Best Management Practices for the Control of Disinfection by-Products in Drinking Water Systems in Newfoundland and Labrador

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Government of Newfoundland & Labrador
Department of Environment and Conservation
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Executive Summary

Provincial guidelines require that drinking water supplies be disinfected and maintain a disinfectant residual in the water distribution system in order to ensure the destruction of potentially harmful pathogens. Chlorine is the most commonly used form of disinfectant in the province. Disinfection by-products (DBP) are chemical compounds formed by the reaction of a water disinfectant with a precursor in a water supply system. DBPs are undesirable in drinking water as there is some evidence that long-term exposure may cause health risks. While minimizing disinfection by-products is important, the risks of not disinfecting water far outweigh the risks created by disinfection by-products. There is a wide array of mitigative options available to deal with DBP issues, and any action taken to reduce one type of DBP is likely to help reduce other forms as well. The main DBPs of concern in Newfoundland and Labrador are trihalomethanes (THMs), bromodichloromethane (BDCMs), and haloacetic acids (HAAs).

The problem of high disinfection by products in drinking water systems is not an isolated issue, but affects approximately a third of public drinking water systems and up to half the population of the province. The seriousness of DBP issues ranges from minor to very major, but to date only limited action in the form of infrastructure upgrades has been taken to address the issue. This report is intended to provide a comprehensive overview of the extent of the DBP problem, factors contributing to the problem, possible solutions and their effectiveness, and how to determine the most appropriate solutions to address DBP issues in individual community drinking water systems.

The two main products of this report are:

- The Best Management Practices for the Control of Disinfection By-Products
- The Decision Making Framework for the Selection of DBP Corrective Measures

The above tools are included in Appendix B and C of this report respectively.

The Decision Making Framework for the Selection of DBP Corrective Measures was developed as an iterative process based on known DBP formation behaviour and best management practices used to deal with DBPs in other jurisdictions; assessment of DBP characteristics and response to existing corrective measures in Newfoundland and Labrador; and through modeling of several water distribution systems that are experiencing DBP problems in the province. The framework developed has been tailored towards addressing THM issues; however, the approach is a holistic one that can be used to mitigate issues associated with other DBPs.

There is no standard solution that will address the issue of high DBP levels in drinking water for all communities. There are numerous probable causes that may be contributing to the formation of DBPs as identified in this report, just as there are numerous potential corrective actions that can be taken to address the problem. The difficulty lies in selecting the most appropriate corrective measure in light of what might be contributing to DBP levels. The selected corrective measure must address the issue of DBPs, but it must also be sustainable, i.e. fit the community involved in terms of available resources

and other solution constraints. Once a preferred corrective measure is selected and implemented, further monitoring and review is required to ensure that the DBP problem has been corrected by the action taken.

The Best Management Practices (BMPs) for the Control of Disinfection By-Products can be used to help reduce THMs and other DBPs for new, upgrading, and existing water distribution systems. These BMPs have been shaped by the understanding developed of THM characteristics and behaviour, the assessment of various corrective measures, and through modeling of water distribution systems. The adoption of BMPs by consultants, owners and operators of water systems, and government departments would be a first step towards dealing with DBP issues.

For a number of years the province has been monitoring drinking water systems for different DBPs to try and determine the degree and status of the problem as part of the Multi-Barrier Strategic Action Plan (MBSAP). This report is part of a more proactive drive by the Department of Environment and Conservation to introduce a new element to the MBSAP of issue analysis and identification of potential sustainable corrective measures to drinking water quality issues.

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List of Acronyms

AC	Asbestos Concrete
ANOVA	Analysis of variance
ANSI	American National Standards Institute
AWWARF	American Water Works Research Foundation
BDCM	Bromodichloromethane
BMP	Best management practice
BWA	Boil water advisory
C	Central
CCME	Canadian Council of Ministers of the Environment
CDM	Chlorine demand management
CFD	Computational fluid dynamics
CI	Cast iron
Cl	Chlorine
CSTR	Continuous stirred tank reactor
CT	Contact time
CWWA	Canadian Water and Wastewater Association
DAF	Dissolved air floatation
DBA	Dibromoacetic acid
DBCM	Dibromochloromethane
DBP	Disinfection by-product
DCA	Dichloroacetic acid
DE	Dead ends
DI	Ductile iron
DICL	Cement Lined Ductile Iron
DOC	Dissolved organic carbon
DOEC	Department of Environment and Conservation
DOEL	Department of Environment and Labour
DPD	Diethyl-p-phenylene diamine
DWQ	Drinking water quality
E	Eastern
EC	Enhanced coagulation
EDR	Electrodialysis reversal
EPA	Environmental Protection Agency
EPANET	Environmental Protection Agency network model
FILO	First in-last out
GAC	Granular activated carbon
GCDWQ	*Guidelines for Canadian Drinking Water Quality*
GIS	Geographic information system
HAA	Haloacetic acids
HAN	Haloacetonitriles
HDPE	High density polyethylene
HRT	Hydraulic residence time
HTH	High test calcium hypochlorite
IAO	Insurance Advisory Organisation
iMAC	Interim maximum allowable concentration

IX	Ion exchange
kPa	Kilopascals
L	Labrador
LIFO	Last in-first out
MAC	Maximum acceptable concentration
MBA	Monobromoacetic acid
MBSAP	Multi-barrier strategic action plan
MCA	Monochloroacetic acid
MCL	Maximum concentration limit
MDPE	Medium density polyethylene
MIOX	Mixed oxidants
N	Northern
N	Sample size
NDMA	N-Nitrosodimethylamine
NL	Newfoundland and Labrador
NOM	Natural organic matter
NSF	National Sanitation Foundation
NTU	Nephelometric turbidity units
OETC	Operator education, training and certification
O&M	Operation and maintenance
PE	Polyethylene
POE	Point of entry
POU	Point of use
PPWSA	Protected public water supply areas
PSI	Pounds per square inch
PVC	Polyvinylchloride
PWDU	Potable water dispensing unit
RMS	Root mean square
RO	Reverse osmosis
RPM	Rotations per minute
RTM	Retention time management
SCADA	Supervisory control and data acquisition
TCA	Trichloroacetic acid
TCU	True colour unit
TDH	Total dynamic head
TDS	Total dissolved solids
THM	Trihalomethane
TOC	Total organic carbon
UV	Ultraviolet
UV_{245}	Ultraviolet absorbance at 245nm wavelength
VIF	Variance inflation factor
VOC	Volatile organic compounds
W	Western
WDM	Water demand management
WHO	World Health Organization
WTP	Water treatment plant

1.0 Introduction

Disinfection by-products (DBP) are chemical compounds formed by the reaction of a water disinfectant with a precursor in a water supply system. For example, in water disinfection, chlorine and natural organic matter are precursors to trihalomethanes (THM). DBPs are undesirable in drinking water as there is some evidence that long-term exposure may cause health risks. While minimizing disinfection by-products is important, the bottom line is that the risks of not disinfecting water far outweigh the risks created by disinfection by-products.

There are hundreds of different types of known DBPs associated with different forms of drinking water disinfection. In Newfoundland and Labrador chlorine disinfection is the most commonly used form of disinfection. Out of approximately 536 public water supply systems in the province, 459 use some form of chlorine (gas, liquid, powder) for disinfection.

Monitoring for DBPs in the province has focused on THMs, a form of THMs known as bromodichloromethane (BDCM), and haloacetic acids (HAA). These are the most abundant DBPs formed by the disinfection of water with chlorine. Out of approximately 536 public water supply systems, 124 display issues with high levels of THMs, and 45 with high levels of BDCMs in comparison to the Guidelines for Canadian Drinking Water Quality (GCDWQ). Using the US guideline, approximately 184 public water supply systems in the province display issues with high levels of HAAs. The majority of these exceedances occur in small, rural drinking water systems, and usually in combination with other parameter exceedances, particularly colour.

This report is meant to be a holistic document that can be used to address different types of DBP issues. The focus of the document, however, is on THMs, which are considered to be the most pressing of the known DBPs in the province's drinking water systems. Not all DBPs are the same, each having its own different formation mechanism and associated health risks. Any action taken to reduce one type of DBP, however, is likely to help reduce other forms as well. DBPs are usually present in complex mixtures that can vary greatly as conditions very. The most widely studied process is chlorination and there are comparatively few studies on other disinfectants and their DBPs.

Water distribution, disinfection and treatment offer special challenges for small water systems. While large cities can provide specialized treatment, have highly trained staff, and monitor water quality on a daily basis, this is not always the case in smaller communities where operation of the water system is often done on a volunteer basis. The sparse geographical distribution of small communities in the province along with low populations of generally 100 to 250 people does not lend itself to easy solutions to deal with drinking water quality issues. Small towns simply do not have access to the same resources (human or financial) that larger systems do. Smaller communities have a lower median household income, and there are fewer businesses and industry resulting in a lower tax base. Populations in most small communities in Newfoundland and Labrador tend to be aging and declining in size. These factors make it more difficult for small towns to afford the infrastructure and qualified operators necessary to provide high quality drinking water to their populations if there are any water quality issues. Money is hard to come by for repairs, upgrades and even daily operation. Even large communities, however, are not immune to issues with disinfection by-products.

For every type of DBP related water quality issue, there is a corresponding array of mitigative options available. Solutions to the DBP problem do not come cheap, however, and many of the corrective measures available will be beyond the fiscal and human resource capacity of small communities in Newfoundland and Labrador. Implementation of water quality improvement measures must be managed in a reasonable time frame, in light of the need of individual communities for water quality improvements, and the feasibility of that community being able to implement improvements.

This report is divided into five main sections that act as a starting point to answering the question of how to address DBP issues in the province's drinking water systems:

- *Probable Causes*- what is causing DBPs to form in drinking water systems in Newfoundland and Labrador?
- *Characteristics of DBPs*- what are the characteristics of the main DBPs found in Newfoundland and Labrador?
- *Corrective Measures*- what are the possible solutions to high levels of DBPs?
- *Distribution System Modeling*- how do systems with DBP issues respond to corrective measures?
- *Managing DBPs*- how to decide which corrective measures are best suited to a community's needs?

1.1 Objectives and Scope of Report

The scope of this report encompasses providing a holistic approach to dealing with DBPs in drinking water systems in Newfoundland and Labrador. Objectives this report aims to achieve in order to provide a comprehensive BMPs for communities in the province to deal with DBP issues include:

1. Identifying what is causing DBP problems in public drinking water systems in the province
2. Identifying behavioural characteristics of major DBPs in the province
3. Identifying potential corrective measures for the mitigation of DBP issues
4. Determining the effectiveness of existing DBP corrective measures already in place in the province
5. Developing water distribution system models for the evaluation of probable causes of DBPs and the effectiveness of corrective measures to address DBP issues
6. Identifying relevant constraints to the implementation of corrective measures
7. Providing a decision-making framework so that suitable mitigative measures can be selected based on identified DBP triggers, with final selection based on an assessment of relevant solution constraints
8. Developing BMPs for the control of DBPs in new, upgrading and existing water distribution systems

9. Identifying gaps in current activities from across jurisdictions in relation to DBPs and making appropriate recommendations to address these gaps

This report does not offer definitive solutions for any individual community's DBP issues, but is meant to provide strategic direction government, communities, consultants and the public.

1.2 Analysis Tools

A variety of analysis tools were used in the completion of this report including two software analysis programs:

- Minitab 14– statistical methods
- EPANET– water distribution system modeling

Various statistical analyses was performed with the help of Minitab including generation of descriptive statistics, correlation analysis, analysis of variance (ANOVA), regression analysis, time series plots, etc. For correlation and ANOVA analysis, a p value of less than 0.05 or 0.01 was used to indicate the level of significance (or alpha- α). The smaller the p-value, the smaller is the probability that you would be making a mistake by rejecting the null hypothesis (that the data populations are equivalent), when in fact the hypothesis is true.

The accuracy of the above statistical analysis may have been affected by:

- the sample sizes being compared
- the censored nature of the data sets– different laboratories, data gaps, less than detect data protocols
- the pre and post date population divisions are assumed to be from the date of issue of the environmental permit to construct, but actual construction could have been up to a year following the permit date
- data sets might be more heavily weighted with samples taken from the end half of the distribution system

The use of EPANET is discussed in detail in Section 5.

2.0 Causative Factors in the Formation of DBPs

A disinfection by-products is an undesirable chemical compound formed by the reaction of a drinking water disinfectant with a precursor substance in a water supply system. Water quality changes in drinking water distribution systems, such as the formation of DBPs, occur as a result of complex and often interrelated physico-chemical and biological processes. The main factors affecting the level of DBPs in water coming out of the tap include:

- Source characteristics
- DBP precursor characteristics
- Distribution system characteristics
- Distribution system operation and maintenance practices

The main DBPs of concern in the province are trihalomethanes (THMs) and haloacetic acids (HAAs) formed as a result of the use of chlorine in disinfection. Of approximately 536 public drinking water supplies in the province, 124 systems displayed issues with THMs, 45 displayed issues with one of the four species that make up THMs known as bromodichloromethanes (BDCM), and 184 systems displayed issues with HAAs (DOEC, 2008).

The following sections provide further information on the main factors affecting DBP levels in drinking water systems in Newfoundland and Labrador.

2.1 Source Supply Characteristics

The majority of water sources in the province of Newfoundland and Labrador (61 %) are surface water sources– rivers, lakes, ponds, streams, canals, reservoirs. The percentage of the province's population serviced by surface water sources is even greater.

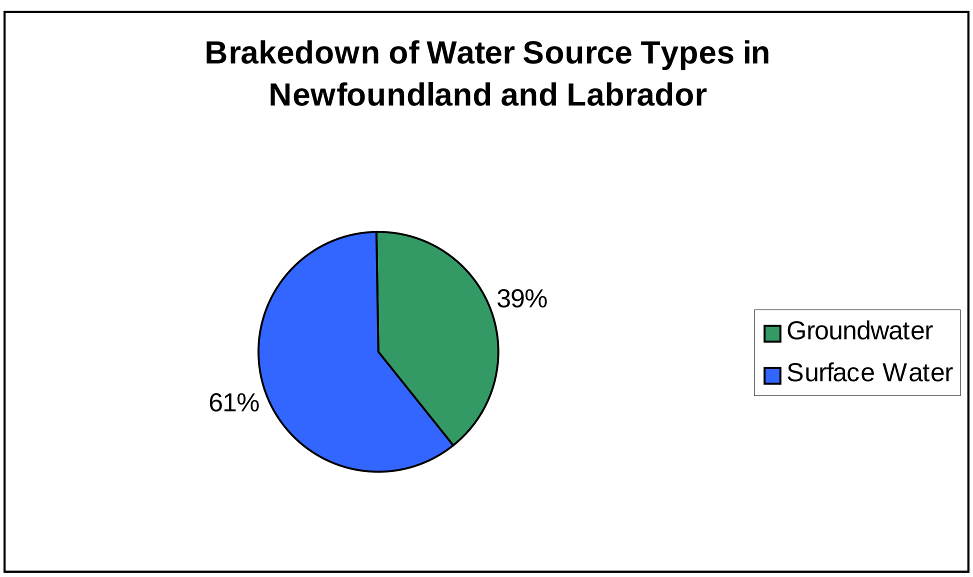


Figure 1: Percentage of source water types in Newfoundland and Labrador

Characteristics of the raw water source greatly affect DBP levels. Surface water is typically higher in organic content than groundwater, while groundwater typically has a higher mineral content. Natural, as opposed to anthropogenic, conditions tend to play the predominant role in influencing water quality in Newfoundland and Labrador. The occurrence and level of DBP precursors in water sources depends on geological, physical and environmental factors such as:

- Drainage area land cover
- Drainage area land use
- Drainage area soil characteristics
- Bedrock geology
- Trophic stage
- Lake size
- River flow rate
- Salt water influence
- Climate and season

The location of the intake structure in a surface water supply will have an effect on the quality of water entering the system. For surface water supplies, if the intake structure is located at the surface of the water supply, there is an increased risk of operational hazards, such as blockages due to the accumulation of ice, organic material, debris, etc. A submerged intake structure will draw water from below the surface and will minimize the risk of blockages. However, the intake should not be installed directly on the bottom of the water body to avoid any sediment being drawn into the system. Major water level fluctuations should also be considered when designing the intake infrastructure and location. Similarly, poorly constructed groundwater wells can have a major impact on groundwater source quality.

2.2 Source Water Quality Characteristics

The following table provides a snapshot of drinking water source quality characteristics from communities across Newfoundland and Labrador using data from January 2003 to January 2006. The mean, maximum and minimum values were derived from over 1000 source water samples gathered from surface and groundwater sources during the period of interest. There are approximately 536 public drinking water systems in the province.

Table 1: Surface and groundwater source water quality statistics Jan 2003-Jan 2006

Water Quality Parameter	GCDWQ	Mean	Maximum	Minimum
Colour (TCU)	15	24.5	282	0
Turbidity (NTU)	1	1.1	100	0
pH	6.5-8.5	6.9	9.6	4.1
DOC (mg/L)	-	3.82	25.2	0
TDS (mg/L)	500	135.3	2550	0
Nitrogen (mg/L)	-	0.20	1.73	0
Phosphorous (mg/L)	-	0.26	0.5	0
Bromide (mg/L)	-	0.042	3.74	0

Iron (mg/L)	0.3	0.20	17.8	0
Manganese (mg/L)	0.05	0.054	1.77	0
Temperature (ºC)	-	9.6	25.0	-0.5

The data above is province wide, including both surface and groundwater sources which typically display slightly different water quality characteristics. Of particular note is that average ambient source water colour, turbidity and manganese are above the recommended Guideline for Canadian Drinking Water Quality (GCDWQ).

2.2.1 Colour

Colour in drinking water is due to the presence of coloured organic substances or metals such as iron, manganese and copper. Highly coloured industrial wastes (eg. paper mill effluent) can also contribute to colour. The presence of colour is not a direct health risk, but it is aesthetically displeasing. Colour is of concern, however, as its presence in untreated source water is an indirect indicator of THM formation potential when water is chlorinated.

Bogs and wetlands produce large amounts of dissolved organic materials such as tannins, lignins, humic and fulvic acids, which can give water a tea-like colour. Calcium carbonate from regions with limestone bedrock may give water a greenish colour, while ferric hydroxide (iron) may impart a reddish colour. The degree of colouring will depend on the concentrations of these and other substances. Water colour is highly influenced by land cover in a basin. Bogs and wetland drainage will contribute high levels of colour to surface runoff, while less organic soils or exposed bedrock in a basin will contribute little to colour. It has also been demonstrated that there is a gradual decrease of colour as one goes downstream on a watercourse as a result of physical, chemical and microbial mechanisms in the water (AwwaRF, 1994). As most surface water sources are located in headwater areas, they may be expected to have higher colour.

Most surface water sources in the province have naturally high concentrations of organic matter. In some cases high colour can be linked to siltation events. Colour in groundwater supplies is usually due to iron and manganese, however, in some cases surface water infiltration or pore casing design is the culprit.

2.2.2 pH

The pH of water is an indicator of the natural buffering capacity of that body of water. Waters of pH 7.0 are considered to be neutral, those below pH 7.0 are considered to be acidic, and those above pH 7.0 are considered basic. Acidic waters have a low buffering capacity (ability to accept a large amount of acid before significant changes in pH will occur) and are typical of runoff from peatlands, bogs and wetlands. Most drinking water in Newfoundland and Labrador tends to be on the acidic side.

2.2.3 Turbidity

Turbidity refers to water's ability to transmit light or the cloudiness of the water and is the result of fine organic and inorganic particles in the water that do not settle out. Turbidity in source waters results from suspended solids and materials such as clay, silt or microorganisms in the water. Turbidity may be caused by naturally occurring silt and sediment runoff from watersheds. Disturbed areas, such as those with road construction, tend to have higher levels of turbid water than undisturbed areas because of increased sediment input and siltation. Increases in turbidity often occur after rainfall events and may provide bacteria with particles for attachment which protect them from disinfectants when applied. Increased turbidity of drinking water is less aesthetically pleasing and may interfere with the disinfection process.

2.2.4 DOC

DOC is the result of microbial degradation of organic matter, oxidative polymerization of phenolic compounds in plants and soil, and photolytic degradation of NOM (Singer, 1999). In drinking water supplies, organic carbon compounds consist of humic and fulvic acids, polymeric carbohydrates, polysaccharides, proteins, carboxylic acids, and low molecular weight acids. Organic carbon is often considered the growth-limiting nutrient in water distribution system bacterial re-growth. Organic compounds are present from natural processes and are frequently called natural organic matter (NOM). Increases in dissolved organic carbon (DOC) concentration are generally observed after heavy rainfalls, and are attributed to increased leaching from soil organic matter during high river discharges. DOC levels are significantly higher in surface water sources in the province than in groundwater sources as can be seen in Table 2.

Sources of DOC can be categorized as allochthonous, entering the system from the terrestrial watershed, and autochthonous, being derived from biota growing in the water body (AwwRF, 1994). In temperate climates most of the DOC originates from the degradation and leaching of organic detritus in the watershed and is transported by surface runoff and shallow groundwater flow. Plant material may be present as vegetation, litter, or modified in highly organic or peaty layers. Marshy areas produce water with high DOC as water moves directly from being in contact with vegetation into streams with no contact with adsorptive material such as clays or oxides. Sandy areas produce water with high DOC as water moves through soil with very low adsorption capacity for DOC. Areas which have permeable soil horizons rich in clay and oxides produce water with low DOC. Decomposition of aquatic organisms such as phanerogams, algae, plankton, bacterial and animal biomass all contribute to NOM.

The Netherlands Waterworks Association has set a DOC guideline of 5 mg C/L and intends to lower this guideline to 3 mg C/L for the restriction of disinfection by-product formation (AwwaRF, 1994).

2.2.5 Bromide

Bromide is the eighth most abundant solute in seawater with an average concentration in seawater of 67 mg/L. It makes up approximately 0.7% of sea salts found in seawater. Bromide levels in rainwater and snow are known to range from 0.005 to 0.15 mg/L.

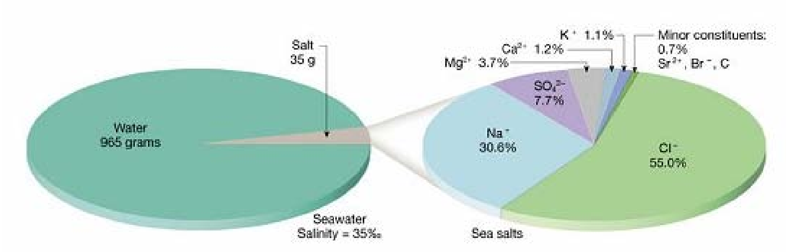


Figure 2: Make up of sea salts in saltwater

The presence of bromide in drinking water is due to a number of factors including:

- Ocean saltwater spray
- Coastal saltwater intrusion
- Saline irrigation drainage
- Soil salinization
- Mixing of surface and groundwater sources
- Geological sources
- Connate water- ancient, geologically trapped seawater
- Road salt
- Tidal influences
- Oil-field brines
- Industrial chemicals

Bromide precipitate from the evaporation of saltwater mist or spray is probably significant, especially in exposed, windward, coastal locations of the province. Newfoundland as a whole has the strongest winds of any province in Canada, with most meteorological stations recording average annual wind speeds greater than 20 km/h. Generally, coastal areas have stronger winds than inland, valleys have lighter winds than elevated terrain, and winter is decidedly windier than summer. Bonavista on the East Coast is the windiest location in the province, with an average annual wind speed of 28 km/h. Winds are predominantly from the west year-round, but variations are common both from location to location and from month to month. Prevailing wind directions are west in winter and west-southwest in summer (Environment Canada, 2006).

Ocean saltwater intrusion, where saline coastal aquifers discharge groundwater into freshwater aquifers, is not unheard of in the province, and is more particularly associated with flat coastal areas. Higher elevations or hills just off the coastline act like freshwater reservoirs that drive the salt water/ fresh water interface outward. Fogo Island is known to be prone to saltwater intrusion, as groundwater wells in the area have had to be abandoned due to their brackishness.

On average, groundwater has approximately five times the TDS and bromide concentration of surface waters. Some surface water drinking sources are heavily under the influence of groundwater in the form of springs and as evidenced by their elevated TDS levels. Bedrock geology known to contain high concentrations of bromide include sedimentary rock of marine origin and evaporite rock. The main areas of evaporite rock in Newfoundland are in St. George's Bay and the Codroy Valley. Mixed groundwater and surface water distribution systems, such as in Port au Choix and Port au Port West, provide the necessary bromide and organic carbon mix necessary for the formation of brominated THMs. Bromide is also a common impurity found in road salt. Soil salinization and saline irrigation drainage are not considered a source of bromide in waters of the province.

Table 2: Average TDS, bromide and DOC in groundwater and surface water- Jan 2003 to Mar 2006

Supply Type	Average TDS, Jan 2003-Mar 2006 (mg/L)	Average Bromide, Jan 2003-Mar 2006 (mg/L)	Average DOC, Jan 2003-Mar 2006 (mg/L)
Surface Water	40	0.015	6.37
Groundwater	230	0.068	1.27

2.2.6 Long Term Water Quality Trends in Newfoundland and Labrador

In a recent study of water quality trends in ambient water bodies of Newfoundland and Labrador it was discovered that even in pristine watersheds without any significant level of development activity, changes in water quality were observed over the period since 1986 (Dawe, 2006). Several of these trends may be influencing drinking water quality and the formation of DBPs, including an observed increase in colour and turbidity throughout water bodies in the province. This trend can be linked to climate variability and an increase in precipitation and hence streamflow in all regions of the province over the same period.

2.2.7 Seasonal Water Quality Fluctuations

Water quality varies throughout the year, even in pristine watersheds unaffected by any significant development activity. The driver of this change is simply annual variation in air temperatures, precipitation and runoff. Figure 3 shows averaged ambient water quality parameter values from the period 1986-1999 from five pristine rivers from across Newfoundland (Spout Cove Brook, Indian Brook, Southern Bay River, Main River, Lloyds River). Water temperature peaks in August, and is at a minimum in January and February. pH likewise peaks in August, and is at it's lowest in April. Conductivity (which can be used as an indicator of TDS) has two low points in the spring (May-June) and fall (Oct-November), and two high points in the winter (February) and summer (September), corresponding with high and low flow periods. Colour varies throughout the year but peaks in the fall in October.

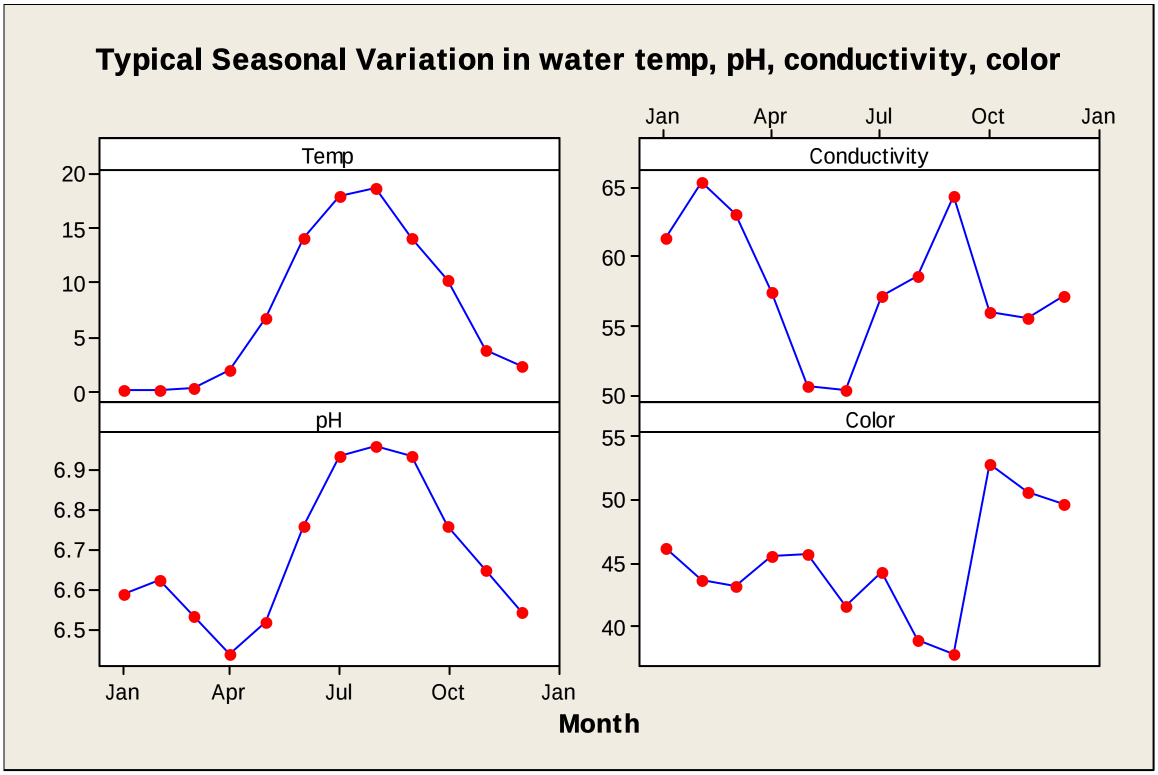


Figure 3: Time series of average ambient water quality values from five pristine rivers in NL

Water pH, colour, bromide and DOC can all vary from one month to the next as indicated in the following figure of average source water quality from Jan 2003 to March 2006. In general high values of colour and DOC do not coincide with high values in water temperature and pH. Higher average bromide concentrations in source waters in the province were observed in the spring, particularly with April and May runoff.

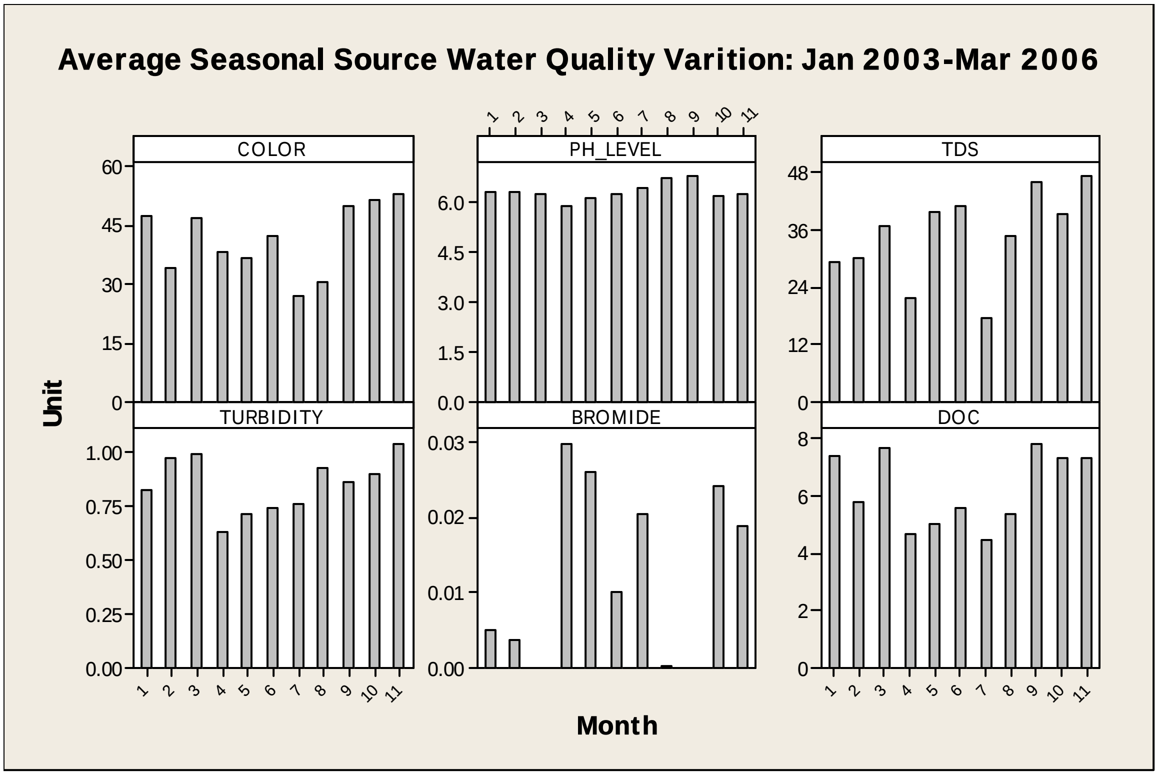


Figure 4: Average seasonal source water quality variation- Jan 2003 to Mar 2006

When examining the influence of water temperature in distribution system, it is important to remember the buffering effect of the ground around the pipes. Increases in water temperature with increasing water age in the distribution system have been observed during some seasons (fall, winter, spring). The opposite can be observed in summer with a decrease in temperature with increasing water age (AwwaRF, 2006).

2.3 Disinfection

The primary reason for disinfecting public drinking water in Newfoundland and Labrador is to destroy or inactivate disease-causing organisms. In addition, the disinfection process protects the distribution system by inhibiting microbial growth in the pipe network. The disinfection process can be divided into two main components, primary disinfection and secondary disinfection. Primary disinfection is executed prior to the delivery of water to the first customer in the distribution system. Secondary disinfection provides the disinfectant residual required for protection of drinking water throughout the distribution system. Primary and secondary disinfection can be achieved by one form of disinfectant or through a combination of disinfection methods.

The majority of public drinking water systems in Newfoundland and Labrador use chlorination for disinfection purposes. In addition to chlorination, other forms of disinfection that are utilized in water supply systems throughout the province include UV, ozone, chloramines and MIOX.

Disinfectants such as chlorine are added to drinking water for a number of reasons including (DOEC, 2006):

- To eradicate and inactivate pathogens
- To act as an oxidant in water treatment
- To remove taste and colour
- To oxidize iron and manganese
- To improve coagulation and filtration efficiency
- To prevent algal growth in sedimentation basins and filters
- To prevent biological re-growth in the water distribution system

2.3.1 Chlorine Disinfection

The goal of water disinfection is the inactivation of micro-organisms (viruses, bacteria, protozoa, etc.) which can cause serious illness and death. Continuous disinfection is mandatory for community water systems as part of the provincial *Standards for Bacteriological Quality of Drinking Water*. Chlorine is the most common chemical used for disinfection in the province, and even when not the primary disinfectant, is still required to provide the water system with sufficient residual disinfectant. The forms of chlorine most often used are chlorine gas and (liquid) calcium or sodium hypochlorite. Figure 5 provides a breakdown of the approximately 459 chlorination systems in the province based on type.

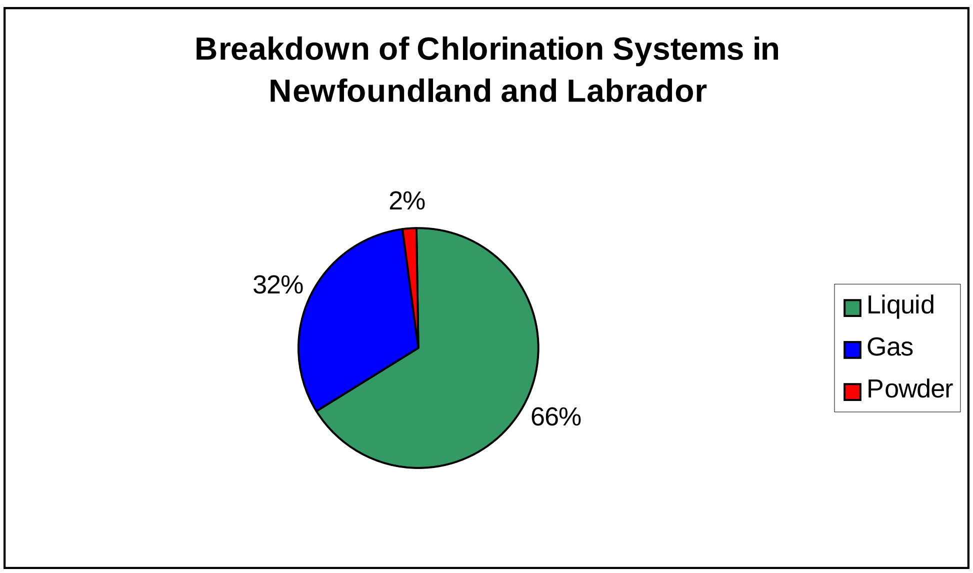


Figure 5: Breakdown of types of chlorination systems in NL

The use of chlorine in water systems to kill disease causing organisms began in 1905 in London, England. According to the AWWA, more than 79,000 tons of chlorine is used each year to treat water in Canada and the US. Chlorine has a lot of practical strengths:

- it can be administered as a liquid or gas
- it is effective against viruses and bacteria (although not very effective at removing *cryptosporidium* and *giardia*)
- it has a high oxidizing potential

- it has a lower cost
- it provides a minimum level of chlorine residual throughout the distribution system that protects against microbial recontamination or re-growth

The term re-growth is used to describe the chronic or periodic appearance of bacteria in the distribution system, either in the bulk water or at the pipe walls in a bio-film. Factors that can impact re-growth include nutrient levels, presence of disinfectants, temperature, hydraulic regime and pipe material.

Chlorine has a limited solubility in water. At 20 °C and atmospheric pressure, the solubility of chlorine in water is 7.29 g/L. The maximum recommended chlorine injection rate by chlorinator equipment manufacturers is 3500 mg/L (AwwaRF, 2004).

Higher chlorine doses favour the formation of HAAs over THMs. Increases in the chlorine dose also shift the formation of DBPs to the less bromine-substituted species (Singer, 1999).

Chlorine is typically added to water distribution systems at the following locations and doses:

- Pre-chlorination of raw water (5-15 mg/L)
- After coagulation and/or before sedimentation (5 mg/L)
- After sedimentation and/or before filtration (5 mg/L)
- After treatment but before distribution (0.5-1 mg/L)
- During distribution (0.5-2 mg/L)
- During maintenance activities (up to 50 mg/L)

In the US, a measurable chlorine residual is typically assumed to be 0.2 mg/L, and must be present at all points of water consumption. Chlorine residuals should not be needlessly large since large chlorine residuals may not appreciably reduce the health risk of pathogen exposure (compared with small residuals in the 0.2 mg/L range), while at the same time reactions of chlorine with naturally occurring organic compounds produce by-products.

2.4 Water Treatment

Several public drinking water systems in the province utilize water treatment processes in the delivery of their drinking water. Treatment processes used in water systems in Newfoundland and Labrador include the following:

- Arsenic removal
- Iron and manganese removal
- pH adjustment
- Sulfur gas removal
- Conventional water treatment plants
- Infiltration galleries

- Potable Water Dispensing Units (small packaged water treatment plants)
- Filtration

2.5 Water Distribution System Design Guidelines

The design, construction and operation of a water distribution system can have a major impact on drinking water quality. In 2005, the Department of Environment and Conservation released updated *Guidelines for the Design, Construction and Operation of Water and Sewerage Systems*. Relevant guidelines that ultimately have an effect on DBP formation include:

- Water which is delivered to consumers will meet current requirements of the DOEC with respect to microbiological, physical, and chemical qualities (3.2)
- Each water supply should take its raw water from the best available source, which is economically reasonable and technically possible (3.2)
- Filtration preceded by appropriate pre-treatment shall be provided for all surface waters (3.2.3.3)
- Withdrawal of water from more than one level if quality varies with depth (3.2.3.4)
- An upground reservoir is a facility into which water is pumped during periods of good quality and high stream flow for future release to treatment facilities (3.2.3.6)
- Site preparation of impoundments and reservoirs shall provide removal of brush and trees to high water elevation (3.2.3.7.1)
- Chemicals shall be applied to the water at such points and by such means as to assure maximum safety to the consumer (3.4)
- Chemical feed rates shall be proportional to flow (3.4.3.2) and a means to measure water flow must be provided in order to determine chemical feed rates
- Provisions shall be made for measuring the quantities of chemicals used (3.4.3.2)
- Weighing scales shall be provided for weighing cylinders at all plants utilizing chlorine gas and should be provided for volumetric dry chemical feeders (3.4.3.2)
- Liquid chemical storage tanks must have a liquid level indicator (3.4.3.9)
- At least 2 pumping units should be provided and shall have ample capacity to supply the peak demand against the required distribution system pressure without dangerous overloading (3.5.3)
- The top water level and location of the treated water storage structures will be determined by the hydraulic analysis undertaken for the design of the distribution system to result in acceptable service pressures throughout the existing and future service areas and should protect the quality of the stored water (3.6.4)
- Frequent cycling of pumps causes increased wear on controls and motors and also increases energy costs (3.6.4.3)
- Fire demands may not occur very often, however, when it does occur, the rate of water use is usually much greater than for domestic peak demand. Also, the required fire storage volume can account for as much as 50% of total capacity of the reservoirs (3.6.4.6)

- The time water stays in storage after disinfectants are added, but before the water is delivered to the first customer can be counted towards the disinfectant contact time (3.6.4.8)
- Supplemental chlorination may be required to maintain minimum chlorine residuals in water from water storage facilities that has insufficient residual chlorine (3.6.4.8)
- A detailed design of the inlet, outlet and baffling is required where storage facilities are used as supplemental chlorination stations (3.6.4.8)
- Some water systems use water from 2 or more sources, with each source having different water quality. The feasibility of blending sources should be investigated, as chemical quality of blended water may affect the integrity of the distribution system (3.6.4.9)
- Excessive storage capacity should be avoided where water quality deterioration may occur (3.6.5)
- Water storage requirements can be calculated using (3.6.5):

$$S = A + B + C$$

Equation 1: Water storage requirement

  - where:
    S = total storage requirement, m^3
    A = Fire storage, m^3, typically established by the appropriate Insurance Advisory Organisation (IAO)
    B = Peak balanced storage, m^3, 25% of maximum day demand
    C = Emergency storage, m^3, 25 % of A + B or 15 % average daily design flow or 40 % of average daily design flow when no fire storage
- When dead storage is present there must be adequate measures taken to circulate the water through the tank to maintain quality (3.6.5.4)
- An objective in both design and operation of distribution system storage facilities is the minimization of detention time and the avoidance of volumes of water that remain in the storage facility for long periods. The allowable detention time should depend on the quality of the water, its reactivity, the type of disinfectant used and the travel time before and after the water's entry into the storage facility. A maximum 72-hour turnover is a reasonable guideline. If it is not possible to have sufficient turnover of water in the storage facility, supplemental disinfection may be required (3.6.5.5)
- A detailed design of the inlet, and outlet, and if required, baffle walls, mixing, etc., is required to ensure maximum turnover of water in a storage tank (3.6.7.1)
- Adequate controls should be provided to maintain levels in distribution system storage structures and changes in water level in a storage tank during daily domestic water demand should be limited to a maximum 9 m to stabilize pressure fluctuations within the distribution system (3.6.7.2)
- The major requirements of a distribution system is to supply each customer with sufficient volume of treated water at an adequate service pressure (3.7)
- Design criteria of transmission and distribution mains should address the following (3.7.3.1):

  - Sizing for ultimate future design flows
  - Sizing and layout to ensure adequate supply and turnover of water storage facilities
  - Looping
  - Elimination of dead ends
  - Adequate valving to provide an efficient flushing program
- For small systems where water usage is strictly residential and there are no water usage records, then the Harmon Formula in conjunction with the theoretical water usage of 340 L/p/d can be used (3.7.3.2)
- The transmission and distribution system should be designed to maintain a minimum pressure of 275 kPa (40 psi) at ground level at all points in the distribution system under normal flow conditions. Fire flow residual pressure should be maintained at 150 kPa (22 psi) at the flow hydrant, and should be a minimum 140 kPa (20 psi) within the system for the design duration of the fire flow event. The normal working pressure in the distribution system should be 410 kPa to 550 kPa (60-80 psi). The maximum design pressure during minimum demand periods should not exceed 650 kPa (95 psi) (3.7.3.3)
- The minimum nominal diameter of pipe should be as follows:
  - 200 mm for primary distribution mains
  - 150 mm for distribution mains
  - 150 mm for service mains providing fire protection
- The minimum size of a watermain in a distribution system where fire protection is not to be provided should be a minimum of 75 mm in diameter. Watermains beyond the last hydrant can have pipe sizes from 50 mm down to 25 mm. For water service connections the minimum pipe size required is 20 mm (3.7.3.5)
- The maximum design velocity for flow under maximum day conditions for transmission mains, primary distribution mains, distribution mains and service mains should be 1.5 m/s. The maximum fire flow velocity should be 3.0 m/s. Flushing devices should be sized to provide a flow that provides a minimum cleansing velocity of 0.75 m/s in the watermain being flushed (3.7.3.6)
- Water distribution systems should be designed to exclude any dead ended primary distribution mains, and distribution mains unless unavoidable. Appropriate tie-ins (loops) should be made whenever practical. Where dead-ends mains occur, they should be provided with a fire hydrant if flow and pressure are sufficient, or with an approved flushing hydrant or blow-off for flushing purposes (3.7.3.7)
- The minimum size of watermain for providing fire protection and serving fire hydrants should be 150 mm (3.7.3.8)
- Chlorine application should be at a point, which will provide a contact time of at least 20 minutes at peak hourly flow with required free chlorine residual. The point of application shall be located in order to minimize the formation of DBPs without compromising the integrity of contact time (4.2.1)
- If primary disinfection is accomplished using some other chemical or process other than chlorine, then chlorine must be added as a secondary disinfectant to provide a residual disinfectant (4.2.1)
- CT factor = residual disinfectant concentration (mg/L) x contact time (min) (4.2.2.1)

- All water entering a water distribution system, after a minimum 20 minutes contact time at peak hourly flow shall contain a residual disinfectant concentration of free chlorine of at least 0.3 mg/L, or equivalent CT value. A detectable free chlorine residual must be maintained in all areas of the distribution system. Higher residuals may be required depending on pH, temperature and other characteristics of the water (4.2.3)
- Chlorine testing should include both free and total chlorine. All systems, as a minimum, should use the DPD (Diethyl-p-phenylene diamine) method for testing chlorine residuals to enable measurement to the nearest 0.02 mg/L in the range of 0.01-4.0 mg/L (4.2.3)
- Where flow varies, an automatic flow proportional system should be installed. If chlorine demand varies than a residual analyzer with recorder should be installed. If both the flow and the chlorine demand vary, then a compound loop system should be installed (4.2.4.4)

2.6 Water Distribution System Characteristics

Water distribution system characteristics that have a major influence on drinking water quality and DBP formation include:

- System configuration
- Pipe age, material and condition
- Water storage tanks
- Hydraulic conditions

2.6.1Water Distribution System Configuration

There are two main distribution system configurations used in the design of water distribution systems in the province: looped (closed or grid system) and branched (tree). The configuration of a water distribution system will impact the quality of drinking water that is delivered to the consumer. In fact, the quality of water may vary throughout the system based partly on the system configuration.

A looped water distribution system is the preferred design configuration. The closed system eliminates the presence of dead-ends which reduces the accumulation of sediment in the distribution system. Another important advantage of using a looped distribution system is that it is easier to maintain an adequate level of chlorine residuals throughout the entire distribution system. In addition, the operator of the system will have increased control during maintenance activities such as flushing.

Branched systems are a very common configuration for small communities in Newfoundland and Labrador. Many rural communities throughout the province consist of one main road that stretches throughout the community with few smaller side roads. Therefore, the branched configuration is the only alternative for a water distribution system. Disadvantages associated with this type of system configuration include the potential for multiple dead-ends, which results in stagnant water, and difficulty in maintaining adequate chlorine residuals throughout the distribution system.

2.6.2 Pipe Characteristics

Drinking water distribution systems may consist of one main pipe material or multiple types of pipe material. The type of pipe material utilized in the distribution system will have an impact on the quality of the water delivered to the consumer. The following is a list of pipe materials that are currently in use throughout the province:

- Cement Lined Ductile Iron (DICL)
- Polyvinyl Chloride (PVC)
- High Density Polyethylene (HDPE)
- Polyethylene (PE)
- Asbestos Concrete (AC)
- Steel or Stainless Steel

Water distribution system pipes deteriorate with age through corrosion, scaling, tuberculation and general ware and tear. Typical useful life for pipe in a distribution network can be anywhere from 25 to over 100 years. The older a pipe is, the worse its hydraulic performance becomes.

Corrosion of metal pipes is of particular concern in relation to disinfection and DBP formation. Corrosion is an oxidative process that occurs at the surface of a metal. The formation of biofilms on the insides of pipes results in the dissolution of the metal and the formation of scales and tubercles. Systems with significant amounts of corrosion by-product mass have been found to contain substantial microbial densities. By increasing the surface area of the pipe there are more sites for biofilm attachment. Cracks and crevices caused by corrosion provide sites that protect bacteria from disinfection. Some types of common corrosion products found on iron piping are also capable of adsorbing natural organic material from the bulk water providing a higher concentration of carbon on the pipe surface for bacterial growth. Combined, the effects of corrosion increase chlorine demand on the system.

Figure 6: Corroded CI pipe

Depending on conditions in the distribution system, pH can fluctuate (increase or decrease) by more than 2 units (AwwaRF, 2006). The main factor affecting such a variation is pipe material. THM concentrations increase with increasing pH while HAA concentrations decrease.

Tests comparing THM formation in a pipe environment versus glass bottles consistently indicate that although chlorine decay rates are higher in the pipe environment, THM formation is also higher. This increase in THM levels is due to the reservoir of organic precursor materials associated with deposits on the pipe wall (AwwaRF, 2006). The fact chlorine decay rates are inversely related to pipe diameter is suggestive of differences between small and large diameter pipes on THM formation, although a conclusive relationship is unknown at this point. Similarly, it is possible that hydraulic conditions affect THM fate in distribution systems, but information on these effects is unknown. Stagnation in pipes increases disinfectant consumption and increases biomass growth, while high flows increase nutrient availability at the pipe wall, disinfectant transport and biofilm detachment.

2.6.3 Water Storage Tanks

Water storage tanks can serve a number of different purposes on a distribution system including: storage of water to meet peak demand and fire flows, creating pressure in the distribution mains if elevated, and providing contact time for chlorine to inactivate pathogens.

There are approximately 75 public water supplies with water storage tanks in the province. The majority of existing tanks can be classified as either standpipe or on ground, share the same draw/fill main, and are pressure controlled. Most storage tanks in the province have problems with poor mixing of water and dead zones.

Lack of water turnover in storage facilities has long been recognized as a primary cause of water quality problems within a distribution system. Disinfectants have more time to react with compounds in the bulk water in storage tanks with dead zones, low water turnover rates or poor circulation. These effects can generally be reduced by proper design and operation of storage facilities, such as appropriate tank sizing, inlet/outlet configuration, mixing and operational schedule.

2.6.4 Hydraulic Conditions

Hydraulic conditions such as water velocity, water age, system pressure and water demand, can have a major effect on water quality in the distribution system. Many water distribution systems in the province were designed with excess capacity to accommodate future population growth or industrial (fish plant) demand. In order to fit fire hydrants, larger pipe sizes are also required than might otherwise be needed.

The residence time of water in a system can play an important factor in the fate of substances in the water distribution system. Oversized pipes cause excessive retention time and the potential for water quality to degrade. As water ages, the chlorine residual decays, bacterial growth increases, DBP formation can occur, and contaminants from the distribution system (pipes, household lines, and fixtures) can potentially leach into the water. Long residence times (greater than 3 days) or large amounts of distribution system storage (greater than 2 days) were observed to increase the chance of a coliform incidence in a study done by the AwwaRF (2003). Organic carbon levels have been shown to decrease as water moves through the distribution system, although not universally.

The hydraulic residence time (HRT) can be calculated by dividing the pipe length by the average water velocity through the pipe or by dividing the volume of water in a pipe length by the flow of water through the pipe as indicated:

$$HRT = \frac{Length_{pipe}(m)}{velocity_{water}(m/s)} \text{ or } HRT = \frac{Volume_{pipe}(m^3)}{Flow_{water}(m^3/s)}$$

Equation 2:Hydraulic residence time

Increased flow velocities can cause shearing of biofilms from the pipe surface, expose the biofilm to increased nutrient levels, and provide greater transport of disinfectants. Stagnant water can cause loss of disinfectant residual and accumulation of sediment, which may lead to microbial growth. Dead end lines have typically shown significant deterioration in microbial water quality. Large changes in water velocity (eg. water hammer) have been observed to increase bacterial levels in pipe systems. Such changes can occur due to pipe network design and pipe size, water main breaks, and distribution system maintenance practices such as flushing.

2.7 Operation and Maintenance

Operation and maintenance requirements for the proper management of water distribution systems is outlined in the *Guidelines for the Design, Construction and Operation of Water and Sewerage Systems* and in the Permit to Operate issued to each community.

Proper operation and maintenance of distribution systems is essential in the control of THMs, and includes such activities as managing chlorine dosage, monitoring chlorine residuals throughout the distribution system, reservoir cleaning, regularly flushing the system, cleaning tanks and maintaining other infrastructure, monitoring pump usage and meter readings, detecting and fixing leaks, etc. These best management practices (BMPs) are recommended to maintain drinking water quality in the distribution system and to extend the life of water system infrastructure.

2.7.1 Blending

A number of communities throughout the province are currently mixing water sources in order to provide sufficient quantity to users. Most common is the mixing of surface with groundwater sources. Combining surface and groundwater can be problematic as different chemical scales and biofilms will form with the new environment formed in the drinking water distribution system. The combination of minerals, mostly from groundwater, and natural organic material, mostly from surface water, can also affect biofilms, corrosion, disinfectant residuals and DBP formation.

3.0 Characteristics of Disinfection by Products

Drinking water is disinfected in order to reduce the risk of pathogenic infection (bacteria, viruses, protozoa) to human health. However, disinfection residues and their by-products may also pose a chemical threat to human health with the presence of organic and inorganic precursors in the water. Since the 1970s, more than 250 DBPs have been identified, but the behavioural profile of only approximately 20 is adequately known (Sadiq et al., 2003). DBP formation involves either halogen substitution and/or oxidation reactions.

[Precursor Material] + [Disinfectant] = [Disinfectant by-product or DBP]

Equation 3: Production of DBPs

When disinfectant is added to water it then reacts with substances already present in the raw water such as:

- Organic substances (humic and fulvic acids, polymeric carbohydrates, polysaccharides, proteins, carboxylic acids, ketones, low molecular weight acids)
- Algae and aquatic plants
- Bromide ion, iodide ion
- Inorganic reducing agents
- Ammonia
- Amino-nitrogen groups

Other factors that play a significant role in the formation of DBPs include:

- concentration and chemical properties of precursors
- water temperature
- pH
- disinfectant type, dose and residual
- contact time

The following is a list of various identified disinfection by-products:

- Trihalomethanes (THMs)
- Haloacetic acids (HAAs)
- Haloacetonitriles (HAN)
- Inorganic compounds: bromate, chlorate, chlorite, iodate, etc.
- Halogenated aldehydes and ketones (HKs)
- Halophenols
- Chloropicrin
- Chloral hydrate
- Cyanogen chloride
- Chlorophenols
- N-organochloramines
- N-nitrosodiumethylamine (NDMA)

- Organic acids
- Ketones
- Epoxides
- Peroxides
- Quinones
- AOC (aldehydes, carboxylic acids, etc.)
- Others

THMs have been identified as the largest class of DBPs detected on a weight basis in chlorinated finished water, with HAAs being the second largest (Singer, 1999). These two groups can be used as indicators for the presence of all chlorinated disinfection by-products in drinking water supplies, and actions taken in their control are expected to reduce the levels of all chlorinated by-products and their corresponding risks to health.

