BAR HARBOR WATER DIVISION
WATER SYSTEM MASTER PLAN

Town of Bar Harbor

Bar Harbor, Maine

April 2005

WOODARD & CURRAN
Engineering • Science • Operations

1 Merchants Plaza
Bangor, ME 04401
(207) 945-5105

www.woodardcurran.com

TABLE OF CONTENTS

SECTION **PAGE NO.**

Executive Summary ES-1

1. INTRODUCTION: PURPOSE OF A MASTER PLAN 1

2. TRANSMISSION AND DISTRIBUTION SYSTEM EVALUATION 2

2.1 History and Overview 2
2.1.1 Introduction 2
2.1.2 Source 2
2.1.3 Intake 2
2.1.4 Filtration Waiver 3
2.1.5 Disinfection 4
2.1.6 Transmission Lines Above Duck Brook Pump Station 5
2.1.7 Historic System Storage – Above Distribution System 5
2.1.8 System Operation 6
2.1.9 New System Storage – Above Distribution System 7
2.1.10 Transmission Lines Below Duck Brook Pump Station 7
2.1.11 System Storage – Southern End of Distribution System 8
2.1.12 System Storage – Northern End of Distribution System 8
2.1.13 Booster Stations 9
2.2 Current & Potential Future Demands 9
2.2.1 Impact of Seasonal Fluctuations 9
2.2.2 Cost of Providing Water 10
2.2.3 Growth Potential 10
2.3 Dam Issues 13
2.4 Distribution System Model 14
2.5 Current & Future Bottlenecks & Deficiencies 15
2.5.1 Seasonal System 15
2.5.2 Small Size Pipe 17
2.5.3 Poor Condition Pipe 17
2.5.4 Bottleneck Pipes 20
2.5.5 System Pressures and Flow Limitations 21
2.6 Storage Tank Operation 27
2.6.1 The Jackson Laboratory Tank 29
2.6.2 New Dreamwood Hill Tank 30
2.7 Unidirectional Flushing Program 32

3. TREATMENT EVALUATION 35

3.1 Introduction 35
3.2 Waiver From Filtration 35
3.3 Issues Surrounding Maintenance of Source Water Quality & Watershed Protection 36
3.4 Future Regulatory Imperatives 38
3.4.1 LT2 38
3.4.2 DBP Rule 39
3.4.3 Other Rules 40

3.5 Disinfection Alternatives 40
3.5.1 Existing Use of Chlorine Gas 40
3.5.2 Sodium Hypochlorite 42
3.5.3 Primary Disinfection Using Ultraviolet Light 42
3.5.4 Use of Chloramines for Secondary Disinfection (System Residual) 45
3.6 Filtration 45
3.7 Duck Brook Pump Station 47
3.8 System Control and Data Acquisition (SCADA) Improvements 48

4. UNIFIED ACTION PLAN 51

4.1 Introduction 51
4.2 Prioritized Recommendations 51
4.2.1 Discussions & Planning Items 51
4.2.2 On-Going Upgrades 52
4.2.3 Special Projects 54
4.3 Summary 58

LIST OF TABLES

TABLE **PAGE NO.**

Table 1: Filtration Waivered Water Systems In Maine (2004) 3
Table 2: Significant Water Users In Bar Harbor's System 12
Table 3: Future Projected Flow Rates For Design 13
Table 4: Low C-Value Transmission Line Pipe Footage 20
Table 5: Piping Bottlenecks In Bar Harbor's Water Distribution System 21
Table 6: Fire Hydrant Hydraulic Analysis (April 2004) 23
Table 7: Flushing Program 33
Table 8: Action Plan Project Summary 59

LIST OF FIGURES

FIGURE **PAGE NO.**

FIGURE 1: System Map ..... Pocket at Back
FIGURE 2: Bar Harbor Water System Seasonal Use Pipe (Surface) 16
FIGURE 3: Bar Harbor Water System Small Diameter Water Lines 18
FIGURE 4: Bar Harbor Water System Large (≥6" Dia.) Water Lines in Poor Condition 19
FIGURE 5: Bar Harbor Water System High Headloss Gradients 22
FIGURE 6: Bar Harbor Water System Available Fire Flows 26
FIGURE 7: Bar Harbor Water System Low Pressure @ 3 MGD Demand Supplied at Eagle Lake Gradient 28

EXECUTIVE SUMMARY

This Master Plan evaluates the treatment and distribution system of the Town of Bar Harbor Water Division. It considers potential needs of the system which may be driven by growth, worker safety, protection of public health, aesthetic concerns, regulatory changes, and equipment and piping deterioration.

The purpose of the plan is to lay out, in an organized fashion, a comprehensive vision of the type of expenditures the system may require over the next several years. It is also to suggest a sequence in which these projects might be undertaken so that one change is not rendered obsolete by subsequent changes and so that adverse impacts on water quality and system capacity are not created. Individual system improvements are summarized in a table at the end of the document, along with a timeframe and budgetary cost.

A system model has been developed in order to determine the effects of piping changes to the system, to identify current system bottlenecks to capacity, and to recognize areas of piping which may be leading to water quality impacts. Generally, piping replacement projects are scheduled as on-going. This reflects the fact that water pipe replacement is usually driven by other projects, such as roadway reconstruction or development. The Master Plan does highlight those pipe segments that are of particular concern for one reason or another so that if multiple roadway projects are under consideration at one time, those that are of most benefit to the water system can be undertaken first.

This document lays out a significant investment in the system. It should be scrutinized carefully and receive considerable public input, particularly in regard to the proposed timeframe for undertaking these projects and the level of public support for potential expansion projects. Other than the on-going piping projects, approximately $2,000,000 in suggested system improvements are discussed for implementation over the next few years. Some pressing concerns, such as the potentially required removal of the abandoned seasonal water transmission line (currently under discussion with the National Park Service) could have widely varying financial and scheduling impacts that cannot presently be predicted. Negotiations are currently underway.

This Master Plan also develops a viable water filtration facility alternative, should the system lose its Waiver From Filtration. The development of a filtration option and consideration of its siting issues are requirements of the State's Drinking Water Program and have been requested by the National Park Service as well. It is important to understand that implementation of filtration will cost on the order of $7,000,000 to $8,000,000. This highlights the importance of the Water Division exercising great diligence in all its activities to protect and maintain its filtration waiver: from watershed protection, equipment maintenance, and technology improvements, to comprehensive monitoring testing and reporting.

1. INTRODUCTION: PURPOSE OF A MASTER PLAN

This document presents a Master Plan for the Town of Bar Harbor's Water Division. The purpose of such a plan is to assess the current condition and future needs of the entire water system in an integrated and comprehensive fashion. Typically, water systems approach capital improvements on an individual basis, responding to sudden growth in one area, problems with water quality in another, and emergency equipment replacements in a third. Frequently, distribution system piping replacement schedules are driven more by other road construction projects than they are by the actual needs of the water system. Each of these reasons is a very real and legitimate driver for undertaking water infrastructure projects. However, without an overall plan for the system in mind, it is often difficult to make wise choices as to which projects to place first, how to size pipe and equipment, or to understand how individual projects will impact budgets and future user rates.

This Master Plan looks at the history of the Bar Harbor water system, it water source, treatment, distribution system, and water quality issues. It starts by giving an overview of the various elements of the system so that all readers have the background to understand the interaction of all the elements of the system. It then discusses the needs of the distribution system and how and where the community may wish to expand and improve the system. This discussion is based on information obtained from a distribution system network computer model which helps assess water flow and pressures within the distribution system under various conditions.

The Plan goes on to evaluate treatment of the water, both present day treatment and possible future modifications. These modifications can be driven by the desire to improve the esthetics of the community's water, by a desire to improve the efficiency of the system or to adopt new or more reliable treatment and monitoring technologies, or by future regulatory requirements.

The goal is to provide a comprehensive overview of the entire system and to make all decision makers aware of the various needs of the system and the priority which should be given to these improvements. Ultimately, a prioritized list of projects is presented along with initial budgetary costs and a proposed implementation schedule.

2. TRANSMISSION AND DISTRIBUTION SYSTEM EVALUATION

2.1 HISTORY AND OVERVIEW

2.1.1 Introduction

The Bar Harbor water system has been operational for at least 130 years. It was originally incorporated in 1874 as the private Bar Harbor Water Company. The water company was in turn taken over by the Town in 2001, and has since been run as a division of Public Works with its own enterprise fund.

Today, the water system serves approximately 1,700 residential, commercial, and governmental customers. The water distribution system covers a majority of the Town, extending from the Village of Bar Harbor to the Villages of Hulls Cove and Salisbury Cove to the north. To the south, the water system extends to The Jackson Laboratory.

2.1.2 Source

The system takes its source from Eagle Lake within Acadia National Park. Eagle Lake in turn is fed from Bubble Pond and the watershed within the Park north of Pemetic Mountain and east of Sargent Mountain. The watershed encompasses an area of approximately 3.6 square miles, entirely within the National Park. Although this has not always been the case, the water system owns no land within its watershed other than the small lot at the north edge of Eagle Lake on which is situated its intake and chlorination building.

In the vicinity of the intake, Eagle Lake is contained by an earthen dam approximately 430 feet in length, constructed by the water company in 1895. A concrete spillway and flash board structure maintains the lake at its normal pool elevation of 276.5 feet. The present-day building along the lake shore at the dam (presently used as the chlorination station) was originally constructed for use as an intake screen house.

2.1.3 Intake

The original intake consisted of a 20-inch diameter cast iron pipe which projected through cribwork at the dam approximately 80 feet into the lake. The depth of the original intake invert is approximately seven feet below the normal water surface, although low lake levels can reduce the depth to the crown of the pipe to less than 4.5 feet below the lake surface. The original intake was subject to occasionally high coliform counts from bank debris and from growth and deposits on the cribbing. In addition, wind and wave action along the north shore of the lake would lead to periods of higher than desirable turbidity at the inlet. The original intake was screened through the use of drop-in screen panels located within the lower level of the present chlorination building. These screens were somewhat ineffective, and were a maintenance issue.

In 1998 the intake was extended 300 feet through the use of 24-inch diameter polyethylene pipe weighted using concrete anchor rings. The pipe was laid along the contour of the lake bed, and was supported in crushed stone. The end of the intake was fitted with a concrete vault structure which supports a 27-inch diameter, 85-inch long cylindrical, tee shaped wedge wire screen. The screen wire provides a clear opening of 0.25 inches. The tee is oriented upright, with the centerline of the screen at a normal water depth of twenty feet. This places it approximately 7.75 feet above the lake floor, approximately 360 feet from the north lake shore.

The screen is provided with a 2-inch flanged connection for the possible future addition of a pneumatic back flushing/purging system. Given that there is relatively low biological activity in Eagle Lake, an automated purging system was not deemed necessary at the time the screen was installed, and it has not proven necessary over the last six years the system has been in use. The Water Division has a diver inspect the screen annually, and manual cleaning is performed when necessary. The 2-inch connection is presently blanked off.

At the time of the intake extension, the drop-in screens were removed and the incoming water was hard-piped directly through the screen vault beneath the chlorination building. This allowed the installation of an inlet magnetic flow meter and improvements to the dosing and feedback controls for disinfectant additions at the chlorination building. This water treatment will be discussed in more detail later.

2.1.4 Filtration Waiver

The Bar Harbor water system presently operates under a waiver from filtration. This waiver was issued to this system and thirteen other systems in the State of Maine in 1992 due to the high source water quality and good watershed protection that these systems offered. Since that time two systems have lost their filtration waivers. The remaining systems within the State and their source waters are listed below, in Table 1.

Table 1: Filtration Waivered Water Systems In Maine (2004)

Portland Water District	(Sebago Lake)
Lewiston Public Works Water Division	(Lake Auburn)
Auburn Water District	(Lake Auburn)
Bangor Water District	(Floods Pond)
Brewer Water Department	(Hatcase Pond)
Great Salt Bay Sanitary District Damariscotta	(Little Pond)
Aqua Maine- Camden Rockport Division	(Mirror Lake)
Vinalhaven Water District	(Round Pond)
Bethel Water District	(Chapman Brook)
Mount Desert Water District – Seal Harbor	(Jordan Pond)
Mount Desert Water District – Northeast Harbor	(Hadlock Ponds)
Bar Harbor Water Division	(Eagle Lake)

A Waiver From Filtration should be considered as one of the Bar Harbor water system's greatest assets, second only to the water supply itself. The value of this waiver, and what should be done to protect it, will be considered in detail later in this report. For now it is sufficient to note that this waiver allows the system to avoid considerable unnecessary cost, complexity, and problems with waste disposal that would be associated with the construction and operation of a filtration plant. This is particularly true given that siting such a facility would be difficult and would involve negotiations with the National Park Service since the water system owns no suitable land for such a facility, having given up most of its easements and ownership to the Federal Government in 1916 during the formation of the National Park.

A filtration waiver does bring with it, however, a significant responsibility to protect source water quality, to provide an unassailable disinfection system, and to conduct enhanced monitoring and reporting. It also means that the Bar Harbor system will remain in the regulatory spotlight, and that the system will receive no regulatory leeway if problems do develop. This places an additional burden on the Division's staff to be diligent in all their activities and reporting. As part of their normal activities, the Bar Harbor Water Division should develop and maintain close contact with each of the waivered systems within the State. The systems will all find that it is in their mutual best interest to stay close to issues affecting waivered systems nationally and to speak with one voice as regulatory proposals are developed.

2.1.5 Disinfection

An unfiltered system has not always been present in Bar Harbor. In 1905, the then private water company installed a series of rapid sand filters on an approximately three acre lot the company owned on the northwest corner of Route 233 and the Duck Brook Road. This system was originally put in place to "…eliminate the disagreeable tastes and odors from the water supply of Bar Harbor arising from uroglena in Eagle Lake.". This treatment system was operational until approximately 1932 when the filters were abandoned and the land was transferred to John D. Rockefeller, Jr. and then ultimately to the Park Service. Filtration was apparently discontinued due to continued odor complaints regarding the finished water. Such issues were relatively common in the days before effective chlorination was practiced and untreated rapid sand filtration was not an effective treatment for taste and odor concerns which often resulted from algae.

The overgrown remains of this system can still be seen in the woods along Route 233. This land is now part of Acadia National Park.

Since the time the filters were abandoned, Bar Harbor's water has been disinfected using chlorine gas. The first chlorination system was installed by the water company in 1933. At that time, the water company's annual report noted, "While in the 59 years that Eagle Lake water has been used for domestic purposes no disease has ever arisen from this source, still as a precautionary measure, a chlorinating plant has been installed at a cost of $8,420.33." This original chlorination facility was actually at what is now the Duck Brook pump house, not at the lake shore screen house building where chlorine addition is currently made. The chlorine handling portion of the pump house building was constructed in 1932.

Today, water flows into the former screen building at the lake shore where chlorine gas, supplied through a principal and a backup chlorinator, is dosed based on a primary flow signal from an inlet magnetic flow meter installed in 1998. Beyond the screen building, the transmission line system consists of 50 feet of 20-inch cast iron pipe, 1,200 feet of 24-inch cast iron pipe, and 5,000 feet of parallel 16-inch and 20-inch cast iron pipe. All this is prior to a pipe manifold at the Duck Brook pump house. The necessary disinfectant contact time is achieved in this varying 16 to 24-inch transmission line between the point of application and the Duck Brook pump house residual monitoring station. In 1998 a tracer study was conducted at varying flow rates in order to determine the actual travel time in this system and the corresponding CT (disinfectant Concentration x Time) that was being achieved.

At the Duck Brook pump house a chlorine residual is measured and used for a manual computation of the chlorine CT. Based on the measured chlorine residual, and the recent trend of that residual, a trim signal is transmitted back to the chlorination building and used to adjust the applied chlorine dose. If the chlorine residual measured at the pump house falls below the acceptable range, an automatic valve will close, isolating the raw water supply from the treated water already in the system. Water will then be drawn from storage downstream from the pump house. The low residual water within the transmission line would have to be flushed from the system at the pump station site.

2.1.6 Transmission Lines Above Duck Brook Pump Station

The transmission line system parallels the Duck Brook Road from Route 233 down to the Duck Brook pump station building near the Duck Brook entrance to the Park's carriage path system. The lines run between the roadway and the wetland that includes Duck Brook. This is an area of very shallow bedrock. For most of their length, the transmission lines are very shallow buried, and in many cases are surface run on bedrock and mounded over with cover. In several instances the lines are exposed where the piping crosses feeder streams and swales.

This line is very susceptible to damage and deterioration and has been repaired on numerous occasions. During recent roadwork conducted by the Park Service during the fall of 2003, the line was damaged by the excavation contractor and was partially dewatered. The resulting air introduced to the line led to significant problems with the intake and ultimately caused a portion of the intake pipe within the lake to separate.

As well as damage and deterioration, the exposed nature of the line makes it susceptible to freezing. Historically, this has not been an issue since the system has primarily operated under gravity flow conditions during cold weather. This has the impact of assuring that there is always a relatively significant flow moving through the line, preventing freezing. This has been augmented through the use of system bleed lines in Town to keep water moving at a sufficient rate.

The potential exposure of the transmission line to freezing is a major consideration in any modifications to the water system that assume conversion to a largely pumped year-round system. Such a pumped system would most likely result in minimal water flow through the transmission line during pump-off periods, usually seen during the night when temperatures are coldest. This will provide a significant limitation to what and where future system changes of any significance could be made. Alternatively, it may be necessary to consider blasting and burying a new transmission line if future changes to a pumped treatment system warrant it.

2.1.7 Historic System Storage – Above Distribution System

In 1895, a stone tower (still present in the woods approximately 2000 feet north of the Eagle Lake shoreline) that was the system's original storage tank and screen house was abandoned. In 1901, the water company completed work on a 700,000 gallon, open top, stone lined reservoir located in the woods near the present day intersection of the Duck Brook Road with the Park Loop Road. This reservoir, with a normal pool elevation of approximately 257 feet, was approximately twenty feet below the Eagle Lake normal level and was used to buffer pressure fluctuations between daytime and nighttime water demands. Although this structure is still present, it was abandoned by the water system sometime prior to 1935, presumably due to the fact that it was uncovered and was at an insufficient elevation to support the continued growth and expansion of the water distribution system into ever higher service elevations.

In 1935 it was determined that additional pressure above that which could be provided by gravity from the lake elevation, would be needed during the summer peak demand period. It was at this time that the Duck Brook (New Mill Meadow) pump station, as it now exists, was constructed alongside the original chlorination facility. This station included facilities for adding lime to the water as a means of adjusting pH to improve alkalinity and reduce the naturally corrosive nature of the water. At that same time, John

D. Rockefeller, Jr. gave permission to build a storage tank on his land across the road from the station. This 60-foot diameter, 20 foot high steel tank, completed in 1936 at a cost of $39,000, is still present and is used during the summer pumping period. It is known as the Great Hill or summer tank. Its capacity is 528,000 gallons. The tank base is at an elevation of 317 feet, and its overflow elevation is 341.

2.1.8 System Operation

From 1935 until 2000, the water system was operated largely in the same manner. For approximately ten months of the year the system operated under strictly gravity conditions, with the water bypassing the pump station's pumps and flowing directly to town at whatever rate the current system demand required. Chlorine dosage was adjusted continuously to match the varying flow. In recent years, typical average daily flows of 0.7 to 0.9 MGD, with peak flows of 1.6 MGD were seen for this period, depending on domestic use and the amount of water lost to flushing and through the use of blow-offs and bleeders.

During July and August when the service area population is greatly increased by both seasonal customers and tourists, tremendous increases in demand have been observed. Average daily demand during these months is typically 1.6 MGD, with peak demand during hot, dry weather reaching 3 MGD. During July and August, distribution system pressures are monitored downtown from the Water Division's office located at 337 Main Street. Typically, during the winter months, a system pressure of 98-102 psi is observed at this office. During the summer peak demand, if gravity flow were maintained, the pressure would be observed to drop to as low as 88 psi at this location.

In order to correct this condition, and maintain system pressures to within a range of 98 to 100 psi at the Division office, the Duck Brook pumps are put into operation to pump water into the Great Hill summer tank. These pumps operate at a constant flow of 3800 gpm and lift the water into the tank, adding an additional 64.5 feet to the hydraulic grade line at this point in the system, for a system pressure increase of 27.8 psi.

The control system for these pumps is somewhat unreliable, and the pumps are operated with considerable human oversight. Presently the pumps can be expected to operate 5-6 times each summer day, for approximately 1.5 hours each cycle. The 3800 gpm flow that the pumps generate in the transmission line/CT chamber is far higher than would otherwise be seen, and has led to problems with sudden changes in applied chlorine dose and the ability to meet proper CT requirements. This pumping rate is more than twice the peak demand that would otherwise be experienced by the system. This will be discussed in greater detail later, under the section on Duck Brook Pump Station Modifications.

The additional 27.8 psi gain that the summer tank provides increases the local system pressure at the pump station to approximately 54 psi which results in a significantly higher pressure than could be tolerated downstream in the 4225 feet of 20-inch cast iron and 2460 feet of 14-inch cast iron transmission line running between the pump station and the distribution system. To correct for this, a pressure reducing valve is located on the outlet line from the summer tank, within the basement of the pump station building. This valve is manually controlled based on the pressure read at the Water Division office.

In order to keep the office pressure within the predetermined range, it may be necessary to reduce the additional pressure supplied by the summer tank by 50 to 68 percent. On peak summer days, the pressure on the downstream side of the pressure reducing valve is initially set to 35-40 psi and is backed down further to a minimum of 26 psi (gravity feed condition) as the pressure downtown comes to within range. At this point, all additional pressure benefit from pumping up to the summer tank is lost.

