Village of Chester
Water System Study
Final Public Report

170807.02 ● Final Public ● June 2018

Prepared for:

Prepared by:

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Solving today's problems with tomorrow in mind

June 27, 2018

Matthew Davidson, P.Eng.
Director of Engineering
Municipality of the District of Chester
PO Box 369
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Chester, NS B0J 1J0

Dear Mr. Davidson:

RE: Water System Study Final Public Report

Please find enclosed the Final Public Report for the Village of Chester Water System Study. This report includes the evaluation of three water supply options and associated treatment options, treated water storage siting and preliminary design parameters, and layout options for treated water transmission and distribution.

We look forward to discussing the contents of this report with you and providing with you a final report.

Yours very truly,

CBCL Limited

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Project No: 170807.02

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A Jozsa Management & Economics “Socio-Economic Effects of a Central Water System for the Village of Chester”

EXECUTIVE SUMMARY

Homes and businesses in the Village of Chester currently draw drinking water from private wells. A survey of well owners indicated that water quantity and quality problems are common throughout the Village. A central water supply would provide a clean, safe and reliable source of water, substantially improving the consistency, quality and access to water for many residents and businesses of the Village. The Village of Chester and the Municipality of Chester has asked CBCL to conduct a high level engineering review to demine construction, operation and maintenance costs of a potential centralised water system compared to upgrading the existing private wells to meet current standards.

This report discusses three potential water supply options for the Village. Development of a surface water source, development of a ground water source, and maintaining the status quo, i.e. private wells. The surface and ground water systems differ in terms of water source location, treatment requirements and reservoir location. They are very similar in terms of the water pipe distribution network. The status quo water supply system would involve maintaining the existing level of service by upgrading the existing wells. Preliminary design parameters were developed for each option, then the capital, operational and maintenance costs for each option were compared.

Water Supply Option 1: Centralised Water from a Surface Water Source

Spectacle Lake is located to the north of the Village and has been identified as a potential surface water source. Previous reports indicate Spectacle Lake likely has adequate capacity to supply water to the Village, however further analysis is required to confirm the lake and discharge stream capacity. A source watershed protection plan should be developed and implemented for Spectacle Lake if it is chosen as the preferred water source for the Village.

Raw water would be extracted from Spectacle Lake by gravity, then pumped to a treatment plant and reservoir located on the high ground near Bond Drive. A municipally owned property has been identified as a potential location for the pump house, treatment plant and reservoir.

Three suitable water treatment processes have been identified; Dissolved Air Flotation, Ultrafiltration and combination Ultrafiltration & Nanofiltration process. These processes are in use elsewhere in the Provence and indicate relatively similar life cycle costs. A preferred process would be selected from these three options at the preliminary design stage.

Water Supply Option 2: Centralised Water from a Ground Water Source

A potential source for groundwater has been identified in Middle River near Lower Grant Road. Exploration work and aquifer testing would be required to quantify the available yield and quality from this location. Potential drilling sites were selected based on geology mapping, thickness of granular material as indicated by water well logs, potential yield as indicated by the geology and airlift yields and site access.

Water would be extracted from the well field and treated on site. Water treatment would likely involve iron and manganese removal, pH adjustment and corrosion inhabitation. There are a number of potential processes to achieve these treatment goals. Oxidation then filtration would likely be recommended as it is commonly used, relatively simple and represents low capital cost.

From the treatment plant water would be pumped to a reservoir located on high ground near the Village. The Haddon Hill area is a potential reservoir site which would work well for a ground water system.

Option 1 & Option 2: Common Elements

The water distribution network for option 1 and option 2 will be similar. Water would likely be supplied to potential customers along the pipe route from the treatment plant to the Village. The surface water option could service customers on Old Trunk 3 while the ground water option could offer water to customers along the Rails to Trails route.

This report assumes that both systems will service the full extent of the Village boundary, however there are some areas within the boundary which demonstrate lower practicality for a centralised water system than others. These areas include lower density developments, locations where construction costs are anticipated to be high or where construction will result in significant loss of property access. These areas include Haddon Hill, the peninsula and lands towards the golf course.

There are also areas outside the village boundary which may warrant water service. The mall has been included within the service zone for both systems based on its socio economic value to the community.

Fire Protection

It is understood that the primary driver for a centralised water system is to provide a clean, safe and reliable source of water for the Village. A secondary goal would be to provide water for fire suppression for the Village. The Fire Underwriters Survey (FUS) publish guidelines for municipal water fire suppression flow and volume. Applying the FUS design criteria to the proposed water system will increase the required pipe and reservoir size, having significant cost implications. For example the total cost of the surface water system, which includes fire suppression water in accordance with FUS is estimated to be approximately $38,715,000, that cost can be reduced by approximately $3,400,000 if the system would be designed to provide potable water only.

It is interesting to note that the "Socio-Economic Effects of a Central Water System for the Village of Chester" attached as Appendix A identifies that installing a central water system with fire protection will reduce insurance premiums over a 25 year period by $8,535,328 in 2018$. This combined community cost savings more than offsets the cost of constructing a central water system with fire protection for the community.

However, it is understood that the village has a reliable tanker shuttle service in service to deliver firefighting water. If the level of service for the tanker shuttle service is accredited "superior", the shuttle service is considered equivalent to hydrant protection from a municipal water supply. It is recommended that firefighting goals for the village be discussed with the local branch of Fire Underwriters Survey, local fire fighters, and municipal staff.

Water Supply Option 3: Maintain Status Quo

Currently property owners typically have either a dug well or a drilled well. A previously study completed by CBCL identified a number of issues with private wells within the Village. Many dug wells do not meet current construction standards, 23% of survey respondents have been affected by water shortages, and water issues were identified in 85% of water samples and 62% of water samples contained coliform bacteria. The identified construction issues and presence of coliform bacteria are an indication that unsafe drinking water is being consumed by the residents.

Each private well and system is different and water treatment processes should be assessed on a case by case basis, however, a reverse osmosis treatment would likely be recommended for the majority of dug wells. An iron and manganese treatment systems would likely be recommended for drilled wills.

Each well system would conceptually be replaced or upgraded over a 25 year design life. As a large portion of the wells do not meet current standards it is assumed those will be upgraded in line with current practices. Dug wells which experience water shortages would likely be replaced by drilled wells. It is anticipated that some residents will continue to experience water shortages. Replacing dug wells with drilled wills may potentially move water shortage issues from one property to the next as each property owner drills deeper into the water source, potentially drawing the water table below their neighbours well.

Lifecycle Cost Comparison

A life cycle cost evaluation of the three water supply options is provided below.

Lifecycle Cost Evaluation

Item	Surface Water Supply	Groundwater Supply	Private Well Supply
Capital Budget	$38,715,000	$40,451,000	$5,270,000
Present Value of O&M	$4,650,000	$1,700,000	$20,500,000
Present Value (Total)	$43,365,000	$42,151,000	$25,770,000

Note the capital costs do not include financing or government funding

It is important to note that the central water system options are eligible for government funding where private wells are not. It is expected that 75% of the capital cost could be funded from provincial and federal governments. The Socio-Economic report in Appendix A (Section 3.1) conducts a financial comparison of the central water supply options versus the upgrade of existing onsite wells and identified that installing a central system provides a higher net benefit to the community compared to upgrading the existing private wells.

Once funding and financing have been added to the financial the central groundwater option becomes the lowest cost central water system at $24,027,826 compared to $26,179,117 for a surface water system. The cost of the ground water system become $27,151,450. All options are relatively close in cost. However, once the benefit of reduced fire insurance premiums ($8,535,328) provided with the central systems are accounted for the central systems provide a much higher benefit to the community compared to upgrading the existing wells.

Next Steps

In order to move forward the community needs to determine if a centralised water system is desired, and if so select the preferred source for the water system.

CHAPTER 1 INTRODUCTION

1.1 Background

The Village of Chester (Village) is located within the Municipality of the District of Chester, in Lunenburg County, Nova Scotia. Homes and businesses in the Village draw water from private wells, and discharge sewage to either private onsite sewage systems or to the municipal collection system. The density of wells in the area is relatively high, particularly within the Village centre.

The Municipality is seeking to determine whether homes within the Village boundary would benefit from a municipal central water supply and what the cost of providing central water service would be. A central supply would provide a clean, safe and reliable source of water to residents and businesses in the community. A demonstrated need for transition from individual supplies to central water would be based on the security, reliability and quality of existing water sources.

In August 2017, CBCL Limited completed a preliminary assessment of the existing private wells serving the residents of the Village. A survey of well owners indicated that water quantity and quality problems are common throughout the Village and that it is very likely that unsafe drinking water is being consumed by residents. The study indicated that a central water system would substantially improve the consistency, quality and access to clean safe water for many residents of the Village. The report recommended a socioeconomic analysis to evaluate the social benefits and costs of a central water system to allow staff to make a more informed decision on a potential central water system. This report presents water service options with opinions of probable construction and operating cost which are used in the development of the socioeconomic assessment that is presented in Appendix A.

1.2 Existing Conditions

Land use within the Village boundary is predominantly residential, with the greatest density of homes located within the Village centre. Residential complexes to the east of the Village centre include the Shoreham Village senior’s complex and the Chandler’s Cove condominium complex. Light commercial activity (office and retail) is concentrated along Highway 3 / Lighthouse Route / North Street, and in the southeast part of the Village centre (Duke, Pleasant, and Queen Streets). Public facilities in the eastern part of the Village include three schools, a recreation complex, and the Chester Golf Club on the peninsula of Chandler’s Cove. Open space, parkland, undeveloped wooded areas, and the southern tip of Stanford Lake comprise the outer parts of the Village, to the north, west, and east. There are two cemeteries, one on Brunswick Street and one on Golf Course Road. There is light agricultural activity in the eastern-most part of the study area, and surrounding the communities of Marriott's Cove-Haddon Hill-Robinson's Corner. Refer to Figure 1.1 for village boundary.

CBCL previously completed a review of private wells within the Village. A number of issues were observed with related to construction of dug wells, including the use of inadequate covers and lack of a concrete apron below the ground surface to prevent short circuiting from the ground surface to the well bore.

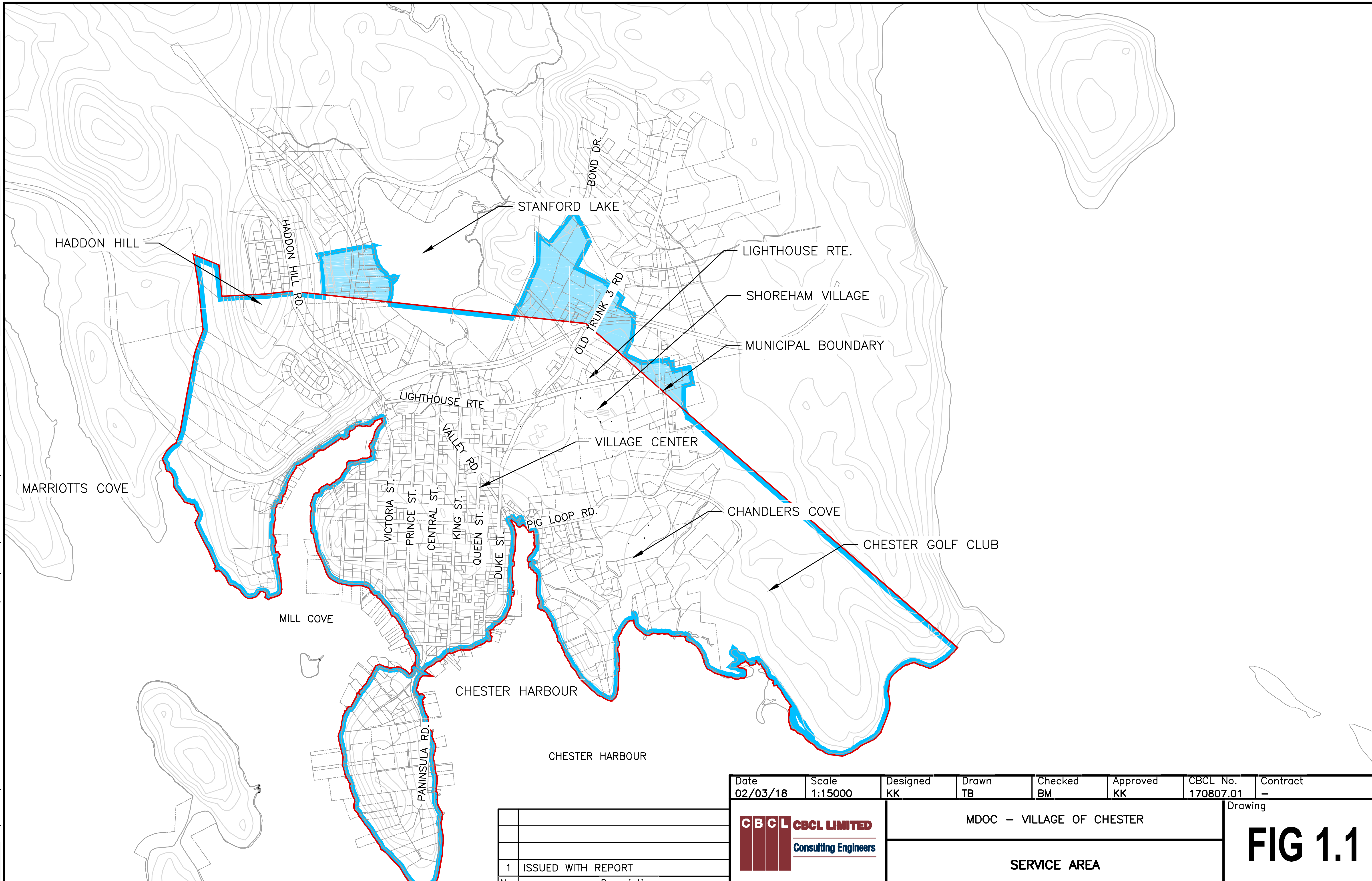
Potential land uses of concern for private well water supplies are limited to a cemetery located on the southwest part of the Village centre, and an Irving Fuel Station at the intersection of Highway 3 and Duke Street. Nova Scotia Department of Transportation and Infrastructure Renewal operates a depot on Lighthouse Road, near the intersection with Marina Drive. The Village's sewage treatment plant is located on Nauss Point Road, 300 metres south of the recreation centre.

The study also included a survey, where 23% of respondents reported having experienced water shortages. Dug well owners reported extensive measures to conserve water; in spite of conservation measures some respondents experienced an interruption in supply for up to four months in 2016. Shortages demonstrate that water resources are stressed in localized parts of the community, and that the potential for further development and onsite improvements is limited as a result.

1.3 Study Objectives

The objective of this project is to investigate options for providing safe and reliable water to the residents and businesses within the Village of Chester. Specifically, the report will include the following tasks:

- Review of background data.
- Estimate water demands based on future population and development projections.
- Identify an appropriate water source.
- Establish preliminary treatment requirements and mechanisms.
- Complete a high level design of treatment, storage, transmission and distribution systems.
- Identify critical or high cost infrastructure items.
- Estimate order of magnitude, preliminary capital and operational costs for each water supply option.
- Conduct a social-economic analysis (Appendix A).



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Date	Scale	Designed	Drawn	Checked	Approved	CBCL No.	Contract
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MDOC – VILLAGE OF CHESTER

SERVICE AREA

Drawing

FIG 1.1

DRAWING NAME: K:\PROJECTS\170807.01 MDOC-VILLAGE OF CHESTER CENTRAL WATER\20 CAD\01 CIVIL\SERVICE BOUNDARY\CONTOUR_MAP_SERVICE ZONES.BAK.DWG LAYOUT NAME: FIG 1.1 SERVICE AREA PLOT DATE:March-02-18 2:17:31 PM CAD OPERATOR: TBUCHER

CHAPTER 2 LONG TERM WATER SUPPLY REQUIREMENTS

2.1 Methodology

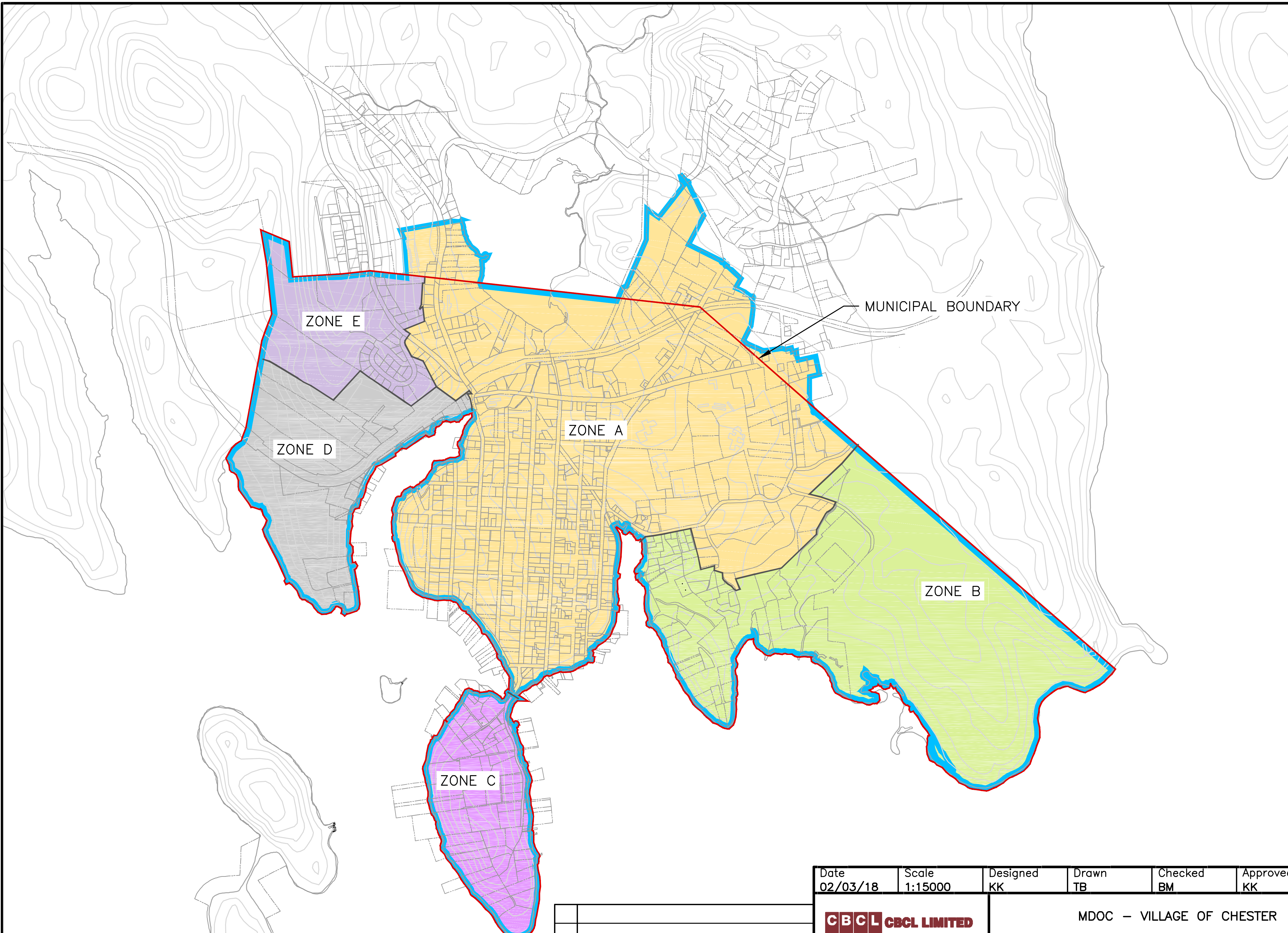
Identifying an appropriate water demand is a critical factor in the development of a central water supply as it dictates the requirements for the water source, treatment and distribution systems. Water demand is based on the number of water users and how much water they use, with allowances for operations, leakage and fire suppression if required. Establishing an accurate demand is of prime importance as the system must have enough capacity to meet the demands of the users while not being oversized which can lead to unnecessary capital expenditure, maintenance and operational issues.

2.2 Service Boundary

The general goal for the central water system is to provide water service to users within the Village boundary. The Village boundary encompasses an area of approximately 390 ha and is shown in red in Figure 1.1. Within the Village there are a variety of different land uses which represent a broad spectrum of consideration in terms of water servicing. We have based this report on the assumption that the water system will service all areas within the Village boundary.

For the purposes of this report the service area has been broken into a number of different zones, illustrated on Figure 2.1. A description of each zone and some servicing considerations for each zone are generally discussed in Chapter 5. Zone A, includes the village core, some surrounding high density residential developments, commercial and institutional uses. Zone A represents approximately 85% of the water system demand and between 75 and 80% of the cost of the distribution system. Zone B, C, D and E represent 15% of the water demand and 20 to 25% of the cost of the distribution system. If development is to occur within the Village it is likely that it will occur outside of Zone A as there is limited space within Zone A to infill. As population increases outside Zone A the demand within Zone A will decrease relative to the total demand of the water system.

The intent to supply all areas within the Village boundary with water may warrant further consideration. Deterrents to offering water supply include low density, access issues and cost. There are also areas outside, but adjacent to, the Village Boundary which warrant consideration to be serviced, these include the mall, properties along proposed water pipe routes and properties currently serviced with wastewater. Water supply to these areas has also been accounted for in this report with costing identified separately for properties outside the Village boundary.



LEGEND

- PROPOSED SERVICE AREA
- MUNICIPAL BOUNDARY
- SERVICE ZONE BOUNDARY
- VILLAGE CORE
- LOW DENSITY
- PENINSULA
- BOTTOM OF HADDON HILL
- TOP OF HADDON HILL

SEE COSTING TABLE FOR ZONE BREAKDOWN

No.	Description
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MDOC – VILLAGE OF CHESTER

LAND ZONING

Drawing

FIG 2.1

Due to the significant capital cost of this project it is unlikely that the full service boundary will be provided with water from the onset. The project will likely proceed in phases. The limits of servicing can be adjusted through future design, however the available supply must be considered when adjusting servicing limits.

2.2.1 Additional Service Area

Outside the village boundary there are some properties which may warrant servicing. For example, it is likely that the reservoir will be located outside the village boundary, potentially near Bond Drive, or Haddon Hill. In both cases it would be a logical and financially reasonable undertaking to service properties along the pipe route between the reservoir and the village boundary.

The mall represents a socioeconomic asset to the Village. It may be in the Village's best interests to include this asset within its water service boundary.

Potential extensions to the service boundary are shown in Figure 1.1, the village boundary is shown in red and the service boundary is outlined in blue.

2.3 Water Use

When evaluating sources for a long-term water supply it is important to account for future projected population and water needs of the planned ultimate service area. As stated in the Atlantic Canada Design Guidelines for the Supply, Treatment, Storage, Distribution, and Operation of Drinking Water Supply Systems (Atlantic Canada Guidelines), the treatment system should be capable of supplying the projected maximum day demand for the 20 to 25 year projected design flows. A 25 year design timeline has been used in this report.

2.3.1 Domestic Water Use

Residents and businesses within the Village obtain water from private wells, there is no available design flow for residential water consumption therefore water use must be estimated. Published per capita consumption in the Atlantic Canada Guidelines as well as Halifax Water Guidelines are summarised in the table below.

Atlantic Water Canada Guidelines:

- 1000 liters per day (L/day) for a 3 bedroom home.
- 1200 L/day for a 3 bedroom home with high use fixtures.
- 1350 L/day for a 4 bedroom home.
- 1500 L/day for a 4 bedroom home with high use fixtures.

Atlantic Canada Wastewater Guidelines:

Waste Water Generation per person in a private Dwelling is 340 l/day. It is generally considered acceptable practice to directly compare wastewater generation to water consumption. 340 l/day has been used herein as the design consumption rate for the Village.

Halifax Water:

- Average Day Demand 410 liters / person / day.
- Based on: 3.35 people per single unit dwelling.

For the purposes of this assessment as per person demand of 340 liters/person /day has been assumed based on Atlantic Canada Wastewater Guidelines. It is important to note that per capita water use rates have been trended downward across Canada. In some situations so much so that total water demand can often be seen to decrease in the face of population growth. This is often attributed to low flow fittings and changes in water use patterns. The 340 liters/person/day is a conservative water demand.

The Village population, based on the 2016 census, is estimated to be 1460 people. The village boundary does not quite match up with the census tracts, however the overlap appears to be within an acceptable margin of error for this level of analysis. Population growth within the proposed water service boundary is estimated at 2% per year. This growth rate is higher than the estimated growth rate of 0.4% as estimated in the Jozsa report, Section 3.3.1.3.3 of Appendix A, indicated the Village population could expand to 1,553 with central water and could decline from 1,369 if private wells are maintained over the next 20 years. The 2% used for the sizing of the source water is a typical engineering value used to provide safety in the design of the supply. Population and domestic water demands have been assumed to grow as follows.

Year	Population (Capita)	Growth (%)	Per Capita Demand (Liters/capita/day)	Domestic Demand (m^3/day)
2018[1]	1,489	2%	340	506
2028	1,815	2%	340	617
2043	2,395	2%	340	814

Note 1: The population in 2018 was based on the 2016 population of 1460 at a growth rate of 2% per year.

2.3.2 Non Domestic Water Use

Beyond domestic water demand, additional sources of demand typically include commercial, industrial and institutional demand.

There are approximately 300 elementary school and 450 middle school students within the Village boundary according to the South Shore Regional School Board published Long Range Outlook (2015). Atlantic Canada guidelines for school flow vary from 50 liters to 115 liters per student per day. An allowance of 60 liters per student per day has been utilised herein to develop an institutional demand of 45m^3/day.

Within the village there are a number of businesses which will be supplied water. It is understood that in some cases the businesses will only be supplied with potable water while other water demands will come from other existing sources. For example the golf course irrigation system will not be connected to the village water supply system, however the clubhouse and other domestic requirements will be.

Approximately 13 hectares of general commercial land use has been identified within the service boundary to be supplied with potable water. The majority of commercial businesses within the Village are service industry based. There are no high water demand business within the village boundary known to CBCL at this time. Examples of high water use business include canning facilities, food processing plants, automatic car washes and breweries. An equivalent residential population of 85 persons per ha for general commercial use is noted in the Atlantic Canada Waste Water Guidelines. Considering the existing established commercial activities within the water service boundary the Atlantic Canada guideline number of 85 persons per ha is considered conservative. CBCL has previously utilised 45 persons per ha, this rate has been established from previous experience with similar types of commercial demands. Based on a 340 liters/cap demand and an equivalent population of 548 persons the current average daily commercial demand is estimated to be 186 m^3/day.

Total Potable Water Demand Estimates based on a 2% growth for residential, institutional and commercial growth is presented below:

Year	Domestic Demand (m^3/day)	Institutional Demand (m^3/day)	Commercial Demand (m^3/day)	Total Potable Water Demand (m^3/day)
2018	506	45	186	737
2028	617	54	226	897
2043	814	73	305	1,192

Equivalent Population:

Year	Domestic	Institutional	Commercial	Total
2018	1,489	132	547	2,168
2028	1,815	161	665	2,641
2043	2,395	216	897	3,508

Peaking factor and max day demand:

Year	Equivalent Population	Average Day Demand (m^3/day)	Peaking Factor	Max Day Demand (m^3/day)
2018	2,168	737	2.25	1,658
2028	2,641	897	2.25	2,018
2043	3,508	1,192	2	2,384

2.3.3 Demand Design Value

The 2043 calculated demand is based on a consistent 2% population growth and constant per capita water demand and an equivalent population of 3,508 people. As discussed earlier trends in water consumption have been shown to offset population growth in many water systems throughout Canada. Also, the Jozsa report (Appendix A Section 3.3.1.3.3) predicts a population of 1,553 in 20 years which is approximately 900 people less than the population based on a 2% growth rate. Based on historical population growth in Chester, and water consumption throughout Canada, it is very unlikely that the 2043 demand, at 2% population growth, will be realised.

Therefore, the water treatment, storage and transmission systems discussed herein are designed to meet the 2018 demands plus an allowance for an annual increase in population of approximately 0.5%, over the 25 year design life from 2018 to 2043. Based on these assumptions the 2043 Max Day Demand is estimated to be 1,900 m^3/day, with a population of 1,687 residents, which will be used to evaluate treatment, distribution, and storage infrastructure to offer more realistic capital and operational costs for the treatment system for the purposes of this assessment.

CHAPTER 3 SOURCE WATER EVALUATION

Three water sources have been evaluated as follows:

- Surface Water (Spectacle Lake).
- Central Groundwater Supply.
- Status quo – continue to use onsite wells.

The following section describes potential water quality, yields and benefits of each source water option.

3.1 Option #1 – Spectacle Lake Surface Water Supply

3.1.1 Watershed Description

The Spectacle Lake Watershed is described in detail in the hydrological study completed by Earth-Water Concepts (2011). The reported noted that the Spectacle Lake sub-watershed is 3.38 km long, 0.5 km wide in the north and 1.46 km wide in the south, and has a surface area of 2,566,640 m^2. The lake has a surface area of 302,000 m^2 and a shoreline of 4,000 m. The lake has two basins with a maximum depth of approximately 18 m, and is estimated to contain approximately 2,172,280 m^3 of water. The lake discharges from a single outlet channel located at the southwest end. A control structure was previously located at the discharge channel, but appears to have eroded. The discharge flow from the lake varied from 1,035 m^3/d during dry periods to 38,970 m^3/d during wet periods from November 2007 to December 2008.

3.1.2 Yield

A number of previous reports have recommended Spectacle Lake as the preferred surface water supply, because of its available yield, location, elevation, and quality. A 2011 study by Earth-Water Concepts estimated the available yield and analyzed raw water quality. Streamflow data from the lake's outflow channel was collected over a period of 40 months between October 2007 and December 2011, and was used to generate a synthetic long-term streamflow record. Flow duration curves from the two datasets were then used to estimate a 95% exceedance flow, which is the guideline value for allocating withdrawals from rivers or lakes in Nova Scotia. The Municipality continues to collect lake level data and this information would be available to for suture yield assessments if required.

The study suggested the withdrawal allocation could range between 1,200 m^3/d to 1,750 m^3/d based on the model results and filed data if no changes are made to the existing outlet structure of the lake. The report suggested using a value of 1,475 m^3/d as the withdrawal allocation with no changes made to the existing outlet structure. The estimated initial maximum day water demand of 1,658 m^3/d fits within this range but is greater than the suggested value of 1,475 m^3/d. The 2043 maximum day demand of 1,900 m^3/d exceeds this range.

The report indicates that the withdrawal allocation from the lake could be increased to between 1,750 m^3/d and 2,175 m^3/d with a conservative withdraw allocation being 1,960 m^3/d if a flow control dam is constructed at Spectacle Lake. Based on the completion of a flow control structure there would be sufficient capacity to support the 2043 maximum day demand of 1,900 m^3/d, based on 0.5% growth rate. If the population growth rate is higher the future maximum day demand will also be higher. Based on a 2% growth rate the future maximum day demand would be 2,384 m^3/d which exceeds the withdrawal allocation from the lake.

It is interesting to note that the report identifies a large degree of uncertainty with respect to the safe withdrawal allocation. The estimated current and future maximum day demands fit within the ranges provided. A water withdrawal approval should be made following the NSE approved methodology to confirm lake yields to remove this uncertainty from the evaluation process.

Changes to the lake water level, which would occur if an outlet flow control structure was constructed, will impact the surrounding and downstream aquatic and terrestrial environments, so approvals will be required from the Department of Fisheries and Oceans (DFO) and NSE. This approval will require an evaluation of the existing aquatic and terrestrial habitat and species with an assessment of the potential impacts the proposed works may trigger.