3.1 Pathogenic and Chemical Risk Analysis

There is a risk associated with drinking untreated water of acute illness, chronic illness and even death due to pathogenic contamination. Water disinfection can reduce and/ or eliminate such risk by inactivating the pathogens that can cause such illness. While reducing this major risk, water disinfection introduces a chemical exposure risk in the form of DBPs. A risk trade-off analysis between pathogenic and chemical risks becomes necessary as depicted in the following figure (LaVerda, 2001).

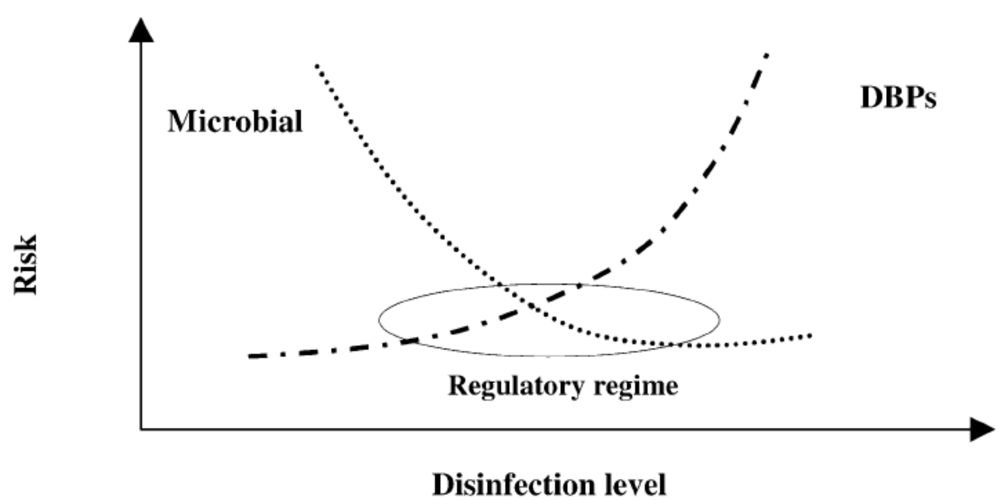


Figure 7: Risk trade-off analysis between pathogens and DBPs

In practice, such a trade off constitutes a major challenge both in terms of general acceptance by the public at large, and due to operational and financial constraints. The province in the form of Guidelines for Canadian Drinking Water Quality has already established acceptable levels of risk for both pathogenic and chemical agents. The general consensus amongst water quality and health experts is that the risk posed by consuming water that hasn't been disinfected is much greater than that of consuming disinfected water containing DBPs.

3.1.1 Health Effects of Common Pathogens

Adverse health effects of some common pathogens contaminating drinking water are summarized in the following table. It is in order to reduce the risk of pathogenic infection to human health that disinfection of drinking water is practiced. Disinfection of drinking water has helped prevent untold death and illness over the past hundred years. Health effects caused by common pathogens in drinking water are typically deemed acute. The young, old and immuno-suppressed individuals are particularly vulnerable to pathogens in drinking water, which in some cases can lead to death.

Table 3: Adverse health effects of common pathogens in drinking water

Contaminant	Potential Health Effect from Ingestion of Water
Chyriptosporidium	Gastrointestinal illness (e.g., diarrhea, vomiting, cramps)
Giardia	Gastrointestinal illness (e.g., diarrhea, vomiting, cramps)
Legionella	Legionnaire's Disease, a type of pneumonia
Viruses (enteric)	Gastrointestinal illness (e.g., diarrhea, vomiting, cramps)
E.Coli (bacteria)	diarrhea and abdominal cramps, hemolytic uremic syndrome
Hepatitis A (virus)	Gastrointestinal illness (e.g., diarrhea, vomiting, cramps)
Norwalk (virus)	Gastrointestinal illness (e.g., diarrhea, vomiting, cramps)
Toxoplasma (protozoa)	Developmental effects, personality changes
Camphlobacter (bacteria)	Gastrointestinal illness (e.g., diarrhea, vomiting, cramps, fever)

3.1.2 Toxicological Effects of Common DBPs

Adverse health effects of some of the more important DBPs are summarized in the following table. Many of these compounds are suspected carcinogens and as a result, tight regulated limits have been imposed on their concentration in drinking water. Most of the observed toxicological effects have been from tests performed on laboratory animals (rodents, dogs). Similar effects are expected in humans. Health effects caused by common DBPs in drinking water are typically deemed chronic as a result of long term exposure (up to 70 years).

Table 4: Toxicological effects for DBPs

Class of DBPs	Compound	Rating	Effects
THM	Chloroform	B2	Cancer, liver, kidney and reproductive effects
	Dibromochloromethane	C	Nervous system, liver, kidney and reproductive effects
	Bromodichloromethane	B2	Cancer, liver, kidney and reproductive effects
	Bromoform	B2	Cancer, nervous system, liver and kidney effects
HAN	Trichloroacetonirile	C	Cancer, mutegenic and clastogenic effects
Halogenated aldehydes and ketones	Formaldehyde	B1	Mutagenic
Halophenol	2-Chlorophenol	D	Cancer, tumour promoter

HAA	Dichloroacetic acid	B2	Cancer, reproductive and developmental effects
	Trichloracetic acid	C	Liver, kidney, spleen and developmental effects
Inorganic compounds	Bromate	B2	Cancer
	Chlorite	D	Developmental and reproductive effects

A: Human carcinogen; B1: Probable human carcinogen (with some epidemiological evidence); B2: Probable human carcinogen (sufficient laboratory evidence); C: Possible human carcinogen; D: Non classifiable.

Epidemiological studies relating consumption of chlorinated water and cancer at specific organ sites have displayed mixed results, suggesting it may be too early to conclude that a causal relationship exists. Studies involving bladder cancer have provided the most consistent results. Similarly, epidemiological studies relating THM exposure to reproductive effects (stillbirth, spontaneous abortion, fetal growth, birth defects) have proven inconclusive, indicating the need for more data and further study (AwwaRF, 2006).

Pathways for THM exposure from municipal tap water include:

- Ingestion- drinking water, beverages made with tap water, food prepared using tap water
- Inhalation- showering, cooking food
- Dermal adsorption- bathing, swimming in pools, washing dishes, washing children

Studies indicate that inhalation dominates the absorbed dose estimates for THMs. The contribution of the ingestion and dermal routes are similar, however, it is unclear which route provides the most exposure (AwwaRF, 2006). Other studies indicate that activities associated with inhaled or dermal exposure routes result in a greater increase in blood THM concentration than does ingestion. It is generally assumed that a large proportion of THMs present in drinking water are transferred to air as a result of their volatility.

3.1.3 Relevant Drinking Water Quality Guidelines

Guidelines for any water quality contaminant are based on evidence of their adverse human health effects as determined by toxicological studies. The following table summarizes relevant drinking water quality guidelines recommended by the World Health Organization (WHO), US Environmental Protection Agency (EPA) and Health Canada. The Health Canada guideline is based on the Canadian Council of Ministers of the Environment (CCME) GCDWQ and are used as drinking water standards in Newfoundland and Labrador.

Table 5: Drinking water quality guidelines (in mg/L unless otherwise stated)

Compound	WHO (2004)	US EPA (2003)	CCME/ Health Canada (2008)
Total THMs		0.080	0.100
Bromodichloromethane	0.060		0.016

Chloroform	0.200		
Bromoform	0.100		
Dibromochloromethane	0.100		
HAA5		0.060	0.080
Chlorite	0.700	1.000	1.000
Chlorate			1.000
Bromate	0.010	0.010	0.010
Chlorine	5.0	4.0	
Chloramines		4.0	
Chlorine dioxide		0.800	
E. coli			0 per 100 mL
Cryptosporidium		zero	
Giardia lamblia		zero	
Total coliforms		zero	0 per 100 mL
Protazoa			None
Enteric viruses		zero	None
Turbidity			1.0 NTU
Colour		15 TCU	15 TCU (aesthetic)
pH		6.5-8.5 (no units)	6.5-8.5 (no units)

Compliance with THM and HAA guidelines is based on a locational annual running average of quarterly samples. Measurement of DBP concentration in drinking water usually requires gas chromatography analysis, which is a time consuming and relatively expensive technique.

3.2 Trihalomethanes

Trihalomethanes are volatile substances defined as halogenated methane compounds that form during chlorination of waters containing naturally occurring organics (DOEC, 2000). Trihalomethanes are single-carbon organics with three of the carbon bonds being occupied by halogens such as chlorine, bromine or iodine. THMs are known as terminal DBPs because they are the final compounds created after a series of intermediary reactions which can form reaction intermediate DBPs (eg. HANs, HKs). Trihalomethanes are rarely found in raw water. THMs are formed when chlorine reacts with natural organic matter (NOM) and/ or inorganic substances present in the raw water as follows:

[Natural Organic Matter] + [Chlorine] = [Trihalomethanes or THMs]

[Natural Organic Matter] + [Bromide ion] = [Trihalomethanes or THMs]

[Natural Organic Matter] + [Chlorine] + [Bromide ion] = [Trihalomethanes or THMs]

Equation 4: Production of THMs

NOM is a mixture of humic and non-humic substances that contribute to DBP precursor levels in drinking water. Humic substances serve as the most important DBP precursor, with low molecular weight acids serving as biodegradable organic matter within a distribution system (AwwaRF, 2006). The most problematic of the humic substances are aromatic organics– humic acids such as tannins. Aromatic organics are more reactive than other organics having a double bond ring structure that results in free electrons that are readily available to react with other molecules. Non-aromatic (aliphatic) organics, such as fulvic acids, tend to be less reactive. Elemental composition of humic and fulvic acides includes, in order of predominance: carbon, oxygen, hydrogen, nitrogen, sulphur and phosphorous.

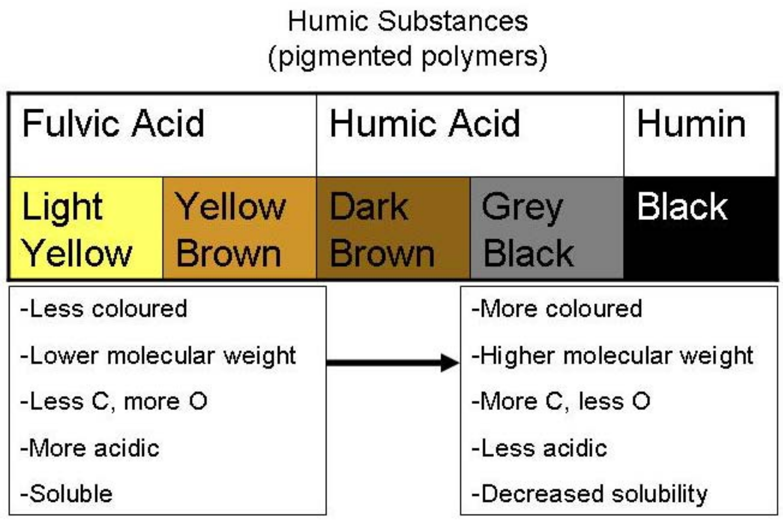


Figure 8: Chemical properties of humic substances

NOM is typically measured by surrogate parameters such as TOC, DOC, UV absorbance or fluorescence. DOC is indicative of the mass of material, UV absorbance is indicative of the NOMs aromatic character, and fluorescence has been correlated with molecular weight. All of these surrogates display high variations of intensity versus time, or seasonal variation. Sophisticated testing methods such as chromatography and mass spectrometry allow for identification of the individual compounds that compromise NOM. These techniques are more applicable to molecules with high molecular weight or strong ionic character and usually only allow for identification of between 5-15 percent of dissolved organic compounds. NOM is an exceedingly complex, potentially unresolvably complex, mixture. The more poorly defined refractory, non-chromatographable fraction is often referred to as humic or fulvic acids or humic substances. Non-humic substances also make up a fraction of NOM. NOM generally include the presence of highly condensed polyhydroxy-aromatics, proteins, amino-sugars, carbohydrates, polysaccharides, carboxylic and phenolic acid groups, amino-acids, and hydrophilic acids (AwwaRF, 1994). Both humic and non-humic NOM and both lower and higher molecular weight NOM can for DBPs, however it is difficult to distinguish which NOM fraction in waters is the most problematic.

In addition to humic substances, algae can be a source of DBP precursors. Algae, both their biomass and extracellular products, reach readily with chlorine to produce THMs. There is some evidence that the extracellular products, on reaction with chlorine, generally produce greater quantities of THMs. It was further observed that high-yielding

THM precursors were liberated by algae in greater abundance during the late exponential phase of growth than at any other time during the algae life cycle (Singer, 1999).

There are currently four major and regulated THM compounds including:

- Chloroform [$CHCl_3$]
- Bromoform [$CHBr_3$]
- Dibromochloromethane (DBCM) [$CHBr_2Cl$]
- Bromodichloromethane (BDCM) [$CHBrCl_2$]

Chlorine, the main disinfectant used in water treatment, reacts with organic compounds through three important routs: by oxidation of reduced functions, by addition onto unsaturated carbons, or by electrophilic substitution on nucleophilic sites. At relatively low chlorine doses, substitution reactions dominate; at high chlorine doses oxidation and cleavage reactions tend to dominate (AwwaRF, 2006).

Figure 8 provides a percentage breakdown of the average composition of total THMs in the province by individual THM species. The average percentages were derived from over 6000 THM samples collected over the period from 2000 to 2005. Chloroform contributes most to THM totals followed by bromodichloromethane. Bromine-containing DBP species are known to form faster than the chlorinated species, and as such, the fraction of chlorine-containing THMs should increase with increasing water age in the distribution system (AwwaRF, 2006). Also the rate of THM formation is higher in waters with increased concentrations of bromide (Singer, 1999).

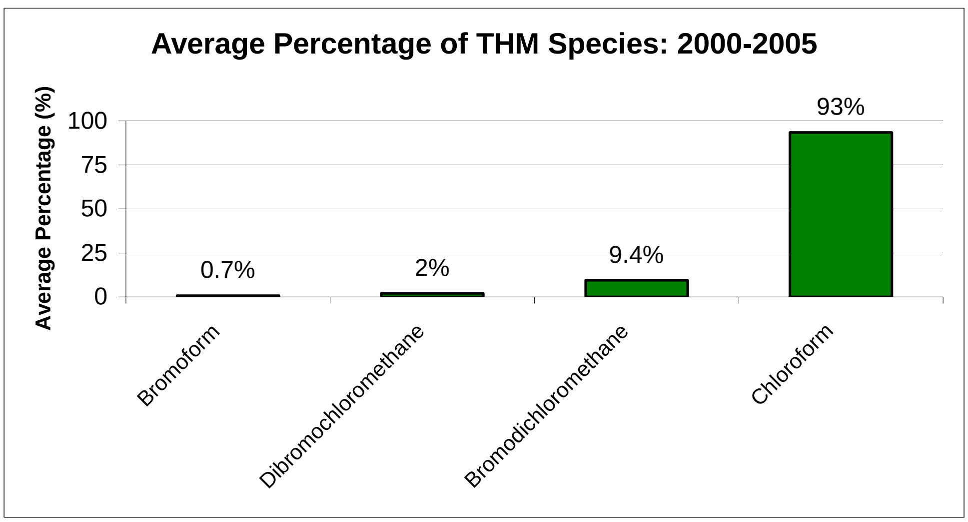


Figure 9: Average makeup of total THMs- 2000 to 2005

Iodine present in the water can also form a class of THMs known as iodomethanes, but these are currently not included in the total THM count. In most circumstances, chloroform is the dominant compound. Bromine-containing compounds may be of greater health concern than their fully chlorinated counterparts, and research suggests that a number of these iodine-containing THM species are more hazardous than those containing only chlorine and bromide.

When chlorine is added to water, either in gas or liquid form, it reacts with water to form two weak acids: hypochlorous acid (HOCl) and hydrochloric acid (HCl).

$$Cl2 + H2O \rightarrow HClO + HCl$$

Equation 5: Production of hypochlorous acid and hydrochloric acid

Hypochlorous acid (HOCl) is a weak acid and dissociates to form hypochlorite (OCl^-) and hydrogen ion (H^+) as shown:

$$HOCl \rightarrow OCl^- + H^+$$

Equation 6: Production of hypochlorite and hydrogen ion

Hypochlorous acid and hypochlorate ions are collectively known as free available chlorine, and are strong oxidants.

The HOCl can also undergo subsequent reactions resulting in the formation of THMs. It oxidizes the bromide (Br^-) present in the water to form hypobromous acid, which reacts readily with NOM to form brominated THMs.

Many factors influence the rate and degree of THM formation including:

- Chlorine dose
- Concentration and nature of NOM (mainly humic subsantces)
- Water residence time in the distribution system
- pH
- Temperature
- Bromide ion concentration
- Inorganic chlorine demand

Typically, higher THM concentrations are expected at higher levels of the above listed parameters. Studies have shown that higher disinfectant dose (as a result of higher chlorine demand) will increase THM formation potential in water. Longer retention time in the distribution system, and therefore reaction time, generally leads to higher consumption of residual chlorine and results in more THM formation as indicated in Figure 10. In general, THM formation increases with an increase in pH. Temperature has a positive effect on THM formation potential and increases the rate of reaction. High bromide levels contribute to the formation of brominated THMs in the presence of high NOM and chlorine.

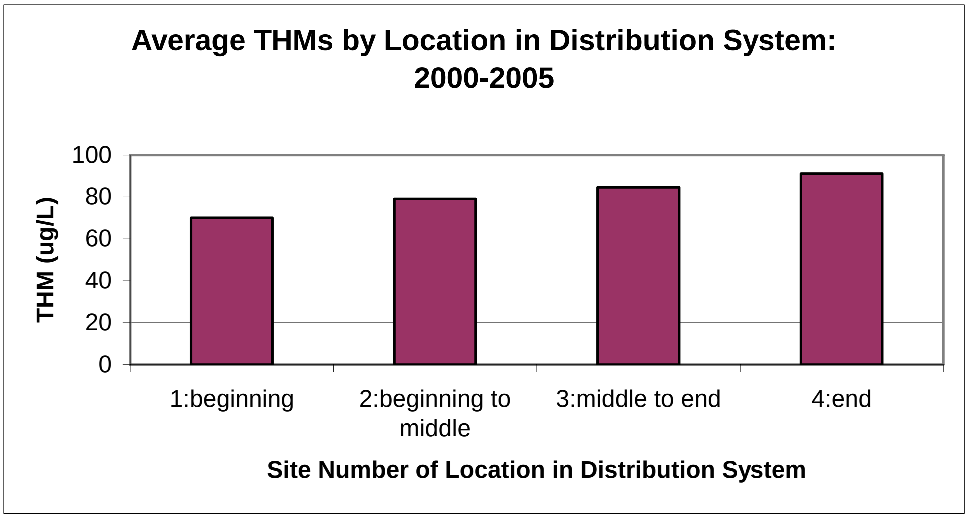


Figure 10: Average total THMs based on sample site location in the distribution system- 2000 to 2005

More recent research suggests that some chlorinated DBPs (including HAAs and HANs) may actually degrade in extremities of distribution systems. Degradation of DBPs in distribution systems can be caused by chemical degradation (instability of the compounds themselves), DBP hydrolysis at specific pH values, interaction with corrosion by-products, adsorption in the biofilm, biodegradation and bioaccumulation by microorganisms. Decreases in THMs in the distribution system, however, are not common.

Seasonal variations in water quality can have a significant impact on drinking water quality and the formation of DBPs. Typically, peak THM levels for any given community are expected in the summer season. The following graph indicates seasonal THM averages from across the province from 2000 to 2005. Fall is the season with the highest THM average, while winter has the lowest. Increases in DOC levels due to decaying organic matter (leaves, etc.) is most likely contributing to the higher THM levels in the fall, and seems to play a more significant role in THM formation than high water temperatures which peak in the summer.

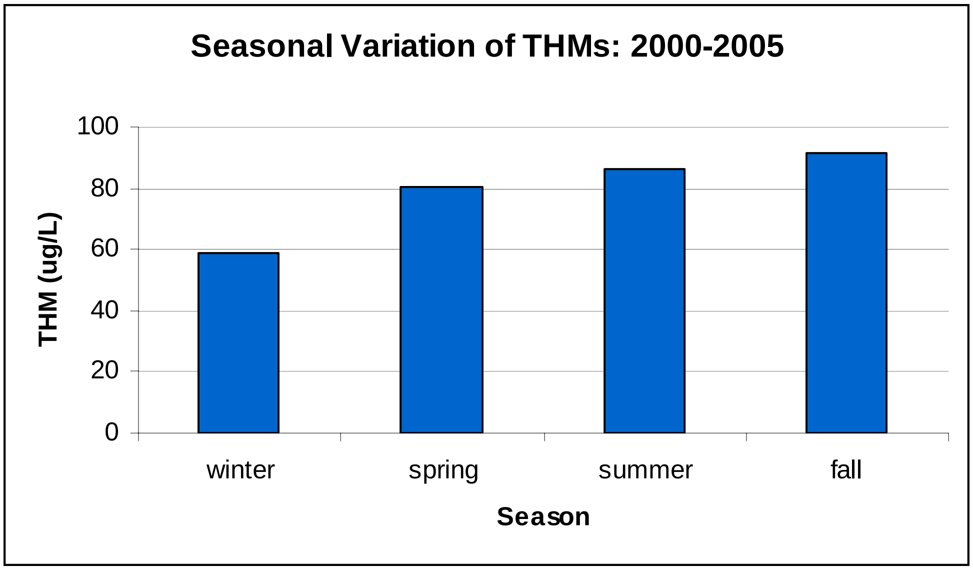


Figure 11: Seasonal variation in average THM levels- 2000 to 2005

In temperate climates, such as in Newfoundland and Labrador, THM levels in drinking water are significantly affected by seasonal conditions. In winter when ice cover protects surface water sources, THM concentrations are generally lower due to lower water temperature and NOM. Under these conditions, chlorine demand is reduced, and the chlorine dose required to maintain an adequate residual in the distribution system is also reduced. High THM concentrations have been observed, particularly in the extremities of water distribution systems in the summer months. Reaction kinetics are higher at warmer temperatures. Below 10 °C, THM concentrations in distribution systems do not increase significantly. Heavy rainfall events, typical in the spring and fall, also have an effect on NOM and bromide concentrations. THM concentrations have also been shown to vary significantly over the course of a day.

Based on over 6000 THM samples collected in the province from 2000 to 2005, THM concentrations from systems across the province ranged from 0-708 µg/L, with a median value of 57.4 µg/L.

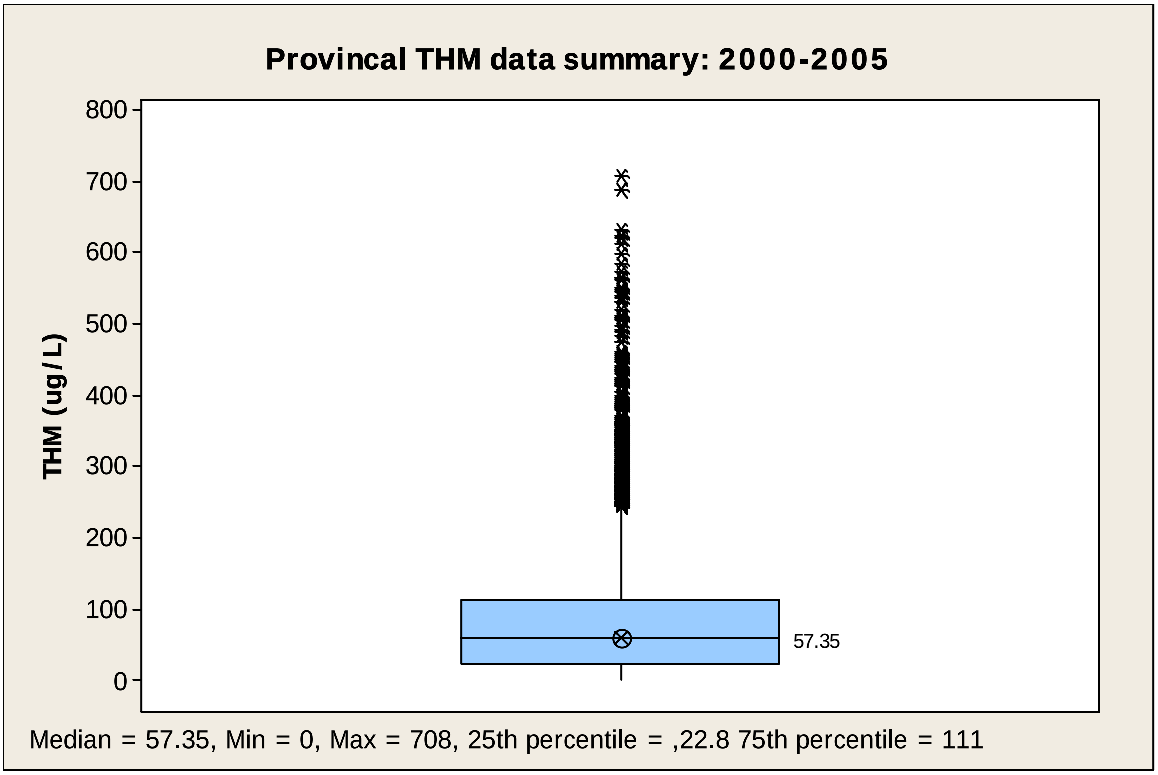


Figure 12: Summary of provincial THM data- 2000 to 2005

Increasing the total chlorine dose, at the main chlorinator or at booster points in the distribution system will increase the final concentrations of THMs. Increased THMs caused by increased chlorine dosages can be estimated from the following equation (AwwaRF, 2006):

$$[THM]_1 = \frac{\left([Cl_2]_1\right)^{0.5}}{\left([Cl_2]_0\right)^{0.5}} \times [THM]_0$$

Equation 7: Calculating THMs based on chlorine dose

where:

$[THM]_1$ = THM concentration given by new chlorine dose
$[Cl_2]_1$ = new chlorine dose current
$[Cl_2]_0$ = initial chlorine dose
$[THM]_0$ = THM concentration given by initial chlorine dose

Using Equation 7, ideal chlorine doses that would maintain THMs at below guideline levels were calculated for several communities. The maximum observed THM reading observed in that community was used along with the most current information on main chlorine dosage. For the majority of communities the calculated ideal chlorine dose is unrealistically low.

Table 6: Required chlorine dose to maintain THMs below 100 µg/L

Community	Current Chlorine Dose	Current Maximum Observed THMs	Future THMs	Required New Chlorine Dose

	(mg/L)	(µg/L)	(µg/L)	(mg/L)
Hawke's Bay	5.10	116	100	3.8
Brighton	6.28	300	100	0.70
Cartwright	4.90	270	100	0.67
Ferryland	6.48	249	100	1.05
Burlington	12.2	214	100	2.66
St. Paul's	12.6	309	100	1.32

Typical chlorine dosages for small water distribution systems in Newfoundland and Labrador range between 5 and 15 mg/L.

3.2.1 Communities with THM Issues

The extent of the THM issue can be judged by looking at the number of communities with THM levels above the guideline. Under the GCDWQ, an annual running average of 100 µg/L is considered the maximum acceptable concentration (MAC) for THMs. Looking at tap water quality data from the period 2003-2006, the total population impacted by THM exceedances has remained fairly constant over the period of interest averaging at 24%. In total there are 124 communities with THM issues in the province; 42 communities have major THM issues, 48 moderate THM issues, and 34 minor THM issues (DOEC, 2008). In the analysis of THM exceedances summarized in the following table, individual exceedances greater than 100 µg/L are considered, not the annual running average. THM exceedances are broken down into the following descriptive categories:

- Minor– exceedances are detected but average is generally less than 120 ug/L
- Moderate– exceedance average is generally between 120 and 150 ug/L with individual levels not exceeding approximately 200 ug/L
- Major– exceedance averages above 150 ug/L or individual samples exceeding 200 ug/L

Table 7: Communities with THM issues

Major THM Issues	Moderate THM Issues	Minor THM Issues
Birchy Bay	Avondale	Aquaforte
Black Tickle-Domino	Baie Verte	Arnold's Cove
Bonavista	Bauline	Beachside
Brighton	Bird Cove (+Brig Bay)	Bellburns
Burgeo	Come By Chance	Campbellton
Cape Freels North	Cook's Harbour	Carmanville
Cartwright	Corner Brook	Channel-Port Aux Basques
Clarenville	Cow Head	Clarenville
Corner Brook	Crow Head	Comfort Cove-Newstead
Cottlesville	Fairbanks-Hillgrade	Conne River
Ferryland	Flower's Cove	Embree
Gander	Fox Roost-Margaree	Gaskiers
Garnish	Harbour Breton	Gillams

Howley	Harbour Main-Chapel's Cove-Lakeview	Hampden
Keels	Hawke's Bay	Happy Adventure
Lamaline	Hermitage	Herring Neck
Little Bay Islands	Joe Batt's Arm-Barr'd Islands-Shoal Bay	Irishtown-Summerside
Lourdes	Leading Tickles	Isle aux Morts
Mary's Harbour	Lewisporte	Little Burnt Bay
McCallum	Little Bay	Mount Moriah
New-Wes-Valley	Little Catalina	Musgrave Harbour
Pidgeon Cove-St. Barbe	Loon Bay	Nameless Cove
Pleasantview	Lushes Bight-Beaumont-Beaumont North	Placentia
Port Hope Simpson	Main Brook	Point Leamington
Port Saunders	Makkovik	Port au Choix
Pouch Cove	Marystown	Port au Port West-Aguathuna-Felix Cove
Purcell's Harbour	Massey Drive	Port Albert
Ramea	Merritt's Harbour	Seal Cove (WB)
Rigolet	Millertown	Seldom-Little Seldom
Salvage	Milltown-Head of Bay D'Espoir	Shoe Cove
Smith's Harbour	Pasadena	St. Lunaire-Griquet
South River	Placentia	Torbay
St. George's	Point May	West St. Modeste
St. Lewis	Point of Bay	Gaultois
St. Pauls	Port au Choix	
Straitsview	Portugal Cove-St. Phillips	
Summerford	Postville	
Sunnyside (T.B.)	Queen's Cove	
Terrenceville	Rocky Harbour	
Tilting	St. Lunaire-Griquet	
West Bay	Tizzard's Harbour	
Wild Cove	Trinity Bay North	
	Triton	
	Twillingate	
	West Bay	
	Whiteway	
	Burlington	
	New-Wes-Valley	

3.2.2 Brominated THM Compounds

Bromide is part of the chemical makeup of three out of the four THM compounds currently regulated. THM data from Nov 1998 to Nov 2005 indicated that bromodichloromethane levels (one bromide ion) were higher than dibromochloromethane levels (two bromide ions), which were in turn higher than bromoform levels (three bromide ions). This indicates the ease of formation based on the number of bromide ions from which the compound is composed. As the bromide ion concentration in water increases, the speciation within the individual DBP classes shifts toward the bromine substituted compounds. The two main reactions involved with brominated DBPs are as follows:

$$HOCl + Br \rightarrow HOBr + Cl$$

$$HOCl + HOBr + NOM \rightarrow DBPs$$

Equation 8: Reactions involved with brominated DBPs

The extent of the BDCM issue can be judged by looking at the number of communities with BDCM levels above the guideline. The GCDWQ maximum acceptable concentration (MAC) for BDCMs is 16 µg/L which only came into effect in 2006. Looking at tap water quality data from the period 2003-2006, the total population impacted by BDCM exceedances has remained fairly constant over the period of interest averaging at 2.6%. In total there are 45 communities with BDCM issues in the province; 8 communities have major BDCM issues, 5 moderate BDCM issues, and 32 minor BDCM issues (DOEC, 2008). BDCM exceedances are broken down into the following descriptive categories:

- Minor– average BDCM levels do not exceed 16 ug/L and their were few individual exceedances
- Moderate– average BDCM level is just above the 16 ug/L level but BDCM exceedances are not on a consistent basis
- Major– BDCM levels are consistently above the 16 ug/L limit and the average BDCM level may be well above the limit

Table 8: Communities with BDCM issues

Major BDCM Issues	Moderate BDCM Issues	Minor BDCM Issues
Black Tickle-Domino	Brighton	Avondale
Cook's Harbour	Herring Neck	Bellburns
Crow Head	Port au Port West-Aguathuna-Felix Cove	Bonavista
Lourdes	Tilting	Burin
Parson's Pond	West Bay	Cavendish
Pidgeon Cove-St. Barbe		Cottlesville
Ramea		Cow Head
St. Pauls		Fairbanks-Hillgrade
		Garnish
		Grand Bank
		Heart's Delight-Islington
		Joe Batt's Arm-Barr'd Islands-Shoal Bay
		Keels
		Lamaline
		Little Bay Islands
		Merritt's Harbour
		Norris Point
		Piccadilly Head
		Pleasantview
		Port au Choix
		Port au Port West-Aguathuna-Felix Cove
		Port Albert
		Port Saunders
		Portland Creek
		Portugal Cove-St. Phillips

		Pouch Cove Purcell's Harbour St. Lunaire-Griquet Straitsview Summerford Sunnyside (T.B.) Whiteway

In order to better understand the source of BDCM issues, several communities were examined in more detail. Table 9 is a list of communities with BDCM levels consistently over the GCDWQ of 16 ug/L, looking at the entire period of record available in some cases going back to 1988. Prior to March 31, 2004, bromide results that were less than the detection limit were reported as half the detection limit. After this date, bromide values that were under the detection limit were assigned a zero value making the bromide dataset highly censored.

Table 9: Source of BDCM issues

Community	Region	Max BDCM value (ug/L)	Average Bromide in Source Water (mg/L)	Probable Bromide Source	Notes
Ramea	W	176	0.20	ocean saltwater spray, geological source, saltwater intrusion	Intake 200 m (W) to coast, BDCM > Chloroform occasionally
Bellburns (Northern Pen)	W	33.0	0.03	geological source, source spring influenced, ocean saltwater spray	Intake 600 m (W) to coast
Bonavista	E	21.1	0.03	ocean saltwater spray	Intake 3 km (W) to coast
Brighton	C	29.0	0.03	ocean saltwater spray	Intake 300 m (NW) to coast
Cooks Harbour (Northern Pen)	W	42.1	0.09	ocean saltwater spray, geological source, saltwater intrusion	Intake 300 m (NW) to coast
Cow Head (Northern Pen)	W	30.0	0.03	ocean saltwater spray	Intake 1.2 km (NW) to coast
Crow Head (New World Island)	C	59.0	0.09	ocean saltwater spray, geological source, saltwater intrusion	Intake 700 m (NW) to coast
Garnish (Burin Pen)	E	18.4	0.02	ocean saltwater spray	Intake 2.5 km (NW) to coast

Herring Neck, Hatchet Harbour, Salt Harbour, Shoal Cove, Sunnyside (New World Island)	C	26.3	0.09	ocean saltwater spray, geological source, saltwater intrusion	Intake 200 (N) to coast
Joe Batt's Arm-Barr'd Islands-Shoal Bay (Fogo Island)	C	26.0	0.02	ocean saltwater spray, geological source, saltwater intrusion	Intake 900 m (W) to coast
Lourdes, West Bay (Port au Port)	W	66.0	0.03	geological source, ocean saltwater spray	Intake 900 m (E) to coast, BDCM > Chloroform occasionally
Merritt's Harbour (New World Island)	C	34.2	0.02	ocean saltwater spray, geological source, saltwater intrusion	Intake 600 m (NW) to coast
Parson's Pond (Northern Pen)	W	40.2	0.17	geological source, source spring influenced	Intake 600 m (W) to coast, BDCM > Chloroform typically
Port au Bras (Burin Pen)	E	26.7	0.03	ocean saltwater spray	Intake 400 m (S) to coast
Port au Choix (Northern Pen)	W	35.0	0.026	groundwater influence from mixing with wellfield water in distribution system	Intake approx 1.1 km (S) to coast
Port au Port West (Port au Port)	W	32.0	0.05	groundwater influence from mixing with well water in distribution system	Intake 400 m (S) to coast
Port Saunders (Northern Pen)	W	25.6	0.03	ocean saltwater spray, geological source, saltwater intrusion	Intake 800 m (W) to coast
Purcell's Harbour (New World Island)	C	38.6	0.026	ocean saltwater spray, saltwater intrusion	Intake 600 m (E) to coast
St. Paul's (Northern Pen)	W	36.0	0.03	ocean saltwater spray	Intake 1.1 km (NW) to coast

Summerford, Cottlesville (New World Island)	C	20.3	0.015	ocean saltwater spray, geological source, saltwater intrusion	Intake 1.3 km (NW) to coast
Tilting (Fogo Island)	C	53.7	0.025	ocean saltwater spray, geological source, saltwater intrusion	Intake 200 m (NE) to coast

Several cluster areas of high BDCMs were identified including the Fogo Island, New World Island, the Northern Peninsula, the Burin Peninsula, and the Port au Port Peninsula. Communities with higher average bromide values tended to have higher BDCM values. Communities where all three bromomethanes (bromoform, dibromochloromethane, bromodichloromethane) make up a significant portion of the total THM value include:

- Cooks Harbour
- Crow Head
- Lourdes
- Parson's Pond
- Port au Port West
- Ramea

Figure 12 illustrates how close is the separation between the freshwater supply of Northwest Pond in Ramea and the ocean. Ramea has the highest observed BDCMs in the province.

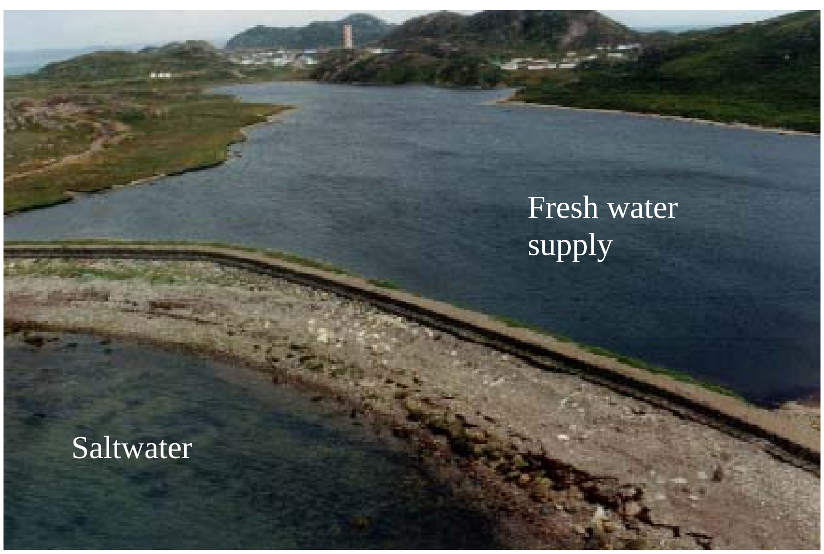


Figure 13: Separation between freshwater and saltwater on the Ramea source water supply

Bromide levels can change from day to day because of variations in the raw water. For example, the bromide level in the Ohio River can be reduced by as much as 50% after a significant runoff event. Bromide is not affected by conventional water treatment yet will significantly affect both the rate and ultimate formation of total THMs, in addition to affecting the distribution of the four individual THM species. When NOM decreases in the presence of bromide ions, an observed shift takes place towards formation of more highly brominated THM species (AwwaRF, 1994).

3.3 Haloacetic Acids

Haloacetic acids or HAAs are a family of organic compounds based on the acetic acid molecule (CH3COOH) where one or more hydrogen atoms attached to carbon atoms are replaced by a halogen (chlorine, bromine, fluorine and/or iodine). There are nine different species of HAAs, however, not all are regularly tested for. HAAs are colourless, have a low volatility, dissolve easily in water and are fairly stable. Exposure to haloacetic acids from drinking water through inhalation and skin contact is not considered significant.

HAAs form when chlorine reacts with natural organic matter and/or bromide ions in raw water supplies. The most commonly measured haloacetic acids include:

- Monochloroacetic acid (MCA) [$ClCH_2COOH$]
- Dichloroacetic acid (DCA) [$CHCl_2COOH$]
- Trichloroacetic acid (TCA) [CCl_3COOH]
- Monobromoacetic acid (MBA) [$BrCH_2COOH$]
- Dibromoacetic acid (DBA) [$Br_2CHCOOH$]
- Bromochloroacetic acid
- Bromodichloroacetic acid
- Chlorodibromoacetic acid
- Tribromoacetic acid

THMs are the predominant DBP formed, followed by HAAs when water is disinfected with chlorine. The HAAs present in the greatest concentrations are typically dichloroacetic and trichloroacetic acid. Looking at HAA data from 2001 to 2007 from across the province, the HAA species with the highest concentrations are trichloroacetic acid followed by dichloroacetic acid. Together they account for approximately 95 percent of total HAAs in the province. The rate of formation of TCA is significantly favoured by low pH (Health Canada, 2007). Bromine is more reactive than chlorine in reactions that form HAAs and the HAA speciation will also depend on the ratio of bromide to chlorine (Singer, 1999).

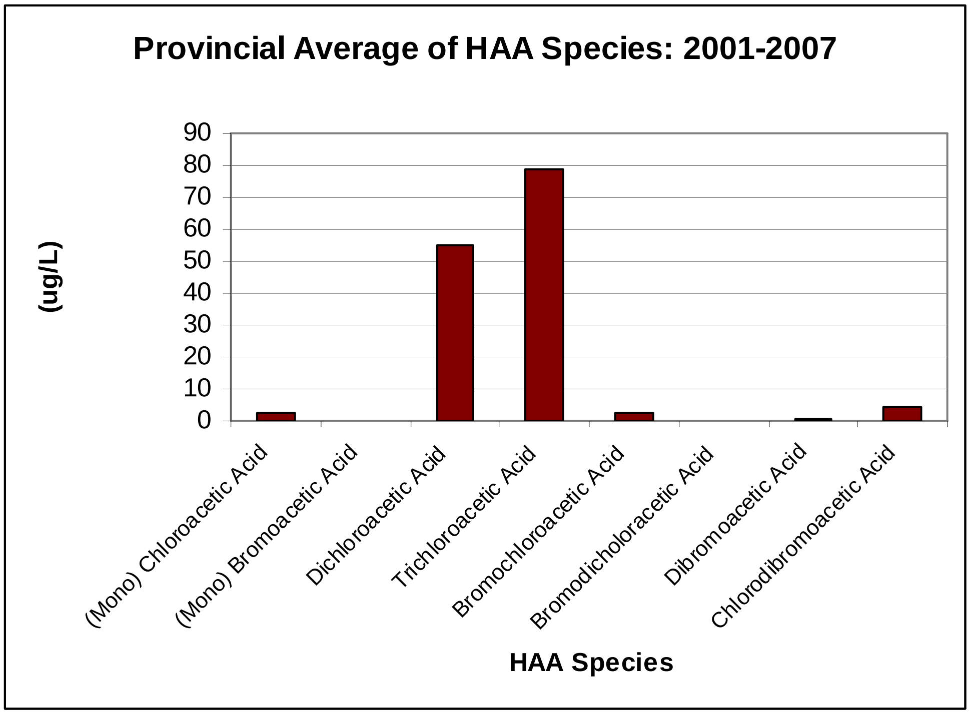


Figure 14: Provincial average of HAA species: 2001-2007

There is sufficient scientific data available to determine the health-based effects of several of the individual HAA species as follows:

- MCA- Group IV- unlikely to be carcinogenic to humans
- DCA- Group II- probably carcinogenic to humans
- TCA- Group III- possibly carcinogenic to humans
- MBA- Group VI- unclassifiable with respect to carcinogenicity in humans
- DBA- Group II- probably carcinogenic to humans

These five HAA species make up what is commonly referred to as HAA5. The two species that display the highest health risk, dicloroacetic acid and dibromoacetic acid, make up 39% and 0.3% of average provincial HAA totals respectively.

The process of HAA formation and decay differs somewhat from that of THMs in that HAAs are a group of acetic acids, they are more likely to be formed under low pH conditions, and peak levels observed in distribution systems do not occur at the point of maximum residence time as with THMs due to microbial decomposition in the network. HAA formation is similarly temperature dependant. More study is required to fully understand the dynamics of HAA formation potential in the province. The presence of brominated HAAs also depends on the presence of bromine in the source water.

Provincial HAA data prior to the spring of 2008 was collected in order to determine background levels in preparation for the selection of a HAA guideline by the CCME. Although all communities that are disinfecting drinking water have been sampled for

HAAs at least once, continued samples were only collected if levels indicated were of concern.

While THMs showed an average linear increase from the beginning to end of the distribution system, HAA levels in the province appeared to be greatest at the beginning to middle of the distribution system. This would appear to concur with other findings that HAAs do not peak at the end of the distribution system due to microbial and other degradation mechanisms in the network. However, the data set used in this analysis was censored with significantly more HAA samples collected at sites 2 and 3 (612 and 872) on the distribution system than at the beginning and end (31 and 56).

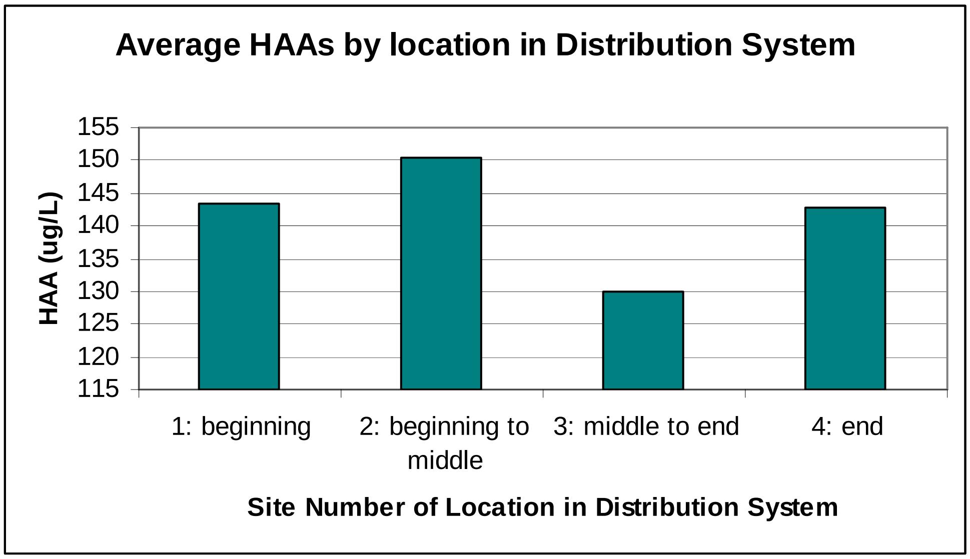


Figure 15: Average HAAs by location in distribution system: 2001-2007

For individual communities with samples collected from multiple locations on the distribution system, the trend in HAA levels throughout the distribution system is similar to that observed for the provincial HAA averages. The majority of community systems examined with multiple HAA samples from different locations in the distribution system typically indicated declining HAA levels with distance travelled through the network.

The seasonal averages in HAAs in the province followed a similar pattern to that of THMs with HAAs increasing throughout the year and peaking in the fall. The lowest average HAAs were observed in the spring rather than winter, however. Sample size was roughly even across all seasons (320 to 518).

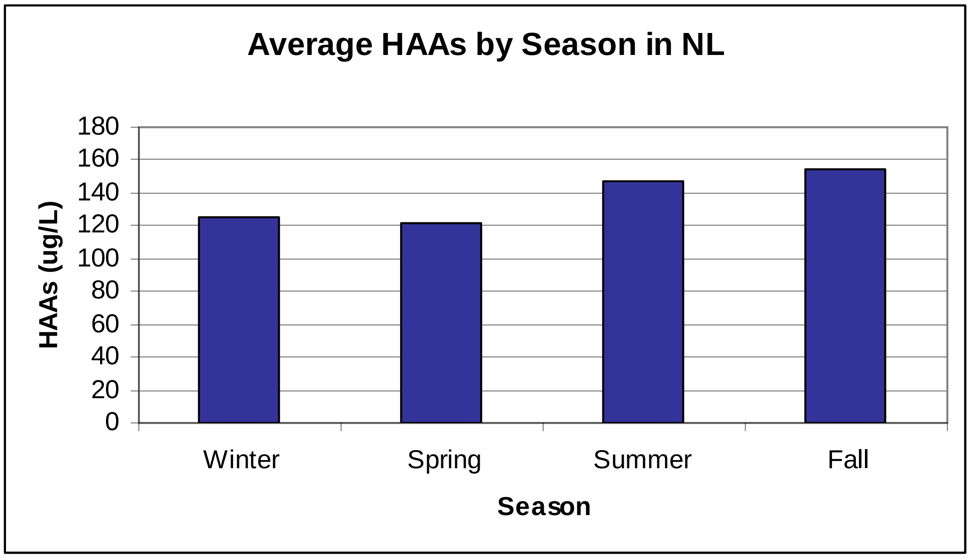


Figure 16: Average HAAs by season in NL: 2001-2007

To discover which variables played the most significant role in HAA formation a correlation analysis was performed using data from 63 samples from different communities across the province covering the period from 2001 to 2007. MINITAB statistical software was used to calculate the Pearson product moment correlation coefficient between each pair of variables listed. The correlation coefficient measures the degree of linear relationship between two variables. The correlation coefficient assumes a value between -1 and +1. If one variable tends to increase as the other decreases, the correlation coefficient is negative. Conversely, if the two variables tend to increase together the correlation coefficient is positive. For this analysis DOC was used as a surrogate of natural organic material, as it is an indicator of the mass of organic substance in water.

Table 10: Correlation analysis between HAAs and HAA precursors in NL

Variable	Pearson Correlation Coefficient for HAAs	P-Value
Sample size	63	
Total THM	0.732*	0.000
DOC	0.387*	0.002
Total Chlorine	0.328*	0.012
Colour	0.264*	0.036
Iron	0.258*	0.041
Free Chlorine	0.204	0.118
Turbidity	0.158	0.216
Water Temperature	0.149	0.256
pH	-0.137	0.286
Nitrate/Nitrite	-0.134	0.295
Site Number	0.129	0.315
Bromide	-0.050	0.696

* statistically significant at $\alpha = 0.05$

HAAs were most closely correlated to THMs followed by DOC, total chlorine, colour and iron. All other correlations were not deemed significant. Although pH was not found to be significant, it did correlate negatively with HAAs. The overwhelming majority of drinking water systems in the province that display high HAAs also display pH levels below the aesthetic guideline of 6.5.

Based on over 1500 HAA samples collected in the province from 2001 to 2007, HAA concentrations from systems across the province ranged from 0-2420 µg/L, with a median value of 94 µg/L. These results are skewed towards the high end as HAA samples in the province have been targeted towards drinking water systems where high HAA levels have been observed.

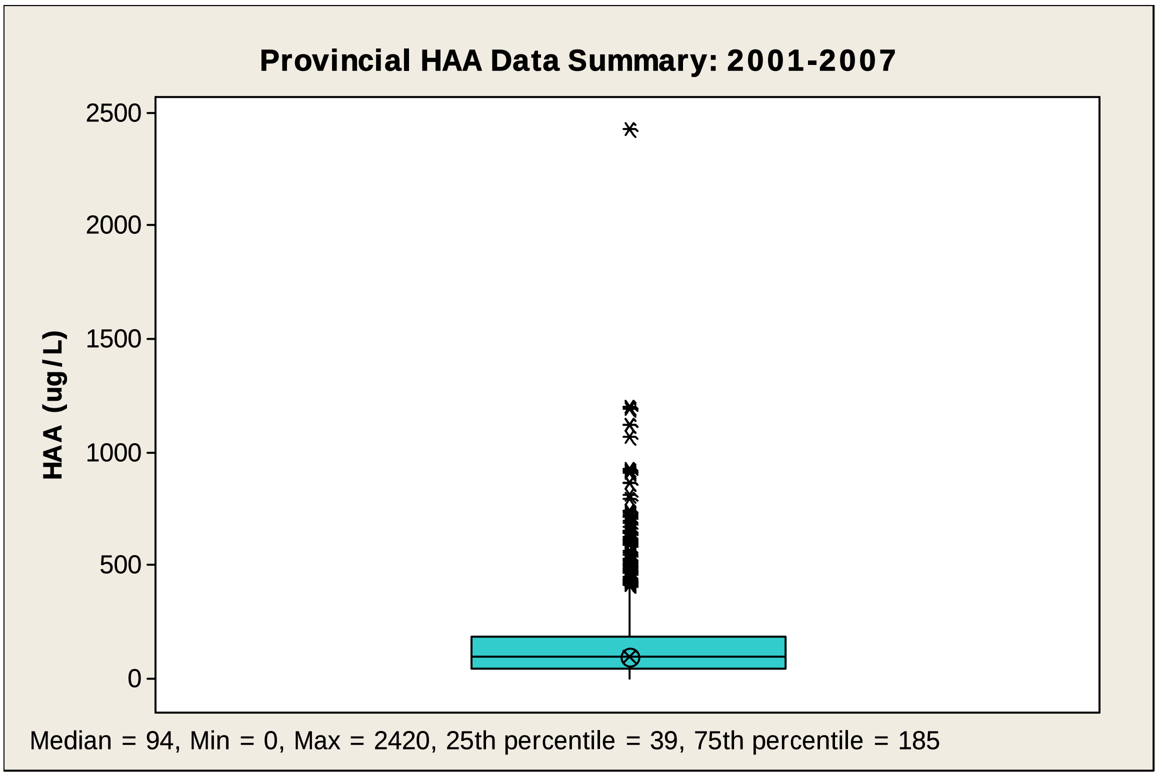


Figure 17: Provincial HAA data summary: 2001-2007

3.3.1 Communities with HAA Issues

HAAs have been handled as an emerging/special parameter by the Department of Environment and Conservation and therefore sampling is on a site-specific basis, and the extent and frequency of sample collection is decided annually. The purpose of collecting samples for HAA analysis has been to accumulate background data in anticipation of a MAC under the GCDWQ. A guideline of 80 µg/L for total HAAs (or HAA5, the five most commonly found HAA species in drinking water) has since come into effect during the writing of this report (Jan, 2008).