This represents a tremendous waste of energy, particularly given the high rate (3800 gpm resulting from 100 horsepower pumps) at which the pumps fill the tank. The water system typically spends approximately $2,000 each month for electricity to operate the pump station to fill a tank that is too high to effectively serve the needs of the system. The good news is that, to date, it has only been necessary to run this system for two months of the year.

During July and August, the summer tank provides one additional benefit. It provides 528,000 gallons of water in storage for fire fighting, approximately 6,000 feet closer to the distribution system than the lake. This treated water is also available as a buffer, should it be necessary to isolate the downstream system from the raw water supply in the event that the disinfectant residual drops below the required level. Without this water in place, the distribution system would soon see negative pressures develop if the water feed from the lake was automatically closed off at the pump station.

The situation is quite different during the September through June period. Even ignoring the wasteful use of energy, it would not be possible to retain water in storage in the steel summer tank during the winter months. As previously noted, a pumped system during this time would lead to stagnation within the upper transmission lines during the pump-off period, and would subject them to the very real possibility of freezing. If smaller pumps were used to keep water flowing to the tank throughout the day and night, there would be insufficient movement of the water level within the tank and an ice plug would form, damaging the tank. The solution has been to convert back to a gravity flow system during the winter months.

2.1.9 New System Storage – Above Distribution System

Without the use of the summer tank, it was not formerly possible to provide full isolation of the raw water supply from the distribution system during the winter months. The system was always at risk of either receiving a low disinfectant residual if the water was allowed to pass into town or negative pressures if an isolation valve was closed.

To rectify this situation, in 2001 the water system added an insulated, below-grade, cast-in-place, 500,000 gallon concrete storage tank across from the pump station approximately half-way up the hill to the summer tank. This tank has a full water elevation of 276.5 the same as the nominal lake level. It is designed to function when the system is operating under gravity flow conditions. The continuous in and out flow through the tank keeps the residence time within the tank low and allows for continuous flow through the upper transmission lines. In the event of an unacceptably low disinfectant CT, the raw water portion of the system can be isolated from the distribution system through the use of the automatic control valve in the pump station basement, leaving 500,000 gallons of treated water in storage.

One nuisance to this dual system is that it is not possible to operate the two tanks at the same time, so there is an annual commissioning and decommissioning required for each tank.

2.1.10 Transmission Lines Below Duck Brook Pump Station

Below the Duck Brook pump station the transmission line continues into Town. The principal line is buried 20-inch cast iron extending approximately 4225 feet before reaching the first customer in the vicinity of Bloomfield Road. Although generally buried to suitable depth, this line has seen past failures due to frost as well as its placement directly on bedrock in places. Damage due to hydraulic shock is a particular concern with this old line.

A second transmission line once split off directly below the Duck Brook carriage path entrance and ran along the stream and through the gorge until it met Route 3 in the vicinity of the present day Days Inn Motel. The original line was surface run 12-inch cast iron with leaded bell joints. This line lies entirely within Acadia National Park land. The line was abandoned due to its poor condition in early 1997, and the line remains in place. It has remained an issue for the National Park Service and the Water Division due to concerns of possible lead contamination of soils in the vicinity of the pipe joints. The Water Division and the Park Service are in on-going discussions as to what will be required to ameliorate the situation and whether the line and contaminated soils will need to be physically removed.

In 1997 a 12-inch polyethylene line was run from the main transmission line on the Duck Brook Road to the same point of connection on Route 3. This surface run line is placed in service only during the summer months, when additional demand along the northern end of the system into Hulls Cove and beyond requires an additional direct feed.

The use of this secondary seasonal transmission line creates an unforeseen problem related to possible additional CT credit. If the 20-inch permanent transmission line was the sole line in use, contact time credit could be taken for the length between the pump station building and the first customer on Bloomfield Road. Because the flow split between the permanent line and the seasonal line is not known, it is not possible to compute and use the contact time afforded by either of these lines toward obtaining the CT credit. Due to the problem of obtaining a residual sample at either junction point to the distribution system, and the inability to isolate the incoming water from the distribution system at either point, this would only be a consideration in the event that a low residual was measured at the pump station and it became necessary to rechlorinate the water within the transmission line, rather than having to dump it.

2.1.11 System Storage – Southern End of Distribution System

The distribution system and its issues will be considered separately, however, several other elements of the system warrant inclusion in this overview.

The system includes year-round treated water storage in a 500,000 gallon steel tank near the southern end of the system along Route 3, opposite The Jackson Laboratory. The Lab is the Water Division's single largest water user. This tank was constructed primarily to supply the domestic and fire protection needs of The Jackson Laboratory, and is located at the end of a long dead end pipe run that limits flow back into the Bar Harbor Village area during high demand periods and fire flow conditions. This tank sits at a base elevation of 235 feet and has an overflow elevation at 265 feet. The tank level is controlled through the use of an altitude valve. Part of the strategy of pressure monitoring of the system at the Water Division office, is to ensure that sufficient pressure is maintained in the distribution system to allow the Jackson Laboratory tank to fill within a reasonable period of time.

2.1.12 System Storage – Northern End of Distribution System

A second, seasonal reservoir, a steel 50,000 gallon tank, is in place at the top of Dreamwood Hill, north of Hulls Cove near the northern end of the system. The tank sits at a base elevation of 190 feet, and has an overflow elevation of 215 feet. Little operational data is maintained on the fluctuation of water in this tank, and there is some confusion as to how well it actually functions within the system. This tank is used only during the summer months and is fed by and serves primarily surface run, seasonal, galvanized, small diameter piping. This tank generally fills by gravity, although pump stations exists near the base of the hill (Gerrish Pump) and at the inlet to the tank. The tank helps provide stable flow to the terminus of the system just prior to King's Creek in Salisbury Cove.

For many years there have been discussions raised regarding the installation of a permanent, year-round tank on Dreamwood Hill and converting much to the distribution system in this area to year-round service. Projects such as the recent Birch Bay Village complex on Crooked Road, development further along Crooked Road, poor fire flow along Eden Street through Hulls Cove and beyond, conversion of formerly seasonal residences to year-round in the area of Degregoire Park north and east of Dreamwood Hill, and the fact that open land in this area has been identified for growth in the Town's Comprehensive Plan, all argue for improvements to the water system in this area. The Water Division owns a parcel of land adjacent to the existing seasonal tank and various plans for a permanent tank on the order of 500,000 gallons storage capacity have been discussed.

Depending on which of these plans move ahead, it may be necessary to add a high pressure zone to the distribution system and increase the hydraulic grade line north of Crooked Road. Other than the two small areas discussed below, the entire year-round portion of the distribution system is currently on a single pressure gradient.

2.1.13 Booster Stations

Two small booster stations are part of the distribution system at Mountain Avenue and at Arata Drive. These stations, in below ground vaults, serve the local neighborhoods at the high elevation of the distribution system. Each station has an output of approximately 100 gpm. There have been some discussions about interconnecting the piping between the two neighborhoods to improve water flow and serving them through a single pump station at Mountain Avenue.

2.2 CURRENT & POTENTIAL FUTURE DEMANDS

2.2.1 Impact of Seasonal Fluctuations

Even in 1933, significant seasonal fluctuations in water demand were seen as an important aspect of the Bar Harbor system. The following was noted in the August 1, 1933 annual report, "An interesting fact has been brought out by the use of the Venturi Meter (newly installed in 1933). On rainy days this summer the consumption has been at the rate of about two million gallons per day, but on warm dry days the consumption has increased to the rate of three and one-half millions per day. The inference is, therefore, that the use of water for lawns and gardens is from 40% to 50% of the day flow at the peak of the load."

Since that time and before, Bar Harbor's water utility has been subject to wide seasonal swings in its water consumption. This is due to its wide seasonal population swing as a tourist destination and its relatively high percentage of seasonal residents. It is also due, however, as the above quote makes clear, to the types of homes and properties located in the community. Extensive lawn plantings and formal gardens can have a significant impact on water demand, a fact that becomes quite clear when comparing water use during hot dry summers against cool, foggy or damp ones.

The overall impact of this swing is that all treatment and distribution facilities need to be sized with peak flows in mind, flows that are normally seen for only a few weeks each year. The rest of the year these facilities may be significantly oversized for the relative needs of the community. The Town has experienced this same frustration when it dealt with its wastewater treatment system. Wide swings in demand and drastic peaking factors for relatively short periods of time can lead not only to the need to construct large expensive facilities that are not easily paid for by a relatively small year-round user base, but also to inefficiencies in the treatment process itself. In the case of drinking water distribution, water quality issues that may develop from poor circulation or lengthy storage times. It becomes critical therefore, to design flexible, staged systems that can be brought on-line in increments, or to provide alternative redundant systems that can be switched on and off between low demand and high demand periods.

One aspect of this problem would be related to treated water storage volume. With peak summer day consumption rates approaching 3 MGD, some guidelines would suggest that a minimum of 3 million gallons of storage be maintained in the system in the event that an equipment failure occurs. At the more typical daily average demand of 0.9 MGD experienced during approximately ten months of the year, 3 MG held in storage would quite possibly result in a loss of disinfectant residual and the need to artificially circulate the stored volume through additional pumping along with rechlorination of the stored water.

In addition, the relatively wide extent of the distribution system and the fact that flow is significantly reduced along many of the outlying margins of the system would dictate that water storage would need to be distributed throughout the system in several tanks rather than centrally located in a single area. This would further add to construction and maintenance costs.

2.2.2 Cost of Providing Water

The community has managed to avoid many of the most significant costs associated with treating and providing water through the maintenance of its waiver from the requirement of filtration as part of its water treatment. Filtration facilities are not only costly to construct and operate, but are notoriously difficult to operate effectively over widely ranging flow rates. In addition, all but large slow sand filters generate a wastewater flow stream in the form of backwash that must be treated.

Bar Harbor’s system was able to obtain a filtration waiver in the early 1990’s due to the very high quality of its source water and due to the high degree of protection afforded its watershed by development controls as a result of being contained within Acadia National Park.

For the foreseeable future it should be a primary focus of the water system to maintain its filtration waiver. Other changes will and should occur in the way the system treats (i.e. disinfects) its water, however, future growth in demand will have significantly less of an impact in terms of facility size and cost than would be the case if a filtration step had to be constructed and sized for a twenty year lifecycle. Due to the considerably lower capital costs associated with the construction of disinfection facilities and the generally shorter lifecycle of this type of equipment, much more conservative growth projections can be used for sizing.

2.2.3 Growth Potential

It is often the case that explosive growth in demand can occur if the ability to simply transport water is improved. Throughout the Bar Harbor distribution system there are areas where old, undersized piping may be limiting excessive consumption. We frequently find that in many communities if system pressures and flow capacity are improved there are sudden increases from pent up demand that leads to a need to upsize treatment or storage facilities. This is similar to the “build it and they will come” phenomenon seen in highway construction.

Unlike the neighboring community of Mount Desert, Bar Harbor collects revenue for its water through the use of water meters. This tends to promote water conservation rather than the alternative, flat rate fixture charges in effect in Mt Desert. Through the years, water consumption on a per capita basis has decreased in Bar Harbor as the cost of water has risen. Since a significant portion of the system serves seasonal piping or seasonally occupied properties, metering does come at a labor premium. Unlike many communities, a significant portion of water meters are removed from service each fall to prevent damage and need to be reinstalled in the spring.

One naturally limiting factor in any water system is the safe yield of the water supply. This is the daily volume that can safely be withdrawn from the source, in this case Eagle Lake on a long term basis. In the case of Bar Harbor, although no recent studies have been undertaken, the safe yield of Eagle Lake is likely far in excess of what would likely be necessary for the foreseeable future. This assumption is made based on the overall storage in Eagle Lake, the area rainfall, historical fluctuations in the lake level, and the lack of competing uses drawing on this water supply. While this is a fortunate situation in that the water source will not be a limiting factor for the foreseeable future, it provides no natural cap to the growth of the system.

Perhaps more limiting is the land area available for development. Much of the central portion of the distribution system is presently fully developed and might reasonably be expected not to generate increasing demand. One exception to this results from the conversion of several smaller properties to larger hotels and condominium projects which could support larger populations and use more water. Overall, these are unlikely to occur in such numbers as to create a significant unanticipated change in water consumption.

Other than The Jackson Laboratory, there are presently few water intensive industries within the service area. This is likely to continue to be the case. The Jackson Laboratory continues to expand, however and is likely to require an ever increasing supply of water, both for its process needs and for firefighting capacity. Water consumption at the Lab has not been observed to increase in recent years at the rate that the overall facility has expanded. During the last three years water consumption has been relatively flat. Typical first quarter (winter) consumption has been 1.3 MM Cubic Feet. Typical third quarter (summer) usage has been 2.1 MM cubic feet.

Changes in Lab water consumption seem to be much more significantly tied to the cost of water, and particularly to the cost of wastewater disposal. Even with growth in the campus, it has been noted that water consumption decreases after significant wastewater rate increases. To a large degree, greater efficiency in water use by automated cleaning equipment and other projects undertaken at the Lab more than offset growth in production or research facilities. However, during the next few years there will be significant growth at the lab. Increases in water use are projected to be on the order of 60%, from the present annual average of 80 gpm to approximately 130 gpm.

Table 2 presents the largest industrial/commercial customers of the water system.

Table 2: Significant Water Users In Bar Harbor's System

	User	2002 Total Usage (ft^3)
Year-Round		
	The Jackson Lab	5,902,211
	Bar Harbor Motor Inn	945,000
	College Of The Atlantic	306,000
	Mount Desert Island Hospital	264,400
	Sonogee Rehabilitation & Living Center	252,700
Seasonal		
	Kebo Valley Country Club	1,666,180
	Regency Hotel	957,800
	Wonder View Inn	429,150
	Days Inn	287,520
	Bar Harbor Campground	283,123

Around the periphery of the distribution system, beginning in the south and west are two natural barriers to future growth of the water system. The first is the considerable land within Acadia National Park, and the second is the increasing elevation of potentially buildable properties. Toward the Mountain Avenue and Arata Drive areas, it is already necessary to operate booster pumps to provide sufficient water pressure. Without a formal decision to create a new consolidated high pressure zone in the distribution system, the system has essentially reached its practical extent toward the west.

The biggest potential for future growth is in the area identified in the Town's Comprehensive Plan to the north, through Hulls Cove and out to Salisbury Cove. Fortunately or unfortunately, this is an area in which the capacity of the water distribution system is already severely restricted.

Due to the long, dead-end, small diameter, older water lines north of Hulls Cove and this area's distance from multiple networked connections to the water supply transmission line, this area suffers from low fire flow capacity. The lack of year-round storage facilities in this portion of the system limits even domestic pressures in the higher elevations, and the seasonal nature of much of the piping further limits growth and development of the area.

Several system improvements will be recommended for this area. It is important to understand, however, that any improvements that are made are likely to lead to significant increases in the water demand for this area, and that this can have an overall impact on transmission lines and water treatment throughout the system.

To some extent, increases in demand may be offset by the on-going improvements to the water system that replace older piping and reduce lost water. Lost water may result from a wide range of sources, from leakage to intentional bleeders and end of line blow-offs. Pipe replacement and pipe extensions to provide better looping or networking of the system brings with it improvements in water quality that reduce the need for bleeders, as well as improvements to pipe burial and insulation that better protect the system from freeze-up and the need to operate seasonal blow-offs, as well as improvement in the reliability of the piping and a reduction in leakage. Presently, the Water Division estimates that lost water accounts for approximately 33 percent of the water that is initially treated. This fraction has been steadily reduced through the years.

With these considerations in mind, it would be prudent to consider the following average and peak daily flows when sizing treatment components for the future:

Table 3: Future Projected Flow Rates For Design

	Average Daily Flow (ADF) (MGD)	Peak Daily Flow (MGD)	Peak Instantaneous Flow Rate (GPM)
September-June	1.2	2.0	3000
July-August	2.0	4.0	6000

Flow rates and overall demands for subareas of the system and individual pipe runs are determined on a case-by-case basis through the use of a computer pipe network model that has been developed for the system. This allows for a characterization of each pipe and storage unit in the distribution system. For individual problem solving or condition assessment, water demands are placed at various locations throughout the system to simulate growth or fire fighting demand. Note is made of the paths water takes to reach that location and the constraints and bottlenecks that result. Individual water demands used in these simulations are based on estimated firefighting needs based on the types of structures encountered in the neighborhood, or on growth estimates based on lot density or proposed individual projects. This model and its use will be described in greater detail below.

2.3 DAM ISSUES

Mention should be made of the earthen dam and concrete spillway in place in the intake area of the northern shore of Eagle Lake. This dam was originally constructed by the Bar Harbor Water Company as part of its improvements to the intake area around 1895. Maintenance of the dam, or at least the spillway structure is still the responsibility of the Water Division.

Prior to 1998, the Maine Emergency Management Agency (MEMA) classified this dam as a Significant Hazard Dam. This was presumably based on the property damage and/or loss of life that could result if there was a failure of the dam. In actuality, placing the dam in this category was likely a default action taken because there was no additional information available or investigation undertaken to warrant a change in classification. Under this classification, the water company by 1998 would have been required to develop an emergency action plan and to increase monitoring, maintenance, and reporting on the structure.

In 1997, MEMA had their consultants perform an inspection of the dam. Among the observations summarized in their report at that time were the following:

- The Eagle Lake Dam is in poor to fair condition.
- There is a reduction of spillway capacity due to the permanent attachment of wooden boards to the spillway crest.
- The sluiceway is not operational.
- The concrete masonry retaining walls are in poor to fair condition, allowing free entry of water.
- There is concentrated seepage at the toe of the west dike masonry wall downstream from the pump house.

- The east and west dikes are in poor condition. This includes undercutting, benching, displacement of riprap, and overgrowth with large trees. The toe of the dikes contained large areas of wet soil and stagnant water.

- There is no Emergency Action Plan (EAP).

There was concern even before this time that the dam could become a considerable liability to the water company. This was prompted by the fact that the water company had observed that there was some saturation and associated leakage around the wing walls of the concrete spillway. The level of repairs that would be necessary, required, or permitted was not clear.

In late 1997 the water company requested that MEMA consider reclassification of the dam based upon the actual damage that could occur from a dam failure. Based on an assessment that was then conducted by the Army Corps of Engineers, MEMA agreed to reclassify the structure to the Low Hazard category. A letter to the water company confirming this reclassification was issued on January 25, 1998.

For now this issue appears not to represent an on-going concern to MEMA. However, it should be recognized that classification criteria, and policy can change over time. Such changes are often driven by crises or failures at other facilities and can result in changes for all dams within a very short period of time. Changes in classification can also result if future development of the flood zone below a dam occurs. While this is not likely, given that much of the area is park land, it may become a consideration if additional water treatment structures are considered for construction in the watershed below the dam.

This issue also serves to point out that the dam, and particularly the spillway structure will require on-going maintenance to ensure that their condition does not deteriorate further and that existing deficiencies are corrected. Although an EAP is not presently required for a Low Hazard Dam, the other issues identified should be addressed. Attention should also be paid in the future to discussions of statewide dam policy and the impact changes may have for Bar Harbor's structure.

2.4 DISTRIBUTION SYSTEM MODEL

A water distribution system network model has been prepared using Haestad Methods', WaterCAD version 4.5 software on a desktop personal computer running under the Windows XP operating system. This computer model is used to simulate flow and pressure conditions throughout the system. The model consists of pipe segments connected through nodes, and includes all major elements of the system such as pumps, pressure control valves and storage units such as tanks and reservoirs. Each pipe present in Bar Harbor's system is included in the model. Nodes can be pipe junctions and hydrants.

In order to calibrate the model, individual pipe characteristics are manipulated until hydrant flow and pressure data match actual data obtained from field tests under normal customer demands. Pipe characteristics include pipe size, material, and roughness coefficients. The coefficient used is known as the Hazen-Williams C-value, a measure of the amount of friction generated as a result of pipe wall material and the wall's condition. The customer demand is based on typical daily water use in the system and is spread between the various nodes in the model network based on the type of housing, its density, and the node spacing. Specific high use customers, industries, and other demands such as bleeders and blow-offs can be assigned to a node with an associated demand.

Once the model is sufficiently calibrated, various demands can be placed in different locations to simulate fire flow needs, line breaks, growth, and other changes. In addition, water level changes within storage units can be observed by running a model simulation over a duration varying from hours to weeks. This can give an idea of water quality by determining the age of the water within a storage tank, how often the water turns over, and the range over which the water level in the tank fluctuates.

The model is also useful for determining where bottlenecks are in the system by determining the path that water takes when flowing through the system and where the greatest headloss occurs. Since the model requires that information on each pipe be input, it also makes a convenient database for storing and displaying this information.

Figure 1, folded in the pocket at the end of this report, shows the overall system, color coded for each pipe size. This is the base map of the system and is provided for reference. Individual, smaller maps are provided within the body of the report and are used to highlight specific features or issues within the system. Note that these additional figures overlie the model network on a street map base. This is done in an effort to make locating individual pipe segments easier. In several locations the registration between the model network and the street base is relatively poor. The model network is simply a schematic drawing of how individual pipe segments are connected to one another. The large scale base map provided in Figure 1 shows the pipe network drawn directly on the street base.

Generally, when considering system problems or shortcomings, and when planning which areas might require future attention, we would choose to highlight the worst 15-20% of the system as it pertains to one particular variable. This has been done for several variables, and each will be discussed in turn.