Other approvals required for a surface water treatment facility will include:

- Yield assessment to comply with 2016 NSE Withdrawal Guidelines.
- Approval to Construct.
- Approval to Operate.

For the purposes of this report, it has been assumed that the existing discharge channel will be used in its current condition. Further evaluation of the discharge stream should be conducted as part of a pre-design study to determine whether additional storage might be required. Increasing the water level in the lake could adversely impact raw water quality and could require construction of a berm at the southeast end of the lake to prevent flooding.

3.1.3 Raw Water Quality

Raw water samples were collected by earth-Water Concepts (2009) on four occasions from three locations at various depths. The data was reviewed and is summarized in Table 3.1.

Table 3.1: Spectacle Lake Raw Water Quality (earth-Water Concepts, 2011)

Parameter	Units	GCDWQ	Average	Range
Hardness	mg/L as $CaCO_3$		5	4 - 6
Total Alkalinity	mg/L as $CaCO_3$		<5	<5 - 5
Chloride (dissolved)	mg/L	< 250	16	10 - 20
Colour	TCU	< 15	18	8 - 30
Total Organic Carbon (TOC)	mg/L		3	1.4 - 3.7
pH		7 – 10.5	6	5.2 - 6.0
Turbidity	NTU	0.2/0.1/1.01	1	0.2 - 1.3
Conductivity	uS/cm		68	44 - 83
Aluminum (dissolved)	ug/L	<100/2002	107	28 - 180
Arsenic (dissolved)	ug/L	10	<2	<2 - 2
Copper (dissolved)	ug/L	< 1000	<2	<2
Iron (dissolved)	ug/L	< 300	213	91 – 5803
Lead (dissolved)	ug/L	10	<2	<2
Manganese (dissolved)	ug/L	< 50	100	24 - 360
Calcium (dissolved)	mg/L		1	0.8 - 1.4
Magnesium (dissolved)	mg/L		1	0.5 - 0.6
Sodium (dissolved)	mg/L	< 200	10	6 - 12

Based on the available data, Spectacle Lake is considered to be typical of surface water sources in Atlantic Canada, as it is soft with low turbidity, pH, alkalinity, colour, and total organic carbon (TOC) concentrations. However, the available data is from a limited number of samples, and therefore is not expected to capture the source's full range of water quality as lake water quality can vary significantly and rapidly. To obtain a full understanding of the water quality range, and therefore to adequately size the treatment process, weekly (daily would be better) pH, color, TOC analysis should be conducted with monthly iron and manganese analysis.

The main objectives with respect to treatment are turbidity removal, pathogen reduction, removal of colour and disinfection by-product precursors, and removal of iron and manganese. Under the GCDWQ, the maximum colour limit (15 TCU) is considered to be an aesthetic objective, meaning it does not have any negative health effects; however, compounds that cause colour, such as natural organic matter (NOM) also react with chlorine to form disinfection by-products (DBPs), such as trihalomethanes (THMs) and haloacetic acids (HAAs), which are carcinogenic compounds and have maximum acceptable concentrations listed in the GCDWQ. Therefore, treated water colour is typically reduced well below the aesthetic objective to levels that are imperceptible to most people.

DBP formation potential (fp) tests were completed as part of the study by earth-Water Concepts. The single test yielded a THMfp of 530 µg/L, which exceeds the GCDWQ value of 100 µg/L. The testing simulates DBP formation under relatively extreme conditions that are generally not representative of actual distribution system conditions (I.e. very high chlorine dose, high temperature, and long reaction time); however, it does indicate that organics treatment is required to reduce DBP formation. Compared to other sources in Nova Scotia, the water appears to have relatively low colour and total organic carbon

(TOC) concentrations, which should help to reduce the required level of colour/TOC reduction to control DBP formation.

Iron and manganese are both presently aesthetic based parameters in the GCDWQ, where the aesthetic objectives are 0.3 mg/L for iron and 0.05 mg/L for manganese. Both metals may cause staining of plumbing components and laundry, as well as taste and odour issues. Health Canada has recently proposed new objectives for manganese, with a maximum acceptable concentration of 0.1 mg/L and an aesthetic objective of 0.02 mg/L. These guidelines have not yet come into effect, but it is anticipated that they will be implemented in the near future. Iron and manganese content in Spectacle Lake raw water are above the GCDWQ. It is expected, given the raw water iron levels on record, and that iron is readily precipitated from solution at pH 6.0, that treated water iron level will be reduced to below the 0.3 mg/L guideline by conventional treatment. Manganese, on the other hand, may require special measures such as oxidant addition to ensure adequate removal.

The pH in the source water is also below the GCDWQ, which can cause aggressive corrosion of metallic components in the distribution system and premise plumbing. Therefore, treatment should also include pH and alkalinity adjustment using a chemical such as soda ash or caustic soda.

3.1.4 Lake Watershed Protection Plan

The water quality in the lake is heavily dependent on the terrestrial conditions within the lake watershed. Changes to, or use of, the watershed will very like cause changes to water quality. It is strongly recommended that development or activities within the lake catchment be strictly limited. The Municipality has purchased a number of properties surrounding the lake, it is advisable that the district continue to purchase properties surrounding the lake, especially the lower lying areas.

The Nova Scotia Treatment Standard also requires a comprehensive Source Water Protection Plan be developed to protect the source water from contamination. There are five steps outlined in the Treatment Standard, including:

1. Form a Source Water Protection Advisory Committee.
2. Delineation of a Source Water Protection Area Boundary.
3. Identify Potential Contaminants and Assess Risk.
4. Develop a Source Water Protection Management Plan.
5. Develop a Monitoring Program to Evaluate the Effectiveness of a Source Water Protection Plan.

Generally, a watershed protection plan will include:

- Watershed boundary signage.
- Security camera at dam.
- Land acquisition policy.
- Water quality monitoring within the watershed.
- Public awareness campaign.
- Emergency spill response plans.
- Restrictions on commercial/industrial/recreational/residential activities and use.
- Protection for the majority of the watershed under the Wilderness Areas Protection Act.
- Designation of the watershed as a Protected Water Area.

3.2 Option #2 – Groundwater

3.2.1 Description

Metamorphic rock underlying the central and northern parts of the Chester area are not expected to be capable of providing yields on this order, and could be limited by salt water intrusion effects. Anecdotal evidence suggests that limestone beds within Windsor Group rocks in the west part of the Village (and further west), may be capable of providing moderate yields, however:

- Windsor Group rocks do not typically provide yields greater than 390 m^3/d.
- Water pumped from Windsor group rocks is typically difficult and costly to treat, exhibiting elevated concentrations of iron, manganese, chloride, sulphate, and other scale forming minerals.

The hydrogeological setting of the Middle River area and preliminary mapping by NSDNR indicate that there may be potential to develop a well field. Extensive thicknesses of granular material supply domestic wells to the west and east of the river. These deposits could be part of a buried regional valley feature, and to the east of the river are mapped as glaciofluvial outwash. These areas are well situated to provide adequate source water protection, but would require a six to seven kilometre pipeline to be connected to the Village.

A typical well field may consist of several wells to meet targeted demands. The characteristics of existing wells in the Middle River Area suggest that a municipal well field would consist of three or more production wells:

- Well depths would be on the order of 30 to 50 metres.
- Wells would be constructed using stainless steel, wire wrapped screens.
- Well diameters would be 200 to 250 mm, depending on individual well capabilities and corresponding pump sizes.
- Required individual well yields would need to be on the order of 325 to 650 m^3/d.
- If needed, the use of storage reservoirs to meet peak demands could reduce the number of wells required.

Potential drilling sites were shown in the groundwater assessment report submitted by CBCL in 2017. Potential drilling sites were selected based on geology mapping, thickness of granular material as indicated by water well logs, potential yield as indicated by the geology and airlift yields, and access for a drilling rig. Land ownerships and permission for access to these properties has not been investigated.

3.2.2 Exploration and Aquifer Testing

Exploration work would focus on drilling of 150 mm diameter boreholes. Test well drilling can require several test holes in order to locate an adequate thickness of high capacity aquifer material. As exploration work proceeds, if potentially productive zones were identified in areas not accessible by water well drilling rigs, road building would be required.

Ideally geotechnical boreholes would be used to narrow the focus of test well drilling and locate the most productive and secure zones of the aquifer. This step would improve the overall design of the wells and well field, while offsetting costs associated with installing screened test wells. Depending on the rig used, monitoring wells would be installed as part of this step for later use in pumping test interpretation, or a steel casing would be installed and perforated to provide a rudimentary test well. Specifics on well depths, completion details, and number of boreholes would need to be determined based on observed conditions at the time of drilling.

Test drilling typically includes on-site supervision and co-ordination with a third party drilling contractor, who is registered and certified in the Province of Nova Scotia. CBCL Limited would supervise all drilling work and work with the driller to collect periodic samples of the formation cuttings. Cuttings samples and reports from the driller on the nature of the material encountered (hardness, water encountered) would be used to assemble a borehole log. Completed logs indicate changes in material type and provide an interpretation of water bearing zones.

Borehole drilling and logging includes airlift yield tests at periodic intervals, or at a depth where the driller reports that a water bearing zone has been encountered. The driller initiates an airlift test by blowing air from the rig down into the borehole to evacuate water from the well. The approximate rate of flow is estimated using dams and a weir or pipe discharging to a calibrated bucket. If airlift testing indicated that the yield of the test well could be adequate to meet community demands, recommendations for full instrumentation would be provided (e.g. final screen diameter, length, and slot size).

Provided that airlift testing indicated that the well yield could be adequate, a step test would be performed on each completed test well. Step tests typically include four one-hour steps and up to two hours of recovery monitoring. The pumping rate is increased after each hour, meeting or exceeding the targeted pumping rate during the last step of the test. Water level responses are recorded during the test using a data logger in the pumped well and confirmed by hand using a water level tape. A handheld probe can be used to measure in situ water quality parameters in the discharged water (including conductivity, temperature, pH, ORP, dissolved oxygen and turbidity), and a water quality sample should be collected at the end of the test. The water quality sample would be analyzed for inorganic parameters.

Step testing data can be analyzed to provide a preliminary indication of the formation transmissivity, well efficiency and optimal pumping rate for longer term testing. Water quality data would show if concentrations of any inorganic parameters exceed Health Canada Guidelines for Canadian Drinking Water Quality (GCDWQ). The results of step testing would indicate whether a 72-hour constant rate test would be required before proceeding to installation of a production well.

Longer term constant rate testing provides more detail on the likely long-term performance of a production well, and can allow for identification of any negative consequences of or limits to long-term aquifer pumping. Response curves would show any negative boundary conditions, (other pumping wells or physical boundaries, positive boundary conditions (interactions with surface water), or over-pumping / dewatering effects. If observation data are collected, a composite analysis will help to identify delayed storage effects, anisotropy, and the aquifer storage coefficient. Constant rate testing typically includes collection of water quality samples at 2 hours and 36 hours, to be analyzed for general chemistry and metals, and collection of a water quality sample at 72 hours to be analyzed for general chemistry, metals, bacteria, total suspended solids, and volatile organic compounds (VOCs).

3.2.3 Production Well Commissioning

Completed wells would require further step testing, constant rate testing, and comprehensive water quality analyses. Based on this analysis, a formal report would need to be generated with a recommended 20-year sustainable pumping rate and the type, size and depth of the pump in each finished production well. Reporting would form the basis for application to the province for a Water Withdrawal Permit.

3.2.4 Quality

Test wells and production wells have not been developed, therefore raw water quality was inferred using water quality data from the Province's Water Well Database. It is anticipated that the production wells would be installed in a surficial aquifer. Water quality data from several nearby surficial wells were reviewed, and the two sample sets shown in Table 3.2 were selected to represent typical and poor water quality conditions in order to obtain quotations for treatment equipment.

Table 3.2: Representative Groundwater Quality

Parameter	Units	GCDWQ	Typical	Poor
Hardness	mg/L as $CaCO_3$		39	73
Alkalinity	mg/L as $CaCO_3$		0.5	2.5
Chloride	mg/L	< 250	42	74
pH		7 - 10.5	7	4
Arsenic	µg/L	10	1	1
Iron	µg/L	< 300	391	3000
Manganese	µg/L	< 50	64	660
Uranium	µg/L	20	1	1.6
Calcium	mg/L		11	22
Magnesium	mg/L		2.9	4
Sodium	mg/L	< 200	34	42
Potassium	mg/L		1.3	2.5
Sulphate	mg/L	< 500	38	66
Total Dissolved Solids	mg/L	< 500	156	226

The water quality data presented above indicates that the groundwater in this region may have moderate to high concentrations of iron and manganese, as well as neutral pH. Therefore, treatment should include iron and manganese removal, pH adjustment, and corrosion inhibitor. Actual treatment requirements will need to be further evaluated with the development of test wells.

3.2.5 Source Water Protection Areas

Source Water Protection Areas (SWPAs) are zones around municipal drinking water wells, used to protect the community's source water. These zones are the basis for a community's Source Water Protection Plan, which provides guidelines for monitoring and regulation of land uses near the well field. Activities such as chemical and fuel handling, sewage treatment, manure spreading, and large scale industrial pumping are to be avoided in SWPAs, because they have the potential to degrade the quality and quantity of the community's drinking water.

Typically SWPAs are created by delineating capture zones around each well using computer models and time of travel concepts. The outer boundary of a SWPA is associated with the longest considered travel time for contaminants of concern. Typical travel times are on the order of 25 years from the outer edge of the zone to the well head. Additional mapping would be used to finalize a SWPA, subdivided according to property ownership, and land uses. A completed Source Water Protection Plan would comprise the following:

- Mapping to show time of travel capture zones.
- Excerpts from provincial and municipal legislation, by-laws and plans.
- Signage, direct communication with land owners, and public education sessions.
- A groundwater quality monitoring plan, including annual sampling, reporting, inspection and enforcement.
- An emergency-contingency plan for spills within the Protected Water Area.
- Additional terms of reference as defined by potential land uses in the SWPA (e.g. Forestry Management and/or Farm Management Plans).

Provincial and/or municipal legislation would generally provide land use restrictions and/or procedures for existing, non-conforming properties within a SWPA, and the utility may apply for designation of a Protected Water Area (PWA) under the province's Water Act.

3.3 Option #3 – Private Water Supplies

3.3.1 Description

Water quality and availability of existing private wells varies from household to household, without any apparent geographical relationships. A number of issues affecting reliability of individual wells in the Village were noted in the previous report by CBCL, including:

- Many dug wells do not meet current standards for construction.
- Water shortages have affected 23% of survey respondents.
- Aesthetic and other issues were noted by 31% of survey respondents and identified in 85% of water samples.
- 62% of raw water supplies contained coliform bacteria.

These results indicate that a consistent, reliable water supply is not available to a significant proportion of the village residents. However, the majority of the concerns are being addressed at a household level. The intent of this analysis is to capture these household costs for remaining on private well supplies.

3.3.2 Quality

Available water quality data indicates that the hardness of water in the area is low to moderate, and shows moderate concentrations of sulphate, chloride, and other dissolved solids. The dissolved solids content at one location was high. Iron and manganese concentrations appear to be elevated. Arsenic and uranium concentrations are below the GCDWQ in available sample data.

CBCL's survey of residents indicated that approximately 60% of respondents use a water treatment system. Predominant forms of treatment included UV disinfection and softening to reduce hardness, iron, and manganese. Raw water samples were also collected by CBCL from 81 sites.

The raw water of 50 wells (62%; 50 of 81) contained coliform bacteria. These locations were distributed evenly throughout the Village, and 48% (24 of 50) were dug wells. The presence of coliform bacteria in dug well supplies is relatively common, particularly in wells with poor covers (metal, wood, or concrete with cracks/fissures) and/or rock-lined construction. Its presence in drilled wells indicates that the wells have a direct link to surface waters and are therefore at higher risk of contamination from surface runoff. Twelve raw water supplies (15%) showed arsenic concentrations above the guideline limit of 10 mg/L. Other parameters exceeding the GCDWQ in raw water were uranium (1 well) and fluoride (2 wells). Copper and lead exceeded the GCDWQ at one and six locations respectively. These parameters can be associated with corrosion of piping; the pH was below seven at 38 locations, which suggests that corrosion could be an issue. Turbidity exceeded 1 NTU at 48 locations (60%). Iron and manganese exceeded the GCDWQ in 33 and 26 samples respectively (41% and 32%). Iron and manganese contribute to poor taste, odours, staining, scale formation, and encrustation of screens, piping, and filters, and can increase turbidity and colour. Elevated chloride concentrations were observed in 3 samples, however, only one of these wells was within 120 metres of the coastline, and this well was a dug well.

3.3.3 Yield

Drilled wells in the area are generally between 15 and 91 metres deep, and are constructed with at least 9 metres of casing to seal off the overburden material. The till thickness varies significantly, and is generally from 3 to 24 metres thick, but reaches up to 80 metres in selected locations.

Reported airlift yields in bedrock wells are low to moderate, and should generally be capable of sustaining a single household. The data suggests that more than half of the bedrock wells in the study area could provide 20 L/min or more. Reported short-term yields from dug wells are much higher, but are based on the driller's test pumping rate, and generally do not reflect longer term sustainability and water table effects.

Available aquifer testing data varies between locations. Whereas the majority of tested wells showed low to moderate potential for long-term yield ($T < 10\ m^2/d$), two wells in the area showed potential to supply facilities of moderate sizes ($T = 30$ and $43\ m^2/d$). One of these higher potential wells was installed in a surficial aquifer.

Transmissivity data support indications from the airlift yield data that well yields can be expected to vary significantly across the study area. Whereas the highest producing wells may be capable of supporting a larger business or residential complex, others may be capable of supporting only a small household. This is consistent with the presence of isolated individual fractures that may extend for some distance, but which are unlikely to be well interconnected, and which do not intersect all wells drilled on a given property.

Drought conditions in 2016 resulted in lowered water tables and water shortages for many private well owners. From a survey of residents, 23% of responding well owners reported a shortage in 2016. Reported shortages in years preceding 2016 were nearly identical. Of those reporting a water shortage, 10 homes were using drilled wells, and 63 homes were using dug wells or cisterns. Shortages show that water resources are stressed in localized parts of the community, and that the potential for further development is limited.

3.3.4 Other Issues

Dug wells were identified at many of the homes in the study area, including all areas within the Village of Chester, Middle River, and much of the area to the northeast of the Village. In the Village centre dug wells were often ornamented with beach rock. Anecdotal reports of well construction suggest that some or most of these wells are hand dug and rock-lined below the ground surface. Wells of this type can be several decades old and in the study area many of the wells are covered with wood or metal plates that form an incomplete seal. Wells constructed with concrete crocks were also observed in varying apparent ages and conditions. Most dug wells in the study area have likely not been completed with a concrete apron below the ground surface, a feature which helps to block short circuit pathways between the ground surface and well bore.

Many homes in the Chester area draw water from wells adjacent to the coastline. Whereas most dug wells will draw water from local, shallow catchments with low potential to be influenced by sea water, drilled wells in these zones are at higher risk. Mapping by NSDNR indicates that drilled wells in the Village Centre fall into a high risk category (Kennedy, 2012). Remaining properties within the Village have been categorized as 'medium' risk. The Village centre and Kaulback Island peninsula are surrounded by coastline; the risk of saltwater intrusion to these wells depends on cumulative extraction rates, connectivity of fracture sets, and the position of the saltwater interface in intermediate and regional flow systems.

CHAPTER 4 CANDIDATE WATER TREATMENT PROCESSES

4.1 Treatment Objectives

This report is investigating public type treatment systems as well as private onsite well supplies. Public water supplies are systems that provide water used for human consumption that:

- Have at last 15 service connections.
- Regularly service 25 or more people per day for at least 60 days of the year.
- Serve a day care facility, food establishment, or commercial property for accommodation of travellers.

The treatment requirements differ based on the type of system as described below.

4.1.1 Municipal Water System

The new central treatment facility must meet all water quality objectives set forth in the Guideline for Canadian Drinking Water Quality (GCDWQ) and the Nova Scotia Environment (NSE) Nova Scotia Treatment Standards for Municipal Drinking Water Systems (Treatment Standard). The GCDWQ includes Health-Based Guidelines, Aesthetic Objectives, and Maximum Acceptable Concentrations for a range of parameters including organic and inorganic compounds, metals, minerals, and other identified water contaminants. These guidelines are provided by Health Canada through the federal government. Enforcement of particular standards is the responsibility of individual provincial agencies.

NSE requires the adherence to the GCDWQ and has also included other specific treatment objectives in the Treatment Standard. For supplies using surface water and groundwater under direct influence (GUDI) of surface water, these include 99.99% removal or inactivation of viruses, and 99.9% inactivation or removal of *Giardia lamblia* cysts and *Cryptosporidium* oocysts through a combination of engineered filtration and disinfection. For non-GUDI sources, overall treatment requirements must meet a minimum of 99.99% inactivation of viruses through disinfection, and turbidity levels must not exceed 1.0 NTU in at least 95% of measurements.

Redundant filtration and disinfection systems must also be provided.

4.1.2 Registered Water Supplies

A registered water supply system is one that supplies general public such as restaurants, bars, schools, day cares, senior’s homes, trailer parks, hotels, and campgrounds.

Owners of a registered water supply are responsible for the delivery of water in accordance with provincial standards and for meeting their requirements for due diligence. Owners are responsible for all water quality sampling, testing, and monitoring requirements in accordance with the *Water and Wastewater Facilities and Public Drinking Water Supplies Regulations* and *Guidelines for Monitoring Public Drinking Water Supplies*. Water quality must comply with the microbiological, chemical, and physical water quality requirements established in the Nova Scotia Treatment Standard.

4.1.3 Private Water Supply System

Private water supply systems are systems that service homes and other systems not used by the general public. These systems do not need to comply with the CDWQG. However, for the purpose of this report we are evaluating all options, including private system, on the requirement to meet CDWQG.

4.2 Treatment Capacity

The 2018 Max Day Demand is estimated to be 1,658 m^3/day. The water treatment, storage and transmission systems discussed herein are designed to meet the 2018 demands plus an allowance for an annual increase in water demand of a 0.5%, over the 25 year design life from 2018 to 2043. The design capacity of the infrastructure systems is 1,900 m^3/day.

4.3 Summary of Candidate Treatment Processes

The main objectives for surface water treatment options will be to provide pathogen reduction, organics removal, and manganese removal. For groundwater, it is anticipated that the main objective for treatment will include iron and manganese removal and virus inactivation. Potential treatment processes are described in detail in the sections below.

4.3.1 Surface Water

Candidate treatment processes must meet the treatment objectives outlined above using a design that is conducive to the hydraulic, geographic, and operational characteristics of the proposed water supply system. A primary goal of the treatment facility will be reduction of organic matter, which can be accomplished through several approaches. These are classified under the general categories listed in Table 4.1.

Table 4.1: Overview of Treatment Processes

Treatment Category	Example Processes
Conventional Processes	• Conventional Treatment (Sedimentation). • Direct Filtration. • Dissolved Air Flotation (DAF). • Plate Settlers.
Membrane Processes	• Microfiltration. • Ultrafiltration. • Nanofiltration. • Reverse Osmosis.

Conventional processes use coagulation and flocculation to precipitate the colour from the water and condition it for removal. Coagulants, which may be aluminium or iron based, are chemicals that can be added to water to induce dissolved and colloidal species to agglomerate into larger particles known as flocs. The flocs may then be removed in a clarification step, which is followed by filtration, or through direct filtration. These processes generally have relatively high capital cost, due to the large tank volumes, and are relatively complex to operate because of the requirement for chemical treatment.

Membrane systems are available in a wide variety of types and configurations. Most notable are Microfiltration (MF), Ultrafiltration (UF) and Nanofiltration (NF). These processes are pressure driven sieve processes that separate particulates by moving water through pores in the membrane and collecting the particulates on the membrane surface. The nominal pore sizes in each class of membranes is presented in the figure below, which also shows the nominal pore size of reverse osmosis (RO) membranes and the effective exclusion size of sand filtration.

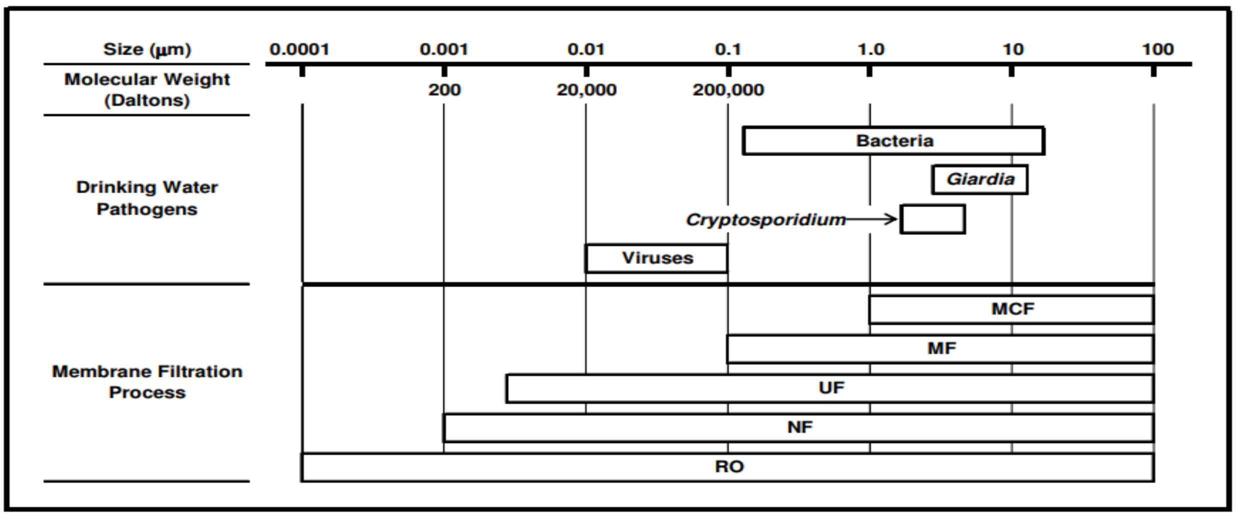


Figure 4.1: Membrane Pore Size Range (USEPA, 2005)

Membrane processes are generally broken down into low pressure and high pressure classifications. Low pressure membrane filtration processes, such as MF and UF systems, are effective at treating water with high levels of turbidity. Depending on the composition of the source water, coagulation/flocculation may be necessary for sufficient reduction of dissolved organic carbon (DOC) and DBP formation potential.

High pressure membrane filtration systems, such as NF, are generally capable of DOC and colour removal without the addition of a coagulant because they filter at a molecular level; however, pre-filtration is generally required to remove suspended solids from feed water to prevent plugging and premature failure of the membrane elements.

Membrane processes provide an effective barrier to bacteriological contaminants including Cryptosporidium and Giardia; however, many jurisdictions do not grant log-removal credit for NF processes because direct integrity tests are not available to confirm system integrity. This testing is available for MF and UF processes, so many jurisdictions often grant 3-5 log removal credit for MF or UF processes.

4.3.1.1 DAF

Flocs formed in low turbidity water tend to have relatively low densities and are generally more effectively removed by flotation compared to settling. In the DAF process, fine bubbles are injected into the water and attach to the floc particles, causing them to float to the surface where they can be collected and removed. The process is popular in Atlantic Canada, where surface waters typically have low turbidity and high concentrations of NOM. Locations in Nova Scotia that use the DAF process include New Glasgow, Antigonish, Shelburne, Canso, Mulgrave, Windsor, and Port Hawkesbury.

A schematic diagram of the DAF process is presented in Figure 4.2. DAF processes generally consist of a pre-fabricated set of tanks, piping, pumps and valves which when combined with chemical feed systems allow for enhanced coagulation, clarification and filtration.

Coagulation is required because the multimedia filters are not capable of removing DOC. Positively charged coagulant is metered into a raw water feed pipe prior to a mixer, which evenly disperses the coagulant. The coagulant reacts with negatively charged particles and dissolved NOM in the raw water to agglomerate small and dissolved contaminants into larger masses (flocs). The tanks are arranged into a series of two flocculation cells followed by a DAF clarifier cell. The flocculation tanks include a large mixer with paddles, which slowly rotate to encourage particle collisions within the tank. The flocculated water then flows into the DAF cell, where microscopic bubbles, which are continuously produced by the recycle system, are injected into the bottom of the tank. The microbubbles attach to the floc particles and rise to the top of the DAF tank, where the sludge blanket forms and is scraped off to the waste stream.

Treatment must also be provided for iron and manganese removal. Dissolved iron and manganese can be converted to iron and manganese oxides by adding a strong oxidant such as $KMnO_4$. Depending on the chemistry of the raw water, this may result in the formation of colloidal manganese oxides, which can be removed through the coagulation process, where colloidal manganese clumps together into flocs, which are then removed through filtration.

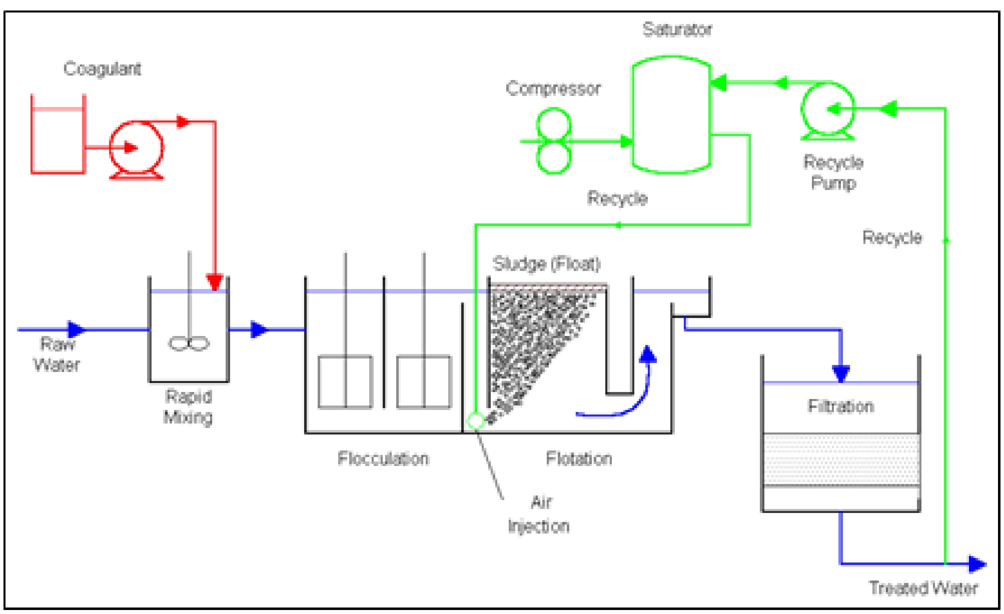


Figure 4.2: Schematic Diagram of DAF Process

The clarified water is withdrawn from the bottom of the DAF cell, and is sent to dual-media (anthracite-sand) filters. Following filtration, the water is disinfected using chlorine and pH adjustment chemicals are used to increase the water to an appropriate pH to prevent corrosion in the distribution system. These filters operate for 24-48 hours depending on loading conditions before a backwash is initiated to remove all particles captured by the filter. After a backwash, a filter-to-waste (rinse) cycle is initiated until the filtered water quality improves to the required standard.

While the backwashes are relatively infrequent, a large volume of water is required for filter backwashing and filter-to-waste (rinse) operations, which is incorporated into the treated water storage design. The DAF system would use approximately 10% of the total flow for backwashing the media filters and carrying sludge from the DAF clarifier.

