool's Hill Storage Tank located off the end of Summit Avenue; the Pigeon Hill Storage Tank located at the end of Landmark Lane; and the South End Storage Tank located off Thatcher Road. These tanks are depicted on the Existing Water Distribution System Nodal Map included as Plate No. 1 in Appendix C. Table 2-2 presents the storage volume, dimensions and overflow elevations for each of the tanks.

As shown in Table 2-2, all three tanks operate at the same hydraulic gradient of 234 feet (MSL) and serve the Main Service system. Dewberry is currently designing the in-kind replacement of the Pigeon Hill Tank which is expected to be constructed in the Summer of 2015.

TABLE 2-2
EXISTING STORAGE FACILITIES

Storage Tank	Year Built	Tank Type	Overflow Elevation (feet)	Height (feet)	Diameter (feet)	Storage Volume (MG)	Gallons/ Foot
Pool's Hill	1997	Concrete	234.0	33.0	124.60	3.000	90,909
Pigeon Hill	1930s	Riveted Steel	234.0	27.5	44.00	0.313	11,382
South End	1962	Welded Steel	234.0	115.0	35.00	0.830	7,217

2.4 DISTRIBUTION SYSTEM

Rockport's present water distribution system includes one pressure zone, referred to herein as the Main Service system, and is comprised of approximately 40 miles of water main varying in diameter from 6 inches to 16 inches. The majority of the original distribution piping has been replaced and is primarily composed of cement-lined ductile iron, with sections of unlined cast iron. There are approximately 425 hydrants within the system for fire protection purposes. The Main Service system operates at a gradient of 234 feet as maintained by the three water storage tanks listed in the previous section. The major components and layout of the existing distribution system are shown on the Existing Water Distribution System Nodal Map included as Plate No. 1 in Appendix C.

Finished water is pumped from the water treatment plant's clearwell into the Main Service System through a 16-inch ductile iron pipe that extends along the plant property, and into Main Street. This 16-inch transmission main continues northerly along Main Street towards the center of town where it connects to a 16-inch transmission main in Granite Street that feeds the northern section of Rockport, and a 10-inch transmission main in Broadway that continues in a northeasterly direction intersecting with Mount Pleasant Street. At this intersection, a 12-inch transmission main continues along Mount Pleasant Street to feed the southern section of Rockport.

Service elevations within the Main Service system range from as low as 5 feet at the south end of Town off Old County Road to as high as 191 feet towards the north section of Town off Landmark Lane near the Pigeon Hill Tank. Based on the current operating gradient of 234 feet as maintained by the three storage tanks and the fact that one pound per square inch (psi) equals 2.31 feet of water, system pressures would expect to be in the range of approximately 20 to 110 psi as calculated below:

234 feet - 191 feet = 43 feet x 1 psi/2.31 feet = 18.6 psi
234 feet - 5 feet = 229 feet x 1 psi/2.31 feet = 99.1 psi

The State Plumbing code recommends that any incoming services with pressures greater than 85 psi should be provided with a pressure-reducing valve to control water hammer and minimize water loss. The minimum DEP recommended service elevation pressure for domestic service is 35 psi, which equates to approximately 81 feet in elevation (35 psi x 2.31 feet /1 psi). Therefore, the Main Service system should provide adequate pressures for service elevations up to approximately 153 feet (234 feet-81 feet), depending on the condition and associated head losses of the existing mains in the area.

The Main Service system provides sufficient pressure for domestic service to most areas within the Town, except for several locations that fall outside the recommended range of 35 to 85 psi. These locations are shown on Plate 1 included in Appendix C and are also identified in Table 2-3 below. From information provided by the Town, the neighborhood that consistently experiences low pressure along with the buildup of air is known as the Great Hill area. This area is located north of the water treatment plant along the highest elevation of State Highway Route 127 (Main Street). It serves a number of residential homes, a few businesses and the municipal facilities of the Police Station, the Public Works Garage and the Rockport Water Treatment Plants.

TABLE 2-3
HIGH AND LOW SERVICE ELEVATIONS

Location	Service Elevation(1) (ft)	Calculated System Pressure(2) (psi)	Nearest Tank
Wildon Heights (Great Hill Area)	169	28	Pool's Hill
Summit Ave/Squam Rd	161	32	Pool's Hill
Pigeon Hill St/Landmark Ln	191	19	Pigeon Hil
Old County Rd/Thatcher Rd/Old Penzance Rd	5	99	South End
Gap Head Rd	21	90	South End

1) Estimated from Town's GIS contour map
2) Based on system operating gradient of 234 feet.

From discussions with Town staff, prior to 1998, a standpipe-type storage tank was located approximately where the new DAF facility now stands, and operated at the same hydraulic gradient of 234 feet as the current system does. Based on recommendations made at that time, the existing standpipe was not replaced after it was taken down to accommodate the construction of the new plant. Since the removal of the standpipe, the residents of the Great Hill area have noticed issues with low water pressure and entrapped air.

Although the current system gradient in this area has not changed due to the removal of the standpipe, its removal seems to be the cause of low pressure within the Great Hill area. As shown in Table 2-3 above, the Great Hill Area is calculated to be below the minimum 35 psi recommended by DEP and complaints of low pressure are to be expected. It is our understanding there are a large number of fittings installed within the piping that serves this area, which can further reduce service pressures due to resulting friction and minor losses. In addition, sudden changes in direction such as from bends within the distribution piping can generate surges and turbulent flow that could potentially introduce air into the piping, which if not released, can become trapped at high points affecting system pressures. A detailed discussion including possible solutions to address the Great Hill area issue is presented in Section 6 of this report.

SECTION III – WATER SYSTEM DEMANDS

3.1 GENERAL

Water usage projections are utilized to assess the impact of future increases in water demands on the supply, storage and distribution components of a system. The adequacy of a water system to meet these future water consumption needs is determined by evaluating historic water consumption data and projecting future system needs. This analysis establishes water usage trends that are then used to forecast future water consumption based on population projections. This section presents an analysis of historic water consumption information and estimated future water demands based upon projected population and water usage through the study period of 2034.

3.2 POPULATION PROJECTIONS

In order to develop population projections for this report, Dewberry first reviewed past population data for the Town of Rockport. Past population figures as reported by the Town since 2006 are included in Table 3-1. The data shows a somewhat stable population with an actual reduction of approximately 270 people over the past eight years.

TABLE 3-1
HISTORICAL POPULATION

Year	Population	Percent Change
2006	7,314	n/a
2007	7,475	2.2%
2008	7,480	0.1%
2009	7,405	-1.0%
2010	7,472	0.9%
2011	7,303	-2.3%
2012	7,279	-0.3%
2013	7,053	-3.1%

To assist in forecasting future growth for the Town, Dewberry next researched available population data as published by the US Census, UMass Donahue Institute (UMDI, formerly known as MISER), and the Massachusetts Metropolitan Area Planning Council (MAPC). Table 3-2 presents the population projections as developed by the UMDI and MAPC up to the year 2030 (the US Census only reports on National and State level population projections and therefore was not considered in this evaluation).

As shown in Table 3-2, the MAPC provides two population projection scenarios: The Status Quo Scenario (SQ) and the Stronger Region Scenario (SR). The SQ scenario is based on the continuation of existing rates of births, death, migration and housing occupancy. The SR scenario is based on assumptions and trends that could potentially produce higher population growth, greater housing demand, and substantially larger workforces. Since the SR scenario is intended for urban areas, it is unlikely that this scenario would be representative of future growth in Rockport.

However, both the UMDI and MAPC SQ/SR projections predict the population in Rockport will steadily decline over the next 16 years. The UMDI forecasts an overall population decline of 35%, the MAPC SQ predicts a 5% decline, and the MAPC SR predicts 3% reduction in population

DDTABLE 3-2
POPULATION PROJECTIONS BY UMDI/MAPC

Year	UMDI	MAPC SQ	MAPC SR
2010(1)	6,952	6,952	6,952
2015	6,385	-	-
2020	5,854	6,791	6,877
2025	5,243	-	-
2030	4,547	6,604	6,763

(1)Actual population as reported from the 2010 US Census.

According to the 2011 "Downtown Master Plan" prepared by the Rockport Planning Board, there is one commercially developed parcel with a lot area of 1,050 square feet (sf) and 4 undeveloped commercial land parcels totaling 3,245 sf within the downtown area (Refer to Table 3: Current Land Use Profile in the Downtown Master Plan). This study also noted that the population from 2000 to 2010 has declined, citing an increase in ownership by seasonal visitors who are not counted as residents, and smaller household sizes as the reason.

From discussions with Town staff, there are two developments currently in the planning process including: the Landing at Pigeon Cove, a 14-lot subdivision off Granite Street along the waterfront that includes one commercial and 13 residential properties, and Woodland Acres, a 40-lot residential subdivision off Woodland Road in the northern section of Town. These two developments are depicted on Plate 2 included in Appendix C. In reviewing the Town's GIS plan, we have identified approximately 52 vacant residential-zoned parcels within the limits of the Town's existing water distribution system, with 34 of these parcels designated as undevelopable and 18 of these parcels designated as developable. Out of the 18 developable parcels, 4 are currently outside of the Town's current sewer district. However, 3 of these parcels are located within the supplementary sewer districts as identified in the "Sewer Capacity Needs Assessment" prepared by Kleinfelder dated October 29th, 2013. Assuming that the existing sewer system is expanded in the future to include these supplementary sewer districts, there are 17 vacant parcels that could be potentially developed as residential properties and connected to the water system.

The "Sewer Capacity Needs Assessment" also estimated that the Town would experience an average of 5 bedroom additions per year over a twenty year period, which for a reported volume of 35 gallons per day (gpd) per bedroom, would result in a total increase in wastewater flow of 3,500 gpd. Assuming that these bedroom additions would occur to properties already served by the Town's water system, a similar increase in water usage would also be expected.

Another possible source of expansion that would affect water supply is the future connection of private well owners to the water system. Based on available GIS data provided by the Town, Dewberry identified eighty-five (85) residential properties within the limits of, or close proximity to (< 1,000 feet), the existing water system that currently use private wells. These properties are shown on Plate 2 included in Appendix C. As noted in Section 2.4, based on the existing gradient of the Town's water system, adequate pressures can only be provided to service elevations up to 150 feet. Therefore, any property with private wells above an elevation of 150 feet would need to remain on private wells. Approximately 10 of the 85 private well properties are above a service elevation of 150 feet, which results in a total of 75 residential properties on private well that could potentially connect to the water system over the next twenty year planning period.

Based on data published by the US Census for 2010, the average number of people per household in the Town of Rockport is 2.14 as compared to the average household size in the United States of 2.59. To provide a conservative estimate for planning, we will use a value of 2.4. From the data presented above, there are approximately 145 residential parcels/properties that could potentially connect to the water system within the study period, which would result in approximately 348 additional residents to be served (145 households x 2.4 people per household). Accounting for the additional 100 bedroom additions estimated in the "Sewer Capacity Needs Assessment", the population served by the Town's water system could increase by up to 448 people. Based on the reported population served of 7,053 for 2013, this would represent an overall growth of approximately 6.4%. Taking into consideration the projected decline in population of 3% to 5% as predicted by the MAPC, the population served at the end of the study period is predicted to increase by 1.4% to 3.4%, respectively.

To allow for some contingency in planning for the future water supply needs of the Town, Dewberry has projected a 5% increase in the total population served by the Town's water system to the year 2034. This projection results in a total population served of approximately 7,406 people. A summary of the projected population for the Town's water system over the study period is shown in Table 3-3. Dewberry will use this percent increase in population served to establish future residential water demands as presented in the following section.

TABLE 3-3
POPULATION PROJECTIONS
BY DEWBERRY

Year	Projected Population
2014(1)	7,053
2019	7,141
2024	7,729
2029	7,317
2034	7,406

(1) Year 2013 population of 7,053 assumed for year 2014

3.3 WATER DEMAND PROJECTIONS

3.3.1 Average Day Demands

Average day demands are commonly defined as the total water supplied in one year divided by 365 days, expressed in millions of gallons per day (MGD). This would include water used for residential, agricultural, commercial, and industrial consumption, as well as unaccounted-for water. Historical average day water consumption for the Town of Rockport was obtained for years 2006 through 2013 and is presented in Table 3-4 on the following page.

As shown in Table 3-4, the total average day demand had been decreasing from 2006 through 2010, but has been increasing slightly for the past three years, with a 5% and 7% increase in 2012 and 2013, respectively. Since 2011, the demands for each user classification has been relatively stable with residential usage making up approximately 70% of the total average day demand, commercial usage making up approximately 7%, and municipal usage making up approximately 8%. The remaining percentage includes institutional, industrial, agricultural and recreational (Other) usages at approximately 2% of the total average day demand, and unaccounted-for water usage making up approximately 11%. The Town of Rockport maintains individual meter reading records for all residential, industrial, institutional, commercial and agricultural water users in town.

TABLE 3-4
HISTORICAL AVERAGE DAY DEMAND BY USER CLASSIFICATION

Year	Residential Demand (MGD)	Commercial Demand (MGD)	Municipal Demand (MGD)	Other [1] Demand (MGD)	Unaccounted-For Water (MGD)	Total Average Day Demand (MGD)
2006	0.427	0.032	0.052	0.012	0.072	0.595
2007	0.424	0.041	0.024	0.025	0.075	0.589
2008	0.395	0.036	0.025	0.022	0.060	0.538
2009	0.384	0.035	0.028	0.027	0.039	0.513
2010	0.397	0.038	0.031	0.016	0.007	0.489
2011	0.372	0.034	0.031	0.019	0.041	0.497
2012	0.373	0.038	0.049	0.019	0.044	0.524
2013	0.370	0.033	0.020	0.025	0.115	0.563

[1] The user classification "Other" includes institutional, industrial, agricultural and recreational usages.

Unaccounted-for water within the system typically consists of water used for street cleaning, flushing, leakage from the system, meter losses, and firefighting. From-Table 3-4, with the exception of 2013, the unaccounted for water usage since 2009 has been approximately 8%, which is below the Town's WMA Permit performance standard of 10% unaccounted-for water. There were significant leaks in 2013 that resulted in the noted unaccounted-for water usage of 17%. The Town has since repaired these leaks and it is expected that the unaccounted-for water for 2014 and beyond will be more consistent with the percentages observed in previous years. Based on the Town's ongoing policies with respect to water conservation, we have assumed that the unaccounted-for water will remain constant throughout the study period at 10% of the total water supplied.