2.5 CURRENT & FUTURE BOTTLENECKS & DEFICIENCIES

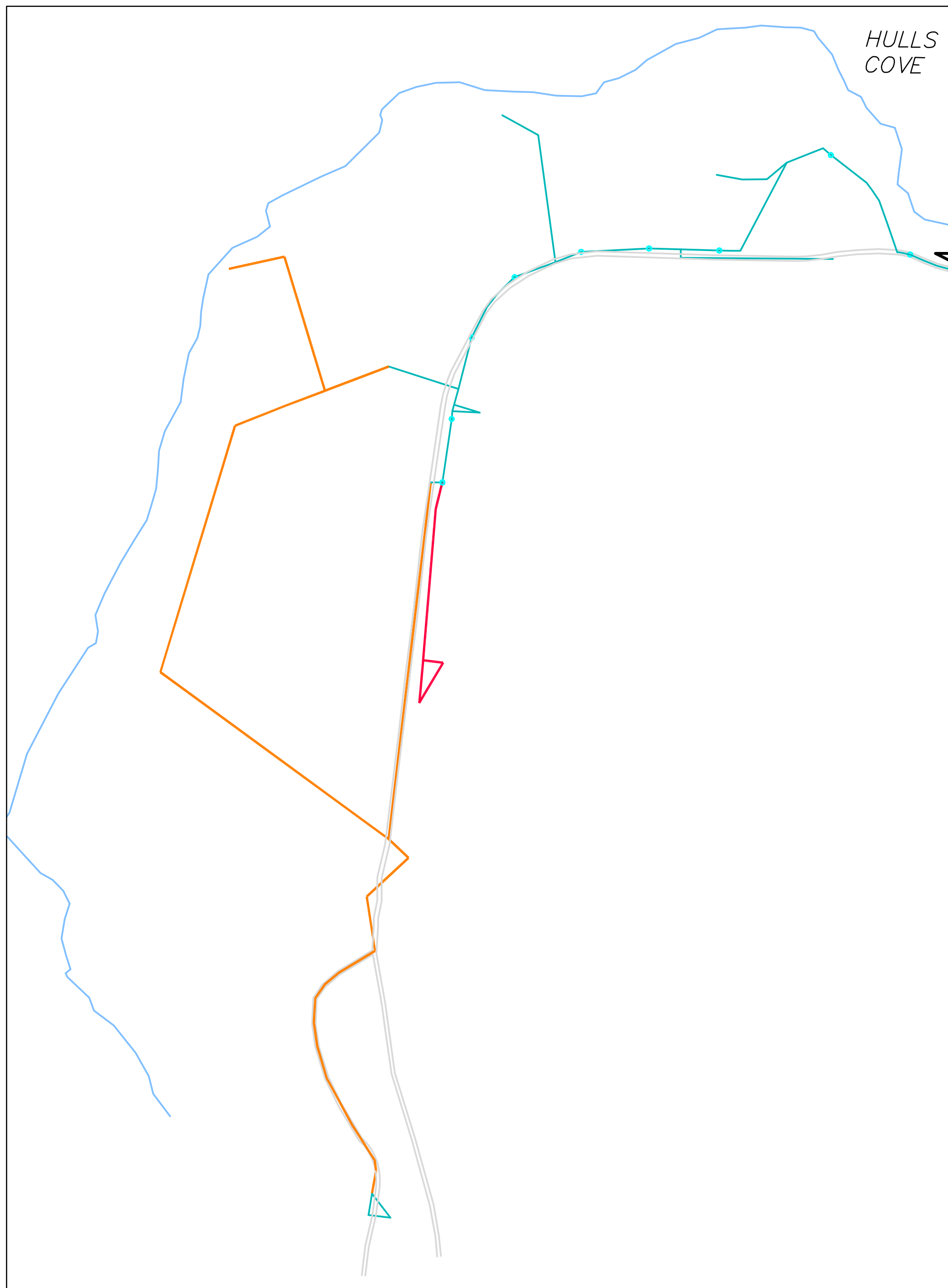
2.5.1 Seasonal System

One measure of the amount of maintenance that areas of the system might require has to do with whether the piping is buried, intended for year-round service, or is surface run, intended for summer service only. Within the seasonal piping category, there tend to be two subgroups of piping, newer high density polyethylene (HDPE), and the older galvanized and cast iron pipe. The HDPE pipe is fairly rugged, and we have assumed that places where it has been installed are places where a decision has been made to keep the system as a seasonal system for the near future.

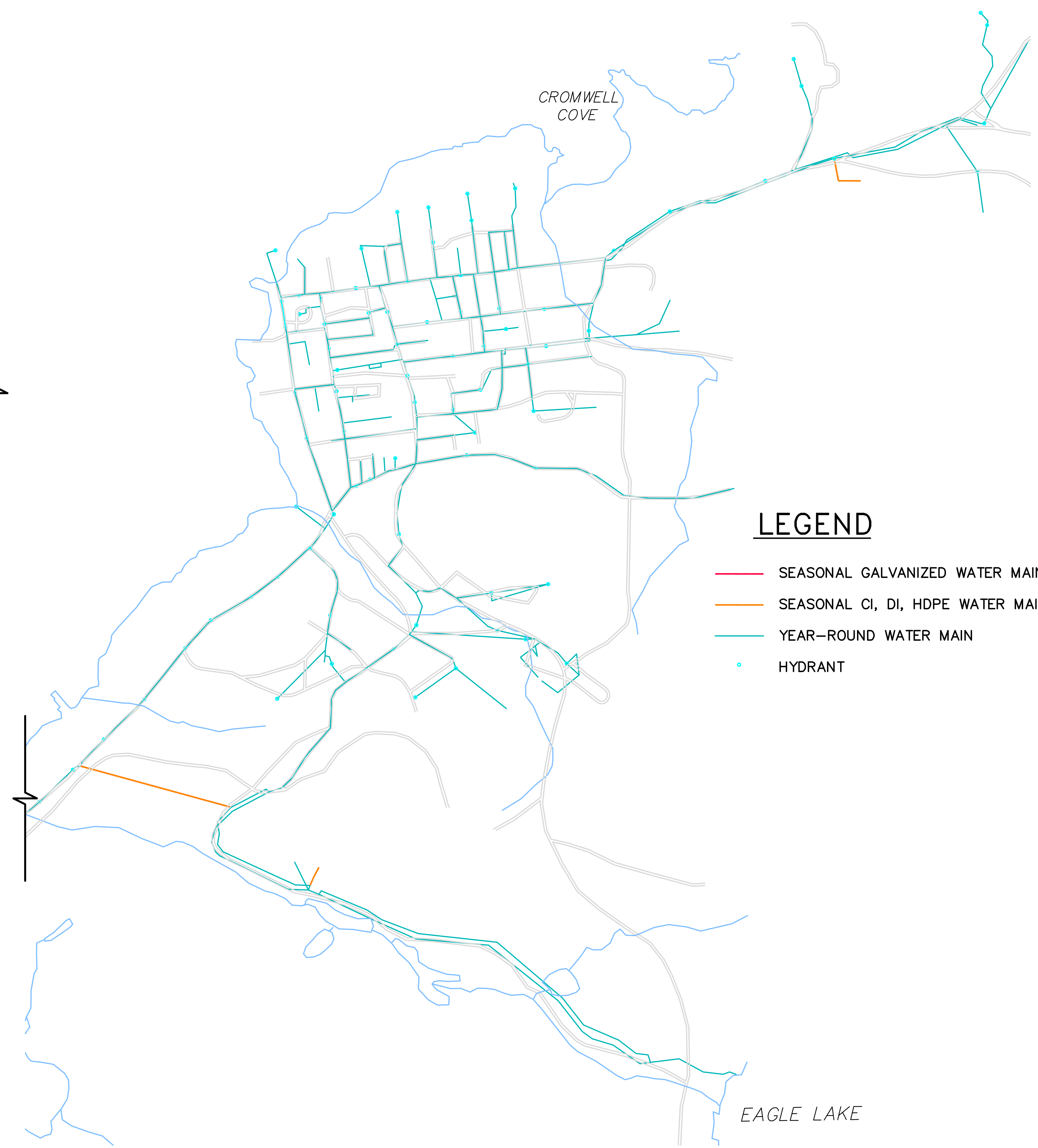
It is the galvanized lines that generally require the most maintenance, and likely have the most negative impact on water quality. Leakage, valve condition, and pipe flow capacity are all issues that are of concern on these lines. In addition, these lines likely represent lines that have not been upgraded recently and would be due for some type of planning or consideration.

Figure 2 highlights the seasonal piping within Bar Harbor's system, and further indicates in red the seasonal galvanized piping. Of the 29 miles of piping in the system, slightly more than 3 miles (11%) of pipe is part of the seasonal system. Of this approximately 6800 feet consists of 2 and 3-inch galvanized pipe. Galvanized pipe replacement should be considered as an area of focus for system upgrades. This would also be the time to consider whether it would be prudent to convert these pipe runs to buried, year-round service if they can be tied into other year-round lines.

As Figure 2 shows, much of the seasonal system is clustered around Hulls Cove north of Lookout Point Road and out to Salisbury Cove. This has been identified as an area for significant future growth and consideration should be given to providing year-round water to this area. Other areas of seasonal piping



HULLS COVE AREA



LEGEND

- SEASONAL GALVANIZED WATER MAIN
- SEASONAL CI, DI, HDPE WATER MAIN
- YEAR–ROUND WATER MAIN
- HYDRANT

WOODARD & CURRAN
Engineering · Science · Operations
BANGOR, MAINE
800-564-2333

BAR HARBOR WATER SYSTEM
SEASONAL USE PIPE (SURFACE)

DESIGNED BY:	TEN/RHH
DRAWN BY:	NTD
CHECKED BY:	RHH
20321501-U002-RPT.dwg	

TOWN OF BAR HARBOR
WATER DIVISION

WATER SYSTEM MASTER PLAN

JOB NO:	203215.01
DATE:	APRIL 2005
SCALE:	1"=1500'

FIGURE 2

include the East Strawberry Hill Road area, the Town Pier, and the 12-inch transmission line along Duck Brook.

2.5.2 Small Size Pipe

Small diameter pipe is not necessarily undersized pipe. However, piping less than 6-inches in diameter is rarely installed these days, and this has been the case for some time. The cost of pipe material is generally a small fraction of the cost of a pipe installation project, and the modern need to provide sufficient fire flows almost always dictates that in-ground piping be at least 6-inches in diameter. For this reason, small diameter piping is probably indicative of older pipe and, for in-town year round service areas, is most likely indicative of unlined cast iron or galvanized pipe. These pipe types are most likely to contribute to diminished water quality through tuberculation (leading to red or brown water), to leaching of metals to the water, and to increasing the chlorine demand of the system (loss of residual).

Within Bar Harbor's water system there are approximately 18,220 feet of buried pipe that is 4-inch diameter or less. This represents approximately 12% of all pipe in the system. This quantity does not include the small diameter pipe that is part of the seasonal system.

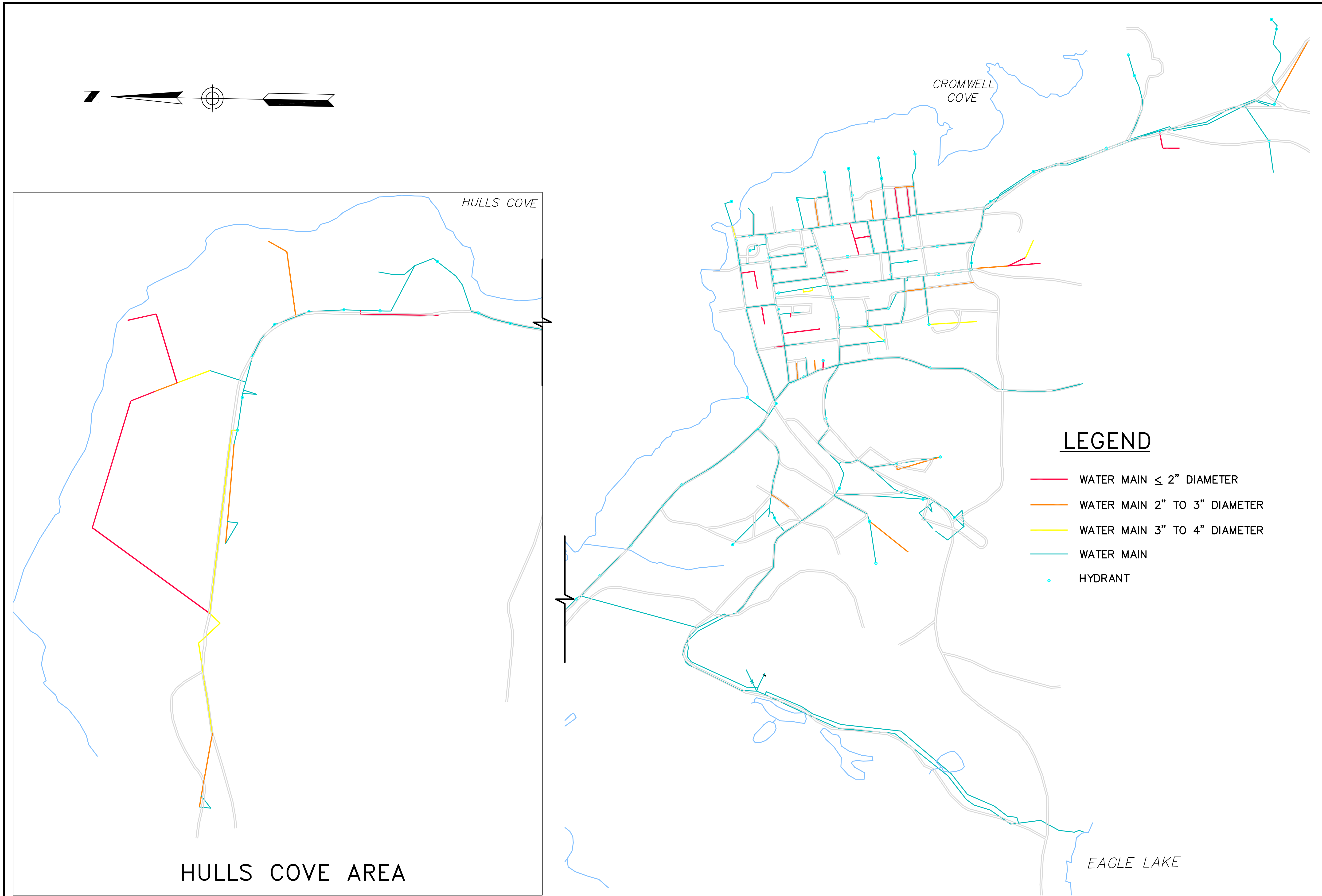
Figure 3 highlights all pipe that is 4 inches or less in diameter. It includes small pipe within the seasonal system. As this figure indicates, small diameter piping is located throughout the downtown area. Some of the smallest pipe (2-inch and under) is located on dead-end runs between Cottage Street and West Street, in the Newton Way area, and on Oliver and Center Streets. This pipe is also some of the oldest in the system.

2.5.3 Poor Condition Pipe

The above two conditions are perhaps more indicative of pipe age rather than actual pipe condition, although they likely reflect pipe condition as well. They do, however, focus on the small piping that tends to be on the margins of the system or on local side streets that are not major flow paths for water moving through the system. The next variable presented is a direct measure of actual pipe condition for major trunk lines within the system.

The Hazen-Williams C-value is a coefficient used to reflect pipe wall condition when computing friction generated by water flowing through a pipe. Depending on the pipe material, C-values of 100-130 are generally used for design for new pipes in good condition. During the model calibration process, C-values are adjusted for individual pipes until the flows and pressures actually observed are reflected in the model output. In most cases the C-value is lowered for older pipe (reflecting poorer condition) until the model output is valid over a wide area and range of flow conditions. Attention to C-value manipulation is generally only paid to the major transmission lines that move water from one neighborhood to another. Changes in C-value made to small, local lines usually do not have much effect on the overall system and are typically not made. For this reason, small diameter lines that may be in poor condition are not necessarily highlighted on a plot showing low C-values.

Figure 4 highlights major transmission pipes with C-values lower than 70, 60, and 40. These are likely to represent major pipes in the worst condition in the entire system. Note that only older pipes in the 6-inch and 10-inch sizes were found to have particularly low C-values. The breakdown of pipe footage in these low classes is as follows:



WOODARD & CURRAN
Engineering · Science · Operations
BANGOR, MAINE
800-564-2333

BAR HARBOR WATER SYSTEM
SMALL DIAMETER WATER LINES

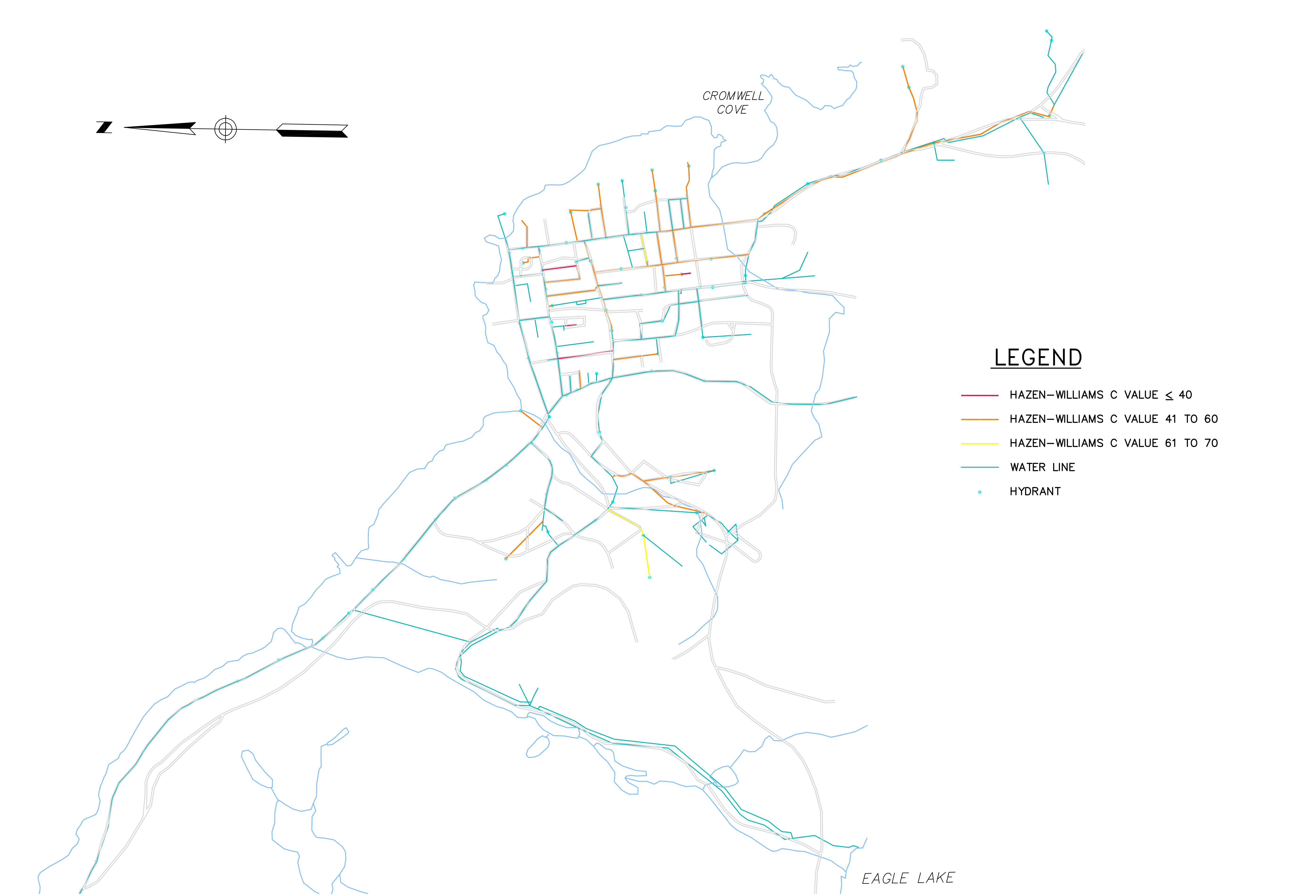
DESIGNED BY: TEN/RHH
DRAWN BY: NTD
CHECKED BY: RHH
20321501-U003-RPT.dwg

TOWN OF BAR HARBOR
WATER DIVISION

WATER SYSTEM MASTER PLAN

JOB NO: 203215.01
DATE: APRIL 2005
SCALE: 1"=1500'

FIGURE 3



WOODARD & CURRAN
Engineering · Science · Operations
BANGOR, MAINE
800-564-2333

BAR HARBOR WATER SYSTEM
LARGE (≥6" DIA.) WATER LINES
IN POOR CONDITION

DESIGNED BY: TEN/RHH
DRAWN BY: NTD
CHECKED BY: RHH
20321501-U004-RPT.dwg

TOWN OF BAR HARBOR
WATER DIVISION

WATER SYSTEM MASTER PLAN

JOB NO: 203215.01
DATE: APRIL 2005
SCALE: 1"=1500'

FIGURE 4

Table 4: Low C-Value Transmission Line Pipe Footage

C-Value	10-Inch Pipe	6-Inch Pipe
70	216	2441
60	1512	25551
40	0	1929
Total	3729	29921

Table 4 indicates that approximately 33,650 feet of significant pipe in the system is suspected of having a poor wall quality. This represents approximately 23% of all pipe in the system. When considering only 6-inch pipe, it represents almost three-quarters of the year-round 6-inch piping in the system, much of which is unlined. For piping larger than 6-inch, the likely newer, cement lined pipe, it represents only 2% of the year-round system pipe.

Figure 4 highlights the fact that this pipe, particularly the 6-inch pipe, is located throughout the downtown area. Frequently, piping in this category is replaced when local roads are upgraded or reconstructed as part of the normal rotation of pavement and sewer maintenance. Mountain Avenue (2002), Spring Street (2003) and Roberts Avenue (2004) are examples of street reconstruction projects that also included the replacement of old water main. The remaining highlighted lines should be considered for replacement as their streets come up for reconstruction.