A DAF process would produce two streams of treatment plant residuals. The first stream would consist of solids that are skimmed from the top of the DAF clarifier. This stream represents a relatively small volume with respect to the total plant flow, and might have solids concentrations of 2-5%, depending on raw water quality.

The second residuals stream consists of backwash wastewater. This represents a substantially larger quantity, at a far lower concentration of solids. Generally, the first few minutes will have high concentrations of organics, but this gradually diminishes as the filter is progressively cleaned through the backwash cycle. Backwash waste represents a high flowrate over a short time, so it must be discharged to an equalization tank.

Following equalization, the residuals must be discharged either to a municipal collection system or treated on-site to reduce TSS and aluminum concentrations prior to disposal. The preferred method is generally to discharge to the municipal collection system. When the collection is too far from the water treatment plant, residuals may be discharged to lagoon or to a mechanical thickening process, such as a DAF or sedimentation clarifier. Depending on the size of the facility, thickened sludge may be sent to a dewatering process, such as a centrifuge, to further reduce sludge volumes. Treated wastewater, meeting aluminum and TSS requirements, may be discharged to a nearby water body.

Solids must be disposed through appropriate means. For this application, it is assumed that thickened sludge will be discharged to a holding tank and trucked off-site for treatment and disposal.

4.3.1.2 Ultrafiltration

As described above, MF or UF membranes may be capable of removing sufficient DOC and colour compounds if a coagulant is applied upstream of the membrane system. Depending on water quality, it may be possible to achieve sufficient removal using inline coagulation without the use of large contact basins to form floc particles. The amount of floc formed in source water affects the flux rate at which a membrane can operate. Higher amounts of floc formation require a lower operating flux and more membrane area for a given capacity. This process is used at the water treatment plants in Lunenburg and Mahone Bay.

A schematic diagram showing a typical ultrafiltration and coagulation system process is provided in Figure 4.3. The process equipment for membrane filtration is different than with DAF and the additional area required for backwash pumps and backwash water storage is eliminated.

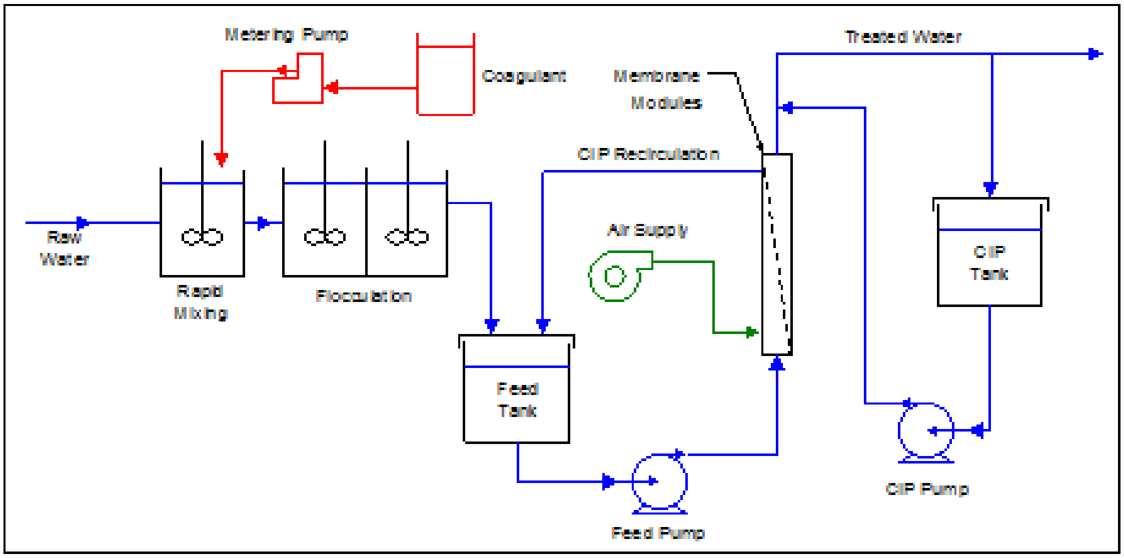


Figure 4.3: Schematic Diagram of Ultrafiltration Process

A coagulant, such as alum, would be applied upstream of a UF membrane. The coagulant would be applied at a low dose to create pin flocs to remove DOC and colour, which are DBP precursors. The membrane-filtered water, called permeate, will be chlorinated and pH adjusted prior to being pumped to the reservoir.

While not shown in the schematic diagram, pre-oxidation would also be included for iron and manganese removal. A membrane system optimized to remove iron and manganese would include a pre-oxidation step to oxidize the reduced metal species into solid or colloidal form. It may also include some form of pre-filtration step to remove particulate oxides that could otherwise quickly foul the membrane. The oxidized water would then be sent through the membrane filter to remove any remaining colloidal metal oxides.

A UF process produces several streams of wastewater. The UF membranes typically backwash approximately once per hour. Once per day the backwash will be chemically enhanced using either high concentrations of chlorine or citric acid to achieve improved cleaning. These backwashes represent relatively high flowrates over very short periods, and therefore must be discharged to an equalization tank. Chemically enhanced backwashes are neutralized prior to being discharged to the equalization tank.

Following equalization, the residuals must be discharged either to a municipal collection system or treated on-site to reduce TSS and aluminum concentrations prior to disposal. Treatment strategies for the UF option will be similar to the DAF residuals treatment. Treated wastewater, meeting aluminum and TSS requirements, may be discharged to a nearby water body. Solids must be disposed through appropriate means. For this application, it is assumed that thickened sludge will be discharged to a holding tank and trucked off-site for treatment and disposal.

On a monthly or bi-monthly basis, the UF membranes are subjected to a higher strength chemical clean-in-place (CIP) process. A high pH CIP, for reducing organic foulants, typically uses high concentrations of sodium hypochlorite and caustic soda, whereas a low pH CIP, for reducing inorganic foulants, will use citric acid. These wastes are relatively high in solids concentration and are performed relatively infrequently compared to the UF backwashes, so they should be sent directly to the same holding tank as the thickened sludge.

4.3.1.3 Integrated Membrane Filtration

NF is an effective solution for small water treatment plants treating source waters with low turbidity, and high colour and dissolved organics. The process avoids the use of chemicals within the treatment process, using a NF membrane that retains not only viruses and microbes, but also the dissolved organic humic and fulvic acids that create DBPs after chlorination. The systems are relatively simple to operate, automated and tolerant to spikes in feed water quality from storm events or spring melt. An additional benefit to not using chemicals is that the waste stream is simply a concentrated form of the source water and in most cases, can be returned directly to the source or discharged to a receiving surface water without concern for chemical contaminants introduced in other common water treatment processes. This process is used at the water treatment plants in Collins Park and Middle Musquodoboit, which are both owned and operated by Halifax Water.

As described above, NF units are not awarded log-removal credits for Giardia or Cryptosporidium, because direct integrity testing is not available. Therefore, pre-filtration using UF is required to achieve particulate and pathogen-removal requirements.

In this arrangement, the UF membrane, which is capable of removing nearly all particulate matter from the raw water, acts as a pre-filter for the NF, which removes the dissolved NOM.

The UF membrane would be configured in a similar arrangement to the NF, with automated membrane skids sized to meet the design flow with one unit out of service. The skids would be sized to produce a filtrate (filtered water) flow large enough to feed the NF skid(s), which would be sized to produce a permeate flow to meet the design flow. The UF reject rate is approximately 5% of the incoming flow, which is significantly lower than the 15 to 25% produced by the NF membranes.

The UF membrane filtrate would not be produced at a sufficient pressure to feed the NF skids directly. Therefore, an intermediate storage step would be required where the UF filtrate fills a transfer tank, which is emptied by feed pumps to the NF skid.

The membranes require a regular recovery clean approximately once per month. The recovery cleans consist of soaking in a citric acid or sodium hypochlorite solution. The waste stream could be held in a chemical holding tank that is pumped out on a periodic basis or directed to a sanitary sewer.

The UF system will generate backwash, chemically enhanced backwash, and CIP waste streams. The NF system will generate a continuous waste stream, known as concentrate, which is the fraction of membrane reject that has higher concentrations of dissolved contaminants from the raw water.

Because this treatment arrangement does not require coagulation, the only parameters that must be reduced in the UF backwashes are solids. The UF backwash may therefore be held in an EQ tank and blended with NF concentrate to reduce TSS prior to discharge. Chemically enhanced backwashes must be neutralized prior to discharging to the EQ tank.

CIP waste from both membrane systems are high in solids and contain chemical cleaning waste. CIP wastes must be stored in a storage tank for periodic removal by hauler truck.

4.3.1.4 Surface Water Treatment Equipment Costs

Budgetary quotations were obtained for the three treatment processes listed above. These costs are summarized in Table 4.2.

Table 4.2: Capital Costs for Surface Water Treatment Equipment

Treatment Process	Process Equipment Costs
Dissolved Air Flotation (DAF)	$1,321,000
Low Pressure Membrane Filtration (UF)	$1,135,000
Integrated Membrane Filtration (UF/NF)	$1,265,000

The costs presented above are only for the major process equipment, and any associated components that might be required to operate the treatment process, such as coagulant feed systems. The costs do not include installation or other components, disinfection systems, finished water chemical feed systems, or for the building and associated site works. All systems will require potassium permanganate dosing systems, which have also not been included in the costs above.

Operations costs for treatment systems are shown in Table 4.3. These costs include power for pumping, chemicals, replacement components, and labour costs for preventive maintenance.

Table 4.3: Annual Operation and Maintenance Costs for Surface Water Treatment Processes

Item	DAF	UF	UF/NF
Labour	$70,000	$75,000	$75,000
Power	$12,000	$29,000	$54,000
Chemicals	$39,000	$32,000	$23,000
Residual Management	$76,000	$36,000	$50,000
Miscellaneous Replacements	$1,000	$14,000	$29,000
Annual O&M	$198,000	$186,000	$231,000

Annual operation and maintenance costs were based on operation at average day flow. Labour costs were estimated based on hourly rates of $35 per hour for a supervisor, and $25 per hour each for a primary and secondary operator.

Power costs were calculated only for major pumping equipment. They do not include for building heating, ventilation, or lighting. They also do not account for power consumption from control panels, mixers, blowers, metering pumps, computers, or other auxiliary equipment. Chemical costs were based on unit rates provided by chemical suppliers.

The UF/NF membrane system requires the least use of chemicals, as it does not require coagulation. The UF system would require chemical coagulation for organics removal, but it is expected to require a lower dose than the DAF system. All systems require pre-treatment oxidation for removal of iron and manganese. It has been assumed that soda ash will be used to increase treated water pH, and that a corrosion inhibitor would be dosed to the finished water. Membrane cleans would likely require citric acid for the low pH clean to remove inorganic foulants, such as iron and manganese, and a high pH wash using caustic soda to remove organic foulants.

Annual costs associated with residuals management are anticipated to be a significant fraction of total annual costs for all systems. For the DAF and UF options, it was assumed that the plants could achieve approximately 95% recovery, which equates to a daily production of approximately 95 m^3/d of residuals. It was assumed that the treatment facility could not be connected to the municipal sewer system, therefore, waste needs to be treated on-site. We assumed that a thickening process using DAF clarification could reduce solids and aluminum concentrations prior to discharge. Waste from the second DAF process would then be discharged directly to a wastewater holding tank that would be pumped out approximately every two weeks. NF concentrate and UF backwash waste could be directly discharged. NF and UF CIP waste would need to be discharged to a holding tank for period pumping.

Miscellaneous replacements include items that wear out or are consumed. This includes metering pump components, calibration kits and reagents, lubricants, and cartridges filters. For the membrane options, it was assumed that UF membrane modules and NF membrane elements would need to be replaced after 5 years per skid (10 years life of total system). While the cost for membrane replacement would be all at one time, it was divided over the 10 year timeline to provide an annual cost.

To further assess the differences between treatment systems, it is appropriate to consider the overall cost of all treatment related equipment and the treatment equipment operating costs. To complete this evaluation of the three processes, the total budgetary cost of capital equipment was estimated along with annual operating and maintenance costs. The UF system appears to have the lowest capital and operating cost. However, because it requires the use of a coagulant, some form of treatment will be required to reduce aluminum concentrations from the backwash waste prior to discharging to the environment. Options for thickening residuals prior to disposal could include geotextile filter bags, DAF clarification, or treatment in a lagoon. These should be reviewed in greater detail as part of the pre-design study to confirm that discharge requirements can be met.

4.3.2 Centralized Groundwater

Iron and manganese are often removed together or with very similar technologies such as oxidation, filtration or adsorption processes. Iron and manganese are usually present in groundwater in their dissolved forms, Fe^{2+} and Mn^{2+}. They can be removed through oxidation and filtration (with or without coagulation), biological filtration or ion exchange. Manganese has a higher redox potential than iron, making it more difficult to oxidize and remove. Thus, more complex processes are usually required for manganese removal than for iron removal.

Methods for iron and manganese removal include:

- Oxidative media.
- Biological filtration.
- Pre-oxidation and coagulation.
- Pre-oxidation and membrane filtration.
- Ion exchange.

If the groundwater option is selected as the preferred approach, then the optimum method should be determined through analysis, treatability testing, and possibly pilot testing. For the purposes of this report, it has been assumed that oxidative media will be used.

4.3.2.1 Oxidative Media

Oxidation and filtration is a well-established iron and manganese removal technology and is generally favoured because of its simplicity and low capital cost. Common oxidants used for iron and manganese removal include oxygen (aeration), chlorine and potassium permanganate.

Dissolved Fe^{2+} is oxidized to Fe^{3+} and precipitates out of solution as $Fe(OH)_3$. This precipitate can then be removed through filtration. Iron oxidation is generally very quick, no matter which oxidant is used. It can, however, be slowed by the presence of organic matter such as humic or fulvic acids in the raw water. Precipitated iron can be removed through physical filtration.

Manganese is present in groundwater as Mn^{2+}, which is its reduced, dissolved form. It reacts very slowly with weak oxidants such as free chlorine. It can, however, be oxidized to MnOx using stronger oxidants such as potassium permanganate. Unfortunately, the resulting oxides tend to be colloidal in nature, making them very difficult to remove through simple filtration.

Most water treatment plants that employ pre-oxidation followed by media filtration to remove both iron and manganese use manganese-oxide-coated media. The most common of these is known as manganese greensand. The media is made of glauconite coated with manganese oxides, which have an affinity for Mn^{2+}. As the Mn^{2+}-laden water passes through the filter the ions adsorb to the manganese oxide coating. When all of the adsorption sites are filled, the media is washed with an oxidant (usually $KMnO_4$) to regenerate the oxides. A number of manufacturers have developed similar, proprietary oxide-coated manganese removal media that function in the same way as traditional greensand media. Pre-oxidation and media filtration systems can operate in several different modes that are differentiated by the point of application of the regenerant, $KMnO_4$. In the 'continuous' regeneration mode potassium permanganate is added at the front end of the filter throughout the production cycle. Free chlorine, a cheaper, weaker oxidant, is often added ahead of the KMnO4 to precipitate the iron, thus minimizing chemical costs. The precipitated iron and manganese are retained on the filter media and later removed through backwashing. Oftentimes, a layer of coal-based media will be placed at the top of the filter to filter out the precipitates while the greensand layer will act as a buffer and remove any un-reacted $KMnO_4$ or Mn^{2+} remaining in the water. This method of operation is generally recommended for systems where iron removal is the primary goal of the treatment system because manganese is not always removed effectively.

In the ‘intermittent’ regeneration mode, chlorine is added continuously at the front of the filter to oxidize any iron in the raw water. The iron precipitates are removed through physical filtration at the top of the filter while manganese is removed through adsorption to the greensand media. Regeneration with KMnO4 occurs only when the greensand media is no longer adsorbing manganese effectively. At that point, the media is washed with KMnO4, which reacts with the Mn^{2+} adsorbed to the manganese oxides on the greensand media to form more manganese oxides, thus regenerating the adsorption capabilities of the media. This method of operation is generally recommended when manganese removal is of particular importance. It can, however, result in high chemical costs as the filter media may have to be regenerated frequently. This can sometimes be avoided by adding more free chlorine than is required for iron oxidation ahead of the filter. The excess free chlorine will continuously regenerate the media by encouraging the oxidation of Mn^{2+} ions adsorbed to the manganese oxide coating. This solution will not be feasible if the source water has significant concentrations of organic matter, as these organic molecules will compete for the chlorine and may form disinfection by-products such as THMs and HAAs.

Table 4.4: Capital Costs for Groundwater Treatment Equipment

Item	Oxidative Media
Process Equipment Costs	$414,000

4.3.3 Onsite Groundwater

For onsite systems, it is assumed that sites with dug wells will require RO filtration systems and sites with drilled wells will require iron and manganese treatment systems. Despite a large proportion of the sites being private systems, it is assumed that treatment will be provided to meet the GCDWQ.

CHAPTER 5 TRANSMISSION AND DISTRIBUTION

Water will be pumped from the treatment plant to a reservoir. Water will then flow by gravity from the reservoir, through a large diameter transmission main towards the village. The transmission system feeds the distribution system, it is the main artery of water from the reservoir to the service area and represents a critical piece of infrastructure. The transmission main will feed the distribution system which consists of smaller diameter water mains, service connections and hydrants.

5.1 Reservoir Location

The reservoir location and design is heavily influenced by the service catchment. The majority of the Village is below the 40m contour, and could be serviced within one pressure zone with static pressures ranging from 40 psi to 100 psi. Individual pressure reducing valves would be required for homes within a static pressure zone which exceeds 80 psi. There is an area of high elevation on Haddon Hill which requires special consideration and is discussed in more detail below. A low water level of approximately 60 meters elevation is required to service the system by gravity, with the exception of the afore mentioned high elevation area.

The reservoir will provide peak balancing, fire storage (if fire protection is provided), and emergency storage. The location of the groundwater and surface water sources is significantly different. To provide the most appropriate transmission and distribution system two reservoir locations have been selected. The reservoir locations have been identified through review of a number of sites. Considerations included: proximity to the supply center, proximity to the source, elevation, site access, power, soil conditions, land owner ship etc.

5.1.1 Surface Water Reservoir

The surface water reservoir is proposed to be located on municipal land in the Bonds Drive area. This reservoir location is suited to the surface water source option. The treatment plant would be located on the same site with access from Bond Drive. High level, preliminary road alignment and profile designs have been completed to verify accessibility. An access road to the reservoir would have a design grade of approximately 10% which is considered acceptable. The site elevation is relatively high, with a potential reservoir base elevation of approximately 65m el, eliminating the need for a water tower or significant dead storage.

5.1.2 Groundwater Reservoir

The groundwater reservoir is proposed to be located west of Stanford Lake in the Haddon Hill area on high ground. The site would preferably be at an elevation of approximately 70 m. The groundwater treatment plant would not be located on the same site therefore the requirements for tank turnover would be more demanding. There is no known municipal land which would be suitable for the reservoir in this location.

5.1.3 Transmission and Distribution System

The recommended reservoir location for the groundwater system is the Haddon Hill area, the recommended location for the surface water system is the Bond Drive area. As the reservoirs are in different locations the transmission lines which run from the reservoir to the urban center differ for both systems. The location in the distribution system where the transmission line terminates would be determined at a later stage in the detailed design based on a full water system model.

The distribution system for both water sources should be very similar, the majority of watermains within the system will be 200mm diameter, with some 300mm and some 150mm watermains. The sizing, design and layout of the distribution system needs to take into account fire flows, acceptable operating pressures, and water quality issues.

The reservoir, transmission and distribution system will need to provide maximum hour potable water flows while maintaining a minimum operating pressure of 42 psi in the distribution system. The size of the reservoir, transmission line and distribution mains will largely be driven by fire flow requirements which are discussed below.

5.1.4 Fire Flow Requirments

There is a desire to provide Fire Fighting Water for the Village. Providing fire protection in a water system of this size has significant cost and operational implications which include installing and maintaining fire hydrants, providing adequate storage capacity, meeting the minimum pipe size requirements and water quality management. The benefits of providing fire protection can include preservation of human life, reduction of human suffering, protection of the tax base from destruction by fire and reduced insurance costs. There are numerous ways to calculate required fire flow. The American Water Works Association manual of water supply practices for Distribution System Requirements for Fire Protection outlines 3 methods of calculating fire flow and storage requirements which offer significantly different results. During the pre-design phase of the project the Village and Fire Department should establish a fire protection plan which outlines the fire flow rate and duration requirement from the proposed water system.

This report utilises the most commonly cited method of Fire Flow Calculation – the Fire Underwriters Survey. This method is considered to be the most conservative for the Village, likely offering the highest fire flow, storage and pipe sizing requirements.

Preliminary fire flow calculations have been completed for a number of buildings in the village core. It is assumed that the buildings are wood frame, non-sprinklered and have shingle roofs. The calculated values are as follows:

	Metric	Imperial	Note
Fire Flow	236 l/s	3,740 USGPM	Based on 42 Queen St
Fire Storage	2,600 m^3	686,847 USG	
Total Reservoir Vol	3,770 m^3	995,928 USG	
Transmission Main	500mm	20 inch	From Bond Drive to Village Core

The design intent of the distribution and transmission system is to accommodate fire flows during a maximum day event while maintaining a system pressure of 20 psi within the distribution system. Calculating fire flow in accordance with FUS is the primary driver determining the reservoir size and transmission main noted above.

5.1.5 Operating Pressures

It is generally considered acceptable to operate water systems between 40 psi to 100 psi. This 60 psi pressure range allows for an elevation variance of roughly 40 meters across the pressure zone. It is desirable to operate a water system within a lower pressure range, for example from 60 psi to 80 psi. This 20 psi pressure range would allow an approximate 14 meters elevation variance across a pressure zone. A reduced pressure range is desirable to provide more constant water pressures to customers throughout the service boundary and operating a lower pressure range also reduces the potential for leakage. However, operating with a lower pressure range allows less of an elevation variance across the distribution zone which may require the installation of pressure reducing valves, booster pumping stations or additional reservoirs.

The elevation within the village boundary ranges from 0m el (sea level) to 65m el. To service the entire village boundary within one pressure zone would result in a working pressures at the sea level of up to 140 psi. This is beyond the limits of standard practice in Nova Scotia and is therefore not recommended.

The vast majority of the Village is between 2m el and 40m el. There are approximately 15 existing homes and several undeveloped lots in the Haddon Hill area that are above the 40m contour. The village should assess the cost benefit of including these homes in the service boundary. To provide potable and fire suppression water service to this area would likely require the installation of a water reservoir on Haddon Hill, with a PRV on East Wind Drive or Haddon Hill Road. This option would work well with the groundwater reservoir location. Alternatively the Village could install a booster pumping station on East Wind Drive or Haddon Hill Road, this would be required based on the surface water reservoir location (Bond Drive). At this point we have included this area within the service boundary and broken out an approximate cost to service these homes.

If the Haddon Hill area is not included in the initial phase the village could be serviced from the 40m contour down to sea level within one pressure zone. Individual PRV's would be required for buildings below the 14 meter contour as the plumbing code allows for a maximum of 80 psi within buildings. However, provisions would need to be incorporated in to the design to allow for the future servicing of Haddon Hill.

5.2 Phasing

A general rule of thumb for central water system planning is that areas of higher density require less pipe per customer and are generally more cost effective to service, whereas areas of lower density require more pipe per customer and are therefore more costly to service.

The Village core represents an area of high density where there will generally be a significant number of water users in a small area. For a comparison example there is less dense development towards the golf course. Areas of lower density which should be reviewed by staff for servicing cost, one area in particular is illustrated in the figure below. It is anticipated that the water system would be built out over a number of years, in a number of phases. Based on the well study completed by CBCL previously there is no discernible pattern to the water supply issues within the Village. Meaning there is no area which has been identified with a notable poor groundwater supply which should get preferential treatment in terms of water system schedule. Phasing could be established based on a cost benefit assessment considering density, socioeconomic benefit topographic restrictions etc.

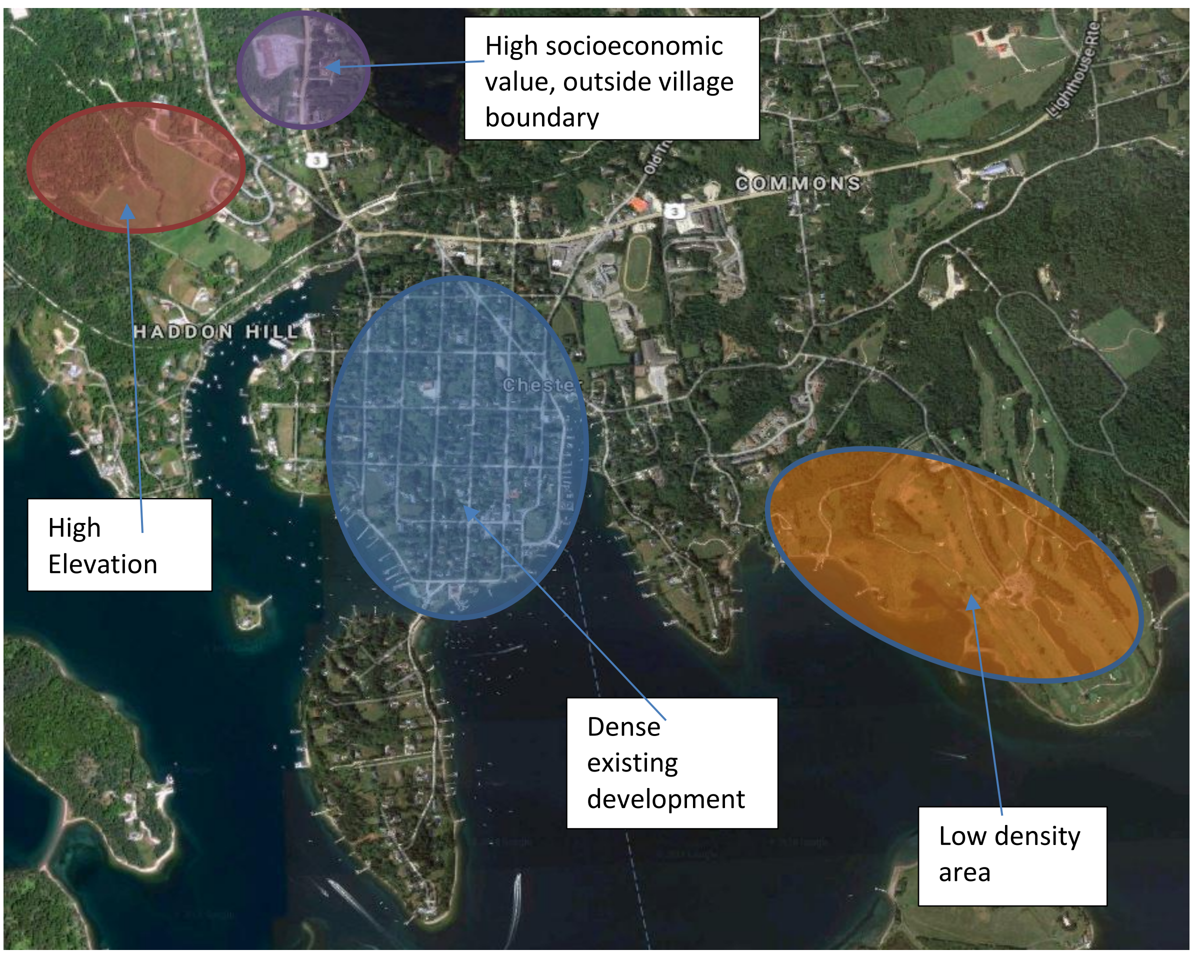


Figure 5.1: Phasing Considerations

5.3 Constructability Issues

5.3.1 Lot Layout and Servcing

There are a variety of lot configurations through the village. There are commercial buildings with 0m setback from the road right of way and there are pan handle rural residential lots with common driveways where homes are located a significant distance from the road right of way. In this report it is assumed that the Village will install a service connection for each property and terminate that service connection at the municipal property line. It has been considered the property owners responsibility, and direct cost, to extend the water service from the property line to their home or business.

The peninsula is an area where there are a significant number of properties with common driveways. A plan will need to be developed on how to service each home. Options could include extending a municipal owned lateral down the common driveway and proving service connection for each home, the service connection would be paid for the home owner and the municipal service lateral would be paid for by the Municipality. Other options could include only installing the water main with the municipal street right of way and having each property own install a private lateral and connect to the pipe within the municipal right of way.

One of the more extreme examples is given below, the home is located approximately 350 meters from the public right of way. A private, or shared, service connection would be required within the right of way.

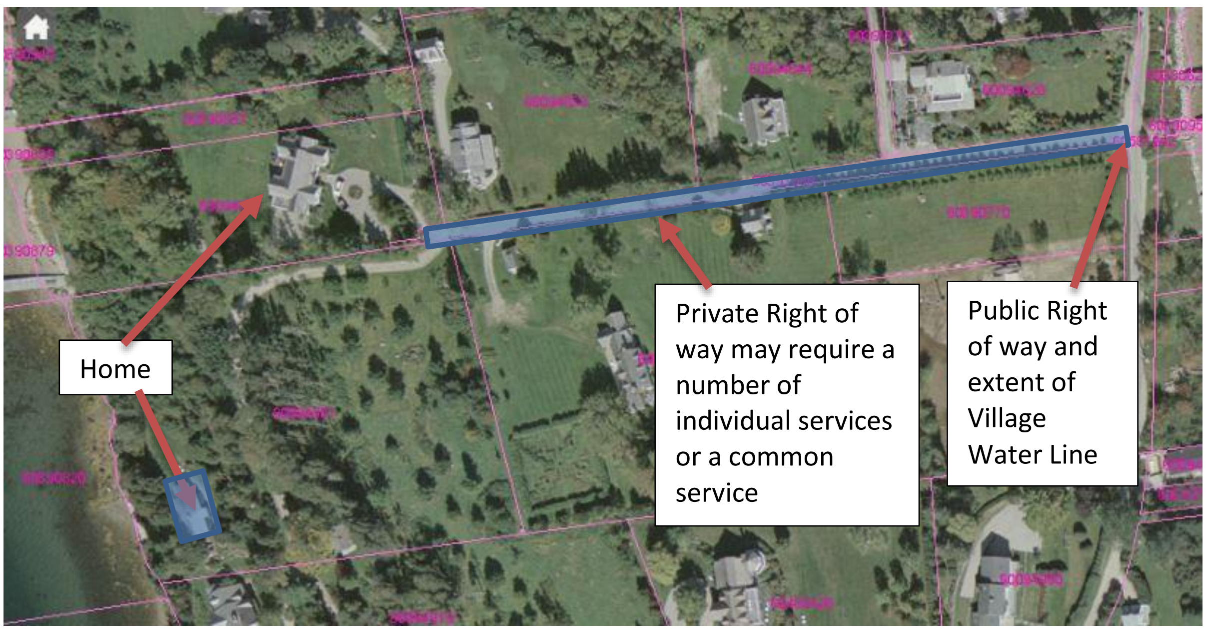


Figure 5.2: Servicing Considerations

5.3.2 Unobstructed Property Access

The streets in the village core are in a grid formation. This allows for easy looping of the water system which can help keep water distribution pipe sizing smaller. Throughout the construction period the grid road system also facilitates detours and allows access to be maintained to properties with relatively ease.

There are a number of areas within the Village boundary which may present access issues during construction:

- Nauss Point Road (south end).
- The peninsula.
- Walker Road (from Victoria Street).
- Chandler Road (from Millennium Drive).