As shown in Table 3-4, the combined demand of institutional, industrial, agricultural and recreational water usage represented by the "Other" user classification has remained consistent with an average usage of approximately 0.020 MGD from 2010 to 2013. Municipal and commercial water usages have also remained consistent over the same period averaging 0.036 MGD and 0.038 MGD, respectively.

An increase in municipal and "Other" usage categories is not anticipated to occur over the study period given that there is only one service connection listed in the 2013 ASR per each agricultural, industrial and institutional category, and there are no planned developments in these types of categories. Therefore, as with the unaccounted-for water, we have assumed that water usages for these two user classifications will remain constant throughout the study period and have applied the noted averages to estimating future water needs.

As previously noted, there is one planned commercial property as part of the Landing at Pigeon Cove development and there are several small, undeveloped parcels available within the downtown area that could support some commercial growth. Given the Town's unique status as a seasonal tourist destination, there is also the potential that an increase in seasonal visitors over the next twenty years could lead to the expansion or redevelopment of existing commercial establishments. To account for such a scenario, we have estimated that the commercial demands will increase by approximately 25% of the estimated total average demand through the study period. This will provide a conservative estimate for water supply planning and allow for some growth within the commercial zones of the Town.

To estimate future residential water usage, a per capita day consumption based on past water usage data is first determined which would then be multiplied by the projected population served for a given year. The historical per capita day consumption for the Town of Rockport is presented in Table 3-5 below. The average per capita day consumption for the years 2006 through 2013 was approximately 53 gallons per capita day (gpcd). Comparatively, the average per capita day consumption from 2006 to 2008 was approximately 56 gpcd, where the average per capita day consumption from 2009 through 2013 was approximately 52 gpcd. This reduction in per capita usage can be attributed to the water conservation methods that the Town has implemented over the years to meet the performance standards of 65 gpcd required in their WMA Permit.

TABLE 3-5
HISTORICAL PER CAPITA AVERAGE DAY DEMANDS

Year	Residential Demand (MGD)	Population Served	Per Capita Consumption (gpcd)
2006	0.427	7,314	58.4
2007	0.424	7,475	56.7
2008	0.395	7,480	52.7
2009	0.384	7,405	51.8
2010	0.397	7,472	53.2
2011	0.372	7,303	51.0
2012	0.373	7,279	51.3
2013	0.370	7,053	52.5

Public awareness of water conservation, for both economic and environmental reasons, has increased in recent years due in part to efforts by municipalities and regulatory agencies. For this reason, per capita water consumption should remain relatively constant over the next twenty years. As noted above, the Town's WMA Permit allows a per capita water consumption of up to 65 gpcd. Although there is no reason why the Town should ever approach this value given its recent history, it would not be practical for planning to assume that the Town would continue to maintain such a low per capita consumption rate over the next twenty years. For the purposes of estimating future residential demands, we have assumed that the Town's per capita water consumption will gradually increase every five years beginning in 2014 at 53 gpcd and ending in 2034 at a maximum value of 60 gpcd. Based on the previous population projections developed as presented in Table 3-3, we have estimated the future residential average day demands for the study period as shown in Table 3-6 on the following page.

Based on the projected residential water usage presented in Table 3-6, and the criteria established for the other user classifications including commercial, municipal, "Other" and unaccounted-for usages, we have estimated the future total average day demands for the Town as presented in Table 3-7 on the following page.

TABLE 3-6
PROJECTED RESIDENTIAL AVERAGE DAY DEMANDS

Year	Projected Population Served	Per Capita Consumption (GPCD)	Projected Residential Demand (MGD)
2014	7,053	53.00	0.374
2019	7,141	54.75	0.391
2024	7,229	56.50	0.408
2029	7,317	58.25	0.426
2034	7,406	60.00	0.444

TABLE 3-7
PROJECTED TOTAL AVERAGE DAY DEMANDS

Year	Residential Demand (MGD)	Commercial Demand (MGD)	Municipal Demand (MGD)	Other [1] Demand (MGD)	Unaccounted-For Water (MGD)	Total Average Day Demand (MGD)
2014	0.374	0.036	0.038	0.020	0.037	0.506
2019	0.391	0.038	0.038	0.020	0.039	0.526
2024	0.408	0.040	0.038	0.020	0.041	0.547
2029	0.426	0.042	0.038	0.020	0.042	0.569
2034	0.444	0.045	0.038	0.020	0.044	0.592

[1] The user classification "Other" includes industrial, agricultural, recreational and miscellaneous usages.

3.3.2 Maximum Day Demands

In determining the adequacy of the existing water supply system, the available supply must be capable of meeting the maximum day demands of the system. Maximum day demand is defined as the highest one day demand which occurs during the year, and is commonly expressed as a percentage of the average day demand. Table 3-8 presents the reported historical maximum day water demands from 2006 through 2013. As shown, historical maximum day demands have varied from 175% to 238% of the average day demand, with an average maximum day/average day ratio of approximately 2.09. From discussions with Town staff, the reported maximum day demand of 1.162 MG for 2013 which occurred on January 1, 2013 was a result of a major leak and does not reflect actual water usage. Historically, the maximum day demands for the Town have occurred primarily during the months of July and August as shown in Table 3-8. For 2013, the month with the highest amount of finished water pumped was reported to be July.

For the purposes of this study, a maximum day/average day ratio of 2.10 will be used to evaluate the adequacy of the Town's water system to meet high demand periods. Based on this ratio, we have estimated future maximum day demands as shown in Table 3-9. The future average day and maximum day demands estimated in this section will be used in the following sections to evaluate the adequacy of the existing water system to meet future needs, and to evaluate system improvements utilizing the computerized hydraulic model created as part of this study.

TABLE 3-8
HISTORICAL MAXIMUM DAY DEMANDS

Year	Average Day Demand (MGD)	Maximum Day Demand (MGD)	Maximum Day Date	Maximum Day Ratio
2006	0.595	1.155	7/4/06	1.94
2007	0.589	1.260	7/27/07	2.14
2008	0.538	1.220	9/3/08	2.27
2009	0.513	1.223	8/15/09	2.38
2010	0.489	1.140	7/6/10	2.33
2011	0.497	0.870	7/23/11	1.75
2012	0.524	0.966	7/9/12	1.84
2013	0.563	1.162	1/13/13	2.06
		Average Maximum Day Ratio		2.09

TABLE 3-9
PROJECTED MAXIMUM DAY DEMANDS

Year	Projected Average Day Demand (MGD)	Projected Maximum Day Demand (MGD)
2014	0.506	1.063
2019	0.526	1.105
2024	0.547	1.149
2029	0.569	1.195
2034	0.592	1.243

3.3.3 Peak Hour Demands

Peak hour demands are the highest hourly demands that occur during a 24-hour period and generally occur in conjunction with the maximum day demand. Peak hour demands can vary anywhere from 1.5 to 6 times the average day demand, and therefore should be met through system storage. Consequently, peak hour demands are considered when evaluating the adequacy of system storage and the ability of system pipelines to deliver such demands. To calculate peak hour demands for a system, hourly recorded pumping and tank level data for the maximum day event are needed. Unfortunately, this data is not currently available for the Town's water system. Based on our experience, peak hour demands for towns similar to Rockport typically range from 1.5 to 2.0 times the maximum day demands. In the absence of hourly tank level and pumping data, we will use a peak hour demand/maximum day ratio of 2.0 to provide a conservative estimate for evaluating system storage and capacity, and to ensure the Town will be able to meet potentially high demand periods. Based on this ratio, we have estimated future peak hour demands as shown in Table 3-10. The impact of peak hour demands on system storage is discussed further in Section 4. The impact of peak hour demands on the distribution system with respect to pipeline capacities is discussed in Section 6.

TABLE 3-10
PROJECTED PEAK HOUR DEMANDS

Year	Projected Maximum Day Demand (MGD)	Projected Peak Hour Demand (MGD
2014	1.063	2.126
2019	1.105	2.210
2024	1.149	2.298
2029	1.195	2.390
2034	1.243	2.486

3.3.4 Seasonal Day Demands

Rockport is a popular summer vacation destination given its location in the northern tip of Cape Ann and is reported to experience an increase of almost twice its year round population during the summer months due to a significant number of summer homes and transient accommodations. This seasonal surge in population results in a higher average day demand for the months of July and August as evident from past pumping records. Table 3-11 presents the historical average day demand along with the seasonal day demands as estimated from the finish water pumping records from 2006 through 2013 for July and August. As shown in Table 3-11, the seasonal day demand for the months of July and August are about 30% greater than the average day demand for the year. Since the historical maximum day demands account for this seasonal fluctuation, the projected maximum day and peak hour demands to evaluate system storage and pipeline capacity will also account for this seasonal fluctuation. The impact of seasonal day demands will be on the ability of the Town's water supplies to meet future water needs, which we discuss in Section 5.

TABLE 3-11
SEASONAL DAY DEMANDS

Year	Average Day Demand (MGD)	Seasonal Day Demand (1) (MGD)	Seasonal Ratio
2006	0.595	0.791	1.33
2007	0.589	0.794	1.35
2008	0.538	0.688	1.28
2009	0.513	0.607	1.18
2010	0.489	0.673	1.38
2011	0.497	0.658	1.32
2012	0.524	0.722	1.38
2013	0.563	0.748	1.33

(1) Seasonal Day Demand is the reported average monthly demands for July and August.

SECTION IV – WATER STORAGE REQUIREMENTS

4.1 GENERAL

Distribution storage is required to meet peak demands of short duration, minimize pressure fluctuations during periods of demand changes in the distribution system, and furnish a reserve for fire fighting. Storage may also serve to provide an emergency supply in case of temporary breakdown of pumping facilities or major transmission main failure. Required volumes of storage, denoted as "usable", are allocated at specific levels within a storage facility to ensure the storage volume will be available at a hydraulic gradient adequate for the intended purpose. There are three components to consider when evaluating storage capacity. The first is equalization storage provided at the top portion of the tank for meeting peak demands and hourly demand fluctuations, with fire storage directly below. Emergency storage is a function of the system characteristics and is determined based on the assessed risk and need. The following presents an analysis of the Town's water storage requirements.

4.1.1 Equalization Storage

Equalization storage is required to meet water system demands in excess of delivery capabilities from pumping facilities. The volume of equalization storage needed depends on pump capacity, transmission delivery, distribution system capacity, and system demand characteristics. Equalization storage can be determined by developing a diurnal demand curve for the system that identifies the duration of system demands in excess of maximum day demand and the storage volume needed. As previously noted in Section 3, hourly pumping and tank level data are currently not available to calculate equalization storage. For planning purposes, we have estimated equalization storage as the amount of volume required to sustain a peak hour demand duration of four hours. This will provide a conservative estimate and minimize the need to operate pumping and/or treatment facilities above the maximum day demand for prolonged periods, particularly during the summer months when the Town experiences higher seasonal demands.

From Table 3-10, the peak hour demand for 2014 was estimated to be 2.13 MGD, which is equivalent to a flow rate of approximately 1,491 gpm (2.13 MGD x 700 gpm/MGD). Based on a peak hour demand period of 4 hours, a volume of approximately 357,840 gallons would be required for equalization storage *(1,491 gpm x 60 minutes/hour x 4 hours).* Applying the same criteria to the projected peak hour demand of 2.49 for 2034, a future volume of approximately 418,320 gallons would be required for equalization storage *(1,743 gpm x 60 minutes/hour x 4 hours).*

4.1.2 Fire Flow Storage

Fire flow storage is based on recommendations of the Insurance Services Office (ISO), which determines needed fire flows for insurance rating classifications of buildings within the distribution system. Results of hydrant flow tests conducted by ISO in 2008 indicated needed fire flows ranging from 500 gpm for residential areas to 7,000 gpm for commercial areas. According to DEP, municipalities are only required to provide up to a maximum fire flow of 3,500 gpm for a three-hour duration. Any additional fire flow required is typically the responsibility of the building owner and would be met by on-site fire protection systems. Therefore, based on a maximum fire flow of 3,500 gpm for a duration of three hours, a volume of approximately 630,000 gallons would be required for fire protection *(3,500 gpm x 60 minutes/hour x 3 hours).*

4.1.3 Emergency Storage

Any storage provided within a tank beyond the volumes for equalization and fire flow storage is considered emergency storage and would be available for pipeline breaks, equipment failures, and other emergency situations. The volume required for emergency storage is a function of risk with respect to an interruption of supply. DEP guidelines recommend that municipalities provide emergency storage equal to the average daily consumption if supply sources for a system are not equipped with sufficient standby power to meet this consumption during a power failure, or lack system redundancy with respect to pumps and/or pipelines.

The Town's water treatment plants are equipped with standby power along with its raw water pump stations. The Town's ability to divert the Mill Brook Well Field and the combined source of Carlson's Quarry and Flat Ledge into Cape Pond for treatment through either the RSF or DAF treatment facility also provides some redundancy. The only potential risk would be the loss of the raw water pipeline or the pump station serving Carlson's Quarry since two of the four primary sources would be lost including Carlson's Quarry and Flat Ledge. However, the new bedrock well soon to be constructed would assist to offset the loss of these two supplies due to pipeline or mechanical failure. In addition, the Town currently has two interconnections with the City of Gloucester that could supplement the Town's supplies on a short-term basis.

Considering the infrastructure and hydraulics of the existing water system, we have estimated the future emergency storage to be 50 percent of the average daily consumption. This will provide the Town with some storage to offset the possible loss of supplies, pipelines or pumping facilities over an extended period. From Table 3-7, the estimated total average day demands for 2014 and 2034 were 0.506 and 0.592, respectively. For emergency storage to be 50 percent of the average day demand, a volume of approximately 253,000 gallons is required for 2014, and a volume of approximately 296,000 gallons will be required for 2034.

4.2 ADEQUACY OF EXISTING STORAGE FACILITIES

Based on the criteria presented for determining system storage needs, we have the estimated the storage requirements to meet the current and future water need for the Town as shown in Table 4-1 below.

TABLE 4-1
ESTIMATED STORAGE REQUIREMENTS

Storage Component	Volume Required (MG)	
	2014	2034
Equalization	0.358	0.418
Fire	0.630	0.630
Emergency	0.253	0.296
Total	1.241	1.344

Under current operating conditions, the total combined storage volume for the Town's water distribution system is approximately 4.143 MG including the Pool's Hill, Pigeon Hill and South End storage tanks. However, all of this storage volume cannot be considered "usable".