The extent of the HAA issue can be judged by looking at the number of communities with HAA levels above the guideline. The US EPA has declared a maximum concentration limit (MCL) of 60 µg/L for HAAs. For discussion and comparison purposes within this section, the US EPA limit will be referenced. Looking at tap water quality data from the period 2003-2006, the total population impacted by HAA exceedances has gone as high as 48%. In total there are 184 communities with HAA issues in the province; 22 communities have very major HAA issues, 57 communities have major HAA issues, 41 moderate HAA issues, and 64 minor HAA issues (DOEC, 2008). In the analysis of HAA exceedances summarized in the following table, individual exceedances greater than 60 µg/L are considered, not the annual running average. Also, the HAA total is comprised of the sum of all 8 HAA species in the provincial drinking water quality database. HAA exceedances are broken down into the following descriptive categories:

- Minor– average level is between 60-100 µg/L
- Moderate – average level is between 100-150 µg/L
- Major– average level is between 150-250 µg/L
- Very Major– average level is above 250 µg/L with some extremely elevated values

Table 11: Communities with HAA issues

Very Major HAA Issues	Major HAA Issues	Moderate HAA Issues	Minor HAA Issues
Cartwright	Baie Verte	Appleton	Aquaforte
Clarenville	Bauline	Avondale	Bay de Verde
Fox Roost-Margaree	Black Tickle-Domino	Burin	Beachside
Keels	Bonavista	Burnt Islands	Bellburns
Lamaline	Brig Bay	Cook's Harbour	Botwood
Little Catalina	Brighton	Corner Brook	Buchans Junction
Mary's Harbour	Brigus	Corner Brook	Burin
New-Wes-Valley	Buchans	Cupids	Burlington
New-Wes-Valley	Burgeo	Dover	Carmanville
Parker's Cove	Campbellton	Flower's Cove	Centreville-Wareham-Trinity
Pleasantview	Cape Freels North	Grand Bank	Centreville-Wareham-Trinity
Port Blandford	Come By Chance	Greenspond	Conne River
Port Hope Simpson	Comfort Cove-Newstead	Harbour Main-Chapel's Cove-Lakeview	Deer Lake
Purcell's Harbour	Corner Brook	Hare Bay	Elliston
Rigolet	Cottlesville	Heart's Delight-Islington	Fleur de Lys
Smith's Harbour	Cow Head	Hopedale	Gambo
St. George's	Crow Head	Lewin's Cove	Grand Falls-Windsor
St. Pauls	Dildo	Little Bay Islands	Hampden
Sunnyside (T.B.)	Embree	Lourdes	Hant's Harbour
Terrenceville	Fairbanks-Hillgrade	Lushes Bight-Beaumont-Beaumont North	Happy Valley-Goose Bay
Trinity Bay North	Fermeuse	Makkovik	Happy Valley-Goose Bay
Wild Cove	Ferryland	Millertown	Hawke's Bay
	Fogo	Pasadena	Heart's Content
	Fortune	Pidgeon Cove-St. Barbe	Herring Neck
	Garnish	Point Leamington	Hughes Brook
	Gaskiers	Point of Bay	Joe Batt's Arm-Barr'd Islands-Shoal Bay

	Glenwood	Ramea	Lewisporte
	Glovertown	Rattling Brook	Little Bay
	Happy Adventure	Reidville	Long Harbour-Mount Arlington Heights
	Harbour Breton	Rocky Harbour	Loon Bay
	Hermitage	South Dildo	Lower Lance Cove
	Howley	St. John's	Middle Arm
	Irishtown-Summerside	St. Lunaire-Griquet	Miles Cove
	Isle aux Morts	St. Lunaire-Griquet	Milltown-Head of Bay D'Espoir
	King's Point	Steady Brook	Mount Moriah
	Leading Tickles	Summerford	Musgrave Harbour
	Main Brook	Tilting	Nain
	Marystown	Torbay	Nameless Cove
	Massey Drive	Trinity	Norris Arm
	Merritt's Harbour	Triton	Norris Point
	New Perlican	West Bay	Peterview
	Old Perlican		Petty Harbour-Maddox Cove
	Placentia		Phillip's Head
	Port Albert		Piccadilly Head
	Portugal Cove-St. Phillips		Port au Choix
	Postville		Port au Choix
	Queen's Cove		Port au Port West-Aguathuna-Felix Cove
	Salvage		Port Saunders
	Seal Cove (WB)		River of Ponds
	South River		Robert's Arm
	Southern Harbour		Roddickton
	St. Bernard's-Jacques Fontaine		Salmon Cove
	St. Lewis		Seldom-Little Seldom
	Tizzard's Harbour		Springdale
	Twillingate		St. Anthony
	Whiteway		St. Anthony Bight
	Woodstock		St. Lawrence
			Stoneville
			Straitsview
			Upper Island Cove
			Victoria
			West St. Modeste
			Wooddale
			Bay Roberts

3.4 Formation Behaviour of THMs

The formation of THMs is not instantaneous. Typically, the rate of formation is fastest in the initial hours after chlorine has been added and then slows down. THM formation can proceed for several days in a distribution system as long as there is free chlorine residual.

Modeling of THMs has been used to:

- Identify the significance of diverse operational and water quality parameters controlling the formation of THMs
- Investigate the kinetics of THM formation
- Predict THM levels as an alternative to field monitoring

Predictive modeling of THMs involves establishing empirical and kinetic relationships in order to ascertain the variables such as water quality (NOM, bromide, pH, water temperature) and operational parameters (disinfectant dose, contact time) that can significantly explain THM formation potential.

To discover which variables played the most significant role in THM formation a correlation analysis was performed for each region of the province on a dataset covering the period from May 2001 to Sept 2005. MINITAB statistical software was used to calculate the Pearson product moment correlation coefficient between each pair of variables listed. The correlation coefficient measures the degree of linear relationship between two variables. The correlation coefficient assumes a value between -1 and +1. If one variable tends to increase as the other decreases, the correlation coefficient is negative. Conversely, if the two variables tend to increase together the correlation coefficient is positive. For this analysis DOC was used as a surrogate of natural organic material, as it is an indicator of the mass of organic substance in water. Site number was used as a surrogate for retention time in the distribution system ranging from 1 (beginning of the system) to 4 (end of the system).

Table 12: Correlation coefficients between THMs and THM precursors in NL

Variable	THMs- Eastern Region (p-value)	THMs- Central Region (p-value)	THMs- Western Region (p-value)	THMs- Labrador Region (p-value)
Sample size	551	496	350	59
DOC	0.454 (0.000)*	0.323 (0.000)*	0.415 (0.000)*	0.638 (0.000)*
Bromide	-0.167 (0.000)*	-0.148 (0.001)*	-0.122 (0.037)	-0.175 (0.186)
Water Temperature	0.128 (0.006)*	-0.047 (0.312)	0.239 (0.000)*	0.425 (0.012)
Free Chlorine	0.129 (0.002)*	0.280 (0.000)*	0.377 (0.000)*	0.569 (0.000)*
Site Number	0.017 (0.684)	0.022 (0.638)	0.146 (0.006)*	-0.054 (0.686)
pH	-0.085 (0.047)	0.136 (0.003)*	-0.187 (0.000)*	-0.439 (0.001)*
[Total Chlorine]	0.120 (0.009)	0.167 (0.056)	0.219 (0.000)*	0.591 (0.000)*
[colour]	0.226 (0.000)*	0.093 (0.043)	0.186 (0.000)*	0.483 (0.000)*

* statistically significant at $\alpha = 0.01$

For correlation coefficients with p-values smaller than 0.01, there is sufficient evidence at $\alpha = 0.01$ that the correlations are not zero, in part reflecting the large sample sizes. From the table above it is obvious that DOC plays the most significant role in THM formation followed by free chlorine, pH and water temperature. Colour and total chlorine were also fairly well correlated with THMs, but were not ranked in the above table as related indicators (DOC and free chlorine) provided a much more significant indicator of THM formation. Bromide appears to negate THM growth, however, this is thought to be due to the censored nature of the bromide dataset. THMs are supposed to increase with increasing pH, however pH showed a majority of negative correlations. All parameters except for bromide appeared to be significantly correlated with THMs in the Western

Region of the province. From the above correlation, DOC is the best surrogate available for THM precursors (Poole, 2006).

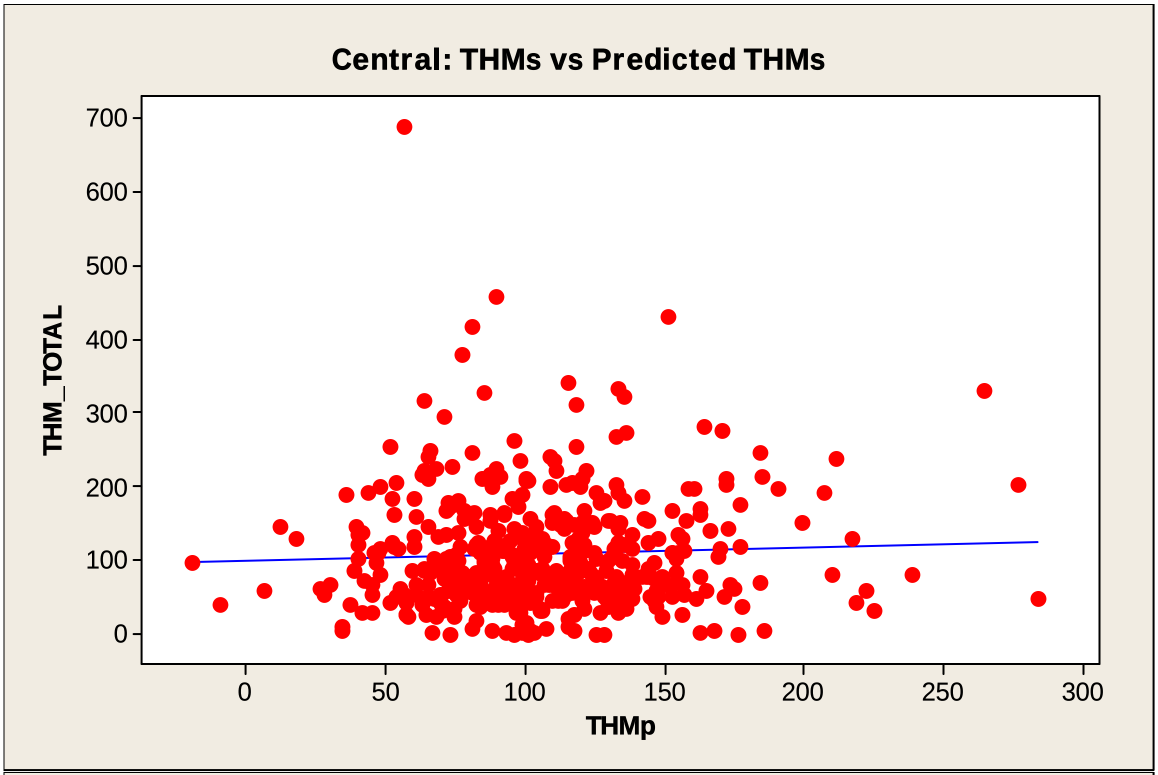
3.5 Empirical THM Models

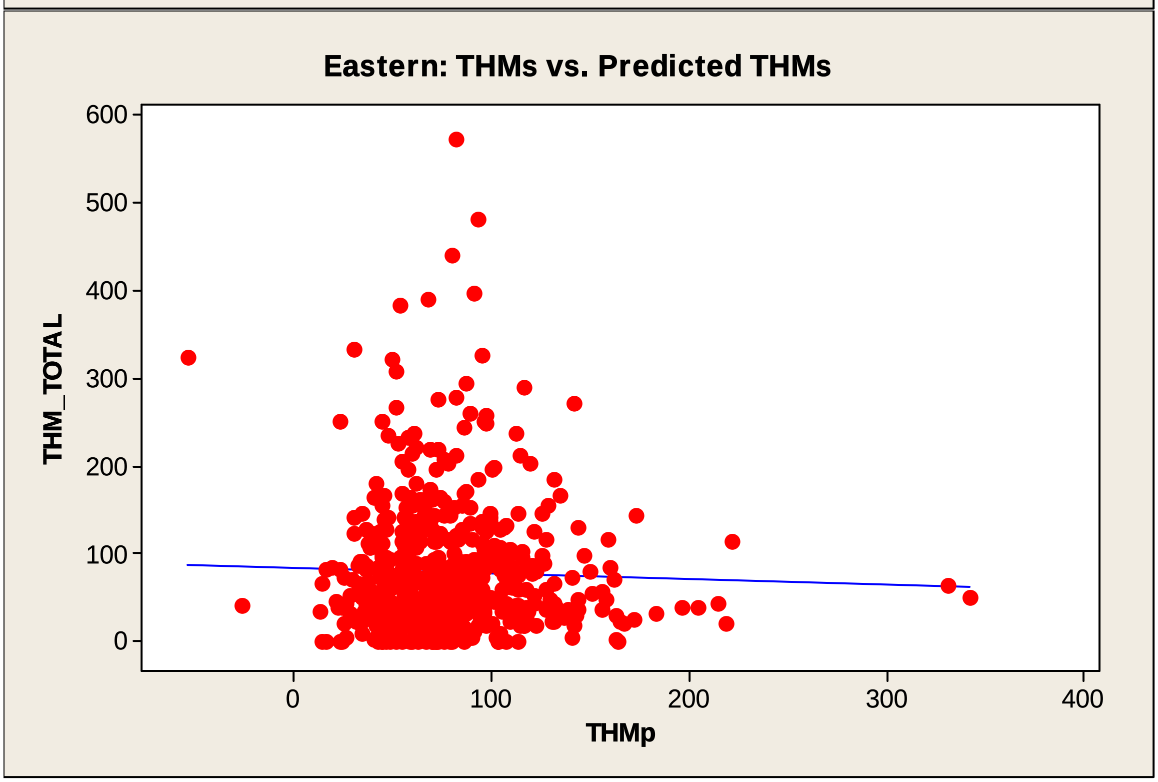
Due to the diverse nature of chlorine and natural organic material reactivity, THM formation models are often empirical curves fitted to observed data for specific water conditions, and model coefficients can be highly site specific. Multiple linear regression analysis was performed (using MINITAB) for each region of the province to develop simple predictive THM models and to discover which of the six major explanatory variables would be included in the regression equation.

Table 13: THM empirical models to predict THMs in NL

Region	Predictive model	R^2	Cp	Max VIF	PRESS
Eastern	THM = - 67.3 + 29.2 FREE CHLORINE + 1.60 WATER TEMP + 11.0 DOC + 11.9 PH - 416 BROMIDE	23.8%	5.9	1.1	2150068
Central	THM = - 124 + 10.0 DOC + 36.3 FREE CHLORINE + 23.9 PH - 434 BROMIDE + 5.30 SITE NUMBER	25.9%	5.3	1.1	2188068
Western	THM = - 145 + 13.7 DOC + 53.7 FREE CHLORINE + 4.77 WATER TEMP + 11.0 PH + 8.53 SITE NUMBER	40.2%	5.0	1.4	1669794
Labrador	THM = - 34.3 + 4.74 WATER TEMP + 21.1 DOC + 50.0 FREE CHLORINE	70.5%	1.6	1.3	147274

The table above summarizes the best-fit linear regression equations for each region. Best-fit was determined by evaluating a number of statistics including R^2, Mallows Cp, the variance inflation factor (VIF) and the PRESS value. Residual plots for each region, except for Labrador, indicate the presence of outliers, non-normal datasets, and non-consistent variance, which affects the accuracy of the regression analysis. Details on the regression analysis can be found in Appendix A. DOC and free chlorine appear as variables in the regression equation for each region of the province. A comparison of measured THM data from each region versus predicted THM values using the above linear regression equations indicated only modest agreement as can be seen in the following figure.


Central: THMs vs Predicted THMs
THM_TOTAL
700
600
500
400
300
200
100
0
0
50
100
150
200
250
300
THMp


Eastern: THMs vs. Predicted THMs
THM_TOTAL
600
500
400
300
200
100
0
0
100
200
300
400
THMp

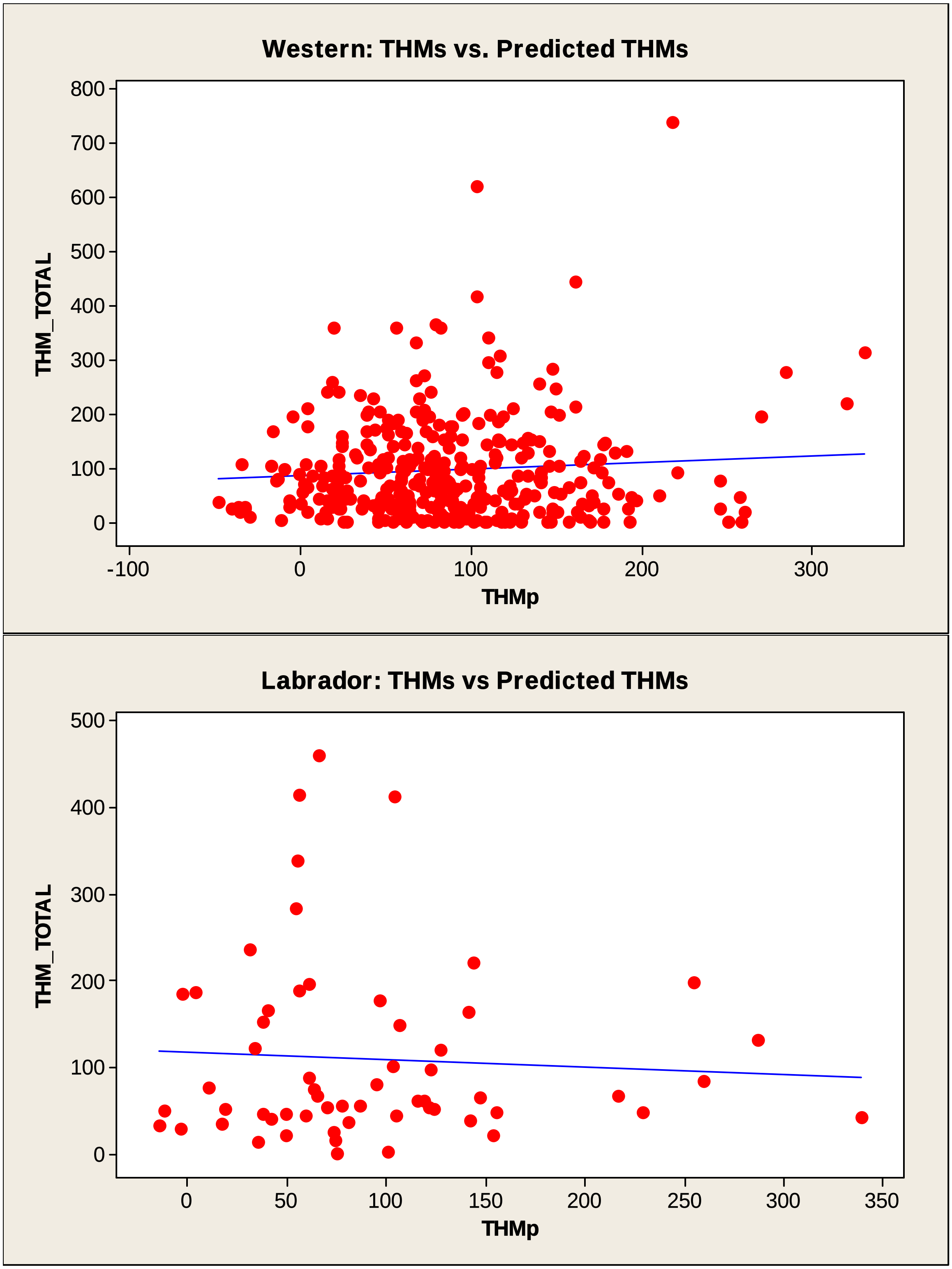


Figure 18: Measured THM data versus THM model predictions for each region

The multiple linear regression models developed for each region can be used to predict THM formation based on various input parameter values (DOC, free chlorine). These equations can be used as a rough guide to indicate which regions have higher THM formation potential, removal requirements of THM precursors, and optimization goals for chlorine dosage. THM formation potential is highest in the Western Region followed by Labrador, Central and then Eastern. Even at low levels of DOC and low levels of free chlorine, the models indicate that THM formation potential is above guideline levels for most of the situations examined. Conditions for THMs to be maintained below guideline levels would include a maximum DOC range of 2.5-6 mg/L, and a maximum free chlorine residual range of 0.2-1.0 mg/L, depending on the region.

A non-linear empirical model to determine THM concentrations has been incorporated into the USEPA water treatment plant (WTP) model using a variety of explanatory variables:

$$[THM] = X_0[TOC]^{X_1} pH^{X_2} t^{X_3} T^{X_4} [Br]^{X_5} [Cl_2]^{X_6} [UV]^{X_7}$$

Equation 9: Non-linear empirical model to determine THMs used in USEPA WTP model

where:

[THM] = THM concentration
X_0 to X_7 = fitted coefficients
[TOC] = total organic carbon concentration
pH = pH
t = time
T = temperature
[Br] = bromine concentration
[Cl_2] = chlorine dose
[UV] = UV absorbance

This model was derived from data collected from treatment processes, rather than the distribution system, where there are higher chlorine doses and shorter contact times. The validity of this model within the distribution system is uncertain, but it has been applied previously with some success (AwwaRF, 2006).

The fraction of NOM most favoured in DBP formation are the aromatic and unsaturated components, which are best detected using UV 245nm wavelength. The greater the absorption of UV light at the UV 254nm wavelength, the higher the amount of aromatic organics. Other organic test parameters, such as DOC or colour, have slightly different biases which make them less ideal for predicting DBP formation. DOC focuses on dissolved organics as well as some non-organic carbons. Colour is generally not recognized as an accurate organic test parameter and it is possible to have water low in colour but still having a large amount of organics. UV_{245} is considered a more direct predictor of source waters to form THMs and HAAs (USEPA, 2007).

3.6 Kinetic Models

The growth or decay of a substance can be modelled using kinetic equations. The rate of most chemical reactions is typically defined by an equation of the following form:

$$\frac{dC}{dt} = -k\,f\,\{\text{reactants}\}$$

Equation 10: Rate of chemical decay

where:

C = concentration
k = reaction rate constant (units dependent on order of reaction)
f{reactants} = some function of the concentration of the reactants

The reaction order is defined as the order of the differential rate of the above equation. Where the reaction rate is directly proportional to the concentration of a single reactant, the reaction is classed as first order. A second order reaction can be due to the reaction rate being proportional to the square of the concentration of a single reactant, or due to the reaction rate being proportional to the product of two reactants. Many water quality reactions are complex and involve numerous different reactants, so simplification is necessary. If one part of the reaction is much slower than the rest, it may be reasonable to only model this rate determining step, or when all but one reactants are present in large quantities, and only the concentration of that reactant changes significantly during the course of the reaction, the reaction rate may appear to follow a first order pattern.

In modeling software programs (eg. EPANET), free chlorine decay is typically modeled as a first order exponential decay with time. THM growth is typically modeled as a first order logistic growth with time.

3.6.1 Chlorine Decay Kinetic Models

Within a distribution system, chemical reactions are assumed to occur both within the bulk flow and with the pipe-wall material or bio-film, based on first order kinetics. Chlorine decay kinetics is a function of both water quality parameters (NOM; inorganic compound concentrations- iron, manganese; temperature; pH) and operational conditions (disinfectant dose, pipe size, residence time, treatment processes). The rate of chlorine decay is highly variable, affected by numerous different parameters which themselves show significant variation between different distribution systems. The majority of chlorine gets consumed early on in the decay reaction. The following table summarizes factors affecting chlorine decay.

Table 14: Factors affecting chlorine decay

Factor	Effect on chlorine decay
Chlorine dose	As chlorine dose increases, the relative demand also increases. However, the rate of bulk decay will decrease if chlorine dose is increased or water re-chlorinated.
Temperature	Temperature has minimal effect during the first few hours of decay. After that, disinfectant residual decays faster at higher temperatures. The decay rate can increase two to three-fold for every 10°C rise in temperature.
pH	Chlorine decay changes with pH, however there is no definitive pattern and the effect is specific to the water.

Pipe sediments	As the amount of iron pipe sediment increases, the decay rate also increases resulting in lower chlorine residuals.
Blending	Blending waters of different age results in a re-chlorination of the old water. Blended water has more stable chlorine residual than either the new or old water.
Re-chlorination	The residual decay after re-chlorination depends only on total chlorine dose.
NOM and Turbidity	The higher the turbidity and concentration of natural organic material, the greater the chlorine demand and chlorine decay rate.
Biofilm	The larger the density of microbiological populations in the biofilm, the greater the chlorine demand.
Pipe size	Chlorine decay increases with decreasing pipe size.
Pipe material	They type of pipe material can potentially have the biggest impact on the rate of chlorine decay. The rate of decay due to unlined iron pipes is typically 10-100 times greater than in cement lined pipes and 100-1000 time greater than in plastic pipes.

The equation for first-order decay of a substance (eg. chlorine) in a distribution system is:

$$C_t = C_0 \exp(-k_t t)$$

Equation 11: First-order decay of a substance

where:

C_t = concentration of substance at any time, (mg/L)
C_0 = initial concentration of chlorine, (mg/L)
k_t = total decay rate, a function of bulk phase decay constant (day^{-1}), wall reaction constant (m/d), molecular diffusivity of the substance, water's kinematic viscosity, velocity and pip radius

A first order model will often give reasonable results for bulk chlorine decay; however, it tends to underestimate the initial chlorine decay in the first 15 minutes to 4 hours following chlorination. It also ignores the concentration of the principal reactants with which chlorine reacts, for example if the DOC varies significantly, the decay rate will change and the first order model will not be accurate. The first order decay coefficient will decrease if the chlorine dose is increased and will also decrease significantly upon re-chlorination. These limitations probably occur because the various compounds with which chlorine reacts limit the rate of the reaction, however, drinking water contains many different compounds, some of which react faster than others and are therefore depleted before other compounds.

The rate of chlorine decay is strongly controlled by temperature. A common approximation is that the reaction rate doubles for every 10°C rise in temperature, otherwise known at the van Hoff approximation. The Arrhenius equation is widely accepted for modeling the effect of temperature on reaction kinetics:

$$k = F \exp\left(\frac{-E}{R(T + 273)}\right)$$

Equation 12: Arrhenius equation

where:

k = reaction coefficient
F = frequency factor
R = ideal gas constant (8.31 J/mol°C)
E = activation energy
T = temperature (°C)

In this study, temperature effects on reaction rates were mostly ignored. Bulk chlorine decay can also be described using a first-order kinetic rate with respect to TOC and temperature as follows:

$$K_b = a \times [TOC] \times e^{(\frac{-b}{T})}$$

Equation 13: Bulk chlorine decay with respect to TOC

where:

K_b = bulk decay constant
a = 1.8x106 L/mg-h
b = 6050 °K
T = temperature in °K
TOC = total organic carbon in mg/L

The following equation can be used to predict chlorine residuals with time in pipes given temperature and UV_{254} (AwwaRF, 2005):

$$Cl_{2(t)} = Cl_{2(0)} \times \exp[-(K_b \times UV254 + \frac{K_W}{D_p}) \times A^{(Temp-20)}] \times time$$

Equation 14:Predicting chlorine residual in pipes

where:

K_b = bulk decay constant (cm/hr)
K_w = wall decay constant (in/hr)
$Cl_{2(t)}$ = chlorine concentration at time t (mg/L)
$Cl_{2(0)}$ = initial chlorine concentration (mg/L)
A = temperature correction coefficient
T = temperature in °C
t = time (h)
D_p = pipe diameter (inches)
UV254 = ultraviolet absorbance at 254 nm wavelength (cm^{-1})

Many different factors affect the rate of chlorine decay (pipe material, water temperature, organic content, pipe diameter) and so reaction coefficients tend to be highly site specific.

The following table gives an indication of the range of chlorine decay rates observed from various sources. The use of previous values recorded in similar systems, while the simplest approach, is only really suitable as an indicative approach as decay coefficients can very by a factor of 100 for similar pipe materials.

Table 15: Chlorine decay rates in different pipe material

Bulk Decay: K_b (1/d)	Wall Decay: K_w (m/d)	Pipe material	Temperature Correction Coefficient	Location/ Source
2.7	0.5	AC		AwwaRF, 2006
0.12	2.48	Cast Iron		AwwaRF, 2006
2.1	12.6	Cast Iron		AwwaRF, 2006
2.3	3.3	Lined DI		AwwaRF, 2006
2.3	1.3	MDPE		AwwaRF, 2006
2.7	3.6	Cast Iron		AwwaRF, 2006
2.0	2.8	PVC		AwwaRF, 2006
1.3	1.1	Cast Iron		AwwaRF, 2006
1.3	0.5	Lined DI		AwwaRF, 2006
1.2	1.6	PVC		AwwaRF, 2006
1.2	0.6	MDPE		AwwaRF, 2006
0.36	-	-		Gander
0.83	-	-		Gander
1.54	-	-		Gander
0.69	-	-		Burlington
0.56	-	-		Burlington
1.53	-	-		Burlington
4.02	-	-		Burlington
0.26	-	-		Brighton
1.15	0.004	PVC	1.15	AwwaRF, 2005
2.02	0.004	Lined cast iron	1.18	AwwaRF, 2005
4.13	0.038	Cast iron	1.13	AwwaRF, 2005
14.74	0.033	Galvanized iron	1.04	AwwaRF, 2005
1.58	0.040	Grey cast iron		AwwaRF, 2006
2.88	0.18	Cast iron		AwwaRF, 2006
1.16		AC		AwwaRF, 2006
17.70	1.50	Cast iron		AwwaRF, 2006
0.77	0.031	Lined DI		AwwaRF, 2006

The relative reactivity of pipe materials in chlorine decay from highest to lowest is as follows:

- Cast iron (most reactive)
- Ductile iron
- Asbestos cement
- PVC
- Polyethylene (least reactive)

Besides using previous values, wall reaction coefficients can be estimated using field measurements, laboratory tests, and calibration against network field tests. In this study, the latter method was of the most practical use. It is a reasonably common practice to collect chlorine spot data from around the distribution network and then adjust the decay coefficients until there is a good fit between modeled and observed data.

3.6.2 THM Formation Kinetic Models

THM growth tends to decrease with time asymptotic to a maximum formation potential. THM formation is often predominantly controlled by bulk water reactions and therefore only modeled as a bulk water reaction. With regard to the formation of a substance (eg. THMs), the rate of formation is a function of time. It has been postulated that DBP formation is governed by the following first order saturation growth equation (Clark et al., 1998):

$$DBP = DBP_u(1 - e^{-kt})$$

Equation 15: First-order saturation growth of DBPs

where:

DBP = DBP concentration, (mg/L)
DBP_u = the ultimate formation potential of the DBP
DBP_0 = initial concentration
t = time , (days)
k = reaction coefficient, (day-1)

The limitation of this model is that it does not take account of the concentration of the various precursors, which control THM formation. The growth coefficient will be highly site specific and if the water quality is not consistent, the coefficient may show considerable variability over time.

The two parameters required for modeling first order saturation growth are: a growth rate coefficient and an ultimate formation potential concentration. In theory, both of these can be determined in the laboratory using bottle studies. The ultimate formation potential is site-specific and generic values may prove meaningless, as it is highly dependent on the amount and nature of the natural organic matter in the water, as well as its removal by any treatment process. Any prediction of THM growth parameters performed in the lab should be carried out using the predicted retention time or water age of water in the distribution system. The source THM as well as both the growth rate coefficient and the ultimate formation value can also change from day to day because of variations in the raw water, particularly in the bromide content. To get an idea of possible THM ultimate formation potential, the maximum observed THM value in each region of the province is listed in Table 16.

Table 16: Ultimate formation potential of THMs in NL by region

Region	THM_U (ultimate formation potential), ug/L
Eastern	573

Central	688
Western	740
Labrador	460

From a limited number of EPA studies, the growth rate coefficient for THM formation has ranged from 0.5 to over 2 per day. As a rough approximation, a THM level of half the ultimate formation potential can be expected after roughly 10 hours. THM_U levels can be expected anywhere from 25 to over 200 hours after chlorination as indicated in Figure 19. Ultimate THM formation potential for specific distribution systems should be based on bulk water growth rates up to the maximum residence time in the distribution system. The extent to which THMs can be controlled by retention time management depends upon their formation rate. Fast forming DBPs will be largely formed in the beginning of the distribution system and so cannot be controlled by altering retention times. The slower the growth rate, the greater is the potential for controlling them by managing retention time.

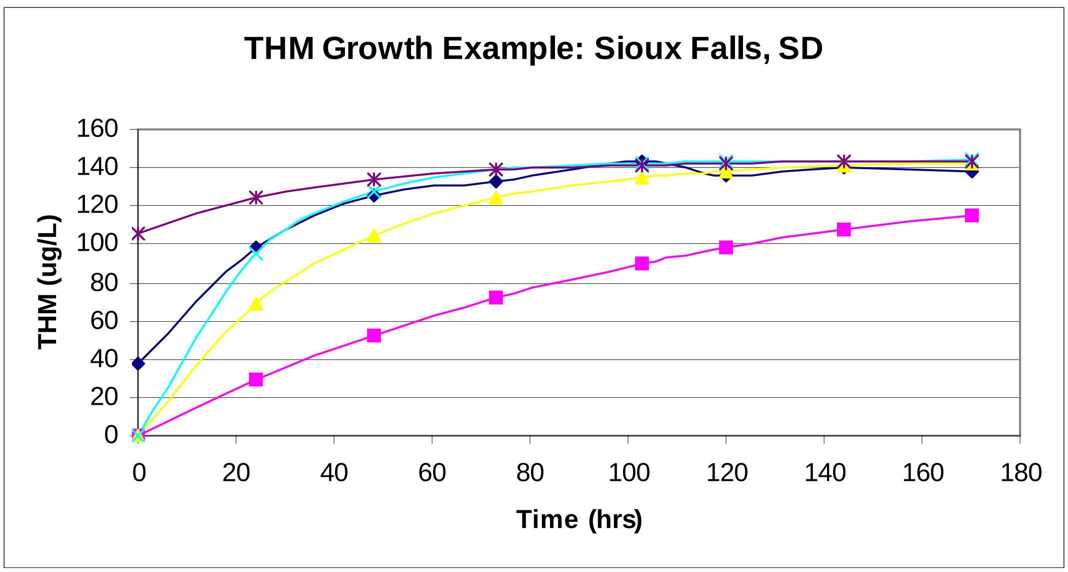


Figure 19: Example of typical THM growth rates- Sioux Falls, SD

It is possible to measure THM levels in distribution systems that are higher than the ultimate level determined in the lab. It is also possible to infer from measured values that THM might be decreasing with residence time in the system when actually it is just the effect of a variation in levels (and reactivity) of THMs entering the system. Simulating THM formation as only a bulk water phenomenon should be undertaken with caution because in some cases organics bound to iron pipe deposits or in the biofilm can cause an increase in DBPs, while adsorption onto iron pipe deposits can cause a decrease in concentrations. The potential for THMs to volatilize from the free water surface in storage tanks is most likely negligible.

3.6.3 Water Age Models

Water age is the time spent by a parcel of water in the distribution network. From a modeling standpoint, water age is treated as a reactive constituent whose growth follows zero-order kinetics with a rate constant equal to 1 (i.e. each second the water becomes a second older). New water entering the distribution network from reservoirs or source supplies enters with an age of zero. As this slug of water moves through the pipe network it splits apart and blends together with parcels of varying age at pipe junctions and storage facilities.

Water age also provides a simple, non-specific measure of the overall quality of delivered drinking water. Water age can be adapted to predict some water quality parameters (such as chlorine and THMs), where those parameters can be defined as a function of time.

4.0 Corrective Measures for Reducing DBPs

There are several different approaches available for dealing with the problem of THMs in community drinking water systems. This section will explore these different approaches, which consist of structural, nonstructural and operational techniques and other best management practices (BMPs). Any corrective measure must be an effective and practical means to contribute significantly to the safety of drinking water and ensure productive use of resources (water, financial, human, etc.).

Because a number of communities throughout the province have been identified as having serious long-term problems with high levels of chlorinated DBPs, the province has been working on various strategies for THM control. The approach taken by the province for THM control was first outlined in the report *Trihalomehtane Levels in Public Water Supplies of Newfoundland and Labrador* (2000), and has evolved over time with more data, research and experience on this issue.

The following is a list of revised broad-based corrective measures that will be explored in further sections of this report. Many of the individual control measures further investigated fall under more than one of these broader categories.

1. Policy measures
2. Source based control measures
3. Chlorine demand management (CDM)
4. Retention time management (RTM)
5. Water demand management (WDM)
6. Water distribution system operational and infrastructural measures
7. Alternative disinfectants
8. Source water treatment
9. Point of use/point of entry measures
10. Water system design measures
11. Operator education and training

The effectiveness of any corrective measures will have to be determined against a list of appropriate criteria such as fiscal capacity of the community, corrective measure cost, feasibility, and level of DBP reduction. A useful tool in the evaluation of certain corrective measures is a water distribution system model such as EPANET. Models can be used to guide decision-making for distribution system design, operational control, maintenance and infrastructure to minimize DBP formation.

4.1 Policy

Providing the public a safe and reliable supply of water at the point of use (tap) became a public health issue in the early 1800’s once it became known that contaminated water supplies spread diseases like cholera and typhoid. At the time, responsibility for water supply treatment and distribution typically fell to government, as they were the only entity with the resources necessary to achieve the goals of providing safe and adequate supply for all. Most people in the Western world now take for granted that a safe supply of water is available to them at the turn of a tap.

Towns in the province have evolved due to resource driven factors (fishery, forestry, mining, transportation routes, etc.) that bring many families together to form a community. These groups then form a collective committee or council to provide needed services and control of community development. These services include water supply, wastewater, roads, street lighting, recreational facilities, and other social and development activities. In Newfoundland and Labrador, the province is the ultimate custodian of all freshwater resources as laid out in the *Water Resources Act* (SNL 2002, W-4.01). Not all communities provide distributed water, disinfection or water treatment, but once they do, they are mandated by the Act to operate such waterworks in such a manner as to provide adequate quality water.

A policy of point of use water treatment would denote a radical shift from the current approach of centralized water treatment of water supplies. Shifting from casual usage of point of use treatment devices by the public, however, to a policy of emergency or regular point of use treatment devices in households would contradict 200 years of precedent and could possibly lead to legal challenges. With such a change of policy, the responsibility for safe, consumable water would now be the onus of the user and not the supplier. The main argument supporting such a shift is the prohibitive cost of water infrastructure, particularly for water treatment and especially in small communities.

Although not strictly speaking a true policy, the prevalence of chlorine use as a disinfectant in Newfoundland and Labrador borders on one. This is in part due to the strength of the chlorine industry in North America. In comparison, ozone sees much wider use in Europe. The following figure shows the breakdown of primary disinfectant by type in 452 drinking water systems in the province.

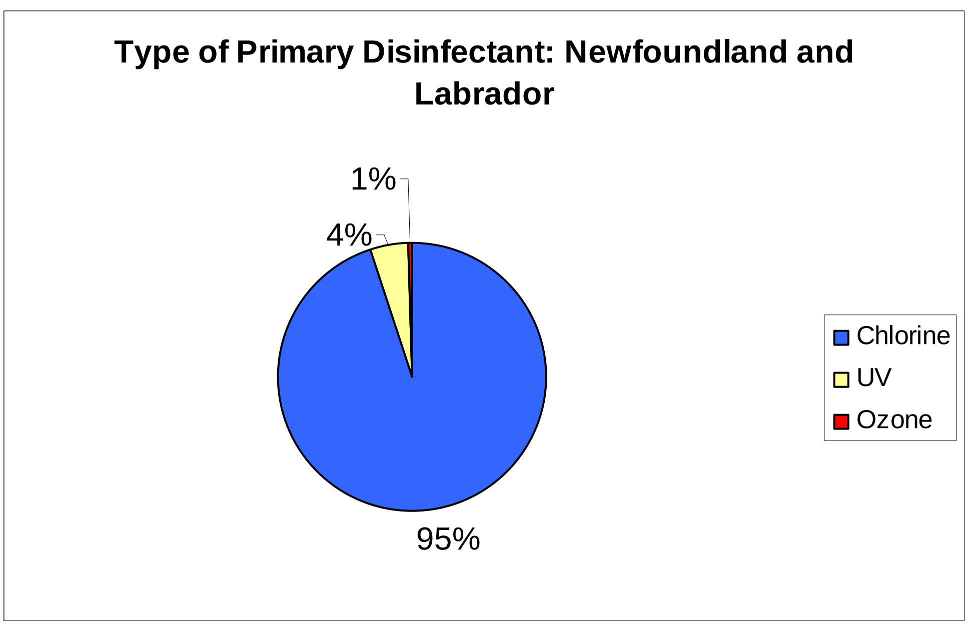


Figure 20: Type of primary disinfectant used in Newfoundland and Labrador

Key Messages:

- It should remain the mandate of any community with a centralized water distribution system to provide adequate quality drinking water to users; the onus for providing potable water meeting GCDWQ should not be placed on the water consumer.
- In very small and small communities with THM levels significantly above the guideline value, a policy of point of use household treatment devises can be implemented as a temporary or emergency measure. A temporary measure should be considered as lasting three months or less.
- More diversity in water disinfection treatment options should be promoted and implemented in the province.

4.2 Watershed Protection

Keeping water sources safe from contamination from various sectoral development activities such as forestry, agriculture, mining, etc., is the essence of watershed protection. Prevention is significantly easier and more cost effective than having to treat water to remove contaminants later on. With respect to the formation of DBPs in drinking water, watershed protection involves long-term management and control of NOM and bromide in raw water supplies through prevention of algal growth, soil erosion, fertilizer runoff, and waste discharges into raw water sources. Minimizing saltwater influences on freshwater sources also needs to be considered.

There are currently 259 surface water supplies with their watersheds designated as protected under Section 39 of the *Water Resource Act*. Once a watershed is protected no new development activity can take place without review and certain protection measures, such as buffer zones around all waterbodies within the protected public water supply area (PPWSA). Any new or expanded development activity is guided by the *Policy Directive on Land and Water Developments in Protected Water Supply Areas W.R. 95-01*. The majority of PPWSAs in the province can be classified as pristine with very little development activity of any kind ongoing within their boundaries. This makes control of DBP precursors through watershed protection a difficult task, as NOM and bromide simply occur naturally in many surface waters throughout the province.

There appears to be a link between surface water supplies exposed to ocean salt water spray and BDCM levels, particularly surface water supplies (i) close to the coastline with little cover from trees, (ii) exposed to prevailing westerly winds, or (iii) exposed to coastal winds from more than one direction. A correlation analysis was performed looking at the relationship between THM averages (as a substitute for BDCM values) and the distance of protected surface water intakes from the coastline as shown in the table below. While the correlation was not significant, it did indicate an inverse relationship between THMs and distance to the coastline (ie. the greater the distance from the coast, the lower the THM average). Approximately 16% of water supply intakes in the province are within 500 m from the coastline. Although there is no available scientific evidence to support it, there is potential to possibly reduce bromide levels in exposed coastal surface water supplies from salt-water spray by providing windbreaks (trees, fencing) around such water sources.

Table 17: Correlation of mean community THMs with distance of intakes to coastline

	Correlation Coefficient for THMs (p-values)
Distance of Intake to Coastline	-0.058 (0.371)

* statistically significant at $\alpha = 0.05$

An analysis of variance (ANOVA) statistical test was carried out to see if there was any difference in THM levels from surface water sources disinfected with chlorine and whether the source was protected or not. Results indicate there is no significant difference in mean THM levels from surface water sources with protected versus unprotected water supply areas as indicated in the following table.

Table 18: ANOVA of mean community THMs with watershed status as analysis factor

Status of Watershed	Number of Watersheds	Mean THM (Standard Deviation)	p-value
Unprotected	46	81.1 (± 65.20)	0.831
Protected	231	83.2 (± 59.27)	

* statistically significant at $\alpha = 0.05$

Key Messages:

- Designation of surface source water watersheds as Protected Public Water Supply Areas should be promoted across the province. PPWSA designation has shown little effect on lowering THM levels due to the pristine condition of most source protection areas and the non-anthropogenic origins of the vast majority of THM precursors present in source waters. PPWSA designation does minimize the risk of additional levels of THM precursors of anthropogenic origin.
- Waters sources and source water intakes should be located as far as possible from the coastline and prevailing coastal winds. Water sources should be sited in locations sheltered (by trees, differences in elevations, berms, fences, etc.) from ocean salt-water spray, and prevailing westerly and coastal winds.

4.3 Changing Raw Water Sources

The main precursor that can be used as a surrogate for DBP levels is DOC, the two being directly proportional. Average surface water DOC is 6.4 mg/L, but typically, any water over a DOC of 2 mg/L can produce unacceptably high levels of DBPs with the addition of chlorine for disinfection. The histogram below indicates the spread of DOC levels across the province. High levels of NOM (of which DOC is a measure) occur naturally in watersheds with a large percentage of wetland areas (bog, marsh, fens, swamp, open shallow water) of which there are many in the province. Flooding of vegetated areas to create more storage volume for surface water supplies is common practice throughout the province. Of 309 public surface water supplies, 114 (37%) have dams holding back water. The percentage of these dams that have flooded significant vegetated areas is unknown.

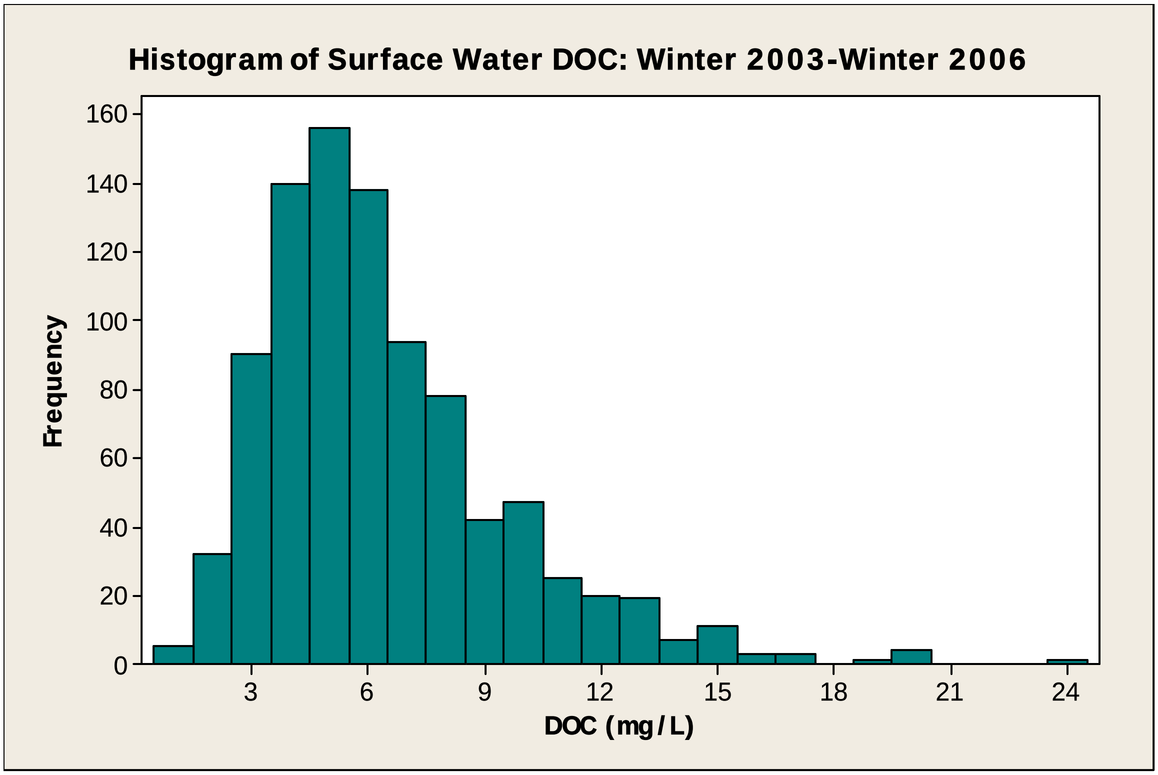


Figure 21: Histogram of surface water source DOC

Based on quartile ranges DOC can be classified as follows:

- Low: DOC <= 4.2 mg/L (25% of data less than or equal to 1st quartile)
- Medium: 4.2 > DOC <= 7.9 mg/L
- High: DOC > 7.9 mg/L (25% of data greater than or equal to 3rd quartile)

To minimize DBP problems with waters disinfected by chlorine, only surface waters with DOC less than 4.2 mg/L should be used as source water supplies to minimize DBP formation potential. When scouting new surface water supplies, this criterion should be kept in view.

Of 509 public water supplies in the province, 39% are from groundwater and 61% are from surface water. Source water can be further broken down into different source water types including rivers, ponds, lakes, brooks, reservoirs, canals, springs, drilled wells, and dug wells as indicated in the following figure. Ponds, drilled wells and brooks are the most common type of public water sources in the province.

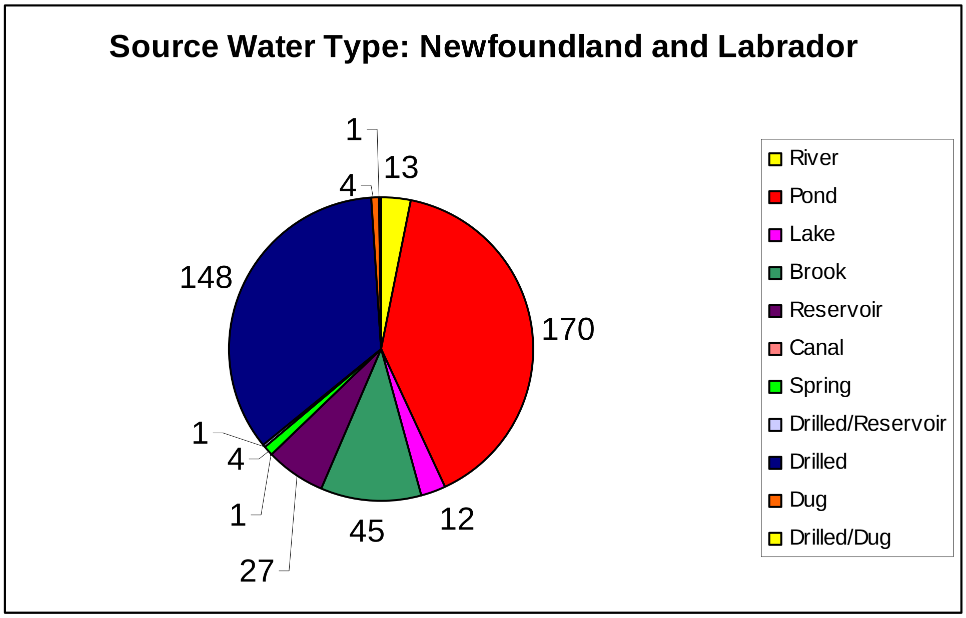


Figure 22: Source water type in Newfoundland and Labrador

An ANOVA analysis was performed to see if there was any difference in THM averages from the different source water types as shown in the following table. A significant difference was detected with rivers, ponds and lakes on the high end of THM levels and drilled and dug wells on the lower end. It is interesting to note that a large proportion of water supply dams are located on ponds in order to create greater storage volumes, where the potential for flooding land area is greatest. It is interesting to note that the type of surface water sources with the highest average THMs are typically from larger drainage areas.

Table 19: ANOVA of mean community THMs with source water type as analysis factor

Status of Watershed	Number of Different Source Types	Mean THM (Standard Deviation)	p-value
River	13	86.6 (± 58.36)	
Pond	170	85.8 (± 65.14)	
Lake	12	73.0 (± 55.29)	
Brook	45	59.6 (± 51.67)	
Reservoir	27	50.9 (± 50.74)	
Canal	1	50.29	0.000*
Spring	4	31.8 (± 4.71)	
Drilled/Reservoir	1	21.6	
Drilled	148	15.7 (± 27.7)	
Dug	4	6.73 (± 5.66)	
Drilled/Dug	1	3.06	

* statistically significant at $\alpha = 0.05$

An ANOVA analysis was performed to see if there was any difference in THM averages with increasing watershed area for surface water sources, as the results based on source type seemed to indicate larger watersheds had higher THM averages. The relationship between watershed size and THM average was not significant as indicated in the following table. Medium sized watersheds actually had the highest THM averages.

Table 20: ANOVA of mean community THMs with watershed status as analysis factor

Size of Watershed	Number of Watersheds	Mean THM (Standard Deviation)	p-value
Large (greater than 20 km^2)	59	82.93	
Medium (5-20 km^2)	71	90.91	0.440
Small (less than 5 km^2)	132	79.76	

* statistically significant at $\alpha = 0.05$

A correlation analysis to see if there is any relationship between average THM levels for each community with a protected water supply area and certain watershed characteristics was also performed as shown in the following table. Watershed characteristics examined include:

- watershed area
- percent of watershed area that is unclassified
- percent of watershed area that is exposed
- percent of watershed area that is covered by water
- percent of watershed area that is non-forest vegetation
- percent of watershed area that is wetlands
- percent of watershed area that is forest

Table 21: Correlation between average community THMs (Spring 2001-Spring 2006) with watershed land cover characteristics

Watershed Characteristic	Correlation Coefficient for THMs (p-value)
Watershed Area	0.003 (0.968)
% Unclassified	0.052 (0.398)
% Water	0.088 (0.152)
% Exposed	-0.123 (0.046)*
% Non-forested Vegetation	-0.048 (0.441)
% Wetlands	-0.039 (0.524)
% Forest	0.037 (0.554)

* statistically significant at $\alpha = 0.05$

Of the characteristics examined, the percent of the watershed area classified as exposed was the only significantly correlated characteristic– the larger the percent exposed, the lower the THM levels. All other correlations were not deemed significant, three being positive and three being negative. Surprisingly, wetlands and non-forested vegetated areas had a slight negative correlation. Watershed area seemed to have no influence on THM formation.

Reservoirs filled by small streams/springs and groundwater sources are the best source water types when trying to maintain DBPs within guideline levels. Blending or alternating raw water sources, if possible, is another alternative.

The mixing of surface water with groundwater in the distribution system, however, was found to elevate THM levels, particularly BDCM. As noted in a previous section, when surface water high in DOC is mixed with groundwater with naturally high levels of bromide, and the water is chlorinated, elevated BDCMs above guideline levels result as was seen in the case of Port au Port West and Port au Choix. The practice of mixing ground and surface water when chlorinating should be avoided in future. Shallow ponds used as water sources with long fetch lengths in the direction of prevailing winds are also prone to wave generation and agitation of bottom sediments leading to turbidity. Communities displaying such problems with their surface water source include Cow Head and St. Paul's.

Prior to 2002, Stephenville's drinking water originated from two ponds, Ned's Pond and Noel Pond, which had average THMs of 207.63 and 232.50 µg/L respectively. A well field has since replaced both water supplies and THMs now average 9.3 mg/L. Below is a picture of Ned's Pond in Stephenville taken in June 2006 after water levels in the pond had been lowered with the removal of a retaining structure. All the uncut trees that were flooded to increase the volume of the reservoir were undoubtedly contributing to the organic load and subsequent THM problems with this water supply.

Figure 23: Ned's Pond, former water source for Stephenville, after water levels lowered

Key Messages:

- Water source options and recommendations are conditional on water availability.
- Only surface waters with a DOC level of less than 4.2 mg/L should be used as new source water supplies.
- As long as water demand from all potential users can be met, a surface water source from a smaller sized drainage area should be selected over a surface water source from a larger sized drainage area for all new source water supplies for ease of management of the watershed area.
- Reservoirs filled by small streams/springs and groundwater sources are the preferable source water type when trying to maintain DBPs within guideline levels.
- Groundwater and surface waters should not be mixed in the same distribution system if the only source of treatment is disinfection through chlorination.
- Where a land area is to be flooded to create a surface water reservoir, vegetation must be removed from the area prior to inundation as per permit requirements. Where a vegetated area has already been flooded to create a source water reservoir, water levels should be lowered and vegetation removed if DBP levels warrant.
- Shallow ponds with long fetch lengths in the direction of prevailing winds should be avoided as water sources.