These areas are generally identified on Figure 2.1. These four areas appear to be accessed by narrow dead end roads. During construction access to these areas would be very difficult for private vehicles and likely not possible for fire services or other large emergency vehicles. It is understood that many of the residents on the Peninsula are seasonal and therefore construction could be completed during the shoulder seasons to limit the impact to residents. Nauss Point Road, Chandler Road and, most severely, Walker Road residents will severely impacted access restrictions during construction. It is recommended that these areas be reviewed, if the Village intends to install services in these areas there should be significant public engagement as well as emergency contingency plan preparation. For the cost estimates included herein we have assumed that vehicular access will be blocked to all these areas during construction with transportation through the construction zone at schedule times.

5.3.3 Water Infiltration

There are areas within the Village boundary where installation of groundwater (sea water) into the excavated trench during watermain installation at typical depths of 1.6m cover may be of concern. Of particular note is Water Street, based on discussions with staff it is anticipated that water levels may be at as little as 1.2m below surface during high tides. In these locations it is likely that the pipe would be installed at less than standard cover, with insulation board as required to provide frost protection.

CHAPTER 6 CONCEPTUAL DESIGN

Conceptual design has been developed for each source water. Additional engineering will need to be complemented as part of a predesign phase to evaluate options and alternatives. The purpose of this section is to outline the system that was used to develop the capital cost estimate.

6.1 Surface Water

The concept design is based on a UF process, which also includes coagulation and flocculation. The following is a description of the process and how the system would operate.

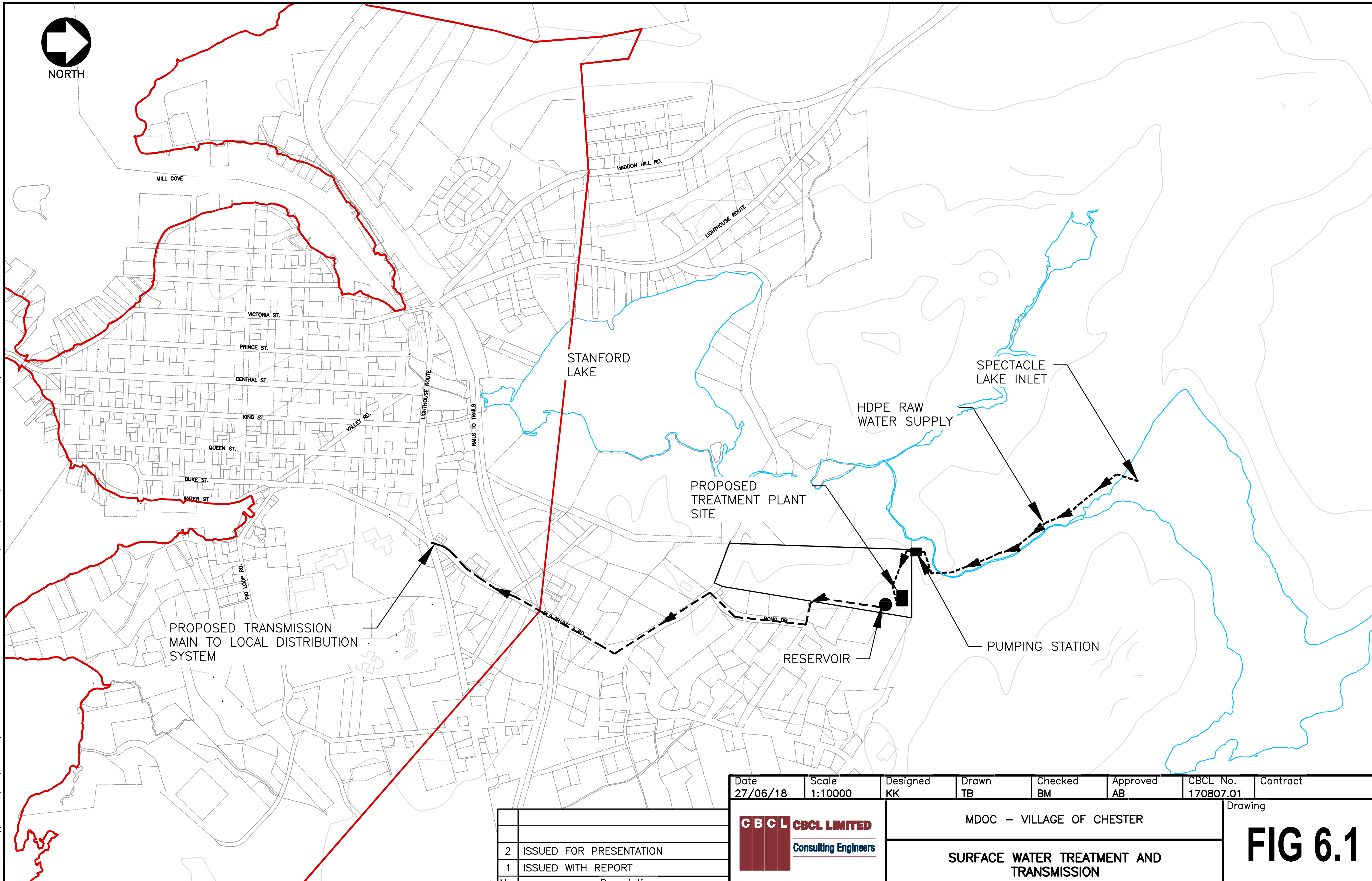
6.1.1 Treatment Plant Site

The proposed site for the treatment plant is the municipality owned property of Stanford Lake Road as shown on Figure 6.1. The plant has been located adjacent to the reservoir. The design intent is to extract water from the lake by gravity and run a gravity raw water line to the municipally owned property, from there pump raw water to the treatment plant.

The site has relatively steep grades. Preliminary road design has been undertaken and it indicates that the most cost effective access option from a construction stand point is from Bond Drive where a maximum road grade of 10% would provide access to both the treatment plant and the reservoir. Alternative access could be gained from Standford Lake Road, however the site has step grades and appears to have bedrock close to the surface near Stanford Lake Rd making driveway construction to the lower portion of the site difficult. Geotechnical investigation will be required to confirm soil conditions.

6.1.2 Intake

An intake will be required to withdraw raw water from the lake. The intake could consist of a screened concrete structure on shore with a channel to divert water to the structure or a submerged screened pipe that extended out into the lake. The piped option is the more common intake structure and will be used for the concept design.



No.	Description
2	ISSUED FOR PRESENTATION
1	ISSUED WITH REPORT

Date	Scale	Designed	Drawn	Checked	Approved	CBCL No.	Contract
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CBCL LIMITED Consulting Engineers

MDOC – VILLAGE OF CHESTER

SURFACE WATER TREATMENT AND TRANSMISSION

Drawing

FIG 6.1

DRAWING NAME: \\HFXDATA4\DATA\PROJECTS\170807.01 MDOC-VILLAGE OF CHESTER CENTRAL WATER\20 CAD\01 CIVIL\MUNICIPAL BOUNDARY 14-02-18\CONTOUR_MAP_PUBLIC.DWG LAYOUT NAME: FIG 6.1 SURFACE WATER TREATMENT PLANT PLOT DATE:June-27-18 3:03:44

The intake point would be located underwater at a location and elevation which promotes consistent water quality. The screen would be sized to comply with Department of Fisheries and Ocean (DFO) intake velocity requirements. It would also be located at a sufficient depth and distance from the shore to avoid impacts from the shore and surface. Screens are large and intake velocities are kept quite low, however they can be prone to sediment buildup. Sediment can be removed by divers and/or automatically by low pressure compressed air. The intake would ideally be located at a depth in the lake which would be low enough to avoid photosynthetic driven growth on the screen.

6.1.3 Low Lift Pump Station

Raw water will be supplied to the pump station through a 250 mm diameter HDPE main. Within the pump station a common header will supply two inline centrifugal pumps, which will operate on a duty/standby configuration, where each pump is capable of meeting maximum day flow requirements with the other unit out of service. Each pump will be designed to provide a flow to meet the treated water demands.

6.1.4 Treatment Process Description

Raw water will enter the plant through a 200 mm diameter pipe and will be dosed with soda ash, potassium permanganate, and a coagulant to oxidize dissolved manganese in the raw water and remove organics. Water will discharge to a tank that is sized to provide 20 minutes of flocculation time. Low pressure UF feed pumps will then supply pre-treated water to the UF system for pathogen reduction and particulate removal. UF permeate will discharge to a transfer tank, where high lift pumps will pumped the water to the treated water reservoir.

The UF membrane must be backwashed approximately once or twice every hour. The skid will automatically go through a reverse filtration (RF) cycle where the RF flow will be returned through the membrane at a rate of 150% of the normal flow. A chemical injection system pumps a small amount of chlorine into the reverse filtration water as it is fed to the modules. This will serve to remove some of the accumulated solids from the membrane surface and module and to prevent any biological growth from occurring on the membrane surface. Dechlorination of the backwash water is typically not required as the solids present in the waste exert enough of a chlorine demand to virtually eliminate any chlorine residual.

The modules may also require an additional air cleaning, where instrument-grade compressed air will be injected into the feed side of the module rack while maintaining the feed flow through the modules. This air scrubbing serves to shake the membrane surface and helps to dislodge foulants from the fibre surfaces.

Periodically, membrane systems require a more intensive cleaning in order to maintain the design flux rates and pressures for the membranes. This is called the clean-in-place (CIP) process. To do this, the membrane modules are taken off-line and isolated. Cleaning chemicals are then added to the system and recirculated as required to restore lost performance through the module. The CIP solution is also heated to allow the membranes to undergo a heat soak. The UF system is expected to require chemical cleaning once every month.

6.1.5 Disinfection

Chlorine remains the most cost-effective and attractive disinfectant as a result of the relative simplicity of operation and the creation of a persistent and measurable residual. Concerns regarding chlorination by-products, such as THM's, have focussed attention on the need to remove organic material from the source water prior to the application of chlorine. Provided this is effectively done, through the operation of a water treatment plant, chlorine remains the most viable of the disinfectant options. The conceptual design has therefore been developed based on the use of chlorine as the means of disinfection.

The treatment process must be capable of providing 3 log (99.9%) reduction of protozoa and a 4 log (99.99%) reduction in virus. The 3 log reduction in protozoa is the dominating criteria and if this condition is achieved the 4 log reduction in virus will be met.

Modern treatment systems provide greater than 2.5 log removal for protozoa. However, under the Nova Scotia Surface Water Treatment Standards the maximum credit that can be given to a treatment process is 2.5 logs for protozoa, this is to ensure that 0.5 log credits are achieved through disinfection. In order to provide the required 0.5-log inactivation credit by chlorination, 42 minutes of specific chlorine contact times must be provided prior to water reaching the first customer. This contact time can be achieved in a CT tank and the transmission main leading to the treated water storage tank. A substantial length of piping is available between the reservoir and the first customer service; however, flowrates in this section of piping are affected by fire flows, so residence time can vary significantly. Therefore, a CT tank has been included in order to meet CT requirements under fire flow conditions.

During the predesign phase a cost comparison between using UV to reduce the CT requires, and therefore the cost of CT system (piping or tanks) should be conducted.

6.1.6 Finished Water Conditioning

Downstream of the membrane system, the pH of the finished water will be adjusted to approximately 7.5 using soda ash. An orthophosphate corrosion inhibitor will then be added to the finished water to enhance corrosion protection within the distribution system. The chemical feed components will be similar for the various chemical feed systems (i.e., metering pumps and mixing/feed tank).

6.1.7 Residuals Management

Under the proposed option, the residuals streams would include:

- UF membrane backwashes.
- UF and NF CIP waste.

The UF backwash and CIP wastes may require treatment prior to discharge back to the source water due to the high solids that will be present in these streams.

Dealing with these high solids content waste streams often comprises a significant portion of the operating cost of a water treatment plant if there is no access to a municipal sewer. Generally, these wastes may be discharged directly to a sewer or treated on-site if the plant is not within close proximity to the sewerage system. As described in Chapter 4, the backwash waste will be pumped from an equalization tank to a residuals DAF system. Clarified subnatant will then be discharged to the nearby stream. Solids removed from the top of the clarifier will discharge to a holding tank. CIP waste will also discharge directly to the holding tank. The holding tank will be pumped out on a periodic basis. During the pre-design stage, residuals management should be further evaluated to determine if extending the collection system might be a viable option.

6.1.8 High Lift Pumping

Two high lift pumps will be provided to supply treated water from the UF transfer tank to the treated water reservoir. The high lift pumps will operate on a duty-standby configuration.

6.1.9 Facilities

The concept water treatment plant building is shown in Figure 6.2 and includes the following facilities:

- Office/Control Room.
- Laboratory.
- Washroom and shower.
- Workshop with storage.
- Process area.
- Chemical storage room.
- Electrical room.

6.1.9.1 LAYOUT

Access to the WTP is from the main door between the Office and the electrical room. Any visitors to the WTP must enter through the main doors between the Control Room and the Laboratory.

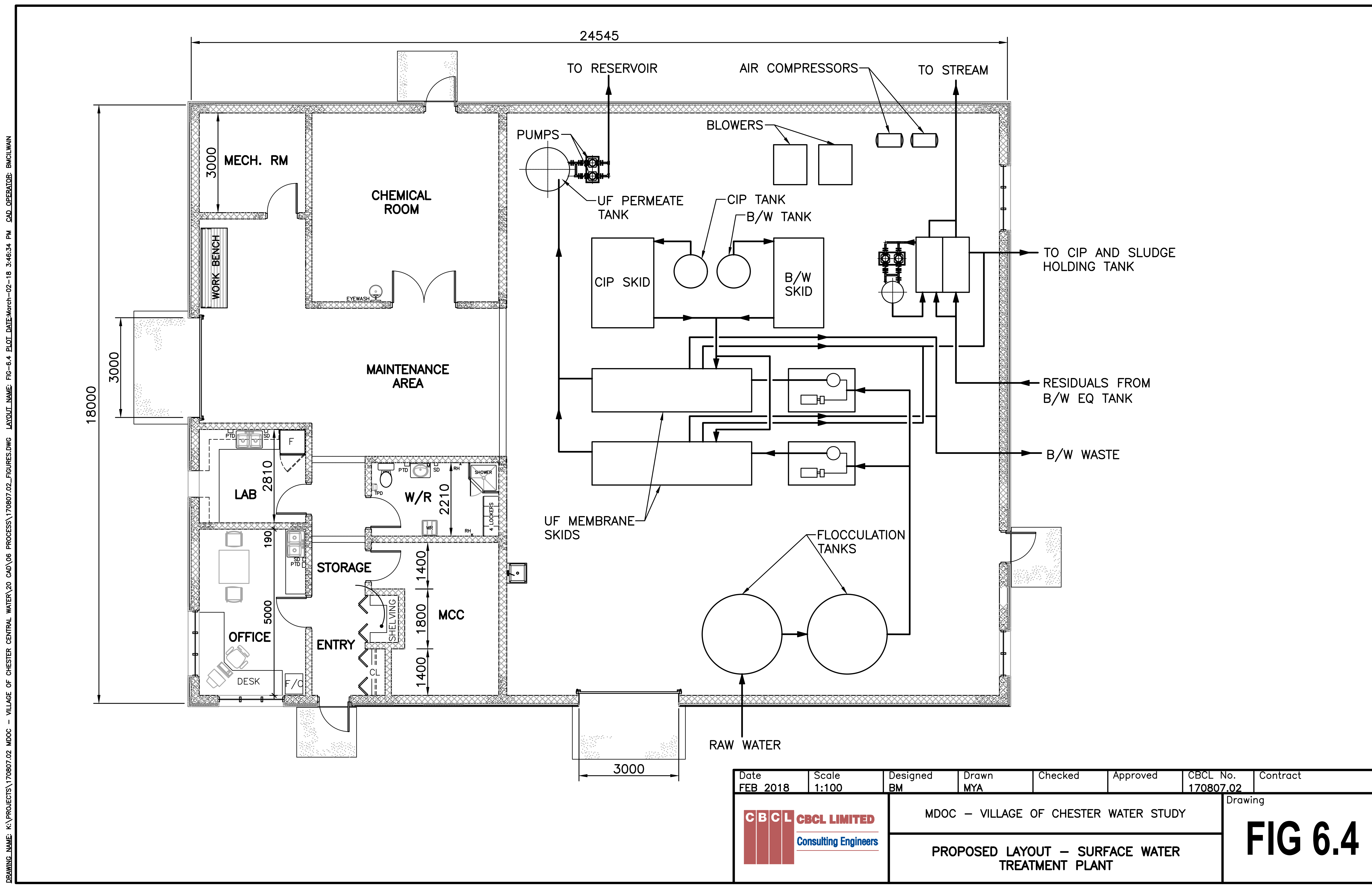
The treatment equipment will be located in a large Process Room. The space is sized to accommodate online instrumentation and air compressors while providing adequate headroom over the filter vessel skids. The height of the Process Room is dictated by the height of the membrane systems. The Process Room will have an overhead door to allow for movement of large equipment and delivery of supplies.

A maintenance area consisting of a workbench and double door exit is located adjacent to the Process room and Chemical Storage room. The Workshop has its own overhead doors and sufficient space to park a vehicle inside the room. The only access to the Chemical Storage room is through the Maintenance Area.

Chemicals (permanganate, soda ash, corrosion inhibitor, and chlorine) will be stored in a dedicated chemical storage room. The room will be separated into areas for the various chemicals, each with their own secondary containment. Chemicals will be delivered to the WTP in 200L drums on pallets, therefore materials handling equipment will be provided to allow delivery through the workshop overhead doors.

The electrical room contains the motor control centre, distribution panels and PLC equipment. The emergency backup generator is located outdoors, with buried electrical conduits routed directly to the electrical room.

The parking area and access roads at the WTP are sized to accommodate full sized transport truck deliveries of chemicals. The access road and parking areas consists of gravel.


24545
TO RESERVOIR
AIR COMPRESSORS
TO STREAM
BLOWERS
PUMPS
MECH. RM
3000
CHEMICAL ROOM
WORK BENCH
EYEWASH
UF PERMEATE TANK
CIP TANK
B/W TANK
CIP SKID
B/W SKID
TO CIP AND SLUDGE HOLDING TANK
MAINTENANCE AREA
18000
3000
RESIDUALS FROM B/W EQ TANK
B/W WASTE
LAB
2810
F
W/R
2210
SHOWER
LOCKERS
UF MEMBRANE SKIDS
FLOCCULATION TANKS
STORAGE
1400
1800
1400
MCC
5000
OFFICE
ENTRY
SHELVING
CL
DESK
F/C
190
RAW WATER
3000
Date FEB 2018
Scale 1:100
Designed BM
Drawn MYA
Checked
Approved
CBCL No. 170807.02
Contract
CBCL LIMITED Consulting Engineers
MDOC – VILLAGE OF CHESTER WATER STUDY
PROPOSED LAYOUT – SURFACE WATER TREATMENT PLANT
Drawing FIG 6.4
DRAWING NAME: K:\PROJECTS\170807.02 MDOC – VILLAGE OF CHESTER CENTRAL WATER\20 CAD\06 PROCESS\170807.02_FIGURES.DWG LAYOUT NAME: FIG-6.4 PLOT DATE:March-02-18 3:46:34 PM CAD OPERATOR: BMCILWAIN

6.1.9.2 Architectural Finish

The treatment plant building consists of a concrete slab on grade. Walls will be concrete block with fibre cement board siding. Interior walls will be constructed of masonry block. The roof is a hip roof constructed of timber trusses with metal roof finish. The underside of the timber trusses are sealed from the process areas below with plywood with epoxy finish.

6.1.9.3 Mechanical

Building heating and cooling is provided by a split heat pump system in the administrative areas (Control Room, Meeting Room, Laboratory and Washroom). The Process Room, Chemical Storage Room each include heating, ventilation and dehumidification. Building heat is supplemented by unit heaters and baseboard heaters in the administrative area.

The Chemical Storage Room has a separate heating and exhaust system. Mechanical devices, vents, ductwork, heaters, etc. in the chemical room will be selected to provide maximum corrosion resistance. No return air will be drawn from the chemical room.

6.1.9.4 Electrical

An emergency backup power generator will provide sufficient power to operate the entire WTP and low lift pump station upon main power loss, and will be located adjacent to the plant in a sound enclosure with dedicated integral fuel storage tank. Power is transferred to the generator by an automatic transfer switch in the Electrical Room.

6.2 Groundwater Option

6.2.1 Treatment Plant Siting

The site would need to accommodate the production wells as well as a treatment building to house pump controls, monitoring equipment, and any treatment equipment. The concept facility we are proposing would be located to the west of the Village, as shown in Figure 6.3. No particular parcel has been selected, as the location will be dependent on the location of the well field.

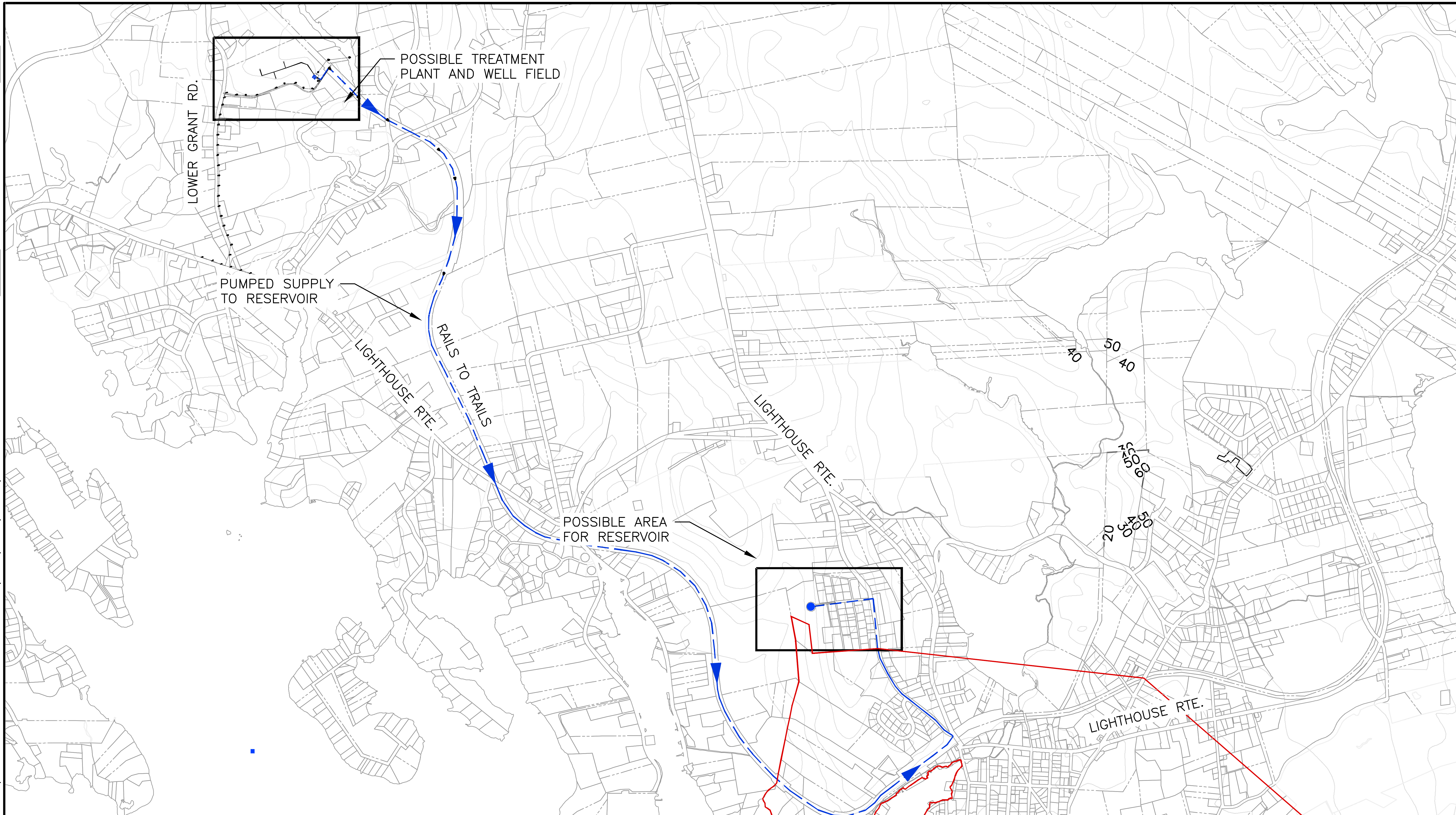
6.2.2 Wellfield

As described in Section 3.2, there may be potential for a well field in the Middle River area, approximately six to seven kilometres from the Village. This location was used to develop the concept design for groundwater supply.

The characteristics of existing wells in the Middle River area suggest that a municipal well field would consist of three or more production wells:

- Well depths would be on the order of 30 to 50 metres.
- Wells would be constructed using stainless steel, wire wrapped screens.
- Well diameters would be 200 to 250 mm, depending on individual well capabilities and corresponding pump sizes.
- Required individual well yields would need to be on the order of 325 to 650m^3/d.

DRAWING NAME: \\HFXDATA4\DATA\PROJECTS\170807.01 MDOC-VILLAGE OF CHESTER CENTRAL WATER\20 CAD\01 CIVIL\SITE PLAN\170807.01 GROUND WATER SITE PLAN PUBLIC.DWG LAYOUT NAME: FIG 6.5 WELL FIELD AND TRANSMISSION MAIN PLOT DATE:June-27-18 3:43:1



No.	Description
2	ISSUED FOR PRESENTATION
1	ISSUED WITH REPORT

Date	Scale	Designed	Drawn	Checked	Approved	CBCL No.	Contract
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CBCL LIMITED
Consulting Engineers

MDOC – VILLAGE OF CHESTER

WELL FIELD AND TRANSMISSION MAIN

Drawing

FIG 6.3

6.2.3 Treatment Process Description

The layout for a groundwater treatment process is included in Figure 6.4. The system will be designed to allow both chlorine addition and permanganate addition as oxidants for iron and manganese removal upstream of the oxidative media filters. It is anticipated that under normal operating conditions that chlorine alone will be used as the oxidant in a catalytic oxidation process arrangement. To allow system flexibility, a permanganate dosing system will be designed to allow the system operate in an intermittent or continuous regeneration process arrangement if desired, where permanganate is dosed in conjunction with chlorine, or alone; continuously or at intervals to allow media reconditioning on an intermittent basis.

Oxidants will be dosed into the pre-filtered water prior to the greensand filters in a common pipe. An injection quill will be used to deliver a metered dose of chemical to the center of flow in the pipe, prior to the filter inlets. Mixing will be achieved using a static mixer element, which will be inserted into a segment of vertical pipe in the common greensand filter inlet header.

6.2.4 Disinfection

Similar to the surface water treatment option, disinfection will be achieved by chlorination. The disinfection requirement changes depending if the well is groundwater under the direct influence (GUDI) of surface water or non-GUDI. Wells that are classified as GUDI are treated as surface water under the Nova Scotia Treatment Standards for Municipal Drinking Water Systems. This standard defines a GUDI well as “any water beneath the surface of the ground with:

- Significant occurrence of insects or other macro-organisms, algae, organic debris, or large-diameter pathogens such as *Giardia lamblia* or *Cryptosporidium*.
- Significant and relatively rapid shifts in water characteristics such as turbidity, temperature, conductivity, or pH which closely correlate to climatological or surface water conditions.”

We have made the assumption that the new wells will be constructed to current standards and will be non-GUDI. Since the wells will be non-GUDI they should not contain protozoa and therefore only need to meet the 4 log inactivation of viruses. Adequate chlorine contact time must be provided to the finished water prior to reaching the first customer. This will be provided in the transmission main with no requirement for a contact chamber.

6.2.5 Finished Water Conditioning

The treatment process will include chemical feed systems for potassium permanganate, and depending on raw water quality, pH adjustment and corrosion inhibitor chemicals. These systems would include a batch tank, mixer, and metering pumps.

6.2.6 Residuals Management

The residuals formed during the treatment process contain high concentrations of iron and manganese. Typically this waste stream is of low volume but it is generated over a very short period of time. If the water treatment plant is located near the wastewater collection system, then the preferred option is to discharge to the sewer. In remote areas, with no access to municipal wastewater treatment plants, settling basins/tanks may be used to settle the suspended metals allowing the clarified water to be discharged back to the environment.

For the concept provided, backwash wastewater will discharge to an equalization tank and then be pumped to a small unlined lagoon, sized to hold one month's storage. Overflow from the lagoon will discharge to the Middle River.

6.2.7 Facilities

The concept water treatment plant building layout is shown in Figure 6.4 and includes the following:

- Office/Control Room.
- Laboratory.
- Washroom and shower.
- Workshop with storage.
- Process area.
- Chemical storage room.
- Electrical room.

6.2.7.1 Layout

Access to the WTP is from the main door between the Office and the electrical room. Any visitors to the WTP must enter through the main doors between the Control Room and the Laboratory. An entry canopy at the main entrance provides shelter for visitors while waiting at the front door.

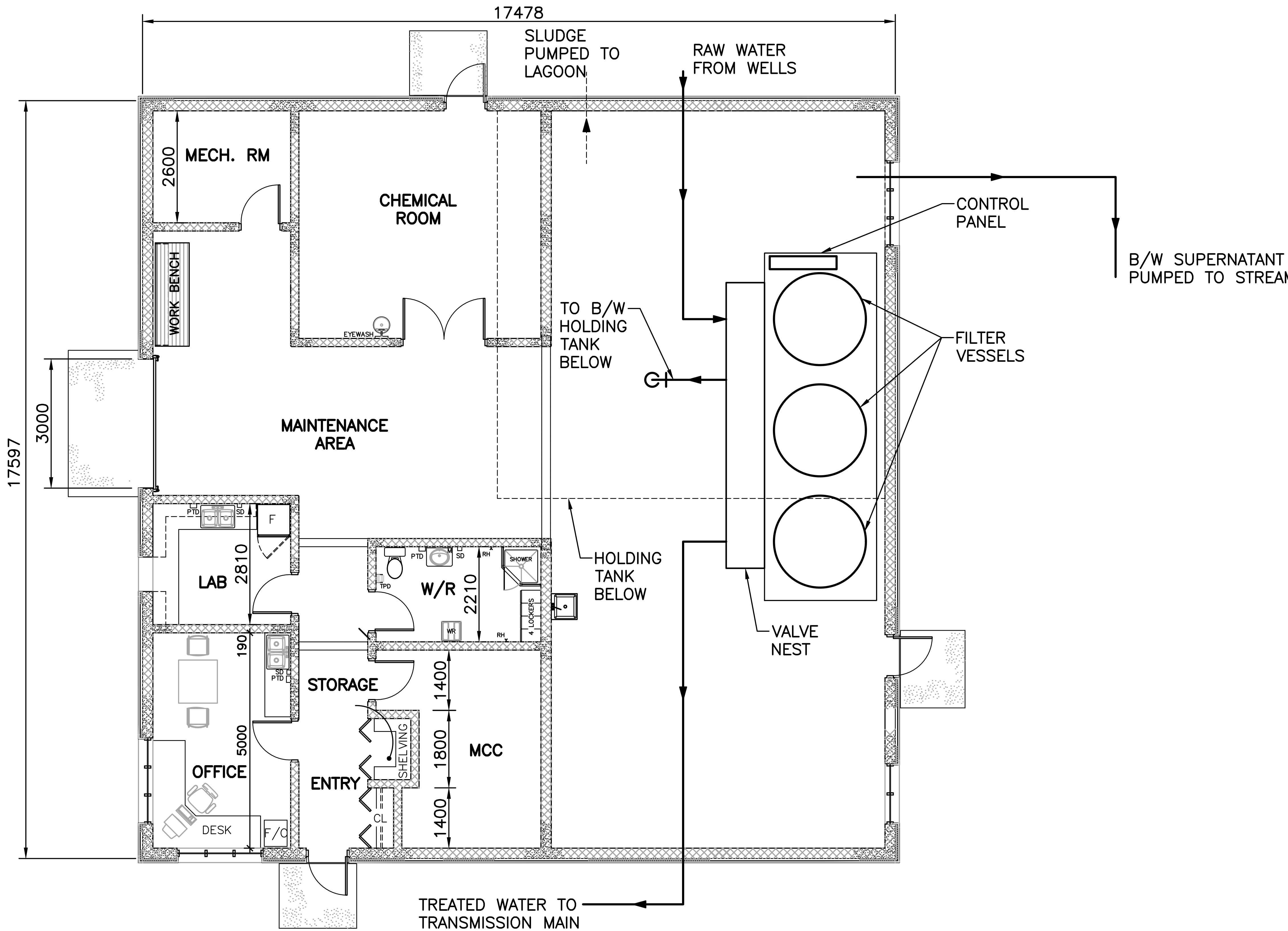
The treatment equipment will be located in a large Process Room. The space is sized to accommodate online instrumentation and air compressors while providing adequate headroom over the filter vessel skids. The height of the Process Room is dictated by the height of the filter vessels. The Process Room will have an overhead door to allow for movement of large equipment and delivery of supplies.