According to DEP Guidelines, equalization storage should be provided to customers under domestic system demand conditions at a minimum pressure of 35 psi (81 feet of hydraulic head). Thus, only the volume of water within a tank that will provide a pressure of 35 psi to the highest service elevation can be considered "usable" equalization storage. From Table 2-3, the highest service elevation within the water distribution system is approximately 191 feet (USGS) which would require the volume allocated for equalization storage to be provided above an elevation of 272 feet (191 feet + 81 feet). Since the overflow elevation for all three tanks is 234 feet, none of the tanks' volumes can be considered available for equalization. However, in reviewing the system, there are only a few services located at the end of Pigeon Hill Street and Landmark Lane which approach this elevation. These services are not representative of a neighborhood or area that is being served, and it would be inappropriate to evaluate the adequacy of the Town's entire storage system on serving these few isolated services. Therefore, they will not be considered in this evaluation.

The next highest service elevation would be approxinately 170 feet which is located along Wildon Heights in the Great Hill Area. Based on serving this elevation, the storage volume allocated for equalization storage would need to be provided above an elevation of 251 feet (170 feet + 81 feet) which is still above the overflow elevation forall three tanks. Therefore, none of the tanks' volumes can be considered available for equalization. Given the tanks' existing dimensions, a combined operating height of approximately 4 feet would provide approximately 0.421 MG of equalization storage which would satisfy the estimated future equalization volume of 0.418 MG. This means that based on an overflow of 234 feet, the highest service elevation that can be adequately served would be 149 feet (234 feet – 4 feet – 81 feet).

With the exception of the isolated services noted above that were not considered in this evaluation, the only remaining services that are above the 149 feet threshold are located at the end of Summit Avenue near the Pool's Hill Tank, and along Hodgkins Road, Wildon Heights, Sandy Bay Terrace, Sheehan Terrace and Main Street within the Great Hill Area. For the few isolated services noted herein, it is suggested that the Town install individual booster pumps to alleviate low pressure issues. For the services within the Great Hill Area, one approach to address both low system pressures and the lack of equalization storage is to create a high pressure zone to serve this area. This new zone would require a new booster pump station, hydropneumatic tank, check valves and some piping modifications to isolate the system.

These two approaches will improve system pressures for residents in these areas, and will lower the highest service elevation within the existing system from 191 feet to approximately 149 feet, thereby eliminating the noted equalization storage deficit. A detailed discussion on the hydraulics and system impacts of creating a high pressure zone for the Great Hill Area is presented in Section 6.5.

For fire flow storage, DEP Guidelines define "usable" fire storage as the volume of water within a storage tank that will provide a pressure of 20 psi (46 feet of hydraulic head) to the highest service elevation in the system. Based on the maximum service elevation of 170 feet used above for evaluating equalization storage, the volume of water required for fire protection should be above an elevation of 216 feet (170 feet + 46 feet) within the tanks to be considered "usable". Based on the tanks' diameters and overflow elevations, a combined total "usable" volume of approximately 1,999,500 gallons is available for fire protection. Based on the required fire storage volume of 630,000 gallons, there is ample storage for fire protection. Taking into consideration the potential approach of creating a high service zone for the Great Hills Area, the top four feet of volume with the storage tanks would be allocated for equalization which would reduce the total "usable" volume available for fire protection to approximately 1,578,600 gallons. This would still exceed the required fire storage volume of 630,000 gallons.

With respect to emergency storage, the remaining volume of approximately 1,272,700 gallons below the fire storage volume within the Pool's Hill Tank alone would meet the estimated volumes of 0.253 MG for 2014 and 0.296 MG for 2034, respectively. Therefore, there is ample emergency storage. Table 4-2 presents a summary of the estimated "usable" storage volumes for the Town's three water storage tanks.

TABLE 4-2
ESTIMATED USABLE STORAGE(1)

Storage Component	Pool's Hill (MG)	Pigeon Hill (MG)	South End (MG)	Total Usable Storage (MG)
Equalization	0	0	0	0
Fire	1.727	0.135	0.137	1.999
Emergency	1.273	0.060	0.144(2)	1.477
Total	3.000	0.195	0.281	3.476

1. Based on a highest service elevation of 170 feet.
2. Based on using volume of water above elevation 195 feet only.

SECTION V – ADEQUACY OF EXISTING WATER SUPPLIES

5.1 GENERAL

Typically, water supply sources are considered adequate if the total safe yield of the supplies can meet the projected water needs of a system. Safe yield is defined in the Water Management Act as the *"maximum dependable withdrawals that can be made continuously from a water source including ground or surface water during a period of years in which the probable driest period or period of greatest water deficiency is likely to occur; provided, however, that such dependability is relative and is a function of storage and drought probability."*

As noted in Section 2, the safe yield of the Town's existing water supplies including Cape Pond, Carlson's Quarry, Flat Ledge and the Mill Brook Well Field were compiled from previous reports and MassDEP-approved WMA permits. From Table 2-1, the total combined estimated safe yield for the Town's existing water supplies is 1.02 MGD. However, this total estimated safe yield does not take into account operational issues that may limit access to the full supply volumes, such as maintaining minimum water levels over intakes for pumping, thereby reducing the available safe yield of the supply. The adequacy of the Town's water supplies to meet projected water demands should be based on the available safe yield of each supply so that an accurate assessment for future planning can be made.

5.2 AVAILABLE SAFE YIELD

From the 1997 "Water Supply Management Plan" prepared by Metcalf & Eddy, the safe yield of Cape Pond and Carlson's Quarry was based on estimated storage capacities of 108 MG and 99 MG, respectively. From Table 2-1, the useable volume of Cape Pond based on maintaining a minimum water level of 6.75 feet over the intake screen is reported to be 106 MG, which is approximately 98% of the estimated storage volume of 108 MG noted in the 1997 report. From a review of past pumping records for Cape Pond from 2006 to 2013, the average withdrawal rate was approximately 0.305 MGD with a maximum withdrawal rate of 0.409 MGD noted for July and August of 2013. As such, the total estimated safe yield of 0.310 MGD will be considered available.

For Carlson's Quarry, the useable volume as estimated from the hydrographic study completed in 2006 is 81 MG, or approximately 82% of the estimated storage volume of 99 MG noted in the 1997 report. This usable volume is based on the Town's ability to divert raw water from this supply into Cape Pond for treatment through the RSF plant if required due to excess iron and managnese levels. The only potential limitation to this supply is the ridge that separates Carlson's Quarry into two distinct bowls. According to the pumping records from 2006 through 2013, the average withdrawal rate for Carlson's Quarry was approximately 0.273 MGD with a maximum withdrawal rate of 0.317 MGD noted for July and August of 2013. It appears that the ridge does not present any limitation to the use of Carlson's Quarry since its has historically been withdrawn at or beyond its estimated safe yield. This may be a result of the fact that the Town has typically only relied on the top 20 feet of this supply which would not draw water levels below the top of the ridge. The approach of diverting this supply to Cape Pond to access the additional volume below the top 20 feet would only improve its reliabiity. As such, the total estimated safe yield of 0.220 MGD will be considered available.

For Flat Ledge, the total storage volume as determined by the hydrographic survey completed per the 2006 "Water Supply Operations Plan" prepared by SEA is 62 MG, when full based on a total depth of 100 feet . To maintain a targeted minimum water level of 30 feet above the pump intake, the total "useable" volume would be approximately 59 MG,or 95% of its total storage volume. In reviewing past pumping data from 2007 to 2013 as reported in the *"Public Water Supply Annual Statistical Reports"*, the highest monthly withdrawal rate reported for this source was approximately 0.09 MGD. However, this withdrawal rate is a function of the existing pumping system which has a limited capacity and is set 40 feet or less below the full water level. Pumping records for Flat Ledge from 2001 and 2002, when a larger pump was being used, show an average withdrawal rate of 0.240 MGD from the months of August through February, which is approximately 83% of its permitted safe yield. As previously noted, the Town is considering the construction of a new pump station at Flat Ledge that will include a larger pump and an intake set closer to the bottom of the reservoir to maximize the use of this available source. Having the ability to utilize this existing source at or near its safe yield would greatly improve the Town's supply strategy to meet high seasonal demands and drought conditions.

Dewberry recommends therefore, that this upgrade be implemented within the next year. Considering the construction of a new pump station at Flat Ledge, and the fact that the supply has been utilized close to its estimated safe yield in the past, we have estimated the available safe yield of Flat Ledge at this time to be 0.20 MGD to provide some protection from overpumping the source. Upon construction and operation of the new larger capacity pump station, the Town should revisit this safe yield estimate based on actual pump and water level data to determine if changes to the estimate are warranted.

With regard to the Mill Brook Well Field, its estimated safe yield is equal to its maximum WMA permitted withdrawal of 0.20 MGD. In reviewing raw water pump records for 2013, the two highest months of pumping for this source were reported to be in May and October with average withdrawal rates of 0.122 MGD, and 0.086 MGD, respectvely. The three gravel pack wells that make up the well field operate over a 20-foot drawdown leaving 10 feet of water over the pump intake. Although the well field can be pumped at a rate of 0.20 MGD per the WMA Permit, based on discussions with Town staff, pumping these wells at this rate for a prolonged period of time results in excessive drawdown requiring the wells to be de-activated. However, pumping records from 2002 and 2003 show that this well field was pumped at an average rate of 0.21 MGD from November 2002 to April 2003, and at an average rate of 0.184 MGD from January 2002 to April 2003. But as shown in Table 5-3 presented in Section 5.4, annual rainfall amounts recorded for 2002 and 2003 were substantial at approximately 51 inches and 53 inches, respectively, which likely contributed to the extended use of this well field. To provide a conservative estimate and to enure the reliability of these wells when needed, we have estimated the effective safe yield of this wellfield to be 0.10 MGD, which is equivalent to pumping the well field at its permitted rate over a 6-month period. Table 5-1 on the following page presents the available safe yields estimated for the Town's water supplies based on current operations that will be used to determine their adequacy in meeting projected water demands.

5.3 SUPPLY VERSUS DEMAND ASSESSMENT

Based on the definition of safe yield noted in Section 5.1, the Town's water supplies need to meet projected average day system demands during a critical dry period, which would occur during the summer months. This is particularly true for Rockport as evident from the seasonal demands that the Town experiences during the months of July and August. Therefore, the Town's supplies must be able to meet this seasonal fluctuation on a yearly basis.

TABLE 5-1
SUMMARY OF ESTIMATED AVAILABLE SAFE YIELDS

Supply	Total Surveyed Volume (MG)(1)	Total Usable Volume (MG)	Estimated Safe Yield (MGD)	Estimated Available Safe Yield (MGD)	Estimated Available Volume (MG)
Cape Pond	175	106(2)	0.31	0.31	113
Carlson's Quarry	92	81(3)	0.22	0.22	80
Flat Ledge Quarry	62	59(4)	0.29(5)	0.20(6)	73
Mill Brook Wells	n/a	n/a	0.20	0.10	37
Totals	**329**	**246**	**1.02**	**0.83**	**303**

1. These values represent the hydrographic surveyed volumes presented in the 2006 "Water Supply Operations Plan" prepared by SEA Consultants, Inc.
2. Based on pumping Cape Pond over an operating range of 8 feet.
3. Based on maintaining 18 feet over pump intake.
4. Based on maintaining 30 feet of water over new pump intake.
5. Based on the 2002 WMA Permit Application.
6. Based on constructing new pump station and intake

From Table 3-11, the seasonal day demand for the months of July and August from 2010 through 2013 were approximately 35% greater than the average day demand for that year. Applying this percentage to the 2014 and 2034 projected total average day demands presented in Table 3-7, the seasonal demands for the same years would be approximately 0.683 MGD and 0.799 MGD, respectively. Based on the available safe yield estimate of 0.830 MGD above, the Town's existing supplies are adequate to meet the projected seasonal day demands, with a surplus of approximately 0.031 MGD in 2034. Table 5-2 presents a summary of the demand versus safe yield assessment.

TABLE 5-2
ADEQUACY OF EXISTING WATER SUPPLIES

Year	Projected Average Day Demand (MGD)	Projected Seasonal Day Demand (MGD)	Estimated Available Safe Yield (MGD)	Surplus / Deficit (MGD)
2014	0.506	0.683	0.830	+0.147
2019	0.526	0.710	0.830	+0.120
2024	0.547	0.738	0.830	+0.092
2029	0.569	0.768	0.830	+0.062
2034	0.592	0.799	0.830	+0.031

Since population trends and related water consumption can vary from what is projected over a twenty year period, actual future water needs of the Town may also be different. Given the Town's recent history with respect to water shortages that have been experienced due to severe drought conditions, it would be prudent to have a reserve source of capacity to provide some redundancy. It should be noted that both Cape Pond and Carlson's Quarry can be overpumped on a temporary basis to offset the seasonal surges in demand that the Town experiences as evident from historical pumping records. In comparing the projected average day demands to the estimated available safe yield of the existing supplies, there would be a surplus of 0.324 MGD in 2014,and 0.238 MGD in 2034, respectively. Excluding the higher seasonal demands that occur within the Town, there is ample supply available to meet projected system demands.

5.3.1 New Bedrock Well Supply

As previously noted in Section 2, construction of a new 8-inch diameter bedrock well and pump station will be completed by the Summer 2015 which will have a permitted maximum withdrawal, or safe yield, of 0.29 MGD. This new source will be pumped directly into Cape Pond to supplement its capacity, similar to the diversion of Flat Ledge into Carlson's, and the Mill Brook Well Field into Cape Pond. This new source has been designed with the intent to be operated during the months from May to September when higher system demands are experienced such as the seasonal day demands. Since the well has not yet been continuously operated over a course of several months, it cannot be determined if in fact the maximum withdrawal of 0.29 MGD will be the available safe yield of this source over its operating period.

However, unlike the Mill Brook Well Field, this bedrock well is approximately 400 feet deep and will have a significant drawdown from which to operate, improving its reliability. In addition, this source will only be operated for several months to meet the higher seasonal demands. The Town should be able to pump this well at its permitted rate of 0.29 MGD over this short duration since it will have ample time to recover. To account for varying hydrogeologic conditons and the impacts of dry weather on groundwater levels, we have estimated the safe yield of this supply for the period of May through September to be 0.20 MGD, or 70 percent of its permitted withdrawal rate.