4.4 Relocation of Water Intakes

Intakes must be designed to provide adequate quantities of water under all conditions (low flows, ice conditions), and supply water of the best quality available from the source. Intake structures generally consist of an intake conduit, screen, and a raw water pumping station. On smaller shallow streams a channel dam may be required to provide adequate intake submergence. Inlet anchor cribs are common to elevate the inlet off the bottom where siltation can be a problem. Multiple inlet towers, which permit varying the depth of withdrawal, can also used. Intake depth must try to ensure conduit openings are not clogged by bed-load deposits (silt, sand, gravel, debris) and submersion during extreme low water events. Intake galleries are sometimes installed on small streams and other sources to resolve sediment, flow levels, and icing problems.

Water quality in reservoirs varies with both time and depth. The quality is usually best at mid-depth. Close to the surface water quality is variable due to surface wave action and for brief periods in spring and fall when overturns may occur or when non-point sources of pollution are an issue. The lower water levels of deep impoundments are normally cool and change little in temperature during the year. Surface water varies in temperature with the air and during most of the year is warmer than the lower levels. The water at the bottom of an impoundment is normally low in dissolved oxygen and high in organic matter (McGhee, 1991). The optimal elevation for withdrawal is likely to change during the year.

Most surface water supply intakes in the province consist of an intake pipe extending out into the source water body, located off the bottom of the pond, reservoir, etc. so that material carried in traction will not cover the structure. Most intakes do not extend more than 150 m from the shore (range: 1-200 m) and typically not into the deepest water available. Intake depths are typically not more than 2 m from the water surface (range: 0.5-8 m). In recent years a number of horizontal intake berms extending out into intake ponds that filter water through layers of rock, gravel and sand before being drawn into the intake have also been constructed. Although they might reduce turbidity, such intake filters have a negligible effect on reducing NOM and bromide, the main DBP precursors.

Drawing water from lower levels of the surface water source can reduce water temperatures and potentially reduce THM formation potential. A 10°C fall in water temperature will typically half reaction rates (THM growth, chlorine decay, microbial growth). Water temperatures at the surface can typically vary from 0 to 24°C in Newfoundland and Labrador depending on the season. By installing a deep level intake, it is believed that the peak temperatures could be reduced by as much as 10°C. The following figure illustrates the variation in lake water temperature during the summer where the surface of the lake gets warmer while the bottom layer grows cooler with increasing depth.

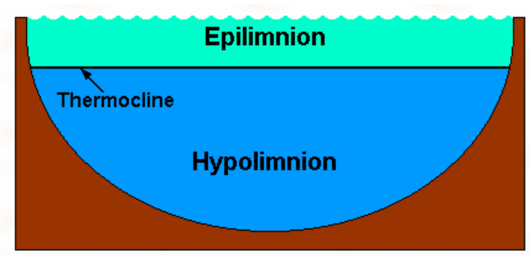


Figure 24: Lake water temperature stratification in summer, surface of lake gets warmer while bottom layer temperature grows cooler with increasing depth

The organic content in source water increases with depth, which may also have an effect on THM formation. It has also been found that in temperate climates, DOC levels in the upper layers of lakes are at a maximum during spring and summer (Singer, 1999) contributing to THM formation potential. It is a reasonable assumption that bromide (and other salts) would increase slightly with water depth as more saline water has a higher density and would sink beneath less dense and fresher surface water.

Extending water supply intakes into deeper water is a viable option for reducing THMs for many surface water supplies located on ponds, lakes and rivers. If feasible, intakes should be located below the summer thermocline, or the separation layer between warm surface water and cooler bottom water. The epilimnion is the top most layer in a thermally stratified lake, while the hypolimnion is the bottom layer. Multiple level intakes to alternate with seasonal changes may also be an option in certain cases. However, more research is required to have a better understanding of provincial surface water behaviour and dynamics for optimal intake location.

Key Messages:

- The optimal type of surface water intake is one that permits varying the depth of water withdrawal to alternate with seasonal changes.
- The intake should be located off the bottom of the waterbody to ensure conduit openings are not clogged by bed-load deposits (silt, sand, gravel, debris), and deep enough below the water surface to ensure submersion during extreme low water events.

- The optimal depth for an intake structure is below the summer thermocline, typically in deeper water, but not at the lowest level in the waterbody.
- Horizontal intake filtration berms have a negligible effect on reducing DBP precursors.

4.5 High Quality Water Storage and Recovery

The principle behind this measure is simply to store high quality surface water during periods of plentiful supply. The water can be treated or untreated prior to storage either in a tank, pond, or groundwater aquifer. When water quality from the main source has deteriorated sufficiently these high quality reserves are then drawn upon, in this context, when DBPs or DBP precursors are likely to peak.

Up-ground reservoirs are mentioned in the *NL Guidelines for the Design, Construction and Operation of Water and Sewerage Systems* as facilities into which water is pumped during periods of good quality and high stream flow for future release to treatment facilities. Aquifer storage and recovery is a management approach used in the US and other dry parts of the world which allows utilities to draw and treat excess amounts of surface water, store the treated water in an underground aquifer, and then draw from the aquifer when raw water volume, contaminants, or precursor concentrations are elevated.

To a certain degree a handful of such systems exist in Newfoundland and Labrador, however, most can be classified as emergency supplies. For example, the communities of Humber Arm South and Daniel's Harbour have higher quality primary supplies, with poorer quality backup supplies on standby. The town of Long Harbour-Mount Arlington Heights has two separate intakes on two separate ponds and switches back and forth when required depending on both water quantity and quality. In the case of Fermuse, St. John's (Windsor Lake) and Corner Brook, water from a secondary source is pumped in to augment the main water source. Overall, this measure has limited potential for reducing DBPs in the province.

Key Messages:

- Where a high quality drinking water source is available either as a primary, secondary, or emergency supply, use of this source should be made to lessen the formation potential of DBPs, especially during the periods of maximum DBP formation potential, typically summer and fall.

4.6 Chlorine Dosage and Application Point

95 % of water disinfection systems in the province use chlorine as the primary form of disinfectant. Typically in Newfoundland and Labrador, disinfection with chlorine is the only form of water treatment. Most distribution networks have the chlorination system located either at or near the intake to couple it with other infrastructure (eg. pump house).

DBP formation resulting from chlorination is influenced by the following treatment variables:

- Application point

- Chlorine dose
- pH
- Water temperature

The provincial standards for bacteriological quality of drinking water have specific requirements for both primary and secondary disinfection. The purpose of primary disinfection is typically to provide for some percentage (eg. 99.9%) inactivation of pathogens prior to water being consumed by the first user on the distribution system. Secondary disinfection is necessary to provide residual protection to prevent growth of biofilms in the distribution system and to maintain disinfection capacity in case of contaminant intrusion at a point on the distribution system. Primary chlorination requires that all water entering a distribution system, after a minimum 20 minutes contact time (and assuming adequate mixing) at peak hourly flow, contain a residual disinfectant concentration of free chlorine of at least 0.3 mg/L at the first point of use or equivalent CT factor value of 6 (based on a mid-range water temperature and pH values for the province). Secondary chlorination requires a detectable free chlorine residual be maintained in all areas of the distribution system. There is some debate over what constitutes a detectable chlorine residual (ranges from 0.02-0.10 mg/L). The commonly used Hach pocket colorimeter has an error range of ± 0.02-0.05 mg/L (at 25ºC) depending on model, year, reagent and other chemical interference (Hach, 2003). To be on the safe side, a detectable chlorine residual should be taken as anything over 0.05 mg/L. Readings within the range of error do not mean that free chlorine is not present, it may be present at higher levels, but caution should be taken. The presence of free chlorine can also be confirmed by testing for total chlorine.

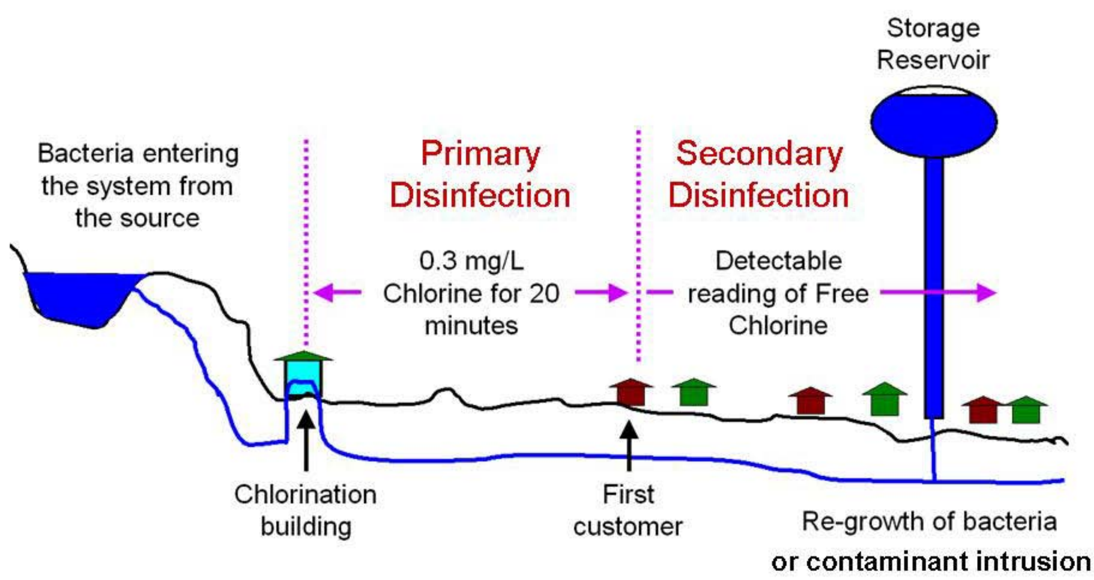


Figure 25: Disinfection of drinking water

A CT value of 6 is considered adequate for chlorine to reduce viral populations to below 4-log or 99.99% removal, however, a much greater contact time is required for the destruction of *giardia*. Inactivation of *giardia* cysts to 3-log or 99.9% removal requires a

CT of 200 or greater. This increased CT value requirement is only an issue when *giardia* is known to have contaminated a distribution system in the past and is not of concern for most communities. The level of chlorine required to inactivate *giardia*, in combination with enumeration techniques and determining viability is still highly unreliable. CT factor values are dependent on the level of micro-organism inactivation (ie. 90% versus 99.999%), pH, water temperature, and other interference factors. CT values should be calculated under worst-case conditions using peak daily flow to determine contact time and minimum observed chlorine residual (within the normal observed range) at the point of interest.

CT factor = residual disinfectant concentration (mg/L) x contact time (min)
Example: 6 mg-min/L = 0.3 mg/L x 20 min

Equation 16: CT factor

In many cases the distance from the point of application of chlorine to the first user on the distribution system is excessive and provides more than the minimum required contact time of 20 minutes. Consideration should be given to optimizing the location of chlorine application so as to provide sufficient but not excessive contact time, which in turn minimizes time for DBP formation. If there is a likelihood of future development (eg. residential, commercial) back towards the chlorination point, this will reduce the available contact time for primary disinfection. Similarly, if there is future development at any point past the first user, this will increase water demand and reduce the available contact time and CT value.

Storage tanks located after the point of chlorination but before the first user will also significantly increase contact times. On systems with a pump and storage tank, chlorine is only dosed to the network when the pump is operating, depending on the tank filling/emptying cycle. Without constant chlorine application, the system has to be super-dosed with chlorine in order to maintain residuals in the network during the tank emptying part of the cycle when the pump and chlorinator are off line. Locating the chlorination system down-pipe of the tank would result in less variation in chlorine residuals and a reduced chlorine dosage, as long as primary and secondary disinfection requirements could still be met.

The typical initial chlorine dose for most systems in the province is around 5 mg/L (typically ranges from 2-15 mg/L). Chlorine dosage is dependent on the amount of chlorine used and the amount of water being treated and is calculated using the following equations:

$$Chlorine\ dose = Chlorine\ demand + Residual\ chlorine$$

Equation 17: Chlorine dose

$$Chlorine, (lbs) = (Hypochlorite, gal)(8.34 lbs / gal)\left(\frac{Hypochlorite, \%}{100\%}\right)$$

Equation 18: Pounds of liquid chlorine used

$$Chlorine\_Dose,(mg/L) = \frac{ChlorineUsed,(lbs)}{WaterTreated,(million-lbs)}$$

Equation 19: Chlorine dose

Optimizing the pH and temperature of water in the distribution system can lead to more productive use of chlorine, and potentially lower THM formation potential (THMs increase with increasing pH and temperature). Surface waters in Newfoundland and Labrador have naturally low pH, which is favourable for disinfection with chlorine. Historical data has also shown that pH adjustment has had little discernable impact on THMs (AwwaRF, 2004). Colder water temperatures and higher pH levels require a higher CT value in order to achieve the equivalent log inactivation of *giardia* or other pathogens in primary disinfection. In theory, this means that communities should be increasing their chlorine dosage in winter. In reality, most communities alter their chlorine dosage in response to chlorine residuals taken in the distribution system, and chlorine is typically increased in spring and fall when there is an increased potential for turbidity from storm water runoff and therefore greater chlorine demand. The City of Corner Brook is one exception, having no current treatment plant, and sufficient system capacity for primary disinfection, the city increases the chlorine dosage in the winter, having suffered through a major *giardia* outbreak in the past.

Water characteristics and chlorine dosage tend to alter seasonally as indicated in the following table. Fall (September-November) is the season that sees the biggest peak in DOC and in chlorine dosage, which contribute to making it the season with the highest average THMs. Adjusting chlorine dosage under different seasonal conditions may help reduce the formation of DBPs.

Table 22: Occurrence of peaks in THMs and THM precursors

	Spring	Summer	Fall	Winter
pH		peak		
Temperature		peak		
DOC/ Colour			peak	
Chlorine Dosage	peak		peak	
THMs			peak	

There is no database on community chlorine dosage rates in the province, however seasonal averages of free chlorine in the distribution network indicate that chlorine residuals are highest in winter and lowest in the spring as shown in the following figure. These results most likely indicate that chlorine demand is actually highest in the spring.

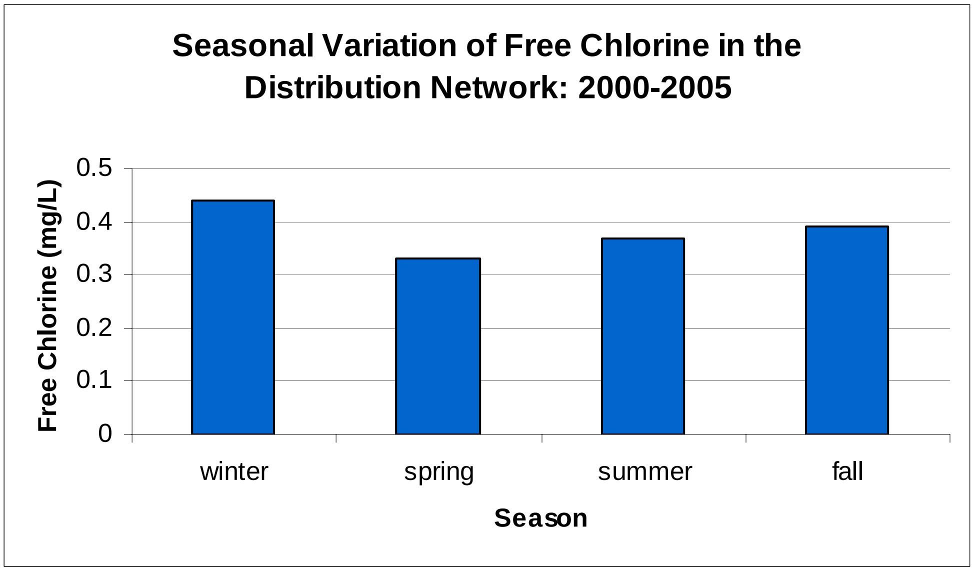


Figure 26: Average seasonal variation of free chlorine in NL distribution systems

Balancing chlorine dose with chlorine demand and the requirement to achieve sufficient primary chlorination and adequate residuals throughout the distribution system can be extremely challenging for many communities. The USEPA guidelines require that chlorine residuals be below 4 mg/L for all water consumers. Chlorine residuals above this level can cause known or expected health risks such as eye nose irritation and stomach discomfort. This standard should also be used as a guide in this province. If chlorine residuals at the beginning of the system exceed this value, the chlorine dose should be reduced and other options like booster systems to maintain residuals investigated. Likewise, if chlorine residuals at the end of the distribution system habitually exceed 0.1 mg/L, there is potential for reducing the chlorine dosage. Chlorine dose needs to be optimized, as there is a direct relationship between chlorine use and DBP levels.

Key Messages:

- The province should adopt a maximum residual disinfectant level for chlorine of 4.0 mg/L for all water consumers. Chlorine residuals above this level can cause known or expected health risks such as eye nose irritation and stomach discomfort.
- A detectable free chlorine residual should be considered anything greater than or equal to 0.05 mg/L unless accompanied and confirmed by a total residual chlorine test. A free chlorine residual of 0.02 mg/L may be acceptable if total chlorine residual confirms presence and removes the possibility of tester error.
- A contact time or CT value for inactivation of *giardia* should only be used when the distribution system has experienced a previous *giardia* contamination event and relies on chlorine disinfection as its only form of treatment.
- The chlorine dosage should be kept as low as possible while still maintaining required primary and secondary disinfection objectives. If chlorine residuals at all points (particularly end points) in the distribution system are typically over 0.1 mg/L, there is potential to reduce the chlorine dosage to achieve "detectable" levels.

- The application point of the chlorine dose should be as close to the first user as possible while still achieving the required primary and secondary disinfection objectives.
- A buffer above the minimum contact time and CT value should be incorporated into the required primary disinfection objectives for chlorine to take into account future developments either down-pipe or up-pipe of the design *First User*. The buffer should not exceed 2-10 times the minimum contact time or CT value.
- Consideration should be given to locating the point of chlorination after water storage tanks in systems where a sufficient contact time or CT value is available. This may increase system maintenance requirements.
- Once an optimal point of chlorination has been identified based on an established *First User* location, future residential, commercial, institutional or industrial development up-pipe of this *First User* site should be restricted.
- Calculation of CT values and contact time is important for system design purposes and should be reviewed regularly with each season and with any new developments on a distribution system. For everyday purposes, chlorine residual readings taken from the distribution system should be used to determine if any alteration in chlorine dosage is warranted.
- For Calculation of the CT value, worst-case scenario conditions should be evaluated: the contact time at peak daily flow should be used, and the minimum observed chlorine residual (within the normal observed range) at the first point of use for the period of interest.
- THMs in the province tend to peak during the fall and are relatively high during the spring and summer in response to peaks in THM precursors. THMs are at their lowest during the winter. Chlorine demand is at its highest during the spring and at its lowest during the winter. Adjusting chlorine dosage, or targeting the use of other specific corrective measures during periods of highest THM formation potential or highest chlorine demand may help reduce formation.
- Where removal of DBP precursors is not possible, practical or affordable, lowering the chlorine dosage (while still maintaining required primary and secondary disinfection objectives) can be used as a first response to high DBP levels.

4.7 Booster Chlorination Systems

A booster chlorination strategy involves multiple coordinated doses of chlorine applied throughout the distribution system. Once primary disinfection has been achieved, it does not need to be re-achieved with the booster system. Booster or satellite chlorination systems are used in water distribution networks in order to (Uber, 2003):

- maintain chlorine residuals towards the end of the distribution network
- reduce an unacceptably high chlorine dose required from a single chlorination system to maintain adequate residuals throughout the network
- reduce the total amount of chlorine used in the system per day
- reduce chlorine fluctuation at sites throughout the distribution network
- increase operational flexibility for maintaining chlorine residuals in the network as usage characteristics change over time
- potentially reduce exposure to chlorine disinfection by products

There are currently several booster chlorination systems located on different distribution networks throughout the province. In theory, if you reduce the overall chlorine dose to a distribution network using booster systems, you can also reduce THM formation potential. However, recent research has indicated that THM formation under booster conditions shows no long-term reduction. The only reduction in THM concentration was found to be prior to the boost dose between the source and the booster station. After the application of the boost dose, THM concentrations reached the level of an equivalent single dose (AwwaRF, 2006).

To determine what effect chlorine booster systems have had on THM levels in the province, an ANOVA analysis of THM levels prior to and post installation of a booster chlorination system was performed with results summarized in the following table.

Table 23: ANOVA of mean community THMs pre/post chlorination booster

Community	Region	N pre booster/ N post booster	THM mean pre booster (ug/L) (± StDiv)	THM mean post booster (±StDiv)	p-value
Cartwright	L	4/3	236.8 (± 161.1)	356.0 (± 110.1)	0.324
Ferryland	E	18/5	204.0 (± 127.4)	177.5 (± 37.5)	0.776
Lewisport	C	59/3	130.5 (± 38.0)	153.7 (± 40.6)	0.308
Corner Brook	W	2/16	101.5 (± 4.95)	167.2 (± 37.2)	0.027*
Come by Chance	E	21/15	70.0 (± 29.0)	134.6 (± 60.7)	0.000*
Cupids	E	16/13	70.3 (± 33.8)	71.5 (± 20.2)	0.917
Harbour Main	E	15/15	62.9 (± 18.4)	115.4 (± 48.0)	0.000*
Little Catalina	E	27/16	161.2 (± 91.6)	103.9 (± 80.7)	0.045*
Torbay	E	34/8	43.0 (± 21.9)	105.7 (± 44.4)	0.000*
Trinity Bay North	E	28/16	153.6 (± 93.8)	118.2 (± 83.1)	0.218
Whiteway	E	20/16	84.1 (± 53.1)	117.8 (± 52.9)	0.067
Whitbourne	E	15/16	91.9 (± 43.0)	62.7 (± 31.2)	0.038*
New-Wes-Valley (Westleyville)	C	18/20	94.1 (± 33.1)	145.0 (± 72.9)	0.010*

* statistically significant at $\alpha = 0.05$, N is sample size

Of the 13 communities with boosters, 7 showed statistically significant differences in mean THMs pre and post installation of the chlorine booster. In the case of Little Catalina and Whitbourne, there was a significant reduction in THM means. In the case of Corner Brook, Come by Chance, Harbour Main, Torbay, and New Wes Valley, there was a significant increase in THM means. Of the 6 communities where no significant difference was found between THM means pre and post installation of chlorine boosters, 4 still showed an increase in the mean THM value, while 2 showed a decrease. Taken as a whole, chlorine boosters tend to aggravate the THM problem in most cases. However, in some instances they appear to have reduced THM levels. The success of booster chlorination in reducing THMs may be a factor of the degree of monitoring and level of control by the operator.

The large number of communities throughout the province with high levels of THMs is in part a reflection of the growing number of communities now regularly chlorinating their water supply in accordance with government standards for bacteriological quality of drinking water. With the installation of new booster systems, communities have tended to become more diligent about chlorinating their water systems properly, and as a consequence chlorine use has actually increased. This explains the higher THM levels observed pre and post installation of chlorine boosters. Communities who have confirmed using more chlorine include: Cartwright, Torbay, and New Wes Valley. Communities who have confirmed using less chlorine include: Ferryland, Trinity Bay North, and Little Catalina. Communities where chlorine consumption had remained the same include: Cupids. Communities where no assessment on chlorine consumption could be made include: Come by Chance, Whitbourne, Corner Brook, Lewisport, Harbour Main and Whiteway due to the unavailability of information.

The US EPA has a maximum residual disinfectant level for chlorine of 4.0 mg/L. The only potential a chlorine booster has for reducing DBPs is if the total combined chlorine dose is less than the chlorine dose from a single chlorination system. Practically speaking, installing chlorine boosters as a measure to reduce DBPs is only an option when the initial chlorine dose is unacceptably high and/or the overall chlorine dose can be reduced.

Key Messages:

- Chlorine boosters have limited application for reducing DBPs, and should only be used for this purpose where the initial chlorine dose is high (over 7 mg/L) or when the residual reading at the first point of use is over 4 mg/L. The only potential a chlorine booster has for reducing DBPs is if the total combined chlorine dose from primary and booster chlorination systems is less than the chlorine dose from a single primary chlorination system.
- Water distribution systems with existing booster chlorination systems need to optimize their chlorine dosages so as to minimize overall chlorine use.

4.8 Chlorine Residual Feedback Control

Most communities in the province have single chlorination systems located at or near the water source with manual control over chlorine dosage based on continuous feedback of manually measured distribution system chlorine residuals. As already explored in a previous section, the source may not be the best location for the addition of chlorine to maintain residuals throughout the entire distribution system. A large dosage may be required at the source in order to maintain minimal residuals at the system periphery. Large fluctuations in chlorine residuals (or conversely chlorine demand) can occur on a daily, weekly and annual basis. Typically, the standard deviation in chlorine residuals decreases with increasing distance from the point of chlorination, indicating greater fluctuation in residuals observed in the beginning of the system than at the end. Fluctuations can be caused by the filling and draining of storage tanks, leaks in the distribution system, changes in temperature, water demand and source water quality. Responding to such fluctuations manually, without on-line feedback of either chlorine residuals or (in many cases) system flows, is the only option for most communities. As a safety factor, common practice is to then use excessive amounts of chlorine.

An intelligent on-line system coupled with properly located actuators and sensors can lead to more reliable chlorine regulation potentially reducing the total chlorine dosage (AwwaRF, 2003). Automatic regulation of distribution system chlorine residuals can also incorporate the use of booster systems. Optimizing chlorine usage through feedback control has good potential for reducing THM formation potential; however, practically speaking this control measure may only be a viable option for larger communities with dedicated and well-trained water system operators and well-maintained systems.

The methods of chlorine dosage control common in the province include:

- Uncontrolled chlorination systems- manual with no flow meter
- Non-automatic flow proportional control- manually varying the rate of chlorine feed in proportion to the flow as determined visually by a metering device or based on average pumping capacity when the pump cuts in
- Open loop flow proportional control- automatic variation of the rate of chlorine feed in proportion to the flow as determined by a metering device
- Closed loop flow proportional control- water quantity (metering device) and quality (chlorine residual analyzer) feedback controls chlorine feed

Manual or automatic flow proportional chlorine control requires a flow meter measuring the water volumes being drawn through the distribution system. According to Department of Environment and Conservation OETC infrastructure records, only 56% of 532 public water distribution systems are equipped with flow meters as indicated in the following figure. Experience in the field indicates that up to 50% or more of installed flow meters may not be functioning properly. Problems with flow meters include general lack of maintenance, improper installation, improper calibration, air in the water mains interfering with readings, inability of meters to read low flows, and corrosion if chlorination systems are placed upstream. The main types of flow meters in use are turbine meters, Mag meters and ultrasonic meters, although there are very few of the latter in the province.

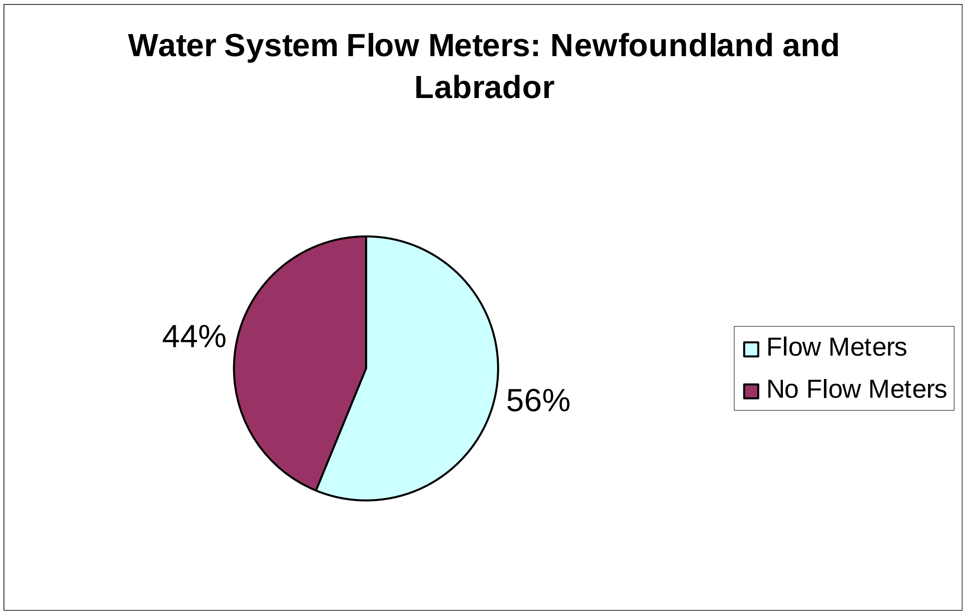


Figure 27: Percentage of NL public water distribution systems with flow meters

Not enough information was available to be able to determine percentages of the other chlorine control methods, although manual and automatic flow control is thought to dominate. Chlorine control through feedback from a single chlorine residual analyzer has been used recently on a number of systems with limited success. Typically, only large systems have combined automatic flow and residual analyzer control of chlorine dosage. St. John's has closed loop chlorine control with combined feedback from chlorine residual analyzers and flow meters. St. Paul's is one smaller community with a residual analyzer located in the water storage tank that maintains chlorine residuals at 2 mg/L before water leaves the tank for the community.

Once the chlorine analyzer detects chlorine levels have fallen below a pre-set level, a signal is sent back to the chlorination system and the dosage is increased accordingly. Problems have arisen, however, with widely fluctuating residuals and improperly calibrated residual analyzers. At any site on a distribution system, chlorine residuals will fluctuate, however a chlorine residual peak that occurs at the beginning of the system will only occur at the end of the system after some lag, as it takes time for that particular plug of water to travel through the distribution network. Therefore, chlorine control using a fixed residual analyzer can only optimize chlorine levels at a specific point, with mixed results elsewhere on the system.

Automatic flow control systems are set up to increase the chlorine dosage with increased water demand in order to maintain required CT values. With increased demand, water moves faster through the system, water age is reduced, chlorine residuals are actually higher, and there is less time for DBPs to form. However, the net product of contact time and residual disinfectant concentration is typically unchanged. Potentially, the advantage of increased demand and reduced residence time may be counteracted by the increased chlorine dosage in terms of DBP formation.

Key Messages:

- Manual chlorine residual readings should be collected from multiple points on the distribution system on a daily basis as per Permit to Operate requirements. Values should be recorded and archived.
- Fluctuations in chlorine residuals at a fixed location can be the result of the filling and draining of water storage tanks, leaks in the distribution system, changes in water temperature, water demand, and source water quality. Fluctuations in residuals are typically greater in the beginning of the system than at the end.
- All water distribution systems should be equipped with a flow meter. Communities should take regular flow meter readings (at least once a week), with values recorded and archived. Flow meters should be properly sized, sited, installed, maintained and calibrated.
- All communities using chlorine for disinfection should be equipped with a field chlorine test meter.
- As a minimum, all communities disinfecting with chlorine should use flow meter readings and manual chlorine residual readings in order to make decisions concerning chlorine dosage control.
- Combined automated flow and residual analyzer control of chlorine dosage should only be considered for large communities or communities with dedicated and well-trained water system operators and well-maintained distribution systems.
- Chlorine residual feedback controls have limited application for reducing DBPs.
- Chlorine control using a fixed location residual analyzer can only optimize chlorine levels at a specific point, with mixed results elsewhere on the system.
- Automated flow and/or residual analyzer controls should not be installed with the expectation that they can replace water distribution system operators, or negate the need for manual chlorine residual readings.

4.9 Modify Tank Operation or Configuration

Water storage tanks can serve a number of different purposes on a distribution system including: system demand equalization, system balancing, providing residence time, emergency flow, fire flow, pressure surge relief, water blending, pressure head if elevated, and contact time for disinfectants to inactivate pathogens.

According to Department of Environment and Conservation records there are 75 public water supplies with water storage tanks in the province. The majority of existing tanks can be classified as either standpipe or on ground as indicated in the following figure, share the same draw/fill main, and are pressure controlled. Most storage tanks in the province have problems with poor mixing of water and dead zones.

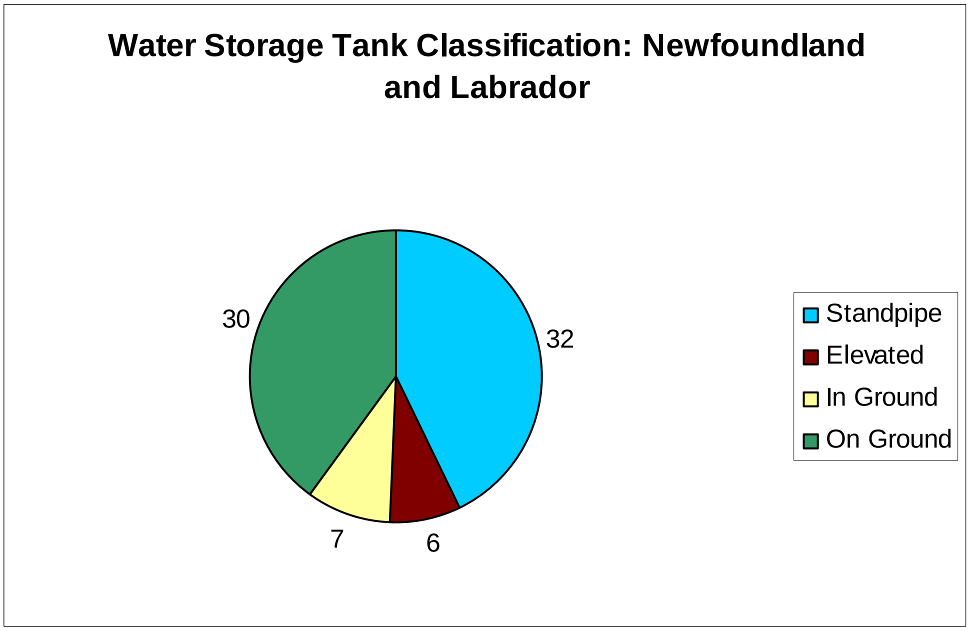


Figure 28: Water storage tank classification in Newfoundland and Labrador

An ANOVA analysis was performed to see if there was any difference found in THM levels from serviced areas with water storage tanks and those without. A significant increase in mean THMs between water distribution systems with tanks and those without was detected as indicated in the table below.

Table 24: ANOVA of mean community THMs with storage tank presence as analysis factor

Presence of Storage Tank	Number of Watersheds	Mean THM (Standard Deviation)	p-value
No Tank	318	62.3 (±59.72)	0.046*
Tank	69	78.4 (±63.59)	

* statistically significant at $\alpha = 0.05$

Lack of water turnover in storage facilities has long been recognized as a primary cause of water quality problems within a distribution system. Disinfectants have more time to react with compounds in the bulk water in storage tanks with dead zones, low water turnover rates or poor circulation. These effects can generally be reduced by proper design and operation of storage facilities, such as appropriate tank sizing, inlet/outlet configuration, mixing and operational schedule.

Ideally, tanks should be fully mixed, but in practice, this is rarely the case and there will be pockets of water which are not well mixed with the bulk of the water resulting in stagnant zones where the water age can be considerably higher than the average age of water in the tank. Since retention time is directly proportional to storage volume, it is important to avoid unnecessary storage. Reducing the volume of storage can be a relatively cheap and simple method of achieving significant reductions in retention time, thereby increasing turnover and reducing the risk of stagnation. However, any reduction in storage must be balanced against the need to provide security of supply and sufficient pressure and volume for fire fighting. When storage is primarily provided for pressure, use of elevated tanks (rather than standpipes) having a smaller storage capacity is recommended.

The location of the tank on the distribution system can also affect chlorine residuals, water age and DBP levels. The majority of tanks in the province are located at the beginning of the distribution system, after the chlorinator and before the first user. Tanks located at the end of the distribution system tend to increase water age in the tank and the distribution network, and increase the variability of chlorine residuals throughout the system.

4.9.1 Adjusting Pump Schedules

Altering pump schedules can be used as a control method to reduce residence times in storage tanks by several different methods:

- Enabling reduction of storage volume by optimizing the balance between supply and demand.
- Increasing the daily variation in water level in the tank can force turnover of water in the tank.
- Increasing the pumping rate for a short period each day can increase the velocity at the inlet and thereby improve mixing in the tank

One of the purposes of storage is to balance the variation in water demand from consumers supplied by single speed pumps, which are either on or off. It is common practice to operate pumps during periods of low demand, with system storage meeting the demand at other times. By adjusting the pumping regime, it is often possible to improve the balance between network demand and the supply from the pumps and thereby reduce the volume of storage required. For tanks with separate inflows and outflows located on opposite sides of the tank, the through flow forces turnover of water in the tank and the greatest benefit will be achieved by minimizing the volume of storage through this method.

Changing the water level in the tank not only forces water in and out of the tank, but also changes the mixing patterns within the tank reducing the likelihood of stagnant water remaining in the tank. This is particularly beneficial for tanks with a common inlet/outlet (ie. standpipe design).

Where the flow is controlled by variable speed pumps or multi-pump installations, it may be possible to increase the pumping rate for a short period each day so as to increase the velocity at the tank inlet and improve mixing. Such a measure would require engineers check that the transmission main has been designed to run at the proposed pumping rate, the adequacy of existing thrust blocks and surge control.

4.9.2 Reducing Storage Capacity

Removing surplus capacity from the distribution system is a simple and effective means of reducing retention time when there are no issues with supply or pressure. Reducing the maximum water level in the tank is one way to achieve this. This method of altering storage volume can be varied with seasons providing more storage in the network when demand is high or it can be reduced permanently. Alternatively, if storage is not required, consideration should be given to removing it entirely.

4.9.3 Reducing Stagnant Zones

Storage tanks should be designed and configured so as to prevent pockets of stagnant water. Altering the configuration or internal geometry of the storage tank can promote greater mixing thereby reducing the maximum retention time. Methods can include:

- replacing a common inlet/outlet with separate pipes
- installing baffles
- moving the location or orientation of the inlet
- increasing the distance between the inlet and outlet
- reducing the diameter of the inlet
- installing a duckbill valve to increase the velocity of the inlet jet
- install paddle or impellor devices to improve mixing within the tank

Tanks with a common or closely located fill/draw pipe are liable to turn over water only in the vicinity of the inlet/outlet leaving a large dead zone. Only the net flow (difference between supply and demand) passes into a tank of this kind, and this quantity may be low, meaning little new water in the tank. Installing the draw line on the opposite side of the tank to the inlet forces water to flow across the full width of the tank. Altering the level of the inlet may also be beneficial. With circular tanks, it is more desirable to have the inlet in the center of the tank so that water flows our radially in all directions. Tank behaviour with common and separate fill/draw lines are indicated in the following figure.

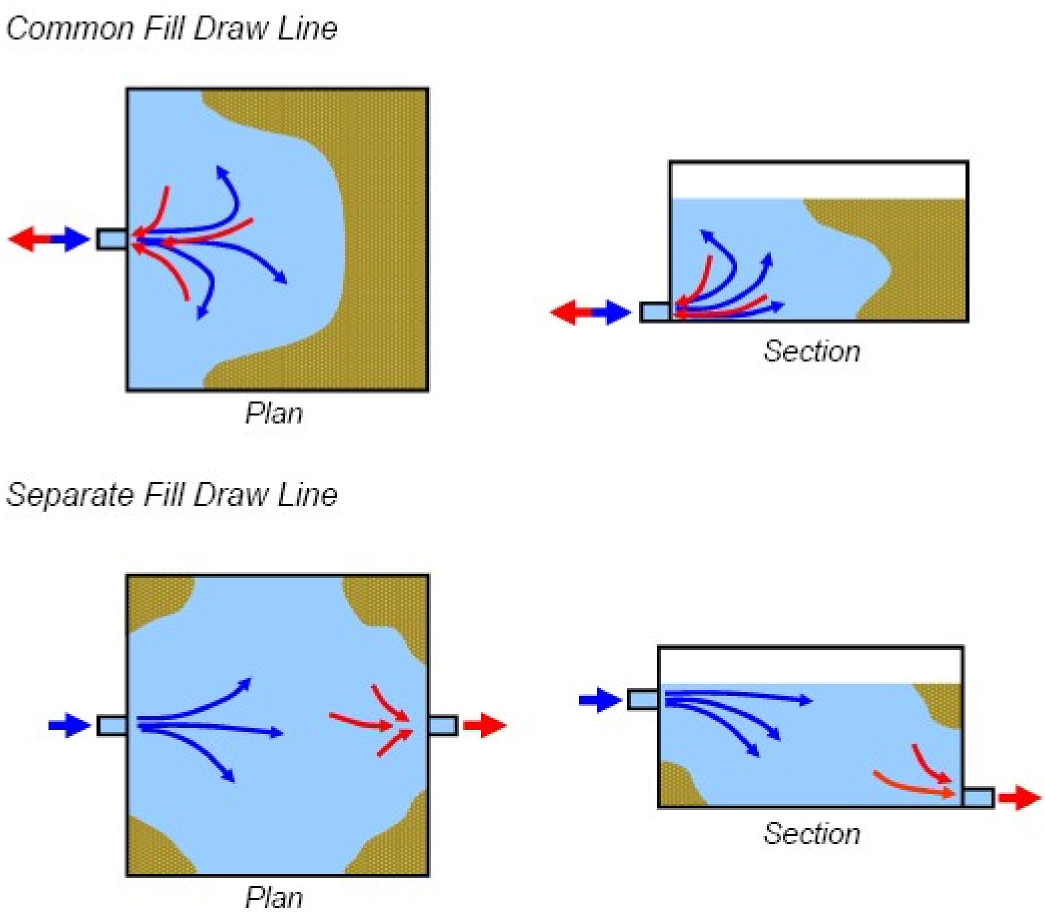


Figure 29: Tank configuration with same and separate fill/draw lines

In cases where the fill/draw main leading to the tank is of considerable length, duplicating this stretch of pipe may not be viable (too expensive) and promoting better mixing within the tank the preferred option.

Fitting baffles into tanks can direct water through regions of a tank that would otherwise have poor turnover. Optimum configurations can be difficult to determine, however. Baffles, walls and other obstructions in the path of the inlet jet tend to dissipate the strength of the inlet jet and so generally have a detrimental effect. Columns or other obstructions cause resistance to the flow of water resulting in stagnant zones. Columns in circular tanks have been observed to cause water to swirl around the perimeter of the tank. Water behaviour in tanks with obstructions and baffles is indicated in the following figure.

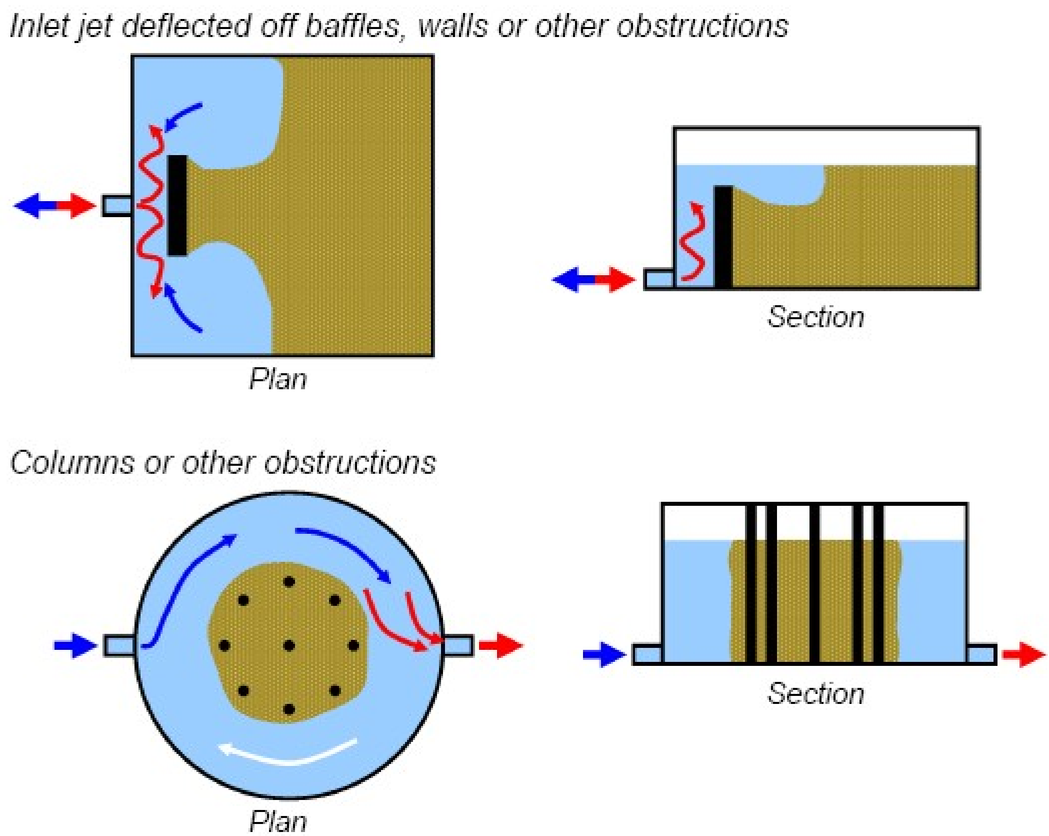


Figure 30: Tank configurations with obstructions and baffles

The energy of the inlet jet can be used to stir the water in the tank. Altering the angle of the inlet jet can have a marked effect on mixing. Tangential inlets tend to promote a flow path around the perimeter of the tank resulting in a stagnant zone in the center as indicated in the following figure. This is most likely to occur in circular tanks, although it can happen to a lesser extent in rectangular tanks.

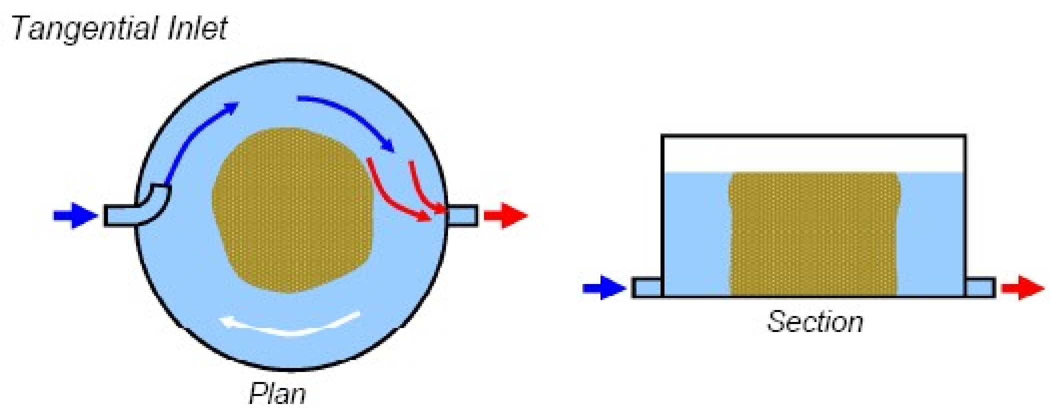


Figure 31: Tank configuration with tangential orientation of inlet

Reducing the diameter of the inlet pipe will increase the velocity and kinetic energy of the water entering the tank and improve mixing. Duckbill valves can reduce the size of the inlet under low flows and thereby increase velocity. In tall narrow tanks, there is a tendency for poor turnover of water at the top of the tank. Directing the inlet jet upwards and ensuring it is powerful enough to mix water throughout the tank can alleviate this problem. Alternatively, installing a high level inlet will ensure that water is forced to flow from top to bottom throughout the full depth of the tank. Water behaviour in standpipes and duck-bill valves are illustrated in the following figure.

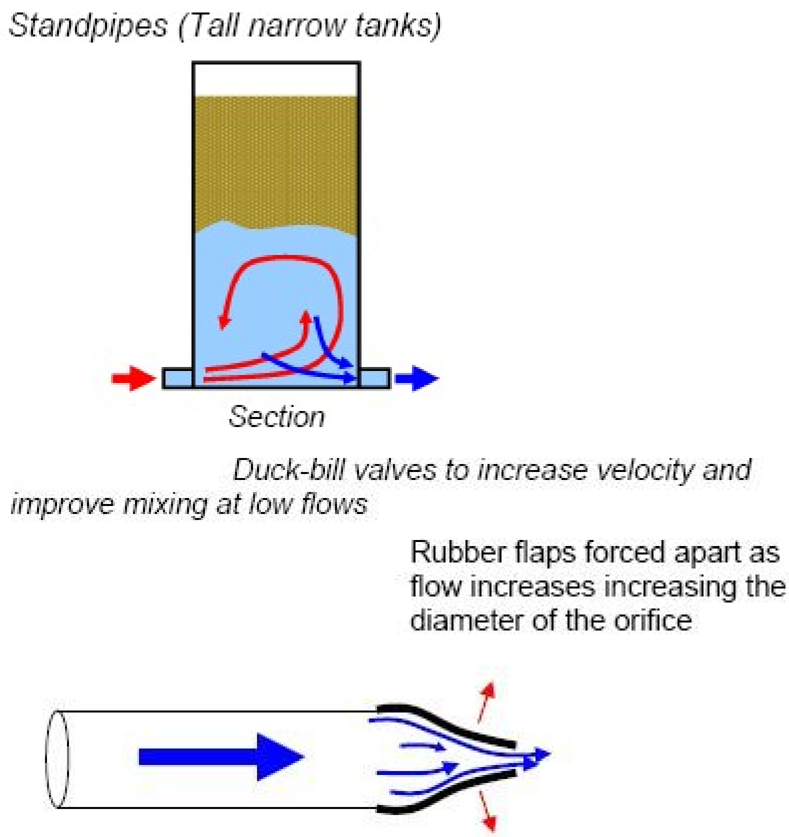


Figure 32: Increasing velocity at tank inlet

Paddles and impellors can also be installed in tanks as a mechanical means of mixing water and preventing stagnant zones. There are a number of commercially available devices on the market some of which are solar powered or can incorporate secondary disinfection dosing equipment.

No matter what type of storage configuration, some pockets of stagnant water may still occur. Stratification can also be an issue with storage tanks. Cold water is denser than warm water and will sink to the bottom of the tank (water reaches peak density at 4°C). If the water flowing into a tank is significantly colder than the general water temperature in the tank, the cold inflowing water will sink to the base of the tank with the warmer, older water floating on top. Relatively little mixing will occur between the warm (old) water and the cold (fresh) water. Alternatively, stratification can also occur when the inflow is significantly warmer than the general water temperature in the tank. In this case the warmer fresh water will float to the top of the tank leaving a body of cold and older water at the bottom of the tank. This is most likely to occur in winter with above ground steel tanks. During the spring and fall, water will also turnover in the tank due to temperature differentials. Turnover of water in tanks (and even in surface water ponds, reservoirs, lakes) has also been associated with short periods (1 or 2 days) of increased turbidity in the distribution system due to agitation of sediments in the tank. Stratification is a particular problem with tall, narrow tanks built above ground. Water behaviour in tanks due to temperature stratification is illustrated in the following figure.

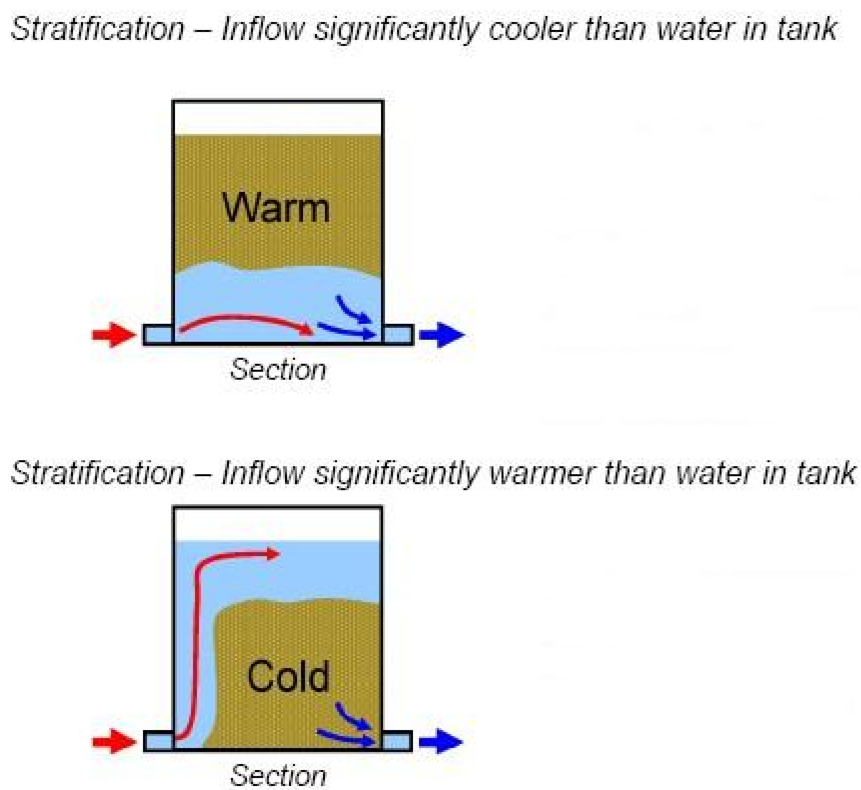


Figure 33: Stratification of different temperature water in tanks

4.9.4 Tank Aeration

Aeration is the process by which air is circulated through, mixed with, or dissolved in a liquid substance. Some disinfection by products, such as THMs, are volatile compounds that can be reduced through aeration. In Suisun City, California, an aeration system was installed in one of the water system's storage tanks which saw a reduction in THMs of 70% (Walfoort et al., 2008). The aeration system comprised of a recirculation pump that drew water from the bottom of the tank and discharged it into the atmosphere via a spray nozzle above the water surface. The cost of the system was highly economical. Aeration of chlorine disinfected water, however, can result in a reduction of chlorine residuals and potential failure of secondary disinfection requirements.

Figure 34: Aeration system in water storage tank

Key Messages:

- Water storage tanks contribute significantly to DBP levels in a distribution system due to dead zones, low water turnover rates, and poor circulation. These effects can generally be reduced by proper design and operation of storage facilities, such as appropriate tank sizing, inlet/outlet configuration, mixing, and operational schedule.
- Storage tank volumes should be minimized to avoid unnecessary storage. Stored water volumes should be optimized to meet requirements for storage, pressure and volume for fire fighting.
- Where the main purpose of a water storage tank is to provide pressure to the water distribution system, elevated storage tanks should be used as opposed to standpipe tanks.
- Tanks located at the beginning of the distribution system tend to reduce overall water age in the tank and distribution network, and reduce variability in chlorine residuals.
- The balance between supply from the pumps and network demand should be optimized in order to reduce the volume of storage required.
- Variation in water level in the tank should be maximized to force turnover of water in the tank.
- Systems with variable speed pumps or multi-pump installations can be configured to increase the pumping rate for a short period each emptying/filling cycle so as to increase the velocity at the tank inlet and improve mixing.
- When there are no issues involved (with supply, pressure or CT value), absolute storage capacity on a distribution system can be reduced by taking storage tanks off line or reducing the maximum water level in a tank.
- Tank design must incorporate the need for greater mixing through replacing a common inlet/outlet with separate pipes, installing baffles, moving the location or orientation of the inlet, increasing the distance between the inlet and outlet, reducing

the diameter of the inlet, installing a duckbill valve to increase the velocity of the inlet jet, or installing a paddle or impellor devices to improve mixing within the tank.

- Stratification is a problem with tall, narrow tanks built above ground.
- According to design guidelines, the maximum allowed water retention time in a storage tank is 72 hours. Water retention times in storage tanks should be minimized.
- According to design guidelines, changes in water level in a storage tank during daily domestic water demand should be limited to a maximum 9 m.
- For water storage tanks with long residence times, aeration systems can be used to strip volatile DBP compounds from the water. With the installation of a water storage tank aeration system, consideration must be given to the resulting loss of chlorine residuals.