A maintenance area consisting of a workbench and double door exit is located adjacent to the Process room and Chemical Storage room. The Workshop has its own overhead doors and sufficient space to park a vehicle inside the room. The only access to the Chemical Storage room is through the Maintenance Area.

Chemicals (permanganate, corrosion inhibitor, and chlorine) will be stored in a dedicated chemical storage room. The room will separated into areas for the various chemicals, each with their own secondary containment. Chemicals will be delivered to the WTP in 200L drums on pallets, therefore materials handling equipment will be provided to allow delivery through the workshop overhead doors.

The electrical room contains the new motor control centre, distribution panels and PLC equipment. The emergency backup generator is located outdoors, with buried electrical conduits routed directly to the electrical room.

The parking area and access roads at the WTP are sized to accommodate full sized transport truck deliveries of chemicals. The access road and parking areas consists of gravel.



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CBCL LIMITED
Consulting Engineers

MDOC – VILLAGE OF CHESTER WATER STUDY

PROPOSED LAYOUT – GROUND WATER TREATMENT PLANT

Drawing

FIG 6.6

6.2.7.2 Architectural Finish

The treatment plant building consists of a concrete slab on grade. Walls will be concrete block with fibre cement siding. Interior walls will be constructed of masonry block. The roof is a hip roof constructed of timber trusses with metal roof finish. The underside of the timber trusses are sealed from the process areas below with plywood with epoxy finish.

6.2.7.3 Mechanical

Building heating and cooling is provided by a split heat pump system in the administrative areas (Control Room, Meeting Room, Laboratory and Washroom). The Process Room, Chemical Storage Room each include heating, ventilation and dehumidification. Building heat is supplemented by unit heaters and baseboard heaters in the administrative area.

The Chemical Storage Room has a separate heating and exhaust system. Mechanical devices, vents, ductwork, heaters, etc. in the chemical room will be selected to provide maximum corrosion resistance. No return air will be drawn from the chemical room.

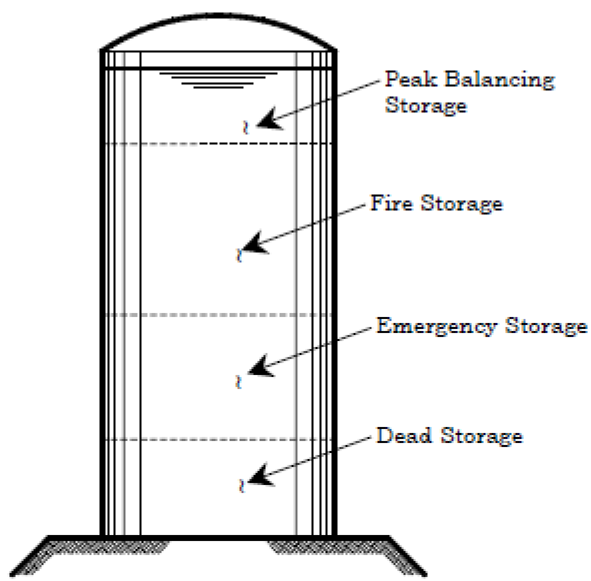
6.2.7.4 Electrical

An emergency backup power generator will provide sufficient power to operate the entire WTP and well pumps upon main power loss, and will be located adjacent to the plant in a sound enclosure with dedicated integral fuel storage tank. Power is transferred to the generator by an automatic transfer switch in the Electrical Room.

6.3 Storage, Transmission & Distribution

6.3.1 Treated Water Storage

The storage tank has been sized to provide peak balancing storage, fire storage, emergency storage, and dead storage. An illustration of a typical reservoir is provided to the right.



Peak balancing storage provides storage to balance peak systems demands with exceed the treatment capacity of the system. The volume is typically a function of diurnal demand in the community and is commonly estimated at 25% of the maximum day demand.

Fire storage is based on the level of fire protection the community selects. For this report we have used a conservative fire flow. The conservative storage volume would be calculated based on 3 hrs of fire flow flows. The calculated fire flow is 236 l/s (3,740 usgpm) which equates to a storage volume of 2,520 m3 (665,713 usgal).

Emergency storage is the volume of water in reserve to cover maintenance shut-down or emergency situations such as source of supply failure or other major incidents. A standard method of calculating this volume is 25% of the sum of the peak balancing volume and fire storage.

Dead storage is the volume of water that cannot be used as it is not an elevation to generate adequate flows and pressures in the system.

A summary table is given below.

	Metric	Imperial	Note
Fire Flow	236 l/s	3,740 USGPM	Based on 42 Queen St
Fire Storage (FS)	2,520 m^3	665,713 USG	Fire flow x 3 hrs operation
Peak Balancing Storage (PBS)	475 m^3	125,600 USG	25% Max day (1900 m^3/d)
Emergency Storage	750 m^3	197,601 USG	25% (PBS + FS)
Total Reservoir Vol	3,745 m^3	990,740 USG	
Transmission Main	500mm	20 inch	From Bond Drive to Village Core

6.3.1.1 Surface Water

Figure 6.1 shows the proposed location for the treated water storage tank for the surface water option. The tank would be relatively close to the treatment facility and located on the same parcel of land. The site elevation is relatively high, with a potential reservoir base elevation of approximately 65m el.

6.3.1.2 Groundwater

Figure 6.3 shows the proposed location for the treated water storage tank for the groundwater option. The reservoir would be located west of Stanford Lake on Haddon Hill. No particular site has been identified but a site with an elevation of approximately 70 m would be preferred. It is, however, located further from the treatment site than the surface water option, so the requirements for a transmission main or operations to ensure tank turnover would be more demanding. It is also anticipated that there may be challenges associated with land acquisition in this area.

6.3.2 Supply and Distribution

The distribution system service area has been established based on the Village boundary. Further refinement is recommended to identify areas that are impractical to service and additional areas where water main could be installed at a reasonable cost per service.

Treated water will be supplied to the Village through a 500 mm diameter PVC transmission main. Water mains within the centre of the Village will consist of 250 mm diameter and 200 mm diameter PVC pipes. A 375 mm diameter water main will supply the commercial area that includes the mall. A 300 mm diameter water main will supply the area that includes the golf course. All other areas of the Village will be serviced by 200 mm diameter water main.

6.4 Private Wells

Conceptually the onsite well supplies would continue to operate as the currently do. However, these system should be brought up to current standards which would requiring upgrading or replacing many of the existing dug wells and possibly adding or upgrading associated treatment systems.

Over a 25 year design period, households and businesses will pay a cost for maintaining their water supply systems. Assumptions have been made to capture this cost in the analysis for a centralized water supply system.

Approximately 820 civic addresses were counted within the proposed water servicing boundary. The following proportions, obtained from the survey that was conducted during the previous phase of work, have been used to estimate the extent of upgrades that would be required for private well owners:

- 55% of survey respondents reported having a dug well.
- 34% of survey respondents reported having a drilled well.
- The remainder of respondents had cisterns, multiple wells, or no wells.
- 30% of dug wells and 7.5% of drilled wells reported experiencing water shortages.
- 21% of existing drilled wells did not already have a treatment system.

A large proportion of the existing dug wells do not meet the current construction standard. Therefore, it has been assumed that all existing dug wells will be replaced within a five year period. Dug wells that had previously experienced water shortages would be replaced with a drilled well, whereas those that have not experienced water shortages will be replaced with a new dug well that meets current construction standards. It is assumed that 7.5% of all drilled wells will continue to experience shortages and will require new drilled well and that 21% of drilled well will require a treatment system. We have assumed the "other" water source types, i.e. cisterns, multiple wells, will require treatment only.

Treatment for dug wells would include reverse osmosis filtration to achieve pathogen reduction, as well as improve aesthetic parameters. Treatment for drilled wells would include softener systems to reduce iron and manganese concentrations. Existing drilled well owners who do not currently have treatment systems will also require softener systems for metals removal. The number of anticipated upgrades are listed below in Table 6.1.

Table 6.1: Modifications to Existing Private Wells

Parameter	Total	Dug	Drilled	Other
Number of Existing Water Supplies	820	444	280	96
Sites Requiring Complete Upgrades	465	444	21	
Sites Requiring Treatment Upgrades Only	155		59	96

6.5 Opinion of Probable Costs

6.5.1 Capital Costs

Capital cost opinions have been developed for the WTP and distribution system based on industry experience, past plant construction costs, and budgetary equipment quotations. Additional costs incurred for civil works, building, storage tanks, additional process piping and electrical requirements have been added using unit rates developed by CBCL Limited based on similar projects.

It should be noted that there are several variables in the assessment that require further study, which have significant impacts on the overall cost of the project. This includes the location of the plant and the storage tank, the type of treatment selected, and selection of residuals handling methods. Consequently, the values presented below are only developed for the purpose of comparing the three main source of supply options: centralized surface water, centralized groundwater, or private wells. They do not represent the actual cost to construct the facility, but instead, provide relative values to highlight the cost difference between the options.

The capital cost breakdown is shown in Table 6.2.

Site works includes all yard piping which is piping outside the WTP such as wastewater piping, raw water supply, treated water piping, and service connections. Site works will also include site finishes such as reinstatement, power extensions, fencing, etc.

For the surface water source option, costs were based on an integrated membrane system using UF for pathogen reduction and NF for organics removal. Costs will vary for other membrane systems, such as a UF system with chemical pre-treatment.

For the groundwater supply option, costs were based on a conventional oxidative media filtration system. Costs may vary for other processes, such as biological filtration or ion exchange.

Based on discussions with staff it is anticipated that no significant rock breaking will be required in or around the Village core. Rock, if encounter, is reported to typically be soft shale which does not require rock breaking equipment to be removed.

For private well systems, costs were based on replacement of all dug wells, installation of new treatment at dug wells and at existing drilled well sites that previously did not include treatment. It has been assumed that all sites that experienced water shortages will have a new drilled well installed. Capital costs were estimated at $6000 for a new dug well, $6000 for a new drilled well, and $4000 for new treatment equipment for both types of wells. These cost are used were developed to allow for an equal comparison in terms of water quality standards between onsite wells and a central water system. It must be understood that this cost only considers improving the construction of the wells and water quality issues, these costs do not address quantity issues and it is expected that quantities issues will continue to be a concern after well improvement are made.

The opinion of probable costs is considered a Class D level of effort and is presented on the basis of experience, qualifications, and best judgement. It has been prepared in accordance with acceptable principles and practices. Sudden market trend changes, non-competitive bidding situations, unforeseen labour and material adjustments, and the like are beyond the control of CBCL Limited and as such we cannot warrant or guarantee that actual costs will not vary significantly from the opinion provided.

Table 6.2: Class D Capital Cost Opinion

Description	Source		
	Surface Water	Groundwater	Private Well
Treatment			
Supply & Treatment	$6,313,000	$4,137,000	
Storage Reservoir	$1,495,000	$1,495,000	
Transmission & Distribution	$20,451,000	$23,894,000	
Subtotal	$28,259,000	$29,526,000	
Design Development Contingency (15%)	$4,239,000	$4,429,000	
Construction Contingency (7%)	$1,978,000	$2,067,000	
Professional Services (15%)	$4239,000	$4,429,000	
Total	$38,715,000	$40,451,000	
HST	$5,807,000	$6,068,000	
Total Construction Cost (with HST)	$44,522,000	$46,519,000	$5,270,000

Supply and treatment covers all items associated with the treatment system and water treatment plant site, including the intake, raw water main, wellfield and well pumps, treatment equipment, instrumentation, and the building. Transmission and distribution system components include treated water main to the storage tank, treated water transmission main from the storage tank, and all distribution system water mains. Costs associated with the reservoir are provided separately.

Note these cost include servicing all areas noted within the proposed service area on Figure 1.1, which includes the areas outside the Village boundary. The estimated cost to services the areas outside the Village limits are $600,000 (including contingency) and could be deducted from the total before HST presented above if these areas were not to be serviced. Also, if only Zone A was to be serviced, per Figure 2.1, meaning the areas outside the service boundary and areas B, C, D and E were not serviced the total capital cost reduction, before HST, would be about $6,600,000 which includes contingency.

6.5.2 Operation and Maintenance Costs

O&M costs are based on annual average water use. The estimated O&M costs for each proposed treatment system are summarized below in Table 6.3. A description of each O&M cost item is described below the table.

Table 6.3: Annual Operation and Maintenance Costs

Description	Source		
	Surface Water	Groundwater	Private Well
Labour	$75,000	$27,000	$820,000
Power	$29,000	$15,000	Incl.
Chemicals	$32,000	$11,000	Incl.
Miscellaneous Replacements	$14,000	$5,000	Incl.
Residuals	$36,000	$10,000	none
Total Annual O&M	$186,000	$68,000	$820,000

6.5.2.1 Labour

Annual operation and maintenance costs were based on operation at average day flow. Labour costs were estimated based on hourly rates of $35 per hour for a supervisor, and $25 per hour each for a primary and secondary operator.

For the surface water option, it was assumed that a full time operator would be required. A part-time secondary operator would be required to assist with CIPs and other miscellaneous tasks for several days per month.

For the groundwater option, it was assumed that the plant would only require a few site visits per week to perform visual inspections, calibrate instrumentation, and top-up chemicals. A secondary operator was not included for the groundwater system.

For the private well option, it was assumed that all sites pay an annual service fee to a well treatment system supplier to maintain private well systems.

6.5.2.2 Power

Power costs were calculated only for major pumping equipment. They do not include for building heating, ventilation, or lighting. They also do not account for power consumption from control panels, mixers, blowers, metering pumps, computers, or other auxiliary equipment.

6.5.2.3 Chemicals

The UF membrane system requires a coagulant to reduce dissolved organic matter. It would also require pre-treatment oxidation for removal of iron and manganese using potassium permanganate and soda ash to achieve a high pH. It has been assumed that soda ash will be used to increase treated water pH to a suitable level, and that a corrosion inhibitor would be dosed to the finished water. Membrane cleans would likely require citric acid for the low pH clean to remove inorganic foulants, such as iron and manganese, and a high pH wash using caustic soda to remove organic foulants.

For the groundwater option, chemicals would include potassium permanganate, sodium hypochlorite for disinfection, and soda ash and orthophosphate for finished water adjustment.

6.5.2.4 Replacement Costs

The largest component of the O&M costs added by the membrane based treatment systems is from membrane module replacement. This cost is based on a 5-year replacement frequency period. The projected frequency of membrane replacement is typical based on operating experience. Modules are expected to see a diminished output over successive years of operation. Should the plant output and water quality be sufficient at 5 or more years from commissioning the module replacement can be postponed until required. This has been common in past installations. Chemical feed pumps also have a life expectancy of 5 years on average, though they may last 10 years or longer.

Process pumps typically have a life span of 20 years in this environment with proper maintenance such as seal replacement and lubrication. Valves and piping will typically have a life span of more than 20 years. To account for unexpected failures and premature failures which may occur at any time an annual allowance was provided. In some years this may not be used while in others it may be exceeded.

6.5.2.5 Residuals

UF backwash residuals will be treated by a DAF system. Subnatant will be discharged to the environment. Thickened sludge will be stored in a wastewater holding tank and pumped out every two weeks.

Residuals from the groundwater treatment option are anticipated to be minimal. Backwash will discharge to an equalization tank. The solids will be allowed to settle before the clarified water can be discharged to the nearby river. Sludge will be pumped to a lagoon.

6.6 Financial Assessment

The Social-Economic report, see Appendix A, evaluated the monetized costs to the Village to install a central water system, considering the 75% funding and financing costs, and compared this cost to the forgone cost of upgrading the existing private well system. This is a summary of the cost of installing a central water system versus the benefits of abandoning the existing private well system. Please refer to Appendix A for assumptions and additional information on how these numbers were developed. Tables 6.4 and 6.5 were taken from the Social-Economic report and represent the cost of central water systems to the Village.

Table 6.4: Monetized Cost of Surface Water System

Monetized Costs to Village Ratepayers(2018$): Surface Water System					
	Connection Cost (includes financing)[1]	Delayed Capital Charge (includes financing)[2]	Connection from Property Line to Buildings (includes financing)[1]	Operating & Maintenance[3]	Total Costs
Year 1	$835,721		$379,325	$186,000	$1,401,046
Year 2 - 10	$835,721		$379,325	$186,000	$1,401,046
Year 11 -25		$625,244		$186,000	$811,244
Total Yr. 1 -25	$8,357,210	$9,378,653	$3,793,254	$4,650,000	$26,179,117
1. Financing by ratepayers at 6% rate, 10 year amortization.					
2. Financing by MoDC at 4% rate, 25 year amortization charged in 15 installments.					
3. Covered by the base rate and consumption charges.					

Table 6.5: Monetized Cost of Groundwater System

Monetized Costs to Village Ratepayers(2018$): Groundwater System					
	One Time Connection Cost (includes financing)[1]	Delayed Capital Charge (includes financing)[2]	Connection from Property Line to Buildings (includes financing)[1]	Operating & Maintenance[3]	Total Costs
Year 1	$835,721		$379,325	$68,000	$1,283,046
Year 2 - 10	$835,721		$379,325	$68,000	$1,283,046
Year 11 -25		$678,491		$68,000	$746,491
Total Yr. 1 -25	$8,357,210	$10,177,362	$3,793,254	$1,700,000	$24,027,826
1. Financing by ratepayers at 6% rate, 10 year amortization.					
2. Financing by MoDC at 4% rate, 25 year amortization charged in 15 installments.					
3. Covered by the base rate and consumption charges.					

Once funding and financing have been added to the financial the central groundwater option becomes the lowest cost central water system at $24,027,826 compared to $26,179,117 for a surface water system.

The cost benefit to the Village of abandoning the private well system, including the foregone cost to bring the wells up to current standards and to provide adequate treatment and maintenance, is provided in Table 6.6. Also included in this table is the reduction in fire insurance premiums which would be gained by a central water system designed to provide fire protection.

Table 6.6: Monetized Benefits of Abandoning Private Wells

Monetized Benefits to Village Ratepayers(2018$)[1]				
	Foregone Well Capital Upgrades	Foregone Well Operating & Maintenance	Reduction in Fire Insurance Cost	Total Benefits to Ratepayers
Total Yr. 1 - 25	$6,651,450	$20,500,000	$8,535,328	$35,686,778
1. Benefits (foregone costs and fire insurance cost reduction) are the same for both central water systems.				
2. Includes financing at 10%, 5 year average amortization.				

The monetized benefit to the village by forgoing the upgrades to the private wells is about $35,686,778 if fire protection is added to the forgone cost. If fire protection is not provided in the central system the monetized benefit of the private well system would be $27,151,450. With or without fire protection the central system has a lower cost to the village compared to upgrading the private well systems.

In summary the cost of each system is as follows, assuming fire protection is provided with the central system:

- Central Groundwater System - $24,027,826
- Central Surface Water System - $26,179,117
- Upgrading Private Wells - $35,686,778

If fire protection is not provided with the central system the costs this reduces the capita cost of the central system and removes the fire insurance benefit from the private well option. The cost are as follows:

- Central Groundwater System - $20.627,826
- Central Surface Water System - $22,779,117
- Upgrading Private Wells - $27,151,450

In all cases the groundwater option is slightly less expensive, however, at this point in the development of cost the surface water system would be within the accuracy of the groundwater cost and considered roughly equal.

CHAPTER 7 CONCLUSIONS AND NEXT STEPS

7.1 Conclusion

- The status quo option of continuing to use private water supplies was identified to be the lowest cost option; however, issues with water quality and quantity are anticipated to persist and possible get worse as weather patterns change. A central water system would substantially improve the consistency, quality and access to water for many residents of the Village.
- Installing a central water system has a lower cost to the Village compared to upgrading the individual private well systems.
- The groundwater central water system has a lower capital cost compared to the surface water central system when financing and funding are accounted for.
- A pre-design study should be completed in order to determine the design parameters. The pre-design study should include: additional raw water sampling over a period of at least one year to identify seasonal water quality trends; treatability testing, which could include jar testing and piloting; a detailed analysis of residuals treatment and disposal options; and refinement of the design flows based on future population of the finalized distribution system service area; update on surface water yield; test wells for ground water.

7.2 Next Steps

1. **Determine if the community would like to have central water or continue with onsite wells (2018).**
2. **Select surface water or groundwater (2018/19):**
   2.1. If a groundwater source is preferred further effort would be required in order to identify and develop a suitable wellfield. This would consist of drilling test wells, identify long terms yields, and preparing hydrological reports for approval by NSE. This process can be lengthy and costly depending on the number and depth of wells required to meet the yield requirement. This options also comes with the risk that sufficient groundwater is not identified.
   2.2. If surface water is selected collect addition water quality information and complete an updated yield assessment based on the new NSE standard.
   2.3. A predesign study would be required once this is completed and water yield and quality are known.

3. **Prepare a Predesign Report to address the outstanding questions such as (2019):**
    3.1. Define the servicing area as it will impact the overall service population, design parameters, and costs. There are a number of areas within the Village boundary that have a high cost of servicing, and should potentially not be included in the initial development phase of the water system. Alternatively, there are several areas outside of the Village boundary that could be serviced at a relatively low additional cost, and be considered for inclusion within the servicing area.
    3.2. Confirm future growth estimates to allow for sizing of the treatment and distribution for phasing and future expansion.
    3.3. Confirm fire flow requirements. We would recommend a stakeholder engagement session be held to discuss firefighting procedure with local fire fighters. The goal of this meeting would be to outline the impacts of providing the village with firefighting water via the proposed water system. An appropriate level of service should be agreed upon.
    3.4. Confirm reservoir location, transmission & distribution pipe sizing and layout.
    3.5. Confirm treatment process and facility requirements.
    3.6. Review residuals treatment options. If the water treatment facility is not connected to the central sewer system, then residuals discharge requirements should be confirmed and treatment options should be evaluated to identify a low-cost option that meets discharge requirements. Generally, these specify that discharge must not be toxic to aquatic life, maximum total suspended solids must be below 25 mg/L, and aluminium concentrations must not exceed 0.1 mg/L or the background levels of the receiving water.
    3.7. Develop construction methodology to address construction issues such as limited access and high groundwater.
    3.8. Develop higher level cost estimates

4. **Engage UARB (2019)**
    4.1. The process of establish water utility regulations and financial framework should be initiated at this step for presentation to the URAB

5. **Develop Source Water Protection Plan to protect the source water from contamination. There are five steps outlined in the Treatment Standard (2019), including:**
    5.1. Form a Source Water Protection Advisory Committee.
    5.2. Delineation of a Source Water Protection Area Boundary.
    5.3. Identify Potential Contaminants and Assess Risk.
    5.4. Develop a Source Water Protection Management Plan.
    5.5. Develop a Monitoring Program to Evaluate the Effectiveness of a Source Water Protection Plan.

6. **Design and Approval Stage (2020):**
    6.1. Develop preliminary designs of accepted processes
    6.2. Prepare design drawings
    6.3. Develop higher level cost estimates
    6.4. Apply for permits and approvals such as wetland alternation, water withdrawal approval, permit to construct and operate
    6.5. Tender

6.6. Construction
6.7. Operation

7. Construction to start 2021

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APPENDIX A

Jozsa Management & Economics “Socio-Economic Effects of a Central Water System for the Village of Chester”

Socio-Economic Effects of a Central Water System for the Village of Chester

Prepared For:

Municipality of the District of Chester

THE MUNICIPALITY OF CHESTER

Submitted By:

JOZSA MANAGEMENT & ECONOMICS

June 2018

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1 INTRODUCTION

This document describes the key socio-economic effects of the provision of a central water system to the Village of Chester in the Municipality of the District of Chester (MoDC).

It:

- considers monetized costs and benefits of providing a central water system to ratepayers in the Village of Chester;
- measures incremental changes in population, jobs, residential assessment and commercial assessment in the MoDC that are plausibly linked to the construction of a central water system; and
- identifies non-monetized impacts that would accrue to the Village, the rest of the MoDC and to neighbouring communities.

1.1 NEED FOR A CENTRAL WATER SYSTEM

The Village has been exploring the idea of central water for nearly 50 years. Central water can assist in mitigating negative impacts associated with four main issues areas.

First, CBCL's 2017 water supply assessment concluded that there are currently water quality issues as well as water shortages within the Village. Businesses in the Village have reported that water shortages have constrained their business operations, and by implication constrain investment expansions. Issues affecting the reliability of individual wells in the Village included the following:

- Many dug wells do not meet current standards for construction.
- Water shortages have affected 23% of survey respondents (Section 1.2 Existing Conditions).
- Aesthetic and other issues were noted by 31% of survey respondents and identified in 85% of water samples (Section 3.3.1 Description).
- 62% of raw water supplies contained coliform bacteria (Section 3.3.1 Description).
- Many homes in the Chester area draw water from wells adjacent to the coastline. Whereas most dug wells will draw water from local, shallow catchments with low potential to be influenced by seawater, drilled wells in these zones are at higher risk (Section 3.3.4 Other Issues).
- Mapping by NSDNR indicates that drilled wells in the Village Centre fall into a high-risk category for seawater intrusion (Kennedy, 2012). The remaining properties within the Village have been categorized as medium risk (Section 3.3.4 Other Issues).

Second, a central water system would help address about 16 of the Weaknesses, Opportunities, Threats, Key Sustainability Issues and Uncertainties, and Gaps to be Filled identified by the Integrated Community Sustainability Plan of the MoDC. Section 3.2 Non-monetized Benefits documents the 16 points.

Third, the Village is among the very few small communities in Nova Scotia whose potable water is not protected via a central system. This leaves the Village and the MoDC in general, with less to promote to attract investment.[1] Appendix 1: Nova Scotia Water Utilities provides a list of the utilities. Appendix 2: Nova Scotia Water Utilities' Current Quarterly Water Rates provides the rates for NS water utilities as published by the NS Utilities and Review Board.

Fourth, as concluded by the Nova Scotia Government, "Nova Scotia is particularly susceptible to climate change because most of our population lives along the coastline, and much of our infrastructure is located in vulnerable areas." It further concludes that for NS climate change means:

- warmer, wetter winters;
- hotter, drier summers;
- changes in nature of precipitation – more floods and more droughts;
- more-frequent extreme storms;
- rising sea levels; and
- accelerated coastal erosion[2]

A central water supply in the Village would help mitigate the risks to water supply that can stem from climate change.

[1] As of April 2018, there are 60 water utilities in Nova Scotia. The 61st will be in Pugwash. Construction is nearly complete. Thirty-four of the water utilities serve communities that are similar in size, or smaller than, the Village of Chester. (https://nsuarb.novascotia.ca/mandates/water)

[2] https://climatechange.novascotia.ca/facts-on-climate-change

1.2 Review of Existing Small NS Central Water Systems

We conducted interviews with representatives of nine water utilities in six small communities in Nova Scotia. The objectives were to identify the reasons for the construction of the central systems and the extent to which economic development considerations were part of the construction decision-making process.

None of the central systems examined had economic development as one of their principal objectives. The municipalities installed the systems to deal with contamination, supply shortages and/or aesthetic issues. Appendix 3: Experience with Central Water Systems at Selected NS Communities provides the findings from our conversations with utility and municipal staff. The table below provides a brief overview of the results of the conversations.

Review of Small NS Central Water Systems: Summary

Community	Reason for Central Water	Year Established	Source Water
Mahone Bay	Quality	1940-50s	Surface
Tatamagouche	Quality	1960s	Surface
Debert	(former) Military Base Requirements	1940s	Ground
Municipality of the County of Annapolis (Bridgetown, Cornwallis Park, Margaretsville)			
Bridgetown	Quality	2009	Surface water converted to Groundwater in 2009 due to need regulations and need for treatment
Cornwallis Park	(former) Military Base Requirements	1970s	Surface
Margaretsville	Quantity		Surface
Lawrencetown	Quality	2000s	Groundwater
Baddeck	Quantity & Quality	1950s	Groundwater; switched to surface water about 5 years ago due to quality issues
Pugwash	Quality	2018	Groundwater

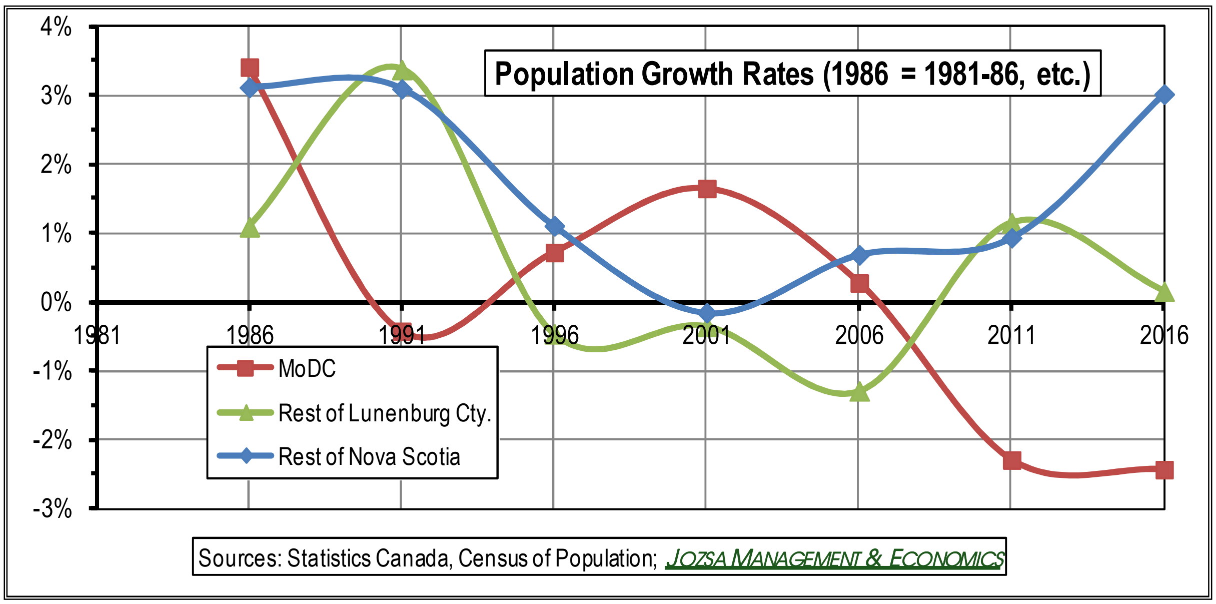
Source: *Jozsa Management & Economics* and CBCL

2 SOCIO-ECONOMIC BASELINE CONDITIONS

2.1 DEMOGRAPHY

In 2016, the MoDC's population was about 10,310 (about 0.8% higher than its 1981 population of 10,230) of which roughly 14.2% (1,460) reside in the Village of Chester.[3] The Village population is an estimate because Census dissemination areas in the MoDC are split between the Village and the rest of the MoDC.