The Devon/Woodbury Road area piping was found to require a particularly low C-value setting. This may be due to poor pipe condition, but may also be an artifact of the model. C-values are also used to lower flow rates through parts of the system where hydrant elevations are not accurately known. Very low fire flows have been observed on Devon Road, and may be partially due to the elevation of the hydrants. However, other hydrants throughout the system located on 6-inch lines with similar static pressures are found to produce upwards of three times the fire flow while retaining significantly higher residual pressures. This would tend to suggest that the poor condition of the pipe plays at least as large a role as the elevation of this area of the system.

2.5.4 Bottleneck Pipes

Poor wall condition, leading to a low C-value, is certainly indicative of poor condition pipes, and likely has an impact on overall water quality, however, it may not be limiting to the overall distribution system. A low C-value pipe may not ordinarily need to pass a significant flow, or there may be alternative flow paths around it during periods of high flow such as would be experienced during a fire demand.

An important measure of which individual pipes are limiting to the system, those that are bottlenecks to efficient flow distribution, is the headloss gradient that is seen along an individual pipe. This measure takes into account both the condition of the pipe wall and the amount of flow that ordinarily would have to flow through the pipe. It also removes from consideration the overall length of the pipe by being expressed in units of feet of headloss/thousand feet of pipe.

Figure 5 highlights those pipes with the highest headloss gradients, broken down into three individual groupings. The headloss gradients are determined under an assumed system demand of 3 MGD, a peak summer day. In total, the pipes highlighted represent approximately 21% of the overall piping in the system. Pipe sizes for these problem sections range from 4-inch to 16-inch. The worst pipe segments of this group (red) should be considered as the limiting bottlenecks to the system and should be the first pipe runs considered for replacement. They represent approximately 5% of the overall system piping.

An individual list of these pipe segments is presented below, in Table 5.

Table 5: Piping Bottlenecks In Bar Harbor's Water Distribution System

Rank	Street Name, Bounds	Length (feet)	Pipe Size (inches)	Headloss Gradient (ft/1000ft)
1	Main Street (Cromwell Harbor Road to Schooner Head Road)	2,745	6	10
2	Along Route 3 towards Hulls Cove from Summer Duck Brook Transmission line to just beyond Crooked Road	2,822	8	5-8
3	Main Street (Second Street to Oliver Street)	401	6	5.7
4	Mt Desert Street (from Kebo Street to Spring Street)	596	10	6-7
5	Cromwell Harbor Road (from Glen Mary Road towards Ledgelawn)	96	10	5

2.5.5 System Pressures and Flow Limitations

Beyond looking at individual pipe elements, the principal purpose of a system hydraulic model is to assess how the overall system responds to various stresses placed on it. Generally, these can take the form of three types of events: average daily demand throughout the system; peak demand (whether from a hot dry summer day when many seasonal users are present, or from a cold winter day when bleeders are active to protect the system from freeze-up, or from flows throughout the system during the semi-annual system maintenance flushing); and fire demand placed on an individual or group of hydrants by fire department pump trucks used during fire fighting.

For modeling purposes, we considered the average daily demand (average daily flow or ADF) to be 1 MGD, just slightly more than the ADF seen through ten months of the year, and slightly less than the ADF over the entire year, including summer months. The peak daily demand was considered to be 3 MGD, just slightly greater than current peaks seen in July and August.

Modeling fire flows is somewhat more difficult to present. A fire demand on the order of 1200-2500 gpm could be applied to any individual hydrant in the system. A better means of determining the available fire flow in various parts of the system is to actually flow hydrants in the field and record the flow available while a residual pressure of 20 psi remains available at an adjacent hydrant. This has been recently completed for each hydrant in the system by an independent contractor working for the Water Division.

The results are presented below in Table 6. Hydrant numbers are keyed to the full-scale system map. Note that in some cases the measured flows at various hydrants differ significantly from those computed

WOODARD & CURRAN
Engineering · Science · Operations
800-564-2333
BANGOR, MAINE

REV	DESCRIPTION	DATE

DESIGNED BY: TEN/RHH
CHECKED BY: RHH
DRAWN BY: NTD
FILE: 20321501-U005-RPT

BAR HARBOR WATER SYSTEM HIGH HEADLOSS GRADIENTS

TOWN OF BAR HARBOR WATER DIVISION

WATER SYSTEM MASTER PLAN

JOB NO.: 203215.01
DATE: APRIL 2005
SCALE: 1"=1500'
SHEET: OF

FIGURE 5

by the model. Many of the measured flows were not taken while maintaining a system residual pressure of 20 psi. Others may have incorrect information regarding the hydrant elevation. In a few instances the hydrant static pressure added to the listed hydrant elevation equals a hydraulic grade line higher than the initial Eagle Lake water level. On a single zone gravity system this would never be the case.

Table 6: Fire Hydrant Hydraulic Analysis (April 2004)

				Pressure		Pitot		
Hydrant #	Location	Main Size (inches)	Elev Hyd. (ft above Sea Level)	Static (psi)	Residual (psi)	Diameter (inches)	Pressure (psi)	Flow (gpm)
1	Albert Meadow	6	45.25	88	75	4	10	1360
2	Amory Lane	6	67.57	79	75	4	5	960
3	Ash Street	6	34.28	83	75	4	2	615
4	Atlantic Ave	6	40.76	97	58	4 5/8	5	1220
5	Backyard- York St.	6	NA	95	65	4	15	1665
6	Cleftstone Rd. & Devon Rd.	6	NA	55	17	4	2	615
7	Cleftstone Rd. & Howes Park	16	NA	52	44	4	24	2110
8	Cleftstone Rd. & West St. Ext.	12	156.94	52	44	4 5/8	25	2770
8	School Street & Ball Park	6	NA	98	82	4 1/2	3	940
9	Cottage & Eden Street	10	47.40	96	82	4	16	1720
10	Cottage & Greeley Ave.	8	52.91	96	74	4 1/4	16	1950
11	Cottage & Holland Ave.	6	45.91	96	82	4 1/2	17	2235
12	Cottage St. & Federal St.	10	53.54	93	69	4 5/8	32	3080
13	Cottage St. & Main St.	8	54.79	97	60	4 1/2	32	3080
14*	Cottage St. & Roberts Ave.	6	NA			4		
15	Cottage St. & Rodick St.	8	48.57	92	52	4	24	2110
16	Derby Lane	8	NA	95	72	4 5/8	7	1440
17	Devon Road & West St. Ext.	12	NA	50	33	4 5/8	19	2365
18	Devon Road (Cunningham)	6	142.41	63	60	4	3	740
19	Eagle Lake Rd. & Cromwell Hbr Rd.	6	200.30	58	56	4	2	615
20	Eagle Lake Rd. & Cross St.	12	116.68	67	58	4 5/8	22	2550
21	Eagle Lake Rd. & Woodbury	8	NA	60	36	4	19	1870
22	Eagle Lake Rd. & Woodbury Rd.	6	134.17	57	25	4	18	1820
23	Eden St. & Atlantic Oaks	10	75.51	87	61	4 5/8	22	2550
24	Eden St. & Ferry Terminal	10	64.23	85	58	4 1/2	31	3080
25	Eden St. & Highbrook Rd.	10	70.87	91	61	4	20	1920
26	Eden St. & Mt. Desert St.	10	8410.00	79	59	4 5/8	19	2365
27	Eden St. & Myrtle Ave	10	61.61	91	75	4	25	2190
28	Eden St. & West St.	10	37.68	96	74	4	32	2430
29	Eden St. (Bob Collier)	8	64.12	90	40	4	14	1610
30	Eden St. (Davis Center)	10	62.32	88	60	4	27	2280
31	Eden St. (Duckbrook)	8	35.28	97	69	4	20	1920
32	Eden St. (Helmut Weber)	10	67.49	89	80	4	27	2280
33	Eden St. (Nat. Park Ent.)	8	62.81	88	62	4 5/8	11	1800
34	Eden St. (Regency)	8	49.51	91	67	4	27	2280
35	Eden St. (Sonogee)	8	62.94	102	69	4 1/2	22	2550

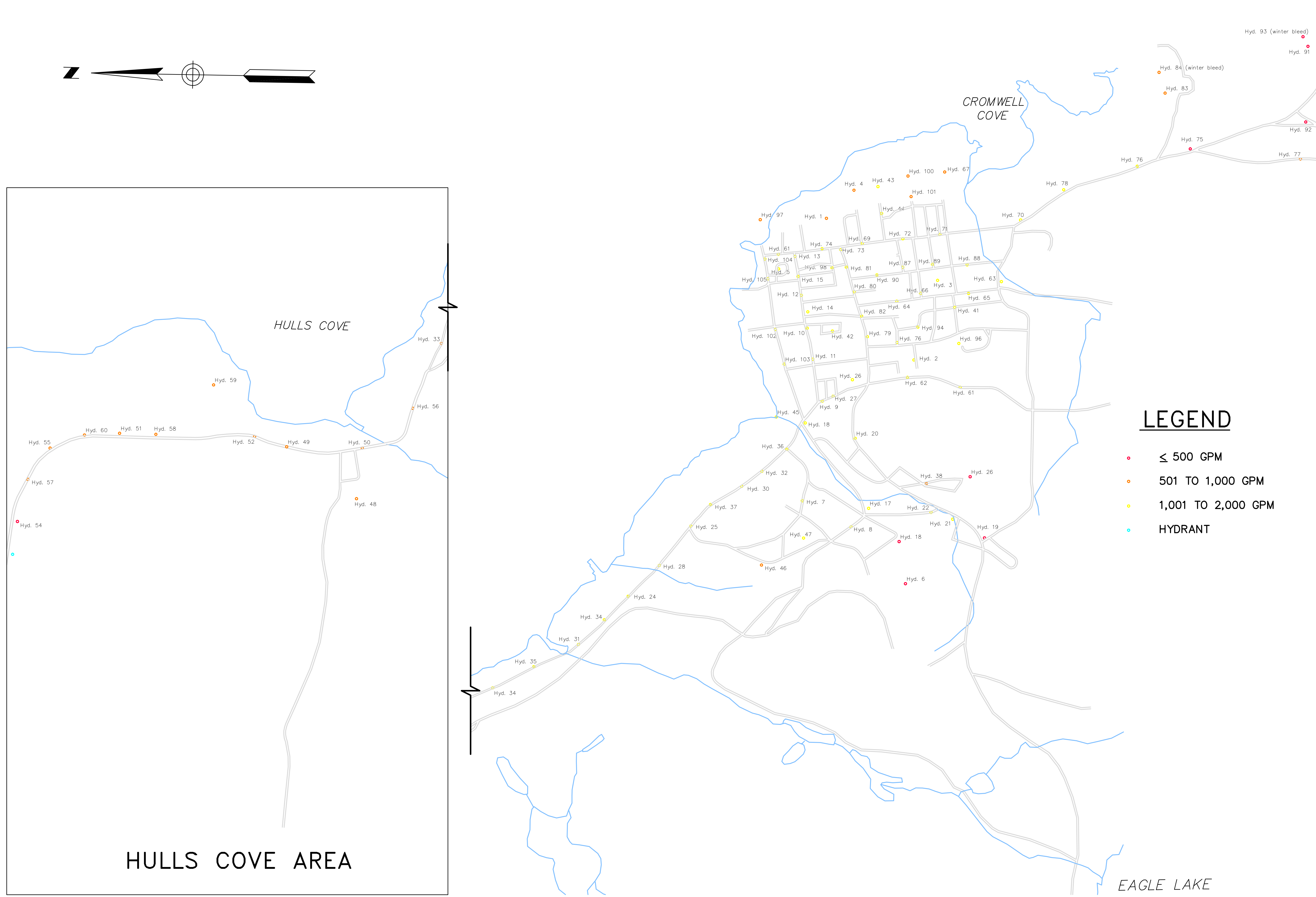
				Pressure		Pitot		
Hydrant #	Location	Main Size (inches)	Elev Hyd. (ft above Sea Level)	Static (psi)	Residual (psi)	Diameter (inches)	Pressure (psi)	Flow (gpm)
36	Eden St. (Squeak Sawyer)	10	65.44	104	93	4 1/2	31	3080
37	Eden St. (Turetts)	10	58.40	90	52	4	29	2350
38	Forrest St. & Pine Street	6	72.94	91	65	4 1/2	13	1960
39	Forrest St. & Pine St.	6	135.64	57	25	4 1/2	1	550
40	Forrest St. (End)	6	141.79	55	25	4 1/2	2	775
41	Glenmary Rd. & Park St. Entrance	6	45.54	95	90	4	5	960
42	Greely Ave.	6	66.77	93	89	4 1/2	2	775
43	Hancock St. (End)	6	33.31	92	58	4 5/8	3	775
44	Hancock St. (Hospital)	6	49.58	95	85	4 5/8	3	940
45	Harbor Lane	6	35.14	102	DE	4 1/2	4	1100
46	Highbrook Rd. (Howes Park)	6	NA	52	47	4 1/2	9	1640
47	Highbrook Rd. (Old Greenhouse)	16	156.11	52	44	4	18	1820
48	Hulls Cove (Crooked Rd.)	6	50.54	108	90	4	3	740
49	Hulls Cove (Frazier)	8	19.64	108	45	4 5/8	12	1890
50	Hulls Cove (Gen Store)	8	22.26	88	22	4	11	1420
51	Hulls Cove (Goodwin)	8	116.44	71	13	4 5/8	5	1220
52*	Hulls Cove (Hamor Lane)	8	33.47			4 1/2		
53	Hulls Cove (Hinkley)	8	167.68	63	21	4	3	740
54	Hulls Cove (Hutchins)	8	124.28	68	18	4	5	960
55	Hulls Cove (Milliken)	8	111.41	75	20	4	3	740
56	Hulls Cove (Ocean Ave.)	8	NA	89	25	4	9	1290
57	Hulls Cove (Pot & Kettle)	8	104.55	68	14	4	5	960
58	Hulls Cove (Stetson Carter)	8	112.23	90	35	4	5	960
59	Hulls Cove (Syndicate Rd.)	8	65.06	108	50	4 1/2	10	1730
60	Hulls Cove (Walls)	8	95.87	70	2	4	6	1050
61	Kebo St. (Hadley)	8	NA	44	16	4 5/8	6	1340
62	Kebo St. (Malvern Belmont)	8	96.76	79	70	4 5/8	11	1800
63	Ledgelawn Ave. & Cromwell Hbr	10	40.85	94	57	4 1/2	24	2660
64	Ledgelawn Ave. (Lambert)	8	52.76	96	66	4	22	2020
65	Ledgelawn Ave. (Porier)	8	NA	96	55	4	20	1920
66	Ledgelawn Ave. & Pleasant St.	8	46.36	96	81	4 1/2	17	2235
67	Livingston Rd.	6	43.31	100	92	4 1/2	1	550
68	Main St. (Life Sports)	10	46.79	107	65	4 5/8	18	2310
69	Main Street & Atlantic Ave.	8	51.19	90	65	4	13	1550
70*	Main Street & Cromwell Hrb. Rd.	6	34.99			4 5/8		
71	Main Street & Edgewood Street	6	35.74	98	65	4 5/8	7	1440
72	Main Street & Hancock Street	8	51.19	90	65	4 5/8	15	2105
73	Main Street & Mt. Desert Street	10	58.38	98	61	4 5/8	29	2980
74	Main Street & Rodick Place	10	65.38	95	67	4 5/8	28	2880
75	Main Street & Schooner Head	6	100.19	75	75	4 1/2	1	550
76	Main Street (Ogden)	8	46.4	98	95	4 5/8	41	3520
77	Main Street (Rt. 3 Jackson Lab)	10	NA	95	86	4	26	2190
78	Main Street (St. Andrews)	6	41.47	100	98	4 5/8	3	940
79	Mt. Desert Street	10	75.06	79	63	4 5/8	35	3260

				Pressure		Pitot		
Hydrant #	Location	Main Size (inches)	Elev Hyd. (ft above Sea Level)	Static (psi)	Residual (psi)	Diameter (inches)	Pressure (psi)	Flow (gpm)
80	Mt. Desert Street & High Street	10	71.86	85	63	4 1/2	25	2770
81	Mt. Desert Street & Kennebec St.	10	64.42	83	57	4	19	1870
82	Mt. Desert Street (Anchorage)	10	72.93	83	59	4	24	2110
83	Old Farm Road	6	83.19	97	97	4 1/2	4	1100
84	Old Farm Road (End)	6	NA	75	16	4 1/2	1	550
85	Old Police Station	6	NA	92	62	4 1/2	20	2430
86	Sand Point Road	6	NA	69	16	4 1/2	4	1100
87	School Street & 1st South St.	6	46.05	94	60	4 1/2	4	1100
89	School Street & Edgewood	6	37.45	96	69	4 1/2	7	1440
90	School Street & Newton Way	6	50.32	83	75	41/2	3	940
91	Schooner Head Rd. & Seely	6	NA	69	0	4 1/2	1	550
92	Schooner Head Rd. Jackson Lab	6	NA	71	70	4 1/2	1	550
93	Seely Road	6	NA	85	2	4 1/2	5	1220
94	Shannon Road	10	NA	89	57	4 1/2	41	3520
95	Shannon Road & Spring St.	10	NA	83	54	4	29	2350
96	Spring Street & Norris Ave.	6	61.12	95	91	4	4	870
97	Town Pier	6	NA	107	60	4 1/2	16	2180
98	Upper Rodick Street	6	NA	92	41	4 1/2	25	2770
99	Waldron Rd.(Glen Mary Pool)	6	NA	97	60	4 1/2	17	2235
100	Wayman Lane (End)	6	38.42	98	49	4 1/2	2	775
101	Wayman Lane (Hospital)	6	41.84	96	89	4	3	740
102	West Street & Bridge Street	10	48.91	100	73	4	19	1870
103	West Street & Holland Ave.	10	39.39	102	82	4	26	2190
104	West Street & Main Street	8	23.13	107	75	4 5/8	25	2770
105	West Street & Rodick Street	8	22.26	96	72	4 5/8	18	2310

*out of service

Hydrant fire flows have been color coded by flow range in Figure 6. This shows that pipe constraints can be offset somewhat by low elevations, so that even some hydrants along the system margins can have moderate flows if they are close to sea level.

Fire flows are known to be low in the Hulls Cove area and beyond, due to the long run of 8-inch pipe north of the juncture of the 12-inch HDPE surface run seasonal line. The lowest flow hydrants in the area are at the end of the year-round piping near the Gerrish pump station serving the Dreamwood Hill summer tank. Here flows of 250-500 gpm at residual pressures of 20 psi are experienced. Once problem areas such as this are identified, pipe changes in the model can be made in order to determine the impact on the fire flow. By changing the 16,650 feet of 8-inch pipe to 12-inch pipe, fire flows to Hydrant 57 are improved to 500 gpm. Typical results show an increase in available fire flows of 1.5 to 2 times greater than those modeled under existing conditions throughout the Hulls Cove area.



WOODARD & CURRAN
Engineering · Science · Operations
BANGOR, MAINE
800-564-2333

BAR HARBOR WATER SYSTEM
AVAILABLE FIRE FLOWS

DESIGNED BY: TEN/RHH
DRAWN BY: NTD
CHECKED BY: RHH
203215.01-U006-RPT.dwg

TOWN OF BAR HARBOR
WATER DIVISION

WATER SYSTEM MASTER PLAN

JOB NO: 203215.01
DATE: APRIL 2005
SCALE: 1"=1500'

FIGURE 6

Alternatively, the 8-inch feed line could remain in place and the Dreamwood Hill standpipe could be upsized to provide for adequate storage to support fire fighting in the area. This would necessitate increasing the feed line size from the Gerrish booster station up to the tank. This is presently 1500 feet of 3-inch surface run galvanized seasonal line. This change could provide 300,000 to 500,000 gallons of water in storage and deliver it to Hydrant 57 at rates of approximately 1,350 gpm, a significantly better improvement requiring significantly less pipe replacement.

Under ADF conditions of 1 MGD, certain areas of the system see relatively low pressures. The ADF model case assumes that the system is operating under its normal Eagle Lake grade line (276.5 feet) with the in-ground 500,000 concrete storage tank on line. Since ADF conditions generally reflect water being consumed for domestic purposes, rather than major leaks, fire demand, or flushing, water velocities are generally low throughout the system. Low pressures, therefore are generally reflective of high ground elevations rather than due to pipe constrictions. If domestic pressures can fall to less than 35 psi, booster stations should be considered.

When operating at the Eagle Lake 276.5 gradient under the 1 MGD ADF condition, all hydrants and nodes within the system were found to be able to maintain pressures in excess of 40 psi. Under peak flow conditions (3 MGD) it is assumed that the Great Hill summer tank has been brought on line and that the pressure reducing valve at the Duck Brook pump station is being manipulated to maintain a system pressure of 100 psi at the water division office on Main Street. This has the effect of making the system respond the same way it would if it were seeing a 1 MGD demand while operating at the lake level gradient. As a result, the same pressures are sustained system-wide for this condition as well.