4.10 Modify Distribution System Operation and Configuration

When looking at the distribution network, it is necessary to consider both the retention time and the condition of the pipe network. Network solutions should always be targeted towards those pipes or sections of the system which are most responsible for contributing to the problem. The formation of THMs is unique within each distribution system, but as a general rule, half the expected ultimate formation potential is likely to be achieved 10-24 hours after chlorination. Ultimate formation potential can typically be reached anywhere from 24-200 hours after chlorination.

System Flushing

System flushing can be achieved by:

- Increasing demand
- Manual flushing
- Automated flushing

Retention time is directly controlled by water demand. In recent years there has been a move towards greater water conservation by the water resources sector to help preserve the resource, an aim diametrically apposed to the concept of deliberately increasing water demand by artificial means. Water demand need not be increased artificially, however. In communities with newly developing areas, there may be some flexibility about the location of new water connections so that increased demand is placed on areas of the network with high retention times. While water conservation should be promoted wherever possible and wasting of water avoided, appropriate system operation and maintenance practices (bleeding, flushing) should take precedence.

Periodic flushing of distribution systems can remove sediment built up in pipes, reduce water age in dead ends, increase water velocities in sections of pipe, and be used to obtain higher disinfectant residuals. Manual flushing is often used as the first remedial measure following a water quality failure (microbial, discolored water, low chlorine). The primary purpose of this method is to expel contaminated water. Some utilities have weekly flushing programs to control water quality in problem areas. A major limitation of using flushing as a means of controlling retention time (and by extension chlorine residuals or THM levels) is that the period between flushing must be less than the return period of a water quality violation event. For many systems this may require weekly or even daily flushing which is expensive in terms of manpower and wasteful of resources. For example, if a distribution network with high THM levels has a maximum retention time of only 24 hours, flushing would be required at least twice a day to have any effect in reducing THM levels.

The Department of Environment and Conservation encourages communities to flush their distribution systems periodically (once or twice a year) as a BMP. Flushing requirements are also part of the Permit to Operate issued to communities by the province. In communities that have performed proper flushing of their distribution system (eg. Burgeo, Gander, St. Lunaire-Griquet), very turbid water and in some cases solid biofilm cake has been observed in the flushed effluent. From anecdotal information, after flushing, total chlorine dosage has dropped (in one case by more than 65 percent), and free chlorine residuals have improved in the distribution system.

Figure 35: Water flushed from the St. Lunaire-Griquet distribution system during flushing (left), after flushing (right)

Options are also available for the automatic programmed flushing of distribution systems to remove sediment and reduce water age in dead ends and low velocity sections of pipe. Automated flushers purge the system at regular intervals either by discharging water to a sewer, watercourse or to surrounding ground. The use of automated flushing can be complicated by the volumes and value of water wasted, difficulties in disposing of water in urban areas or in freezing conditions, issues with vandalism or tampering if installed in public areas. Automated flushing devices are best suited to rural networks. In long systems terminating in dead ends there may be few other alternatives to flushing for controlling retention time. An alternative to automated flushing sometimes practiced is to have the system continuously bleed water at a dead end. This will prevent stagnation, but again there are problems with wastage and disposal.

Automated flushing units are typically either special flushing units or hydrant flushing attachments that connect to standard fire hydrants as shown in the following figure. Assessment of flushing duration, rates and locations would need to be carried out, and not all flushing products are designed for freezing conditions.

Figure 36: Hydro-Guard automatic flushing devises

Under the federal *Fisheries Act*, it is illegal to discharge chlorinated water (with a total residual chlorine level greater than 0.02 mg/L) into a receiving surface water body (fresh or marine). This restriction needs to be considered with any water distribution system flushing program.

4.10.1 Altering Valving of Networks and Recirculation

By changing valve arrangements and hydraulic boundaries, travel times can be reduced and water rerouted to increase velocities in low flow pipes. Retention times are often highest at dead ends or valves, which have been shut to create internal boundaries. It is possible to reduce retention time in localized parts of a distribution network by changing valve arrangements. Minimizing the number of shut valves required to produce a hydraulic boundary, and locating valves in areas with relatively high demand on either side of the shut valve can reduce retention times except in systems with linear flow in long ribbon development and one main line.

Shut valves can also be used in a network to re-route flows through parts of the system with low demand and high retention time where otherwise such flows would pass directly to points of high demand as shown in the following figure. Although the age of water may be increased at the point of high demand, other parts of the network will benefit from higher turnover. In highly diffuse systems with no easily controllable flow path, any re-routing would require the installation of numerous closed valves, which are generally detrimental to water quality.

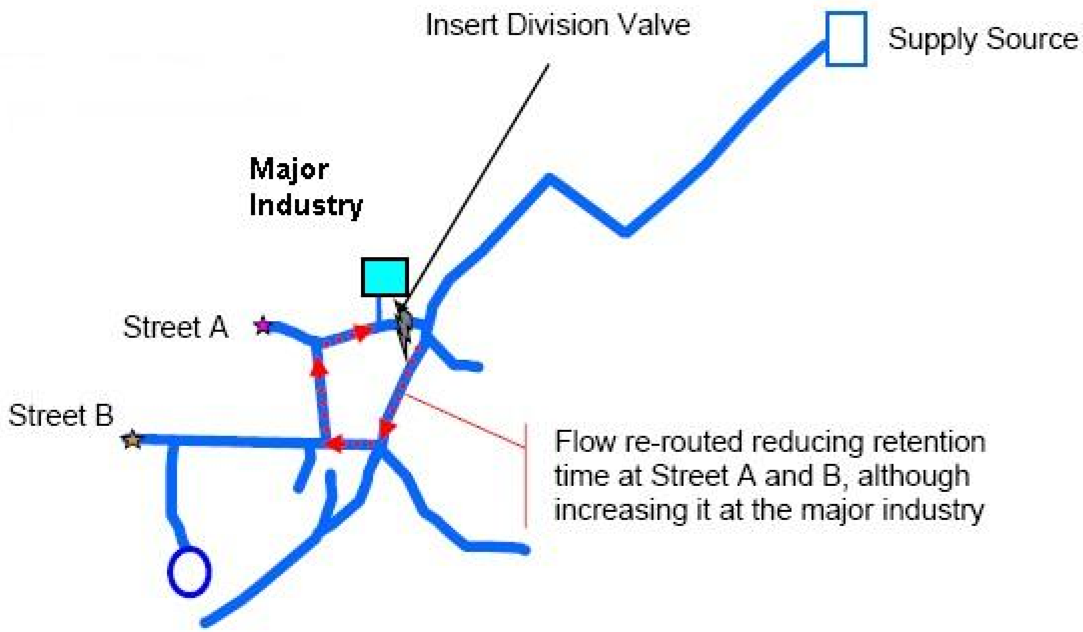


Figure 37: Insertion of a gate valve to re-route flow in distribution system

In some cases, pumping water from one zone in a distribution system to another in order to re-circulate water can be used to reduce overall peak retention times.

4.10.2 Abandoning or Downsizing Mains

Abandoning mains involves removing surplus capacity from the system, while downsizing mains entails reducing overall system capacity to increase water velocities and reduce retention times. Many distribution networks have been steadily added to over the years, invariably resulting in a non-ideal and often haphazard network. Abandoning sections of poor condition and obsolete pipes may alleviate water quality problems, even if it does not significantly reduce overall water age in the network.

Most distribution networks have been designed to meet a minimum hydraulic capacity. Additional capacity is generally built in at the design stage to accommodate future growth, fire flows, and allow more flexibility in the configuration of the network. Most water distribution networks in the province were designed and installed at a time when future population growth was expected to be steady or increase. The reality is that the province's population has been declining for the past 25 years due to low birth rates and out-migration, particularly from rural areas. The average rate of population change in communities throughout the province over the period from 1996 to 2006 is -1.7%.

A utility may also have a policy to limit the number of different pipe diameters within the system in order to simplify construction and maintenance. The smallest diameter pipe a fire hydrant can be fitted to according to provincial guidelines is 150 mm. Consequently, on systems designed to accommodate fire flows, network pipes tend to be larger than necessary to meet the daily demand from the network, leading to increased retention times. Larger mains may be replaced with smaller diameter pipes and still achieve the required hydraulic capacity. Old pipes, particularly unlined cast iron pipes should either be replaced or relined.

In the Netherlands, some utilities have started to design new networks specifically with the intention of minimizing retention time (AWWA, 2006). The Dutch design criteria have been revised so that pipes are sized to achieve a daily peak velocity of at least 0.4 m/s. This velocity is considered sufficient to prevent sediment accumulating and produce a ‘self-cleansing’ network. This design approach allows for reduced fire flows to be agreed with local fire departments, taking into account the improved performance of modern fire fighting equipment. Now there are 40 mm and 63 mm diameter pipes feeding domestic users in local networks at dead ends where before the minimum pipe size may have been 100 to 150 mm to cater for fire flows.

Chlorine demand increases with decreasing pipe diameter as the two are inversely related as can be seen in the following equation where k_{w*} is wall reactions, k_w is wall reactivity, k_f is mass transfer coefficient, and d is pipe diameter:

$$k_{w*} = \frac{4k_w k_f}{d\left(k_w + k_f\right)}$$

Equation 20: Pipe wall reaction rate

Consequently, the same distribution network with a uniform pipe size of 50 mm would require a higher chlorine dose than a network of pipe size 150 mm, in order to meet primary and secondary disinfection requirements. Potentially the benefits of having reduced the retention time on the distribution system through downsizing pipes would be counterbalanced by the necessary increase in the chlorine dosage.

Key Messages:

- Effort should be made to locate new water connections, and manual and automated flushing sites on areas of the distribution network with high retention times so that demand is increased in these areas.
- Distribution system flushing can be used as a first response measure to water quality failures, including high levels of DBPs.
- Manual or automatic flushing for the control of DBPs must occur so that the period between flushing is less than the maximum retention time in the distribution system.
- A distribution system can be bled continuously in order to lower retention times under certain conditions.
- Continual system flushing (manual, automated or through a continuous bleed) and reducing overall system capacity (abandoning mains, downsizing mains) offers positive potential for reducing DBP levels, but must be weighed against water conservation needs, and contact time or CT requirements.
- Minimizing the number of shut valves required to produce a hydraulic boundary, and locating valves in areas with relatively high demand on either side of the shut valve can reduce retention times. Shut valves can be used in a network to re-route flows through parts of a system with low demand and high retention times.

- Pumping water from one zone in a distribution system to another in order to re-circulate water can be used to reduce overall peak retention times.
- Old pipes greater than 25 years, particularly unlined cast iron pipes, should either be replaced or relined if known to be contributing to water quality problems.
- New development in communities should be controlled so as to promote optimal water distribution system layout. They should be designed to avoid branching, to minimize the number of dead ends, and to maximize looping of the system.
- The design of water distribution systems needs to reflect current long term declining population trends in the province when estimating future water demand.
- Pipe size should be optimized to meet required hydraulic conditions.
- Consideration should be given to a design guideline requiring the achievement of a daily peak water velocity for all pipes in a distribution system in the range of 0.2-0.4 m/s.
- In Newfoundland and Labrador, design guidelines for fire flows, fire storage and other fire fighting requirements are established by the Insurance Advisory Organization and the Fire Commissioners Office. The justification for such requirements is not well documented and should be investigated more comprehensively.

4.11 Alternative Disinfectants

The purpose of disinfecting water is to kill or inactivate all pathogens that might be present in the water including bacteria, amebic cysts, algae, spores and viruses. The ideal disinfectant needs to be:

- effective against all pathogens
- provide a residual that will remain in the water to continue to disinfect and be measurable
- be cheap, reliable, and easy to produce
- not create harmful byproducts

The formation of chlorinated DBPs in drinking water has emphasized the need to explore alternate disinfectants and new water treatment technologies. Alternative disinfectants to chlorine offer two separate but related methods for reducing chlorinated DBPs in drinking water: i) by not forming chlorinated DBPs in the first place, and ii) by the chemical destruction of DBP precursors through oxidation.

Controlling chlorinated DBPs in drinking water can be achieved using alternative disinfectants either alone or in combination with chlorine. Alternative disinfectants include:

- Ozone [O_3]
- Chloramines [NH_2Cl]
- Chlorine dioxide [ClO_2]
- UV
- MIOX (mixed oxidants)

In using alternate disinfectants, however, consideration must be given to the fact that they may form non-chlorinated DBPs of which less in known. In order to leave a residual, ozone and UV disinfection must be paired with chlorine as a secondary disinfectant. If NOM is present in the water, DBP formation will still occur even with an alternative primary disinfectant.

The prevention or removal of DBP precursors prior to disinfection provides the best assurance that DBPs will not form. Chemical destruction of DBP precursors by oxidation is partly a treatment measure, however, the main oxidizing agents can also be used for disinfection. Ozone, chlorine dioxide and permanganate are all at least somewhat effective in reducing THM formation potential by reducing precursor (NOM) levels.

4.11.1 Ozone

Ozone is generated on site by passing dry air or oxygen (O_2) through an electric charge, converting it to ozone (O_3). The ozone gas is then bubbled through the water. Uses of ozone include disinfection, oxidation of iron and manganese, taste and odour control, enhancement of coagulation and filtration, reducing chlorinated DBPs and oxidation of hydrogen sulfide. Ozone is an excellent disinfectant for bacteria and viruses, is capable of inactivating c*ryptosporidium*, and of breaking down pesticides. Ozone has been shown to cause a change in the fractional makeup of NOM, but results in only a slight drop in the total concentration of NOM. Ozonation results in significant reduction in UV absorbance suggesting reaction with carbon double bonds and aromatic materials in the NOM that are considered more favorable for DBP formation (AwwRF, 1994). The most significant by-product formed by ozonation is bromate, which depends on the presence of bromide and ammonia ion concentrations. Ozone does not produce a residual disinfectant and so must be coupled with another disinfectant such as chlorine. Ozone equipment has proven to be less reliable than other methods of disinfection and can lead to higher rates of corrosion in the distribution system. Ozone is widely used in Europe, less so in the US and even less in Canada. Ozone costs roughly four times as much as chlorine disinfection due to the large amounts of electricity used to generate the ozone.

4.11.2 Chloramines

Chloramines are formed from chlorine and ammonia. Use of chloramines include final disinfection, persistence and ability to reach remote areas in the distribution system, penetration of biofilms, formation of lower levels of THMs (levels 40 to 80 percent lower) and other DBPs, and taste and odor control. Chloramines are moderately effective against bacteria, but not so good at killing viruses. Chloramines produce similar DBPs to that of chlorine but at much lower concentrations. Available information indicates that chloramines can reduce other DBP formation as well. However, recent research suggests that chloramines can cause the formation of cyanogen chloride and N-Nitrosodimethylamine (NDMA), a proven carcinogen. Chloramines are known to cause nose and eye irritation, stomach discomfort and anemia. In addition, recent research has shown that iodinated by-products are higher in systems using chloramines than in systems using free chlorine (Singer, 2006). Use of chloramines can also lead to biological nitrification problems and has been linked to elevated levels of lead in tap water. Chloramines are used widely in the US and cost roughly twice as much as chlorine.

Chloramines are more often used as a final disinfectant in the distribution system after primary disinfection has been achieved with some combination of filtration, ozone, chlorine dioxide and or free chlorine. Chloramines are in use in less than a handful of communities across the province. An ANOVA analysis was performed to test for significant differences in THM values before and after the commissioning of the chloramine system in the community of Dunville as shown in the following table. The difference in THM means before and after the switch from chlorine to chloramines was significant with a 92% reduction of THM values. The reduction in average BDCM was also significant.

Table 25: ANOVA of mean community THMs with chloramines as analysis factor

Community	Region	N pre/ N post alternative disinfection	THM mean pre alternative disinfection (ug/L) (± StDiv)	THM mean post alternative disinfection (ug/L) (±StDiv)	p-value
Placentia (Dunville)- THM	E	56/29	184.1 (±96.48)	15.3 (±53.34)	0.000*
Placentia (Dunville)- BDCM	E	48/29	9.844 (±4.661)	0.538 (±1.637)	0.000*

* statistically significant at $\alpha = 0.05$, N is sample size

While not a silver bullet for dealing with the province's DBP problems, the use of chloramines as a disinfectant offers huge potential for reducing THM levels.

4.11.3 Chlorine dioxide

Uses of chlorine dioxide (ClO_2) include disinfection, oxidation of iron and manganese, taste and odor control, enhancement of coagulation and filtration, reducing chlorinated DBPs and control of biological growth in open treatment basins. Chlorine dioxide is effective at killing pathogens (including c*ryptosporidium* and g*iardia*) and it leaves a residual. Disinfection with chlorine dioxide does not form THMs, however inorganic DBPs such as chlorite and chlorate are formed which have human health risk implications. Chlorine dioxide is known to cause anemia as well as nervous system effects in infants and young children. Chlorine dioxide use is more common in Europe and is almost ten times as expensive as chlorine.

4.11.4 Ultraviolet Radiation

Ultraviolet radiation (UV) is generated by special light bulbs that are immersed in water where the UV rays work by damaging the genetic material of pathogens (including *cryptosporidium* and *giardia*), preventing them from reproducing. UV is an evolving technology in water treatment seeing increasing use throughout Europe, the US and Canada. Research suggests no direct DBP formation at the doses used for water disinfection, except for some formation of nitrate from nitrite. UV requires minimal contact time and is not affected by pH levels. UV leaves no residual in the distribution system and so must be coupled with chlorination or some other form of disinfectant that does provide a residual. UV does not work well with waters having high turbidity and is a preferred treatment for groundwater. If used on waters with high levels of suspended solids, water must be cleaned (filtration, coagulation/flocculation, settling) well before disinfection. UV radiation costs roughly twice as much as chlorine disinfection.

Figure 38: UV treatment system from Pasadena WTP

4.11.5 MIOX

MIOX is a proprietary product of MIOX Corporation that uses only salt and electricity to generate a dilute mixed oxidant disinfectant solution that can be used in water distribution systems rather than chlorine. Mixed oxidants are effective at microorganism inactivation, preventing the build up of bio-film in the distribution system, improving flocculation treatment performance, maintaining chlorine residuals, reducing taste and odor complaints, reducing colour, and reducing levels of chlorinated DBPs. MIOX systems are also effective on systems that have a low contact time, require no handling of hazardous materials, and leave no chlorine taste to drinking water. MIOX systems are not a good alternative in high bromide waters, and little is know of the potential for formation of alternative DBPs. Capital costs for MIOX systems fall between those of liquid and gas chlorination systems, however, operational savings are significant as compared with traditional chlorine disinfection.

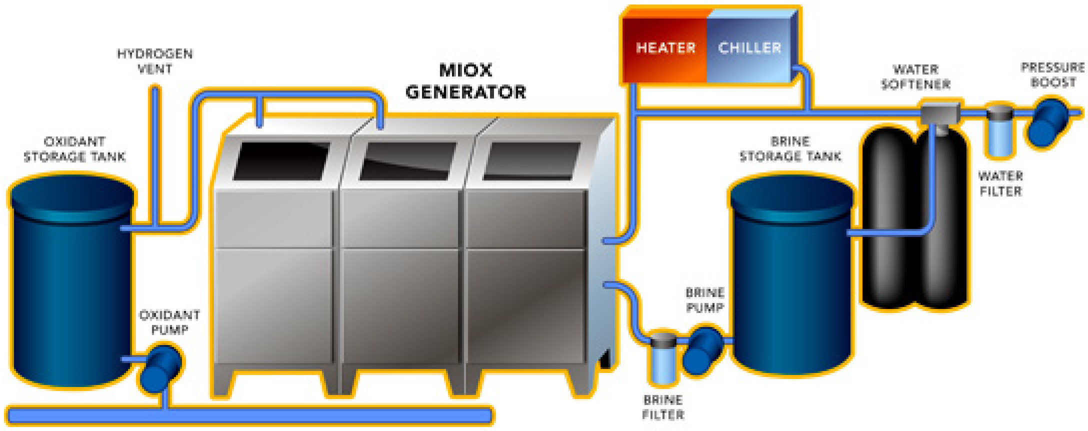


Figure 39: The MIOX process

Several communities in Newfoundland and Labrador have already or are looking to switch to MIOX disinfection systems and preliminary results look encouraging– water colour is reduced, residual levels have improved. MIOX technology use is very much in the pilot stage in the province currently.

Table 26: Communities in NL using or considering use of MIOX

Communities Using/ Considering MIOX	Region
Heart's Delight-Islington	E
Come by Chance	E
Lawn	E
Port au Port West	W
Channel-Port aux Basque	W

Key Messages:

- Alternative disinfectants such as ozone, chlorine dioxide, chloramines, UV and MIOX, can significantly reduce the production of chlorinated DBPs.
- In order to provide a disinfectant residual in the distribution system, ozone and UV must be paired with a disinfectant that does leave a residual, such as chlorine.
- All disinfection methods, except for UV, will produce some form and level of DBPs.
- Ozone, chlorine dioxide and MIOX not only disinfect water, they provide for the chemical destruction of DBP precursors through oxidation.

4.12 Source Water Treatment

Water is treated for a variety of purposes, including removal of pathogenic microorganisms, taste and odours, colour and turbidity, dissolved minerals and harmful organic materials. The water treatment process entails the prevention, physical removal and chemical destruction of unwanted characteristics in the water. The removal of DBP precursors through treatment provides the best assurance that DBPs will not form (Bureau of Reclamation, 2001). Various treatment methods are already in place in many communities across the province including:

- Conventional water treatment plants- screening, coagulation, flocculation, sedimentation/flotation, filtration (granular)
- Granular activated carbon (GAC) filtration
- Membrane filtration- microfiltration, ultrafiltration, reverse osmosis (RO)
- pH adjustment
- Iron and manganese removal

The removal of organic and inorganic substances that act as precursors for DBP formation has proven effective in reducing chlorinated DBP formation potential but that effectiveness depends on the type of treatment involved. It may be that the limited removal of NOM does not give desired results; for example, THMs above guideline values can be expected for waters in the province with DOC values greater than 2 mg/L. Natural organic material can be partially removed using conventional and newer treatment processes; however, bromide is much harder to remove without advanced treatment processes. At a minimum, most communities have pre-screening to prevent large debris from being drawn into the intake and distribution system. Treatment processes on smaller systems are usually installed to deal with specific issues such as low pH. Typically only larger communities in the province will have conventional large-scale water treatment plants.

Treatment based corrective measures are usually capital-intensive solutions. Most processes have significant funding requirements, not only for initial capital costs, but ongoing operational or life cycle expenditures for things like electricity, chemicals and maintenance.

4.12.1 Water Treatment Plants

In the past few years, a number of communities in the province have installed water treatment plants (WTP). Surface water generally requires more treatment than groundwater as it has a higher potential for contamination. Treatment objectives are variable, but usually include the removal of turbidity and associated contaminants (pathogenic organisms, colour). Half the treatment plants in the province provide only mechanical treatment and disinfection. The other half provides mechanical and chemical treatment along with disinfection as indicated by the figure of the conventional water treatment train below. The following tables highlight the WTP train processes for the treatment plants examined:

Table 27: Water treatment plant treatment trains

Heart's Delight-Islington	Conne River	Gander	Grand Falls-Windsor	Pasadena
pH adjustment	Dual media pre-filters	Pre-filters	Prescreening	Pre-filters
Coagulation/ Flocculation	Micro-filtration	Ozonation	Coagulation/ Flocculation	Tread filter
Sedimentation	Ultra-filtration	Rapid sand dual media filtration	Sedimentation	UV

Filtration	Sodium carbonate addition	Carbon filtration	Rapid sand dual media filters	Chlorination
Chlorination	Chlorination	pH adjustment	pH adjustment	
		Chlorination/ Chloramination	Chlorination	

Table 25: (continued)

Deer Lake	Channel-Port au Basques	Happy Valley-Goose Bay	Lourdes	Ramea
Pre-filters	Prescreening	Permanganate, Alum, Cl Addition	Primary multi-media filtration	Prescreening
Thread filter	Coagulation/ Flocculation	Green Sand Filtration	Permanganate Addition	Pre-chlorination
UV	Sedimentation	Sedimentation (backwash)	Multi-media filtration	pH adjustment
Chlorination	Dual media filtration	Post-Chlorination	Chlorination	Coagulation/ Flocculation
	pH adjustment			Sedimentation
	Chlorination			Dual media filtration
				Post-chlorination

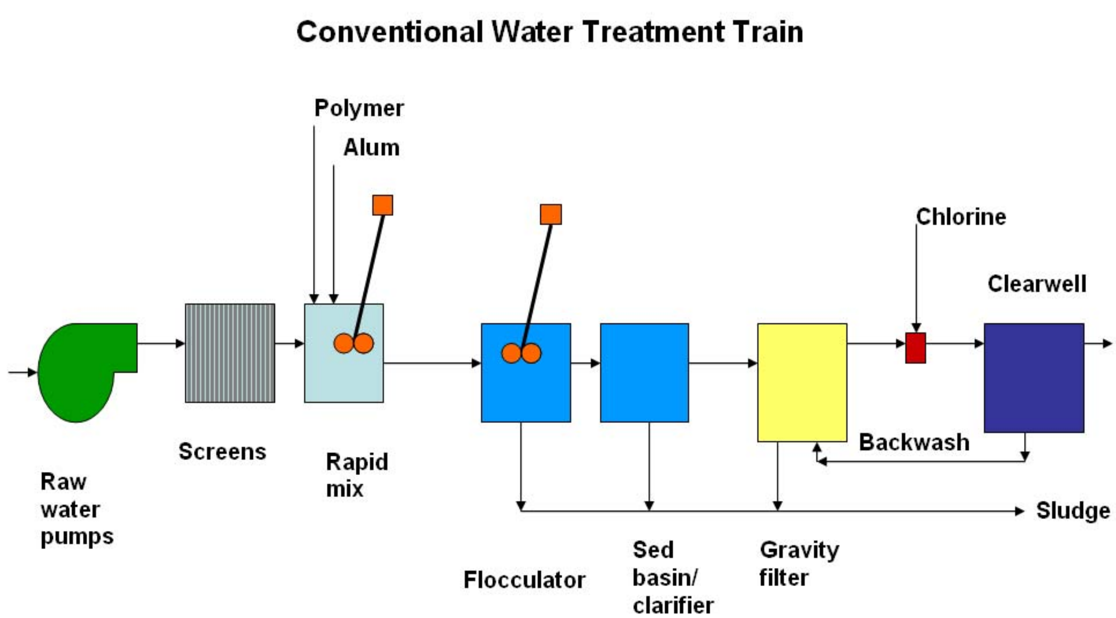


Figure 40: Conventional water treatment train

An ANOVA analysis was performed to test for significant differences in THM values before and after the commissioning of the water treatment plant (WTP) as shown in the following table.

Table 28: ANOVA of mean community THMs with water treatment plant as analysis factor

Community	Region	N pre WTP/ N post WTP	THM mean pre WTP (ug/L) (± StDiv)	THM mean post WTP (±StDiv)	p-value
Conne River	C	68/7	149.9 (±92.66)	58.96 (±16.47)	0.012*
Deer Lake	W	13/25	50.1 (±21.94)	46.16 (±19.85)	0.583
Gander	C	100/6	194.8 (±48.58)	210.5 (±53.66)	0.448
Pasadena	W	31/5	77.3 (±25.99)	150.1 (±37.58)	0.000*
Channel-Port au Basques	W	2/75	36.9 (±0.49)	59.8 (±44.42)	0.471
Grand Falls-Windsor	C	5/95	60.2 (±25.21)	80.2 (±25.65)	0.093
Happy Valley-Goose Bay	L	11/9	19.2 (±9.85)	39.3 (±15.03)	0.002*
Heart's Delight-Islington	E	24/26	159.7 (±82.83)	88.0 (±30.65)	0.000*
Lourdes	W	33/6	193.6 (±112.7)	124.7 (±51.5)	0.153
Ramea	W	9/34	301.2 (±181.9)	313.9 (±175.3)	0.849

* statistically significant at $\alpha = 0.05$, N is sample size

Of the 10 water treatment plants examined, only four showed significant differences in THM values before versus after the commissioning of the treatment plant. Conne River and Heart's Delight-Islington showed a significant decrease in THM levels, while Deer Lake and Lourdes both showed a weak decrease. Pasadena and Happy Valley-Goose Bay both showed significant increases in THM levels. The Pasadena water treatment plant is intending to add additional filtration capacity due to increased water demand, so existing filters may not be adequate. Gander, Channel-Port au Basques, Grand Falls-Windsor and Ramea all showed weak increases. The Gander WTP is not fully commissioned, which might explain the slight increase. In other cases there was very little data for comparison either before or after the commissioning of the WTP.

It appears the presence of a WTP on a distribution system will not necessarily reduce THM levels if the WTP has not been designed specifically to remove DBP precursors (NOM) or if the treatment system has not been sized adequately. Another factor that may be influencing observed THM increases is the practice of pre-chlorination to bacteriologically disinfect WTP infrastructure. Pre-chlorination is the addition of chlorine prior to any other treatment. This practice enhances the production of THMs and other DBPs making the practice undesirable. Chlorine can be added before filtration, but should not be added before coagulation and sedimentation, which provide a substantial reduction in the organic materials, which are the precursors of THMs. The use of UV disinfection for pre-disinfection in the treatment train is also a viable option. The most successful forms of treatment to reduce THMs, of those in use, appear to be chemical treatment (ie. coagulation/flocculation, GAC), multi-media filtration, and micro and ultra-filtration.

4.12.2 Filtration

Filtration is increasingly being used to remove pathogens and suspended solids from drinking water as government regulations become stricter for parameters like turbidity. Alberta and Quebec are the only provinces in Canada that demand filtration and disinfection of surface water sources. The NL *Guidelines for the Design, Construction and Operation of Water and Sewerage Systems* state that "filtration preceded by appropriate pre-treatment shall be provided for all surface waters", however, the vast majority of systems in the province lack any form of filtration other than basic screening at the intake.

Filters remove suspended solids and pathogens by physically preventing them from passing through a filter media. Filter media can be granular (sand, gravel) typically used on larger systems with water treatment plants, chemical (granular activated carbon- GAC, greensand), weave-wire screens, and polymer membranes with very small pore sizes. Simple distribution systems filters are usually either in-line (GAC) pressure filters or membrane filters. Membrane filtration is roughly twenty times as expensive as chlorine disinfection, although costs are steadily decreasing. Pre-filtration and membrane filters can be classified by their pore size as follows (divisions can vary):

Table 29: Pre-filtration and membrane filtration pore size divisions

Level of Filtration	Filter Type	Application	Operating Pressure (psi)	Pore Size (μm)
Pre-filtration	Screen filters			>10
Pre-filtration	Thread filters			>3
Membrane filtration	Micro-filtration	Disinfection, particle removal	7-40	0.1-10
Membrane filtration	Ultra-filtration	Disinfection, particle removal	7-40	0.001-0.1
Membrane filtration	Nano-filtration	Softening, NOM removal	75-130	0.0001-0.001
Membrane filtration	Reverse osmosis	Desalting, synthetic organic chemical and inorganic chemical removal	150-1500	<0.0001

Figure 41: Pre-filtration system at Pasadena WTP

In the past few years, a number of communities across the province have installed pre-filtration and GAC units on their distribution systems to treat source water. The following table outlines the size and type of filtration system currently active in several communities. All of these stand-alone filters can be classified as pre-filters. Membrane filters are typically only in use in the province as part of a water treatment plant train.

Table 30: Communities with stand-alone filters in NL

Community and Type of Filter	Filter Size/Type
Buchans	100 μm
Embree	100 μm
Isle aux Mort	80 μm and GAC and pH adjustment
Lewisporte	100 μm
Port Anson	50 μm
Seal Cove (WB)	50 μm
Steady Brook	50 μm
Gilliams	50 μm
Cox's Cove	100 μm
Grand Bank- GAC	GAC
Churchill Falls	80 μm

An ANOVA analysis was performed to test for significant differences in THM values before and after the commissioning of filtration systems.

Table 31: ANOVA of mean community THMs with filtration as analysis factor

Community and Type of Filter	Region	N pre filtration / N post	THM mean pre filtration (ug/L) (± StDiv)	THM mean post filtration (±StDiv)	p-value

		filtration			
Buchans	C	22/10	50.1 (±21.42)	48.2 (±27.73)	0.836
Embree	C	28/15	92.8 (±82.58)	86.5 (±34.66)	0.780
Isle aux Mort	W	16/3	80.0 (±50.41)	122.8 (±105.02)	0.268
Lewisporte	C	51/11	129.6 (±38.90)	140.9 (±34.19)	0.374
Port Anson	C	20/6	144.9 (± 80.33)	110.4 (±69.50)	0.354
Seal Cove (WB)	W	9/20	76.01 (± 36.73)	93.61 (± 36.58)	0.242
Steady Brook	W	3/25	44.0 (±8.66)	66.3 (±34.94)	0.289
Gilliams	W	12/8	85.4 (±50.12)	92.4 (±40.40)	0.749
Cox's Cove	W	19/4	53.9 (±16.58)	53.3 (±12.79)	0.942
Grand Bank- GAC	E	10/54	46.7 (±11.96)	66.3 (±38.49)	0.118
Churchill Falls	L	3/5	41.33 (±8.62)	47.30 (±16.65)	0.593

* statistically significant at $\alpha = 0.05$, N is sample size

No significant differences in THM levels were detected in any of the ten distribution systems examined with the installation of filtration units. A lack of data either before or after commissioning of the filtration system may be influencing some results. There were weak decreases in THM levels in four communities, while the remaining seven showed weak increases. While such filtration systems might assist in the removal of turbidity and other suspended solids, they appear to have little effect in the removal of DBP precursors such as DOC. In order to remove DBP precursors, filtration systems (granular) must be in combination with chemical treatment, they must be appropriately sized and maintained (all types), or they must be of sufficiently small pore size (ultra-filtration, nano-filtration). Upgrading to more effective filtration systems entails moving from simple filtration (automatic backwash pressure filters) to multiple process treatment trains.

4.12.3 pH Adjustment

pH adjustment is normally implemented as a treatment for corrosion and aggressive water or to add alkalinity and provide buffer as a pre-conditioner for polymer and coagulant addition in conventional treatment processes. Optimizing the pH of water in the distribution system can also lead to more productive use of chlorine, and potentially lower THM formation potential (THMs increase with increasing pH and temperature). Surface waters in Newfoundland and Labrador have naturally low pH, which is favourable for disinfection with chlorine. Historical data has shown that pH adjustment has had little discernable impact on THMs (AwwaRF, 2004). Several communities throughout the province have pH adjustment on their distribution systems to increase pH levels with the addition of lime or soda ash. An ANOVA analysis was performed to determine the impact of pH adjustment on THM levels.

Table 32: ANOVA of mean community THMs with pH adjustment as analysis factor

Community	Pre or post chlorin -ation	Reg-ion	N pre filtration / N post filtration	THM mean pre filtration (ug/L) (± StDiv)	THM mean post filtration (±StDiv)	p-value
Burnt Islands	pre	W	6/5	41.8 (±13.36)	38.8 (±15.88)	0.745
Bonavista	post	E	46/12	120.8 (±73.36)	213.6 (±66.74)	0.000*

Pouch Cove	post	E	22/13	71.9 (±29.86)	202.4 (±142.46)	0.000*
Spaniard's Bay	post	E	27/3	35.3 (±15.07)	40.9 (±6.43)	0.539
Victoria	post	E	18/12	35.4 (±12.46)	39.4 (±12.99)	0.406
Fogo	post	C	27/19	180.79 (±12.46)	71.36 (±25.84)	0.000*
Hermitage	post	C	6/36	108.53 (±53.08)	91.09 (±71.53)	0.572
New-Wes-Valley (Newtown-Templeton)	post	C	39/5	108.96 (±70.43)	93.54 (±34.76)	0.635

* statistically significant at $\alpha = 0.05$, N is sample size

Of the 8 communities with pH adjustment examined only three showed any significant change in means. In the case of Pouch Cove, the increase in THM levels before and after installation of the pH adjustment system is linked to the chlorination system being moved closer to the supply at the same time, thus increasing residence time for THM formation. No discernable reason for the significant increase in THMs in Bonavista could be identified other than a possible increase in chlorine use. In Fogo, the chlorination system was upgraded at the same time as the installation of the pH adjustment system which could explain the significant decrease in THM levels observed. All other communities showed either a weak increase or decrease.

For the most part, pH adjustment has little effect on THM levels. Although pH and THMs are directly proportional, and an increase in THMs would be expected with an increase in the pH, other factors (increased contact time, increased chlorine use) seem to be the main cause where significant increases were observed. Chlorination is considerably slowed down when the pH is high (greater than 8) requiring either an increased contact time or increased initial dose. pH adjustment should always take place post-chlorination.

4.12.4 Iron and Manganese Removal

Iron and manganese are common elements found in both surface water and groundwater throughout the province. Neither is a health contaminant, although aesthetic guidelines exist for both as they contribute to colour, and can cause staining (of clothes, fixtures). A community should look at iron and manganese removal if iron exceeds 0.3 mg/L and manganese exceeds 0.05 mg/L, the aesthetic guideline values. Typically, iron and manganese removal has been used for treatment of groundwater sources in the province. Both iron and manganese are naturally high in NL, even in surface waters.

Iron and manganese oxidize readily in the presence of chlorine, and can be responsible for a large portion of chlorine demand in distribution systems with high iron and manganese in their source surface waters. Removal of iron and manganese from surface water sources can potentially reduce overall chlorine demand, reducing the amount of chlorine dosage required to disinfect the water supply, and potentially reducing THM totals.

Methods to control iron and manganese in the distribution system include adding phosphate to the water to keep iron and manganese in solution, and oxidation and removal with filtration. Oxidation is typically carried out using aeration, chlorine, or permanganate. Permanganate is more effective at oxidizing manganese than either aeration or chlorine. Permanganate (and to a lesser extent aeration) has also been proven to be somewhat effective in reducing THM formation potential by reducing precursor (NOM) levels through oxidation. Use of potassium permanganate should occur as soon as possible in the treatment process.

Most iron and manganese removal systems in Newfoundland and Labrador are on groundwater sources and involve the use of permanganate. Only two iron and manganese removal systems are on surface water sources, in the communities of Long Harbour and Port Hope Simpson. An ANOVA analysis was performed to determine if there was any significant difference in THM levels before and after commissioning of the iron and manganese removal system.

Table 33: ANOVA of mean community THMs with iron and manganese removal as analysis factor

Community	Region	N pre Fe-Mn/ N post Fe-Mn removal	THM mean pre Fe-Mn removal (ug/L) (± StDiv)	THM mean post Fe-Mn removal (±StDiv)	p-value
Long Harbour	E	7/31	108.1 (±37.42)	62.15 (±32.35)	0.002*
Port Hope Simpson	L	2/5	4.0 (±5.7)	203.5 (±134.4)	0.104

* statistically significant at $\alpha = 0.05$, N is sample size

There was a significant decrease in THM levels in Long Harbour after the installation of the iron and manganese removal system. The combination of reducing chlorine demand and therefore chlorine dosage, and the potential for oxidizing DBP precursors (depending on the type of iron and manganese removal implemented) is potentially a very effective method for reducing THM levels. Although there was an increase in THM levels for Port Hope Simpson after the installation of the Greensand filter, the data set is too small for the difference to be significant. It is also doubtful that the town was chlorination prior to the installation date of the filter. More examples of iron and manganese removal on surface water systems need to be evaluated to truly gauge the effectiveness of this treatment process in dealing with chlorinated DBPs.

4.12.5 Advanced Treatment Processes

So far, the only treatment processes that have been examined are conventional or more standard ones common to Newfoundland and Labrador, but a variety of other processes exist of varying technological complexity, effectiveness and cost. The main advanced treatment processes used for DBP precursor removal of NOM include:

- Enhanced coagulation (EC)
- Granular activated carbon (GAC) filters
- Reverse osmosis or nanofiltration

- Peroxide addition- to oxidize difficult to treat organics
- Biofiltration
- Dissolved air flotation (DAF)

Researchers have shown that increased NOM removal using enhanced coagulation, GAC adsorption and nano-filtration lowers overall formation of THMs (and HAAs), but can result in an increase in some of the more brominated forms of these classes of compounds (AwwaRF, 2006). The main advanced treatment processes used for DBP precursor removal of total dissolved solids (including bromide) include:

- Reverse osmosis or nano-filtration
- Electro-dialysis reversal (EDR)
- Ion exchange (IX)

Other advanced treatment processes that can help reduce DBPs include, but are not limited to:

- MIEX® resin process
- Ceramic membranes
- Ozone and bio-filtration combined
- Filtration using iron-oxide coated media
- Adsorption filters
- Advanced oxidation processes (AOP)
- Distillation (multistage)

Advanced treatment processes are not appropriate for smaller distribution systems. For the volumes of water being treated, the cost and operational know how required for advanced treatment processes is simply outside of the capabilities of most small communities in Newfoundland and Labrador.

Key Messages:

- Source water treatment for the targeted removal of DBP precursors provides the best assurance that DBPs will not form.
- Natural organic material can be removed to varying degrees using conventional, standard, and advanced treatment processes. Bromide removal requires advanced treatment processes.
- A water treatment plant (WTP) on a distribution system will not necessarily reduce THM levels if the WTP has not been designed specifically to remove DBP precursors or if the treatment system has not been adequately designed. WTPs in communities with DPB issues must be designed for the removal of DBP precursors.
- The practice of pre-chlorination prior to any other form of treatment in the WTP must be stopped. Depending on the treatment train, chlorine may be added before filtration, but never before coagulation and sedimentation. Pre-chlorination in conventional treatment plants may be necessary on a periodic cycling basis to deal with in-plant vectors such as algae growth and odour conditions.

- The most successful forms of treatment in a WTP train to reduce THMs are chemical treatment (ie. coagulation/flocculation, GAC), multi-media filtration, membrane (micro to nano) filtration, and reverse osmosis.
- Stand-alone pre-filtration systems (of pour size >10 µm) have no significant effect on reducing DBPs.
- To be effective in reducing DBPs, filtration systems (granular) must be in combination with chemical treatment, they must be appropriately sized and maintained (all types), or they must be of sufficiently small pore size (ultra-filtration, nano-filtration).
- pH adjustment has a limited effect on reducing DBPs. pH adjustment should be optimized for each individual system and should occur post chlorination.
- Iron and manganese removal (preferably through the use of permanganate) offers positive potential for the reduction of DBPs through reducing chlorine demand and required chlorine dosage, and the oxidation of DBP precursors. Primary disinfection requirements must still be met with any reduction in chlorine dosage, however.
- Large scale advanced treatment processes are not appropriate for very small to medium sized water distribution systems in the province.

4.13 Potable Water Dispensing Units

A potable water dispensing unit (PWDU) is a type of small scale water treatment system that treats only a fraction of total water demand on a distribution system using many of the same treatment processes found in large scale treatment plants. A PWDU is intended to only treat the drinking water portion of overall water demand or roughly 0.5-3 L/person/day. Water is stored on site at some centralized location for manual collection by users. Other jurisdiction in Canada, including Saskatchewan and the Territories are also making use of PWDU to deal with drinking water quality issues in small, rural communities.

While large scale advanced water treatment plants might be unsuitable for many small communities in Newfoundland and Labrador, such technology may be suitable on a smaller scale. In many communities across the province, residents refuse to drink the water that comes from their tap, and collect their drinking water in containers from potentially unsafe roadside springs, local wells and commercial outlets. In a recent study, 28% of tests from springs used for drinking water were contaminated with *E.coli* and/or had coliform counts above provincial guidelines. Springs that tested fine one week would also prove two be contaminated after tests two weeks later (Nicol et al., 2008). Providing treated potable water that can be collected in containers similar to the current practice of collecting water from potentially unsafe roadside springs would not require much adaptation.

Currently, there are a handful of communities with different water quality issues that have provided small-scale potable water dispensing units from which residents of the community can fill containers for their drinking water. The communities still operate their regular water distribution system, and use the PWDUs as an alternative source of drinking water. Communities with such systems include:

- Burnt Islands (W)
- St. Lawrence (E)
- Buchans (C)
- Howley (W)
- Ramea (W)
- Black Tickle-Domino (L)- no distribution system exits

The systems include a combination of multimedia filtration, activated carbon filtration, ozonation, reverse osmosis, and UV disinfection. Treatment systems, water storage tanks and dispensing stations are typically located in a centralized and easily accessible public building. The capital cost of such a system ranges from $30,000 to $100,000 with annual operation costs of between $600-$25,000 (Park, 2007). The issue of qualified operator attendance for PWDU operation and maintenance in small, rural communities is best dealt with by having provincial management of installed systems. There must be demonstrated approval from residents of the community for installation of a PWDU.

Figure 42: St. Lawrence potable water dispensing unit facility

Preliminary water quality results indicate a marked improvement in drinking water quality in two of the PWDU systems as shown in the following table. After treatment from the PWDU, all parameters were below guideline levels except for pH, and there were significant decreases in colour, turbidity, DOC and copper. Results did indicate a slight increase in aluminum in both systems. To date, PWDUs have been working effectively to provide safe, affordable, accessible, high quality drinking water to public in response to drinking water quality issues experienced in respective communities.

Table 34: Water quality results from before and after PWDU treatment

Parameter	Buchans		St. Lawrence	
	Before	After	Before	After
Colour (TCU)	31	5	47	7
pH	6.44	6.33	4.55	5.12

Turbidity (NTU)	0.6	0.3	1.1	0.7
DOC (mg/L)	5.6	1.1	6.2	2.4
Copper (mg/L)	0.347	0.002	0.756	0.267
Aluminum (mg/L)	0.01	0.12	0.15	0.22
Iron (mg/L)	0.03	0.13	0.37	0.19
TDS (mg/L)	8	9	32	26

In some communities PWDUs may interfere with commercial businesses (store owners) that have been selling treated water to local consumers.

Key Messages:

- Advanced water treatment technology may be appropriate in small to moderate sized communities on a small scale in the form of Potable Water Dispensing Units (PWDUs).
- Collection of drinking water in containers from a centralized location (springs, stores) is common practice in many communities of the province.
- Roadside springs are not reliable sources of safe drinking water.
- PWDUs must not replace regular water distribution systems and should not replace regular water disinfection or treatment systems.
- There must be demonstrated community support for the installation of a PWDU.

4.14 Centralized/Regional Drinking Water Systems

A centralized or regional drinking water system can be defined as any water distribution system that services more than one community. Such systems generally exist because:

- There is only one water source available for each of the communities so the system must be shared.
- Communities are extremely close together, and having separate systems makes no economic sense.
- The source water being centralized is from a center of high population that can afford adequate treatment. Water is then distributed through pipelines to smaller surrounding populations that have typically been characterized by poor source and drinking water quality.

In other jurisdictions in Canada facing THM problems, regional water systems have been developed to distribute high quality drinking water (with low DBPs) to communities up to 20-40 km away. Such a regionalized approach would require the pooling of community financial resources, but would reduce manpower, supervisory and purchasing costs. It would also reduce infrastructure replication, provide efficiencies of scale, and better trained water system operators.

The following table lists all of the current centralized or regional water systems in the province. The St. John’s and Grand Falls-Windsor systems can be more properly classified as regional systems due to the population being serviced and the length of these systems.

Table 35: Centralized or regional drinking water systems in NL

Source Community	Other Communities	Region	Total Population (2001)	Approximate Longest Reach in System (km)
Grand Falls-Windsor (Exploits Regional Service Board)	Bishop's Falls, Botwood, Peterview, Wooddale	C	21,088	36
Bird Cove	Brig Bay	W	314	5
Whiteway	Cavendish	E	466	4
St. John's	Conception Bay South, Paradise, Mount Pearl, Portugal Cove-St. Phillips	E	145,953	-
Summerford	Cottlesville	C	1307	4
Brigus	Cupids, South River	E	1331	7
Hare Bay	Dover	C	1795	5
Appleton	Glenwood	C	1421	3
Burin	Lewin's Cove	E	3048	8
Embree	Little Burnt Bay	C	1234	10
Trinity Bay North	Little Catalina, Port Union	E	2641	8
Corner Brook	Massey Drive, Mount Moriah	W	21608	6
Flower's Cove	Nameless Cove	W	410	4
Deer Lake	Reidville	W	5249	6
Victoria	Salmon Cove	E	2407	-
Eastport	Sandy Cove	C	661	3
Dildo	South Dildo	E	1469	6
Spaniard's Bay	Upper Island Cove	E	4432	7
Lourdes	West Bay	W	803	4
Piccadilly Head	West Bay	W	307	3

An ANOVA analysis was performed to determine if there was any significant difference in THM levels between regional/centralized systems and non-regional/non-centralized systems in the province as shown in the following table. The analysis indicated that there was no significant difference between the two. This means that other factors, such as the presence of a water treatment plant, play a more significant role in reducing THM levels then just centralization of the water system.

Table 36: ANOVA of mean community THMs with regional/centralized system as analysis factor

N regional/ non regional	THM mean regional systems (ug/L) (± StDiv)	THM mean non-regional systems (ug/L) (±StDiv)	p-value
53/242	87.68 (±50.24)	84.86 (±60.6)	0.752

* statistically significant at $\alpha = 0.05$, N is sample size

Centralized systems might not be appropriate for extremely small and dispersed communities. Such systems also work better in relatively flat areas with a deep soil profile for the laying of extensive pipeline. Extremes in elevation could cause pressure problems in the pipeline, and bedrock would make the laying of such pipeline more difficult. The longest regional water system pipeline in the province (Exploits Regional Service Board) has approximately 50 km of pipeline in total.

Key Messages:
- Centralized or regional drinking water systems are most appropriate in high population, high population density areas that are relatively flat with a deep soil profile for the laying of extensive pipeline.
- Centralized or regional drinking water systems should include a water treatment plant if the population being serviced is medium to very large in size.
- Centralized or regional drinking water systems require support from the communities involved.
- Centralized or regional drinking water systems should have dedicated and well trained operators.

4.15 Point of Use and Point of Entry Treatment

Point of use (POU) and point of entry (POE) control measures entail the use of available commercial devises that can successfully reduce the amount of THMs in drinking water for the individual water consumer. A POU devise is a filtration unit that attaches to a tap, while a POE devise is something that attaches to the water service line coming into the home. With such systems, the consumer becomes responsible for maintaining the system– cleaning it, replacing parts, storing treated water. A system that is not maintained properly can actually add more contaminants to drinking water than it removes.

There are various point of use/point of entry treatment methods available that can successfully reduce the amount of THMs and other contaminants in drinking water including:

- Activated carbon filter units (THMs)
- Boiling water (THMs)
- Storage (THMs)
- RO units (THMs)
- Ion exchange units (THMs)
- Particulate filters
- UV disinfection units

The most common type of system is the jug filter that costs about $20-40, and a year's supply of replacement filters (typically activated carbon filters) will run about $40-90. Also common are in-line filter systems that can be installed on your faucet, under your sink, or as water enters your home; such systems can treat larger volumes of water. A simple faucet-mounted filter will cost approximately $30-50, and a year's supply of replacement filters can range from $40-110. The Department of Environment and Conservation recommends that any filters purchased be ANSI/NSF certified for removal of THMs or other contaminants of concern. Filters should be changed frequently and as a factor of the volume of water being filtered rather than recommended time periods as suggested by the manufacturer.

Figure 43: A Brita faucet filtration system

In a recent study performed by researchers at the University of Laval, storing water in a refrigerator for 48 hours reduced THMs by 30 percent, while boiling water before storage cut them by 87 percent. THMs are classified as volatile organic compounds (VOCs), which when in solution in water easily form vapours under normal temperatures and pressures, thus reducing their concentration in water. Using an activated carbon filtering pitcher prior to storage cut THMs by 92 percent (Reuters, 2006). Beyond water storage, boiling water, and using simple point of use water filters, the cost involved with household water treatment units increases exponentially.

A policy of point of use water treatment would denote a radical shift from the current approach of centralized water treatment of water supplies, and safe drinking water being provided at the turn of a tap. However, roughly one in five households in North America already filter their water with point of use treatment devices such as tap filters and filter jugs (Stauffer, 2004). Point of use control measures may be applicable as either short-term solutions or in very select situations as follows:

- For very small communities that cannot afford any water treatment
- As an interim solution while a more permanent solution is being put in place
- For situations where THMs may be high for only limited periods during the year
- For houses located on parts of the distribution system that have extremely high residence times and known THM problems

In September of 2006, residents of Rosebud Alberta were given activated carbon filter systems for their taps due to high levels of DBPs, particularly BDCM. This measure was taken after the Calgary Health Region issued a water usage advisory and the water treatment plant failed to maintain low levels of BDCM (CWWA, 2006). The Government of Newfoundland and Labrador should give consideration to issuing point of use treatment devises to households in communities with DBPs (THMs) consistently over guidelines.

Key Messages:

- POU/POE treatment systems must be used and maintained properly by the consumer including cleaning, replacement of parts, and adequate storage of treated water.
- POU/POE control measures may be applicable for very small communities that cannot afford any water treatment, as an interim solution to water quality problems while a more permanent solution is being sought, for situations where THMs may be high for certain periods of the year, for houses located on parts of the distribution system that have extremely high residence times and known THM problems.
- The Government of Newfoundland and Labrador should give consideration to issuing or subsidizing point of use treatment devises to households in communities with DBPs consistently over guideline levels.

4.16 Improving Drinking Water System Design

The design of water treatment and distribution systems in Newfoundland and Labrador must adhere to the *Guidelines for the Design, Construction and Operation of Water and Sewerage Systems*. Most communities already have a distribution network, but upgrades are constantly being performed. A community will typically hire an engineering consultant to design a new treatment/distribution system or upgrades to an existing system. Funding for the project will come from the town, the Department of Municipal Affaires, and in some instances the federal government. Approvals for the design will come from the Department of Environment and Conservation. The use of distribution system modeling software packages (eg. EPANET, WaterCAD, MikeNet) can assist in optimizing the design of such systems. Design constraints should take into consideration a number of different factors including:

- Technical feasibility
- Cost
- Fire-flow requirements
- Water quality
- Design guidelines

A number of design factors that have an impact on DBP levels have already been discussed in previous sections. Examining older systems to discover their weakness in design is also a useful exercise, so such problems can be avoided in future designs. Recommended improvements to the design of water treatment and distribution systems that have been made in previous sections of this report include:

- Looping of distribution system networks to prevent dead ends
- Self cleaning distribution systems through the requirement of achieving a maximum pipe water velocity

- Minimizing tank storage and maximizing tank mixing
- Minimizing pipe size to reduce unneeded capacity in the distribution system
- Adding new distribution lines to areas of low demand to reduce retention time in these areas
- Multiple level intakes and locating intakes in deeper water where water quality is superior and colder water temperatures slow reaction rates
- Preference towards groundwater and low DOC surface waters as source waters
- Avoid mixing of surface water and groundwater sources
- Avoid exposed coastal surface water sources
- Avoid shallow intake ponds with long exposed fetch lenghts
- Chlorination systems closer to 1^{st} point of use without violating primary disinfection requirements
- Flow meters a requirement on all systems
- Chlorine at least flow controlled
- Size pumps to provide high velocity inflow into tanks and for flushing
- Minimize number of shut valves and locate shut valves in areas of high demand
- Avoid use of cast iron pipe
- Add chlorine after treatment processes in a WTP

Such a list is ever evolving with new information and data contributing to our knowledge on DBP control on a daily basis.