As shown in the figure below the MoDC's population growth rate has fluctuated since 1981-86 but since 2001 it has slowed until it became negative just after 2006 and continued to decline until 2016. The weakening of the MoDC's population growth came at the same time the rest of Nova Scotia showed strengthening in its population growth rate.



Census dissemination areas in the MoDC straddle the boundary between the between the Village and the rest of the MoDC. We arrived at a rough estimate of the population in the Village by estimating the share of population on the Village side of the boundary. We do not have sufficient data to provide historical population estimates for the Village. Since the Village is the oldest built up area of the MoDC is possible that the Village's share of the MoDC's population has declined since 1981 because population growth may have tended to locate in the rest of the MoDC where more land that is residential was available and land costs are lower.

2.2 ECONOMY

Based on labour force survey information and on journey to work by place of work and place of residence information from the 2016 Census we estimate that in 2016 there were about 3,281 jobs in the MoDC, which accounted for about 15% of the jobs located in Lunenburg County. The MoDC's share of jobs located in Lunenburg County declined from about 17.2%

[3] The population of the rest of Lunenburg County is 38,816, about 3.7% higher than it was in 1981. The population of the rest of Nova Scotia is about 12.4% higher than it was in 1981.

in 2001 to about 13.2% in 2011 before recovering to 15.0% in 2016. In this study, we make the relatively conservative assumption that the MoDC will maintain its 15.0% share of jobs in Lunenburg County throughout the projection period.

MoDC residents hold about 72% of jobs located in the MoDC, rest by in-commuters.

About 48% of employed MoDC residents work in the MoDC and the remainder commute to work outside the MoDC. About 30% of employed residents work in the Halifax Regional Municipality and about 17% work in the rest of Lunenburg County. The remaining 5% work in other areas of Nova Scotia.

Data describing the place of work of residents of the Village and the place of residence of those working in the Village are not available.

2.3 Potential External Forces Affecting Demographic and Economic Growth in the MoDC

As noted above about 30% of the employed residents of the MoDC commute to work in the HRM.

We note that the Halifax Regional Municipality (HRM) has experienced consistent population growth due mainly to international migration (accounts for about 73% of its net migration since 2001) and migration from the rest of Nova Scotia (about 39% of its net migration since 2001). The population of the HRM has grown by 15.3% since 2001, just below the 17% growth for Canada overall.

The HRM has shown consistent employment growth since 2001 with only one year of negative growth (2004-05, -0.1%). Over the last five years, employment has grown by about 1,380 jobs per year.

As the twinning of the 100 Series highway between the HRM and Lunenburg County continues it is possible that the population decline in the MoDC will at least be stabilized or even return to positive growth if the improved accessibility to the HRM can be leveraged to attract a larger share of people who work at jobs in the HRM. The availability of a central water system in the Village, and the benefits it provides, could help to increase the population of commuters. This possibility is further supported by the fact that a substantial number of employed residents in the MoDC (30%) commute to work in the HRM.[4] That is, the MoDC is already a very attractive place to live for people working in the HRM.

2.4 Tax Assessment

2.4.1 Residential Assessed Value

The Village is home to about 24% of the MoDC's taxable residential assessment (about 916 taxable accounts). Two residential accounts are tax exempt.

[4] The very large size of this commuter flow becomes clear when one considers that only 4.7% of employed residents in the rest of Lunenburg County commuted to work in the HRM.

The average residential assessment in the Village (excluding tax exempt) is about $372,875 (median is about $207,000). The average for residential tax-exempt accounts is about $55,600.

2.4.2 COMMERCIAL ASSESSED VALUE

The Village is home to about 27% of the MoDC's taxable commercial assessment made up of about 88 accounts. Their average value is about $276,645 (median is about $150,000). There are 34 tax-exempt accounts whose average value is about $496,588.

3 IMPACTS OF A CENTRAL WATER SYSTEM FOR THE VILLAGE

Two options for central water have been investigated, groundwater and surface water. The 2018 CBCL Report "Village of Chester Water System Study" describes these systems in detail.

The well-based system, the status quo, serves as the benchmark against which we measured the benefits and costs of each of the central water alternative.

3.1 MONETIZED BENEFITS AND COSTS

The analysis of monetized benefits and costs:

- spans a 25-year design timeline, the same design timeline that CBCL used for the design of the central water systems; and
- focuses solely on the costs paid by, and benefits accrued by, the ratepayers of the Village.

The Village ratepayers receive all the monetized benefits associated with forgone private well capital and operating costs and benefits associated with the reduction of fire insurance costs.

The estimates of monetized costs to ratepayers are based on our suggested payment plan that spreads the costs of connecting to, and paying for, the central water system over 25 years. The payment plan makes the cost of using the central system less onerous to encourage ratepayers to connect to the new system.

All costs and benefits are measured in constant 2018$.

The ratepayers in the Village capture all monetized costs and benefits.

That is, the ratepayers in the Village pay for the:

- construction of the central system (including financing costs;
- costs to run connections from property lines to the associated buildings (including financing costs; and
- annual operating and maintenance costs of the central system.

3.1.1 MONETIZED BENEFITS

3.1.1.1 DIRECT BENEFITS

The direct monetary benefits of a central system are the costs foregone due to the abandonment of private wells and savings in fire insurance due to the provision of fire hydrants.[5]

A capital expenditure of $5,270,000 (2018$) (HST included) (Table 6.2 Class D Capital cost Options) would be required by Village ratepayers to:

- bring all wells in the Village up to current construction and water treatment standards; and
- mitigate against water shortages.

[5] TD Meloche Monnex (Halifax) conducted simulations to estimate the savings in fire insurance for properties in the Village that move from a semi-protected rating to a protected rating once fire hydrants are installed.

Therefore, we make the assumption that over the next five years (Table 6.1 Modifications to Existing Private Wells):

- all existing dug wells will be replaced and have their treatment systems upgraded;
- 21 drilled wells will be replaced and have their treatment systems upgraded; and
- 155 drilled wells will have their treatment systems upgraded.

Operating costs of the wells would be about $820,000 (2018$) (HST included) per year (Table 6.3 Annual Operation and Maintenance Costs).

Hence, in this benefit/cost analysis the foregone costs associated with wells and reduced fire insurance costs are benefits of the central water system.

All monetized benefits accrue to the ratepayers in the Village.

3.1.1.2 Temporal Distribution of Monetized Benefits

We assumed that:

- Foregone Well Reconstruction and Treatment Upgrades: CBCL estimates that $5,270,000 (2018$) in capital works would be needed to upgrade the well system to meet current standards and to avoid water shortages. We assume that, in the absence of a central water system, the Village ratepayers make these expenditures at various times during Years 1 to 5 and that they would finance the costs at 10% over five years.

  The total foregone cost, including financing, would be about $6,651,450 (2018$). They span Years 1 to 9.
- Foregone Well Operating & Maintenance: Benefits due to foregone well operating and maintenance costs begin in Year 1 and amount to about $820,000 (2018$) per year.

  These costs begin in Year 1.
- Reduction in Fire Insurance Cost: With the provision of a central water system and fire hydrants property owners in the Village will move from a semi-protected fire insurance rating to a protected rating. The average residential property owner will save about $332 per year and the average commercial property owner will save about $301 per year. Put another way, these savings are equivalent to a reduction in the 2018 residential tax rate of about 12.6% and about 7.1% in the commercial rate. We assume fire insurance savings for taxable and tax exempt properties because each class either buys property insurance or self-insures. The benefits from reduced fire insurance costs begin in Year 1.

  Annual benefits from reduced insurance costs amount to about $341,413. The 25 year total is $8,535,328 (2018$).

The following table summarizes the monetized benefits to ratepayers in the Village.

Monetized Benefits to Village Ratepayers(2018$)[1]				
	Foregone Well Reconstruction & Treatment Upgrading[2]	Foregone Well Operating & Maintenance	Reduction in Fire Insurance Cost	Total Benefits to Ratepayers
Year 1	$266,058	$820,000	$341,413	$1,380,919
Year 2	$532,116	$820,000	$341,413	$1,600,424
Year 3	$798,174	$820,000	$341,413	$1,819,930
Year 4	$1,064,232	$820,000	$341,413	$2,039,435
Year 5	$1,330,290	$820,000	$341,413	$2,258,941
Year 6	$1,064,232	$820,000	$341,413	$2,039,435
Year 7	$798,174	$820,000	$341,413	$1,819,930
Year 8	$532,116	$820,000	$341,413	$1,600,424
Year 9	$266,058	$820,000	$341,413	$1,380,919
Year 10 - 25	.	$820,000	$341,413	$1,161,413
Total Yr. 1 - 25	$6,651,450	$20,500,000	$8,535,328	$34,522,965
1.Benefits (foregone costs and fire insurance cost reduction) are the same for both central water systems.				
2.Includes financing at 10%, 5 year average amortization.				

3.1.2 MONETIZED COSTS

The immediate and largest monetary cost is of course the capital cost of constructing the system, estimated at between $44,522,250 (2018$) including HST for a surface water based system and $46,518,650 (2018$) including HST for a groundwater based system (Table 6.2 Class D Capital Cost Opinion).[6]

With cost sharing by senior levels of government, the capital cost to the MoDC could be reduced by 75%. The capital costs include the cost of providing distribution lines up to the property lines of taxable and tax-exempt properties.

3.1.2.1 TEMPORAL DISTRIBUTION OF MONETIZED COSTS

We assumed that:

- Connection Cost (includes financing): The central water system would be completed over a two to three year period. All 1,040 tax accounts in the Village will connect to the central system.

  CBCL estimates that over the next five years the well reconstruction and treatment upgrading of the existing well system would cost $5,270,000 (2018$) including HST. To make the immediate cost of connecting to the central system more attractive we suggest that connection costs to the 1,040 tax accounts be limited to $5,270,000, equivalent to what they would have paid to upgrade wells.

  We assume that ratepayers will finance the $5,270,000 at 10% over a 10-year amortization period.
- Delayed Capital Charge (includes financing): If the 1,040 tax accounts in the Village are charged only $5,270,000 for their connections to the system, the MoDC will be left with a significant amount of unpaid capital and financing costs.

[6] All estimates are D class, often called indicative, and are generally accurate to about +35% and -22.5%.

In the case of the surface water system we assume that the unpaid capital will be financed by the MoDC at 4% over 25 years for a total cost of about:

- o $9,378,653 (2018$) for the surface water system; and
- o $10,177,362 5 (2018$) for the groundwater based system.

We assume that this cost will be charged in 15 installments beginning in year 11. [7]

- Connection from Property Line to Buildings (includes financing: The average cost to connect from property lines to buildings will be about $2,000 plus HST (2018$). Each of the 1,040 tax accounts in the Village pays these costs.

  We assume that ratepayers will finance the cost of these connections at 10% over 10 years.
- Operating and Maintenance: Annual operations and maintenance costs are estimated by CBCL at about $186,000 for the surface water system and $68,000 for the groundwater based system (Table 6.3 Annual Operation and Maintenance Costs). All of these costs would be borne by the ratepayers in the Village. Annual operating and maintenance costs for the central systems would begin to be paid in Year 1 on a cost recovery basis. The base rate plus consumption charges are assumed to cover normal operating and maintenance costs.

The following tables summarize the costs of the surface water based central water system to ratepayers in the Village.

Monetized Costs to Village Ratepayers(2018$): Surface Water System

	Connection Cost (includes financing)[1]	Delayed Capital Charge (includes financing)[2]	Connection from Property Line to Buildings (includes financing)[1]	Operating & Maintenance[3]	Total Costs
Year 1	$835,721		$379,325	$186,000	$1,401,046
Year 2 - 10	$835,721		$379,325	$186,000	$1,401,046
Year 11 -25		$625,244		$186,000	$811,244
Total Yr. 1 - 25	$8,357,210	$9,378,653	$3,793,254	$4,650,000	$26,179,117

1: Financed by ratepayers at 10% rate, 10 year amortization
2: Financed by the MoDC at 4% rate, 25 year amortization but charged in 15 installments.
3: Covered by the base rate and consumption charges

[7] In North America there has been a tendency to under price water by setting prices to cover historic costs rather than future replacement costs. The result is the need to borrow to pay for replacement, which further results in higher future costs, which politically are generally hard to handle.

To avoid this situation in the MoDC we suggest that the Unrecovered Capital Charge be continued beyond year 25. However, it would be charged in 25 installments vs 15.

The following tables summarize the costs of the groundwater based central water system to ratepayers in the Village.

Monetized Costs to Village Ratepayers(2018$): Groundwater System					
	Connection Cost (includes financing)[1]	Delayed Capital Charge (includes financing)[2]	Connection from Property Line to Buildings (includes financing)[1]	Operating & Maintenance[3]	Total Costs
Year 1	$835,721		$379,325	$68,000	$1,283,046
Year 2 - 10	$835,721		$379,325	$68,000	$1,283,046
Year 11 -25		$678,491		$68,000	$746,491
Total Yr. 1 - 25	$8,357,210	$10,177,362	$3,793,254	$1,700,000	$24,027,826
1: Financed by ratepayers at 10% rate, 10 year amortization					
2: Financed by the MoDC at 4% rate, 25 year amortization but charged in 15 installments.					
3: Covered by the base rate and consumption charges					

3.1.3 RESULTS OF THE BENEFIT/COST ANALYSES

The benefit/cost analyses consider all monetized costs and benefits from the perspective of the ratepayers in the Village. All dollar values are stated in 2018 constant dollars.

The cost and benefit streams cover 25 years.

Benefits and costs that were estimated at a Class D level of accuracy (-22.5% to +35%) were subject to stochastic modelling. We used a PERT distribution to model the random normal distribution of accuracy of estimates between -22.5% and +35% when the most likely level of accuracy is +/-0%. The PERT distribution emphasises the “most likely” value (the mode) over the minimum and maximum values.

We assumed that the most likely accuracy was about +/-0% and that the least likely values were -22.5% and +35%. These assumptions result in a stochastic model based on a random normal distribution whose mode is 0% and skews so that values above the mode occur more frequency than those below the mode.

Appendix 5: Risk Analysis via Stochastic Modeling provides details about the application of stochastic modeling and how it was used in the study.

To measure present values (PV) we use a social time preference rate (i.e. the discount rate) of about 2.9% (the current rate of a 20-year Government of Canada bond)[8].

Appendix 6: Detailed Results of the Benefit/Cost Analyses provides the full range of findings stemming from the stochastic modelling of benefits and costs.

The net present value (NPV) is the difference between the PV of benefits of foregoing the well-based system and the PV of costs of constructing and operating the central water system.

[8] Social discount rates are normally based on the rate of long government bonds especially when the majority of expenditures are made by public sector entities. Since all costs and benefits are stated in 2018$ the rate of about 2.9% is equivalent to a discount rate of about 5% if costs and benefits were stated in current dollar terms.

3.1.3.1 SURFACE WATER CENTRAL SYSTEM

There is a 72% chance that the NPV will fall between $6.1 million and $10.0 million (2018$).

The median value of the range of NPVs is about $7.9 million (2018$).

There is a 76% chance that the benefit/cost ratio (BCR) will fall between 1.28 and 1.57.

The median value for the benefit/cost ratio BCR is about 1.39. This BCR reflects a 25-year real social rate of return of about 39%. (about 1.3% on an annual average basis).

3.1.3.2 GROUNDWATER CENTRAL SYSTEM

There is a 65% chance that the NPV will be between $7.8 million and $11.9 million (2018$).

The median value of the range of NPVs is about $9.4 million (2018$).

There is a 72% chance that the BCR will fall between 1.39 and 1.68.

The median value for the BCR is about 1.51. This BCR reflects a 25-year real social rate of return of about 51%. (about 1.7% on an annual average basis).

3.1.3.3 COMPARING THE BENEFIT/COST ANALYSES OF CENTRAL WATER SYSTEMS

The groundwater based and the surface based central water systems are more financially advantageous for the Village ratepayers then the well based system.

The groundwater based system is more financially advantageous for the Village ratepayers than the surface water based system because:

- the median NPV of the groundwater based system is about $1.6 million (20%) greater than the NPV of the surface water system;
- the median real rate of return over 25 years is about 51% for the groundwater based system versus about 39% for the surface water based system; and
- the groundwater based system offers less downside risk (minimum groundwater NPV is $2.9 million vs $2.1 million for the surface water based system).

3.1.3.4 COMPARING THE COSTS OF THE WELL BASED SYSTEM AND THE CENTRAL SYSTEMS

As the tables in Section 3.1.2.1 Temporal Distribution Of Monetized Costs show the 25 year undiscounted cost (including financing) of the:

- surface water based system is $26,179,117 (2018$); and
- groundwater based system is $24,027,826.

The central water systems allow for a full fire protection rating for insurance purposes. Over 25 years the savings in fire insurance amount to $8,535,328. The saving in insurance costs reduces the 25-year net undiscounted cost of the:

- surface water based system to $17,643,789 (2018$); and
- groundwater based system to $15,492,498 (2018$).

The table below shows that the 25 year undiscounted cost (including financing) of the well-based water system is $27,151,450 (2018$). The central water systems are less costly even before consideration of savings in fire insurance.

Costs to Upgrade and Operate the Well Based System (2018$)			
	Well Reconstruction & Treatment Upgrading[1]	Well Operating & Maintenance	Total Costs
Year 1	$266,058	$820,000	$1,086,058
Year 2	$532,116	$820,000	$1,352,116
Year 3	$798,174	$820,000	$1,618,174
Year 4	$1,064,232	$820,000	$1,884,232
Year 5	$1,330,290	$820,000	$2,150,290
Year 6	$1,064,232	$820,000	$1,884,232
Year 7	$798,174	$820,000	$1,618,174
Year 8	$532,116	$820,000	$1,352,116
Year 9	$266,058	$820,000	$1,086,058
Year 10 - 25		$820,000	$820,000
Total Yr. 1 - 25	$6,651,450	$20,500,000	$27,151,450
1.Includes financing at 10%, 5 year average amortization.			

3.2 NON-MONETIZED IMPACTS

The establishment of a central water system in the Village will also bring non-monetary impacts. The non-monetized impacts will accrue to, but not exclusively to, ratepayers in the Village. The non-monetized impacts are similar to public goods that are available for anyone to accrue, not just those that pay the monetized costs associated with non-monetized impacts.[9]

In general, public goods are supplied via collective effort, (e.g. via a voting process). However, because public goods can be accrued by anyone there is an incentive to not join in the collective payment and hence get a free ride in terms of non-monetized benefits. This situation again underscores the need to price connection and operating costs in a manner that in PV terms they are less than the cost to Village ratepayers of continuing the use of wells.

The central water system will produce non-monetized impacts by:

- mitigating the effects of climate change in NS that relate to:
  - hotter, drier summers;
  - changes in the nature of precipitation – more floods and more droughts; and
  - rising sea levels, which could cause saltwater intrusion into some wells located near ocean shorelines;
- solving water quantity and quality problems uncovered in the survey of well owners; and
- addressing the following issues noted in the MoDC's Integrated Community Sustainability Plan (ICSP):
  - Weaknesses;
    - water supply risk,

9 Water consumed by homes and businesses is properly treated as a private good and is readily monetized, as described in section 3.1.2 Monetized Costs.

    - small population limits the number and variety of services,
    - spending leakages to other areas,
    - small population limits the volunteer base, and
    - high cost of maintaining public land,
  - Opportunities;
    - exploitation of surface water supply,
  - Threats;
    - residents asking for more and better municipal services (e.g. potable water),
  - Key Sustainability Issues and Uncertainties;
    - declining school enrollment,
    - volunteerism suffers due to declining population,
    - service providers seeing rising expectations of customers in terms of quality and quantity,
    - few restaurants and local accommodations in the Village of Chester, and
    - higher sea levels due to climate change could possibly impact groundwater salinity, and
  - Gaps to be Filled;
    - water supply areas need to be identified and protected,
    - need for central services such as water and sewer,
    - making eco-friendly lifestyle affordable,
    - need more year round employment opportunities.

3.3 Population, Employment and Assessment Impacts

3.3.1 Impacts Associated with the Status Quo

3.3.1.1 Impacts on the Overall MoDC

If historical trends (1987 – 2016) continue for the next 20 years:

- Taxable commercial assessment could decline from $91.1 million (2018$) to about $85.4 million (2018$). At the 2017-18 general commercial tax rate of $1.53 per $100 of taxable assessment this assessment decline would result in a decrease of $87,210 in commercial tax revenue.
- The number of jobs located in the MoDC could decline from about 3,309 in 2016 to about 3,172.
- Taxable residential assessment could decline from about $1.443 billion (2018$) to about $1.383 billion (2018$). At the 2017-18 general residential tax rate of $0.705 per $100 of taxable assessment this assessment decline would result in a decrease of $423,000 in residential tax revenue.
- The population of the MoDC could decline from 10,310 in 2016 to about 9,881.

3.3.1.2 Impacts on the Village

Assuming the Village maintains its current shares of assessment and population:

- Taxable commercial assessment could decline from $24.3 million (2018$) to about $22.8 million (2018$).

- The number of jobs located in the Village will decline from about 884 in 2016 to about 848.
- Taxable residential assessment could decline from about $341.6 million (2018$) to about $327.4 million (2018$).
- The population of the Village will decline from about 1,460 in 2016 to about 1,369.

3.3.1.3 POTENTIAL IMPACTS FACILITATED BY THE PROVISION OF CENTRAL WATER IN THE VILLAGE

3.3.1.3.1 RESEARCH FINDINGS AND ASSUMPTIONS UPON WHICH PROJECTIONS ARE BASED

Hanemann[10] found that the experience in the United States was that non-monetized impacts such as those listed above could have significant positive economic effects. However, the research also showed that a secure supply of high quality water was necessary, but not sufficient, for economic growth.

The findings of peer-reviewed research deserve special attention. Decades of research in the USA have shown the following relationships between the provision of secure water supplies and economic growth:

- Investments in water supply do not automatically guarantee economic growth. However, areas that persist in lacking an adequate supply will not flourish economically.
- Secure water supply and quality is a necessary condition for economic growth but it not a sufficient condition. Hence, a secure water supply can sometimes show a causal link to economic growth.
- A secure water supply is not a major factor in macro-location (large geographical area) decisions but it does affect a micro-location decision, i.e. the decision of where to locate within a region. In the case of the Village of Chester, this suggests that a secure water supply would not necessarily attract investment that would not have otherwise come to the HRM and South Shore but it could increase the chance of attracting investment destined for the HRM and South Shore area to locate in the MoDC.

The availability of a safe and secure supply of potable water will likely have a modest impact on the demographic and economic sustainability of the Village and the overall MoDC. We base this conclusion on our examination of:

- past trends in population and job growth;
- MoDC building permits;
- planning intentions,
    - o to establish a commercial corridor by tying in commercial activity from North Street to the existing central commercial area of the Village via Pleasant Street, Valley Road and Duke Street; and
    - o to infill vacant land and underused land in the Village core;
- experience at other central water utilities in small NS communities; and
- research findings regarding the relationship of the provision of secure water supplies and economic growth reviews published in peer reviewed journals.

It is not possible to isolate the demographic and economic impacts that are due solely to the provision of a central water system in the Village because:

[10] "The Economic Conception of Water" W M Hanemann, University of California, Berkeley, USA. 2006

- based on experience elsewhere and the findings of peer reviewed research the effects of a central water system in the Village will be modest at best; and
- there are a large number of other forces introduced since 2016, internal to the MoDC and external, that should move the MoDC and the Village away from the 1987 – 2016 trends.

Our projections from 2018 onward assume that with the introduction of a central water supply in the Village:

- the MoDC will:
  - aggressively pursue is current economic development strategy;
  - aggressively pursue its planning intentions to establish a commercial corridor by tying in commercial activity from North Street to the existing central commercial area of the Village via Pleasant Street, Valley Road and Duke Street;
  - continue to move toward ‘as of right’ commercial development in the Village (as indicated in the Chester Village Area Revised Secondary Planning Strategy of 2004);
  - relax current planning policies limiting multi-unit residential building in the Village to four units;
  - relax current planning policies limiting the employment size[11] of light industrial establishments and lot coverage ratios (assuming no significant land use conflicts are created) to allow the Village’s light industrial base to take advantage of the larger and more secure supply of water;
  - allow higher density residential development in the Village (versus the current policy that limits residential density in order to protect the groundwater supply); and
  - market the new water system as part of its value proposition;
- improved water supply conditions in the Village will help it capture a greater share of job growth in the Lunenburg County area;
- improved water supply conditions and improving highway accessibility to the HRM will help the Village and the rest of the MoDC:
  - capture jobs that would otherwise have located in the HRM, particularly those that would otherwise locate in the western edges of the HRM; and
  - increase the number of residents who commute to jobs in the HRM.

3.3.1.3.2 IMPACTS ON THE OVERALL MODC

As noted above the provision of a central water system produce benefits that spread beyond the Village itself. The rest of the MoDC will:

- benefit economically if the Village is able to accommodate more commercial and residential development;
- accrue benefits stemming from the non-monetized impacts; and
- benefit from the implementation of economic development actions and municipal planning intentions that were introduced since 2016 and would be more actionable if the central water system is included in the mix.

[11] The 2004 Chester Village Area Secondary Planning Strategy states that new industrial uses in the Highway 3 Light Industrial Area are limited to a maximum of six employees. We assume that the limitation on the number of employees is being used as a proxy for the scale of an operation. However, technology is changing so rapidly that industrial operations require fewer and fewer employees. Hence, a modern light industrial establishment employing six people will operate at a much larger scale than a six employee establishment 20 years ago.

Based on the findings and assumptions described in Section 3.3.1.3.1 Research Findings and Assumptions Upon Which Projections Are Based we estimate that over the next 20 years:

- Taxable commercial assessment could rise from $91.1 million (2018$) to about $97.0 million (2018$), versus a decline to about $85.4 million in the status quo situation. At the 2017-18 general commercial tax rate of $1.53 per $100 of taxable assessment this assessment increase would result in an increase of $90,270 in commercial tax revenue.
- The number of jobs located in the MoDC will rise from about 3,309 in 2016 to about 3,601, versus a decline to about 3,172 in the case of the status quo.
- Taxable residential assessment could rise from about $1.443 billion (2018$) to about $1.57 billion (2018$), versus a decline to about $1.383 billion in the status quo situation. At the 2017-18 general residential tax rate of $0.705 per $100 of taxable assessment this assessment increase would result in an increase of $895,350 in residential tax revenue.
- The population of the MoDC will rise from 10,310 in 2016 to about 11,217, versus a decline to about 9,881 in the case of the status quo.

3.3.1.3.3 IMPACTS IN THE VILLAGE OF CHESTER

Impacts on Commercial Assessment and Jobs. The Village is currently home to about 27% of the taxable commercial assessment in the MoDC. As noted earlier CBCL's water supply assessment reports found water shortages and water quality issues in the Village likely limit additional development in the Village. Elimination of water supply and quality issues should allow commercial growth in the Village. The secure supply should facilitate the intention of current planning concepts to:

- direct growth into a commercial corridor from North Street to the existing central commercial area of the Village; and
- infill vacant space currently zoned for commercial type uses in the core of the Village.

Assuming these planning directions are vigorously perused the Village should be able to attract at least 27% of the incremental growth in commercial assessment to maintain its current share of the MoDC's taxable commercial assessment. However, the improved conditions regarding water supply, lower cost of water with a central system and lower fire insurance costs suggests that, on the high end, the Village could double its share of new commercial development.

Assuming the Village can increase its share of the growth in assessment to about 40% from its historical 27% share over the next 20 years:

- Taxable commercial assessment could increase from $24.3 million (2018$) to about $26.7 million (2018$), versus a decline to about $22.8 million in the status quo case.
- The number of jobs located in the Village will increase from about 884 in 2016 to about 1,001, versus a decline to about 848 in the case of the status quo.

Assuming the incremental commercial assessment is composed of an equal mix of retail, office space, restaurant and apartment construction the increase in commercial buildings will be about 8,138 square feet.[12] Given the land available for commercial development in the Village, the additional commercial space should be easily accommodated.

[12] Average cost per square foot of about $290 (2018$).

Impacts on Residential Assessment and Population. Given the high cost of land in the Village (relative to the rest of the MoDC) and the limited amount of vacant land for residential development in the Village it may be difficult for the Village to maintain its current share of taxable residential assessment and population. However, the development of multi-story rental accommodation as part of the growth in commercial assessment would help the Village to maintain its share of the overall MoDC population.

Assuming the Village can maintain its share of the MoDC taxable residential assessment over the next 20 years:

- Taxable residential assessment could rise from about $341.6 million (2018$) to about $371.6 million (2018$), versus a decline to about $327.4 million in the status quo situation.
- The population of the Village could rise from about 1,460 in 2016 to about 1,553, versus a decline to about 1,369 in the case of the status quo.

Home values in the Village are about 2.4 times greater than the average in the MoDC. Based on the assumption that construction costs would be about 2.4 times the average cost the incremental change in residential assessment in the Village would allow for the construction of about 37 residences (2,000 ft^2) in the Village.

3.3.2 Incremental Costs Due to Population and Business Growth

There will be incremental costs to provide municipal services to the increased population and increased commercial and residential assessments. However, because the increases are not large and are spread over 20 years the marginal cost of providing municipal services to the incremental growth should be less than the current average cost.

However, taxes on the incremental growth will be charged at full rates, not according to the incremental costs to the MoDC.

Therefore, other things being equal, the incremental growth in costs compared to the tax receipt growth should leave the MoDC's finances in a net positive position.

APPENDIX 1: NOVA SCOTIA WATER UTILITIES

Town of Amherst Water Utility

Annapolis County Water Utility

Town of Annapolis Royal Water Utility

Municipality of the County of Antigonish (Fringe Area, St. Andrews, Lr. South River)

Town of Antigonish Water Utility

The Village Commissioners of Baddeck Water Utility

Bridgetown Water Utility

Bridgewater Public Service Commission

The Village Commissioners of Canning Water Utility

Cape Breton Regional Municipality Water Utility

Debert Water Utility

Town of Digby Water Utility

East Hants Regional Water Utility

Falmouth Water Utility

Greenwood Water Utility

Halifax Regional Water Commission (including Eight Small Water Systems)

Hantsport Water Utility

Hazel Hill - Canso Water Utility

Inverness County Water Utility

Kentville Water Commission

The Village Commissioners of Lawrencetown Water Utility

Town of Lunenburg Water Utility

Town of Mahone Bay Water Utility

Town of Middleton Water Utility

Town of Mulgrave Water Utility

Town of New Glasgow Water Utility

The Village Commissioners of New Minas Water Utility

Town of Oxford Water Utility

Parrsboro Water Utility

Pictou County Water Utility

Town of Pictou Water Utility

Town of Port Hawkesbury Water Utility

Village of Port Williams Water Utility

Region of Queens Water Utility

Richmond County Water Utility

Town of Shelburne Water Utility

Sherbrooke Water Utility

Springhill Water Utility

Town of Stellarton Water Utility

Town of Stewiacke Water Utility

St. Peter's - Samsonville & Area Water Utility

The Village Commissioners of Tatamagouche Water Utility

Three Mile Plains/Wentworth Water Utility

Town of Trenton Water Utility

Town of Truro Water Utility

Victoria County Water Utility

Town of Westville Water Utility

Town of Windsor Water Utility

Town of Wolfville Water Utility

Town of Yarmouth Water Utility

The construction of a central water system for the Village of Pugwash is underway and a new utility will be established.