Although the intent is to operate this well during the higher seasonal demand periods to augment supply from Cape Pond, this source could be operated from April to October if needed. Based on pumping the new bedrock well at its estimated safe yield of 0.20 MGD over this extended period, its effective safe yield would be approximately 0.12 MGD. With this new source, the total estimated available safe yield of the Town's water supply will be increased to approximately 0.950 MGD. This will result in a supply surplus of 0.151 MGD in 2034 (0.950 MGD – 0.799 MGD) during the higher seasonal demand periods when needed, and a supply surplus of 0.358 MGD (0.950 MGD – 0.592 MGD) during the other months of the year. Upon construction and operation of the new bedrock well, the Town should revisit this safe yield estimate based on actual pump, water level and recovery data to determine if changes to the estimate are warranted.

5.3.2 Potential Sources of Additional Supply

In addition to this new bedrock well source, there may be unused capacity within Cape Pond that the Town could access in the future if needed. As shown in Table 5-1, the total surveyed volume for Cape Pond is 175 MG which represents the amount of water above the intake elevation. The Town currently utilizes the top 8 feet of this volume to preserve the raw water quality entering the RSF plant and to maintain a minimum water level above the intake for operational purposes. As the water level in Cape Pond approaches the lower portion of this 8-foot operating range, levels of iron and manganese increase resulting in the need to use potassium permanganate for its removal. The Town has been discussing the option of extended the existing intake to a deeper portion in the reservoir. Based on the hydrographic survey completed in 2006, there is a depression about 200 feet south of the existing intake that could potentially lower the intake by two to three feet, thereby increasing the operating range from 8 feet to 10 or 11 feet. Based on the volume tables presented in the 2006 "Water Supply Operations

Plan", this increase in operating range would provide an additional 22 MG to 30 MG of storage volume for the Town to pump. Based on the projected seasonal day demand of 0.799 MGD for 2034, this additional volume would sustain the Town for another 28 to 38 days.

Extending the intake and pumping down Cape Pond another two to three feet could impact raw water quality entering the RSF plant including increased levels of iron and manganese that may require enhancements to the on-going treatment processes including the aeration and potassium permanganate feed systems. Also, improvements to the existing vacuum priming system may be needed to allow the existing low lift pump system to operate effectively while drawing more suction.

The Town has also been investigating and planning for the potential expansion of Flat Ledge Quarry including the completion of a feasibility study, and has obtained the necessary permit for constructing the expansion which will require the construction of a concrete dam across the quarry opening. This permit is valid until June 2018. This new expansion will result in capturing a larger portion of runoff from Carlson's Quarry, which now flows out to the ocean. The pumping of water from Flat Ledge back up to Carlson's Quarry does recover some of this runoff, but this can occur when Carlson's Quarry is low enough.

Access to Flat Ledge's supply is dependent on Carlson's Quarry since it is pumped directly into Carlson's Quarry. One option that would allow the Town to utilize Flat Ledge to its fullest capacity including the runoff from Carlson's Quarry would be to operate it independently as an individual source. This would require the installation of a new raw water transmission main to connect Flat Ledge into the existing raw water main utilized by Carlson's Quarry. Having Flat Ledge as an independent source would provide flexibility in managing these two supplies and improve future system redundancy. With this new connection, the Town would be able to pump from either Flat Ledge or Carlson's Quarry individually or simultaneously. Even if the new dam was to be constructed without this new transmission main, supply from Flat Ledge will still be dependent on the status of Carlson's Quarry.

5.4 SUPPLY IMPACTS FROM POTENTIAL DROUGHT CONDITIONS

As noted in Section 5.3, the Town's current water supplies can meet the projected system demands up to the year 2034 under normal operating conditions. With the activation of the new bedrock well, the Town will have even more ability to meet normal system demands. However, the Town's supplies should also be evaluated based on their ability to sustain water needs during a drought condition similar to the one that was experienced in 2001 and 2002. From review of past pumping and reservoir level records, in April 2001, Cape Pond and Carlson's Quarry were 100% full, which is normal for these supplies at this time of year. Historically, these supplies have been at or near full capacity almost every April. By December 2001, Cape Pond and Carlson's Quarry were approximately 26% full and 53% full, respectively, which is far below normal for these supplies. Historically, in December, both these supplies are somewhere in the range of 73% to 75% full. As such, the Town had to pump Flat Ledge at a substantial rate to keep Carlson Quarry as full as possible. From July 2001 to March 2002, the Town pumped approximately 57 MG of supply from Flat Ledge, which equates to an average pump rate of 0.210 MGD. To access this volume, the Town had to move a portable pump down into Flat Ledge on a continuous basis as the water level within Flat ledge was pumped lower and lower. This was because Flat Ledge is not equipped with a suitable pump station and intake.

In reviewing rainfall and system demand data as reported from 1997 through 2003, similar conditions that contributed to the 2001-2002 drought also occurred in 1997 and 1999. As shown in Table 5-3 on the following page, the annual rainfall amount in 1999 was almost as low as in 2001 with a slightly higher demand. Reservoir level records from 1999 indicate that Cape Pond was at its lowest level of 2.2 feet in November, and Carlson's Quarry was at its lowest level of 71.1 feet in September. To supplement the low levels with Carlson's Quarry, the Town pumped approximately 24 MG out of Flat Ledge from September 1999 to March 2000, and enacted Phase II water restrictions to reduce water usage within the Town. The Town also requested in November 1999 that MassDEP declare a State of Emergency to allow the Town to utilize available supply from Steel Derrick, a privately owned quarry, to supplement Carlson's Quarry. Although the Town never used Steel Derrick, water levels within the two reservoirs improved in the next coming months, and sufficiently recovered by May 2000 with Cape Pond at 89% capacity and Carlson's Quarry at 100% capacity.

TABLE 5-3
SUMMARY OF ANNUAL RAINFALL VERSUS SYSTEM DEMANDS (1997 TO 2003)

Year	Annual Precipitation (inches)	Annual System Demand (MGD)	Annual System Demand (MG)
1997	42	0.71	259
1998	57	0.71	259
1999	45	0.71	259
2000	57	0.69	252
2001	43	0.67	245
2002	51	0.65	237
2003	53	0.65	237

As shown in Table 5-3, rainfall amounts in 2000 were 12 inches higher than in 1999, which likely contributed to the recovery of the supplies. In comparison, the rainfall amounts in 2002 were only 8 inches higher than in 2001. This might be one reason why levels within the two reservoirs, particular Cape Pond, did not recover as quickly as they did in 2000.

During the two drought events of 1999 and 2001, the Town's primary supplies included only Cape Pond and Carlson's Quarry. Flat Ledge at that time was not yet permitted and was not being used as part of the Town's normal daily operations to meet system demands. In addition, the Mill Brook Well Field was not available due to its need for repairs. In fact, the 1999 drought prompted the Town to move ahead with the upgrades to the Mill Brook Well Field, which included replacing the existing tubular well system with three new gravel packed shallow wells. The new wells were completed and placed into service in January 2002. Based on this supply scenario, it is not surprising that the Town had difficulty dealing with the 2001 drought event. As noted in Table 5-1, the combined available safe yield of Cape Pond and Carlson's Quarry is 0.53 MGD. From Table 5-3, in 2001, the Town's average system demand was 0.67 MGD, which is above the safe yield of these two primary supplies. The Town's seasonal summer demands were probably closer to 1 MGD or more, almost double the safe yield of these two supplies, which placed a significant stress on these two supplies, and required the Town to pump Flat Ledge as much as it did to keep up with demands.

Even with pumping Flat Ledge, the Town could not restore the levels within Cape Pond or Carlson's Quarry. From the 2001 supply records, the Town began pumping Flat Ledge in July. However, levels within Cape Pond and Carlson's Quarry continued to drop. In December 2001, Cape Pond was at its lowest operating level of 1.6 feet and Carlson's Quarry was at an operating level of 73.9 feet. Dewberry believes this was a result of the fact that the system demands still exceeded the safe yield of the available supplies. From Table 5-1, the estimated available safe yield of Flat Ledge is 0.20 MGD. Adding this supply to the combined safe yield of 0.53 MGD as provided by Cape Pond and Carlson's Quarry equals a total available safe yield of 0.73 MGD, which is still below the seasonal summer demand that occurred in 2001.

This is evident by the fact that the levels within Cape Pond and Carlson's Quarry did not begin to recover until the Town activated the Mill Brook wells in January 2002. From reservoir level records, the Town pumped approximately 83 MG from the newly upgraded Mill Brook wells from January 2002 up to April 2003, with 50 MG diverted to Cape Pond and 33 MG pumped to the DAF treatment plant. In March 2002, the Town was eventually able to stop pumping Flat Ledge to allow it to recover also. Although some volume of water was siphoned from Steel Derrick to Carlson's Quarry in May 2002, the recovery of the Town's two major reservoirs was due primarily to the availability of the Mill Brook wells along with the additional rainfall that occurred.

Based on the supply volume of 83 MG pumped over the 15-month duration from the Mill Brook wells, the Town increased its safe yield of 0.73 MGD by approximately 0.184 MGD. With the Mill Brook wells in operation, the Town's safe yield was approximately 0.914 MGD, which was sufficient to meet system demands and restore the levels of Cape Pond and Carlson's Quarry. The stress placed on the Town's water supplies during the 2001-2002 drought event was primarily because the Mill Brook wells were not available to contribute supply to the system when needed. As previously noted, water supplies need to meet system demands during a critical dry period, which is defined as the "safe yield". If system demands exceed the safe yield of its available supplies for an extended dry period, such as the one that occurred in 2001/2002, it is reasonable to expect that supply deficits similar to what the Town experienced will result.

This is more evident by comparing annual rainfall amounts, system demands and available safe yield of the Town's supplies over the last several years. Table 5-4 includes annual rainfall amounts, system demands and safe yields from 2007 to 2013. During these years, the Town's available supplies included Cape Pond, Carlson's Quarry, Flat Ledge and the Mill Brook Well Field.

As shown in Table 5-4, 2012 and 2013 exhibited low rainfall amounts similar to that experienced in 1997 and 2001. However, according to reservoir level records, in 2012, both Cape Pond and Carlson's Quarry were at 100% full capacity by May. In 2013, both Cape Pond and Carlson's Quarry were at 100% full capacity by June, even with Cape Pond being at its lowest operating level of 4.6 feet in December, and Carlson's Quarry being at its lowest operating level of 78.1 feet in December. In addition, both of these supplies were at 100% full capacity by April 2014 further proving that the lack of rainfall that occurred in 2012 and 2013 had minimal impact on their ability to meet system demands. This is because as shown in Table 5-4, the safe yield of the supplies during these periods of low rainfall exceeded the system demands, which was not the case during the 2001/2002 drought event. This is the basis for ensuring that the safe yield of the Town's supply system meets or exceeds current and future system demands.

TABLE 5-4
SUMMARY OF ANNUAL RAINFALL, SYSTEM DEMANDS & SAFE YIELDS (2007 TO 2013)

Year	Annual Precipitation (inches)	Annual System Demand (MGD)	Available Safe Yield (MGD)
2007	47	0.59	0.83
2008	56	0.54	0.83
2009	55	0.51	0.83
2010	52	0.49	0.83
2011	57	0.50	0.83
2012	43	0.52	0.83
2013	41	0.57	0.83

Since the 2001/2002 drought, the Town has improved the ability of its water system to handle a future drought condition by:

- Upgrading the Mill Brook Well Field in January 2002
- Adding Flat Ledge as a permitted supply to its Water Management Act (2002)
- Implementing water conservation and leak detection measures that have considerably reduced annual water consumption and unaccounted-for water

Even with the above improvements, the Town would be vulnerable to an extended drought condition if it lost any of its current sources due to maintenance or mechanical failure, similar to what occurred in 2001-2002 with the Mill Brook Wells. The Town has since reduced this potential vulnerability with the addition of the new permitted bedrock well supply, which is currently under construction and should be ready to be placed into service by Summer 2015. This new permitted source will supplement the volumes within Cape Pond to reduce the impact of higher summer seasonal demands, and improve the Town's ability to mitigate through a drought event. To further improve its supply reliability, the Town is in the process of modifying the existing pumping system at Flat Ledge with a larger size pump that will hang off a floating structure. This is an interim step to allow the Town to access more of Flat Ledge's available supply volume until the new permanent pump station and intake is constructed.

5.4.1 Revised Water Supply and Operational Strategies

Figure 5-1 presents an overall schematic of the Town's water supply system including its three primary surface supplies, the Mill Brook wells and the soon to be constructed bedrock well along with individual capacities and operating parameters. From Table 5-1, the total available supply volume from the primary surface water supplies of Cape Pond, Carlson's Quarry and Flat Ledge is approximately 266 MG, which based on the 2014 average day demand of 0.506 MGD presented in Table 5-2, would provide approximately 525 days of supply. For the 2034 average day demand 0.592 MGD presented in Table 5-2, approximately 450 days of supply would be provided.

For the Mill Brook wells, operating this source at its effective safe yield of 0.10 MGD estimated in Section 5.2 would result in approximately 37 MG of additional supply volume to meet system demands. For the new bedrock well, operating this source at its effective safe yield of 0.12 MGD would provide approximately 44 MG of additional supply volume to meet system demands. Combined, the two well sources would contribute approximately 81 MG of supply to supplement the Town's surface water supplies. In terms of operational impact, if both the new bedrock well and the Mill Brook wells were pumped at the noted rates from May 2001 until October 2002 during the 2001-2002 drought, Cape Pond would have been approximately 4.5 feet higher in December 2001. Based on the recorded operating level of 1.6 feet, Cape Pond would have been at an operating level of about 6.25 feet, which is above the level that Cape Pond is normally at that time of year. Table 5-5 below summarizes the estimated available safe yield of the Town's revised water supply including the new bedrock well.

TABLE 5-5
REVISED WATER SUPPLY SAFE YIELD ESTIMATES

Supply	Estimated Available Safe Yield (MGD)	Estimated Available Volume (MG)	Estimated Days of Supply (2014)[1]	Estimated Days of Supply (2034)[2]
Cape Pond	0.31	113	223	191
Carlson's Quarry	0.22	80	158	135
Flat Ledge Quarry	0.20	73	144	123
Mill Brook Wells	0.10	37	73	63
Bedrock Well	0.12	44	87	74
Totals	**0.95**	**347**	**685**	**586**

1. Based on estimated annual supply need of 185 MG (0.506 MGD x 365 days)
2. Based on estimated annual supply need of 216 MG (0.592 MGD x 365 days)

From the 2006 "Water Supply Operations Plan", the recommended operational strategy for optimizing the Town's supplies while achieving maximim recovery was based on meeting target combined volumes of Cape Pond and Carlson's Quarry as determined from weekly level monitoring along with operating the Mill Brook wells on a daily basis from June to October. This is a reasonable approach given that the higher summer seasonal demands that occur within Rockport place the most stress on its supplies, and utilizing the Mill Brook wells during this time would help to preserve as much volume as possible within the two reservoirs for a potential drought condition. This is the same concept that served as the basis for the new bedrock well, which will be pumped directly into Cape Pond from May to September to minimize the impact of the summer seasonal demands on Cape Pond. As such, we recommend that the Town continue following the Operational Plan currently in place with the inclusion of the new bedrock well being operated on a daily basis from May to September.