Key Messages:

- The design of water distribution systems and water treatment plants is not static. New concerns, scientific knowledge, methods and innovations occur over time and those who design drinking water systems must be flexible and knowledgeable enough to incorporate such changes.
- The NL *Guidelines for the Design, Construction and Operation of Water and Sewerage Systems* should be updated at least every 10 years.
- Distribution system modeling should be used as a tool in the design of water distribution systems.
- Training should be provided to consultants involved in the design of water infrastructure to apprise them of changes to design guidelines, new concerns, scientific knowledge, methods and innovations.

4.17 Water System Operator Education and Training

Operator education and training is an essential and often overlooked component of any THM control methodology to be adopted, and in many cases is the only control measure in place. Operator Certification in Newfoundland and Labrador is voluntary at this point in time, although an increasing number of municipalities are requiring their operators to become certified. Operator certification is moving towards a phased mandatory requirement starting with large municipalities and then moving on to medium and small communities. This process is anticipated to take approximately five years. Operators can be certified from Class I-IV in Water Distribution, Water Treatment, Wastewater Treatment and Waste Water Collection. The Class an Operator can be certified to for water distribution depends on the population being serviced by the distribution system. The range is as follows:

Table 37: Certification levels according to population

Class	Population
I	0-1500
II	1501-15,000
III	15,001-50,000
IV	>50,000

For water treatment, operator certification levels depend on they type of treatment system in operation.

The availability of trained and qualified water system Operators who are able to properly operate and maintain their distribution systems is essential in the control of THMs. Operators often directly manage chlorine dosage, monitor chlorine residuals throughout the distribution system, regularly flush the system, clean tanks and maintain other infrastructure, monitor pump usage and take meter readings, detect and fix leaks– all activities that can have a major impact on THM levels.

Approximately 221 water system operators working with 117 different municipalities or band councils have at least Class I certification for either Water Distribution or Water Treatment. A correlation analysis was performed to see if there was any correlation between average community THM results and the highest level of certification held by an operator working on that community's distribution and treatment system (if one exists) as shown in the table below.

Table 38: Correlation between mean community THMs and operator certification level

	Correlation Coefficient-Water Distribution Class	Correlation Coefficient-Water Treatment Class
Average THMs	0.076 (0.419)	-0.165 (0.556)

* statistically significant at $\alpha = 0.05$

The results indicate that for communities with certified operators, there was no significant correlation between the level of certification and THM levels either positive or negative. This indicates that other factors (such as chlorine dosage and retention time) have a much greater effect on resulting THM levels than does the level of operator certification. THMs were negatively correlated with Water Treatment Certification level and positively correlated with Water Distribution Class level. An ANOVA analysis was also performed to see if there is any significant difference in THMs between communities with trained operators and those without as indicated in the table below. Results indicate that THMs are significantly higher in communities that have a trained operator. This is likely due to the fact that trained operators are more likely to be chlorinating properly.

Table 39: ANOVA of mean community THMs with a trained operator as analysis factor

N with/without	THM mean without a	THM mean with a trained	p-value

trained operators	trained operator (ug/L) (± StDiv)	operator (ug/L) (±StDiv)	
115/349	46.05 (±57.79)	84.96 (±57.70)	0.000*

* statistically significant at $\alpha = 0.05$, N is sample size

While operator certification level does not appear to have a statistically significant effect on reducing THM levels, the importance of having a knowledgeable and reliable Operator cannot be underestimated.

Key Messages:

- Operator education and training is an essential component of any DBP control methodology.
- Communities should require that their water system operators be certified.
- The OECT program should design a module on managing DBPs for incorporation into their training curriculum.

4.18 Impact of Corrective Measures

The impact of any corrective measure must be examined in terms of the main goals of reducing DBPs while maintaining disinfectant residuals. The impact of certain corrective measures has been examined in previous sections of this report based on experience here in the province and scientific research from other jurisdictions. The following table looks at the likely impact of the different identified corrective measures identified in this report.

Table 40: Impact of corrective measures on disinfectant residual and DBP formation

Corrective Measure	Control Method	Disinfectant Residual	DBP
Policy		NE	B
Source-based	Watershed protection	LB	LB
	Alternative water sources	LB	B
	No mixing ground and surface water	LB	B
	BMPs for reservoir flooding	LB	LB
	Wind breaks around exposed sources	NE	LB
	Intake in deeper water	LB	LB
	High quality water storage and recovery	B	B
CDM	Optimize disinfectant dose	B	B
	Relocate chlorination system	B	B
	Chlorine booster	B	I/D
	Chlorine dose control	B	LB
RTM	Tank location	LB	I/D
	Adjust pump schedule	LB	LB
	Reduce storage capacity	B	B
	Increase tank mixing	B	B
WDM	Flushing	LB	LB
	Continuous bleed	LB	LB
	Increase demand	LB	LB
Operation/ Infrastructure	Optimize valving	I/D	I/D
	Re-route or re-circulate water	I/D	I/D
	Downsize mains	B	LB
	Clean, replace or reline pipes	B	LB

	Loop distribution network	I/D	I/D
	Upgrade distribution system	B	B
	System maintenance	LB	LB
	Increase capacity of WTP	B	B
	Regionalization	I/D	I/D
Alternative Disinfectants	Chloramination	B	B
	Ozone	B	B
	Chlorine Dioxide	I/D	I/D
	UV	LB	NE
	MIOX	B	B
Treatment	WTP	B	B
	No pre-chlorination in WTP	LB	LB
	Filtration	B	B
	pH	I/D	I/D
	Iron and manganese removal	B	LB
	Advanced treatment	B	B
Point of Use Treatment		NE	B
Training		LB	LB
Design		B	B

*B-Benefit, LB-Limited benefit, NE-No significant effect, I/D-Could improve or deteriorate, D-Deterioration

The likely impact or effectiveness of each control measure must be evaluated as rigorously as possible. Methods to determine effectiveness include information from other jurisdiction, scientific studies, previous experience, modeling, etc.

5.0 Distribution System Modeling for Evaluating DBP Probable Causes and Corrective Measures

In order to help evaluate the probable causes of high DBP levels and the effectiveness of different corrective measures that can be implemented to reduce them, several water quality distribution models were developed. These community distribution system models represented very small and small sized water distribution systems from across the province representing Eastern, Central, Western and Labrador regions. Work on this modeling project first started in 2001 (Khan et al., 2002).

Based on a review of each distribution system for probable causes of high THMs observed in that community, different corrective measures to address these causes were selected and run as separate scenarios in the distribution system models developed. The development of a network model required an extensive review of the existing distribution system from which probable causes for excessive THM formation could be identified. Once probable causes for high THM levels were identified, specific corrective measures to address these problems could then be identified. Knowing which THM control methods offered the best probability of lowering THM levels, the identified solutions were then modeled (if possible) to determine their effectiveness. Measured against a set of solution constraints, each corrective measure can than be ranked for effectiveness, with the highest scoring measure offering the preferred solution(s). Distribution system models were used to develop and test out this integrated decision making framework approach for dealing with THMs, and in the development of generic BMPs to reduce THMs (and other DBPs) in problem water distribution systems of the province of Newfoundland & Labrador.

Although the results from this study can be used for any size distribution system, of particular concern in Newfoundland and Labrador is the number of very small and small communities with limited resources that have high THM levels. For this study, distribution system size has been defined based on population as follows:

- *Very small systems*: pop < 501
- *Small systems*: 501-1500
- *Medium systems*: 1501-15,000
- *Large systems*: 15,001-50,000
- *Very large systems*: > 50,000

The following table summarizes the communities with DBP problems identified for inclusion in this study, and each community's classification in terms of system size and region.

Table 41: Classification of modeled communities

Classification	1	2	3	4
Very small systems	Brighton	Burlington	St. Paul's	Hawke's Bay
Small systems	Cartwright	Ferryland		
Eastern	Ferryland			
Central	Brighton			

Western Labrador	Burlington Cartwright	St. Paul's	Hawke's Bay

5.1 Modeling Objectives
The success or failure of a model scenario is dependent on whether it not it meets a set of predetermined modeling objectives. All scenarios modeled as part of this report had to meet the following criteria in order to be deemed successful:

- All water entering the distribution system shall have at least a 20 min contact time at peak hourly flow (using the Harmon Formula or peaking table), and shall contain a free Cl residual of at least 0.3 mg/L at the first point of use, or equivalent CT value
- Maintain detectable free Cl residual (0.05-0.10 mg/L) in all areas of the distribution system (ie. end points)
- Satisfy a maximum residual chlorine disinfectant level of 4.0 mg/L
- Maintain system pressure between 275-650 kPa
- Maintain system water velocity below 1.5 m/s
- Maximum water retention time in a storage tank is 72 hours
- Changes in water level in a storage tank during daily domestic water demand should be limited to a maximum 9 m

The above criteria are mostly all derived from *Guidelines for the Design, Construction and Operation of Water and Sewerage Systems* released by the Department of Environment and Conservation in 2005.

5.2 Water Quality Modeling of Water Distribution Systems
Water distribution network models are computer models that can simulate the hydraulic behaviour of distribution networks. If accurate, the models can simulate hydraulic behaviour at any point in the network without direct measurement. They can also predict what would happen if different conditions were imposed on the network.

EPANET 2.0 was the network modeling software package used in this study. EPANET was selected because it is a combined hydraulic/water quality model, models bulk phase and pipe wall reactions, and has proven useful for modeling THM formation and maintenance of Cl residuals by other users. Any model that examines water quality in a distribution system requires modeling of hydraulic components first and a predefined set of assumptions. Assumptions made in conducting this hydraulic/water quality modeling included (Rossman, 2000):

- Conservation of mass within pipe lengths
- Complete and instantaneous mixing of water
- 1^{st} order decay of chlorine
- Solution to the hydraulic model of network established for water quality modeling
- Tanks are continuously stirred tank reactors (CSTR)
- Various component and operational assumptions

The different components of a hydraulic/water quality model include information on network components and their connections; pipe diameter, length, material, and roughness; characteristic operating curves for pumps; diameter, lower and upper water levels for each tank; control rules for pumps, pipes, and tanks; water demand and changes in water demand over time; initial water quality at all nodes and changes in water quality over time at source nodes; time step; volume in tank at start of simulation; and direction, velocity and flow in each link from the solution of the hydraulic model for use in the water quality model.

Table 42: Components, parameters and likely data sources for distribution system model development

Component	Parameters	Data Source	Comments
Network configuration	• Connectivity • x,y coordinates • node elevation	• Maps • Plans • GIS	• Records may contain errors
Pipes	• Internal diameter • Length • Age • Roughness	• Maps • Plans • GIS • Typical values	• True diameter by be less than nominal diameter due to corrosion, scaling
Valves	• Loss coefficient	• Assumed	
Storage	• Dimensions	• Drawings • Site survey • Field testing	• Can be difficult to measure storage volume with depth based on tank shape
Pumps	• Pump curve	• Manufacturer	• Pump performance will deteriorate with age due to wear
Demands	• Consumption • 24 hr profile	• Flow metering • Assumed	• Meter errors and under-recording a significant issue • 24 hr profiles can vary significantly across networks
Control rules	• Pump, tank, and valve control logic	• Consultation with operators	
Water quality coefficients	• Chlorine decay • THM growth	• Measure • Assumed values	• Some coefficients can

	vary by several orders of magnitude, making assumed values unreliable

All current distribution system water quality models are limited to track the dynamics of a single species as it is transported and changed via chemical reactions throughout the distribution system network. This is a serious limitation, however, as all water quality dynamics result from reactions between chemical species. The most common assumption for modeling chlorine within a distribution system is first-order chlorine decay kinetics. The bulk decay coefficient is a function of temperature, initial chlorine concentration and organic content in the bulk water. Wall demand coefficients depend on pipe characteristics such as material, age and rate of corrosion.

5.2.1 Modeling Storage Tanks

There are four basic models commonly used to simulate storage within a distribution network model such as EPANET (Rossman, 2000):

1. *CSTR*: Assumes that as soon as water enters the tank it becomes fully mixed with the water in the tank. Throughout the tank the water will be of uniform age and quality. Fill/draw tanks can often be simulated as a CSTR, providing that the inlet jet has reasonable momentum and there are no internal baffles or other obstructions.
2. *Plug Flow*: Assumes that the water progresses along a fixed path from the inlet to the outlet with no mixing. In this model, the oldest water is located at the outlet.
3. *Last in/First out*: Similar to a plug flow, but with a connected or adjacent inlet and outlet. No mixing of water and so the water drawn from the tank is always the youngest water in the tank. This model could be suitable for standpipe tanks with a common fill/draw where there is insufficient momentum in the inlet jet to promote mixing.
4. *Multi-Compartment*: The above models cannot simulate tanks which exhibit non-uniform behavior (eg. short-circuiting, dead zones or stratification). More complex behaviour can be simulated when storage is divided into zones.

The following figure demonstrates the difference between different tank models used in EPANET.

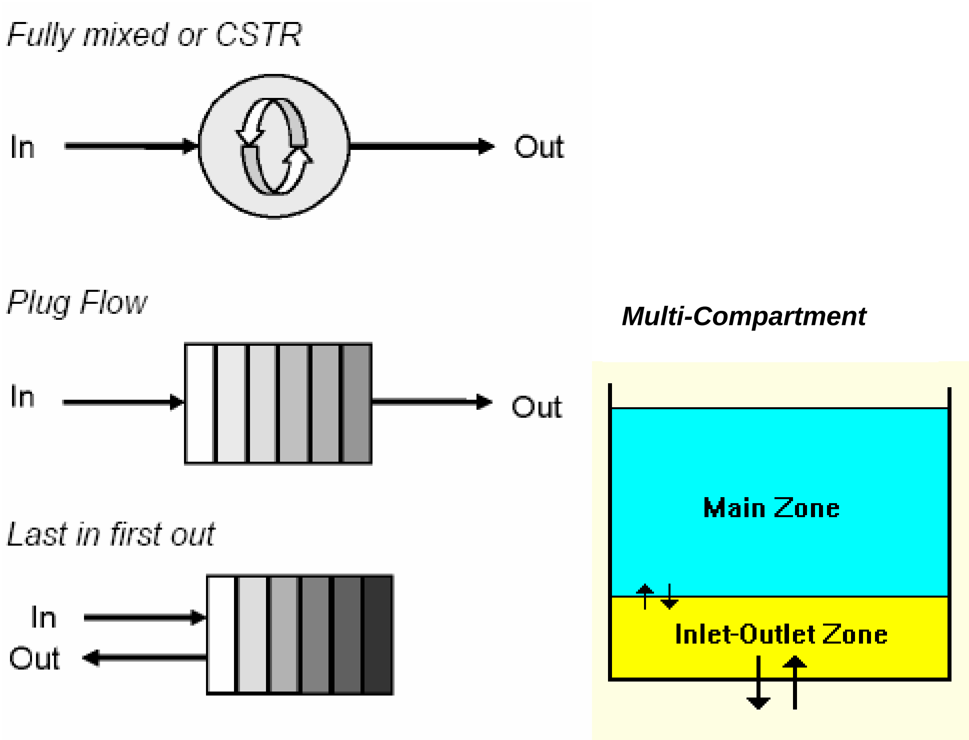


Figure 44: Typical tank model configurations

Although not used in this study, Computational Fluid Dynamic (CFD) modeling examines the motion of fluids in three-dimensions, and is the most comprehensive analysis tool available for evaluating water storage tanks. It can model hydraulics, chemical reactions, heat transfer, multi-phase flow and their interaction, all of which are useful in water quality modeling.

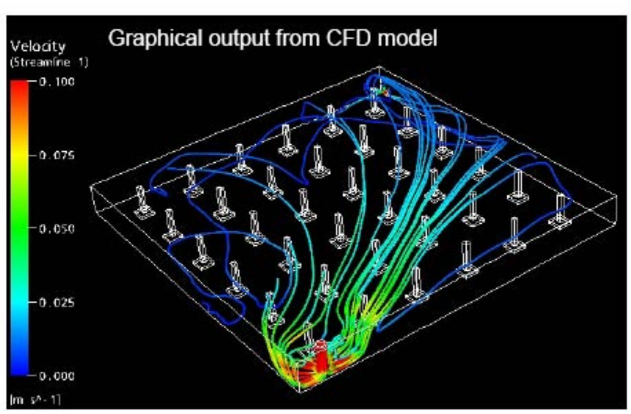


Figure 45: Graphical output from CFD model

5.3 Bulk Chlorine Decay and THM Growth Tests

Bulk chlorine decay and THM formation can be measured using water samples taken from the distribution systems of communities of interest. The following test procedure can be used to measure chlorine decay and THM growth coefficients:

1. Several clean 500 mL brown glass (organics) sample bottles were filled to the top, leaving no headspace, with water as close to the point of chlorination as possible.
2. Measured initial chlorine concentration and took one THM sample for analysis by laboratory and noted time as start time.
3. Bottles were stored at the same temperature as the sample water.
4. Measured the chlorine concentration from one of the 500 mL bottles and took a THM sample for analysis by laboratory and noted the time. Measured chlorine and took THM samples at intervals up to the maximum hydraulic retention time for that particular distribution system.
5. Plot chlorine and THM concentration versus sample time to establish decay and growth curves and determine decay coefficients using best-fit lines and curves.

The following table summarizes observed reaction rates from studies identified earlier in this report. All calculated and estimated bulk chlorine reaction coefficients derived for specific communities were within the normal range of observed coefficients from previous modeling studies.

Table 43: Typical chlorine and THM reaction coefficients

Reaction Coefficient Type	Normal Range
Bulk Chlorine decay (1/d)	0.26-17.7
Bulk THM growth (1/d)	0.5-5

5.4 Model Calibration

The initial model set up is based on assumed values for many parameters which are not directly measurable (eg. pipe roughness) or for which there was no data available (eg. daily demand pattern). Calibration involves comparing model predictions with field measurements and then adjusting model parameters to improve the fit between predicted and observed data. Parameters adjusted during calibration are typically those in which there is the greatest uncertainty.

The benefits of calibration include:

- Improved predictive capability of the model
- Provides an indication of the degree of confidence that can be placed in the performance of the model
- Learning about the behaviour of the network and any hydraulic limitations

The objective of model calibration is to reduce the percentage error between actual and modeled results as much as possible. Percentage error can range from 0 to several hundred percent, however, for modeling purposes in this report, a percentage error of 25 % or less was deemed desirable (if attainable). Models with the majority of parameters displaying less than 25 % error were thought to provide a fair representation of the actual system. Percentage error is calculated from:

$$\%Error = [(Experimental\ Value - Calibrated\ Value) / Accepted\ Value] \times 100$$

Equation 21: Percentage error

The inaccuracy of the model can be attributed to a number of factors including:

- Use of long term averaged values with seasonal data
- Inconsistencies in data due to system configuration changes
- The sensitivity of the system to changing water demand
- Use of approximate or design data for comparison purposes

5.5 Specifics of Corrective Measure Modeling

As certain factors have been identified as enabling excessive DBP growth in water distribution systems, several generic solutions to correct the problem have also been identified. However, not all of these corrective measures are conducive to water distribution system modeling, particularly the policy, source water, water treatment, alternative disinfectants, operator education and training, and POU/POE treatment measures.

Without water treatment to remove DBP precursors such as NOM and bromide, the only factors in THM formation that can be readily controlled are the chlorine dose and retention time of the water in the distribution system. These two factors can be managed through various identified infrastructural and operational mitigative measures.

The priority water quality parameters modeled in this study are chlorine and water age. THMs were not modeled at this stage due to a lack of reliable formation data and calibration data sets. Examining the percent of water from a particular source or tank can also be evaluated, but was not relevant for any of the distribution systems modeled. In most cases individual corrective measures were evaluated one at a time, however, some scenarios were run with multiple corrective measures incorporated into the model.

Each scenario selected for further investigation was run under extended period simulation. Initial parameter values were assigned at the source while all other nodes started from zero. The system was then simulated for a period (less than 10 days) until it reached dynamic equilibrium. Results of that particular scenario were only evaluated once dynamic equilibrium had been reached.

6.0 Brighton Water Distribution System Model

The Brighton water distribution system is typical of many small towns in Newfoundland & Labrador– a long linear system with a surface water supply whose raw water displays above average colour. When the pump is operating, it supplies water to both the community and the tank at a constant flow rate. When the pump is off, the tank supplies the community directly. Water levels in the tank direct the operation of the pump, cutting in when water levels fall below one quarter full, and cutting off once the tank is three quarters full. The chlorinator cuts in automatically once the pump does, and provides a constant dose that is proportional with flow. When the distribution system is operating on an automated rather than a manual basis, the pumps might not cut in for two days and at irregular hours based on tank level or system pressure. The Brighton distribution system can be classified as very small and from the Central Region of the province.

Figure 46: Brighton water distribution system network

Descriptive data for the Brighton water distribution system is detailed in following sections. This data was then input into the Brighton EPANET hydraulic/water quality model. The next step involved calibrating the Brighton model with system data also highlighted in the following sections. Different corrective measures and modeling scenarios were then selected based on observed problems with how the distribution system is currently operating. The potential effectiveness of the given solution or modeled scenario was then weighted against solution criteria and constraints.

6.1 Reservoir

The water supply for the town of Brighton is Hynes Cove Pond, located just off of Highway 380 and within half a kilometre of town. The intake is located 200 m into the pond and a berm or wet well was constructed around the intake to help deal with turbidity and colour problems. That berm system has since been flooded over due to an increase in the water level in the pond. The surface of the reservoir is at an elevation of 24.8 m.

Table 44: Average source water quality values for Brighton

Water Quality Parameters	Average Values 1988-2005
Colour (TCU)	39.7
pH	6.9
Turbidity (NTU)	0.67
Bromide (mg/L)	0.027
Chloride (mg/L)	6.5
DOC (mg/L)	6.4
Temp (ºC)	9.9
Iron (mg/L)	0.10
Manganese (mg/L)	0.022

Figure 47: Hynes Cove Pond

The Hynes Cove Pond watershed area is small at only 0.44 km^2.

6.2 Pumps

There are two Flygt submersible pumps operating on this distribution system. They are configured in parallel with only one pump operating at a time, and then operation switched over to the other pump the next time the pump cuts in. The pumps are supposed to cut in on an automated basis once the tank volume falls below one quarter full, and cut out once the tank volume is three quarters full. As the two pumps function in relay, it has been observed by the System Operator that the two pumps do not pump at the same rate, a fact indicated by chlorine consumption at the pump house. Flow meter observations indicate that when operational, the pumps produce a continuous flow of around 7.15 L/s, meaning the pumps are operating at peak efficiency. When the system is running automated, the tanks run for approximately 6 hours and are off for 30 hours.

Table 45: Brighton pump data

Pump Type	Power	Diameter	Rpm
Flygt 2070 submersible pump	4.5 KW	129 mm	3350

The following graph displays the performance curve for the two Flygt submersible pumps in use in the Brighton distribution system.

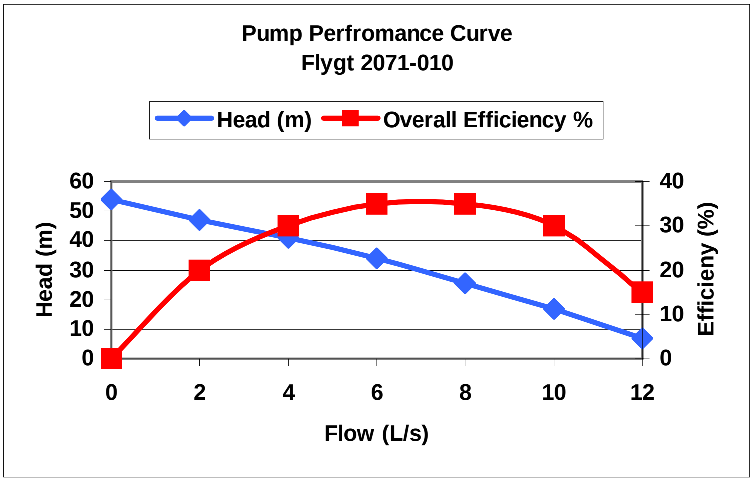


Figure 48: Brighton pump performance curves

Figure 49: Brighton pumps

6.3 Tank

There is a T-connection joining mains from the tank, pump house, and the rest of the distribution system. When the pump is in operation, water flows towards both the tank and the community. When the pump is off, the tank supplies water to the community directly. Water levels in the tank direct the operation of the pumps as previously mentioned. An overflow pipe siphons off water from the top of the tank once the tank is full. The inlet and outlet of the tank are at the same opening located at the base of the tank, meaning that the tank status is either filling or emptying. In this kind of standpipe tank, the water drawn from the tank is always the youngest water in the tank and mixing is potentially very poor. Based on observations made by the Brighton System Operator, when the system is running automated, the pumps might only cut in once every 2 days, resulting in a tank filling/emptying cycle of approximately 36 hrs (6 hours to fill and 30 hours to empty).

Table 46: Brighton tank data

Elevation	Height	Diameter	Volume	Max Water Level	Min Water Level
54 m	7.3 m	6.4 m	238 m^3	5.5 m	1.83 m

Figure 50: Brighton water storage tank

6.4 Pipes

The pipes in the current Brighton distribution system were installed over 6 phases beginning in 1986 and ending in 1992. Pipe from the source to just past the causeway is made of ductile iron starting at a diameter of 250 mm and then reducing down to 200 mm. From just past the causeway on, pipe material is PVC starting at a diameter of 200 mm and reducing down to 150 mm at the end of the system. In total there is approximately 3.1 km of trunk main laid down in the Brighton distribution system.

The Hazen-Williams head loss formula was selected for this model in order to determine energy losses throughout the system. Roughness factors were selected for each type of pipe: 130 for ductile iron pipes and 140 for PVC pipes.

6.5 Demand

From meter readings taken from the pump house over the month of September 2004, an average daily demand of 92.8 m^3/d was determined. An instantaneous flow reading of 7.15 L/s was observed in the pump house just after noon during the site visit. Based on a total of 104 water connections and a census town population in 2001 of 233 people, average daily water demand is 398 L/person/day. This overall demand was then attributed to 6 different junctions throughout the distribution network based on housing density surrounding that junction. Elevation of junctions with assigned demands ranged from 7.2 m to 1.2 m above sea level.

Meter readings had not been taken at a frequency to establish a daily demand pattern for the Brighton distribution system. Peaks in the morning, noon and evening are usual however. The following generic demand pattern was used in the Brighton model.

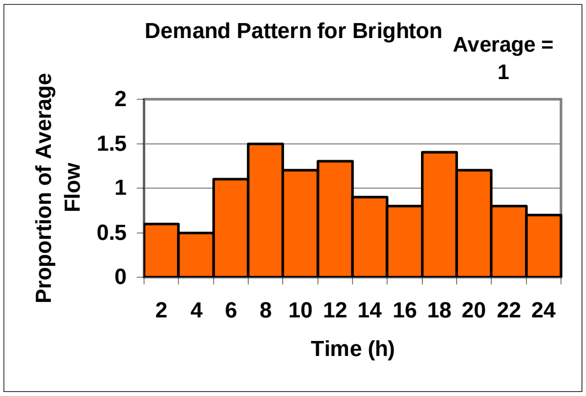


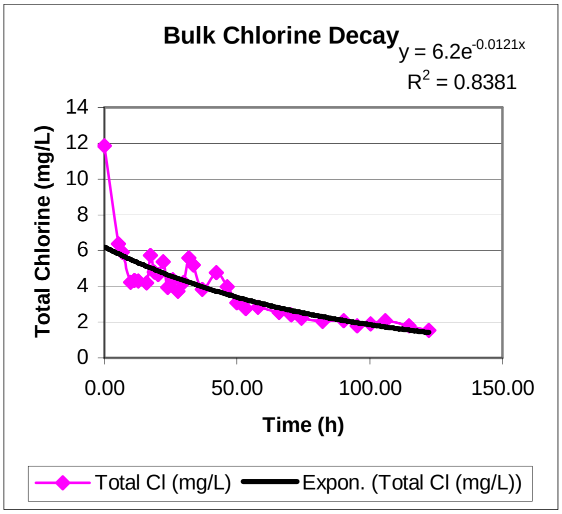
Figure 51: Typical daily water demand pattern

6.6 Chlorine Decay

The Brighton water distribution network has a liquid hypochlorination system. The chlorinator cuts in automatically once the pump does, and provides a constant dose that is proportional with flow. According to the operator, the system meets the following disinfection standards: i) all water entering the system, after a minimum 20 minute contact time, shall contain a free chlorine residual of at least 0.30 mg/L, or equivalent CT value; ii) a detectable free chlorine residual maintained in all areas of the distribution system. With the chlorination system at the pumphouse, and using an average daily flow of 92.8 m^3/d, the contact time at the first point of use is 145 minutes (water coming direct from reservoir) and 1021 minutes (water coming from the tank). Under worst-case conditions, using a chlorine residual of 0.08 mg/L taken from the field, the CT value at the first point of use is 11.6. Primary disinfection requirements are met on the Brighton system.

Figure 52: Brighton hypochlorination system

A bulk chlorine decay test was performed using water taken from the pump house directly after chlorine injection. Six, 1 L amber glass bottles were filled with water and kept at the source water's ambient temperature (9.3°C) during the decay test. Both total and free chlorine were tested over 5 days using a Hach portable chlorine meter. As the Hach meter only reads up to 2.20 mg/L, samples over this value were diluted down with de-ionized water so that readings could be taken. Bulk decay coefficients were determined for both free and total chlorine, -0.0108 h^{-1} (-0.2592 d^{-1}) and -0.0121 h^{-1} (-0.2904 d^{-1}) respectively. From these results a bulk decay coefficient of -0.3 d^{-1} was used for the model. A default wall decay coefficient of –1 m/day was also used prior to calibration.



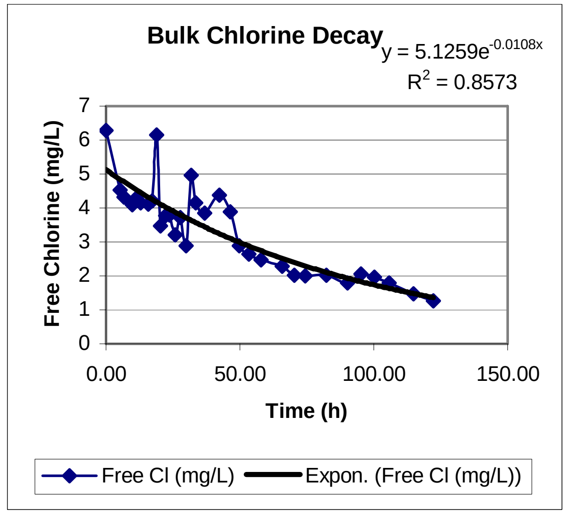


Figure 53: Chlorine decay test coefficients

6.7 Site Visit of Sept 24, 2004

Three members of the Water Resources Management Division visited the town of Brighton on Sept 24, 2004 in order to gather data on the distribution system. This involved discussions with the system operator, Edmond Fudge, and town clerk, Gloria Fudge, and also measurements taken on the system.

At the time of the site visit, the system was being run manually with the pump being turned on 8:30 in the morning, and being shut down 4-5 hrs later. The automated switch that is supposed to turn the pump on and off at certain tank water levels had not been functioning since May 2004. During the site visit, the tank was observed to be overflowing before the pump was shut off.

There are approximately 14 fire hydrants located on the Brighton distribution system. Using a pressure gauge that attached to the hydrants, pressure readings were taken at the end and middle of the system. The pressure gauge in the pumphouse was also checked against a field gauge, and readings were found to agree. Pressures in the system are within normal range.

Table 47: Brighton network pressures

Location	Junction	Pressure (psi)	Pressure (kPa)	Pressure (m)	Comments
End of System	7	90	621	63	
Middle of System	8	87	600	61	
Pump house	Pump2	53	365	37	Tank full

Figure 54: Brighton system pressure reading from fire hydrant

Chlorine readings were also taken at different points on the distribution system. The chlorine reading at the end of the system was taken after flushing the hydrant for 10 minutes and may have affected that result. During the site visit, the chlorinator was observed to have an air bubble blocking the flow of chlorine into the distribution system. Chlorine readings at the pump house were only taken after the blockage was removed. It must be assumed that chlorine found in other parts of the system was at least 24 hours old.

Table 48: Brighton network chlorine residuals during site visit

Location	Junction	Time	Free Chlorine (mg/L)	Total Chlorine (mg/L)
Town Council Office	8	11:10	0.10	0.14
End of system fire hydrant	7	11:40	0.19	0.50
Tank	Tank2	13:00	0.07	0.33
Pumphouse	Pump2	13:10	6.28	11.84
1st house on system	4	13:30	0.08	0.33

6.8 Chlorine and THM Data Gathering

Besides the chlorine tests taken in the field, chlorine tests are regularly made by the Brighton System Operator and by Department of Environment staff. The following table summarizes average chlorine and total THM results. Negligible chlorine readings at the beginning and middle of the distribution system indicate problems with the operation of the system and so chlorine readings for those dates were removed.

Table 49: Average chlorine, THM and BDCM (1998-2006) readings on Brighton network

Location in Network	Junction	Free Chlorine - Town (mg/L)	Free Chlorine-DOEC (mg/L)	THM Total-DOEC (ug/L)	BDCM-DOEC (ug/L)
Beginning	5	1.00	-		
Beginning	4	-	0.92	300	10.5
Middle	6	-	0.71	271	11.9
Middle	8	0.48	-		
End	7	0.30	0.22	258	14.5

The CCME maximum acceptable concentration (MAC) for total THMs is 100 ug/L. As shown in the table, THM levels in Brighton are well over the limit.

6.9 Calibration of the Brighton Model

In order to first calibrate the Brighton hydraulic/water quality model, results were compared with flow, pressure, tank filling/emptying cycles and chlorine residual data collected from the Brighton distribution system. The collection of this data is outlined in previous sections. Because of issues with the system operation (air bubble in the chlorine line) on the day of the site visit, only some of the chlorine data gathered on Sept 24th, 2004 can be used for comparison.

Comparison of initial model results to calibration data is described in the following table, along with actions taken to compensate for any discrepancies, and final associated percentage errors found in the calibrated model. Average values from the model are taken for comparison once equilibrium or periodic behaviour from that parameter had been reached.

Table 50: Calibration of Brighton model

Issue	Percentage Error	Action	Percentage Error After Calibration
-6.5 L/s model flow during tank filling vs. observed instantaneous flow of 7.15 L/s	9.1%	-changed to higher design demand regime 450 L/p/d -adjusted pump curve by increasing head values by 5m to be more in line with constant pump power of 4.5kW	6.3% (6.7L/s)
-node 7 model pressure ranges from 54-57m vs. observed 63m -node 8 model pressure ranges from 53-57m vs. observed 61m	-11.9% -9.8%	-reduced PVC pipe roughness coefficient to 140 -reduced DI pipe roughness coefficient to 130 -increased tank elevation by 3 m	-5.5% (59.5m) -3.3% (59m)
-tank fills on a 40hr cycle vs. observed approximate 36hr filling/emptying cycle	16.7%	-increased tank elevation by 3 m -adjusted pump curve by increasing head values by 5m to be more in line with constant pump power of 4.5kW	2.8% (35hrs)
-headloss across pump 32m vs. observed pumphouse pressure of 37m	13.5%	-reduced PVC pipe roughness coefficient to 140 -reduced DI pipe roughness coefficient to 130 -increased tank elevation by 3 m	2.7% (36m)
-node 7 equilibrium (after 45hr) Cl of 0.17 mg/L vs.	-34.6%	-increased bulk reaction rate from -0.3 d^{-1} to -0.4d^{-1}	-53.8% (0.12mg/L)

observed average of 0.26 mg/L		-increased wall reaction rate from –1 to –1.5	
-node 8 equilibrium (after 20hr) Cl of 0.49 mg/L vs. 0.48 mg/L	-2.1%	-increased bulk reaction rate from -0.3 d^{-1} to -0.4d^{-1} -increased wall reaction rate from –1 to –1.5	-14.6% (0.41mg/L)
-node 6 equilibrium (after 15hr) Cl of 1.2 mg/L vs. observed average of 0.99 mg/L	-21.2%	-increased bulk reaction rate from -0.3 d^{-1} to -0.4d^{-1} -increased wall reaction rate from –1 to –1.5	-1.0% (1.0mg/L)
-node 5 equilibrium (after 12hrs) Cl of 1.5 mg/L vs. 1.0 mg/L	-50%	-increased bulk reaction rate from -0.3 d^{-1} to -0.4d^{-1} -increased wall reaction rate from –1 to –1.5	-30% (1.3mg/L)
-node 4 equilibrium (after 10hr) Cl of 2.0 mg/L vs. observed average of 1.49 mg/L	-34.2%	-increased bulk reaction rate from -0.3 d^{-1} to -0.4d^{-1} -increased wall reaction rate from –1 to –1.5	-20.8% (1.8mg/L)

Once results predicted by the model were felt to adequately reflect observed field data– matching pressures, tank filling/empting cycles, flows, chlorine residuals– through the adjustment of certain network parameters, a baseline model was established. The different model scenarios will then be run on this baseline model, adjusting only selected network parameters.

The following table and graph show calibration statistics for pressure in the Brighton distribution system. Observed pressure readings taken from the field were assigned times at the midpoint of the pressure cycle to give an indication of how closely matched simulated and measured values are. There was very little error observed between field and modeled system pressures indicating a near perfect correlation.

Table 51: Brighton calibration statistics for pressure

Location	Num Obs	Obs Mean	Comp Mean	Mean Error	RMS Error
8	1	61.00	58.41	2.592	2.592
7	1	63.00	58.91	4.094	4.094
Network	2	62.00	58.66	3.343	3.426

Correlation Between Means: 1.000

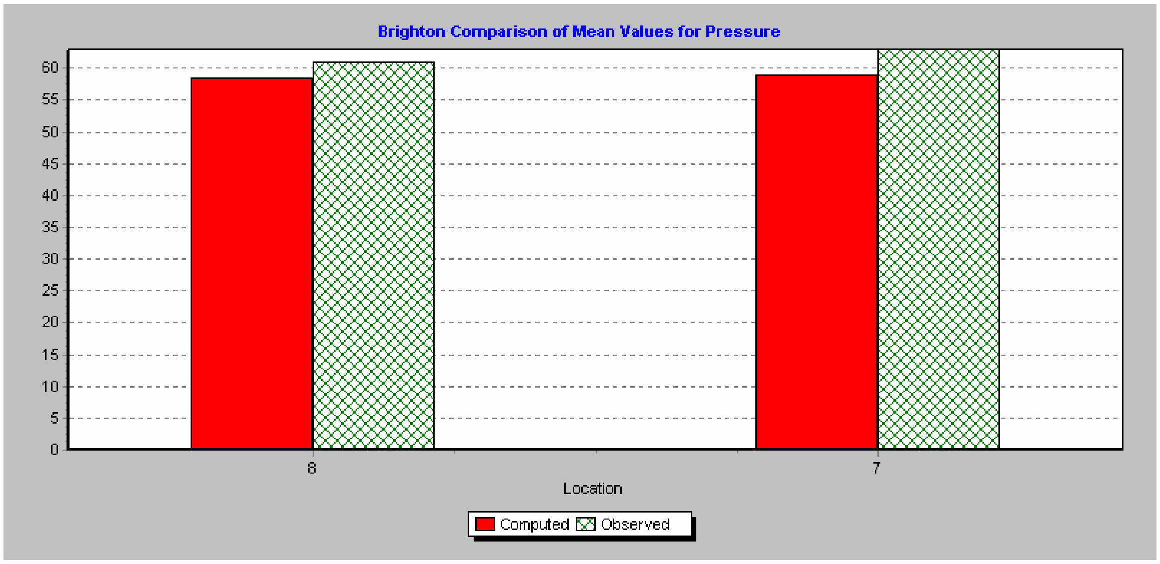


Figure 55: Mean observed and mean simulated value for pressure in Brighton

The following graph shows tank water level variation over the 7-day simulation period. It indicates the tank is on a 36-hour filling/emptying cycle, similar to observed tank operation.

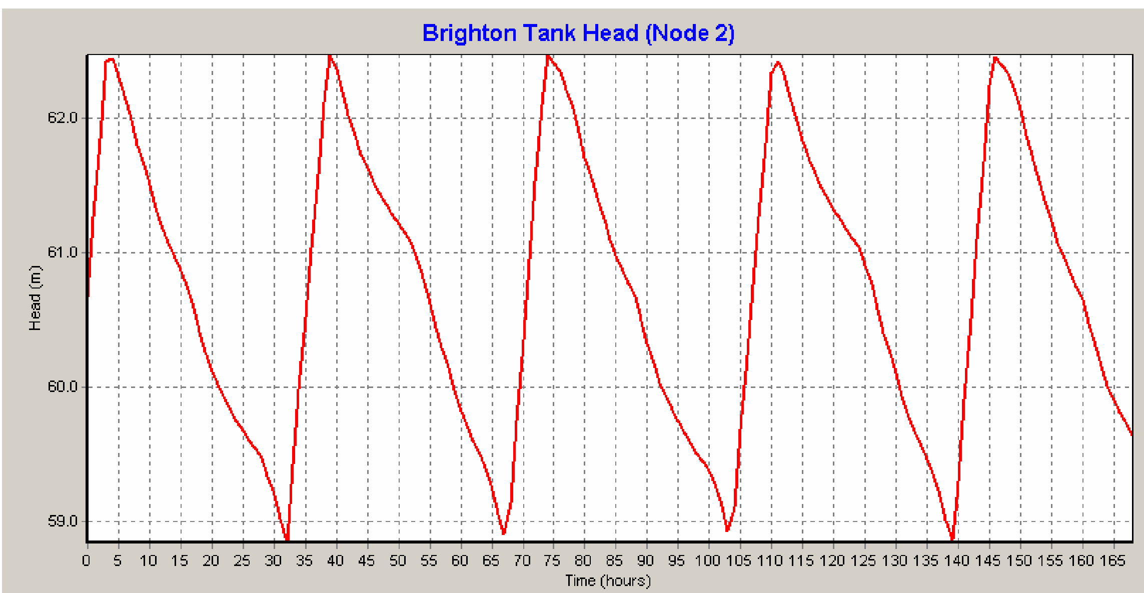


Figure 56: Brighton tank operation

The following graph shows flow coming from the pump over the 7-day simulation period. When the Brighton system is run on automated, the pump cuts in when water levels fall below a certain level in the tank. The instantaneous flow of 7.15 L/s observed in the field from the pump house meter is matched closely by simulation results.

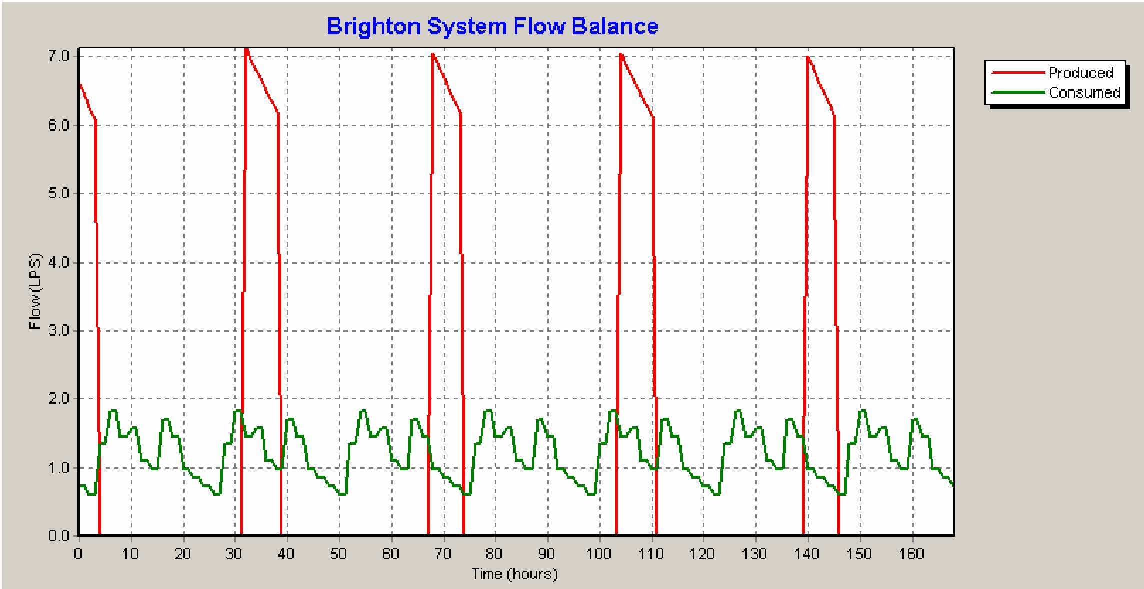


Figure 57: Brighton pumped flow and system demand

The following table and graph show calibration statistics for free chlorine residuals taken from five different points in the Brighton distribution system. Observed chlorine readings taken from the field were assigned times after equilibrium had been reached for each node. Once chlorine reached equilibrium, it still varied significantly, pulsing with pump operation. A median point along this chlorine pulse cycle was used to compare simulated to observed results. There was little error observed between field and modeled chlorine residuals indicating a very good correlation.

Table 52: Brighton calibration statistics for chlorine

Location	Num Obs	Obs Mean	Comp Mean	Mean Error	RMS Error
5	1	1.00	1.30	0.298	0.298
4	1	1.49	1.84	0.353	0.353
6	1	0.99	0.99	0.002	0.002
8	1	0.48	0.41	0.070	0.070
7	1	0.26	0.12	0.138	0.138
Network	5	0.84	0.93	0.172	0.218

Correlation Between Means: 0.988

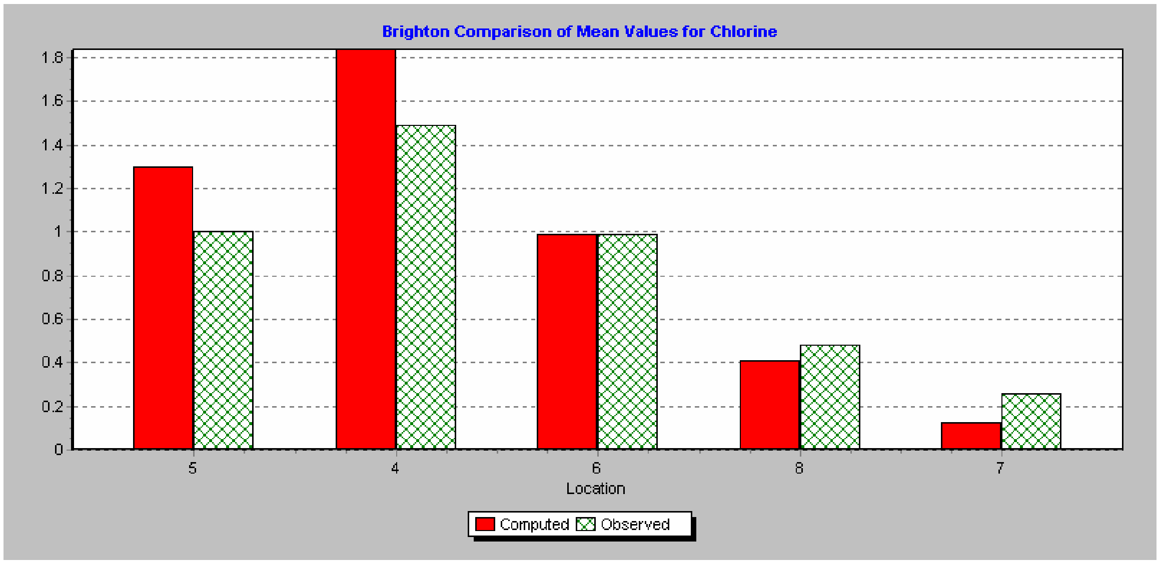


Figure 58: Mean observed and mean simulated value for chlorine residuals in Brighton

6.10 Problems with the Brighton Distribution System and Appropriate Corrective Measures

By gathering detailed background information on the Brighton distribution system and establishing a calibrated baseline model, we were able to identify problems with how the system operates normally. According to the model results, chlorine residuals appear adequate at the end of the system, corroborating observed Dept of Environment field data and observations made by the Brighton System Operator. Adequate chlorine residuals are desirable, but in this case, are leading to problems with elevated THM levels. Several contributing factors were identified as contributing to the overall THM problem as outlined in the following table.

Table 53: Problems contributing to high THMs in the Brighton distribution system

	Causative Factors	Quantitative Value
1	Reservoir contains flooded vegetation	
3	Surface water source exposed to saltwater influence	300 m (NW)
5	High DOC in source water	6.4 mg/L
6	High levels of bromide in source water	0.027 mg/L
10	Excessive chlorine demand	-0.4 d-1 (bulk) -1.5 m/d (wall)
12	Long linear system	3.1 km intake to end total = 3.1 km
14	Distance of chlorination system to 1st point of use	1 km contact time= 145 min CT = 174
15	Insufficient chlorination controls on system	manual
16	System is oversized	0.01-0.12 m/s 250-150 mm Q_{avg} = 1.07 L/s

17	High retention time in network	max = 102 hrs
22	Balance between pumped supply and demand not optimized with storage	6 hr to fill/ 30 hrs to empty
23	High retention time in tank	max = 57 hrs
24	Dead zones/poor mixing in tank	25% inactive volume
26	Poor O&M of system	
27	Multiple factors	
28	Poor design of system	

The following figures illustrate some of the problems observed in the Brighton distribution system.

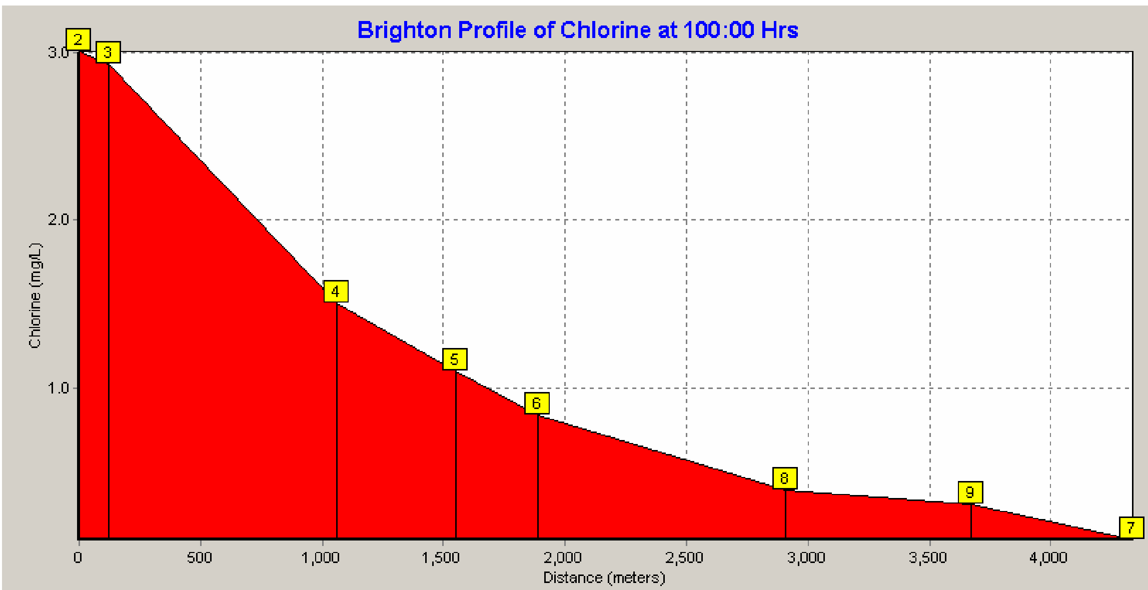


Figure 59: Chlorine decay profile through Brighton distribution system

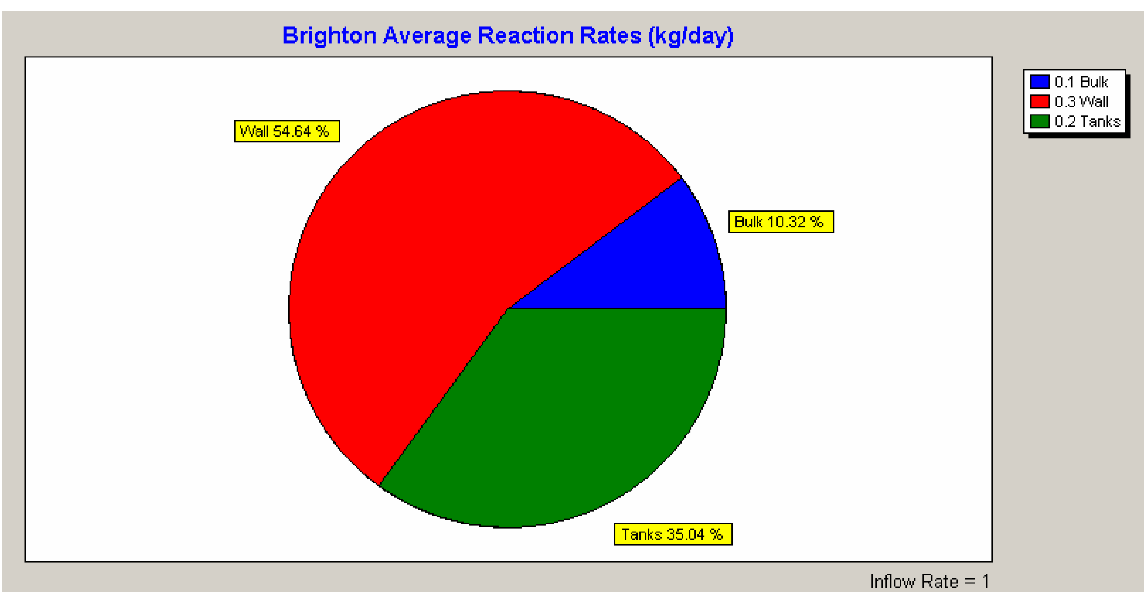


Figure 60: Chlorine decay contributions in Brighton distribution system

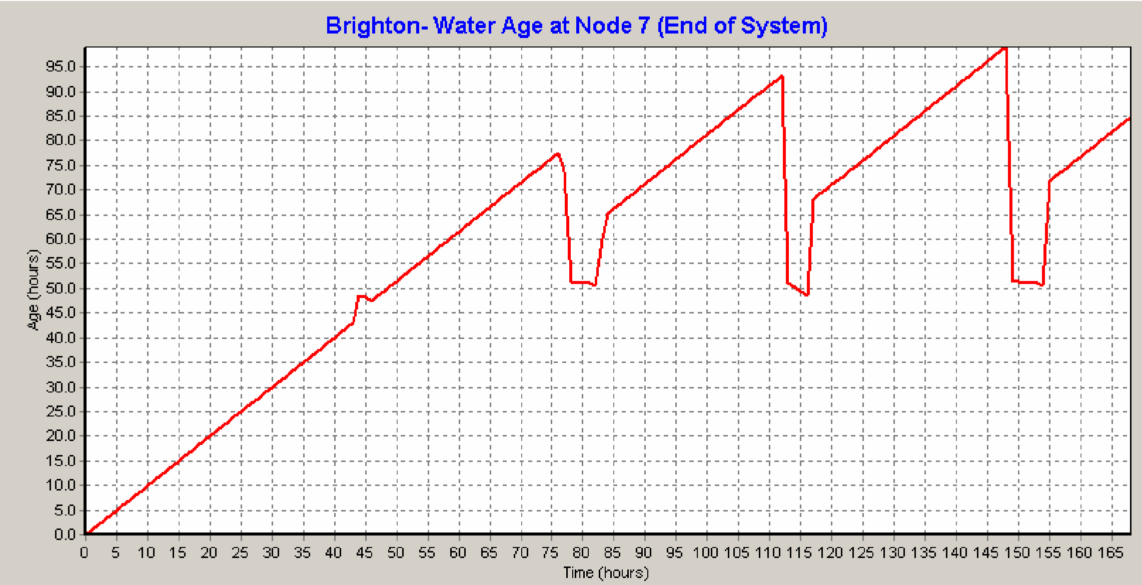


Figure 61: Water age at end of Brighton distribution system

Solutions that might address the probable causes of high THM levels in the Brighton distribution system are outlined in the following table. Those corrective measures highlighted in grey are the only solutions that can potentially be modeled.