APPENDIX 2: NOVA SCOTIA WATER UTILITIES' CURRENT QUARTERLY WATER RATES

Purpose: To Calculate the quarterly billing for each water utility and to have a comparison of the charges for each water utility totalled under the Quarterly bill colum.

Document#: 226053 This document is updated after any water utility change to the rates found in the schedules attached to the order. This is a Quarterly metered table rate.

Gallons (standard) 13.50 Reference Ditation:# 177771

Calculation: 13.5* consumption Rate + 5/8 Domestic charge

Consumption Rate/1000 gallons con rate * 4.54

Comparison of NS Quarterly Water Rates					
Town/Utility	**(Quarterly Bills - Alphabetical)**	**5/8" Domestic**	**3" Industrial**	**Consumption Rate (per 1,000 gallons)**	**Quarterly Bill (5/8" meter & 13,500 gals.)**
AMHERST	July 1/17	29.34	430.83	3.15	71.87
	Apr. 1/18	30.74	451.91	3.24	74.48
	Apr. 1/19	31.11	458.32	3.47	77.96
ANNAPOLIS COUNTY (formerly Cornwallis Park / Granville Ferry & Margaretsville Utilities) - Amalgamated Oct/09	Apr. 1/17	79.59	1250.91	9.30	205.14
	Apr. 1/18	81.78	1285.21	9.94	215.97
ANNAPOLIS ROYAL	July 1/08	76.74	1122.06	3.77	127.64
	Apr.1/09	97.45	1450.18	3.91	150.24
	Apr. 1/10	97.95	1454.92	4.09	153.16
ANT. CO. (FRINGE)	July 1/15	52.28	809.19	2.43	85.09
	Apr. 1/16	56.64	877.19	2.88	95.52
	Apr. 1/17	56.64	877.19	3.16	99.30
ANT. CO. (ST. ANDREW'S AND LOWER SOUTH RIVER)	July 1/15	87.36	1376.57	4.24	144.60
	Apr. 1/16	98.10	1544.91	4.45	158.18
	Apr. 1/17	106.68	1678.64	4.89	172.70
ANTIGONISH (Town)	Apr. 1/17	48.36	427.43	2.64	57.60
	Apr. 1/18	50.65	449.73	2.78	88.18
	Apr. 1/19	53.05	472.44	2.88	91.93
BADDECK	Apr. 1/12	36.44	542.62	3.41	82.47
	Apr. 1/13	46.79	707.42	3.50	94.04
BRIDGETOWN	July 1/15	66.93	986.16	7.63	169.90
	Apr. 1/16	66.93	986.16	9.03	188.90
	Apr. 1/17	66.93	990.53	9.58	196.25
BRIDGEWATER	July 1/16	62.07	923.33	5.49	136.19
	Apr. 1/17	66.38	990.24	5.91	146.17
	Apr. 1/18	68.13	1016.37	6.14	151.02
CANNING	July 1/11	45.60	673.54	4.64	108.24
	Apr. 1/12	49.17	729.77	5.15	118.69
	Apr. 1/13	52.05	774.70	5.52	126.57
CANSO-HAZEL HILL	Apr. 1/14	82.88	1267.97	10.10	219.23
	Apr. 1/15	84.26	2010.94	10.46	225.47
CAPE BRETON REGIONAL	July 1/17	59.11	909.92	6.54	147.40
	Apr. 1/18	61.46	946.67	6.67	151.56
	Apr. 1/19	63.71	981.61	6.90	156.87
DEBERT (Billed bi-monthly)	July 1/11	81.62	1198.58	8.13	191.37
	Apr. 1/12	87.50	1291.23	8.83	206.70
	Apr. 1/13	90.78	1342.18	9.20	214.98
DIGBY	Jul1 1/13	56.78	845.01	2.02	84.04
	Apr. 1/14	59.04	878.64	2.17	88.33
	Apr. 1/15	61.10	909.69	2.30	78.92
EAST HANTS	July 1/17	44.41	666.53	11.00	192.91
	Apr. 1/18	45.91	690.06	12.00	207.91
	Apr. 1/19	47.00	707.18	12.34	213.59
	July 1/14	43.54	674.55	5.44	116.98

FALMOUTH	Apr. 1/15	46.29	634.90	5.70	123.24
	Apr. 1/16	36.29	558.12	6.31	121.48
GREENWOOD	July 1/14	62.14	976.85	8.56	177.70
	Apr. 1/15	65.01	1022.42	9.76	196.77
	Apr. 1/16	67.95	1069.10	10.36	207.81
HALIFAX REGIONAL	May 1/15	39.00	510.00	3.83	90.79
	Apr 1/16	39.00	510.00	4.43	98.80
HANTSPORT	July 1/15	78.51	1184.34	8.17	188.81
	Apr. 1/16	84.84	1283.56	8.99	206.21
	Apr. 1/17	91.12	1381.83	9.85	224.10
INVERNESS CO.	Apr. 1/07	22.16	342.01	3.36	67.51
	Apr. 1/08	27.87	433.01	3.95	81.19
KENTVILLE	May 1/14	32.57	497.49	2.94	72.26
	Apr. 1/15	32.39	494.12	3.17	75.18
	Apr. 1/16	32.67	497.91	3.29	77.09
LAWRENCETOWN	July 1/11	56.95	861.34	3.59	105.41
	Apr. 1/12	61.57	934.00	4.23	118.67
	Apr. 1/13	62.76	951.81	4.41	120.09
LUNENBURG	July 1/16	71.47	1055.63	4.52	132.49
	Apr. 1/17	81.45	1206.95	4.71	145.04
	Apr. 1/18	86.62	1286.91	5.00	154.12
MAHONE BAY (Billed bi-monthly)	July 1/14	77.88	1161.00	11.79	237.05
	Apr. 1/15	88.62	1331.46	12.31	254.81
	Apr. 1/16	101.19	1532.10	12.91	275.48
MIDDLETON	July 1/15	73.99	1087.42	3.69	123.81
	Apr. 1/16	76.98	1132.32	3.90	129.69
	Apr. 1/17	79.55	1170.52	4.18	135.94
MILL COVE PARK	March 1/09	58.26	849.93	7.22	155.73
	Oct. 1/09	70.45	1042.52	8.52	185.47
	Apr. 1/10	78.06	1161.65	8.97	199.16
MULGRAVE	Oct. 1/16	106.20	1614.14	8.37	219.20
	Oct. 1/17	118.33	1806.61	9.69	249.15
	Oct. 1/18	136.98	1053.75	10.88	283.86
NEW GLASGOW	Aug.1/15	45.12	684.06	5.35	117.35
	Apr. 1/16	54.02	825.74	6.06	135.83
	Apr. 1/17	59.23	908.00	6.64	148.87
NEW MINAS	April 1/08	24.07	272.23	2.69	60.38
	April 1/09	25.40	302.00	2.91	64.68
OXFORD	Oct. 1/15	62.51	934.29	2.17	69.35
	Apr. 1/16	64.39	962.51	2.20	71.32
	Apr. 1/17	67.47	1009.62	2.28	74.65
PARRSBORO (Cumberland Co.)	Apr. 1/17	36.83	N/A	N/A	-------
	Apr. 1/18	39.64	N/A	N/A	
PICTOU, Town	Sept. 1/13	45.00	600.00	3.41	91.04
	Apr. 1/14	45.00	600.00	3.49	92.12
	Apr.1/15	45.00	600.00	3.51	92.39
	Apr. 1/16	45.00	600.00	3.49	92.12
PICTOU County	July 1/17	47.41	733.49	4.83	112.62
	Apr. 1/18	56.64	876.81	5.04	124.68
	Apr. 1/19	65.88	1020.66	5.22	136.35
PORT HAWKESBURY	Oct. 1/14	54.40	833.73	6.02	135.67
	Apr. 1/15	60.13	926.05	6.41	146.67
	Apr. 1/16	67.75	1046.86	6.85	160.23
	Apr. 1/16	58.48	891.76	4.18	114.91

PORT WILLIAMS	Apr. 1/17	59.20	902.07	4.27	116.81
	Apr.1 /18	60.61	923.56	4.27	118.26
PUGWASH (Cumberland Co.)	Apr. 1/18	59.12	894.61	4.89	125.19
	Apr. 1/19	63.24	962.40	5.20	133.48
QUEENS	July 1/02	34.62	517.54	1.91	60.41
	Apr. 1/03	38.90	584.70	2.32	70.22
	Apr. 1/04	39.23	588.69	4.36	98.09
RICHMOND COUNTY	Apr. 1/17	34.83	533.50	6.95	128.66
	Apr. 1/18	38.31	588.40	7.58	140.64
SHELBURNE	Apr.1/14	86.15	1312.25	8.40	199.55
	Apr.1/15	100.46	1536.60	8.63	216.95
SHERBROOKE (ST. MARY'S)	Oct. 1/17	78.38	1200.74	9.31	204.07
	Apr. 1/18	83.09	1274.97	9.58	212.42
	Apr. 1/19	88.93	1367.18	9.76	220.69
SPRINGHILL (Cumberland Co.)	July 1/15	63.86	972.43	5.40	136.76
	Apr. 1/16	66.67	1015.90	5.48	140.65
	Apr. 1/17	69.15	1054.01	5.60	144.75
ST. PETER'S/SAMSONVILLE & AREA	Oct. 1/17	97.47	1519.69	5.95	177.76
	Apr. 1/18	97.67	1521.70	6.08	179.80
	Apr. 1/19	97.96	1525.12	6.31	183.15
STELLARTON	May 1/06	28.29	441.36	2.92	67.71
	Apr. 1/07	44.39	698.57	4.52	105.41
	Apr. 1/08	44.96	707.38	4.62	107.33
STEWIACKE	July1/13	40.33	621.65	8.75	158.45
	April 1/14	43.56	672.75	9.24	168.30
	April 1/15	48.98	758.86	9.88	182.36
TATAMAGOUCHE	Jan. 1/18	85.69	1285.14	11.40	239.53
	Apr. 1/18	95.30	1437.09	11.85	255.27
	Apr. 1/19	104.79	1586.30	12.35	271.50
THREE MILE PLAINS/WENTWORTH	Apr. 1/17	36.41	564.60	10.76	181.67
	Apr. 1/18	42.65	663.57	11.35	195.88
TRENTON	July 1/09	20.33	313.40	4.51	81.22
	Apr. 1/10	22.30	344.46	4.77	86.70
	Apr. 1/11	24.42	378.07	5.01	92.06
TRURO	May 1/13	55.66	865.71	4.22	112.63
	April 1/14	59.51	926.80	4.36	118.37
	April 1/15	63.44	988.94	4.45	123.52
VICTORIA COUNTY (former Dingwall, Neil's Hbr/New Haven, Little Narrows and Ingonish) - Amalgamated 2009	July 1/17	109.92	1695.39	14.62	307.27
	Apr. 1/18	109.87	1691.97	15.75	322.50
	Apr. 1/19	111.00	5325.90	17.07	341.45
WESTVILLE	Feb. 1/11	48.33	734.89	5.68	125.01
	Apr. 1/11	48.26	732.70	6.23	132.37
	Apr. 1/12	48.71	738.99	6.68	138.89
WINDSOR	Oct. 1/15	64.26	955.86	5.96	144.72
	Apr. 1/16	68.29	1016.45	6.45	155.37
	Apr. 1/17	69.75	1037.36	6.69	160.07
WOLFVILLE	Jan. 1/02	32.52	474.20	3.08	74.10
	Jan. 1/03	33.36	486.54	3.18	76.29
	Jan. 1/04	33.92	494.33	3.28	78.20
YARMOUTH	Nov. 1/16	50.53	756.98	5.97	131.13
	Apr. 1/17	53.71	806.27	6.16	136.87
	Apr. 1/18	54.87	823.22	6.27	139.52

APPENDIX 3: EXPERIENCE WITH CENTRAL WATER SYSTEMS AT SELECTED SMALL NS COMMUNITIES

Mahone Bay

The central water system was installed in the late 1940s to early 1950s in order to improve the quality of water. It is a surface water based system. It was installed to improve water quality. There are 71 commercial connections and 409 residential connections. The system serves a population of about 1,000 and has sufficient capacity to serve a population of about 1,500.There have not been any expansions to the system.

New connections must pay for connecting from the main service line to the property line and then from the property line to the building in question.

There are still some private wells being used.

Appendix 4: Summary Financial Indicators and Quarterly Rates for Selected Water Utilities provides a summary of financial indicators for the Mahone Bay Water Utility. Its base rates plus consumption charges were sufficient to yield annual positive net incomes for two of the last four years. This situation has resulted in an inability to consistently move its retained equity out of its negative position.

Tatamagouche

The central water system was established in the 1960s because water quality from private wells was poor and private wells near the waterfront had some issues with salt intrusion.

Some homes continue to use private wells.

The secure supply of high quality water was to some extent responsible for Tatamagouche Brewing Company setting up in the village.

Appendix 4: Summary Financial Indicators and Quarterly Rates for Selected Water Utilities provides the current water rates for this utility. Summary financial indicators were not available for the Tatamagouche Water Utility.

Debert

The central water system was installed to meet the demand of the former military base. The base became the property of the Province of Nova Scotia and it was promoted as an industrial park. In 2008 ownership of the Debert Air Industrial Park was transferred to the Municipality of the County of Colchester. The park area is about 45% occupied. Its larger tenants include:

- manufacturing and distribution operations for Peter Kohler Windows;
- Home Hardware distribution;
- a Sobeys distribution centre;
- Newmac Manufacturing;
- Specialty Steel; and
- Tim Horton's distribution centre.

The Debert Industrial Airport has three runways ranging in length from 3,450 to 5,000 meters.

Appendix 4: Summary Financial Indicators and Quarterly Rates for Selected Water Utilities provides a summary of financial indicators for the Debert Water Utility. Its base rates plus consumption charges were sufficient to yield annual positive net incomes for two of the last four years.

This situation has resulted in an inability to consistently move its retained equity out of its negative position.

MUNICIPALITY OF THE COUNTY OF ANNAPOLIS

The Municipality of the County of Annapolis is responsible for central water systems located in Margaretsville, Granville Ferry, Cornwallis Park and Bridgetown.

The system in Bridgetown was made necessary due to uranium contamination issues.

The system in Cornwallis Park was installed to meet the demands of the former CFB Cornwallis.

The construction of a central system in Margaretsville was made necessary due to recurring water shortages.

The water systems by policy do not extend beyond community boundaries.

County Council did at one time consider installing central water systems as aids to economic development. In one case a public-private-partnership was considered as a financing tool for a system. However, the option was turned down because it was too costly. To date no systems have been constructed with the primary objective of aiding economic development.

Appendix 4: Summary Financial Indicators and Quarterly Rates for Selected Water Utilities provides a summary of financial indicators for the combined Margaretville, Granville Ferry and Cornwallis Park water utilities and the Bridgetown Water Utility.

The base rates plus consumption charges of the combined Margaretville, Granville Ferry and Cornwallis Park water utilities resulted in positive net income for only one of the past four years (2012/13 – 2015/16). As a result, the utility's retained equity dipped further into the negative from 2012/13 to 2015/16.

The base rates plus consumption charges of the Bridgetown Water Utility resulted in positive net income for two of the past four years (2010/11 – 2013/14). As a result the utility's retained equity became less negative during the latest two years.

LAWRENCETOWN

The system in Lawrencetown was made necessary due to uranium contamination issues.

Appendix 4: Summary Financial Indicators and Quarterly Rates for Selected Water Utilities provides a summary of financial indicators for the Lawrencetown Water Utility.

The base rates plus consumption charges of the Lawrencetown Water Utility resulted in positive net income for the past four years (2012/13 – 2015/16). As a result the utility's retained equity moved from a negative position in 2012/13 to a positive position in 2015/16.

BADDECK

The central water system was installed in the 1950s. The central system was installed for the following reasons:

- There is substantial gypsum content in the ground and this was affecting the quality of water in many of the private wells.
- Some private wells would run dry over the summer.

- As the village grew there became less room for private dug wells.

The village has fire protection services via hydrants and a fire department.

There are about 550 connections of which 75 are commercial.

About five years ago the system switched sources from a small river to two drilled wells in the Highlands. This was done because the water quality of the river fluctuated by season.

The availability of a secure supply of high quality water is important to the economy of the village because the hotels and resorts require clean and secure supplies to operate effectively.

The central water system, which includes hydrants, has contributed to increases in property values.

Like Chester, Baddeck has a large number of summer residents. The utility will remove water meters of summer residents in the Fall and shut off their service. Customers will re-establish service when they return in the summer.

Appendix 4: Summary Financial Indicators and Quarterly Rates for Selected Water Utilities provides a summary of financial indicators for the Baddeck Water Utility. Its base rates plus consumption charges have been sufficient to yield annual positive net incomes for the last four years. This situation has enabled the utility to consistently increase its retained equity

PUGWASH

The central water system, a groundwater based system, in Pugwash is currently at its commissioning stage. It will begin with about 340 residential connections and 30 commercial connections. Hydrants for fire protection are not part of the current development plan. The system's capacity was designed to accommodate a 2% annual growth rate in population over the next 20 years

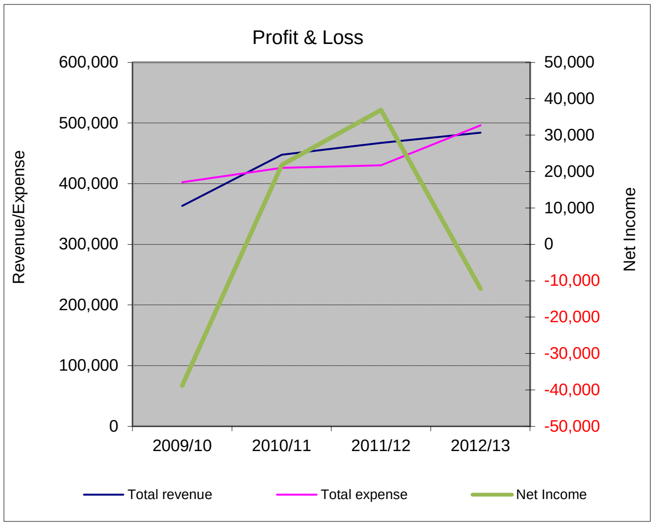
Connection to the system will be voluntary. However, ratepayers who choose not to connect to the system will be required to pay a local improvement charge. The decision to establish a local improvement charge was made despite the fact that during public consultation such a charge was the most mentioned negative aspect of the plan for the central water system.

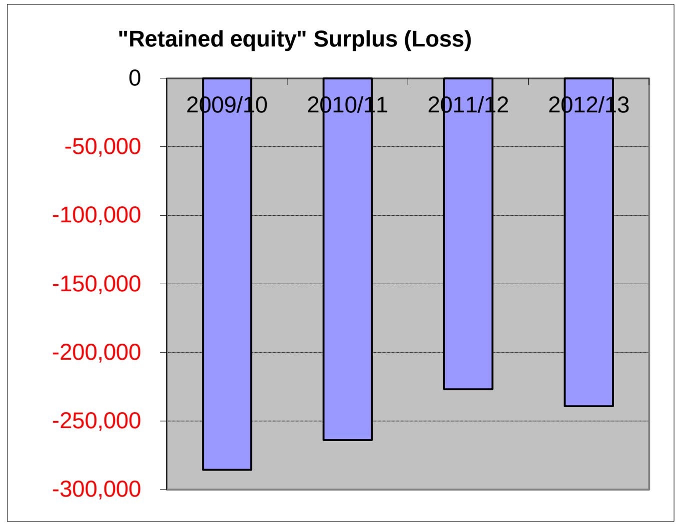
The decision to build a central water system was driven by health related concerns and to a lesser extent by aesthetic issues with the water supply. In addition, there were occasional, but not widespread, water shortages among private wells.

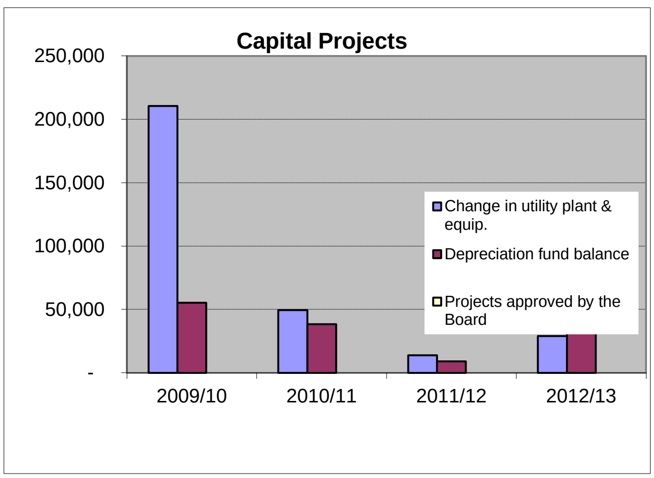
The need for a central water system also addressed by the area's Integrated Community Sustainability Plan (ICSP), which stated that a central water system was needed to help Pugwash to 'come into its own'. The lack of a central water system was the most mentioned weakness during the public during consultations for the creation of the ICSP.

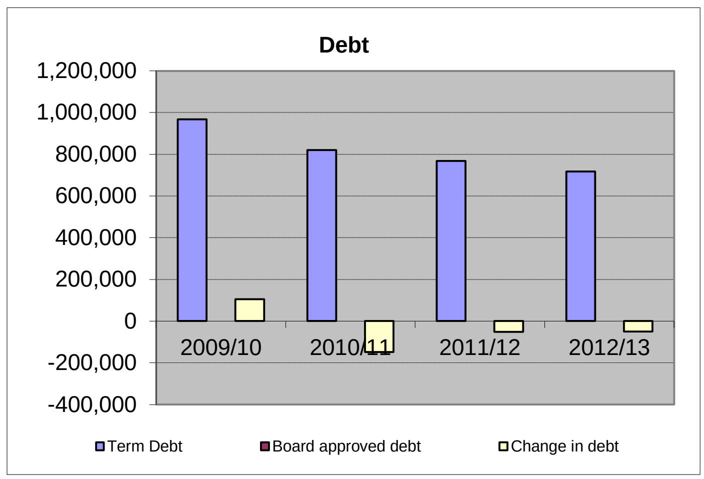
APPENDIX 4: SUMMARY FINANCIAL INDICATORS AND QUARTERLY RATES FOR SELECTED WATER UTILITIES

Town of Mahone Bay Water Utility 2013









Return on Rate Base	2009/10	2010/11	2011/12	2012/13
	0.92	1.97	2.84	0.87

Date of last rate increase	5/8" Domestic	3/4"	1"	1-1/2"	2"	3"	4"	6"	Consumption rate(per 1000 gal)	Bi-monthly bill (5/8" & 9000 gal) 13.5
01-Apr-11	56.18	82.65	135.59	267.93	426.74	850.25	1326.69	2,650.13	9.47	141.41

Document Number: 221234

ORDER **M05893**

NOVA SCOTIA UTILITY AND REVIEW BOARD

IN THE MATTER OF THE PUBLIC UTILITIES ACT

\- and -

IN THE MATTER OF AN APPLICATION of the of the **Municipality of the County of Colchester** on behalf of the **Tatamagouche Water Utility** for Approval of Amendments to its Schedule of Rates for Water and Water Services and its Schedule of Rules and Regulations

BEFORE: Murray E. Doehler, CA, P.Eng., Member

ORDER

WHEREAS the Municipality of the County of Colchester on behalf of the Tatamagouche Water Utility ("Utility"), made application to the Nova Scotia Utility and Review Board ("Board") for approval of amendments to its Schedule of Rates for Water and Water Services and its Schedule of Rules and Regulations;

AND WHEREAS after due public notice, a hearing was held on the 19th day of February, 2014, and the Board was pleased to issue its Decision on the 1st day of May, 2014;

IT IS HEREBY ORDERED that the Schedule of Rates and Charges, attached hereto as Schedules "A", "B" and "C", be approved, for water and water services supplied on and after July 1, 2014; April 1, 2015, and April 1, 2016 respectively;

AND IT IS FURTHER ORDERED that the Schedule of Rules and Regulations, attached hereto as Schedule "D", be approved effective July 1, 2014.

DATED at Halifax, Nova Scotia, this 1st day of May, 2014.

Clerk of the Board

SCHEDULE "A"

MUNICIPALITY OF THE COUNTY OF COLCHESTER
TATAMAGOUCHE WATER UTILITY
SCHEDULE OF RATES FOR WATER AND WATER SERVICES

(Effective for water supplied on and after 1 July, 2014)

RATES

The rates set out below are the rates approved by the Board for water and water services when payment is made within 30 days from the date rendered as shown on the bill.

When payment is made after 30 days from the date rendered as shown on the bill, the rates will include interest charges of 1.25 % per month, or part thereof.

Each bill shall show the amount payable within 30 days from the date rendered as shown on the bill.

In this Schedule, the word "Utility" means the Tatamagouche Water Utility of the Municipality of the County of Colchester.

1. RATES:

(a) **Base Charges** **Quarterly**

Size of Meter	
5/8"	$ 65.40
3/4"	95.54
1"	155.82
1.5"	306.54
2"	487.40
3"	969.68
4"	1,512.25
6"	3,019.40
8"	5,430.83

(b) Consumption Rate

$9.62 per 1,000 imp. Gallons
$2.12 per cubic meter

(c) Minimum Bill

The minimum bill shall be the Base Charge.

SCHEDULE "B"

MUNICIPALITY OF THE COUNTY OF COLCHESTER
TATAMAGOUCHE WATER UTILITY
SCHEDULE OF RATES FOR WATER AND WATER SERVICES

(Effective for water supplied on and after 1 April, 2015)

RATES

The rates set out below are the rates approved by the Board for water and water services when payment is made within 30 days from the date rendered as shown on the bill.

When payment is made after 30 days from the date rendered as shown on the bill, the rates will include interest charges of 1.25 % per month, or part thereof.

Each bill shall show the amount payable within 30 days from the date rendered as shown on the bill.

In this Schedule, the word "Utility" means the Tatamagouche Water Utility of the Municipality of the County of Colchester.

1. RATES:

(a) **Base Charges** **Quarterly**

Size of Meter	
5/8"	$ 73.95
3/4"	108.30
1"	177.00
1.5"	348.76
2"	554.87
3"	1,104.50
4"	1,722.84
6"	3,440.43
8"	6,188.58

(b) Consumption Rate

$10.00 per 1,000 imp. Gallons
$2.20 per cubic meter

(c) Minimum Bill

The minimum bill shall be the Base Charge.

SCHEDULE "C"

MUNICIPALITY OF THE COUNTY OF COLCHESTER
TATAMAGOUCHE WATER UTILITY
SCHEDULE OF RATES FOR WATER AND WATER SERVICES

(Effective for water supplied on and after 1 April, 2016)

RATES

The rates set out below are the rates approved by the Board for water and water services when payment is made within 30 days from the date rendered as shown on the bill.

When payment is made after 30 days from the date rendered as shown on the bill, the rates will include interest charges of 1.25 % per month, or part thereof.

Each bill shall show the amount payable within 30 days from the date rendered as shown on the bill.

In this Schedule, the word "Utility" means the Tatamagouche Water Utility of the Municipality of the County of Colchester.

1. RATES:

(a) Base Charges

Size of Meter	Quarterly
5/8"	$ 81.49
3/4"	119.55
1"	195.67
1.5"	385.95
2"	614.30
3"	1,223.21
4"	1,908.24
6"	3,811.11
8"	6,855.69

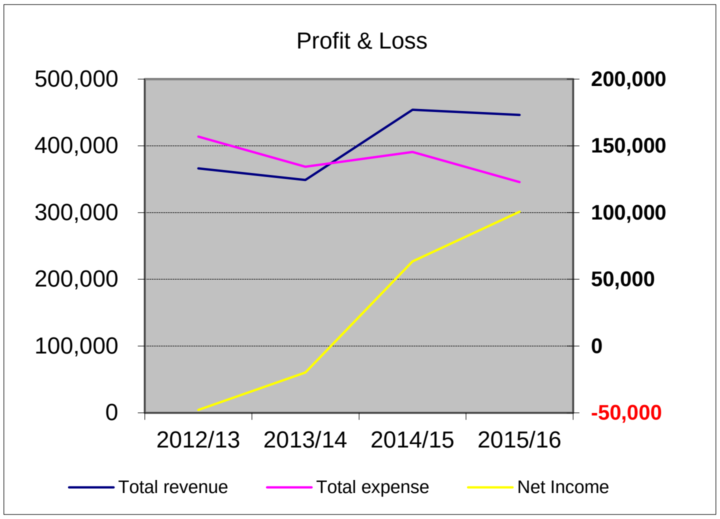
(b) Consumption Rate

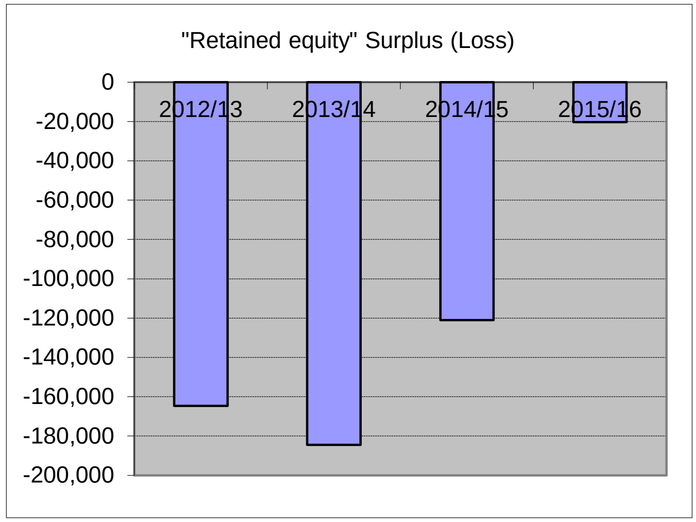
$10.46 per 1,000 imp. Gallons
$2.30 per cubic meter

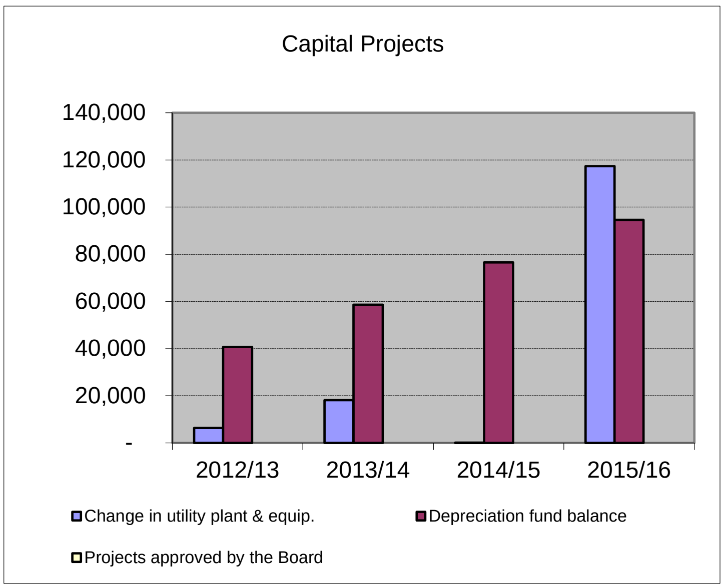
(c) Minimum Bill

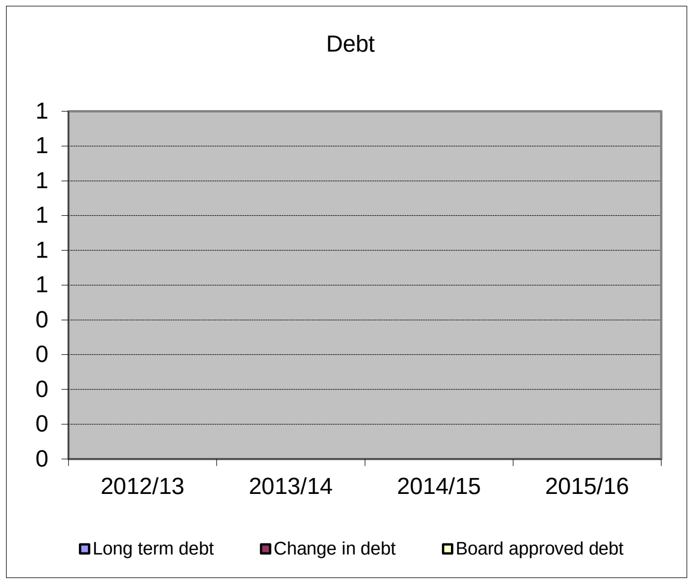
The minimum bill shall be the Base Charge.