Given the fact that the new bedrock well will only be available for operation from April to October, we recommend that this source be operated at the maximum extent possible with the Mill Brook wells used on a less consistent basis or when needed to meet the target combined volumes established in the Operations Plan. This approach will improve the reliability of the Mill Brook wells if needed to mitigate possible shortages in volume during the remainder of the year by preserving their available supply.

In discussing the current operational strategies with Town staff, it is our understanding that the Mill Brook wells, when placed in service, are operated 24 hours a day over a period of several weeks at a time. This is primarily due to the fact that there is no automation for the Mill Brook Wells to be controlled remotely from the water treatment plants or locally off a timer. These wells are required to be manually turned on and turned off. Although the rate at which each well is pumped is moderate (approximately 40 gpm), operating a gravel pack well 24/7 for an extended period of time is not advisable and can lead to shorter service life of pumps and associated equipment along with increasing the maintenance needs of the well itself. Typically, gravel pack wells should be rehabilitated every 5 to 7 years depending on water quality and soil characteristics of the surrounding formation. Overpumping a well can prolong its recovery time and potentially draw fines and other particles into the well from the outer formation. These fines can plug up the gravel pack that surrounds the well screen, thereby reducing the hydraulic capacity of the well and requiring more frequent maintenace, every 2 to 4 years.

With the addition of the new bedrock well, operating the Mill Brook wells on a 24/7 basis to keep Cape Pond levels adequate should not be needed. Dewberry would recommend operating the new bedrock well and the Mill Brook wells on a 8 to 12 hour per day basis as an initial strategy. In addition, we suggest that some form of automation, preferably through the existing SCADA system, be provided for these wells to improve their operation. This will prevent overpumping either well and preserve their reliability when needed to meet higher system demands. If conditions are such that additional supply is needed to replenish levels within Cape Pond, then the Town could pump these well sources more as a temporary measure.

As previously noted in Section 2, the Town has the ability to divert supply from Carlson's Quarry into Cape Pond for treatment through the RSF plant. This is an added strategy that could prove useful during times of high demands or extended dry periods. Under normal operations, Carlson's Quarry is pumped to the DAF plant for treatment. As such, the Town has historically relied on the top twenty feet of Carlson's Quarry due to the fact that pumping below this level results in elevated levels of iron and manganese within the raw water, which is difficult to treat through the DAF plant. This twenty-foot operating range is included as part of the Operations Plan due to this water quality issue.

Since the RSF plant can treat higher levels of iron and managnese than the DAF Plant, the approach of diverting Carslon's Quarry to Cape Pond when surface water levels in Carslon's Quarry goes below its normal 20-foot operating range would allow the Town to access the additional volume below the top 20 feet if necessary to optimize the operation of its current supplies. Based on maintaining a minimum water level of 18 feet over the existing Carlson's Quarry intake, approximately 38 MG of additional supply volume could be pumped into Cape Pond, and treated through the RSF plant.

5.4.2 Emergency Interconnections with Gloucester

In addition to the new bedrock well supply, the Town also has the ability to obtain water on an emergency basis from two interconnections with the City of Gloucester. These are shown on Plate No. 1 included in Appendix C. These interconnections could provide a source of finished water supply and backup to the Town's water supplies if needed to sustain a drought condition, assuming that the City has surplus supply to provide the Town. One operational concern in utilizing these interconnections is that the City of Gloucester recently switched over to chloramines for providing disinfection of its distribution system. Chloramines are formed by adding chlorine and ammonia to the finished water within a ratio of 3.5 to 5 parts free chlorine per 1 part ammonia. Rockport uses sodium hypochlorite for disinfection which produce a free chlorine residual. When mixing chloraminated water with chlorinated water, there is a potential for disrupting the chloramine ratio which could result in excess ammonia being introduced into the distribution system. This can cause water quality issues, or a depletion of free chlorine residual.

However, for water distribution systems that are adequately maintained and provide a high quality of finished water as is the case in Rockport, the introduction of Gloucester's chloraminated water into Rockport's chlorinated water should not pose a problem. There may be a need to slightly increase chlorine dosages at the plants to maintan sufficient free chlorine residual in the system. An important parameter to consider in blending chloraminated water from Gloucester with chlorinated water from Rockport is the finished water pH. Changes in pH through blending two sources of water within a system can temporarily affect corrosion control measures, as well as reducing the effectiveness of chlorine residuals.

It is our understanding that the Town of Rockport will be meeting with the City of Gloucester in the next coming weeks to discuss the use of these interconnections based on improvements that the City of Gloucester is currently undertaking in their system. The goal will be to establish an intermunicipal agreement (IMA) between the two municipalities that would establish the terms and procedures for transferring water supply between the two adjoining systems in the event of an emergency. Specific limitations and issues to provide additional supply through these existing interconnections with respect to water quality, system infrastructure and hydraulic compatibility should be also be identified and addressed as part of the IMA.

5.4.3 Other Sources of Emergency Supply

Another potential source of emergency supply that could be available to supplement the Town's current supplies during an extended drought condition are the several privately-owned quarries that exist within the Town including Steel Derrick Quarry and Johnson's Quarry. Steel Derrick is already designated as an emergency source of supply and is listed in the Town's Annual Statistical Report as such. During the 2001/2002 drought, the Town obtained permission from the property owners and MassDEP to use Steel Derrick as an emergency source to supplement its significantly depleted surface water supplies at that time. However, levels within the Town's existing surface supplies began to improve when the newly renovated Mill Brook wells were placed on-line, so Steel Derrick was never activated as an emergency source. Steel Derrick is located at the end of Rowe Avenue, approximately 1,800 feet northwest of Carlson's Quarry. Overflow from Steel Derrick naturally discharges overland into Carlson's Quarry. There is an existing 12-inch siphon line that also allows water from Steel Derrick to be directly discharged into Carlson's Quarry. In order to initiate the siphon discharge into Carslon's Quarry, the DPW's portable pump is used to prime the siphon line. As such, this source remains available as an emergency supply with the approval of the property owners and MassDEP using the existing infrastructure and portable pump.

Johnson's Quarry has been considered in the past as a potential source of supply but has never been fully evaluated in terms of its caapcity and feasibility. It is located at the end of Johnson Road approximately 1,700 feet southwest of Steel Derrick, and about 2,400 feet west of Carlson's Quarry. Given its location, a pratical approach to using this source as a backup supply would be to feed it directly into Carlson's Quarry via an above-grade line, similar to how Flat Ledge is utilzied. Although Johnson's Quarry is at a higher elevation than Carslon's Quarry, a new pump station would likely be needed to fully access its storage volume along with a new raw water pipeline installed overland.

5.5 CONCLUSION

Based on the water supply assessment completed herein, Dewberry has determined that the Town's existing water supplies can meet the projected water demands within the twenty year planning period. Additionally, the activation of the new bedrock well will provide a permitted supplemental water supply as needed to meet the increased seasonal day demands that occur during the summer months. As noted herein, the available safe yield estimated for the Town's water supplies was contingent on improving the intake and pumping system at Flat Ledge. It is recommended that the Town proceed with the planning and funding for the design and construction of a new larger capacity raw water pump station and intake to maximize the use of this source in its current condition. This project would likely include a new brick and block or pre-cast concrete modular structure to house the electrical, HVAC and pump control systems along with a 90-foot deep intake pipe installed along the side of the quarry, terminating with a screen, very similar to the current system used at Carlson's Quarry. Approximately 1,000 feet of new discharge pipe will also need to be installed to pump raw water from Flat Ledge up to the Carlson's Quarry Dam. As previously noted, the Town will be modifying the existing pumping system at Flat Ledge as an interim measure to access additional supply from Flat ledge until the recommended pump station and intake can be constructed.

Dewberry also recommends that the Town consider evaluating alternatives for providing a new raw water transmission main to allow Flat Ledge to be accessed independently of Carlson's Quarry. Unlike the new pump station and intake, the need for this new raw water transmission main is not immediate based on meeting projected water demands. The Town should consider this recommendation as a long-term improvement to further optimize the use of this supply and take full advantage of the increased storage capacity from the new pump station and intake.

As with the raw water transmission main, the planned expansion of Flat Ledge by constructing a new dam is not necessary to meet projected water demands within the 20-year planning period given the installation of the recommended pump station and intake. Any additional supply capacity from the Flat Ledge expansion would certainly improve the Town's ability to manage its supplies to meet these demands, however it doesn't appear necessary at this time. Dewberry understands that the Flat Ledge expansion will be a significant undertaking and will require a serious commitment of funds to complete. As such, we recommend that the Town consider delaying the expansion of Flat Ledge until 2020 or beyond, and re-assess the need for the additional supply compared to the actual system demands at that time. Based on the current permit for the expansion being valid until June 2018, the Town will need to request an extension of this permit to cover this timeframe along with the expected construction schedule.

To improve the supply redundancy and optimization of Cape Pond, we recommended that the Town evaluate the existing aeration and intake systems to determine potential upgrades that would allow more volume of water to be accessible such as lowering the intake. As previously noted in Section 5.3.1, there has been some discussion of extending the existing intake about 200 feet to a deeper portion within the reservoir, which may allow lowering the intake by two to three feet. This could potentially increase the useable storage volume available to the Town by an additional 22 MG to 30 MG. However, extending the intake to pump down Cape Pond another two to three feet could potentially result in increased levels of iron and manganese within the raw water that could require enhancements to the aeration and potassium permanganate feed systems currently in use. Also, extending and lowering the existing intake will have hydraulic impacts to the low lift pump station which will need to be evaluated. For example, the low lift pump station includes a vacuum priming system which is needed to operate the pumps due to a lack of a flooded suction. Lowering the operating levels of Cape Pond along with adding 200 feet of pipe will increase the suction head, requiring more assistance from the vacuum priming system to operate the pumps effectively.

As discussed in Section 5.4, the inclusion of the new bedrock well into the Town's Operations Plan will improve the redundancy of the Town's water supply system to meet high seasonal demands and be less vulnerable to future drought conditions. In addition, the emergency interconnections in place with the City of Gloucester can supplement the Town's supplies if needed during emergency drought conditions. While the analysis completed for this study shows that the Town should have sufficent redundancy to sustain an extended drought condition, this is predicated on all of its current sources of supply being available.

The available supply from the interconnections with Gloucester during an extended drought condition has yet to be proven. It is reasonable to assume that an extended drought condition would also have an impact on Gloucester's surface water supply volumes, which could limit the amount of supply that Gloucester would allocate for Rockport during such an event. There is also a potential that one of the Town's supplies could become temporarily unavailable during an extended drought condition due to mechanical failure or overpumping of one of the well supplies. This would place a significant stress on the remaining supplies, similar to the 2001/2002 drought when the Mill Brook wells were out of service. The approach of diverting Carlson's Quarry to Cape Pond to access the additional 38 MG of supply that exists below the normal 20-foot operating range could be utilized to address a loss of supply.

As noted in Section 5.4.3, two potential emergency sources of supply to improve system redundancy include Steel Derrick and Johnson's Quarry. Steel Derrick could provide additional capacity to supplement the Town's current supply by siphoning water to Carlson's Quarry, with minimal development and infrastructure requirements. Johnson's Quarry would require the construction of a pump station and pipeline, but could prove to be a feasible option based on its available capacity. As a long-term strategy, Dewberry would recommend that the Town take the necessary steps to develop these sources as future emergency/backup supplies. These sources would provide an added level of protection against an extended drought condition and reduce the Town's vulnerability due to a sudden loss of one of its supplies. A study should be completed to estimate the available storage capacities from these two sources, identify the steps required to develop these sources including water withdrawal agreements with property owners and/or acquisition of property, MassDEP coordination and permitting, watershed protection, and infrastructure upgrades with estimated costs.

SECTION VI – ADEQUACY OF THE EXISTING DISTRIBUTION SYSTEM

6.1 GENERAL

To properly serve its residents, a water distribution system must be able to meet demands during periods of peak consumption and provide adequate fire protection. Both peak demand conditions and fire flows are typically met from system storage reserves and not by relying on supply pumping. The Town's storage facilities have been assessed in the previous section with respect to storage volume requirements and "usable" storage available. This section of the report will evaluate the capability of the distribution system to deliver fire flows at adequate pressures for fire protection during maximum day demands. This demand condition is considered to be the maximum amount of water that a distribution system must realistically supply and has been established as a prime criterion for the evaluation of water distribution systems.

A hydraulic analysis was conducted utilizing the newly created computerized system model completed by Dewberry, and available hydrant flow test results to evaluate the distribution system's ability to meet the above demand condition.

6.2 RECOMMENDED FIRE FLOWS

The Insurance Services Office (ISO) has established certain standards by which the adequacy of a public water system to provide fire protection can be rated. Fire flow requirements for a community are typically established for each type of development within a community. The required flow rate and duration of flow for each type of development are based on structural conditions, type of occupancy, and the congestion of buildings in the area under consideration. The largest fire flow demands generally occur in the major business and industrial districts of a community. The ISO standards are used to set fire insurance rates within a community. In the planning and design of a waterworks system, it is considered good practice to adhere to these standards, not only to minimize fire insurance rates, but to reduce the risk of human casualties and loss of property resulting from fires.

Based on ISO guidelines, recommended fire flows are defined as the required flow rate from a point in the system while maintaining a minimum pressure of 20 psi at all points in the distribution system. For one and two family residential areas, the ISO required fire flows range from 500 gpm to 1,500 gpm, depending on the spacing of the houses. For high-density residential, commercial, institutional, and industrial areas, the maximum required fire flow to be sustained by the distribution system is 3,500 gpm. Individual site requirements greater than 3,500 gpm, which might exceed the capacity of the distribution system, should be met through fire protection systems provided by the property owner.