Table 54: Applicable THM corrective measures for Brighton

Applicable Corrective Measures	Probable Causes Addressed
Policy of POU/POE treatment	All
Policy to promote use of alternative disinfectants	All
Policy to promote PWDU	All
Watershed protection	All
High quality water storage and recovery	All
Alternative disinfectants	All
System maintenance	All
Potable water dispensing unit	All
Point of use/entry treatment	All
Regionalization	All
Training	All
Improved design of system	All
Alternative water sources	1-3-5-6
Remove submerged vegetation	1-5
Wind breaks around exposed costal water sources	3-6
Optimize disinfectant dosage	1-3-5-6
Relocate chlorination system	1-3-5-6-14
Install chlorine booster at optimal location	10
Chlorine dose control	1-3-5-6
Tank location (multiple smaller tanks)	22-23
Adjusting pump schedule	17-22-23-24-25

Reduce storage capacity	17-22-23
Increase mixing in tank	17-22-23
Regular system flushing at dead ends	1-5-12-16-17
Continuously bleed system at dead end	1-5-12-16-17
Downsizing mains	1-5-12-16
Water treatment plants	5
Filtration	5
Advanced treatment	3-5-6
Combination of corrective measures	All

6.11 Results from the Brighton Modeling

The next step was to model the different selected corrective measures and see how the Brighton distribution system responded. Given the ability of the baseline model to reflect current conditions accurately, a reasonable degree of confidence can be placed in the scenario results.

6.11.1 Optimize Chlorine Dosage

The Brighton distribution network has a hypochlorination treatment system located in the pumphouse. An approximate 10% solution of sodium hypochlorite, similar to bleach but 3-5 times stronger, is the most common form of disinfectant used with such systems. The sodium hypochlorite solution is diluted down to the required level using water and stored in the polyethylene or fiberglass hypochlorination container or jug. Typical volumes for this jug are 100-200 L. The small chlorine pump operates only when the system pump is also functioning, and applies a constant chlorine dose directly into the water stream of the pipe just before the water leaves the pumphouse. The hypochlorite stock solution in the container needs to be filled at least once a week. Flow rates for chlorine suction pumps can range from 5.5 L/d to 1442 L/d. The Brighton chlorine pump is on the lower end of this scale, pumping approximately 25 L/d of solution. From collected field data, the free chlorine residual of the water leaving the pump house is 6.28 mg/L and was used as the chlorine dosage in the model.

In EPANET we have chosen to model chlorine at the source as a fixed concentration, by setting the initial quality at the reservoir (node 1) to 6.28 mg/L. This will be the concentration of chlorine that continuously enters the network when the pump is operational. The following graphs show a selection of decreasing chlorine residuals with increasing distance from the source at two different chlorine dosages.

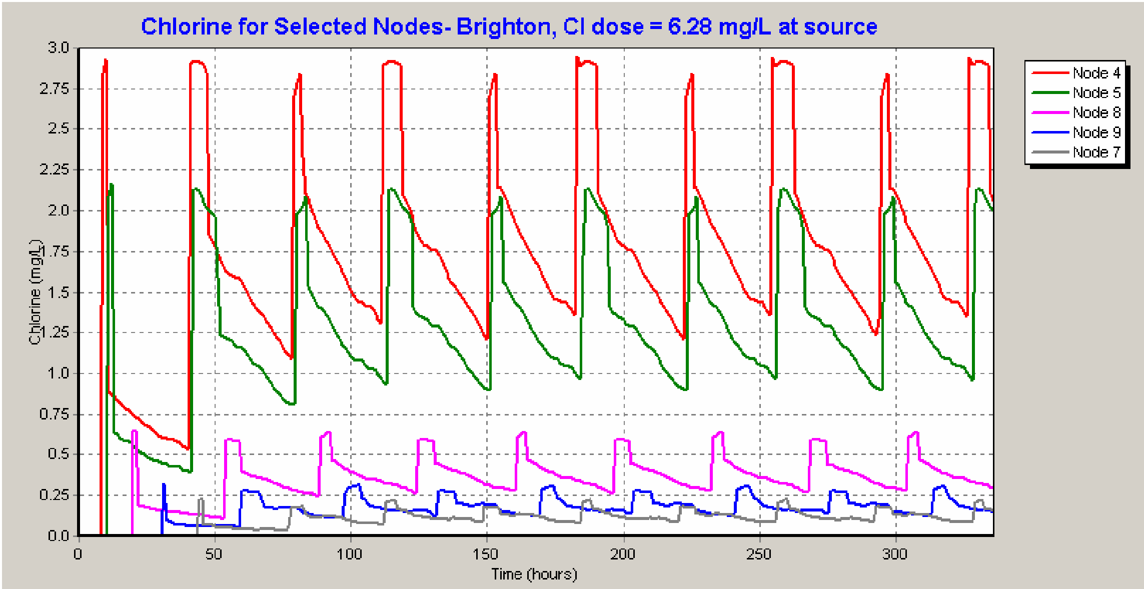


Figure 62: Chlorine levels in Brighton network with a chlorine dose of 6.28 mg/L

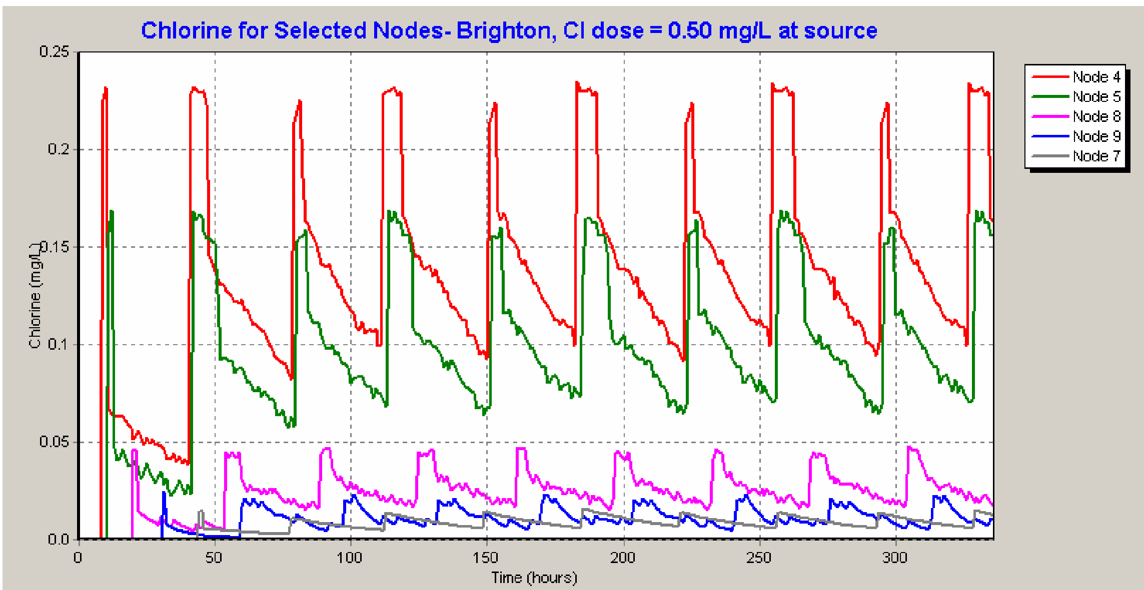


Figure 63: Chlorine levels in Brighton network with a chlorine dose of 0.50 mg/L

The following table summarizes the results of altering chlorine dosage.

Table 55: Altering chlorine dosage in Brighton distribution system

Chlorine Dose (mg/L)	Calculated Contact Time (hrs)	Water Age at Beginning of System-Node 4 (hrs)	Average Chlorine Residual at Start of System (mg/L)	Min Chlorine Residual at Start of System (mg/L)	Average Chlorine Residual at End of System (mg/L)	Min Chlorine Residual at End of System (mg/L)
6.28	5.7	15-65	1.80	1.10	0.12	0.07

5	5.7	14-65	1.50	0.70	0.09	0.06
4	5.7	13-65	1.20	0.70	0.07	0.04
3	5.7	12-63	0.90	0.50	0.05	0.03
2	5.7	11-62	0.60	0.40	0.03	0.02
1	5.7	10-61	0.3	0.20	0.02	0.01
0.5	5.7	10-60	0.16	0.10	0.01	0.01

Adequate contact time is not a problem in the Brighton system, however maintaining an adequate chlorine residual at the end of the system using a single chlorination unit located in the pumphouse at the very beginning of the system is a problem. In this case, the chlorine residual leaving the pumphouse must be kept above 5 mg/L. If we are only trying to maintain a residual of 0.3 mg/L at the first point of use, however (as in the case of adding a chlorine booster somewhere in the system), chlorine residual leaving the pumphouse must be kept above 2 mg/L.

6.11.2 Relocate Primary Chlorination System Location Closer to First User

The chlorination system on the Brighton network is located in the pumphouse next to the reservoir. For this scenario, the system was modeled with the primary chlorination system located on the town side of the T (node 3) where water coming from the pumphouse either goes to the town or the tank. At this location (node 10), water flowing to the community is still being chlorinated if it is coming from either the tank or direct from the reservoir. At this new location, the contact time is decreased from 145 minutes (water coming direct from reservoir) to 136 minutes due to reduced pipe length, more than sufficient to achieve the 20-minute contact time requirement.

At a chlorine dose of 3 mg/L, with the chlorination system located on the community side of the reservoir-tank-community T, adequate chlorine residuals are maintained throughout the system. By bypassing the tank, the average time available for THM formation in the Brighton distribution system is almost halved, reduced by approximately 37 hours from a previous water age of 102 hours. Variation in chlorine residuals is also significantly reduced.

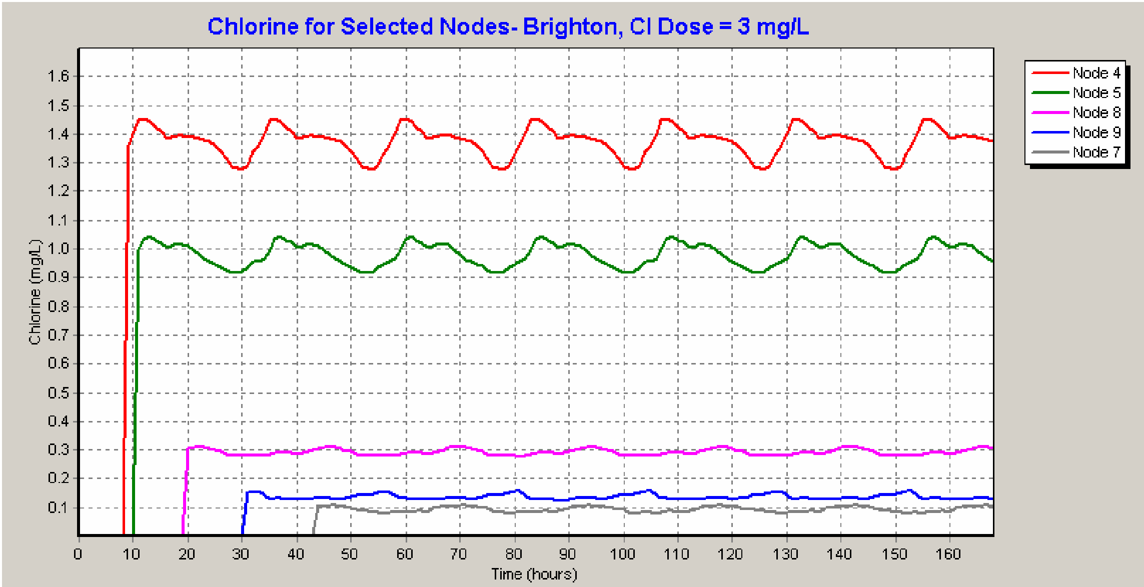


Figure 64: Chlorine levels in Brighton network with chlorination system moved closer to first user

6.11.3 Chlorine Dosage Control

The chlorination system only functions when the pump is operational, injecting a constant chlorine dose with the constant water flow volume of 7.15 L/s for the 6 hours the pump is typically operational. As there is currently no variation in flow from the pump when operational, a constant chlorine dose is appropriate. Water quantity (flow) and quality (chlorine residual) feedback controls could be used to manage the chlorine feed, if the existing chlorination system was upgraded or the location moved.

For this simulation the location of the chlorination system was changed to the community side of the reservoir-tank-community T, as with the previous simulation. The chlorine dose was made to vary with time using a time pattern similar to that used for water demand. Optimizing this time pattern proved rather difficult, however. Typically, feedback control of chlorination systems function by increasing the chlorine dose when flows increase in order to maintain CT values at the first point of use. However, when demand is high, water moves faster through the distribution system, water age is reduced and chlorine does not decay as fast resulting in higher chlorine readings. Chlorine values mimic the peaks and lows of flow values at specific locations, but there is usually some lag. The lag time observed between a peak in flow and the corresponding peak in chlorine increases the further you get towards the end of the distribution system. The following two graphs look at chlorine readings throughout the network if the chlorine dosage increases proportional to flow and inversely proportional to flow. Variation in chlorine readings has increased significantly over a constant dosage.

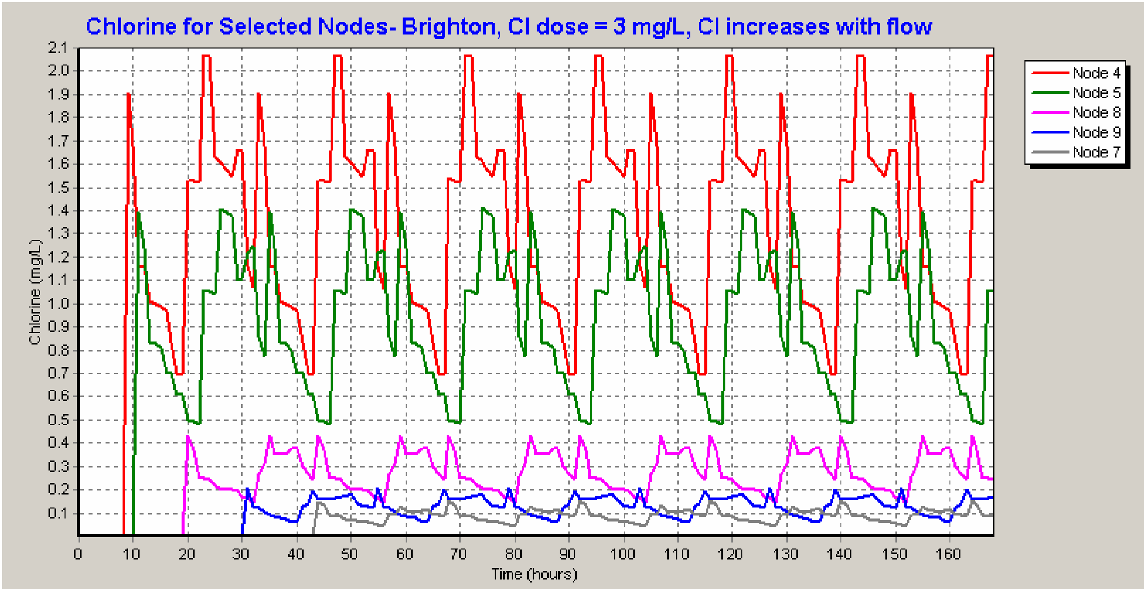


Figure 65: Chlorine control proportional to flow in Brighton network

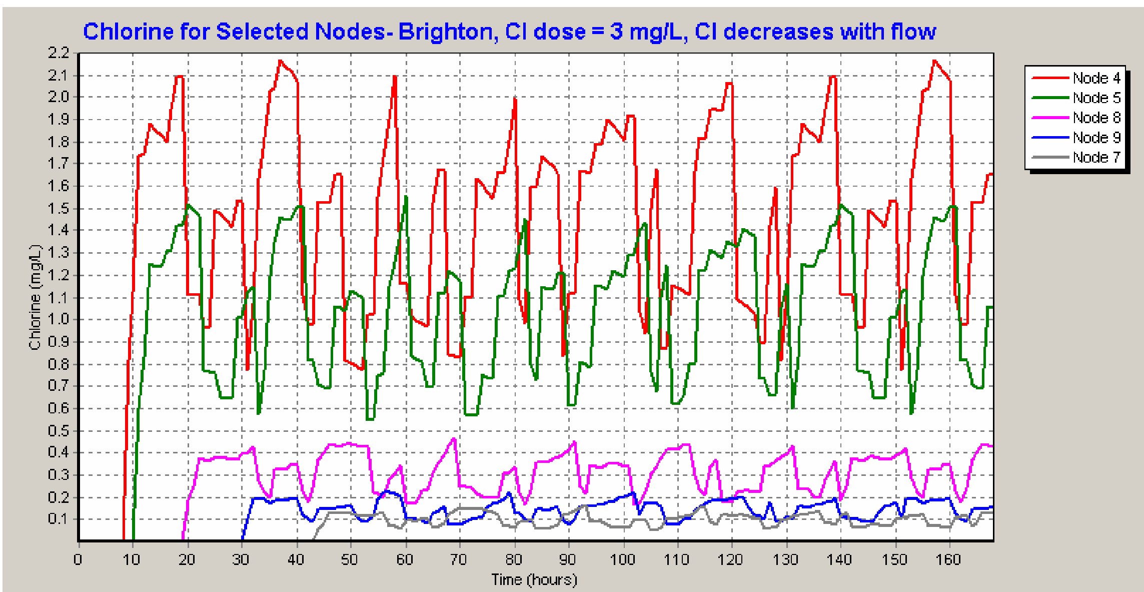


Figure 66: Chlorine control inversely proportional to flow in Brighton network

In the case of Brighton, this lag time increase between peaks in flow and the corresponding chlorine peak was on the range of 20-35 hours from the beginning to the end of the system. As the different nodes are completely out of sink in terms of when their maximum and minimum chlorine values occur, trying to come up with an appropriate time pattern to control chlorine dosage required a middle of the road approach. The time pattern selected to try and decrease chlorine peaks and increase chlorine trough values throughout the distribution system was based on a lag of 10 hours from the flow demand pattern. Where corresponding flow multipliers were above 1, a chlorine multiplier of 1.2 was input, and for flow multipliers below 1, the corresponding chlorine multiplier was 0.8. The chlorine dose remained at 3 mg/L.

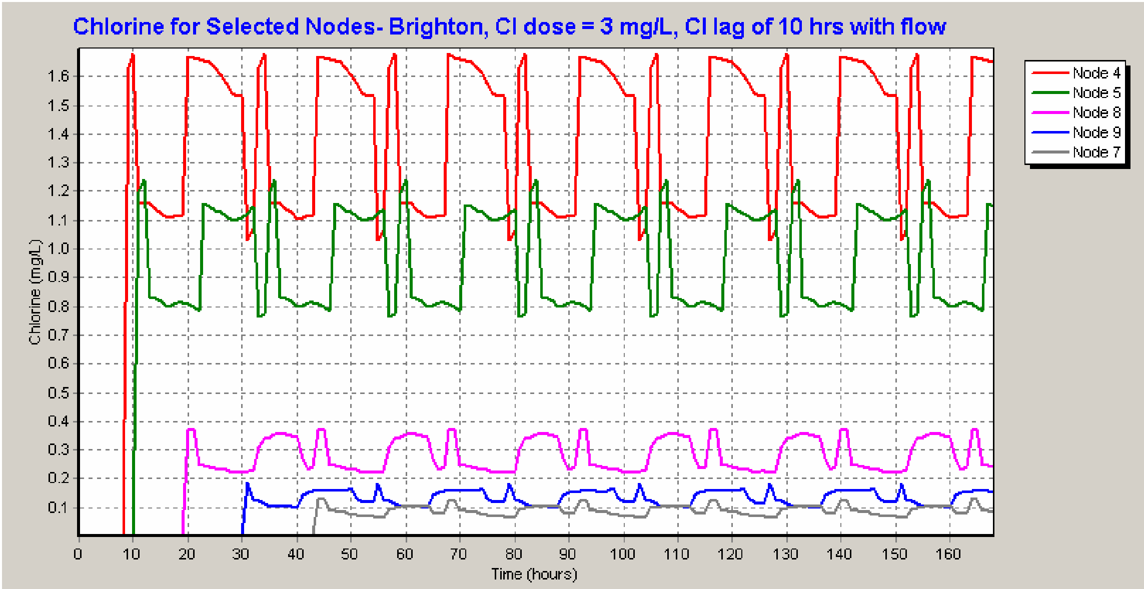


Figure 67: Chlorine control with ten hour lag from flow in Brighton network

As the above graph indicates, there was no advantage to an adjustable chlorine dosage over a constant dose. Chlorine values were no higher, meaning there was no opportunity to reduce overall dosage, and there was greater variability in chlorine values over a constant dose. This scenario indicates that there are complicating factors involved with controlling chlorine dosage that will not make it work for all parts of the distribution system.

6.11.4 Tank Location/ Multiple Smaller Tanks

Two different scenarios were looked at for this potential corrective measure: moving the existing tank towards the end of the distribution system, and having two smaller tanks on the network, one at the beginning and one at the end of the system. Moving the existing tank near the end of the network (off of node 9) while keeping the chlorine dosage at 6.28 mg/L results in wildly varying chlorine readings throughout the system, chlorine values over 4 mg/L at the beginning of the system, and minimum values below 0.05 mg/L at the end of the system. The amount of chlorine decay in the tank decreases significantly however from 35.04 % to 2.6%. The tank still fills and empties on the same 6 to 30 hour cycle it was on previously. Because the tank is now located at the end of the system, it acts as a large demand when filling causing a spike in water flows and velocity throughout the distribution network once a cycle. With the tank at the end of the system, the maximum water age in the tank increases to 113 hrs (maximum water age in the system becomes 127 hrs), which violates the maximum water retention time allowed in a storage tank of 72 hrs. Pressures within the system ranged from 41-50 m which is within acceptable range.

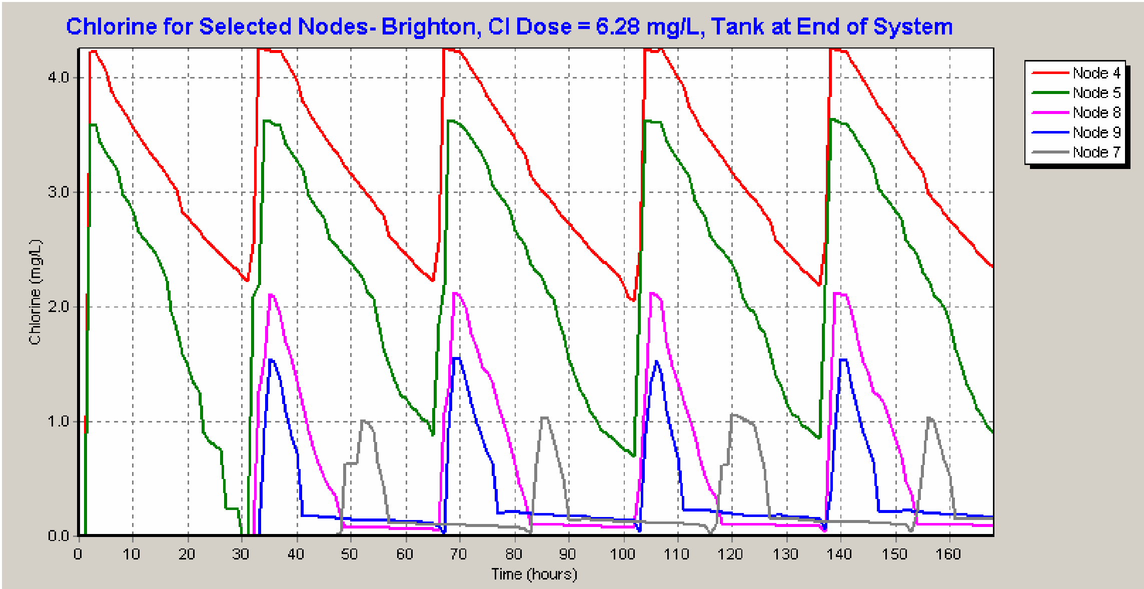


Figure 68: Chlorine levels with Brighton tank at end of system

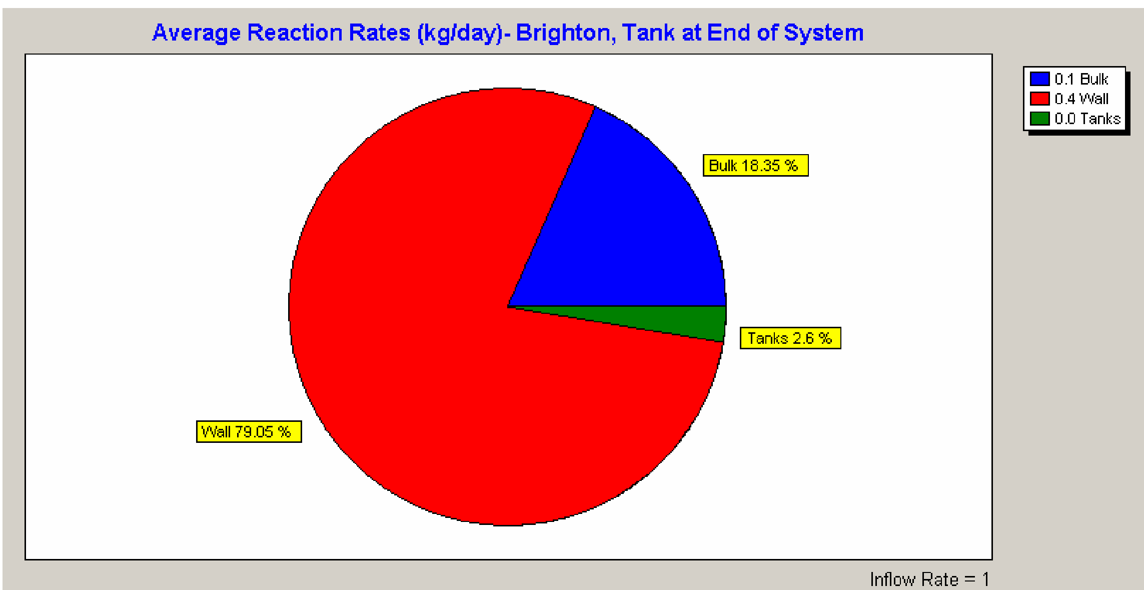


Figure 69: Chlorine decay rates with Brighton tank at end of system

For the second scenario, two tanks half the volume of the existing tank were placed on the system at the beginning and end (off node 9) of the network and operate in tandem, only one supplying water to the distribution system at a time. A chlorine dosage of 6.28 mg/L was maintained. While chlorine values were less variable than with just one tank on the end of the system, there were still peaks above 4 mg/L at the first node, and values that fell below 0.05 mg/L at the end. In this scenario the percentage of chlorine decay in the tanks was only 4.25%. The filling/emptying cycle with two tanks is now 2 hours to fill, 9 hours of draw down, alternating between tanks, meaning much more wear and tear on the pump. When the tank on the end of the system is filling, acting as a large demand on the system, flow and velocity through the distribution system spike. Pressure in the system ranges from 40-57 m, which is within acceptable range. Maximum water age in the beginning tank is 22 hrs, while water age in the end tank and at the end of the system increases constantly with time.

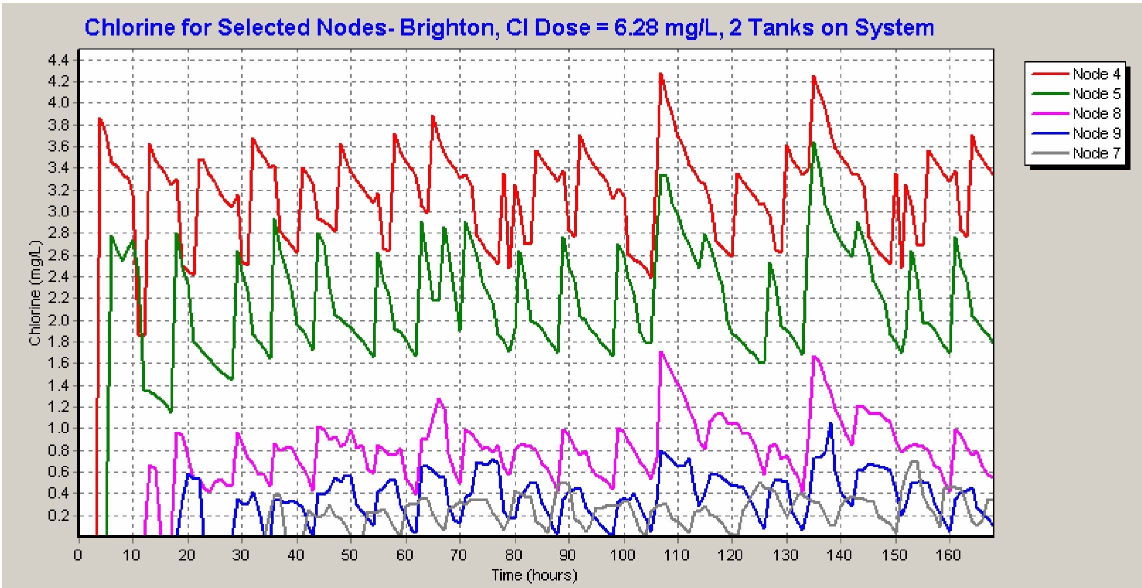


Figure 70: Chlorine levels with two tanks on Brighton network

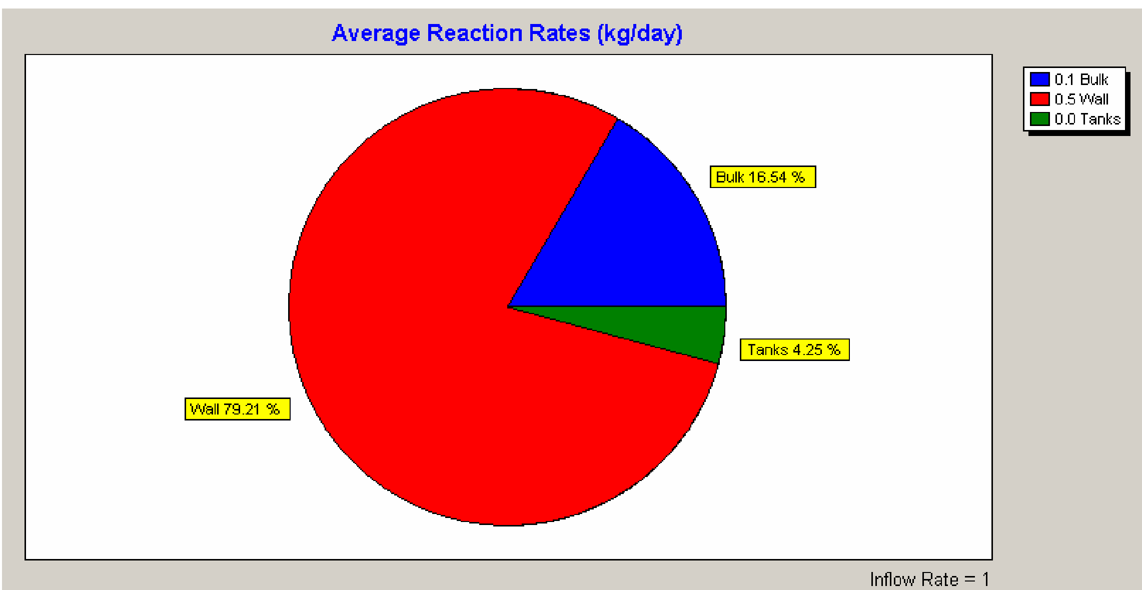


Figure 71: Chlorine decay rates with two tanks on Brighton network

Neither scenario met all requirements to be deemed successful.

6.11.5 Reducing Tank Storage Capacity/ Adjusting Pump Schedule

Reducing the tank storage capacity and adjusting the pump schedule are modeled in the same way. The water levels in the tank are supposed to trigger pump operation when the Brighton distribution is running on automated. At one-quarter full, the pump is supposed to turn on and at three-quarters full, the pump is supposed to turn off, actively utilizing 50% of the tank volume. Water quality degrades as a result of long residence times in storage tanks; chlorine residuals decrease with increased residence times, while disinfection by-products (DBPs) such as THMs increase. The maximum water age in the Brighton tank under current conditions is approximately 57 hrs.

For this corrective measure, two slightly different but related scenarios were investigated. One looked at altering the active storage volume in the tank (as the tank is always 25% full, but this volume is inactive). The other scenario looked at reducing the total volume of water stored in the tank. Pressures in the system were within adequate range for each scenario looked at. A chlorine dosage of 6.28 mg/L was used for each scenario.

The following table summarizes the results from the various scenarios examined. Increasing the active storage volume in the tank, while decreasing the inactive volume saw water age in the tank and in the distribution system decrease slightly. Chlorine residuals remained mostly constant, however. The number of times chlorine was injected (or pulsed) into the system increased with smaller active volumes. As the active storage volume in the tank was decreased (when there was no inactive volume), water age in the tank and throughout the distribution system decreased significantly. Minimum chlorine residuals at the end of the system increased to 0.16 mg/L, meaning there is potential to reduce the overall chlorine dosage.

Table 56: Effect of varying water levels in Brighton tank

Active: Inactive: Dean Tank Volume Used (%)	Min Water Level (m)	Max Water Level (m)	Max Water Age in Tank (hrs)	Max Water Age at End of System (hrs)	Min Chlorine at End of System (mg/L)	Pump/ Chlorine Pulse Cycle (times/day)
10:65:25	4.77	5.5	66	109	0.08	3.5
25:50:25	3.66	5.5	62	105	0.08	1.5
50:25:25	1.83	5.5	57	102	0.08	1.25
75:0:25	0	5.5	56	101	0.07	1
50:0:50	0	3.66	37	83	0.10	1.2
25:0:75	0	1.83	25	68	0.16	1.8
10:0:90	0	0.73	18	58	0.16	3.9

To summarize, the best options for reducing THM formation potential is to reduce the inactive storage volume in the tank (or increase the active volume), or to increase the dead volume by decreasing the maximum water level. Both options provide some potential to lower the chlorine dose. When there is an inactive water volume present in the tank, as water level variation increases, the age of water in the tank decreases and the spread of older water throughout the system also decreases slightly. While increasing the active volume (while still having some inactive volume) had little effect on chlorine residuals throughout the system, there is potential for THM reduction with any reduction in water age. The best option, however, is to have a tank with no inactive volume and with as little storage volume as possible.

6.11.6 Increase Mixing in Tank

When using EPANET to model hydraulic and water quality behaviour the assumption was made that tanks behave as continuously stirred tank reactors (CSTR) where there is complete mixing. Complete mixing is an idealized assumption of the Brighton tank behaviour and in reality it probably functions more on the principle of first in/last out plug flow or two-compartment mixing. These additional tank-mixing scenarios were examined to determine if there were any major differences observed.

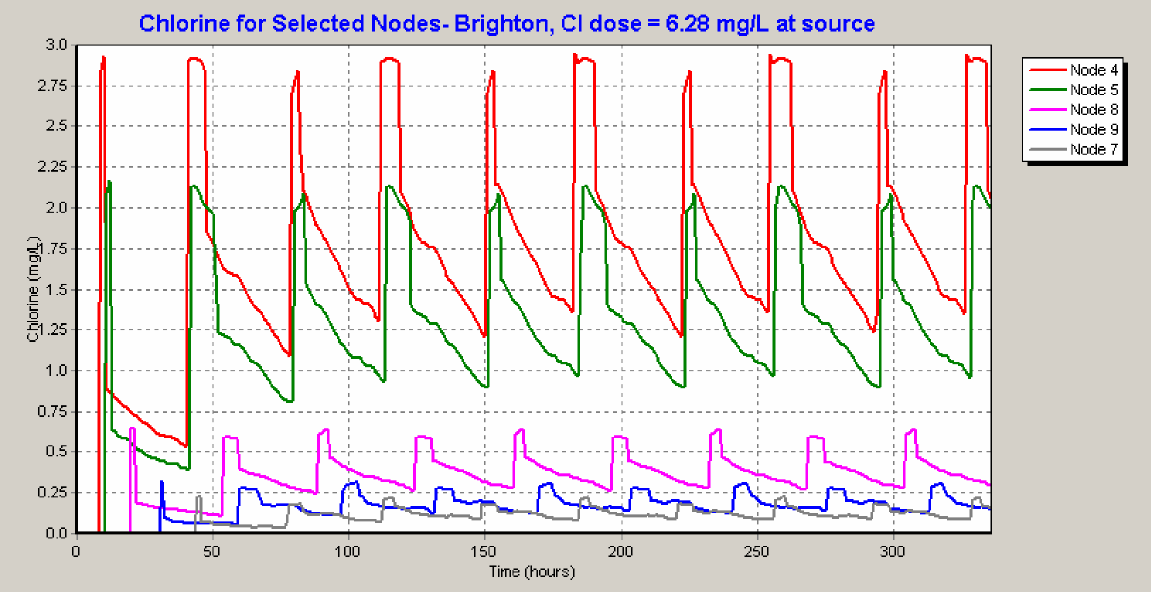


Figure 72: Brighton tank as complete mixing tank model

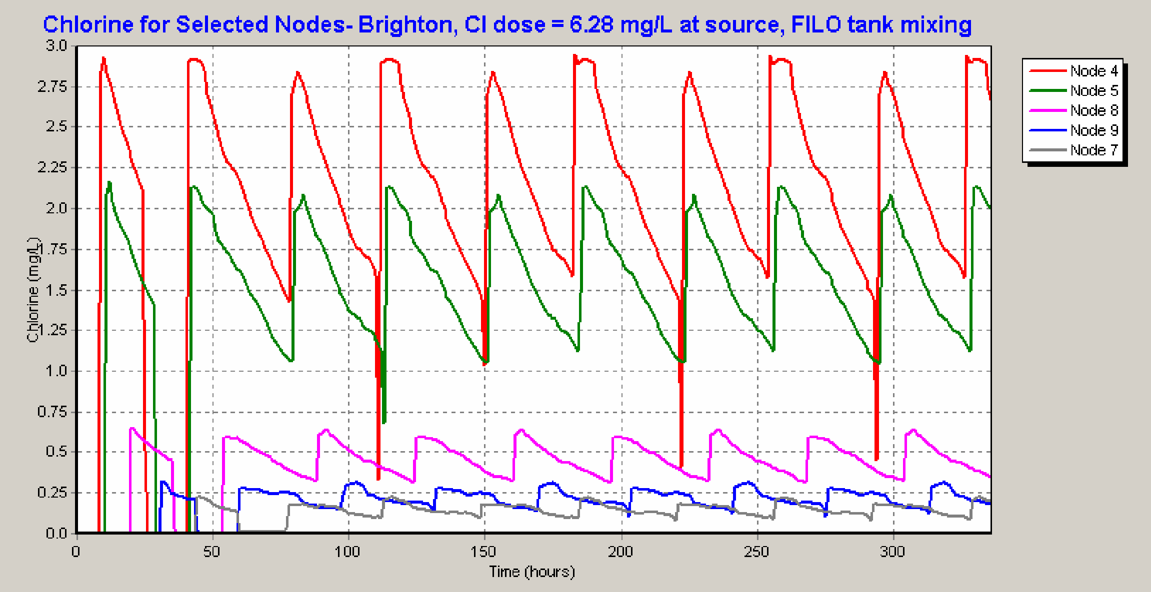


Figure 73: Brighton tank as first in-last out tank mixing model

For the 2-compartment mixing tank model, a parameter that is the fraction of the total tank volume devoted to the first compartment must be input. The first compartment simulates short-circuiting between inflow and outflow while the second compartment represents a dead zone in the tank. It is a reasonable assumption that the well-mixed zone in the tank would extend to the height of the jet input of water. The dynamics of free and confined jets are well known for calculation purposes, but for this scenario a mixing fraction of 0.2 was assumed, corresponding to a free jet height of approximately 1 m high.

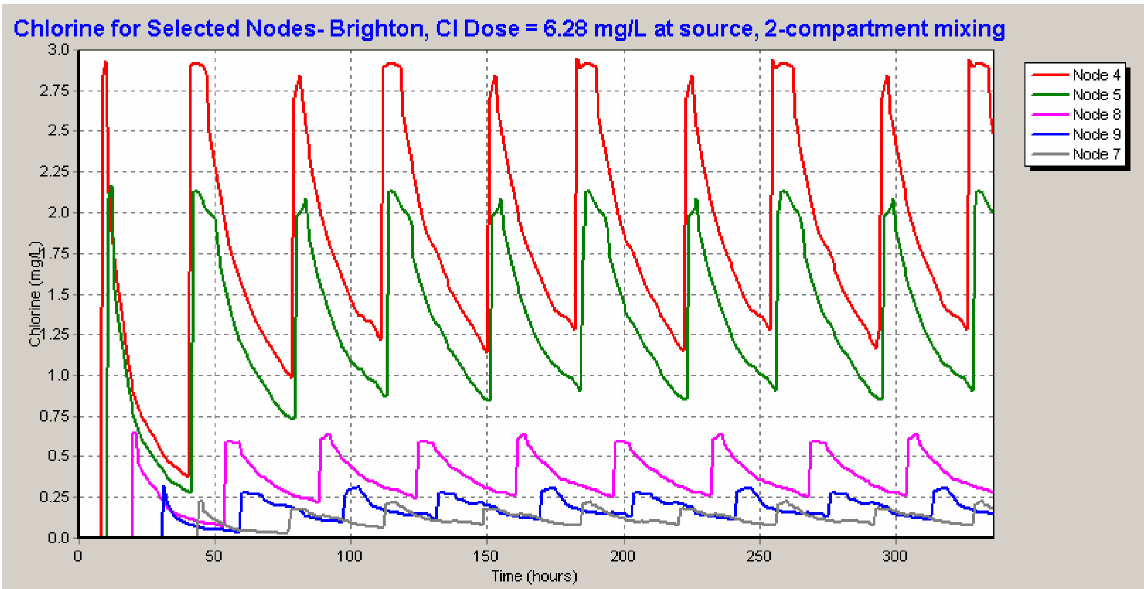


Figure 74: Brighton tank as 2-compartment mixing tank model

Increased mixing of water layers in the tank can also be achieved by reducing the diameter of the inlet into the tank, creating a higher jet effect. To model this scenario a mixing fraction of 0.75 was assumed to coincide with a more powerful input jet. The resulting chlorine levels throughout the system were most similar to the complete mixing tank model, and there were no observed changes in maximum and minimum chlorine levels.

The behaviour of chlorine throughout the system did not change dramatically based on the different tank mixing models selected. The maximum and minimum values were virtually the same, however the decay of chlorine over the chlorine pulse cycle was much smoother and more gradual with the tank modeled as a last in/first out or 2-compartment mixing tank.

Increased mixing in the tank can also be achieved by forcing greater turnover in the tank. This type of simulation was performed in the previous section where the active storage volume in the tank was increased at the expense of the inactive volume.

6.11.7 Regular System Flushing at Dead Ends/ Continuous Bleed at System End

The maximum retention time in the Brighton water distribution system is 102 hours. Any flushing program therefore must occur at a time period less than this, ideally at half the current return period every 51 hours or approximately every 2 days. For this corrective measure, two separate scenarios were looked at: having a regular (either manual or automated) intermittent flushing program, and having a continuous bleed at the end of the system. The average daily flow rate (demand) in the network is 1.07 L/s, and flushing rates will be some multiple of this. Pressures throughout the distribution system are adequate up to 10 times the average daily flow rate, above this, the pump lacks capacity and minimum pressure criteria are violated. Even at the maximum flushing rates the distribution system is capable of, it is impossible to reach a flushing velocity of 0.75 m/s.

For the regular system-flushing scenario, a flushing period of once every 24 hours was selected to coincide with the low demand period that occurs overnight. The system is flushed for 8 hours during the overnight low demand period, and then demand returns to the normal pattern for the remaining 16 hours. The flushing demand was assigned to the end node (node 7). An initial flushing demand rate of 1 L/s (effectively twice average daily flow) was chosen, but there was little observed change in chlorine readings at the end of the system, so the flushing rate was increased to 2 L/s (three times average daily flow). At this rate the minimum chlorine reading at the end node becomes 0.12 mg/L and the maximum water age is 66 hours, indicating potential for a reduction in the overall chlorine dosage. Under this flushing regime, the chlorine dosage could safely be reduced to 3.5 mg/L and adequate chlorine residuals still be maintained throughout the network.

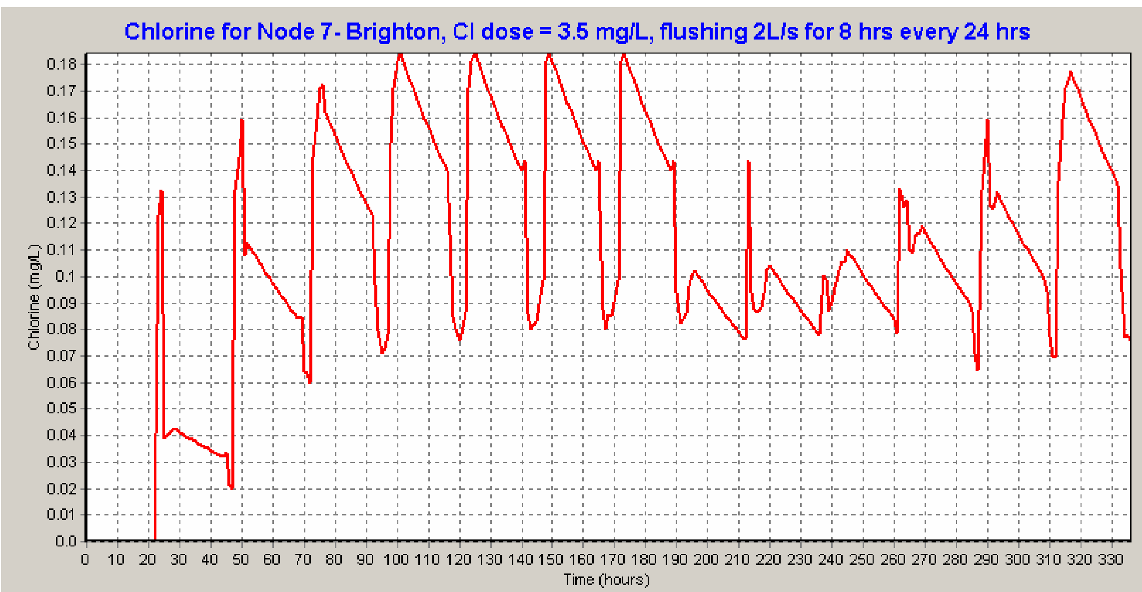


Figure 75: Chlorine levels at the end of the Brighton network with continuous flushing

For the continuous bleed scenario, an additional constant demand of 1 L/s was placed on the end node (node 7). With more demand at the end of the system, water moves faster through the distribution network and the tank filling/emptying cycle changes to 9 hours for filling, 14 hours for emptying. This compares with 6 hours to fill and 30 hours to empty previously, meaning increased wear and tear on the pump. Minimum chlorine readings at the end of the system increase to 0.16 mg/L, while the maximum water age in the network is reduced to 53 hours. This indicates the potential for the overall chlorine dosage to be reduced.

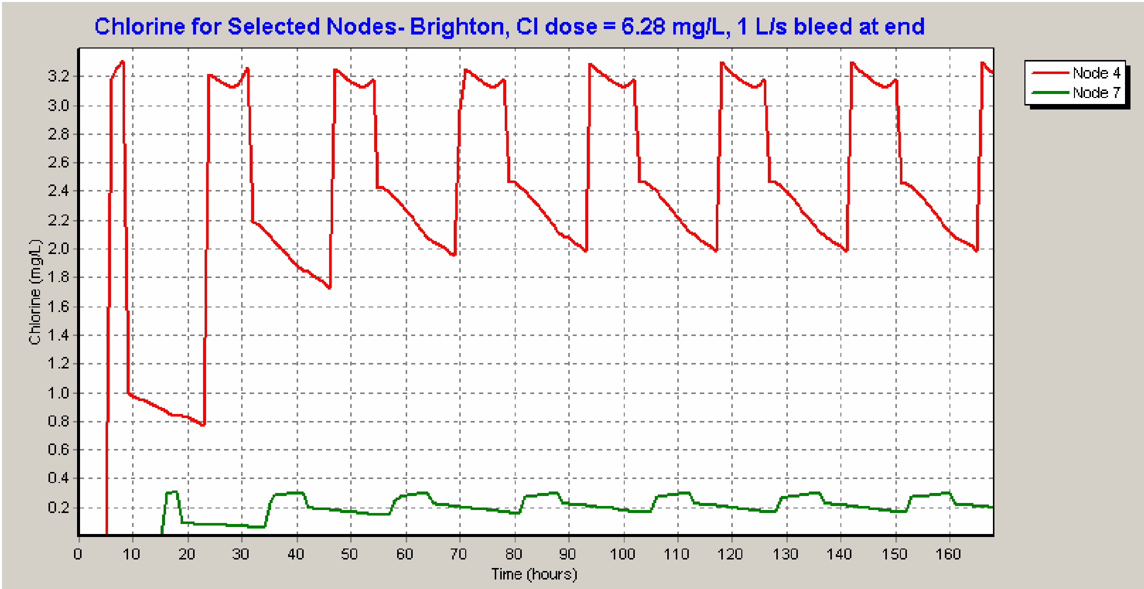


Figure 76: Chlorine levels in the Brighton network with a continuous bleed

With a continuous bleed of 1L/s at the end of the system, the chlorine dosage can be reduced to 3 mg/L and adequate chlorine residuals still be maintained throughout the network.

6.11.8 Downsizing Mains

The Brighton distribution network is oversized for the demand on the system. Pipe sizes range from 250-150 mm, however the resulting maximum water velocity in the system is only 0.12 m/s observed in the pipe used to fill the tank (pipe 1). The use of 150 mm pipes is common even on small, low demand systems in order to fit fire hydrants.

For this scenario, each pipe was resized so as to achieve a peak velocity of approximately 0.4 m/s. The minimum pipe size assigned, however, could not be lower than 40 mm. Under these criteria, pipe sizes in the Brighton distribution system now range from 150-40 mm. The resulting maximum water age in the system now becomes 62 hours. Pressures throughout the system are still within normal range even with the reduced pipe size. Chlorine readings at the end of the system (node 7), however, fall below 0.05 mg/L.

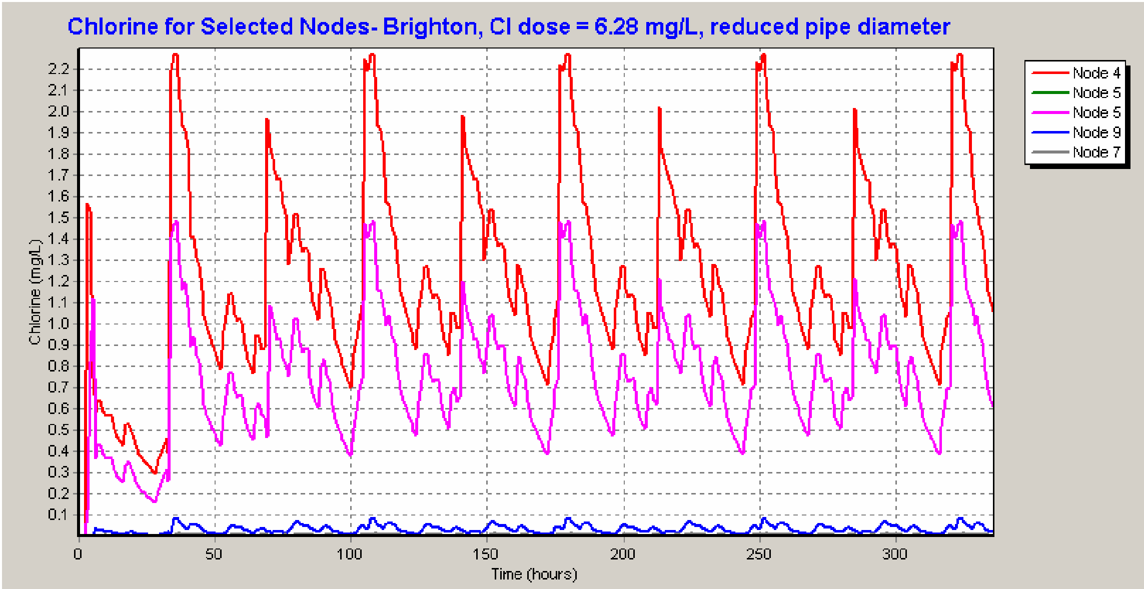


Figure 77: Chlorine levels in the Brighton network with reduced pipe diameters

The rate of reaction of chlorine at the pipe wall is inversely related to pipe diameter, so the smaller the pipe diameter the greater the pipe wall reaction rate, and the greater the amount of chlorine consumed at the pipe wall. As the rate of wall demand in smaller diameter pipes is higher than in large diameter pipes, even though there is a noticeable decrease in water age at the end of the system, chlorine residuals at the end node do not meet required criteria.

With smaller diameter pipes on the Brighton system, the overall chlorine dosage would have to be increased in order to achieve adequate residuals. THM formation is dependent on time and available chlorine, so any benefits in terms of THM reduction that might have resulted from decreased residence time in the distribution system with smaller pipe size are likely to be offset by the increased chlorine dosage, unless a chlorination booster is used.

6.11.9 Install Chlorine Booster

A chlorine booster is a secondary chlorination system located on a water distribution system to boost chlorine residuals to appropriate levels in areas where they may have fallen below acceptable levels. In EPANET a chlorine booster can be modeled in 3 ways from the source quality editor. For the purposes of this study, the chlorine booster is modeled as a setpoint booster, which sets the chlorine concentration that water leaving the node will be boosted to.

From the first scenario looking at reducing the chlorine dosage, a minimum dosage of 2 mg/L is required to produces adequate chlorine readings at the first point of use, and will be used as the chlorine dose in this scenario. With a source chlorine residual of 2 mg/L, the minimum chlorine reading at node 8 is 0.08 mg/L, still above the minimum criteria of 0.05 mg/L. Chlorine values become too low beyond this point, therefore, node 8 (halfway along the distribution system) is the best site for our chlorine booster station.

A booster chlorination residual of 0.5 mg/L leaving node 8 is sufficient to provide desirable chlorine readings at the end of the system. The combined chlorine dosage required to maintain adequate chlorine levels in the Brighton distribution system using a source and booster chlorination system is significantly less that that required using just a source chlorination system. A source dose of 2 mg/L and booster dose of 0.5 mg/L at node 8 provides adequate chlorine levels and uses less than half the amount of chlorine currently used, indicating potential for THM reduction.