Debert Water Utility 2016







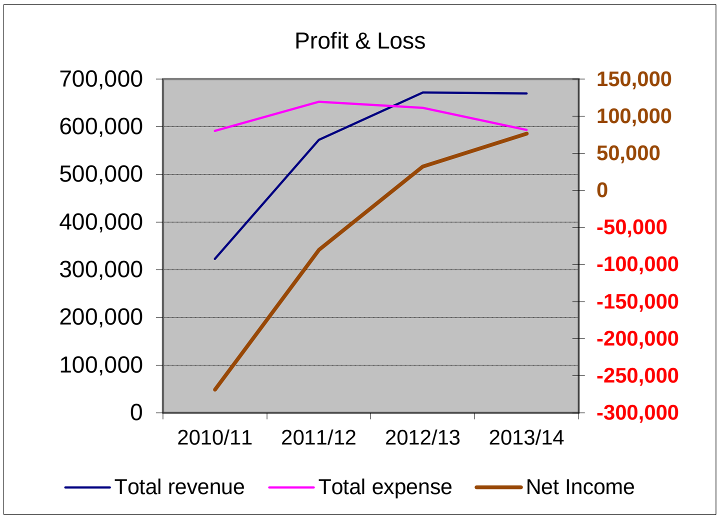


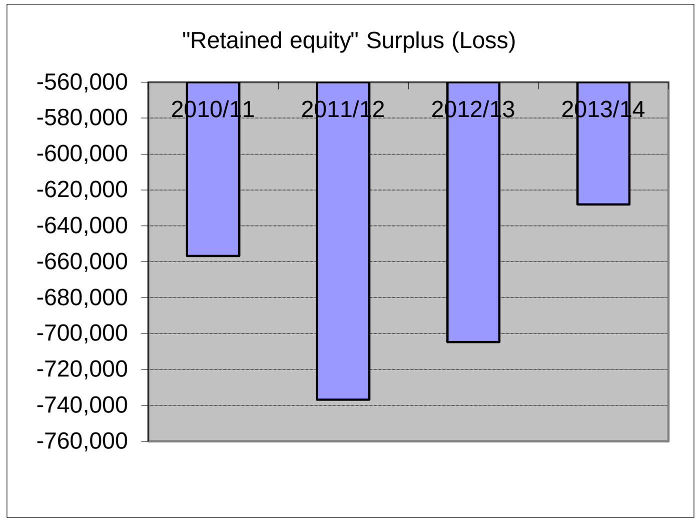
Return on Rate Base	2012/13	2013/14	2014/15	2015/16
	-5.88	0.91	-1.02	4.69

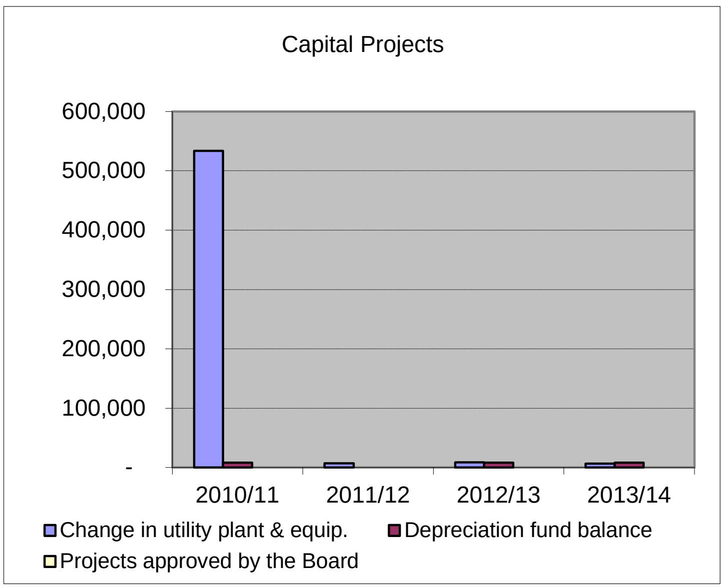
Date of last rate increase	5/8" Domestic	3/4"	1"	1-1/2"	2"	3"	4"	6"	Consumption rate(per 1000 gal)	Quarterly bill (5/8" & 13500 gal) 13.5
1-Apr-13	90.78	132.49	215.92	424.49	674.77	1342.18	2093.09	4,178.68	9.20	214.98

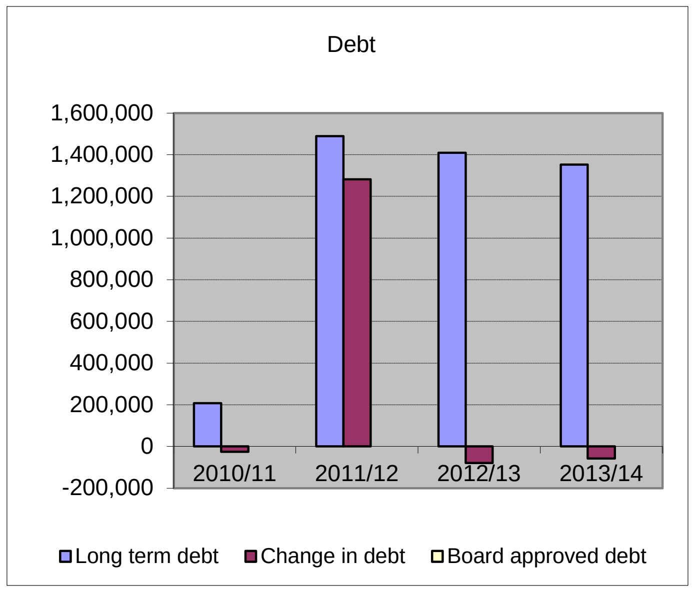
Document Number: 219965

Bridgetown Water Utility 2014







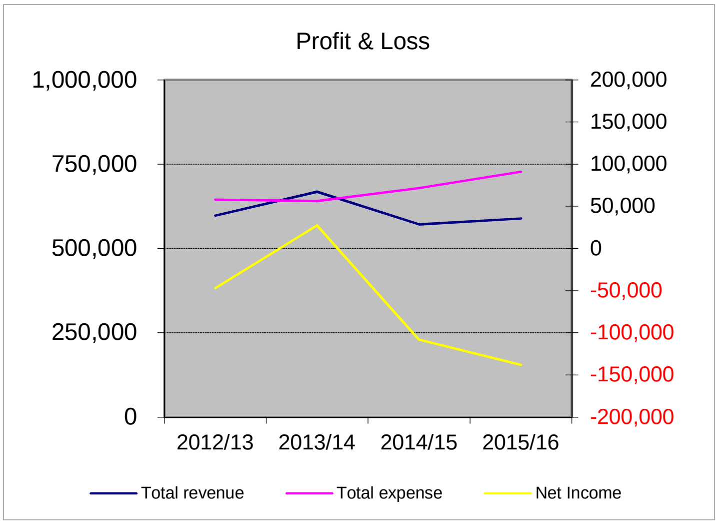


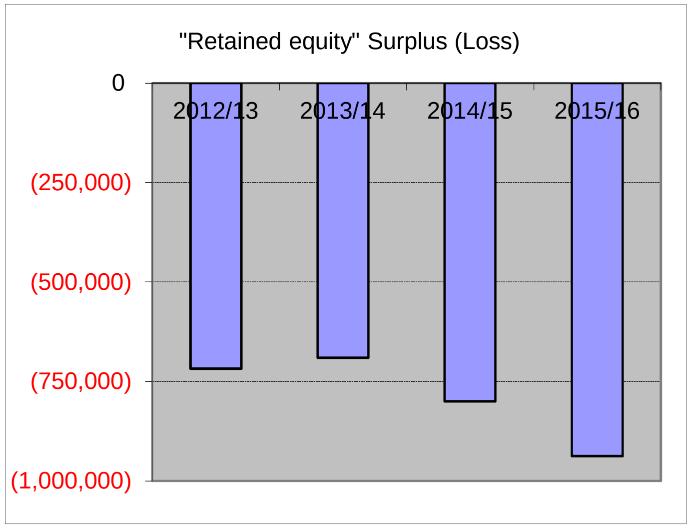
Return on Rate Base	2010/11	2011/12	2012/13	2013/14
	-6.45	-2.64	0.97	2.07

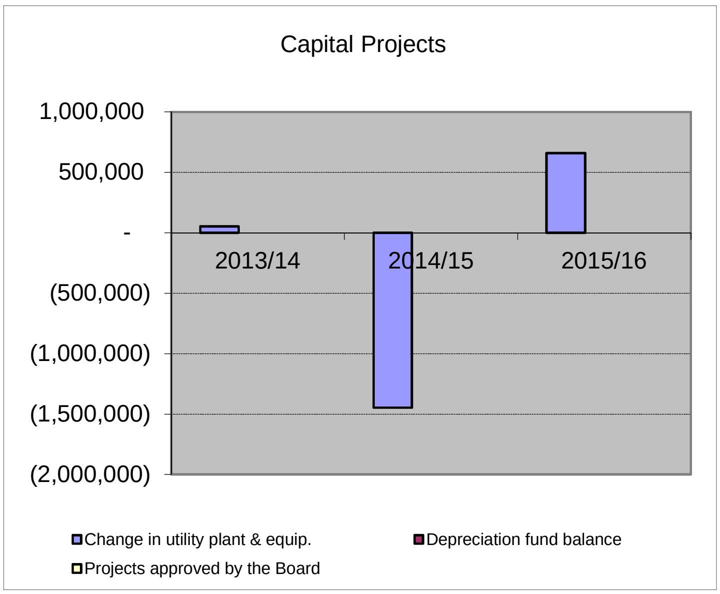
Date of last rate increase	5/8" Domestic	3/4"	1"	1-1/2"	2"	3"	4"	6"	Consumption rate(per 1000 gal)	Quarterly bill (5/8" & 13500 gal)
										13.5
April 1/13	66.93	97.57	158.85	312.05	495.90	986.15	1537.69	3,069.73	6.72	157.64

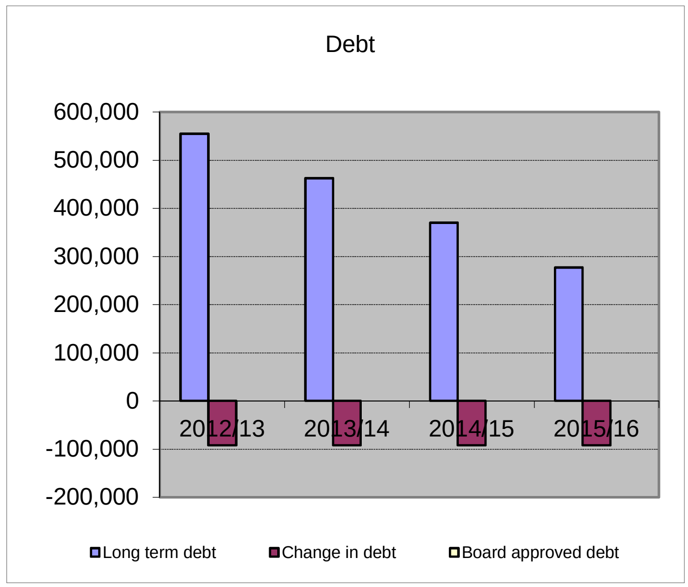
Document Number: 228166

Annapolis County Water Utility 2016







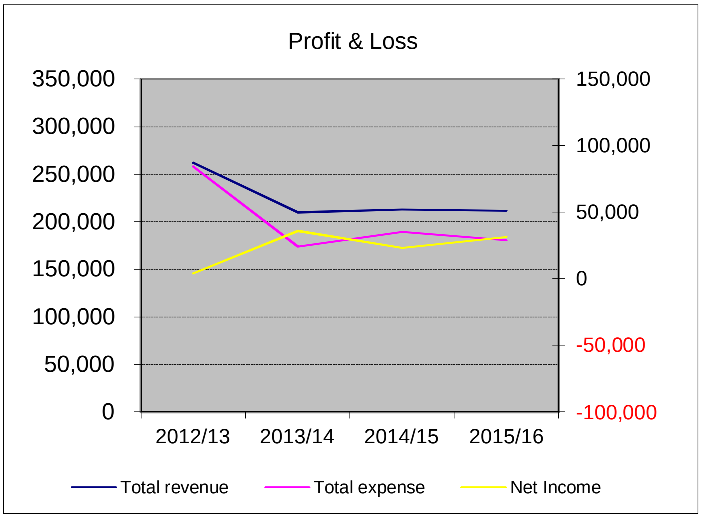


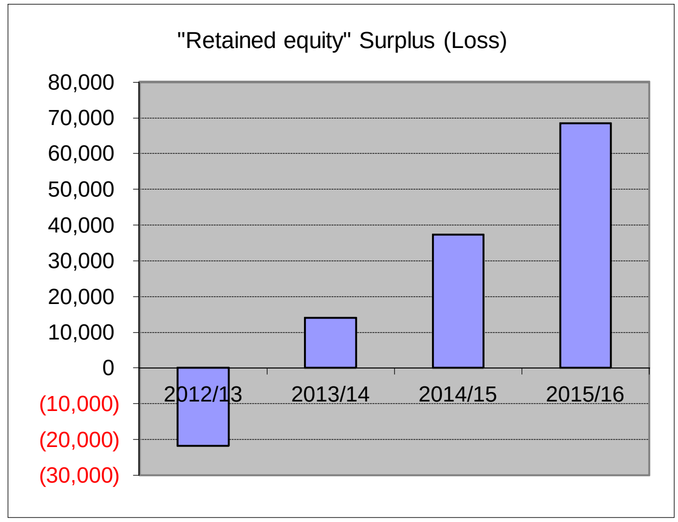
Return on Rate Base	2012/13	2013/14	2014/15	2015/16
	0.84	1.62	0.25	-0.10

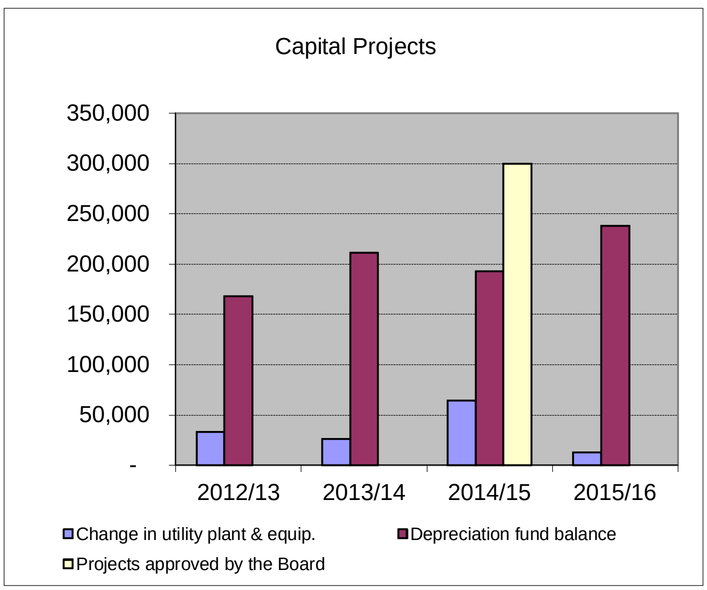
Date of last rate increase	5/8" Domestic	3/4"	1"	1-1/2"	2"	3"	4"	6"	Consumption rate(per 1000 gal)	Quarterly bill (5/8" & 13500 gal)
										13.5
1-Apr-11	69.05	103.12	171.24	341.55	545.92	1090.90	1704.01	3,407.09	6.09	151.27

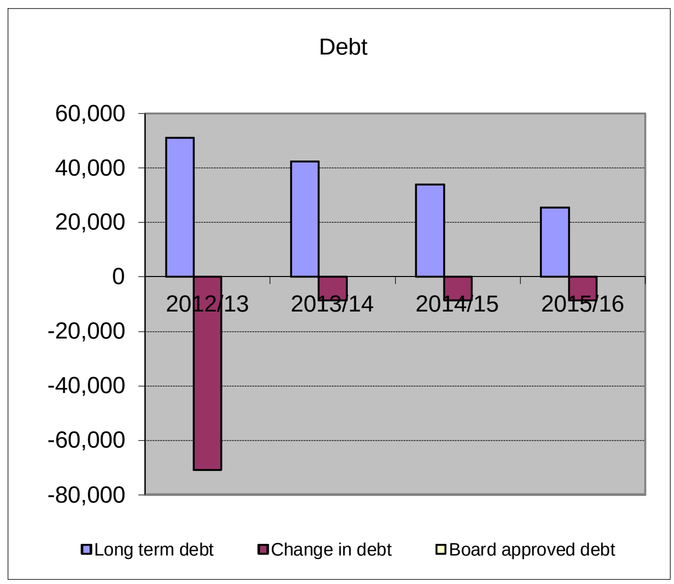
Document Number: 221840

The Village of Lawrencetown Water Utility 2016







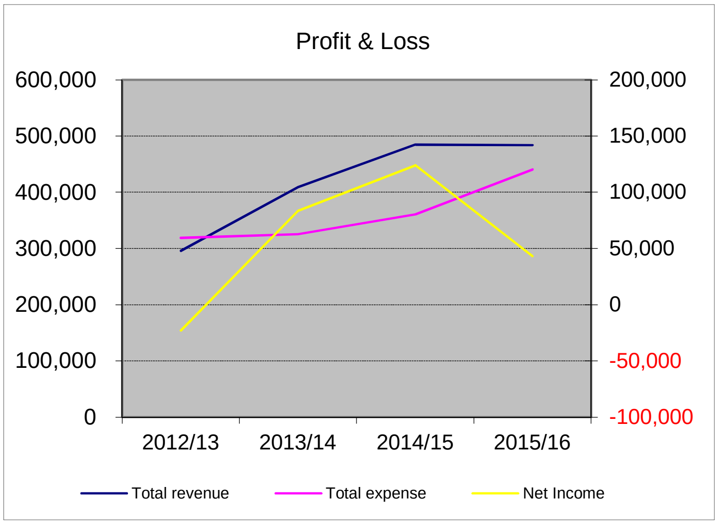


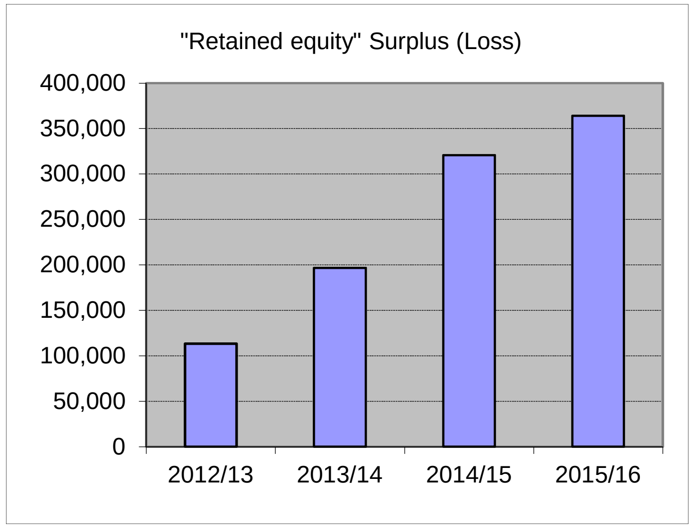
Return on Rate Base	2012/13	2013/14	2014/15	2015/16
	4.51	7.18	4.99	6.59

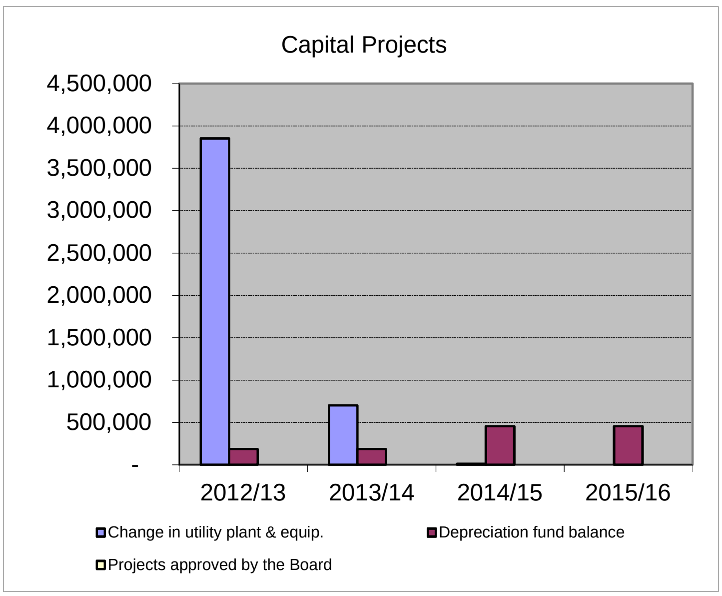
Date of last rate increase	5/8" Domestic	3/4"	1"	1-1/2"	2"	3"	4"	6"	Consumption rate(per 1000 gal)	Quarterly bill (5/8" & 13500 gal)
										13.5
1-Apr-13	62.76	92.39	151.66	299.84	477.65	951.81	1485.25	N/A	4.40	122.21

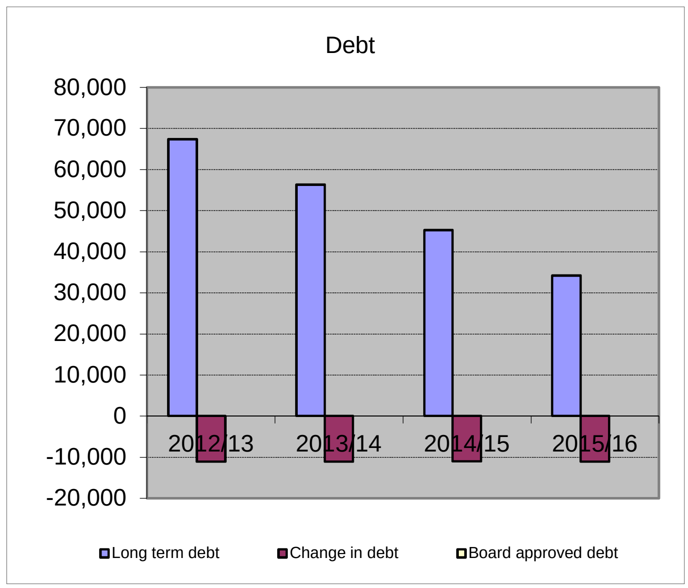
Document Number: 218643

Village Commissioners of Baddeck Water Utility 2016









Return on Rate Base	2012/13	2013/14	2014/15	2015/16
	1.41	0.96	0.12	-1.19

Date of last rate increase	5/8" Domestic	3/4"	1"	1-1/2"	2"	3"	4"	6"	Consumption rate(per 1000 gal)	Quarterly bill (5/8" & 13500 gal)
										13.5
1-Apr-13	46.79	68.81	112.86	222.96	355.59	707.42	1103.80	2,204.85	3.50	93.98

APPENDIX 5: RISK ANALYSIS VIA STOCHASTIC MODELING

Risk Analysis Via Stochastic Modelling

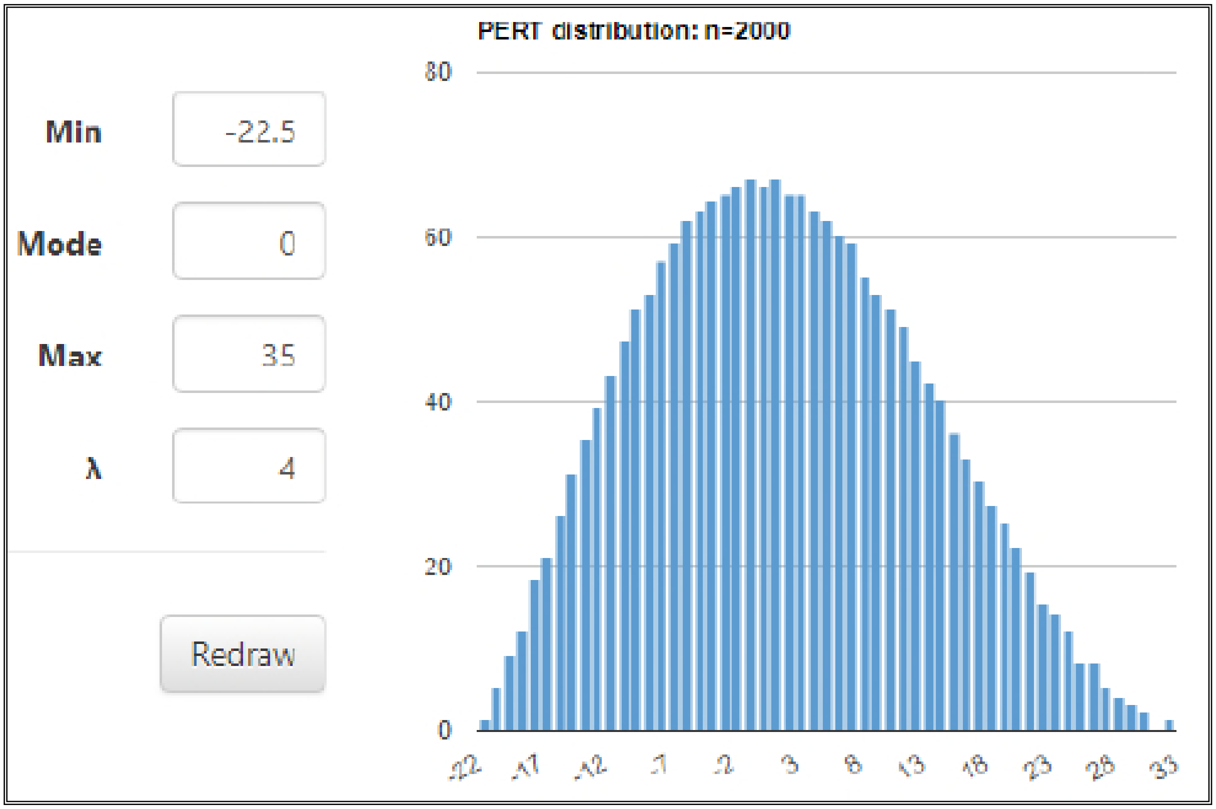
A stochastic model is a tool for estimating probability distributions of potential outcomes by allowing for random variation in one or more inputs over time. The random variation is usually based on fluctuations observed in historical data. Distributions of potential outcomes are derived from a large number of simulations that reflect the random variation in the input(s). In this study the variables subject to stochastic modelling are the construction and operating costs for wells and the central water systems)

To specify the form of the stochastic model we drew information from the 2016 AACE International Cost Estimate Classification System – Bulletin No. 18R-97 to identify the observed low and high ranges for Class D estimates. The bulletin indicates that Class D estimates have an expected accuracy of about 35% on the high side and about -22.5% on the low side. This is to say that estimators are more likely to calculate higher estimates of costs than lower estimates. In addition, the relative frequency of estimates that were low by 22.5% or high by 35% was extremely low compared to the frequency of estimates close the center of the accuracy range.

We assumed that the most likely accuracy was about +/-0% and that the least likely values were -22.5% and +35%. These assumptions result in a stochastic model based on a random normal distribution whose mode is 0% and is skewed so that values above the mode occur more frequency than those below the mode.

PERT Distribution Applied in This Study

The figure below provides an overview of the shape of the PERT distribution of cost estimation accuracy rates used in this analysis. The PERT distribution emphasises the "most likely" value (the mode) over the minimum and maximum values.



Source: https://www.riskamp.com/beta-pert

APPENDIX 6: DETAILED RESULTS OF THE BENEFIT/COST ANALYSES

SURFACE WATER CENTRAL SYSTEM

The PV of benefits to the ratepayers of the Village will likely fall in the following ranges:
- 12% chance that the PV of benefits will be between $22.9 million and $25.3 million.
- 67% chance of being between $25.4 million and $28.8 million.
- 21% chance of being between $28.9 million and $32.8 million.

The PV of costs to the ratepayers of the Village will likely fall in the following ranges:
- 14% chance that PV of costs will be between $14.1 million and $16.0 million.
- 74% chance that the PV of costs will be between $16.1 million and $18.8 million.
- 12% chance that the PV of costs will be between $18.9 million and $22.0 million.

The net present value (NPV) of benefits and costs will likely fall in the following ranges:
- 18% chance of being between $3.7 million and $8.5 million.
- 68% chance of being between $8.6 million and $12.7 million.
- 15%chance of being between $12.8 million and $17.7 million.

The median value of the range of NPVs is about $9.9 million (2018$).

The benefit/cost ratios (BCR) will likely fall in the following ranges:
- 9% chance of being between 1.19 and 1.43.
- 75% chance of being between 1.44 and 1.78.
- 15% chance of being between 1.79 and 2.19.

The median value for the BCR is about 1.58, reflecting a 25-year real social rate of return of about 58%. (1.9% on an annual average basis).

GROUNDWATER CENTRAL SYSTEM

The PV of benefits to the ratepayers of the Village will likely fall in the following ranges:
- 14% chance that the PV of benefits will be between $23.0 million and $25.4 million.
- 66% chance of being between $25.5 million and $28.9 million.
- 20% chance of being between $29.0 million and $33.0 million.

The PV of costs to the ratepayers of the Village will likely fall in the following ranges:
- 11% chance that PV of costs will be between $12.0 million and $13.5 million.
- 78% chance that the PV of costs will be between $13.6 million and $15.9 million.
- 11% chance that the PV of costs will be between $16.0 million and $18.3 million.

The NPV of benefits and costs will likely fall in the following ranges:
- 16% chance of being between $7.0 million and $11.1 million.
- 69% chance of being between $11.2 million and $15.3 million.
- 15% chance of being between $15.4 million and $20.2 million.

The median value of the range of NPVs is about $12.6 million (2018$).

The BCRs will likely fall in the following ranges:
- 11% chance of being between 1.43 and 1.72.
- 69% chance of being between 1.73 and 2.08.
- 20% chance of being between 2.09 and 2.63.

The median value for the BCR is about 1.88, reflecting a 25-year real social rate of return of about 88%. (2.5% on an annual average basis).