The ISO has also established time duration requirements for which fire flows should be maintained to assess the adequacy of system storage. In general, required fire flows up to 2,500 gpm should be maintained for two (2) hours, while required fire flows up to the maximum 3,500 gpm should be maintianed for three hours. These ISO fire flow duration requirements were used in the previous section to evaluate the adequacy of the Town's storage systems to meet current and future storage needs.

6.3 HYDRANT FLOW TESTS

The ISO conducted hydrant flow tests in the Town of Rockport to determine the water system's ability to provide adequate fire protection in April 2008. Table 6-1 presents a summary of the results for the locations tested. The ISO hydrant flow data results are included in Appendix A of this report. As shown in Table 6-1, only one test site, hydrant flow test #9, was not capable of meeting the required ISO flow for adequate fire protection. Dewberry conducted additional hydrant flow testing on May 5, 2013 to evaluate known problem areas and to collect data for the purpose of analyzing, adjusting and calibrating the newly created model. A copy of these flow test results are also included in Appendix A. The locations of the ISO and Dewberry hydrant flow tests are shown on Plate No. 2 included in Appendix C.

TABLE 6-1
SUMMARY OF ISO HYDRANT FLOW TEST RESULTS

Test No.	Street/Location	District Type	Static Pressure (psi)	Residual Pressure (psi)	Observed Flow (gpm)	ISO Required Flow @ 20 PSI (gpm)	Available Flow @ 20 PSI (gpm)
1	Main St. @ Blue Gate Ln	Comm.	51	47	2,210	4,000	6,700
2	Railroad Ave. @ King Drive	Comm.	70	69	2,160	3,000	12,000
3	Granite St. @ Phillips Ave.	Comm.	82	79	1,330	2,250	6,800
4	Jerdens Ln @ High School	Comm.	63	60	1,260	3,000	5,300
5	Broadway @ Mt. Pleasant St.	Comm.	94	75	2,390	3,500	5,000
6	Phillips Ave. @ Cove Ave.	Comm.	80	70	1,210	1,500	3,200
7	Woodbury Ln @ Granite St.	Res.	88	42	1,690	1,000	2,100
8	Granite St. @ Beeah St.	Comm.	87	85	1,860	1,500	12,000
9	**Eden Rd. @ Penzence Rd.**	**Comm.**	**75**	**50**	**1,060**	**2,500**	**1,600(1)**
10	Rte 127 @ Frank St.	Res.	92	51	1,140	750	1,500
11	2nd Hydrant @ Country Club	Comm.	64	61	810	2,250	3,500

1) Available flow as estimated from flow test data does not meet the required ISO fire flow.

6.4 COMPUTER MODELING

To conduct a hydraulic analysis of the Town's water system, Dewberry first created a new WaterCAD-based computerized system model utilizing available Town GIS data as the basis of the model. Various demand and operational scenarios were then simulated within the model to evaluate the existing water system and develop recommendations to alleviate deficiencies with respect to meeting current and future water needs.

6.4.1 Input Data

To develop the computer model, a GIS base plan of the Town that included water main, storage tank, roadway, and parcel layers was first imported into the WaterCAD software to create the system nodal map, or graphically portion of the model. The nodal map is a system of numbered pipe segments and junction nodes that represent the physical pipe network and connections of the water system as well as other major system components including storage tanks, water supplies and pumping stations. The water system nodal map of the Town's existing distribution system used for the anaylses completed is included as Plate No. 1 in Appendix C of this report.

Junction nodes were added at each street intersection, at each change in diameter along a pipeline, and at other appropriate modeling locations. The lengths of pipe segments between junction nodes were automatically calculated by the software during the layout of the system. Additional data necessary for modeling the water system was collected, tabulated, and manually inputted into the computer model including the following:

- Pipeline diameters and material
- Hazen Williams pipe roughness coefficients (C-values)
- Water demands and elevations for each junction node
- Dimensions and overflow elevations for storage tanks and supply reservoirs
- Water supply pump curves

Various sources were utilized to obtain the noted information for the computer model. Pipe diameters were obtained from the Town's GIS base plan and available tie card records. Elevations for the junction nodes were obtained from the Town's GIS base plan and from reviewing respective USGS maps. There was limited information available on the material composition and age of pipes within the Town's system to estimate their hydraulic capacity or roughness coefficient. However, it is our undertsanding that a majority of the older cast-iron unlined water mains have been replaced over the years with cement-lined ductile iron pipe.

The pipe roughness coefficient (C-value) for water main refers to the carrying capacity of the pipe which varies with a pipe's age and material of construction, as well as its diameter. For example, newer cement-lined ductile iron pipe has a smooth interior surface and tends to retain its original capacity for many years where as older unlined cast-iron pipe is prone to corrosion and the formation of turberculation over time. As a result, their carrying capacity tends to gradually diminish over time. In addition, larger diameter mains tend to have higher carrying capacities than smaller diameter mains since larger mains typically transmit greater volumes of flow and are well looped which tends to prevent the buildup and accumulation of sediment and turberculation within the pipe's interior. A C-value of 120-125 is commonly used for newly installed pipe, while C-values for older unlined cast-iron pipe can be as low as 40. In general, pipes with C-values of 60 have only one-half of their original carrying capacity.

Based on our experience with distribution systems and allowing for some loss in carrying capacity with age, cement-lined ductile-iron pipes were initially assigned a C-value between 110 and 120, depending on the pipe diameter. Unlined cast-iron pipe was initially assigned C-values ranging from 80 to 100, depending on pipe diameter and available information on the physical condition of the pipe. During the analysis, C-values were adjusted to reflect the results of the hydrant flow tests conducted in order to balance and calibrate the model.

Water demands were estimated from the Town's 2013 individual meter records which were provided by the Town as a GIS layer. Large commercial, agriculturual and industrial users were first identified and their individual usages as determined from Town meter records were applied to the nearest node that best represented their actual location within the system. A list of these large users along with their representative nodes are shown in Table 6-2. The remaining metered usages were then evenly distributed over the nodes throughout the system.

TABLE 6-2
2013 LARGE WATER USERS

User	Address	Ave. Day Usage (GPD)	Ave. Day Usage (GPM)	Junction Node
Harborlight Community Apts	13 Curtis Street	2,517	1.75	J-288A
Emerson Inn By the Sea	1 Cathedral Avenue	2,880	1.96	J-322
Bayridge Condominiums	1 Bayridge lane	3,370	2.34	J-275, 274
Millbrook Park Elderly Housing	Pooles Lane	3,946	2.74	J-237
Whistlestop Mall	Railroad Avenue	2,040	1.42	J-384
Motel (Local Yokel LLC)	183 Main Street	4,713	3.27	J-21
Rockport Housing Authority	Kitefield Rd	3,768	2.62	J-61, 62, 63
Wastewater Treatment Plant	46 Pleasant Street	9,794	6.80	J-71
Water Treatment Plant	1-4 DPW Way	29,796	20.7	TJ-104
High School/Elementary	24-34 Jerdens Lane	2,357	1.64	J-86
Nursing Home (Ventas Realty)	44 South Street	8,483	5.89	J-124A
Sandy Bay Estates Apts.	Sandy BayTerrace	8,112	5.63	J-7,6

6.4.2 System Model Calibration

As previously mentioned, hydrant flow tests were conducted at various locations throughout the distribution system by ISO and Dewberry staff. The results from the hydrant flow tests including static pressures, residual pressures, and flow were simulated in the computer model. Adjustments to the computerized hydraulic model were made until the results of the model simulations compared favorably to the hydraulic data collected during the field tests. At this point, the hydraulic model was considered to be calibrated and adequately representative of the physical operating characteristics of the existing distribution system. The results of the hydrant flow tests simulated in the model as compared to the actual field data recorded are indicated on Plate No. 2 attached in Appendix C. The following Table 6-3 summarizes the results of the model calibration.

As shown in Table 6-3, there are several highlighted flow tests which required the closing of either existing pipe segments, or increasing the existing pipe size, to simulate the field test results in the hydraulic model. This indicates that there may be closed or partially closed valves that are restricting flow into these system areas, or inaccurate record data with respect to pipe size. The individual pipe segments that were closed or increased in size within the model for the respective hydrant flow tests during the calibration process are indicated on Plate No. 2 included in Appendix C. A copy of Plate No. 2 has been given to the Town Staff for their use. It is recommended that the Town investigate the mains within the noted areas to determine if any improper valve closures exist, and what other possible reasons may account for the noted restrictions. With the exception of the highlighted flow tests, the remaining flow tests were successfully simulated within the model as initially constructed.

TABLE 6-3
RESULTS OF SYSTEM MODEL CALIBRATION

Test No.	Street/Location	Junction Node	Available Flow @ 20 PSI (gpm)	Modeled Flow @ 20 PSI (gpm)	% of Tested Flow
ISO Tests Conducted 04/09/08					
FF1	Main St. @ Blue Gate Ln	J-20	6,700	7,800	116
FF2	Railroad Ave. @ King Drive	J-234	12,000	14,000	116
FF3	Granite St. @ Phillips Ave.	J-306	**6,800(1)**	2,973	44
FF4	Jerdens Ln @ High School	J-85	5,300	5,100	96
FF5	Broadway @ Mt. Pleasant St.	J-53	**5,000(2)**	5,060	101
FF6	Phillips Ave. @ Cove Ave.	J-340	**3,200(5)**	2,000	63
FF7	Woodbury Ln @ Granite St.	J-371	2,100	2,094	99
FF8	Granite St. @ Beeah St.	J-236	12,000	15,100	125
FF9	Eden Rd. @ Penzence Rd.	J-165	1,600	**1,715(3)**	107
FF10	Rte 127 @ Frank St.	J-197	1,500	1,537	102
FF11	2nd Hydrant @ Country Club	J-192	3,500	**1,825(4)**	24
Dewberry Tests Conducted 05/05/14					
FF1	243 Granite Street	J-306	3,678	3,200	87
FF2	1C Pigeon Hill Court	J-299	1,749	1,750	0
FF3	Country Club Road (2nd hydrant)	J-192	1,817	1,825	0
FF4	Thatcher Road @ South Street	J-162	2,639	**2,871(6)**	108
FF5	25 Hodgkins Road	J-13	545	**575(7)**	105

(1) Model suggests that data for this flow test is invalid. Test replaced by Dewberry flow test FF1.
(2) To simulate field results, pipes within Broadway, High St. & Mt. Pleasant St. were closed.
(3) To simulate field results, pipe within Penzence Rd was closed.
(4) Model suggests that data for this flow test was invalid. Test replaced by Dewberry flow test FF3.
(5) Model suggest that data for this flow test is invalid.
(6) To simulate field results, pipe within Thatcher Road was closed.
(7) Finished water pumps were off during test.

6.4.3 Hydraulic Analysis

After the calibration of the computerized hydraulic model, associated data to reflect the estimated water demands through the year 2034 as presented in Tables 3-7, 3-9, and 3-10, was inputted into the model. The computer model was then subjected to various flow and operating conditions to evaluate the adequacy of the distribution system to meet current and future water requirements, and to identify problem areas and system deficiencies. Improvements to the distribution system were then made in the model to evaluate their impacts and to develop recommendations for alleviating the noted deficiencies within the water distribution system. The computer model was also used to assess the operational alternative of creating a separate high pressure zone for the Great Hill area to address the low pressures being experienced in this area of the system. This alternative is discussed further in Section 6.5.

To evaluate system impacts from imposing the various domestic demands and operating conditions in the model, representative junction nodes were selected within the water distribution system to monitor potential reduction in system pressures. These pressure monitoring sites are listed in Table 6-4 below and are shown on Plate 4 included in Appendix C.

TABLE 6-4
PRESSURE MONITORING SITES FOR SYSTEM ANALYSIS

Node#	Elevation (feet)	Location
J-369	80.4	Intersection of Granite St. & Gott Ave.
J-335	51.6	Intersection of Phillips Ave. & Point De Chene Ave.
J-297	90.5	Intersection of Stockholm Ave. & Woodland Rd.
J-283	81.6	Intersection of Row Ave. & Pasture Rd
J-262	132.9	End of Squam Hill Rd.
J-244	27.5	Intersection of Main St. & Beach St.
J-134	59.6	Intersection of Marmion Way & Old Rd.
J-167	34.9	Intersection of Eden Rd. & Luccia Ave.
J-208	23.0	Intersection of Ruthern Way & Old Penzance Rd.
J-200	17.7	Intersection of Thatcher Rd & Highview Rd.
J-89	98.9	End of Jerden's Lane
J-13	158.2	Intersection of Hodgkins Rd. & Great Hill Ln.

Computer simulations were conducted under current and future water system demand allocations for average day, maximum day and peak hour demands. For the analyses, the existing water storage tanks were kept four (4) feet below their overflow elevation. Separate analyses were conducted with the Town's finished water pumps on and off. The results of the system analyses are summarized in Table 6-5 on the following page. As shown in Table 6-5, there were minimal reductions in system pressures (< 0.3 psi) predicted for all the monitoring sites between the 2014 demand allocations and the 2034 demand allocations. In comparing the system pressures as they relate to the operation of the finished water pump, the only monitoring site that exhibited pressure reductions greater than 0.4 psi was Node# J-13 located within the Great Hill area of the system. The model predicted a reduction in pressure of approximately 1.6 to 1.9 psi with the finished water pumps off. This is expected given the fact that this area is in close proximity to the finished water pumps, and is high in elevation compared to the operating gradient of system. As shown in Table 6-5, the system pressure at this site under current demands is already below the DEP recommended 35 psi for domestic service.