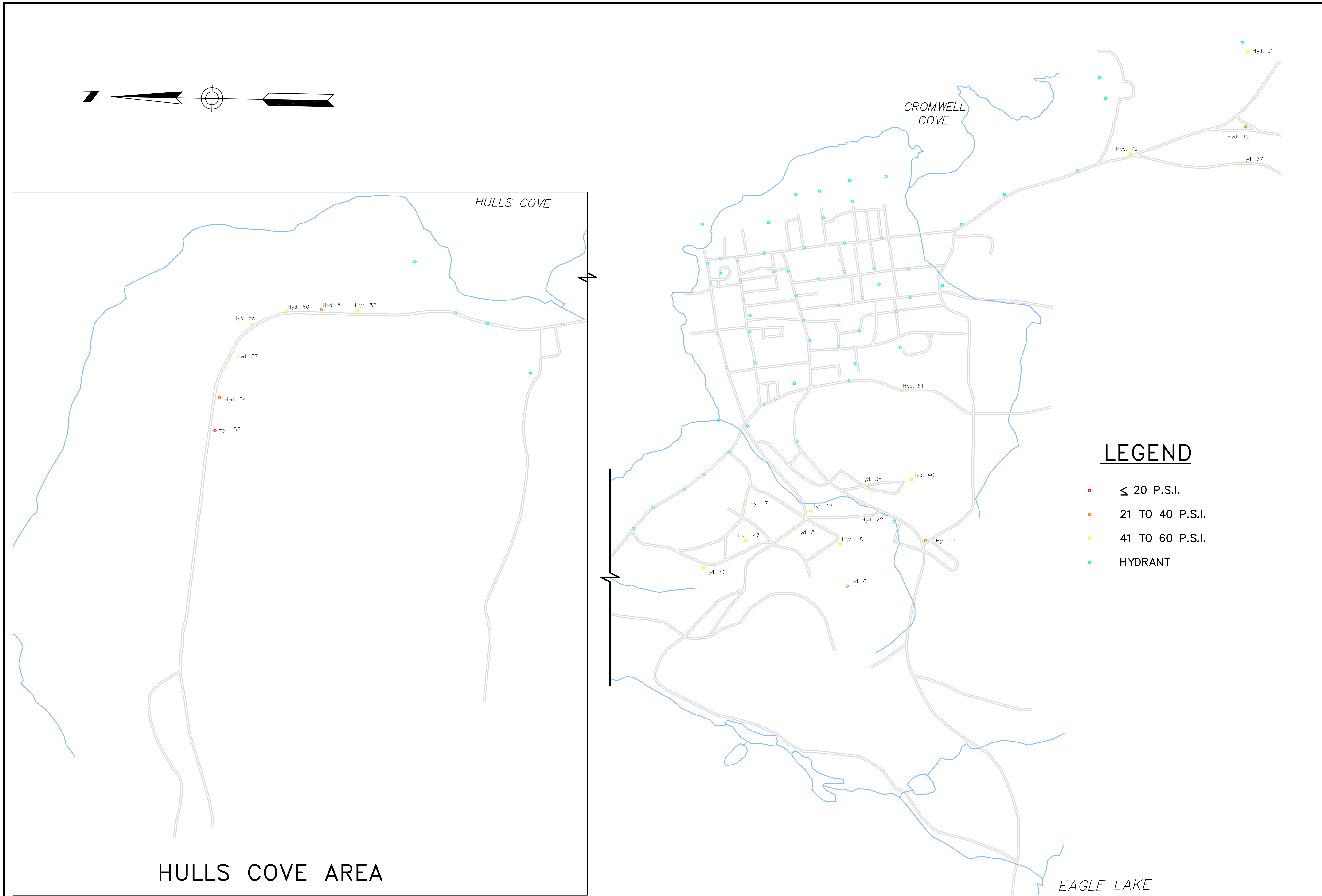
If system demand reaches such an extent that the 100 psi pressure cannot be maintained at the water division office by fully opening the pressure reducing valve, pressures will begin to drop at the higher system elevations. Should the office pressure drop to 88 psi, the lowest pressure experienced historically, Figure 7 shows the hydrant pressures that would result. The higher elevations of the system drop to 40-60 psi, with lower pressures experienced first on the north side of Hulls Cove.

One word of caution regarding this analysis, for all the cases examined by the model, the system demands were spread uniformly over the system. If there are concentrated demands in certain parts of the system, particularly at the higher elevations or at the fringes of the distribution system, there can be a much more significant pressure drop in those areas. This can be a significant impact for users such as the golf course (higher elevation), for commercial users in the Hulls Cove area, and for The Jackson Lab.

High system pressures can also be problematic, leading to excessive leakage and fixture failure. In some isolated areas, individual homes can be fitted with pressure reducing valves. Widespread pressures exceeding 100 psi should be avoided, however.

2.6 STORAGE TANK OPERATION

When considering the operation of storage tanks within the system, the focus will be on the 500,000 gallon tank at the southern end of the system across from The Jackson Laboratory (TJL), and on an as yet unconstructed tank on Dreamwood Hill at the northern end of the system. A new north-end tank was considered since the existing 50,000 gallon seasonal tank is inadequate for a number of reasons, and should be considered as one of the top improvement projects in the coming years. In addition, it is difficult to model the seasonal tank's present operation since there is no detailed flow information for this area and the outputs of pumps at the Gerrish site and at the outlet of the tank itself are not well known. The tank operation itself has been unreliable. It was recently discovered that a knot from the wood



WOODARD & CURRAN
Engineering · Science · Operations
800-564-2333
BANGOR, MAINE

BAR HARBOR WATER SYSTEM
LOW PRESSURE @ 3MGD DEMAND
SUPPLIED AT EAGLE LAKE GRADIENT

DESIGNED BY: TEN/RHH
DRAWN BY: NTD
CHECKED BY: RHH
20321501-U007-RPT.dwg

TOWN OF BAR HARBOR
WATER DIVISION

WATER SYSTEM MASTER PLAN

JOB NO: 203215.01
DATE: APRIL 2005
SCALE: 1"=1500'

FIGURE 7

decking which supported the tank roof had come free and had jammed the tank's check valve in the open position, preventing the tank from filling properly. It is not known how long this condition had been present.

The Great Hill summer tank and its companion below-grade gravity tank will not be covered in this section since they have been dealt with under general system operation and operational changes.

2.6.1 The Jackson Laboratory Tank

The TJL tank is at the end of a long line out of the main distribution area along South Main Street/Route 3. This line consists of 561 feet of 10-inch line along Cromwell Harbor Road, then 2,744 feet of 8-inch line along South Main Street to just beyond Old Farm Road, and then an additional 3,301 feet of 10-inch line out to TJL and the tank. The tank serves the domestic, process, and fire suppression needs of TJL. The 10-inch line from the tank presently enters the TJL campus through the lab's main fire pump building. Pressure is boosted for both fire and domestic flows. Only a portion of the overall campus is served by this system.

In addition, a 6-inch line along South Main Street continues out of town along South Main/Route 3 to Schooner Head Road. It serves part of the Jackson Lab campus and then continues down Schooner Head Road to Seely Road where it ends in a year-round bleeder. Further along Schooner Head Road, the line changes to a year-round 3-inch galvanized line and then to a private, seasonal, 2-inch plastic line which serves the High Seas property.

The 10/8-inch Cromwell Harbor Road and the 6-inch South Main Street lines interconnect at several locations. At the Cromwell Harbor Road/South Main Street intersection, the two lines are interconnected with a length of 8-inch pipe. At the intersection of Route 3 with Old Farm Road, the two lines are again interconnected. A 6-inch line serving Old Farm Road is tapped into the 6-inch line, although it receives water from both the 10/8-inch line and the 6-inch line. Further south along Route 3, the two lines physically cross each other, although it is not known if they are actually interconnected at that point.

The 6-inch system enters the TJL campus and passes through a 3-inch booster pump station. During the winter months, inlet pressure is approximately 50 psi. During the summer, there is a significant inlet pressure drop due to additional demand on the 6-inch line from Seely and Schooner Head Road customers. During both the winter and summer the lab boosts pressure from this line. The boosted 6-inch system is not interconnected with the boosted 10-inch system, however.

The Lab presently has plans to tie all its internal piping into the incoming 10-inch line and eliminate the 6-inch service and its booster station. This would have a number of beneficial effects. It would boost the volume of water drawn through the storage tank, improving turnover and further reducing water age in the tank. During the first quarter of 2004, the Lab consumed a total of 1,240,000 cubic feet of water. Of this total, 436,400 cubic feet, 35% of the total, came through the 6-inch service. This is water that would generally have come down the 6-inch transmission line from South Main Street, quite possibly not having passed through the storage tank. Feeding from the 10-inch line directly would ensure that additional water moved toward the storage tank.

The benefit for the 6-inch transmission line is that without the Lab demand, higher pressure would be available for residential users along Old Farm, Seely and Schooner Head Roads.

The storage tank is connected by a single inlet/outlet line to the 10-inch transmission line at the Lab campus. The tank level is controlled by an altitude valve, which is set to close just before the tank overflows at an elevation of 265 feet. There are no electronic controls or data collection and transmission systems associated with this tank. To determine the water level within the tank, Water Division staff climb the tank access ladder and feel the outside surface of the tank with their hands to determine a temperature or condensation change. Water Division staff report that there are few if any problems with the tank filling and turning over regularly during the winter months when the system is operated at the Eagle Lake hydraulic gradient.

During the high use summer months, staff report that there can be problems filling the tank if the tank level starts out low in the early morning. Under this situation, flow to the tank is improved by increasing the pressure slightly at the Duck Brook pump house pressure reducing valve. As with other system adjustments, the amount of the adjustment is monitored using the pressure indicated at the Water Division office. Through experience, staff have determined that the tank altitude valve will be open whenever normal system conditions lead to less than 102 psi at the Water Division office.

When an extended period simulation of the model is run, it is observed that under peak flow summer conditions, assuming that system pressure cannot be maintained at 102 psi at the Water Division office, water age in the tank increases significantly. The ability to fill the tank during peak system demand periods is greatly reduced, and it may not be possible to completely fill the tank, even after several days. In order to improve turnover in the tank under these conditions, it may be beneficial to isolate the 6-inch line from the more northerly portion of the system near Cromwell Harbor Road and South Main Street so that all local demand in this area is forced to pass through the tank.

Headloss along the 8 and 10-inch lines feeding the tank was found to be high and the hydraulic grade line on the way from the center of Town out to the tank drops approximately 25 feet.

The tank is available to feed water back to the main region of the system in the event of a loss of the source water or in case of a fire demand, although system pressures would drop significantly. The best use of this tank is for storage to meet the needs of The Jackson Lab. With modifications to improve water flow through the tank under peak summer flow conditions, water quality will be improved. Without these modifications, the disinfectant residual in the tank should be checked regularly. It may be necessary to set up a small circulation loop at the tank and rechlorinate the tank contents in order to maintain a sufficient residual.

2.6.2 New Dreamwood Hill Tank

Throughout the years there have been repeated discussions about improving water flow and pressures throughout the northern end of the system through construction of a year-round standpipe on Dreamwood Hill and converting much of the seasonal system in this area to year-round buried service.

In the fall of 1999, the impending development of the Birch Bay Village housing project on a hill overlooking Crooked Road in Hulls Cove drove an analysis of the means to best serve this area. At the time a decision needed to be made by the developers as to whether to support improvements to the public water system or to drill on-site wells for domestic and fire needs and maintain water storage on-site for fire suppression. The Birch Bay site is located at the highest elevation in the area at 220 feet.

To serve the Birch Bay project, it was determined that a storage tank on a lot adjacent to the existing Dreamwood Hill summer tank (ground elevation 200 feet) would have to be approximately 100 feet tall and would have to be served through a separate high pressure zone created for the area. Several alternative routes for piping and for pump station and pressure reducing valve locations were developed.

Due to timing and the costs involved, the Birch Bay project ultimately decided to develop its own water system. Removing this development site from the area to be served would allow adequate service to the remaining northern end of the system through the use of a 60 to 70 foot tall tank on the water division owned lot. Such a tank would set the hydraulic grade line for a north end at elevation 260-270, just slightly below the Eagle Lake gradient. Although the tank could fill under gravity conditions during portions of the year, it would be necessary to provide a booster station in order to assure that the tank filled at a reasonable rate during the summer and that turnover in the tank was adequate. Without pumping, the model estimates that it could take as long as five days to fill the tank when the system is operating at an average demand of 1 MGD. The tank would then empty in approximately 1.5 days. This situation would be aggravated during the summer when system demand would approach the 3 MGD volume and additional demand on a Dreamwood Hill tank would come from the area north to Salisbury Cove.

A new Dreamwood Hill tank should contain a volume of approximately 500,000 gallons in order to meet the storage needs of the system. Options evaluated in 1999 looked at two principal routes for piping to the tank: along the existing Route 3 corridor, or cross country from Crooked Road in the vicinity of the Hulls Cove Wastewater Treatment Facility directly to the tank site.

The cross country route would be through private property, but would open up a significant area of land for development. It would also allow for better looping of the system. The cross country line pressure could be boosted through the use of a pump station, and flow from the tank would be via gravity north to Salisbury Cove and south back to Hulls Cove along Route 3. It was estimated that approximately 1600 feet of 12-inch pipe along Crooked Road, 6000 feet of 12-inch pipe cross-country to reach the tank, and 1800 feet of upgraded pipe from the tank down Route 3 to the vicinity of the Gerrish pump station would be required for this option. The Gerrish pump would become obsolete and would be removed. A booster station would be required at Crooked Road to ensure that the tank would fill under all conditions. A control valve would be required on Route 3 just north of Crooked Road to ensure that the system would not short circuit when the pumps were running and simply pump water out of the tank and back in. This valve would also force water serving local demand to be initially drawn from the tank in order to assure proper turnover. This option would allow for system extensions for a significant distance out Crooked Road and north along Route 3.

The alternative would be to fill and draw from the tank from a single new line along Route 3. For this option a booster station could be located on Syndicate Road and a pressure reducing valve would be located in the same general area on Route 3. This option would require only an 1800 foot upgraded line from the Gerrish pump up to the tank site. The tank would be the same size and height. It would not serve any of the interior land or the Crooked Road outer area, although piping could be extended back in that direction as part of a future phase.

The disadvantage to the second option is that pressure fluctuations in the service lines, both on the suction and discharge sides of the pump station would be much more significant than they would for a booster station on Crooked Road. The longer, small diameter suction line out to a Syndicate Road booster station and the smaller diameter piping that such a booster station would pump into, could see pressure fluctuations of as much as 40 psi between the pump-on and pump-off condition. This effect would be lessened for a Crooked Road booster station with larger cross-country discharge piping (12-inch).

In order to minimize pressure fluctuations under either option, the tank fill rate was limited to 250-500 gpm during the modeling. This still provided for an acceptable tank turnover interval.

2.7 UNIDIRECTIONAL FLUSHING PROGRAM

In an effort to increase capacities of the existing water pipes, an aggressive unidirectional flushing program should be initiated. Unidirectional flushing consists of isolating a particular pipe section by closing appropriate valves and exercising hydrants in an organized, sequential manner. The term unidirectional flushing is often associated with a velocity of 6 feet per second, however the concept of isolating pipe segments and flushing in a sequential manner from the source to the outer reaches of the system can be practiced at lower velocities. In communities that contain areas of aging water mains, like much of the downtown part of the Bar Harbor system, reduced velocities are recommended. Higher velocities may stress the pipe and create bursting or breaking that will require repairs.

According to the American Water Works Association's Guidance Manual for Maintaining Distribution System Water Quality, the following velocity guidelines are for achieving various water quality goals using a unidirectional flushing program.

- ≥ 3 feet per second – remove silt, sediment, reduce disinfectant demand
- ≥ 5 feet per second – promote scouring, remove biofilm, loose deposits, reduce disinfectant demand
- ≈ 12 feet per second – remove sand from inverted siphons

We have selected a target flushing velocity of 3.5 feet per second for the program developed in Bar Harbor. This velocity is greater than the typical ranges for removing silts and sediments but less than that which typically promotes scouring or biofilm removal. In selecting the flushing velocity, we have considered pipe age and did not want to provide a velocity that could potentially damage the existing water mains.

Hydrant flushing, as practiced historically by the water division, is an excellent way to maintain clean service pipes to the hydrants and should be continued. The unidirectional flushing program is similar in nature to a hydrant flushing program except that the hydrants are opened for a longer period of time and system valves are operated to maximize the water flow along the pipe being flushed. All the 6" diameter pipes in the system were found to have very low roughness coefficients (i.e. very high friction losses). Additionally, the 10" diameter pipe along Mount Desert Street was determined to have a very low roughness coefficient.

The proposed flushing program is itemized in Table 7. The required flow to achieve a scouring velocity of 3.5 feet per second in 6", 8", and 10" diameter pipe is approximately 325 gpm, 550 gpm, and 875 gpm, respectively. The amount of time the flushing at each hydrant should take place is equal to twice the travel time in the pipes. Once the flush is initiated, it is expected that the water quality will diminish as the water scours the inside of the pipe. The flush should be run until the water runs clear again.

If the flushing program is not effective in increasing the flows through the system, a more rigorous cleaning or routing procedure may be required, which would include increasing flushing velocities.

The flushing program will work optimally if the work is performed in the sequence listed in Table 7.

Table 7: Flushing Program

	Flushing Streets	Closed Streets (valves)	Open Hydrant	Minimum Flow, gpm	Pipe Length, feet	Flush Duration Minutes
1	Harbor Lane	None	45	325	500	10
2	Devon Road	None	6	325	1500	20
3	Eagle Lake Road	None	19	325	2200	25
4	Forest Street	None	40	325	1200	15
5	Malvern Belmont Service Road	None	2	325	800	10
6	Norris Avenue	None	96	325	800	10
7	Kennebec Avenue	None	98**	325	850	15
8	Greeley Avenue	None	42	325	400	10
9	Mount Desert Street	Valves to Spring, Roberts, Ledgelawn, High, Kennebec, School and on Mount Desert St. near Kebo.	79	875	1600	20
10	Roberts Avenue	Valve to Cottage Street	14	325	850	15
11	Albert Meadow	End of Albert Meadow to Atlantic Avenue	1	325	500	10
12	Main Street Part 1	Valves on First South, Pleasant, Edgewood, and on Main Street near South Main Street	71	325	1700	20
13	Main Street Part 2	Valves on First South, Pleasant, Edgewood, and on Main Street near Mount Desert Street	71	325	1250	15
14	Atlantic Avenue	End of Albert Meadow to Atlantic Avenue	4	325	900	15
15	Hancock Street	None	43	325	900	15
16	Wayman Lane	None	100	325	1000	15
17	Livingston Road	None	67	325	1100	15
18	South Street (First Square)	Valve to School Street	87	325	500	10
19	Second South (Second Square)	Valve on School Street near Pleasant and valve on School Street near Mount Desert Street	66	325	900	15
20	Ash Street	None	3	325	300	10
21	School Street Part 1	Valves on First South, Pleasant (two valves), Edgewood, and on School Street near Cromwell Harbor Road.	88	325	2000	25
22	School Street Part 2	Valves on First South, Pleasant (two valves), Edgewood, and on School Street near Mount Desert Street	88	325	600	10
23	Edgewood Street	Valve to School Street	89	325	500	10
24	South Main Street/Schooner Head Road	Open first hydrant (75) for duration, then close hydrant and proceed to second hydrant (92) for same duration. Close hydrant and open hydrant 93 for same duration.	75, 92, 93	325	3500	35
25	Eden Street	None	32	625	500	10
26	Eden Street	None	30	625	425	10
27	Eden Street	None	37	625	600	10
28	Eden Street	None	25	625	500	10
29	Eden Street	None	23	625	850	15
30	Eden Street	None	24	625	725	10
31	Eden Street	None	34	625	650	10
32	Eden Street	None	31	400	600	10
33	Eden Street	None	35	400	850	15
34	Eden Street	None	29	400	775	10
35	Eden Street	None	33	400	4000	40
36	Eden Street	None	56	400	1250	15
37	Eden Street	None	50	400	1050	15
38	Eden Street	None	48	250	1175	15

	Flushing Streets	Closed Streets (valves)	Open Hydrant	Minimum Flow, gpm	Pipe Length, feet	Flush Duration Minutes
39	Eden Street	None	49	400	2250	25
40	Eden Street	None	52	400	750	10
41	Eden Street	None	59	400	875	15
42	Eden Street	None	58	400	1475	20
43	Eden Street	None	51	400	600	10
44	Eden Street	None	60	400	600	10
45	Eden Street	None	55	400	600	10
46	Eden Street	None	57	400	650	10
47	Eden Street	None	54	400	725	10
48	Eden Street	None	53	400	550	10
** This hydrant was unnumbered on the System Map. It is located on Kennebec Avenue near Firefly Lane.						

3. TREATMENT EVALUATION

3.1 INTRODUCTION

This section presents information on the present methods of water treatment employed by the Bar Harbor Water Division and discusses the impacts of this treatment on water quality and aesthetics. It also presents potential options for future treatment that the Water Division may wish to consider. Future changes to water treatment may be driven by aesthetic concerns, by chemical handling and safety concerns, and by regulatory requirements. Known upcoming regulatory requirements will be discussed as they might pertain to Bar Harbor's system.

3.2 WAIVER FROM FILTRATION

As previously noted, Bar Harbor's system operates under a waiver from filtration. It is important to understand the origin of the waiver program. The Federal legislation known as the 1986 Amendments to the 1974 Safe Drinking Water Act resulted in EPA's 1989 Surface Water Treatment Rule (SWTR). Effective in December, 1990, the SWTR essentially mandated filtration for all public water systems using surface water as their source of supply in order to provide a minimum of 99.9 percent combined removal and inactivation of the microbe Giardia and 99.99 percent of viruses. The adequacy of the filtration process was to be established by measuring turbidity (a measure of the amount of all particles) in the treated water and determining if it met EPA's performance standards.

Several of the largest metropolitan systems in the country, including the New York City and Boston metropolitan areas would have been faced with tremendous costs to implement the then existing filtration technologies to systems of their scale. These systems sought a waiver program from the requirement to filter their water supplies by demonstrating that they had sufficient watershed protection measures in place to assure the continued high quality of their source waters and that their disinfection based treatment systems alone could provide the very same level of contaminant removal or inactivation and, therefore, the same level of public health protection. In addition, due to their generally large staff sizes, these systems were able to demonstrate that they had the manpower and staff oversight to closely monitor their raw and treated water and the treatment process itself to a very high degree.