TABLE 6-5
RESULTS OF SYSTEM ANALYSES

		System Pressures for 2014 Demands (psi)			System Pressures for 2034 Demands(psi)		
Node No.	Elevation (feet)	Average Day	Maximum Day	Peak Hour	Average Day	Maximum Day	Peak Hour
All Tanks Set @ 230 feet, Finish Water Pump @ Town's Plant On							
J-369	80.4	64.7	64.7	64.5	64.7	64.6	64.4
J-335	51.6	77.2	77.1	76.9	77.2	77.1	76.8
J-297	90.5	60.4	60.3	60.1	60.3	60.3	60.0
J-283	81.6	64.2	64.2	64.2	64.2	64.2	64.2
J-262	132.9	42.1	42.1	42.0	42.1	42.1	42.0
J-244	27.5	87.9	87.8	87.6	87.9	87.7	87.6
J-134	59.6	74.0	73.8	73.5	73.9	73.8	73.4
J-167	34.9	84.6	84.4	84.2	84.5	84.4	84.0
J-208	23.0	89.7	89.6	89.3	89.7	89.5	89.2
J-200	17.7	91.8	91.8	91.5	91.8	91.7	91.4
J-89	98.9	57.0	56.8	56.5	57.0	56.8	56.4
J-13	158.2	32.9	32.6	32.3	32.8	32.6	32.2
All Tanks Set @ 230 feet, Finish Water Pump @ Town's Plant Off							
J-369	80.4	64.7	64.6	64.4	64.7	64.6	64.3
J-335	51.6	77.2	77.1	76.8	77.2	77.1	76.7
J-297	90.5	60.3	60.3	60.0	60.3	60.2	59.8
J-283	81.6	64.2	64.2	64.1	64.2	64.2	64.0
J-262	132.9	42.0	42.0	41.8	42.0	41.9	41.8
J-244	27.5	87.6	87.5	87.3	87.6	87.5	87.1
J-134	59.6	73.7	73.6	73.2	73.7	73.5	73.0
J-167	34.9	84.4	84.3	83.9	84.4	84.2	83.7
J-208	23.0	89.5	89.4	89.0	89.5	89.4	88.9
J-200	17.7	91.8	91.7	91.5	91.8	91.7	91.3
J-89	98.9	56.7	56.6	56.2	56.7	56.5	56.0
J-13	158.2	31.0	31.0	30.7	31.0	30.9	30.5

As previously discussed in Section 4, one approach to address the low system presssures within the Great Hill area as well as the lack of "usable" equalization storage due to the noted higher service elevations is to create a high pressure zone to serve this area. A detailed discussion on the hydraulics and requirements to creating a high pressure zone is presented in Section 6.5.

6.4.4 Fire Flow Assessment

Based on the ISO hydrant flow test data summarized in Table 6-1, the only location within the Town's water distribution system that did not meet the required fire flow at a minimum pressure of 20 psi was at the intersection of Eden Road and Penzance Road. The ISO required fire flow was 2,500 gpm and the calculated available fire flow was 1,600 gpm. However, as shown in Plate 2 in Appendix C, in order to simulate the results of this flow test in the model, the 8-inch pipe (P-211) located within Penzance Road had to be closed, suggesting that there is a potential restriction or closed valve in this area which is likely causing the noted deficiency. When opening pipe P-211 in the model to simulate the removal of the restriction, the avalable fire flow at a residual pressure of 20 psi is predicted to increase from approximately 1,710 gpm to 3,000 gpm which exceeds the ISO required fire flow of 2,500 gpm at this location.

The hydrant flow tests conducted by Dewberry indicated two locations that do not meet the ISO required fire flows including FF#3 conducted at the end of Country Club Road and FF#5 conducted in front of #25 Hodgkins Road. As shown on Plate No.2 in Appendix C and in Table 6-1, the ISO required flow at the end of Country Club Road is 2,250 gpm. The ISO test data showed an available flow of 3,500 gpm, however, the data recorded from Dewberry's flow test shows an available flow of only 1,817 gpm at this location. There is an existing 8-inch main that extends about 1,250 feet from the 12-inch main in South Street up to the end of Country Club Road to supply the Rockport Golf Club. With the exception of looping this dead end somehow back to the 12-inch main in South Street, the only way to provide the ISO required fire flow is to replace this main with a new 12-inch main.

For the flow test conducted on Hodgkins Road, the data recorded from Dewberry's flow test shows an available flow at this location of approximately 545 gpm. The closest ISO test that was conducted in 2008 was at the intersection of Main Street and Blue Gate Lane, which had a required fire flow of 1,500 gpm. Assuming that the required fire flow along Hodgkins Road would be in the range of 1,000 gpm to 1,500 gpm, there is a significant deficiency within this area. However, as shown in Plate 2 in Appendix C, in order to simulate the results of this flow test in the model, the 8-inch pipe (P-7) that connects the 6-inch main in Hodgkins Road to the 12-inch main in Sandy Bay Terrace had to be closed, suggesting that there is a potential restriction or closed valve along this pipeline, which is contributing to the noted deficiency. When opening pipe P-7 in the model to simulate the removal of the restriction, the available fire flow at a residual pressure of 20 psi is predicted to increase from approximately 545 gpm to 1,250 gpm, which is within the expected range of flows. As previously discussed, this area of the Town's system experiences low pressures due to their service elevation as compared to the operating gradient which also is contributing to the noted fire flow deficiency. The recommendation to create a separate high pressure zone to serve this area of the system will also improve the fire protection in this area as well.

To assess the ability of the water distribution system to provide adequate fire protection through the planning period, the results of the hydrant flow tests conducted by ISO and Dewberry were simulated in the model under current and future maximum day day demands to identify potential deficiencies within the system. Table 6-6 presents the results of this assessment. For this assessment, the finished water pumps were assumed to be on.

TABLE 6-6
IMPOSED FIRE FLOWS UNDER MAXIMUM DAY DEMANDS

				Predicted Flow @ 20 PSI (gpm)	
Test No.	Street/Location	Junction Node	ISO Required Flow (gpm)	2014 Maximum Day Demand	2034 Maximum Day Demand
ISO Tests Conducted 04/09/08					
FF1	Main St. @ Blue Gate Ln	J-20	4,000	**7,500(2)**	**7,300(2)**
FF2	Railroad Ave. @ King Drive	J-234	3,000	14,000	14,000
FF3	Granite St. @ Phillips Ave.	J-306	2,250	2,800	2,750
FF4	Jerdens Ln @ High School	J-85	3,000	3,650	3,600
FF5	Broadway @ Mt. Pleasant St.	J-53	3,500	5,060	4,900
FF6	Phillips Ave. @ Cove Ave.	J-340	1,500	2,000	1,975
FF7	Woodbury Ln @ Granite St.	J-371	1,000	2,094	2,075
FF8	Granite St. @ Beach St.	J-236	1,500	15,100	15,100
FF9	Eden Rd. @ Penzance Rd.	J-165	2,500	**1,675(3)**	**1,625(3)**
FF10	Rte 127 @ Frank St.	J-197	750	1,525	1,510
FF11	2nd Hydrant @ Country Club	J-192	2,250	1,750	1,690
Dewberry Tests Conducted 05/05/14					
FF1	243 Granite Street	J-306	**2,250(1)**	2,800	2,750
FF2	1C Pigeon Hill Court	J-299	**1,500(1)**	1,745	**1,700(4)**
FF3	Country Club Road (2nd hydrant)	J-192	2,250	1,750	1,690
FF4	Thatcher Road @ South Street	J-162	2,500(1)	2,700	2,675
FF5	25 Hodgkins Road	J-13	**1,000 to 1,500(1)**	**750(5)**	**750(5)**

1. Estimated based on nearest ISO flow tests.
2. Pressures within the Great Hill area noted to be below 20 psi.
3. This flow reflects the noted restriction within pipe P-211.
4. Pressure at end of Pigeon Hill Court noted to be below 20 psi.
5. This flow reflects the noted restriction within pipe P-7.

As shown in Table 6-6, minor reductions in fire protection are predicted to occur under the 2034 maximum day demands. The three deficent sites highlighted in Table 6-6 are the same sites that were noted to be deficient based on the field results of the hydrant flow tests including:

1) Lower end of Eden Road at the intersection with Penzance Road
2) End of Country Club Road at the Rockport Country Club
3) Upper half of Hodgkins Road within the Great Hill Area

As previously noted, the lower end of Eden Road is deficient due to a possible restriction such as a closed or partially closed valve within the 8-inch main in Penzance Road, or one of the upstream mains that feed Penzance Road. The mains that serve this area of the system have the hydraulic capacity to deliver the required ISO fire flow with the restriction removed. For Country Club Road, the deficiency is due to the hydraulic limitation of the existing 8-inch dead end main, which can only provide so much flow. To meet the required ISO fire flow, a new 12-inch main would need to be installed.

For the Great Hill Area, although service pressures are low, the deficiency is primarily due to a possible restriction such as a closed or partially closed valve within the 8-inch main that connects Hodgkins Road to Sandy Bay Terrace. With the restriction removed, the existing system can provide adequate fire flows in this area. As previously noted in Section 4, one approach to address the issue of low pressures within this area as well as the lack of equalization storage was to create a boosted high pressure zone to serve this area which will improve fire flows. This approach is discussed further below in Section 6.5.

6.5 ASSESSMENT OF THE GREAT HILL AREA

According to Town staff, the Great Hill area has consistently experienced low pressures along with the buildup of air since 1998 when the existing standpipe that was located on the Town's water treatment plant site was taken down to construct the new DAF facility. This area is located north of the water treatment plant along the highest elevation of State Highway Route 127 (Main Street) as shown on Plate 1 included in Appendix C. As previously noted in Section 4, based on the water system's current operating gradient of 234 feet, the higher elevated services in this area are below the minimum DEP recommended domestic pressure of 35 psi, and are the reason for the lack of "usable" equalization storage. There are also a number of fittings and high points along the piping that serve this area, which can lead to air generated by turbulent flow through the fittings to become trapped at the high points. It is our understanding that there is an air release valve at one of the high points to help alleviate any trapped air.

One approach to address the low pressures and accumulation of air within this area of the system is to create a separate high pressure zone. From reviewing the Town's water distribution, the Great Hill area can be isolated with minimal impacts to the existing system by installing check valves at the two following locations:

- 10-inch main on Sandy Bay Terrace near the intersection with Main Street
- 10-inch main on Main Street near the intersection with Hill Top Lane

To complete the separation of the high pressure zone, a new 12-inch main will need to be installed along Main Street parallel to the existing 16-inch main from the intersection of Sandy Bay Terrace to the intersection of Hill Top Lane. Elevated services that are currently connected to this section of existing 16-inch main will then need to be transferred over to the new 12-inch main including the 6-inch main in

Lamb Height Road. This will allow the existing 16-inch main to continue serving the Main Service system without being affected by the new pressure zone. During the hydrant flow tests, Dewberry had noted that the flow out of the existing 6-inch main within Hodgkins Road was extremely brown and never really cleared up. In addition, to calibrate the model, this main as well as the 6-inch main in Wilson Heights had to be given a C-value of 70 which is very low and indicates that these mains are heavily tuberculated. The Town should consider replacing these two mains with new 8-inch mains to improve the water quality and hydraulics within the newly created pressure zone.

The new booster pump station should be sized to meet the domestic needs of the newly created pressure zone. Fire protection will still be provided by the Main Service system through the two new check valves which will open once the downstream pressure drops below the upstream pressure due to a fire flow condition. Based on the future water needs and elevations of this service area, the new booster pump station should include two end suction centrifugal type pumps each rated for a capacity of approximately 60 gpm at a total dynamic head (TDH) of 60 feet along with a 5,000 hydropneumatic tank. The range of pressures expected within this new high pressure zone based on current service elevations would be approximately 54 psi to 81 psi, respectively. The station should also be equipped with controls and a surge relief valve to prevent overpressurizing the system.

The most feasible location for the new booster pump station would be at the Town's water treatment plant site. The Town already owns the property and connecting to the new station at this location would require minimal infrastructure upgrades. The limits of the proposed high pressure zone and the associated system upgrades are shown on Plate No. 5 in Appendix C.

One concern that needs to be considered when raising the operating gradient or pressure within a system is the impact to the older existing mains. Any increase in system pressure should be implemented gradually to minimize leaks and allow the mains to slowly adjust to the increased pressure. Nonetheless, there could be incidents of water main breaks and repairs. We recommend that prior to increasing the pressures within this isolated zone, the Town conduct an investigation of the valves to identify which are operational and which should be replaced to facilitate repairs as needed.

6.6 CONCLUSION

Based on the assessment and hydraulic modeling completed herein, Dewberry has determined that the Town's existing water distribution system is capable of meeting current and future average day, maximum day and peak hour demands through the planning period. It has also been determined that the existing water distribution system can deliver the required ISO fire flows under maximum day demands with the exception of three sites which were identified in Section 6.4.4. Out of the three sites, only one is a result of inadequate hydraulic capacity. The other two sites are due to potentially closed valves, which when open, will alleviate the deficiency.

As such, no significant pipeline improvements or upgrades to the Town's existing water distribution system through the planning period are required. With the exception of the low pressure issues for the Great Hill area, no immediate deficiencies were identified. However, a review of the computerized water system nodal map shows that there are several areas within the distribution system that include short segments of smaller mains which are situated between two larger mains. These sections should be replaced with mains equal in size to the connecting main to eliminate possible flow restrictions and/or "bottlenecks. These segments include:

- Replace 300 feet of 6-inch main at the High School off Jerden's Lane with 8-inch main
- Replace 800 feet of 4-inch main in Norwood Avenue with 8-inch main
- Replace 930 feet of 4-inch main in Pasture Road with 8-inch main
- Replace 200 feet of 6-inch main in Bearskin Neck Road with 8-inch main
- Replace 990 feet of 6-inch main in Phillips Avenue (south side) with 8-inch main
- Replace 2,000 feet of 6-inch main in Curtis Street with 8-inch main
- Replace 815 feet of 6-inch main in Pigeon Hill Street with 8-inch main
- Replace 2,870 feet of 6-inch main in Phillips Avenue (north side) with 8-inch main

When improving or upgrading the distribution system, the proper size selection and location of water mains and the manner which they are incorporated into the system are important factors in providing adequate flows and pressures. As a rule of thumb, any water main that extends up to 1,000 feet in length without a cross connection should have a minimum diameter of 8-inches. For lengths less than 500 feet, the minimum pipe diameter may be reduced to six-inches if the water main is used to complete a water main loop and the required fire flow is minimal. For pipeline lengths greater than 1,000 feet that have no cross connections, the minimum pipe diameter should be 12-inches. From the computerized hydraulic model, there is approximately 209,000 feet of water main, with approximately 85,000 feet being six-inch in diamater. Some sections of these 6-inch mains extend greater than 1,000 feet between interconnections. We recommend that the Town develop a continuing pipeline improvements program to replace these longer lengths with more appropriate sized mains. This will improve water transmission as well as localized fire flows.

As previously noted in Section 4, there are several isolated services within the water distribution system in addition to the Great Hill area, which due to their high elevation, are below or barely at the recommended minimum pressure of 35 psi under normal or domestic service conditions. These included a few services located at the end of Pigeon Hill Street, Landmark Lane and Summit Avenue. We recommend that the Town install individual booster pumps to alleviate the low pressure issues as well as to address the lack of equalization storage. Based on our analysis of the water distribution system, Dewberry recommends that the Town encourage the installation of individual booster pumps to existing services above 145 feet. From Plate 1 in Appendix C, there are approximately 35 to 40 services that would require individual booster pumps.

SECTION VII – RECOMMENDED PHASED IMPROVEMENTS

7.1 GENERAL

In the previous section, various improvements to the Town's water supply and distribution system infrastructure have been recommended to address deficiencies with respect to meeting current and future water needs as determined from the hydraulic analysis. To assist the Town in implementing these improvements, Dewberry has prioritized the recommended improvements into the following three categories based on the Town's immediate deficiencies, estimated needs, and benefit to the system.

- Initial System Improvements
- Short-Term System Improvements
- Long-Term System Improvements

The initial system improvements address immediate deficiencies and needs with respect to supply, system operations and fire protection, and should be considered the highest priority for implementation. The short-term improvements address current and future deficiencies with respect to fire protection, hydraulic capacity, and transmission of flow as determined from the computer modeling conducted. These improvements should be implemented over the next 5 years in conjunction with the initial system improvements. The long-term improvements address minor deficiencies within the existing water distribution system such as noted restrictions or undersized mains, and should be implemented within the next 5 to 15 years.

Estimated costs presented in the following tables are total project costs and include construction, engineering and contingencies. Construction costs were developed in part using recent construction cost data for new water mains, pumping stations and appurtenances. Other sources include the latest Means "Building Construction Cost Data" publication and manufacturers' quotes. The estimated costs do not include land acquisition, right-of-way procurement, permitting, and legal fees.

The recommended water system infrastructure improvements presented herein are shown on Plate No. 5 included in Appendix C.

7.2 PHASED CAPITAL IMPROVEMENTS

Based on the categories noted above , we have prioritized the recommended improvements presented in this report as follows:

Initial System Improvements (0-1 Year):

- Design and construction of air release/water main improvements for the Great Hills Area
- Investigate possible closed valves (Refer to Plate 2 in Appendix C for locations)
- Design and construction of new 12" main in Country Club Lane
- Design and construction of new pump station & intake at Flat Ledge for additional capacity and supply redundancy

Short-Term System Improvements (1-5 Years):

- Creation of High Pressure Zone for Great Hills Area to address low pressures and lack of equalization storage

- Evaluate alternatives for new Flat Ledge raw water transmission main
- Evaluate existing aeration and intake systems at Cape Pond to access more volume
- Replace 800 feet of 4-inch main in Norwood Avenue with 8-inch main
- Replace 930 feet of 4-inch main in Pasture Road with 8-inch main
- Replace 815 feet of 6-inch main in Pigeon Hill Street with 8-inch main
- Install individual booster pumps for service elevations greated than 145 feet (approximately 35 to 40)

Long-Term System Improvements (5-15 Years):

- Replace 200 feet of 6-inch main in Bearskin Neck Road with 8-inch main
- Replace 300 feet of 6-inch main at the High School off Jerden's Lane with 8-inch main
- Replace 990 feet of 6-inch main in Phillips Avenue (south side) with 8-inch main
- Replace 2,000 feet of 6-inch main in Curtis Street with 8-inch main
- Replace 2,870 feet of 6-inch main in Phillips Avenue (north side) with 8-inch main

Table 7-1 presents the estimated costs associated with the initial system improvements. The total cost for these improvements is $940,000. Table 7-2 presents the estimated costs associated with the short term system improvements. The total cost for these improvements is $1,375,000 including a ten-percent inflation adjustment. Table 7-3 presents the estimated costs associated with the long term system improvements. The total cost for these improvements is $1,530,000 including a twenty-percent inflation adjustment. All costs are presented in 2015 dollars.

TABLE 7-1
INITIAL SYSTEM IMPROVEMENTS COST

ITEM/DESCRIPTION	COST(1)
Air release/water main improvements for the Great Hill Area	$50,000
Investigate possible closed valves (Refer to Plate 2 in Appendix C)	N/A
1,250' of new 12" main to replace ex. 6" main in Country Club Road	$315,000
New raw water pump station w/ 200 gpm submersible pump & intake at Flat Ledge	$575,000
TOTAL – INITIAL SYSTEM IMPROVEMENTS	**$940,000**

(1) Costs do not include land acquisition, permitting and legal fees.

TABLE 7-2
SHORT-TERM SYSTEM IMPROVEMENTS COST

ITEM/DESCRIPTION	COST(1)
Creation of New High Pressure Zone for Great Hill Area	
New booster pump station w/2 60 gpm pumps & 5,000 gallon hydropneumatic tank	$150,000
10" check valve in Sandy Bay Terrace at Main Street	$35,000
10" check valve in Main Street at Hill Top Lane	$35,000
1,000' of new 12" main in Main Street from Great Hill Lane to Sandy Bay Terrace	$250,000
500' of new 8" main to replace ex. 6" main in Wildon Heights	$100,000
600' of new 8" main to replace ex. 6" main in Hodgkins Road	$120,000
Subtotal – Creation of New High Pressure Zone	**$690,000**
Perform alternatives analysis for Flat Ledge raw water transmission main	$25,000
Perform evaluation of existing aeration and intake systems at Cape Pond	$20,000
Replace 800'of 4" main in Norwood Avenue w/ new 8" main	$160,000
Replace 930' of 4" main in Pasture Road w/ new 8" main	$190,000
Replace 815' of 6" main in Pigeon Hill Street w/ new 8" main	$165,000
SUBTOTAL – SHORT-TERM SYSTEM IMPROVEMENTS	**$1,250,000**
Inflation Adjustment (10%)	$125,000
TOTAL – SHORT-TERM SYSTEM IMPROVEMENTS	**$1,375,000**

(1) Costs do not include land acquisition, permitting and legal fees.

TABLE 7-3
LONG-TERM SYSTEM IMPROVEMENTS COST

ITEM/DESCRIPTION			COST[1]
Replace 200' of 6" main in Bearskin Neck Road w/ new 8" main			$40,000
Replace 990' of 6" main in Phillips Avenue (south side) w/ new 8" main			$200,000
Replace 300'of 6"at the High School off Jerden's Lane w/ new 8" main			$60,000
Replace 2,000' of 6" main in Curtis Street w/ new 8" main			$400,000
Replace 2,870' of 6" main in Phillips Avenue (north side) w/ new 8" main			$575,000
SUBTOTAL – LONG-TERM SYSTEM IMPROVEMENTS			**$1,275,000**
Inflation Adjustment (20%)			$255,000
TOTAL – LONG-TERM SYSTEM IMPROVEMENTS			**$1,530,000**

(1) Costs do not include land acquisition, permitting and legal fees.

APPENDIX A

HYDRANT FLOW TEST DATA

Dewberry

HYDRANT FLOW TEST REPORT

PROJECT:	Rockport Water Needs	TEST NO.	#1
DATE:	5/5/2014	TIME:	9:00 AM
WEATHER:	Sunny 60 F	INSPECTOR:	CB/PC
CITY:	Rockport	STATE:	MA

SUPPLIES IN OPERATION: Finish water pumps at WTP on

WATER TANK LEVELS: All three tanks filling

HYDRANT LOCATION:

Flow Hydrant F1 - 243 Granite Street

Test Hydrant T1 - 239 Granite Street

TEST HYDRANT DATA

Hyd. No.	Observed Pressure (psi)		Corrected Pressure(psi)	
T1	Static(H_s)	53	Static(H_s)	49
	Residual(H_r)	52	Residual(H_r)	48
	Static(H_s)		Static(H_s)	
	Residual(H_r)		Residual(H_r)	

FLOW HYDRANT DATA

	Hyd. No. F1	Hyd. No. F2
Nozzle Diameter(inches)	2.5	
No. Butts Flowing	1	
Discharge Coefficient	0.85	
Main Size, inches		
Static Pressure (psi)	49	
Pitot Pressure (psi)	30	
Flow (Q_f) (gpm)	868	

(Q_f = 29.83 x cx d^2 x p^0.5)

FLOW AVAILABLE AT 20 PSI (Q_{20})

$$Q_{20} = Q_f \times \frac{(Hs - 20)^{0.54}}{(Hs - Hr)^{0.54}}$$

Q_{20} = 5,348 gpm

Dewberry

HYDRANT FLOW TEST REPORT

PROJECT: Rockport Water Needs
TEST NO. #2
DATE: 5/5/2014
TIME: 9:18 AM
WEATHER: Sunny 60 F
INSPECTOR: CB/PC
CITY: Rockport
STATE: MA
SUPPLIES IN OPERATION: Finish water pumps at WTP on
WATER TANK LEVELS: All three tanks filling
HYDRANT LOCATION:
Flow Hydrant F1 - 31 Stockholm Avenue
Test Hydrant T1 - 1C Pigeon Hill Ct

TEST HYDRANT DATA

Hyd. No.	Observed Pressure (psi)		Corrected Pressure(psi)	
T1	Static(H_s)	51	Static(H_s)	54
	Residual(H_r)	42	Residual(H_r)	45
	Static(H_s)		Static(H_s)	
	Residual(H_r)		Residual(H_r)	

FLOW HYDRANT DATA

	Hyd. No. F1	Hyd. No. F2
Nozzle Diameter(inches)	2.5	
No. Butts Flowing	1	
Discharge Coefficient	0.85	
Main Size, inches		
Static Pressure (psi)	54	
Pitot Pressure (psi)	29	
Flow (Q_f) (gpm)	853	

(Q_f = 29.83 x cx d^2 x p^0.5)

FLOW AVAILABLE AT 20 PSI (Q_{20})

$$Q_{20} = Q_f \times \frac{(Hs - 20)^{0.54}}{(Hs - Hr)^{0.54}}$$

Q_{20} = 1,749 gpm

Dewberry

HYDRANT FLOW TEST REPORT

PROJECT:	Rockport Water Needs	TEST NO.	#3
DATE:	5/5/2014	TIME:	9:45 AM
WEATHER:	Sunny 60 F	INSPECTOR:	CB/PC
CITY:	Rockport	STATE:	MA

SUPPLIES IN OPERATION: Finish water pumps at WTP on

WATER TANK LEVELS: All three tanks filling

HYDRANT LOCATION:

Flow Hydrant: F1 - 2nd hydrant on Country Club Road

Test Hydrant: T1 - 14 Country Club Road

TEST HYDRANT DATA

Hyd. No.	Observed Pressure (psi)		Corrected Pressure(psi)	
T1	Static(H_s)	67	Static(H_s)	66
	Residual(H_r)	51	Residual(H_r)	50
	Static(H_s)		Static(H_s)	
	Residual(H_r)		Residual(H_r)	

FLOW HYDRANT DATA

	Hyd. No. F1	Hyd. No. F2
Nozzle Diameter(inches)	2.5	
No. Butts Flowing	1	
Discharge Coefficient	0.85	
Main Size, inches		
Static Pressure (psi)	66	
Pitot Pressure (psi)	44	
Flow (Q_f) (gpm)	1,051	

(Q_f = 29.83 x cx d^2 x p^0.5)

FLOW AVAILABLE AT 20 PSI (Q_{20})

$$Q_{20} = Q_f \times \frac{(Hs - 20)^{0.54}}{(Hs - Hr)^{0.54}}$$

Q_{20} = 1,859 gpm

Dewberry

HYDRANT FLOW TEST REPORT

PROJECT:	Rockport Water Needs	TEST NO.	#4
DATE:	5/5/2014	TIME:	10:05 AM
WEATHER:	Sunny 60 F	INSPECTOR:	CB/PC
CITY:	Rockport	STATE:	MA

SUPPLIES IN OPERATION: Finish water pumps at WTP on

WATER TANK LEVELS: All three tanks filling

HYDRANT LOCATION:

Flow Hydrant: F1 - South Street/Thatcher Road Intersection

Test Hydrant: T1 - 1st Hydrant on Thatcher Road from South Street

TEST HYDRANT DATA

Hyd. No.	Observed Pressure (psi)		Corrected Pressure(psi)	
T1	Static(H_s)	72	Static(H_s)	64
	Residual(H_r)	64	Residual(H_r)	56
	Static(H_s)		Static(H_s)	
	Residual(H_r)		Residual(H_r)	

FLOW HYDRANT DATA

	Hyd. No. F1	Hyd. No. F2
Nozzle Diameter(inches)	2.5	
No. Butts Flowing	1	
Discharge Coefficient	0.85	
Main Size, inches		
Static Pressure (psi)	64	
Pitot Pressure (psi)	44	
Flow (Q_f) (gpm)	1,051	

(Q_f = 29.83 x cx d^2 x p^0.5)

FLOW AVAILABLE AT 20 PSI (Q_{20})

$$Q_{20} = Q_f \times \frac{(Hs - 20)^{0.54}}{(Hs - Hr)^{0.54}}$$

Q_{20} = 2,639 gpm

Dewberry

HYDRANT FLOW TEST REPORT

PROJECT:	Rockport Water Needs	TEST NO.	#5
DATE:	5/5/2014	TIME:	10:40 AM
WEATHER:	Sunny 60 F	INSPECTOR:	CB/PC
CITY:	Rockport	STATE:	MA

SUPPLIES IN OPERATION: Finish water pumps at WTP on

WATER TANK LEVELS: All three tanks filling

HYDRANT LOCATION:

Flow Hydrant: F1 - 25 Hodgkins Road

Test Hydrant: T1 - 7 Wildon Heights1@ Hodgkins Raod Intersection

TEST HYDRANT DATA

Hyd. No.	Observed Pressure (psi)		Corrected Pressure(psi)	
T1	Static(H_s)	28	Static(H_s)	28
	Residual(H_r)	15	Residual(H_r)	15
	Static(H_s)		Static(H_s)	
	Residual(H_r)		Residual(H_r)	

FLOW HYDRANT DATA

	Hyd. No. F1	Hyd. No. F2
Nozzle Diameter(inches)	2.5	
No. Butts Flowing	1	
Discharge Coefficient	0.85	
Main Size, inches		
Static Pressure (psi)	28	
Pitot Pressure (psi)	20	
Flow (Q_f) (gpm)	709	

(Q_f = 29.83 x cx d^2 x p^0.5)

FLOW AVAILABLE AT 20 PSI (Q_{20})

$$Q_{20} = Q_f \times \frac{(Hs - 20)^{0.54}}{(Hs - Hr)^{0.54}}$$

Q_{20} = 545 gpm

APPENDIX B

HYDRAULIC MODEL DATABASE

APPENDIX C

WATER SYSTEM PLATES