The waiver program, although a national program, was applied very unevenly from state to state. In Maine by many accounts the waiver program was applied quite liberally. This was primarily because a number of the water systems had remote, high quality sources in watersheds that were controlled to a large extent by the water utility. By 1992, when the deadline for entry into the program was reached, 14 systems in the State of Maine were granted filtration waivers.

There have been criticisms of the way the waiver program was administered in the State. Chief among these has been the argument that the program was originally primarily intended for large systems that could provide good oversight of their treatment, testing, and reporting programs. Of the 14 systems waivered in Maine, upwards of 10 including Bar Harbor would be considered as small systems both in terms of water production and in terms of staffing levels. Some have argued that these systems are not equipped to provide the technical oversight that an unfiltered treatment system requires. Indeed, since the original granting of the waivers, two small systems have had their waivers revoked due to treatment violations.

Some have argued that even for large systems the waiver program was never intended to be a permanent condition, but was designed to give large systems more time to prepare and fund filtration facilities that would eventually be mandated. There is still considerably wide interpretation of the permanency of the program. In several cases, utilities have been directed to have a plan in place for the construction of filtration facilities so that they could be rapidly brought on-line in the event of violations that could lead to the revocation of the waiver.

Nationally, there is a relatively powerful lobby of the nations biggest water treatment systems that would argue that they are capable of protecting the public health without the need for filtration. This would tend to prolong the life of the waiver program. At the very least, the continuance of the program has allowed new technologies in water filtration to come to market that can provide greater efficiency and smaller space requirements for filtration systems.

With truly pristine waters, low in organics and suspended solids, such as Bar Harbor's, there is limited value in filtering. In fact, under the traditional filtration technologies typically employed in drinking water treatment, forming a large filterable solid particle, known as floc, can be quite difficult. In some cases, inert solids are actually added to the water in an effort to provide enough particles to bind together with a coagulant chemical into a floc which then sweeps other particles from the water as it settles or is filtered.

One additional characteristic places Bar Harbor's water system under intense scrutiny when it comes to meeting filtration waiver requirements: its status as a tourist destination with a relatively large fluctuating population from many parts of the country. Systems such as this are under special scrutiny due to the greater difficulty that a transient population poses in identifying waterborne disease outbreaks. Systems supplying a stable resident population tend to be easier to monitor for the early stages of such outbreaks. This is considerably more difficult when the population served may be quickly dispersed around the country to areas which may not be able to correlate similar cases and tie them quickly to a specific cause. In addition, transient customers may be more susceptible to the water quality of a specific system.

For these reasons, unfiltered systems in destination communities such as Bar Harbor are subject to increased scrutiny and held to higher standards in terms of technical violations than might otherwise be the case.

For the foreseeable future, Bar Harbor should consider it possible to maintain its filtration waiver, however, it should understand that in order to do so it must be vigilant in meeting water quality requirements, testing and reporting requirements, sample hold times and other technicalities, and implementing the very highest degree of overall watershed protection measures.

We will suggest some alternative water filtration processes should filtration become mandated, and provide an idea of the sizes that these treatment units might take. This should not be interpreted as a suggestion that the Water Division move to adopt these measures at this point in time.

3.3 ISSUES SURROUNDING MAINTENANCE OF SOURCE WATER QUALITY & WATERSHED PROTECTION

Watershed protection is a major area of focus for all water systems, and particularly for unfiltered systems. Watershed protection frequently takes the form of rules and regulations for activities within the watershed, however an often overlooked component is public education and outreach.

Bar Harbor is fortunate in many respects that its water supply is contained within Acadia National Park. This considerably relieves the Division from the threat that normal development pressures would ordinarily have on such a scenic watershed. Park Service ownership does bring with it a host of new concerns however. It is possible to have conflicting requirements come into play.

Several years ago, the Mount Desert Water District in Seal Harbor ran into just such a problem. The District's water is drawn from Jordan Pond, a major focal point of the Park. The District was required by the State Drinking Water Program to mark its intake and to place a string of buoys 500 feet from the intake to delineate a zone restricting canoe traffic. The placement of these buoys wasn't acceptable to the Park Service from an aesthetic standpoint and they were ultimately removed after involving the State's Congressional delegation.

Conflicting management plans regarding beavers can also become an issue. It would be in the Water Division's best interest to remove any beavers from the watershed, since these animals are often carriers of the Giardia parasite. The Park Service has taken a position against the removal of naturally occurring beaver populations.

The shoreline of Eagle Lake is an area of a significant amount of human activity. Hiking trails ring the lake and carriage paths run in close proximity to much of the shore line. A public boat launch is approximately 1200 feet from the chlorination building. Recreational use of the area occurs year round, with ice fishing and cross country skiing occurring during the winter. There have been occurrences of human swimming, and of dogs and horses swimming or wading in the water, although these uses are prohibited. The Water Division has a network of signs both along Eagle Lake and along Bubble Pond noting that the water serves as a public water supply and water contact is prohibited. Notice is also posted on the Park-issued hiking trail maps, and the Water Division employs its own ranger during the period from mid-June through Labor Day to monitor use and warn people of improper use. Still, there is a balance between excessive signage and excess paper notices distributed in a National Park setting. This can be an area of friction between the Water Division and the Park Service.

One area of particular concern is ice fishing. Fishing shacks must be outfitted with solid floors, and no ice fishing is permitted within 1000 feet of the intake. These are reasonable and prudent precautions to guard against contamination. ATVs and snowmobiles are permitted on the lake, however automobiles are prohibited from driving on Eagle Lake.

One area that should not be overlooked is public education and outreach. Community and school based programs are excellent ways to increase public awareness as to the value and importance of protecting the source water. For several years a hands-on field laboratory was part of the curriculum of one of the science teachers in the Mt Desert High School. Local water suppliers were asked to help monetarily support the program each year. This type of activity was looked upon highly favorably by the State Drinking Water Program (DWP) during their bi-annual watershed inspections. Such programs can also help to gather baseline data on water quality that can be later used to determine if there has been any trend toward water quality deterioration.

Other than possible microbial and viral contaminants, there is little else that should present a problem in the source water. Turbidity is generally not a problem with this source, and Eagle Lake, as with most of the lakes in this immediate area, is very low in biological activity. Metals, pesticides and other organic contaminants are not an issue. The water has only moderate hardness, but is extremely low in alkalinity and so alkalinity must be added in order to adjust and stabilize pH in the distribution system. Lime is used for this purpose. Raw water pH is generally in the range of 6.3 to 6.5. The system routinely passes its lead and copper testing requirements.

3.4 FUTURE REGULATORY IMPERATIVES

3.4.1 LT2

Upcoming regulatory requirements which are part of the Long Term 2 Enhanced Surface Water Treatment Rule (LT2ESWTR or LT2), will have impacts for systems the size of Bar Harbor. This rule will require testing for and classification (bin rating) of the potential for *Cryptosporidium* to exist in the watershed. *Cryptosporidium* is a pathogenic oocyst. This organism can be removed by filtration and can be inactivated by certain disinfectants. Unfortunately, it is highly resistant to destruction by chlorine. Depending on the bin classification a water system is placed into, a 2-log to 3-log removal (99-99.9%) of this organism is required. It will not be feasible to achieve this level of removal through the use of chlorine, at least not without creating a host of other potential public health problems and aesthetic concerns. Disinfection alternatives will be discussed in a subsequent section.

Exact compliance dates for LT2 are difficult to predict because there have been numerous delays in finalizing the rule. Initially LT2 was due to be promulgated in May, 2002. This date slipped to June, 2003 due to slippage in the date for LT1, a rule that applies to larger systems. Anticipated promulgation of LT2 slipped again to July, 2004 due to a significant number of public comments, and even this date is now widely viewed as unrealistic. The current best guess is that a promulgation of July, 2005 is likely. This date becomes important because it sets the clock ticking on a sampling and compliance schedule that will impact Bar Harbor's system.

There are a number of intermediate steps that must be achieved prior to the eventual compliance date for LT2. Each of these steps is tied to the rule's promulgation date. The entire process is laid out in detail in EPA's "Draft Source Water Monitoring Guidance Manual for Public Water Systems for the Long Term 2 Enhanced Surface Water Treatment Rule (LT2 Rule), June 2003". The steps are as follows:

- Within 27 months of rule promulgation, submit a sampling schedule through the LT2 Data Collection System.

- Beginning 30 months after rule promulgation, collect 2 raw water *E. coli* samples each month for 1 year following a specific protocol developed by the rule. These samples may or may not be the same as *E. coli* samples collected for other reporting purposes.

- Additionally, small systems *may* be required to monitor for *Cryptosporidium* for 1 year, beginning 6 months after completion of their *E. coli* monitoring, and *will* be required to monitor if the *E. coli* annual mean concentrations exceeds 10 col/100 ml.

- Within 45 months of rule promulgation, submit a *Cryptosporidium* sampling schedule through the LT2 Data Collection System.

- Within 48 months after rule promulgation, collect 2 *Cryptosporidium* samples each month for 1 year. This can be a difficult and expensive test to undertake, as this organism is not easily identified. There are also few labs, nationally, presently certified to conduct the test using the EPA's 1622 or 1623 testing methodology. The cost for individual tests is presently on the order of $500.

- Approximately 6 months after the completion of sampling a Bin classification of the system will be made depending on the mean concentration of *Cryptosporidium* oocysts. A cutoff of 0.01 oocysts/liter will determine if an unfiltered system falls into Bin 1 or Bin 2. If in Bin 1, the system will be required to demonstrate a 2-log (99%) removal of oocysts. If in Bin 2, a 3-log (99.9%) removal is required.

Once the Bin classification has been assigned, systems must begin the design of suitable treatment facilities and will have approximately five years to design and bring an effective treatment facility on-line. States may grant an additional two years to complete this process for hardship cases.

Although the final implementation date for this proposed rule may be as much as ten years away, the Town should consider the impacts this rule will have sooner rather than later. This is important because it will not be possible for the treatment levels required by this rule to be met through the use of chlorine disinfection. Even with no other regulatory changes, this rule alone would either drive Bar Harbor's treatment to either some form of filtration or to some other disinfectant. Other pending rules will have the same impact. Fortunately, there are new disinfection technologies coming to market which will provide viable treatment.

3.4.2 DBP Rule

The Stage 1 Disinfectants and Disinfection Byproducts Rule (DBP rule) applied to systems the size of Bar Harbor's beginning in December, 2003. Its immediate impact was to lower the allowable levels of two categories of disinfection by-products, Total Trihalomethanes (TTHM or THMs) and five Haloacetic Acids (HAA5 or HAAs). TTHMs consist of the sum of the concentrations of chloroform, bromodichloromethane, dibromochloromethane, and bromoform. The TTHM allowable concentration was lowered from 100 µg/l (ppb) to the new standard of 80 µg/l. HAA5 is the sum of the concentrations of mono-, di-, and trichloroacetic acids and mono- and dibromoacetic acids. The HAA5 allowable concentration was lowered from 80 µg/l to the new standard of 60 µg/l.

THMs and HAAs are byproducts that primarily result from the interaction of organics found in raw and filtered water with chlorine. In addition, the DBP rule set standards for chlorite (1.0 mg/l) and for bromate (10 µg/l). These are byproducts which form from interactions with the disinfectants chlorine dioxide and ozone, respectively. They would not ordinarily be associated with the type of treatment performed in Bar Harbor. The DBP rule also sets a maximum permissible chlorine residual in systems at 4 mg/l as Cl_2.

THM and HAA compliance is determined based on an annual average of samples, generally collected quarterly. Samples must be collected from within the collection system. Since disinfection byproducts form from contact with chlorine over time, samples should be collected from the far reaches of the distribution system, where the water age is the longest.

Because the level of THM and HAA formation is related to the chlorine dose and the level of organic matter in the water, it stands to reason that systems that either do not filter or that add chlorine to water ahead of a filtration step would be under the greatest risk of DBP formation. Since Bar Harbor's system does not filter and adds a relatively high dosage of chlorine to the water, it would seem to be susceptible to high DBP levels. Fortunately, Eagle Lake water is extremely low in organic compounds (as measured by Total Organic Carbon, or TOC, levels).

With an applied chlorine dose averaging 2 mg/l, and a typical distribution system chlorine residual of 1 mg/l (high compared to more typical filtered system residuals of 0.2 to trace mg/l), Bar Harbor's water develops THM levels of approximately 40 µg/l and HAA levels of 29 µg/l, well below regulated levels and generally below the levels found in many systems around the State, filtered and unfiltered.

The last DBP sample collected from the system was in February, 2004. Samples are generally collected from the Hull's Cove General Store, within the northern reach of the distribution system.

3.4.3 Other Rules

Other rules on the horizon would impact primarily groundwater systems or filtration facilities. Arsenic and radionuclide rules are pending, but are principally related to groundwater sources. It should be noted, however, that the native bedrock throughout Mount Desert Island is granite, and that groundwater sources drawn from wells developed from granitic rock are particularly susceptible to higher radon and other radionuclide levels. This could become a consideration if the Bar Harbor system ever contemplated the use of groundwater as a backup or alternative source of supply.

The lead and copper rule is already in effect for systems the size of Bar Harbor's. This rule requires that at least 90% of households within a system have copper levels less than 1.3 mg/l and lead at less than 0.015 mg/l. Lead and copper levels are primarily influenced by the aggressiveness of the treated water toward plumbing and plumbing fixtures within a customer's home.

Sampling for lead and copper must be done based on a program of sample sites selected from area housing that is deemed as most vulnerable to the problem based on age and plumbing types. Initial sampling must consist of two rounds of samples conducted six months apart. If lead and copper levels are below the action levels, the water system can go to annual sampling for three years. If it passes that round of tests, it can reduce sampling to once every three years.

Presently, Bar Harbor's system tests once every three years. The last round of testing was completed by the end of the summer 2004. Twenty locations are sampled. Testing performed in August of 2001, yielded average lead concentrations of 0.0043 mg/l and average copper concentrations of 0.0531 mg/l. Only a single sample was at the action level for lead and none were above. All copper concentrations were well below the copper action level.

The Bar Harbor system guards against high lead and copper levels through pH adjustment of the treated water using lime. Raw water pH in the range of 6.3-6.5 is adjusted to 9.0-9.2 by adding lime at the Duck Brook pump station. Because of the low alkalinity (buffering) of the raw water, significant pH adjustments can be made through the application of small quantities of lime.

3.5 DISINFECTION ALTERNATIVES

3.5.1 Existing Use of Chlorine Gas

The water system presently disinfects Eagle Lake water using chlorine gas stored at the former screening building located at the Eagle Lake shore. Chlorine dose is paced to water flow measured by an in-line magnetic flow meter installed in the intake line at the screen building. The transmission lines between this building and the Duck Brook pump station are used as the contact chamber for CT computation purposes. A chlorine residual is measured at the pump station and is relayed back to the screen building for use as a trim signal.

Chlorine gas has historically been used as a disinfectant because it was cheap, effective, relatively easy to handle, transport, and store, and because it occupies a small space in the building. The water system typically uses approximately 400-500 pounds of gas a month during periods when average daily water flow is 0.7-1.0 MGD. Generally, four 150-pound cylinders of gas (600 pounds of gas) are stored on-site.

If the system stored in excess of 1500 pounds of gas on-site, under OHSA regulations it would be required to maintain a Process Safety Management (PSM) Plan (29 CFR 1910.119). The 1500 pound threshold quantity need only be exceeded at the time an inspection occurs for the requirement to be in effect. There are 14 elements to these plans, including:

Employee Participation	Mechanical Integrity
Process Safety Information	Hot Work Permits
Process Hazards Analysis	Management of Change
Operating Procedures	Incident Investigation
Training	Emergency Planning and Response
Contractors	Compliance Audit
Pre-startup Safety Review	Trade Secrets

Of these, the compliance audit can be the most onerous in terms of cost.

The EPA Consolidated List of Chemicals Subject to the Emergency Planning and Community Right-To-Know Act (EPCRA) and Section 112(r) of the Clean Air Act (the so-called List of Lists) lists chlorine gas as a Section 302 Extremely Hazardous Substance. It establishes a Threshold Planning Quantity (TPQ) of 100 pounds for chlorine. Facilities storing more than the TPQ are required to submit Chemical Inventory Reporting Form (CIRF), Form R annual reports. In addition, the Maine Emergency Management Agency (MEMA) has adopted the TPQs as the basis for its requirements for Maine Facility Emergency Response Plans. Unless the facility prepares and submits a Spill Prevention, Control, and Countermeasure (SPCC) Plan, all releases must be reported, regardless of size or consequences.

There are several reasons for moving away from chlorine generally, and chlorine gas specifically, for primary disinfecting purposes, principal among them are safety and security concerns and the reporting requirements. The former Water Company has already had an employee injury resulting from chlorine gas leaks during gas cylinder change over, and leakage to the environment has occurred due to plugging of chlorinators with debris.

Chlorine cylinders are stored in a common room with other electronics, analyzers, and equipment. This room has a below-grade pipe vault which could allow heavier than air chlorine gas leakage to accumulate. The building is fitted with an ambient air gas alarm which is tied into the building's autodialer in order to alert Division staff in the event of a leak. However, the building is not adequately protected or ventilated as a secure chlorine gas storage space as would be required if larger stored quantities were present. Nonetheless, the hazard, particularly to Division employees is just as real.

Security concerns and additional paperwork and reporting requirements for chlorine distributors have recently driven up gas prices. Several suppliers have left the market, particularly those willing to handle small deliveries. Water Division staff report that in recent years their cost for purchasing chlorine gas has approximately doubled to a current price of $1.00/pound. Security concerns have not been an issue thus far with the Park Service at the Eagle Lake site, but it can be expected that this will become a point of focus, particularly when considered in the context of the Division's soon to be completed vulnerability assessment.

3.5.2 Sodium Hypochlorite

Bar Harbor's wastewater treatment facility presently uses sodium hypochlorite as a disinfectant. There have been past discussions within Public Works on converting the water system to sodium hypochlorite as well, since weekly deliveries are already made in Town and since there is a familiarity with handling it. If a direct conversion to sodium hypochlorite was made, assuming the same dosing was kept, the 400-500 pounds per month of gas usage would translate to 350-475 gallons per month of 15% sodium hypochlorite solution. In order to accommodate regular deliveries and allow for increased summer usage, on-site storage capacity of 1200-1500 gallons would be necessary.

The conversion to sodium hypochlorite as a direct replacement for chlorine gas at the Eagle Lake site would cause several complications:

- Additional storage space would be required.

- Bulk truck deliveries could not be made on the unpaved roads that access the site. It would likely be necessary to deliver hypochlorite in plastic drums and transfer the contents to a bulk storage tank or work directly from drums and a day tank. The additional handling required would increase the potential for accidents and would require additional storage and work space.

- Secondary containment would have to be provided for a bulk storage tank or for stored drums.

- Sodium hypochlorite exposed to the atmosphere through open top or unpressurized tanks would create a corrosive atmosphere in the building space. Chemical storage would need to be completely separate from sensitive electronic equipment.

In addition, it will not be possible for Bar Harbor to meet the *Cryptosporidium* requirements of the upcoming LT2 rule using chlorine gas or hypochlorite for primary disinfection. A conversion to hypochlorite also does nothing to minimize disinfection byproduct formation (although already low) resulting from the use of free chlorine and does not eliminate any of the potential aesthetic concerns that result from a high free chlorine residual in the finished water. On the plus side, the conversion to hypochlorite would eliminate the drop in pH in the unbuffered water that results from the application of chlorine gas.

3.5.3 Primary Disinfection Using Ultraviolet Light

One alternative to the use of free chlorine for primary disinfection is a conversion to ultraviolet (UV) light for primary disinfection and free chlorine (hypochlorite) or combined chlorine (chloramines) to provide a system residual. UV disinfection is an up and coming technology which has been found to be extremely effective in inactivating *Cryptosporidium* and *Giardia*. It can be less effective at killing some classes of viruses, however, these organisms can be easily killed using small doses of free or combined chlorine. When used in conjunction with a secondary disinfection step that uses some form of chlorine, a very effective disinfection barrier is developed that will meet future LT2 requirements.

UV disinfection has the added benefits that it does not appear to create any disinfection byproducts and that the reaction is essentially instantaneous, completing within a small reactor vessel the size of a pipe tee fitting. As a result, there is no requirement for a post-vessel CT. A UV unit is installed in-line and can be followed by any other treatment or chemical addition step without concern for interfering with the disinfection reaction.

Guidance is currently being finalized on the use of UV for drinking water primary disinfection, and it is anticipated that this treatment technology will find widespread use throughout the water treatment industry once the guidelines are formally approved. This technology has already been officially recognized as an effective treatment for meeting the requirements of LT2.

Ordinarily, the UV treatment step is applied prior to the addition of any other chemicals to the water. If used in a filtered system, the UV step would typically be applied after filtration, but ahead of any other chemical additions. The reason for this is that the required UV dose is highly dependant on the UV transmissivity of the water. In Bar Harbor's case, its raw water is extremely visually clear and low in turbidity. However, this does not necessarily translate to clarity to UV transmissivity. This characteristic can only be determined through sampling and measurement of the transmissivity directly.

Measurement of UV transmissivity is easy to perform, however, it does require the use of a spectrophotometer that is capable of making the UV reading. Unfortunately, the Hach DR/2010 spectrophotometer which many water utilities use for chlorine DPD testing cannot be used to measure UV transmissivity. The simplest commonly available spectrophotometer manufactured by the Hach Company capable of this test is the DR/4000, a unit which costs approximately $6000 with the necessary accessories.

We have previously recommended to the Water Division that it consider collecting samples for UV analysis over a period of one year. Sampling should not necessarily be frequent as rapid changes in UV transmissivity are not expected. The sampling should be made through all four seasons in order to determine the impacts of the spring and fall overturn of the lake, ice cover, wind induced turbidity, and similar effects. It was suggested that several samples be collected during the first month and that thereafter monthly or semi-monthly samples could suffice. Additional samples could be collected if specific events occur such as a hurricane or algae bloom.

The test performed is normally a UV 254 analysis, which looks specifically at the 254 nanometer UV wavelength. A better analysis is to generate a printout of the response to all UV wavelengths between 190-300 nanometers. This can be done using a DR/4000. This is beneficial since there are several clusters of wavelengths of UV energy that are effective as a disinfectant and credit should be obtained for each cluster. If the Water Division does not wish to purchase its own spectrophotometer, commercial labs and many area water districts can perform the test.

Once the variability of the UV transmissivity of Eagle Lake water is known it will be possible to determine the size and operational cost of a suitably sized UV disinfection system. Recent pricing for UV systems for other area water supplies was on the order of $300,000 for a system with an average daily flow in the 3-3.5 MGD range. Operational costs, primarily for electricity, are comparable to chemical costs for disinfectants. Maintenance costs for lamps and analyzers are highly variable, depending on the manufacturer's equipment design and on anticipated regulatory changes which may require adjustments to some equipment designs and rated outputs. As with other treatment steps, redundancy in equipment is mandatory.

3.5.3.1 Location Options for UV Disinfection

A UV disinfection step could be inserted at either the screening building at Eagle Lake or in the vicinity of the Duck Brook pump station. In either case, it is highly unlikely that the equipment could be located within either of the existing buildings. Although relatively compact as far as treatment equipment goes, a small building addition would likely be necessary. Such an addition would principally need to expose an area of transmission piping where a smooth approach and discharge section to and from a UV unit could be maintained. As with flow meters, turbulence up and downstream is to be avoided, so it would be difficult to install such equipment in the basement of the existing pump station building among the existing valves and fittings.

For simplification of control, access, power, and building facilities, one option would be to consider locating a UV disinfection step at the Duck Brook pump station site, immediately upstream from the existing building. Significant yard piping changes would be required. These changes could possibly fit in well with other suggested changes to pumping to the summer storage tank that will be discussed below. The consolidated facility would measure flow, pace UV primary disinfection to flow, continue to adjust pH using lime, add sodium hypochlorite for secondary disinfection, provide contact sufficient to allow for a CT value of 12 (4-log virus kill at a pH range of 6-9, temperature of 0.5°C or less), and send the treated water into the distribution system.

In order to achieve a CT of 12 under peak flow conditions using a reasonable hypochlorite dose, a contact tank volume of 36,000 gallons would be required. If this step were to be inserted at the Duck Brook pump station site, the contact chamber would be below the hydraulic grade line of the system and would have to act as a pressure vessel. For this to happen, it would have to take the form of a pipe rather than a baffled concrete chamber. It would be difficult to install the required volume of pipe on the existing site, and would be a waste of additional piping since the transmission lines to the pump station could already provide this function. The contact time would need to occur prior to sending water to either the summer storage tank or to the cast-in-place gravity fed tank on the side of Great Hill.

The advantages to the Duck Brook site option are that all chemicals, controls, power, and operations are consolidated at one site. Interior building renovations could be made so that hypochlorite, ammonium sulfate, lime and hexametaphosphate could all be handled within the existing building. A minor building addition would be required for the UV equipment. If a filtration step were ever to be required, it could be located upstream from the Duck Brook modifications without requiring any further changes to the Duck Brook site.

The alternative is to locate UV primary disinfection and hypochlorite addition in the vicinity of Eagle Lake by the old filter beds and continue to use the transmission line as a contact chamber for hypochlorite virus kill. The existing transmission lines between Eagle Lake and the Duck Brook pumping station provide more than twice the volume necessary to achieve the required CT for virus kill at the peak design flow rates. Hypochlorite dose could be reduced or an ammonium sulfate addition station (if conversion to chloramines was considered) could be constructed midway along the transmission line. This would result in yet another separate facility and dosing would be difficult since flows would be split between two parallel transmission lines.

The sequencing of the flow measurement, UV primary disinfection, hypochlorite addition, and hypochlorite contact steps near the Eagle Lake site would be very straight forward. The contact chamber (transmission line) is already in place. A building expansion would be required together with a significant upgrade to the incoming power service. Provisions for hypochlorite delivery, handling and storage would be required, although hypochlorite quantities applied and stored on-site would be much less than discussed earlier if hypochlorite were simply used as a direct substitute for chlorine gas as the sole disinfectant.

The biggest negative associated with implementing disinfection upgrades at this site center around the visibility and ownership of the surrounding area. The Eagle Lake site is in close proximity to the lake, a public boat launch, and to the Park's Carriage Road System. The Water Division owns only a very small lot on which is located the screening/chlorination building. Expanding the facility may involve considerable negotiation with the Park Service and may raise serious objections with the Park or with the public.

3.5.4 Use of Chloramines for Secondary Disinfection (System Residual)

Many water systems are finding that they are at or above the disinfection byproduct (DBP) limits set for THMs and HAAs as a result of the interaction of free chlorine with residual organics in their finished water, whether the water is unfiltered or filtered. To reduce the formation of DBPs, many utilities are converting to a combined form of chlorine, known as chloramines, for a system residual. Chloramines are formed through the addition of small amounts of nitrogen in the form of ammonia to a water that is first treated using free chlorine. The combined form of chlorine, or chloramine, although a relatively weak disinfectant, is extremely persistent within the distribution system and therefore provides an excellent method of maintaining a system residual. Chloramines do not generally act to form DBPs.

Conversion to chloramines and their ongoing use is not without risks, however. The proper chlorine/nitrogen ratio must be carefully maintained in order not to form undesirable forms of chloramines such di- and tri-chloramines which can cause taste and odor problems. Nitrogen is a nutrient and can cause regrowth in pipe wall organisms and nitrification of the water system. The additional persistence of the chloramine can create a spike in the residual in pipes that have not seen this condition before. Particularly in old unlined cast iron pipe, this can lead to softening of the tuberculation on the pipe walls causing significant cases of red water. Nitrification is particularly a concern as water temperatures rise. For Bar Harbor's system, we would have concerns that some of the surface run piping and the shallow buried and exposed transmission lines could be susceptible to nitrification as the piping temperature rises in the sun.

Recent studies have suggested that chloramines may also cause elevated selective corrosion of lead from piping and plumbing fixtures. This is a newly identified area of concern and is being actively investigated nationally. This issue alone may cause a reevaluation of the advisability of converting to chloramination for many systems.

Since DBP samples from Bar Harbor's system have shown relatively low concentrations of THMs and HAAs, there is no particular need at this point to try to eliminate free chlorine from the system and risk other problems associated with chloramination. If and when it became advisable to make a conversion, it would be an easy step to add nitrogen through the use of ammonium sulfate. Nothing proposed elsewhere in this plan would limit the ability to add this future step.

At this point in time we would recommend a future conversion to UV disinfection to meet the eventual requirements of LT2 *Cryptosporidium* inactivation and the conversion to hypochlorite to provide a 4-log virus kill and a free chlorine system residual. Applied chlorine doses would be less than at present and system residuals could be run lower. Chloramination would not be recommended unless THM and HAA concentrations increased significantly.

3.6 FILTRATION

If a filtration facility became necessary, the most likely location for such a facility would be on the Park owned wooded parcel north of Route 233, the site of the former rapid sand filters of the early 1900s. Normally, a disinfection step, whether UV, chlorination, or chloramination, would follow filtration so that the maximum amount of organic matter and turbidity is removed from the water ahead of disinfection.

In Bar Harbor's case there are some fairly unique conditions that suggest that the treatment step order could be reversed. Eagle Lake water is apparently very low in both organics and turbidity. If a filtration step became necessary, it is likely that it would simply be a regulatory requirement and that very little in the way of organics would actually have to be removed. Filtration may become necessary if *Cryptosporidium* concentrations place the system at a Bin level that requires multiple barriers for *Crypto* removal (additional barriers beyond simple inactivation using UV). Conventional filtration facilities generally require chemical additions to initiate coagulation and flocculation of particles into settleable floc that can capture microscopic particles and organisms and be captured on filters. Backwashing is ultimately required, and this generates a waste stream requiring disposal. The preferred method of disposal of this waste stream is to a municipal sewer.

The one potential site is over 3000 feet from the nearest sewer. On-site disposal would be difficult given the shallow soils in the area. Even with on-site disposal, large settling lagoons would be required. This would result in expanded clearing of a site and most likely lead to visual impacts.

The principal exception to backwash requiring treatment methods is slow sand filtration. Unfortunately, slow sand filtration requires considerable filter area and would not be suitable for treating a fraction of the 4 MGD design flow that Bar Harbor would require. In addition, the method is not well suited to varying flow rates and so would not lend itself well to continued operation of the system under gravity flow. This is a necessary condition during winter operation in order to keep water moving in the upper transmission lines and prevent freezing of these lines.

Membrane filtration is likely the most promising filtration technology for Bar Harbor. The membrane pores are small enough that they act as a barrier to microorganisms directly, without the need for chemical additions to develop a floc. Membranes do require backwashing, and this typically results in a waste sidestream of 7-10% of the forward flow through the plant.

The discharge of any sidestream to Duck Brook would be virtually impossible from a regulatory perspective, even if it was Eagle Lake water with no chemical additions, and no increased turbidity. However, given the low organic content of the raw water and the likely infrequency of required backwashing, it may be possible to clarify the backwash water through settling in an underground tank or through an additional sidestream pressure filter or separate membrane treatment process and recycle the supernatant back to the rapid filters. This sidestream filtration could proceed at a slower, more controlled rate. The underflow stream would be greatly reduced in volume and could be collected and trucked off-site to the wastewater treatment plant. Alternatively, 3000 feet of new sewer force main could be constructed.

Membrane filters generally need to be operated at fairly constant flow rates. It is necessary, therefore, to stage membrane treatment facilities with numerous parallel filters that can go on and off line as water demand fluccuates. During the winter months, when continuously varying flow through the transmission lines must be maintained, membrane plant production can be controlled by monitoring the minor water level changes in the cast-in-place concrete storage tank on Great Hill and returning that signal to the membrane plant. Filters would be turned on and off in order to keep the tank level as constant as possible.

During the summer months, membrane plant production rates would be tied into the pump output signal from the Duck Brook pump station.

If a membrane plant needed to be constructed, it and any require backwash storage and settling facilities could be built entirely within an enclosed structure. Such a structure would be roughly the size of a

40x70 foot single story building. Tankage could be located below the building floor. From a centralization of facilities perspective, it would be desirable to relocate the UV and hypochlorite addition steps to the tail end of the filtration facility. There would still be plenty of transmission line capacity below the plant to obtain the required CT. The disinfection steps would not necessarily need to be relocated. There would be a need to negotiate with the Park Service over obtaining the plant site. It may be beneficial to offer a reduction in the need to access the existing screen building on the shore of Eagle Lake (for chemical deliveries and other maintenance) in return for obtaining a parcel away from the lake. With a complete relocation of all lake shore activities to a new plant site, it may be possible to remove the screen building structure entirely. This could be offered as a point of negotiation.

3.7 DUCK BROOK PUMP STATION

There have been on-going discussions with the Water Division about improvements to the Duck Brook pump station. These discussions have included recommendations on replacing the existing summer pumps at the station, updating the electrical entrance to the facility, and modernizing, simplifying, and standardizing the control system at the station.

In 2000, a project to upgrade the pump facilities was submitted to the Maine Drinking Water Program for consideration of State Revolving Fund (SRF) funding. The amount requested was $138,750. The project was approved but never undertaken. About this same time the former Water Company was being converted to a division of the Town's Public Works department and a decision was made to hold off on pump replacement until a financial assessment of the water system needs and cash flow could be undertaken and a master plan completed. At this point the project has likely rolled off the SRF eligible project list. However, it could be reinstated if the decision is made to implement the upgrade.

The existing pumps pump to the summer storage tank at a rate of approximately 3800 gpm. This is equivalent to a daily rate of 5.5 MGD, far in excess of even peak demands. The present day impact is to greatly reduce the potential contact time for disinfectant in the transmission lines and correspondingly reduce the computed CT. This is offset to a certain extent by increasing the applied chlorine dose. Still, computed CT dropouts occur when the pumps initially start up before the ramped up chlorine dose actually reaches the downstream residual analyzer.

In the future, if a conversion to UV disinfection is made, UV dose will be directly paced to flow using a flow meter located at the screen building. Unnecessarily high flow rates as a result of pumping will significantly drive up the sizing and cost of UV equipment and will cause efficiency problems when operating at the normal lower flows. Switching UV lamps on and off to accommodate flow increases will unnecessarily shorten their lifespan and will lead to computational problems in determining the actual dosage applied.

Sudden sharp increases in flow rate also has the potential to create turbulence at the intake and lead to unnecessary turbidity spikes. This was demonstrated during the summer of 2003 when a separation in the intake pipe (causes by air entrapment during a dewatering of the transmission line as a result of accidental damage) allowed silt to be drawn into the line when the pumps started.

In addition to the unnecessary transmission line velocities created, pumping at this rate is very inefficient from an energy consumption perspective. It has been determined that one of the two 100 horsepower pumps (original pumps, at 550V, installed at the time the pump station was first constructed) could be replaced with smaller pumps that could pump continuously to the summer tank. One pump could be used for average summer day demand and a second pump could provide peak flows.

Originally, three 20 horsepower pumps were considered, each producing approximately 0.9 MGD. This option made the most sense when pressing the summer tank into year-round service was under consideration. This option was later eliminated with the construction of the below ground tank and the retention of a gravity flow system for ten months of the year. It would still be beneficial to use two smaller pumps, pumping continuously, to meet average and peak summer demand. The remaining original pump could be used as a fire pump in emergencies.

Part of a pump upgrade project would include conversion of the electrical entrance to the building to 480 volt from the present 550 volt system, upgrading the pump control system, alarm and dialer system, and raw water isolation control system (valve closure upon low residual detection). This project should be scheduled for the near future, as it would be beneficial prior to and after a UV disinfection conversion was undertaken. The recorded CT dropouts on pump start-up, even though they are simply an artifact of the way the CT computation is performed and are not related to actual disinfection effectiveness or protection of public health, present a significant risk to the system's filtration waiver since they may be viewed as a technical violation. This became an issue with the Drinking Water Program during the fall of 1999.

Lime addition should continue as is at the Duck Brook pump station. The Water Division has recently installed new lime feed equipment, and the addition of lime has been very effective at maintaining optimum system pH. The addition of hexametaphosphate should also continue at the pump station. No changes to the system's corrosion control program appear necessary.

The system presently adds fluosilicic acid at the Duck Brook pump station as a source of fluoride. This is a straight forward process using a liquid chemical, added in small quantities, that is relatively easy to handle in terms of storage space and deliveries. Its downside is that it is an extremely corrosive chemical which must be segregated from other chemicals and equipment. It also will create a pH drop in the finished water. If large scale changes in treatment are undertaken, such as construction of a filtration facility, consideration should be given to converting to the use of sodium fluoride. Although this is a dry chemical that must be dissolved in a saturator before being added to the water flow, it is much less corrosive and does not have a pH impact on the finished water.

3.8 SYSTEM CONTROL AND DATA ACQUISITION (SCADA) IMPROVEMENTS

The existing water distribution system contains very primitive instrumentation and virtually no remote control capability. Other than the 500,000 gallon cast-in-place concrete tank constructed in 2000, storage tanks are not instrumented for data display. The new storage tank instrumentation provides tank water level data to a local display located in the Duck Brook pump station. The summer tank on Great Hill provides high and low pressure signals to start and stop the summer pumps at Duck Brook. The Jackson Lab tank is controlled only via altitude valve and includes no instrumentation, as is true of the seasonal tank on Dreamwood Hill.

The system is monitored by manually observing the pressure displayed on a gauge at the Water Division office and adjustments are made by having a Division staff member manually adjust the pressure reducing valve at the Duck Brook pump station. Although the pressure signal from the Great Hill summer tank can automatically start and stop the Duck Brook pumps, in recent years this system has been unreliable and a staff member generally checks in with the station several times each day during the summer to assure that the pumps start when called upon. The pumps are manually shut down and locked out at night after the tank has been filled late in the day.

The Duck Brook pump station contains an autodialer connected into alarms for low summer tank level and low chlorine residual. The dial out system together with the summer tank pressure monitoring system and the pump start system have been cobbled together over the years and are poorly documented. It has been a persistent staff complaint that maintenance and troubleshooting of the system is difficult. Individual projects have been proposed for updating and integrating the limited system with what would be considered a more "off the shelf" package. To date, this change has not been made, and at this point it would be more effective to incorporate a controls and communication upgrade with pump replacement and other improvements to the overall station.

A venturi flow meter located at the Duck Brook station records flow on a circular chart recorder located there. Also recorded at this station on a circular chart is the measured chlorine residual.

Chlorine dosing and residual monitoring instrumentation were upgraded during the 1998 changes made to the intake and screen building. Redundant chlorine residual analyzers located at the Duck Brook pump station monitor chlorine residual for CT computations and return a signal to the screen building chlorinators for use as a dose trim signal. The primary dosing signal comes from an in-line magnetic flow meter located at the screen building. System raw water flow, turbidity, and chlorine residual are each recorded on circular charts located at the screen building. CT computations are made manually.

The screen building contains an autodialer which alarms out for chlorine leakage to the ambient air, high raw water turbidity, and two low chlorine low residual alarms, one for low residual at the station and a second warning that the low low residual level has been reached and the isolation valve at the Duck Brook station will be closing.

Water Division monthly reports to its regulatory primacy agency (Maine's Drinking Water Program, DWP), filings with the Public Utilities Commission, and incoming data such as laboratory analyses and flow rates are all compiled manually on paper worksheets and using the desktop computers at the Water Division office. No automated data entry into routine reports is made and there are no provisions in place for real-time trending of any system variables.

While a relatively simple water treatment and distribution system such as Bar Harbor's may not yet benefit from the capability to remotely control the various components of the system, at the vary least an integrated data monitoring, transfer, and storage system should be put in place. Such a system could deliver information such as system flow, storage tank levels, and computed CT as well as residual, trends, and a history of alarm-out conditions to the Division office for monitoring and storage. To a significant degree data can be input directly into routine reports. Such a system should be expandable so that remote control can be implemented in the future should additional treatment steps such as UV disinfection or membrane filtration be added.

A graphical user interface at the Division office should be developed so that it is easy and intuitive for each staff member to monitor variables such as tank levels and equipment operation.

As part of any reasonable instrumentation upgrade, the screen building, the Duck Brook pump station, the Jackson Lab storage tank, the Dreamwood Hill storage tank (existing as well as future) and the few small booster pump stations in the system should each be provided with a PLC for processing incoming data such as pressures, flows, equipment start/stops, security/intrusion alarms, tank levels, ambient air quality (chlorine), chemical scale weights, chemical feed pump outputs, and similar variables. This information should then be telemetered back to the Division office as well as displayed locally at each site.

Transmission of this information can be handled either by radio or over dedicated phone lines. The alternative of choice is generally radio, saving the cost of telephone lines. In Bar Harbor's case this may be difficult to implement at all locations given the terrain. As a preliminary step, the Division should have a specialty firm undertake a radio signal evaluation test for each facility location in order to determine the feasibility of radio communication for the overall system.

4. UNIFIED ACTION PLAN

4.1 INTRODUCTION

The recommendations, conversions, shortcomings, and regulatory issues discussed within this master plan are of little value unless they are organized into a prioritized action plan. Such a plan considers the importance of the various changes, their drivers (regulatory, public health protection, staffing, safety, financial, growth, etc.), financial impacts of the recommendations, and the sequencing required to allow the system to remain in continuous production and continuous compliance. Individual recommendations, their level of importance, implementation schedules, and anticipated costs are presented below.

4.2 PRIORITIZED RECOMMENDATIONS

4.2.1 Discussions & Planning Items

Abandoned Transmission Line Mitigation (Lead Joints) - HIGH

Discussions with the National Park Service are on-going regarding the environmental impacts of lead leaching from the exposed leaded joints of the abandoned Duck Brook seasonal transmission line. Site sampling is presently underway by the Park Service. It may ultimately be necessary to isolate individual leaded joints and remove localized areas of soil. Alternatively, the Park Service may require the Water Division to remove the entire 3000 foot length of the abandoned pipe or specific sections of pipe that are accessible.

Costs for this work are unknown at this point. If complete pipe removal is required, a wide range of costs could be incurred since access to much of the run will be quite difficult. Removal costs can only be determined once a plan is developed for the type of methods used to remove the pipe without causing further environmental or aesthetic damage.

UV Transmissivity Testing - HIGH

As detailed in the treatment section of this plan, the Water Division should begin regular testing of its raw water for UV transmittance. Depending on the ability of the Division to make arrangements with an area utility to perform the testing, it may be necessary to purchase a DR4000 spectrophotometer for use in-house. This should be budgeted as a $6000 cost. Alternatively, samples collected by the Division could be tested at other utility, commercial labs, or at the State lab. Since this is not a regulatory prescribed test, there are no requirements for laboratory certification. For the test samples there are no complicated fixing, holding, or shipping requirements.

Dam Repair - MEDIUM

The Eagle Lake spillway has been identified as being in need of repairs. The Water Division should have a structural and soils evaluation performed on the spillway to determine the extent of recommended repairs and prepare a detailed cost estimate for the repairs. It may be possible for individual repairs to be undertaken by the Town's Public Works Department on a year-by-year basis, or it may be best to consider a single independent contract for the total rehabilitation of the structure.

The inspection should be scheduled for this summer so that the detailed needs and costs of spillway restoration can be factored into the scheduling and funding of other projects identified in the master plan. We would anticipate being able to complete the evaluation for a budget of $8000, including costs for a soils subconsultant and their report.

Future Land Needs Discussions with National Park Service - MEDIUM

At the time the cast-in-place storage tank was constructed on Great Hill, the National Park Service granted an easement to the former Water Company for the tank site. One of the conditions for this easement was that the water system develop a master plan and advise the Park Service of their possible future needs. This was required so that any future land negotiations could take place with an overall understanding of how the water system would remain viable and what would ultimately be requested of the Park Service, rather than responding to an individual emergency or regulatory requirement.

While it may never need to construct a filtration facility, the Water Division should begin discussions with the Park Service to advise it of the available options and test the feasibility of obtaining the rights to develop the former sand filter site. Possible land exchanges should be part of this discussion.

4.2.2 On-Going Upgrades

Galvanized Pipe Replacement - HIGH

Once a decision is made regarding which elements of the existing seasonal system are to remain seasonal for the foreseeable future, the Water Division should consider the replacement of the remaining galvanized lines with HDPE fused plastic lines. Presently there are approximately 6800 feet of 2-inch and 3-inch seasonal galvanized line in the system. The galvanized lines not only contribute to poor water quality along these runs, but are also one of the most frequent locations of leakage in the system. Replacement costs can be highly variable, depending upon whether Division staff perform the installation or hire it out. Material costs are on the order of $1.50-2.50/foot for replacing surface-run galvanized line with surface-run HDPE piping depending on pipe size and wall thickness.

In addition there are approximately 15,000 feet of year-round buried 2-inch and 3-inch galvanized line in the system. Much of this pipe should likely be replaced with 6-inch ductile iron; although for certain limited runs (such as the loop on Roberts Square recently replaced) 4-inch ductile iron pipe would be sufficient. These pipe replacement projects should be undertaken whenever accompanying road work is planned.

If all year-round galvanized pipe was replaced with 6-inch ductile iron pipe, using the same trench and assuming that this work was not done in conjunction with other roadway reconstruction projects, this stand-alone work could be expected to cost on the order of $75/foot, or approximately $1.125MM to complete. Since roadway reconstruction is routinely done in the downtown area as part of the Town's pavement management program, including full depth reconstructions involving storm drainage, wastewater utilities, and curbing, it makes sense to incorporate galvanized water line replacement in these projects as they occur. This would stretch out the galvanized line replacement over a significantly longer period of time, but would allow for a considerable cost savings. If water line replacement is taken into account when scheduling roadway reconstruction projects, the ranking of various projects may change to accommodate those streets with the most problematic water lines.

Poor Condition Pipe Replacement - HIGH

Other major transmission lines in relatively poor condition would include most of the 6-inch unlined cast iron pipe in the downtown area. These lines should be considered for replacement whenever related roadway reconstruction is undertaken. In recent years Spring Street (2003) and Roberts Avenue (2004) are examples of these types of projects.

Two of the poorest condition pipe runs are along Rodick Street and Holland Avenue. A portion of the Rodick Street pipe was replaced a few years ago, but a complete replacement was not made. Holland Avenue has had sidewalk and street work performed several years ago, but utility upgrades were not part of that work. Size increases are likely unnecessary, however the existing unlined pipe is likely to be heavily tuberculated leading to high headloss and diminished water quality.

The total length of these two lines is approximately 1500 feet. If other conditions warrant performing work in these areas, these streets should be ranked high on the reconstruction schedule.

Overall, there is approximately 30,000 feet of old, unlined, cast-iron, 6-inch pipe in the downtown area. If all of this pipe were replaced with 8-inch ductile iron, lined pipe as an independent, stand-alone project (assuming the pipe was set in the same trench so no ledge removal would be necessary), costs could be expected to be approximately $85/foot. This would result in a total project cost of approximately $2.5MM. In addition to the substantial cost, such an undertaking would be highly disruptive to the community, especially given the very short construction window available outside of the peak summer season.

The best way to address old, poor condition pipes is to factor their replacement into individual street reconstruction projects as is presently done. Water line condition should be considered as one of the ranking criteria for scheduling the order of individual street reconstruction projects.

Abandoned Service Replacement/Removal - HIGH

During the last several months there has been a significant increase in line breaks associated with old abandoned services. Speculation is that many of these services may have been left over from properties impacted by the 1947 fire. Apparently, discontinued services were abandoned at the curb stop, rather than being pulled from the main line within the roadway. The services, many of which are galvanized, are now apparently reaching their ultimate life span and are leaking either from the curb stop or from failure of the line. This has resulted in a considerable amount of emergency response by Division staff and a significant amount of lost water.

A review of service line records should be undertaken in an effort to identify other likely abandoned services so that they may be eliminated back to the corporation before they fail. Many of these services are unlikely to be documented and many are unlikely to be found until problems develop. The Water Division should plan on having to deal with spring seasonal problems with old services for the foreseeable future.

Bottleneck Pipe Replacement - MEDIUM

Major transmission lines with high headloss gradients represent the principal limiting bottlenecks to the distribution system. These pipe runs should be some of the first sections considered for planned replacement. Among the worst stretches are 2600 feet of 8-inch cast iron from the summer Duck Brook transmission line out Eden Street toward Hulls Cove, 750 feet of 10-inch cast iron on Mt Desert Street between Kebo Street and Spring Street, and 2400 feet of 6-inch cast iron on South Main Street between Cromwell Harbor Road and Old Farm Road.

The first two projects should consider replacement using 12-inch ductile iron, while for the South Main Street run 8-inch would be sufficient. Costs would be highly dependant upon what other road work was being undertaken in conjunction with water line replacement, the amount of ledge encountered, and whether the existing pipe trench could be used.

Seasonal Line Conversion - LOW

In conjunction with decisions on storage in the Hulls/Salisbury Cove area, decisions should be made on the conversion of seasonal lines to year-round service in areas where housing stock is being converted to year-round use and where growth is anticipated. Conversion of seasonal lines can be viewed as piecemeal projects, undertaken as part of other road upgrades, or a push can be made for the specific purpose of converting a wider area.

Some areas of summer service, such as Degregoire Park contain year-round residences that revert to on-site wells during the winter season. Community input should be solicited to determine the interest these residents may have for remaining on the water system year-round.

There are approximately 14,475 feet of seasonal lines (not including transmission lines) in the Bar Harbor system. The great majority of this footage would likely be replaced with 6-inch ductile iron line if a large scale conversion to year round service was made. At typical costs of $50 to $75 per foot for line replacement (depending primarily on the amount of ledge encountered) the overall cost for construction could range from $725,000 to $1,100,000. Obviously, seasonal line conversions would take place in relatively small increments on a year to year schedule. Some sections of seasonal pipe would likely remain as part of the seasonal system for the foreseeable future.

4.2.3 Special Projects

Pump Station Pump Equipment Upgrade - HIGH

The Duck Brook pump station upgrade should be undertaken regardless of other system modifications that may or may not be made. This pump station will continue to provide a necessary increase in the summer hydraulic grade line for the foreseeable future. The present pump equipment, the electrical service, and instrumentation associated with this system have been sources of reliability concerns for several years.

Preliminary designs developed four or five years ago for changes to the pumps are somewhat obsolete since the decision was made to install the cast-in-place storage tank. However, the underlying idea to reduce the pump rate used to fill the summer tank is still very sound and should be pursued in order to minimize the impact on the CT achieved in the transmission line.

Changing the pump rate will also reduce energy waste from increased headloss in the summer tank piping.

The $139,000 original estimate for pump station work submitted as a State Revolving Fund project in 2000 is likely to be low, given that the project should be expanded to include an electrical service upgrade and controls improvements and given the passage of time. We would recommend that $180,000 be allocated toward this work. This work can only be undertaken outside the summer pumping season. It could likely be completed within 60 days of delivery of the necessary equipment.

SCADA System Upgrade - HIGH

Installation of a SCADA system is somewhat tricky in regard to timing. The water system has very definite needs for certain elements of a modern data acquisition and management system right now and will benefit from better remote control of the system in the near future. The key is selecting and installing a system that is easily expanded to meet future needs, while not over investing in a system that may become obsolete prior to being fully utilized.

For the present, the Water Division should consider instrumenting each of its storage tanks and having those signals transmitted to the Division office. Chlorine residual, flow, turbidity, pH and other data presently collected at the screen building and the pump station should be transmitted to the office as well and all this information should be input into a desktop computer for display and trending. CT computations should be continuous and be done automatically. Pump start and alarm information should also be collected and displayed at the office, and the autodialer dial out alarm system should be simplified with off-the-shelf equipment. In the future, remote control and start up functions can be added.

Similar instrumentation projects presently underway, incorporating simple data monitoring and alarms have cost on the order of $65,000-$85,000, including engineering and installation. We would suggest that a similar expenditure would be appropriate here. This upgrade should be undertaken in the near future, principally to provide better monitoring of the CT achieved by the system.

Hypochlorite Disinfection Conversion - MEDIUM

Conversion to the use of hypochlorite is primarily a safety issue. An interim conversion could be undertaken in short order, but will likely require the use of hypochlorite stored in individual drums, frequent deliveries, and substantial handling of the product. Bulk storage would likely require building modifications and improvements in road access to the screen house site.

The Water Division may wish to consider whether it wants to make this conversion in the near future or hold off until such time as decision is made on a conversion to UV for primary disinfection. With the reduction in hypochlorite use that will occur at that time, it will then be possible to install a smaller hypochlorite system. This decision will hinge on the Division's continued ability to purchase chlorine gas cost effectively and to meet the increasing reporting, training, and monitoring requirements.

The capital cost for conversion will be highly dependant on whether the changes can fit within the existing structure. Changeover will require elimination of the gas chlorinators as well. If the necessary drum storage, scales, pumps and equipment can be installed within the existing building, capital costs would be on the order of $15,000, primarily for purchased equipment that the Division could install on its own.

North End Year-Round Storage Tank Installation (Dreamwood Hill) - MEDIUM

A new, year-round storage tank in Hulls Cove, together with the piping and pumps to serve it, is the single largest project the Water Division should consider in the near future. At this point, the details on pipe routing and seasonal system conversion are wide open and there needs to be considerable public input and discussion on the alternatives and their implications. Improvements to the water system in this area could potentially open up a significant amount of land to development.

The Water Division does own a site that would be suitable for a new standpipe, and this location would be the preferred location for a storage tank under any of the potential options. Tank size and materials of construction have not been finalized; however it is likely that storage on the order of 500,000 gallons should be considered. Since a full-tank water height of approximately 70 feet should be considered in order to properly serve the largest possible area, this dictates that the diameter of the standpipe be approximately 35 feet. This in turn sets the materials of construction, since it is not economic to construct a concrete tank to these dimensions.

A glass-lined, bolted steel tank of this volume and dimensions would cost on the order of $400,000, plus the necessary site work. A booster pump station would also be required. Control valves, pressure regulating valves, instrumentation, and piping improvements would be additional and would be dependant on the option and route chosen.

The overall project could easily reach twice the cost of the standpipe, and upwards of $1,000,000 if a cross-country pipe route is selected. To a large degree, it will be necessary to complete all planned piping changes in conjunction with the standpipe construction, since a new standpipe will be too large to simply function as a direct replacement for the existing seasonal tank without creating water quality issues.

UV Disinfection Conversion - MEDIUM

Promulgation of the LT2 regulation has been held up numerous times. It could be several years before it becomes the driving force towards disinfection conversion to UV treatment. A more immediate driver may be local aesthetic concerns with the taste and odor of the chlorinated water. UV guidance is still in the process of being finalized and there are a number of design, testing, and certification standards for UV equipment that have not yet been fully adopted. Bar Harbor may not wish to be an early adopter of a new technology.

The time frame for wide-spread adoption and installation of UV disinfection by small water utilities is likely to be within the next three to four years. Presently costs for equipment are relatively low since many manufacturers are trying to position themselves for the coming market by having demonstration installations up and running. Future cost trends are difficult to predict. Demand for equipment will be high, but production and research and development costs will be spread much broader. There are several firms in active competition in this market.

Recently quoted costs for UV equipment for a planned project the size of Bar Harbor's was on the order of $300,000. Piping, electrical, and building modifications and engineering would likely double the equipment costs. Bar Harbor should consider waiting four to five years before considering the expenditure of the half to three-quarters of a million dollars required to make this type of conversion. As noted prior, transmittance testing should begin now so that the efficacy of this alternative can be determined.

GIS/Smart Mapping - MEDIUM

One follow-on step to the planning and data management activities undertaken by the Water Division would be to develop a database of all system information, including valve locations and data, hydrants, pipe information, curb stops, meter sizes, etc.. This information could be stored within a database associated with a smart map of the water system. Such a map would give detailed location data for the system and be tied into a Town developed GIS system.

Prior to finalizing the scope of the present master plan, it was discussed that the Town was in the process of developing such a system for all infrastructure for planning, storage, and emergency response. The Water Division would certainly benefit from such a comprehensive system, and this next step should be pursued. The system's pipe network model is also very adaptable for inclusion into such a system. Cost would be dependant on the number of layers of infrastructure developed for the system, features required, whether such a system is developed all at once or is phased, and particularly upon the community's ability to oversee and maintain the system in-house rather than requiring the on-going assistance of outside consultants.

Membrane Filter Plant Construction (Including 3000 ft sewer extension construction) - LOW

As noted in the report, this option is evaluated only in the interest of completeness. The Water Division's first goal should be to avoid the need for filtration through careful attention to maintaining the conditions of its waiver from filtration.

If filtration became necessary, one of the few options which would be effective in this application would be direct membrane filtration. Since solids concentrations in the source water are very low, there would be little need for a pre-filtration chemical addition and flocculation step. The biggest complication to a membrane filtration facility would be the disposal of the backwash wastestream, a problem common to any type of filtration suitable to systems of Bar Harbor's size.

Woodard & Curran recently designed a 3 MGD membrane treatment facility in Walpole, Massachusetts that is presently under construction. This facility does include a number of pre-filtration chemical addition and settling steps. Total treatment project costs are approximately $7,000,000. It is estimated that strictly membrane treatment can be accomplished for $1.60-$1.80/gallon of plant size. This would place the cost of a 4 MGD membrane treatment facility (note: full 4 MGD design size considered for long-lived, capital intensive projects, 3 MGD for shorter-lived, smaller projects such as disinfection conversions) for Bar Harbor at $6.4MM to $7.2MM. In addition, it is assumed that a 3000 foot forcemain to the sewer system would have to be installed, much of which would pass through shallow or surface exposed ledge. This could easily add an additional $250,000-$400,000 to the project.

If a filtration facility became necessary, it should include disinfection treatment using UV for primary disinfection and hypochlorite for secondary disinfection relocated from the lakeside screen building to the new plant site.

Fluoride Conversion - LOW

Conversion from the use of fluosilicic acid to sodium fluoride is recommended to be undertaken only if a unified filtration facility were to be constructed. The principal advantages of this conversion are avoidance of a pH drop in the treated water and a lessening of the safety and corrosive concerns of handling fluosilicic acid.

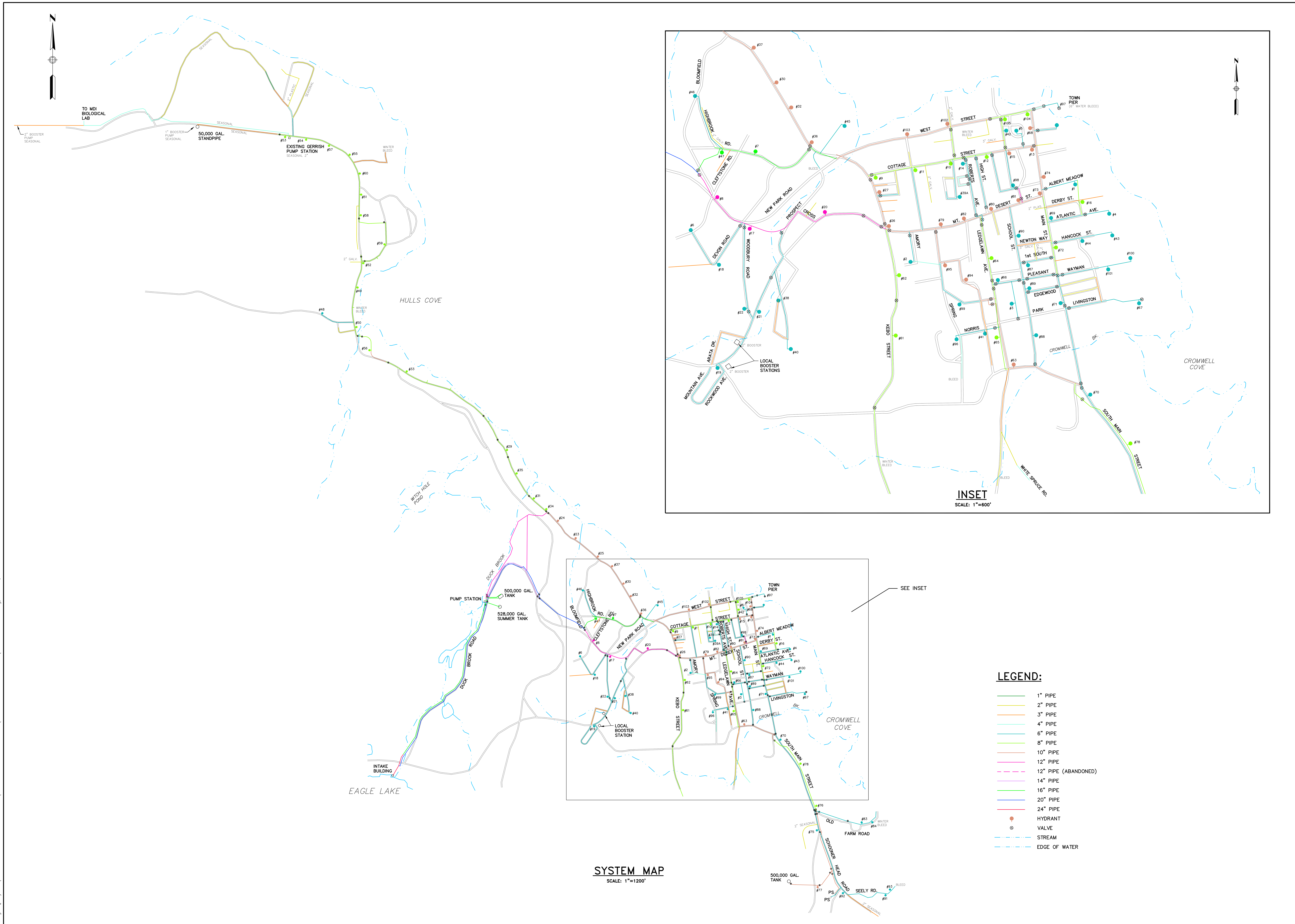
Installation of a saturator and storage area for dry sodium fluoride would be incidental to the costs of sizing and constructing a filtration facility.

4.3 SUMMARY

The proposed management action plan is summarized in the following table. This table highlights the relative priority of each project, a timeframe for undertaking it, and an anticipated planning cost. Several projects are listed as "on-going", reflecting the fact that they are likely to be undertaken as other projects such as roadwork drive them.

Table 8: Action Plan Project Summary

	Project	Importance	Driver	Timeframe	Budgetary Cost
1	Abandoned Transmission Line Lead Abatement	High	NPS	TBD	Unknown (depends on method)
2	UV Transmissivity Testing	High	Future UV Conversion	2005-2006	$5,000 - $10,000
3	Galvanized Line Replacement	High	Repairs/Water Quality	On-Going	$1.125MM as stand-alone project, less if incorporated in scheduled roadway reconstruction projects
4	Poor Condition Pipe Replacement	High	System Capacity/Repairs/Water Quality	On-Going	$2.5MM as stand-alone project, less if incorporated in scheduled roadway reconstruction projects
5	Abandoned Service Removal	High	Leakage/Repairs	On-Going	Unknown
6	Duck Brook Pump Station Upgrade	High	Waiver Maintenance/Reliability	2006	$180,000
7	SCADA System Upgrade	High	Waiver Maintenance/Reliability	2006	$65,000 - $85,000
8	Dam Repair Evaluation	Medium	Safety/Supply Protection	Summer 2005	$8,000
9	Dam Repair	Medium	Safety/Supply Protection	Dependant on Report Findings	Unknown (depends on report findings)
10	NPS Land Discussions	Medium	NPS	2005-2006	-
11	Bottleneck Line Replacement	Medium	System Capacity/Repairs	On-Going	$410,000 if main projects fully completed
12	Hypochlorite Conversion	Medium	Safety/Reporting	2007	$15,000
13	North End Storage Tank Installation	Medium	System Capacity/Growth/Water Quality	2008-2009	$1 MM
14	UV Disinfection Conversion	Medium	Regulatory/Aesthetics	2009	$600,000
15	GIS/Smart Mapping	Medium	Planning	2006-2007	TBD after discussion w/other Departments
16	Seasonal Line Conversion	Low	Development/Comp Plan	On-Going	$725,000 - $1.1MM (If entire conversion made)
17	Filtration Facility Construction	Low	Regulatory	Unknown	$6.6 – $7.6 MM



WOODARD & CURRAN
Engineering · Science · Operations
BANGOR, MAINE
800-564-2333

REV	DESCRIPTION	DATE
1	MISC. LABELS	02-01-05

DESIGNED BY: TEN/RHH CHECKED BY: RHH
DRAWN BY: NTD 20321501-U001.dwg

SYSTEM MAP

TOWN OF BAR HARBOR
WATER DIVISION

WATER SYSTEM MASTER PLAN

JOB NO.: 203215.01
DATE: APRIL 2005
SCALE: AS NOTED
SHEET: OF

FIGURE 1

\\Bangor\projects\203215 Bar Harbor Master Plan\01 Bar Harbor Master Plan\4-2005 REPORT FIGURES\20321501-U001.dwg, Mar 16, 2011 - 9:40am