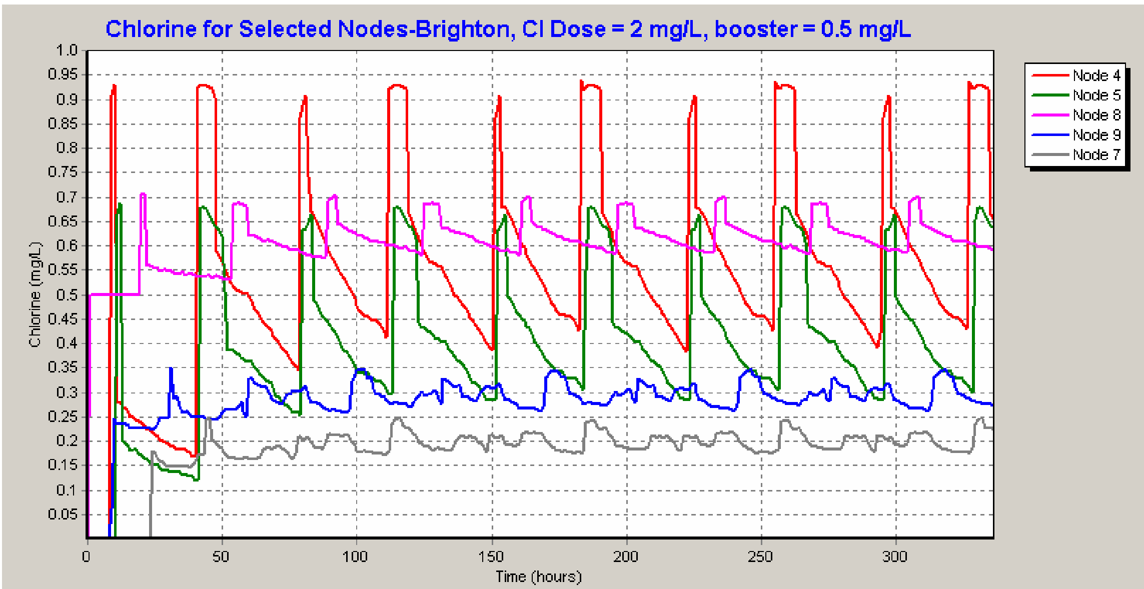


Figure 78: Chlorine levels in Brighton network with a chlorine booster

6.12 Impact of Modeled Corrective Measures

Of the 11 corrective measures identified in a previous section that could be modeled in EPANET in order to access their impact in terms of improving water quality (looking at chlorine, water age, and potential THM formation), two were grouped together with other related scenarios. Not all scenarios met the required criteria in order to be deemed successful. Any scenario that saw a reduction in the overall chlorine dosage and a decrease in water age in the distribution system has potential for lowering THM levels. The following table highlights which scenarios had a positive impact on water quality.

Table 57: Modeled scenarios for the Brighton network and their effectiveness

	Scenario Description	All Criteria Met	Comments
1	Optimize Chlorine Dosage	Yes	-Potential to reduce overall Cl dose slightly
2	Relocate Chlorination System Location Closer to 1st User	Yes	-Overall Cl dose reduced by more than half -Time available for THM formation almost halved
3	Chlorine Dosage Control	Yes	-Greater Cl variability

			-No potential to reduce overall Cl dose
4	Tank Location/ Multiple Smaller Tanks	No	-Primary and secondary chlorination criteria not met
5	Reducing Tank Storage Capacity/ Adjusting Pump Schedule	Yes	-Reducing inactive storage volume in the tank slightly lowers water age in system -Increasing dead tank volume significantly lowers water age and potential for lower Cl dose
6	Increased Mixing in Tank	Yes	-Cl levels constant with different tank mixing models -No potential to reduce overall Cl dose
7	Regular System Flushing at Dead Ends/ Continuously Bleed System	Yes	-Overall Cl dose can be reduced -Water age reduced -Increased wear on pumps
8	Downsizing Mains	No	-Water age will decrease, but higher Cl dose required
9	Install Chlorine Booster	Yes	-Overall Cl dose halved

Any corrective measures that did not meet the necessary criteria should be dropped from consideration and evaluated no further. Scenarios that saw overall chlorine use reduced and water age in the distribution system lowered will be the most effective in terms of lowering THMs. Based on this assessment, the corrective measures (that met criteria) with the most potential for reducing THM formation potential are:

- Optimize chlorine dosage
- Re-locate chlorination system closer to first user
- Reducing tank storage capacity/ adjusting pump schedule
- Regular system flushing at dead ends/ continuously bleed system
- Installing a chlorination booster

6.13 Assessment of Corrective Measure Constraints for Brighton Network

The following table evaluates each remaining corrective measure for the Brighton water distribution system against identified solutions constraints. The selection of the preferred solution(s) to water quality problems can be made based on the corrective measure(s) with the highest score(s).

Based on the resulting scores, there are 3 main tiers of possible solutions. The top three tiers in the decision matrix scoring system comprise the corrective measures that have the most potential for effectively optimizing chlorine dosage, reducing water age, and lowering THMs.

The first tier, which scored at 14, consisted of installation of a Potable Water Dispensing Unit. The second tier of possible solutions, which scored 13, included the general best management practice of improved system design and three “hard” solutions including reducing storage capacity, regular system flushing at dead ends and relocation of the chlorination system. The third tier of corrective measures included continuously bleeding the system and three “soft” solutions including watershed protection, training and adaptive policy to promote PWDUs.

The selection of a preferred solution by the decision making body (town, engineering consultant, Department of Municipal Affairs) can be guided by this decision making framework. The next step in the process involves the implementation of this solution, monitoring and review.

Table 58: Assessment of solution constraints for Brighton

Applicable Corrective Measures	Effectiveness	Cost	Time Scale for Implementation	Permanency of Solution	Adverse Hydraulic Impacts	Adverse WQ Impacts	Acceptable to Stakeholders	Meets Regulations	Total
Policy of POU/POE treatment	1	2	0	0	1	2	1	1	8
Advanced treatment	2	0	0	2	1	2	0	2	9
Alternative water sources	1	0	0	2	1	2	1	2	9
Combination of corrective measures	2	0	0	2	1	1	1	2	9
High quality water storage and recovery	1	0	0	2	1	2	1	2	9
Policy to promote use of alternative disinfectants	1	2	0	1	1	1	1	2	9
Alternative disinfectants	2	1	1	1	1	1	1	2	10
Chlorine dose control	0	2	1	1	1	1	2	2	10
Increase mixing in tank	0	2	1	1	1	1	2	2	10
Regionalization	1	1	1	2	1	1	1	2	10
Water treatment plants	2	0	0	2	1	2	1	2	10
Wind breaks around exposed sources	0	2	0	2	1	1	2	2	10
Filtration	1	1	1	1	1	2	2	2	11
Install chlorination booster	1	2	1	1	1	1	2	2	11
Optimize disinfectant dose	1	2	2	1	1	1	2	1	11
Point of use/entry treatment	2	2	2	0	1	2	1	1	11
Remove submerged vegetation	1	0	1	2	1	2	2	2	11
System maintenance	1	2	2	0	1	2	1	2	11
Continuously bleed system at dead end	2	2	1	1	2	2	1	1	12
Policy to promote PWDU	1	2	0	2	1	2	2	2	12
Training	1	2	0	1	2	2	2	2	12
Watershed protection	0	2	1	2	1	2	2	2	12
Improved design of system	1	2	0	2	2	2	2	2	13
Reduce storage capacity/ Adjust pump schedule	2	2	1	1	1	2	2	2	13
Regular system flushing at dead ends	2	2	1	1	2	2	2	1	13
Relocate chlorination system	2	2	1	1	1	2	2	2	13
Potable water treatment unit	2	2	1	2	1	2	2	2	14

7.0 Burlington Water Distribution System Model

The Burlington water distribution system is typical of many small towns in Newfoundland and Labrador– a long linear system with a surface water supply whose raw water displays high colour and organics. The Burlington system is gravity fed from a source called Waddy's Pond. The system has a screening chamber 130 m down-pipe from the source intake and also receives chlorine disinfection. Upgrades to the Burlington distribution system have been ongoing since 1995 through a phased program (up to Phase 7B in 2002). Only the west end of Burlington, to the Winterhouse Cove area, still has older and smaller diameter pipes. The chlorination system was upgraded in 1995.

Around 2005, the town of Burlington became increasingly concerned with the issue of THM levels being over the GCDWQ. According to town officials, there are problems in maintaining chlorine residuals towards the end of the distribution system. Also during the winter months, the 30 or so homes connected to the old distribution mains must run their water to keep their pipes from freezing. The Burlington distribution system can be classified as very small and from the Western region of the province.

Figure 79: Burlington water distribution system network

Descriptive data for the Burlington water distribution system is detailed in following sections. This data was then input into the Burlington EPANET hydraulic/water quality model. The next step involved calibrating the Burlington model with system data also highlighted in the following sections of the report. Different corrective measures and modeling scenarios were then selected based on observed problems with how the distribution system is currently operating. The potential effectiveness of the given solution or modeled scenario was then weighted against solution criteria and constraints.

7.1 Reservoir

Prior to 2004, Burlington East used Eastern Island Pond as its source water supply, and Burlington West used Goudie's Brook. In late 2003 or early 2004, the Goudie's Brook supply was taken off line and the entire community was put on the Eastern Island Pond supply. The water supply for the town of Burlington, also know by the local name of Waddy's Pond, is located approximately one kilometre northeast of town. Waddy's Pond has reservoir storage of 500,000 m^3 with 1 m of drawdown. The intake is a 300 mm polyethylene pipe. A primary screening chamber exists near the intake to help deal with solids, turbidity and colour problems. The reservoir has a water level of 60.1 m. The catchment area for Waddy's Pond also takes in the PPWSA for Smith's Harbour (Fleshetts Brook) and is approximately 13.1 km^2 in size. Average DOC levels for Burlington are in the 3rd quartile or the highest 25% of average DOC values in source waters in the province.

Table 59: Average source water quality values for Burlington

Water Quality Parameters	Average Values 1993-2006
Colour (TCU)	78.3
pH	6.13
Turbidity (NTU)	0.60
Bromide (mg/L)	0.02
Chloride (mg/L)	2.88
DOC (mg/L)	8.19
Temp (°C)	10.5
Iron (mg/L)	0.19
Manganese (mg/L)	0.008

7.2 Pipes

The majority of pipes in the current Burlington distribution system were installed since 1998. Pipes in the western part of town, including Winterhouse Cove, are over 25 years old. The trunk main carrying water from the intake to the community is composed of high-density polyethylene (HDPE). The intake pipe is 300 mm, while the pipe from the screening chamber to the community is 250 mm. Pipes in the community are all PVC, ranging in diameter from 200 to 150 mm. Pipes left over from the old distribution system are 75 mm in diameter and are buried to a shallower depth than the newer pipes resulting in problems of pipes freezing in winter. In total there is 4.5 km of trunk main laid down in the Burlington distribution system.

The Hazen-Williams head loss formula was selected for this model in order to determine energy losses throughout the system. Roughness factors were selected based on pipe age: 155 for newer HDPE pipe, 150 for newer PVC pipes, and 140 for older PVC pipes.

From information gathered on the system, line pressure has been estimated to range from 145-345 kPa (14.8-35.2 m). However, a quick rule of thumb is that every one-meter in elevation equates to 10 kPa in system pressure. By this estimation, maximum estimated pressure in the Burlington system will be 581 kPa (59.2 m). A pressure range of 345-581 kPa will be used for comparison. In addition, there are at least 11 fire hydrants located at different points on the distribution system.

7.3 Demand

The Burlington distribution system does have an old pulse meter located on the system, however the meter is not operational and there is no record of any meter readings. As no meter readings are available, average flow into the Burlington distribution system was estimated based on a population of 409 and a typical design demand of 450 L/person/d. Average demand is estimated to be 184 m^3/d or 2.13 L/s, and these values will be used for modeling purposes. Types of water users and their number are summarized in the following table.

Table 60: Type and number of water users on the Burlington

Type of Water User	Number
Residential	135
School	1
Institution (Municipal Hall)	1
Commercial (one hotel)	2

Residential demand was allocated to 11 different junctions throughout the distribution network based on housing density surrounding that junction. Non-residential demand is not significant on this system and so was equated to an equivalent number of residential properties.

Information gathered from earlier sources sets the maximum yield of the system at 500 L/s (43,200 m^3/d), with an available yield of 17 L/s (1469 m^3/d), and an average demand on the system of 2.5 L/s (216 m^3/d). Based on this average demand, per capita consumption is 528 L/p/d.

Elevation of junctions with assigned demands ranged from 28 m (end of system in Winterhouse Cove) to 2.0 m above sea level along Highway 413.

Meter readings have not been taken at a frequency to establish a daily demand pattern for the Burlington distribution system. Peaks in the morning, noon and evening for domestic users are typical however. The following two generic demand patterns were used in the Burlington model- one for domestic water use, and one for winter flushing to prevent pipes from freezing. The winter flushing pattern simply adds an additional 50 % to average flows on top of the typical domestic water use pattern.

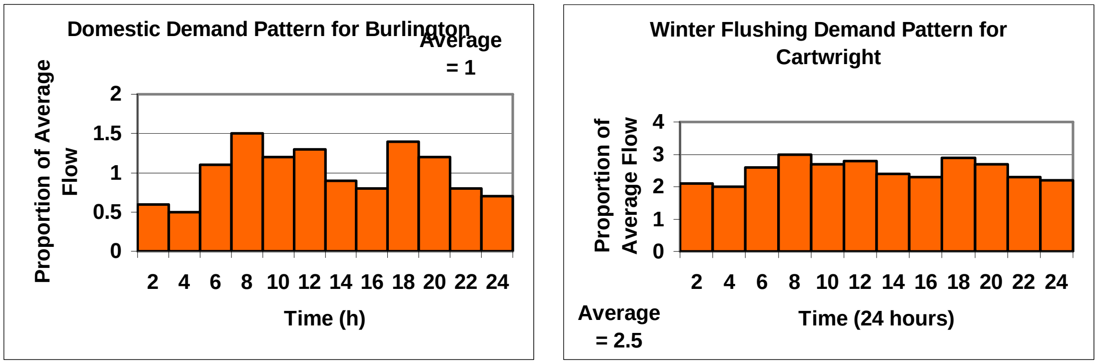


Figure 80: Typical domestic and winter flushing demand patterns

7.4 Chlorine Decay

The Burlington water distribution network has a liquid chlorination system (node 3) that was upgraded in 1995. The chlorination system has a 200 L tank that previously used to be filled every 4 days with 60 L of 12% chlorine solution for every 140 L of water resulting in a 4.5% dilution. Since the fall of 2007, the system operator has been using 200 L of 12% chlorine solution, with the tank needing to be refilled every 8 days. A current chlorine dose of 16.3 mg/L was calculated, however, the previous dose of 12.2 mg/L will be used for modeling purposes. The amount of chlorine injected into the distribution system is manually regulated.

According to gathered information, there are difficulties in maintaining adequate chlorine residuals at the far end of the system. This problem is alleviated somewhat during winter when homes at the end of the system flush their lines to keep their pipes from freezing. Based on an average daily flow of 2.13 L/s, the available contact time at the first point of use is 81 minutes, while the contact time at peak flow is 20 minutes (a minimum of 20 minutes is required). If increased winter flows are used (based on an average daily flow of 3.2 L/s), the CT at the first point of use is 54 minutes, while the contact time at peak flow is only 13 minutes. Based on these calculations, special attention will have to be made to contact times, especially during periods of high demand in the winter months.

Results from a bulk chlorine decay test performed in 1998 on Waddy's Pond water by Water Analysis Laboratories of Mount Pearl, NL were used to determine a value for the bulk chlorine decay coefficient. Four different dilutions were tested over a 24-hour period with bulk decay rates ranging from -0.56 to -4.0 d^{-1}. A bulk chlorine decay coefficient of -1.53 d^{-1} was eventually selected for the Burlington model, selected from the third scenario where chlorine ranged from 19.1-3.76 mg/L. A default wall decay coefficient of -1 m/day was also selected.

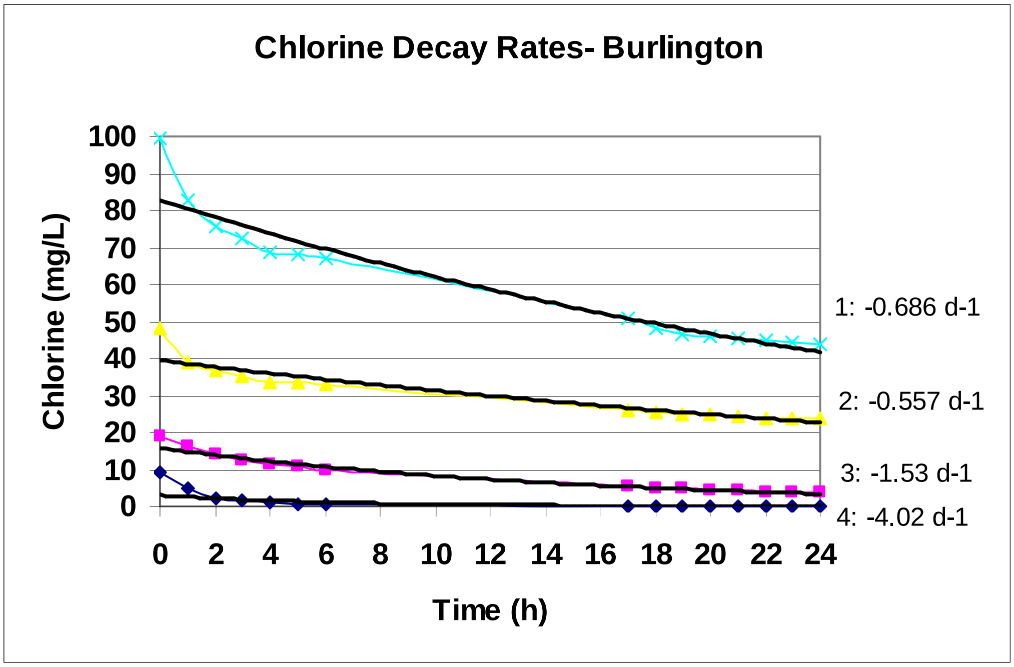


Figure 81: Chlorine decay rates for Burlington

7.5 Chlorine and THM Data Gathering

Chlorine tests are regularly made by the Burlington System Operator and by Department of Environment and Conservation staff. The following table summarizes average chlorine, total THM, and BDCM results. The values of free chlorine in brackets are the highest values observed at this site and were used in calibration of the system. There was very little data available from the end of the system.

Table 61: Average chlorine, THM and BDCM (2002-2007) reading for Burlington

Location in Network	Junction	Free Chlorine-DOEC (mg/L)	Total Chlorine - DOEC (mg/L)	THM Total-DOEC (ug/L)	BDCM (ug/L)
Beginning	8	0.24 (1.39)	0.40	183	3.04
Middle	11	0.13 (0.52)	0.32	214.3	2.6
End	14	0.06	0.22	-	-

The CCME maximum acceptable concentration (MAC) for total THMs is 100 ug/L. As shown above, THM levels in Burlington are well over the limit.

7.6 Calibration of the Burlington Model

In order to first calibrate the Burlington hydraulic/water quality model, results were compared with flow, pressure and chlorine residual data gathered on the Burlington distribution system. The collection of this data is outlined in previous sections.

Comparison of initial model results to calibration data is described in the following table, along with actions taken to compensate for any discrepancies, and final associated percentage errors found in the calibrated model. Average values from the model are taken for comparison once equilibrium or periodic behaviour from that parameter had been reached.

Table 62: Calibration of the Burlington model

Issue	Percentage Error	Action	Percentage Error After Calibration
-average daily model flow of 2.13 L/s (daily range of 1.06-3.19 L/s) vs. average flow of 2.13 L/s	-0%	None- no metered flow readings for comparison	
- node 2 model pressure average of 58.1 m vs. estimated line pressure of 59.2 m -node 15 model pressure average of 31.8 m vs. estimated line pressure of 35.2 m	-1.9% -9.7%	None- model pressures display less than 10% error from estimated line pressures	
-node 8 (beginning of system) equilibrium Cl of 3.15 mg/L (range of 2.45-3.85 mg/L) vs. 1.39 mg/L	-127%	-decreased source chlorine dose from 12.2 mg/L to 6.1 mg/L -increased bulk reaction rate from -1.53 d^{-1} to -2.0 d^{-1} -increased wall reaction rate from -1 m/d to -1.5 m/d	-6.5% (1.30 mg/L)
-node 11 (middle of system) equilibrium Cl of 1.47 mg/L (range of 1.13-1.80 mg/L) vs. 0.52 mg/L	-183%	-decreased source chlorine dose from 12.2 mg/L to 6.1 mg/L -increased bulk reaction rate from -1.53 d^{-1} to -2.0 d^{-1} -increased wall reaction rate from -1 m/d to -1.5 m/d	-5.8% (0.55 mg/L)
-node 14 (end of system) equilibrium Cl of 0.13 mg/L (range of 0.07-0.18 mg/L) vs. 0.06 mg/L	-117%	-decreased source chlorine dose from 12.2 mg/L to 6.1 mg/L -increased bulk reaction rate from -1.53 d^{-1} to -2.0 d^{-1} -increased wall reaction rate from -1 m/d to -1.5 m/d	-33% (0.04 mg/L)

As a site visit was not undertaken, model calibration had to be performed with the best available data, some of which had to be estimated (flow) or was based on estimated data (chlorine dosage). Once results predicted by the model were felt to adequately reflect observed data– matching pressures, flows, chlorine residuals– through the adjustment of certain network parameters, a baseline model was established. The resulting calibration is rough, but is felt to fairly accurately reflect historic operation. The different model scenarios will then be run on this baseline model, adjusting only selected network parameters.

The following graph shows mean observed verses mean simulated values of pressure for nodes 15 and 2 (highest and lowest elevation) on the Burlington system. As can be seen in the graph and calibration table below, actual and modeled pressures match very closely.

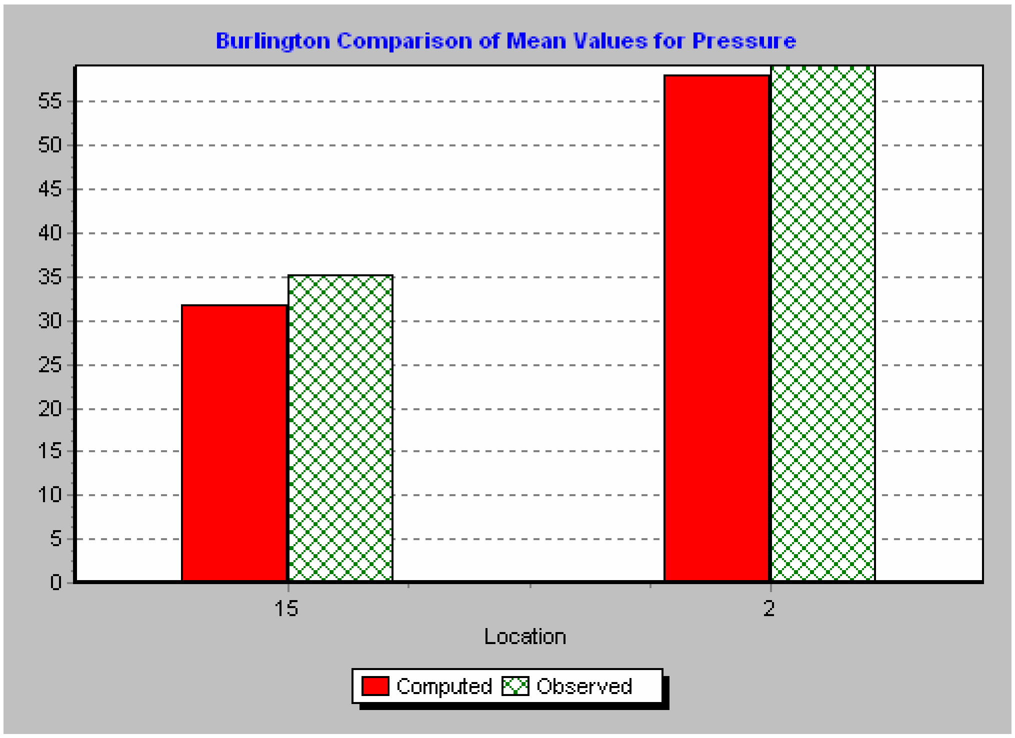


Figure 82: Mean observed and mean simulated values for pressure in Burlington

Table 63: Calibration statistics for pressure

Location	Num Obs	Obs Mean	Comp Mean	Mean Error	RMS Error
15	1	35.20	31.70	3.501	3.501
2	1	59.20	58.09	1.114	1.114
Network	2	47.20	44.89	2.307	2.598

Correlation Between Means: 1.000

The following graph shows system flows over the 7-day simulation period. Given the domestic demand pattern used, the graph indicates expected variation from the average flow of 2.13 L/s.

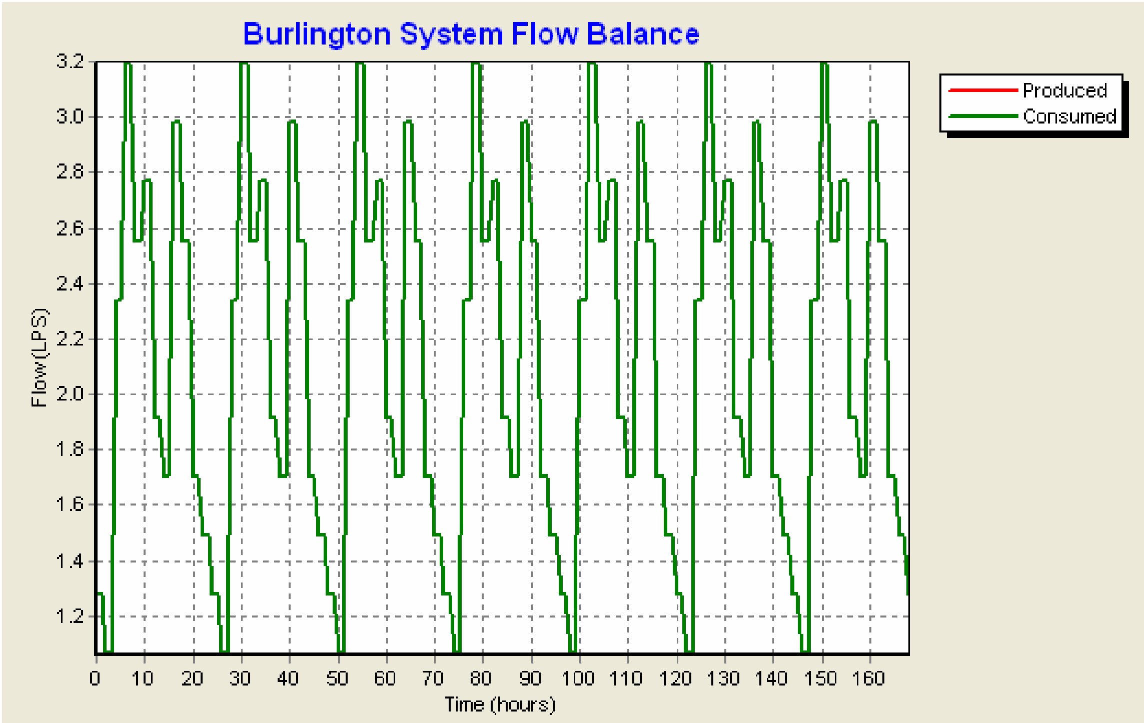


Figure 83: Burlington system water demand

The following graph and table show calibration statistics for free chlorine residuals taken from three different locations on the Burlington distribution system. Observed chlorine readings taken from the field were assigned times after equilibrium had been reached for each node. Once chlorine reached equilibrium, it still varied with changes in system demand. A median point along the chlorine pulse cycle was used to compare simulated to observed results. There was fairly good correlation observed between the field chlorine readings used and modeled chlorine residuals.

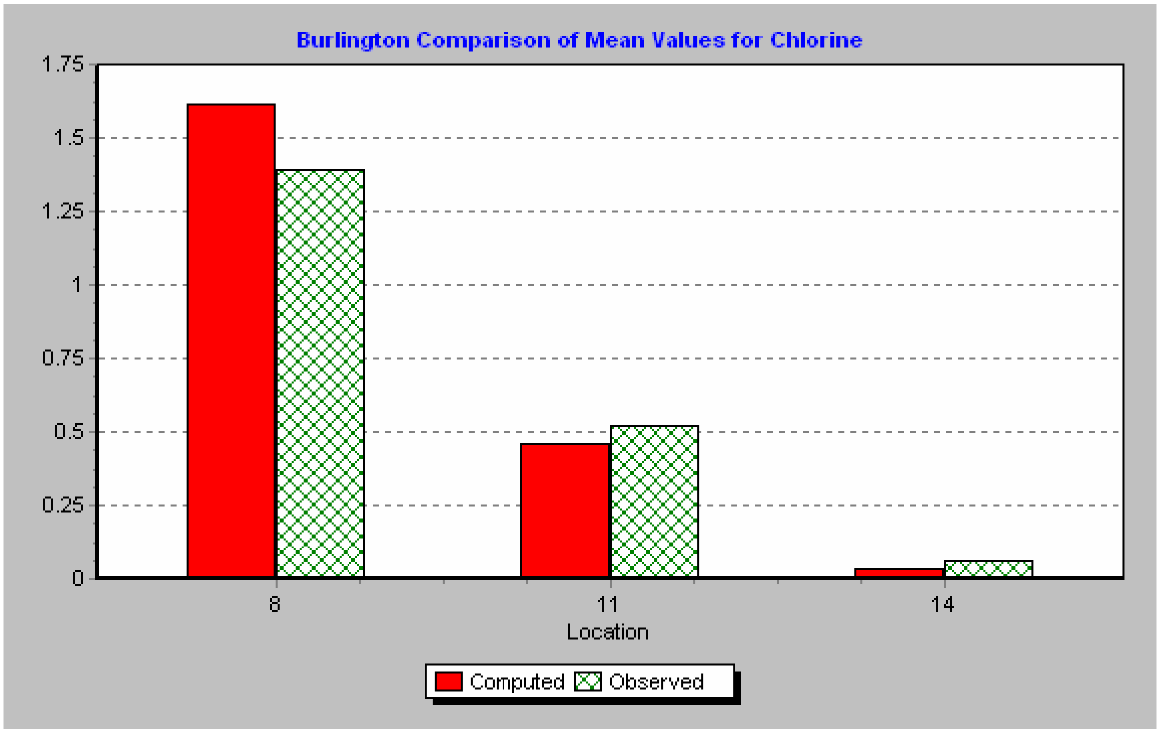


Figure 84: Mean observed and mean simulated value for chlorine residuals in Burlington

Table 64: Calibration statistics for chlorine

Location	Num Obs	Obs Mean	Comp Mean	Mean Error	RMS Error
8	1	1.39	1.61	0.224	0.224
11	1	0.52	0.46	0.061	0.061
14	1	0.06	0.03	0.027	0.027
Network	3	0.66	0.70	0.104	0.135

Correlation Between Means: 0.996

7.7 Problems with the Burlington Distribution System

By gathering detailed background information on the Burlington water distribution system and establishing a calibrated baseline model, we were able to identify problems with how the system operates normally. According to the model results, chlorine residuals, while high at the beginning of the system, are inadequate by the end of the system. Several contributing factors were identified as contributing to the overall Burlington THM problem as outlined in the following table.

Table 65: Problems contributing to high THMs in the Burlington distribution system

	Causative Factors	Quantitative Value
2	Shallow intake	-
5	High DOC in source water	8.19 mg/L
7	High chlorine dose	12.2 mg/L 4.88 mg/L @ 1st user
10	Excessive chlorine demand	-2.0 d-1 (bulk) -1.5 m/d (wall)

12	Long linear system	4.6 km intake to end total = 4.9 km
13	Branched system with multiple dead ends	at least 4 DE
15	Insufficient chlorination controls on system	manual
16	System is oversized	0.02-0.18 m/s 300-75 mm Q_{avg} = 2.13 L/s
18	Pipe material and age	1980
26	Poor O&M of system	Water Dist- Class I
27	Multiple factors	-
28	Poor design of system	-
30	High per capita demand	450 L/p/d winter flushing
32	Problems with chlorine residuals	0 mg/L @ end 4.88 mg/L @ 1st user

The following figures illustrate some of the problems observed in the Burlington distribution system.

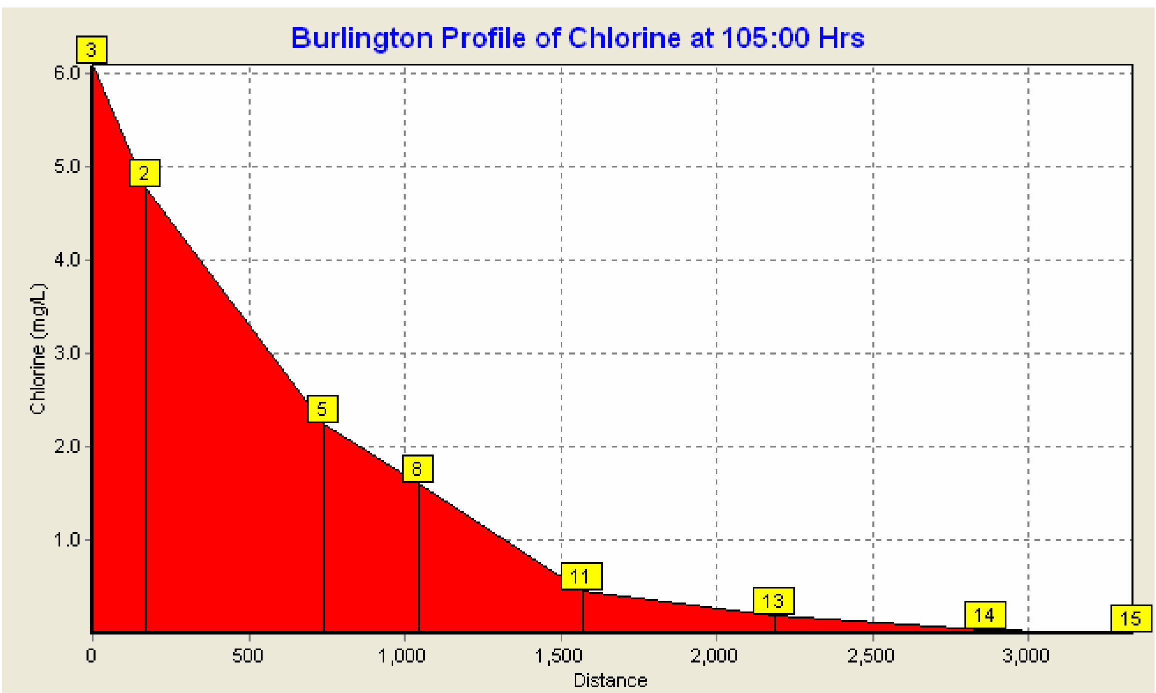


Figure 85: Chlorine decay profile through Burlington distribution system

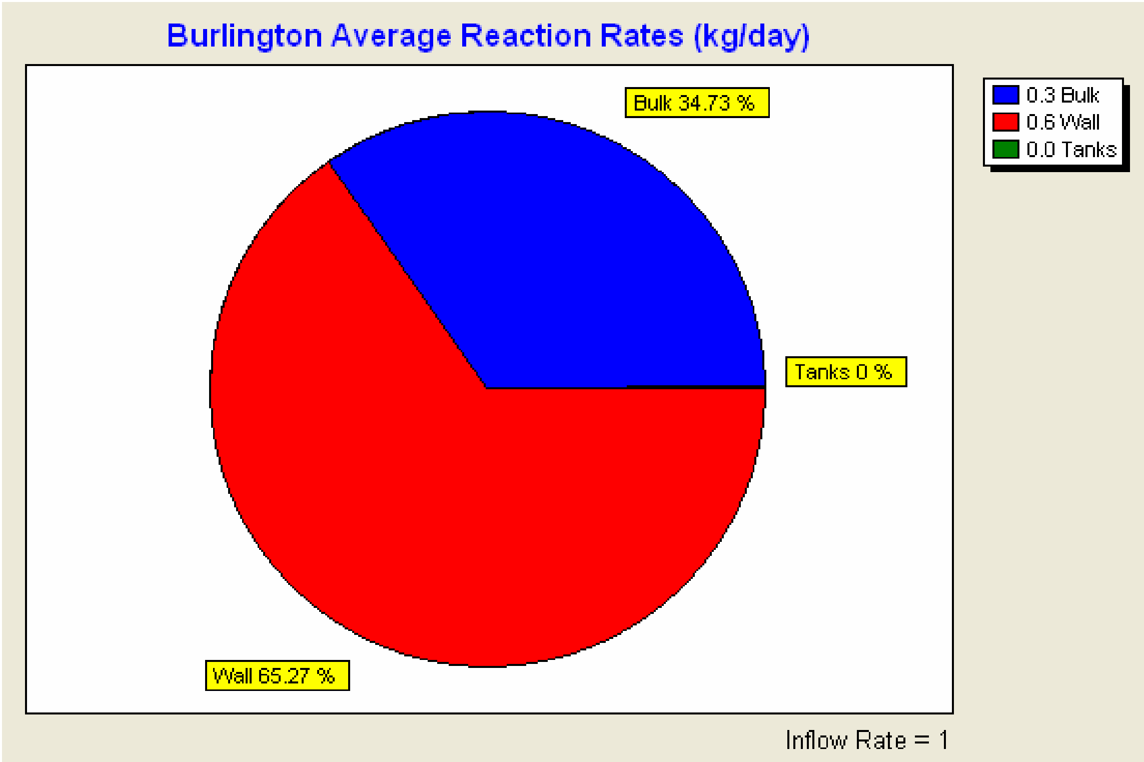


Figure 86: Chlorine decay contributions in the Burlington distribution system

Solutions that might address the probable causes of high THM levels in the Burlington distribution system are outlined in the following table. Those corrective measures highlighted in grey are the only solutions that can potentially be modeled.

Table 66: Applicable THM corrective measures for Burlington

Applicable Corrective Measures	Probable Causes Addressed
Policy of POU/POE treatment	All
Policy to promote use of alternative disinfectants	All
Policy to promote PWDU	All
Watershed protection	All
Alternative water sources	5
Relocate intake in deeper water	2
High quality water storage and recovery	All
System maintenance	All
Regionalization	All
Alternative disinfectants	All
Potable water dispensing unit	All
Point of use/entry treatment	All
Training	All
Optimize disinfectant dosage	2-5-7
Install chlorine booster at optimal location	7-10
Chlorine dose control	2-5-7-15
Regular system flushing at dead ends	2-5-12-13-16
Continuously bleed system at dead end	2-5-12-13-16

Downsizing mains	2-5-12-13-16
Replace or reline pipe	18
Loop distribution network	13
Upgrade distribution network	2-18
Water treatment plants	5-7
Filtration	5-7
Advanced treatment	5-7
Improved design of system	28
Combination of corrective measures	All

7.8 Results from the Burlington Model

The next step was to model the different selected scenarios and see how the Burlington distribution system responded. Given the ability of the baseline model to reflect current conditions accurately, a reasonable degree of confidence can be placed in the scenario results.

7.8.1 Optimize Chlorine Dosage

The Burlington distribution network has a liquid chlorination system located in a chlorination building approximately 210 m inland, just off the main highway. The current chlorine dose is calculated to be 12.2 mg/L, however, without any actual flow data, this value is a best estimate. For modeling purposes, a chlorine dose of 6.1 mg/L was used. The chlorination system varies the dosage manually with the guidance of chlorine residual readings. Primary disinfection requirements for the system are just met, with the contact time at peak flow of 20 minutes.

In EPANET we have chosen to model chlorine as a setpoint booster at node 3, which fixes the concentration of any flow leaving that node. As stated in the objectives, the Burlington system should have a 20 minute contact time, contain a free chlorine residual of at least 0.30 mg/L at the first point of use (or equivalent CT value), and maintain a free chlorine residual of 0.05-0.10 mg/L at the end of the distribution system. The following table summarizes the results of altering chlorine dosage.

Table 67: Altering chlorine dosage in Burlington distribution system

Chlorine Dose (mg/L)	CT Value at 1st User	Min Cl Residual at Start of System – node 2 (mg/L)	Max Cl Residual at Start of System – node 2 (mg/L)	Min Cl Residual at End of System – node 15 (mg/L)
16.3	218	10.9	13.04	0.01
12.2	163	8.16	9.76	0.005
6.1	81.6	4.08	4.88	0
5	67.2	3.36	4.00	0
2	27	1.35	1.60	0
0.50	6.6	0.33	0.40	0

Chlorine dosages generally range from 5-15 mg/L. Simulations were run at various dosages, however, none resulted in an adequate free chlorine residuals at the end of the system. Excessive chlorine levels at the beginning of the system are also an issue at dosages over 5 mg/L.

Maintaining adequate chlorine residuals at the end of the network using a single chlorination system located at the beginning of the system will not meet objectives even at the maximum typical chlorine dosage range. If only trying to maintain a residual of 0.3 mg/L at the first point of use, however (as in the case of adding a chlorine booster somewhere in the system), the chlorine dose can be as low as 0.50 mg/L.

7.8.2 Install Chlorine Booster

A chlorine booster is a secondary chlorination system located on a water distribution system to boost chlorine residuals to appropriate levels in areas where they may have fallen below acceptable levels. For this scenario, a chlorine booster station was located at node 13, south of where Highway 413 enters the community before the bridge at the west end of town (approximately 2.3 km from the main chlorination building).

Figure 87: Optimal location of main and booster chlorinators in Burlington

With an initial chlorine dose of 2 mg/L, chlorine levels in the main part of the system up to node 13 are adequate at 0.06 mg/L or above. A booster chlorine dose of 5.0 mg/L leaving node 13 is sufficient to provide a minimum chlorine residual of 0.05 mg/L at the end of the system.

The combined chlorine dose with the booster chlorination system location optimized at node 13 is 7 mg/L, as compared with a single chlorination system dose of 12.2 mg/L. While overall chlorine use is reduced, a booster dose of 5 mg/L is required, resulting in chlorine residuals above criteria levels.

7.8.3 Chlorine Dosage Control

The chlorination system in Burlington does not have a flow meter. The rate of chlorine solution injected into the water distribution system can be varied manually, based on water demand, season, the rate of consumption of chlorine solution, or other factors. For calibration purposes, chlorine dosage was modeled as a constant dose. Water quantity (flow) and/or quality (chlorine residual) feedback controls can be used to manage the chlorine feed.

The chlorine dose was made to vary with time using two time patterns: one the same as that used for water demand, the other opposite to that used for water demand. Feedback control on chlorination systems typically function by increasing the chlorine dose when flows increase in order to maintain CT values at the first point of use. However, when demand is high, water moves at an increased rate through the distribution system, resulting in reduced water age, less time for chlorine decay and higher chlorine residuals. The variation in chlorine will mimic the peaks and lows of flow throughout the system (for chlorinators that are flow controlled), only the lag time between peaks in flow and peaks in chlorine residuals will increase the further you get towards the end of the distribution system.

The following three graphs look at the variation in chlorine readings as three different points in the network if the chlorine dose is constant, increases with flow, decreases with flow. Variation in chlorine residuals increases significantly with flow control.

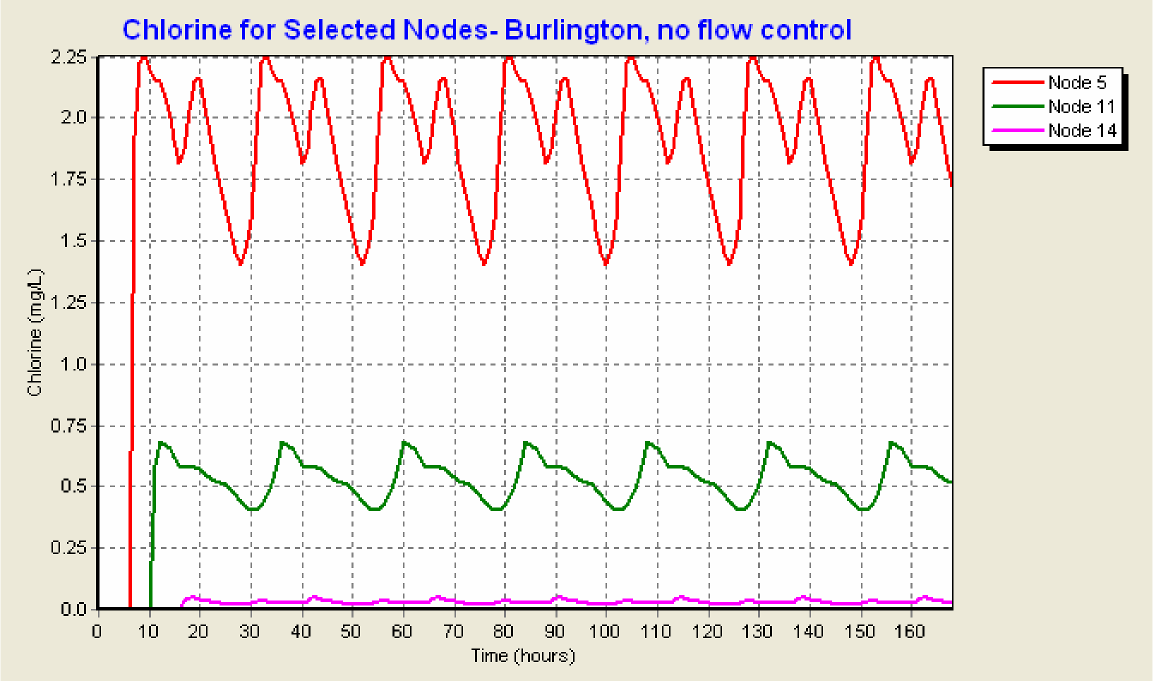


Figure 88: Chlorine levels with constant dose, Burlington

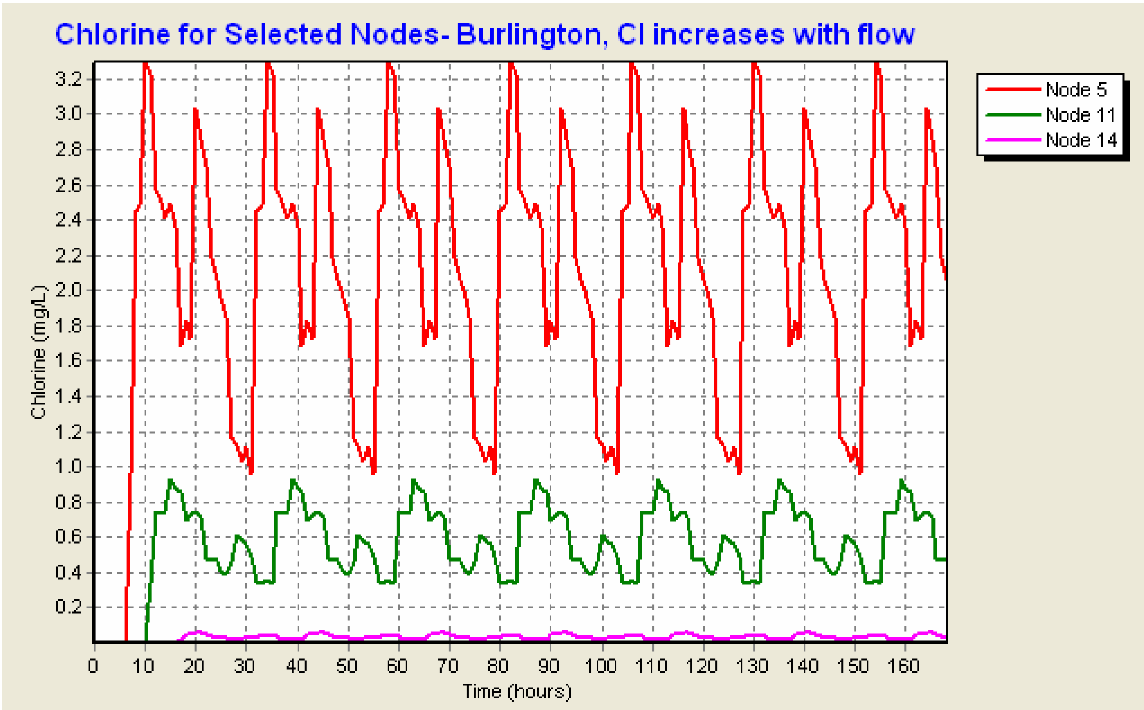


Figure 89: Chlorine levels with chlorine control proportional to flow, Burlington

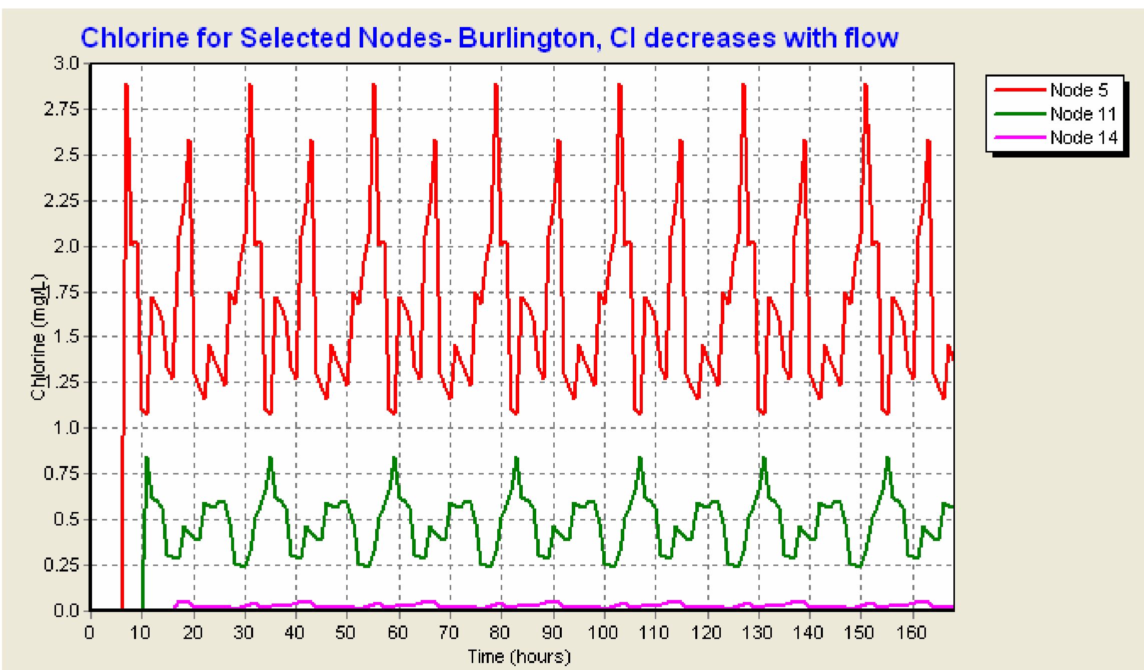


Figure 90: Chlorine levels with chlorine control inversely proportional to flow, Burlington

The lag time between the peak in flow and the corresponding peak in chlorine residual at the end of the system is 22 hours, indicating the difficulty in trying to optimize chlorine through flow control. Chlorine residuals are higher with flow proportional control in

Burlington, but there was no opportunity available to reduce overall chlorine dosage, as secondary disinfection requirements were not met. There are complicating factors involved with chlorine dosage control that make it effective only in parts of the distribution system whether flow or residual controlled.

7.8.4 Regular System Flushing/ Continuous Bleed at System End

The maximum retention time in the Burlington water distribution system at the end node is 30 hours. Cumulative demand decreases towards the end of the network, and this lack of demand combined with older and smaller pipe in the latter third of the network results in barely discernable chlorine residuals at the end of the network. The older pipe has also been buried at a shallower depth than newer pipe, causing residents in this area to leave their taps running in the winter in order to avoid freezing pipes. This practice may actually be improving water quality in the system.

Any flushing program must occur at a time period of less than 30 hours in order to achieve any improvement in water age, ideally at least half the current return period or ever 15 hours. For this corrective measure, two scenarios were examined: flushing twice a day at the end node, a continuous bleed at the end node. The average daily flow rate is 2.13 L/s, and flushing rates will be some multiple of this. At the maximum average demand the system is capable of supplying without pressures becoming negative (5.39 L/s), it is possible to reach a flushing velocity of 0.75 m/L in certain sections of the network.

For the scenario where flushing occurs twice a day at the end node, base demand at node 14 was increased to 1.99 L/s (from 0.142 L/s) for 4 hours at 12 hour intervals during periods of low demand. Maximum water age in the network was reduced from 30 to 20.8 hours. Chlorine residuals increase throughout the network, over 4 mg/L at the first user and to a minimum of 0.01 mg/L at the end of the system. The contact time at peak flow falls from 20 to 11 minutes, violating criteria. Pressure also falls below 28 m at the end node where elevation is highest where there is demand.

For the continuous bleed scenario, an additional constant demand of 1.5 L/s was placed on node 15, increasing average demand by approximately 70%. With more demand at the end of the system, water moves faster through the distribution network and maximum water age is reduced to 13.1 hours. Chlorine residuals increase throughout the network, over 4 mg/L at the first user and over 0.06 mg/L at the end of the network. With a continuous bleed of 1.5 L/s at the end of the system, the chlorine dose can be reduced from 6.1 mg/L to 4.5 mg/L while maintaining chlorine residuals below 4.0 mg/L at the first point of use and above 0.05 mg/L at the end of the system. Contact time and minimum pressure requirements are not met, however.

Manual flushing once a day or more at the end of the Burlington distribution network would not be a practical use of resources. Automatic hydrant flushing units may be a more practical approach. Continuously bleeding the system is wasteful of resources (water, chlorine) and may harm the receiving environment. Neither scenario meets all the required criteria, however.

7.8.5 Replacing or Relining Pipe/ Downsizing Mains

The Burlington distribution system has been designed with a significant amount of excess capacity. Overcapacity in water distribution systems is typical throughout the province for a number of reasons including common design practice, to accommodate potential growth or industry such as fish plants, to achieve sufficient fire flows, and to fit fire hydrants. Pipe sizes range from 250-75 mm, with the majority of trunk main sized at 200 mm. The maximum observed velocity in the system is 0.18 m/s observed in a section of 75 mm pipe.

In the first scenario, old 75mm PVC pipe from 1980 was relined with new 150 mm PVC pipe with a Hazen-Williams C value of 150. The model results indicated no improvement in chlorine levels and water age was increased from 30 to 48.5 hours at the end of the system.

For the second scenario, each pipe was resized so as to achieve a peak velocity of approximately 0.4 m/s or a minimum pipe size of 40 mm. Under these criteria, pipe size in the Burlington distribution system now range from 40-100 mm. The resulting maximum water age at the end of the system is now 8.6 hours (reduced from 30 hours). Pressure at the highest point of elevation (end of the network) has fallen below the minimum pressure criteria. Chlorine residuals at the end of the system have decreased, and are slightly lower throughout the entire network. The contact time at peak flow has fallen from 20 minutes to only 3. The rate of reaction of chlorine at the pipe wall is inversely related to pipe diameter, so the smaller the pipe diameter, the greater the pipe wall reaction rate and the greater the amount of chlorine consumed at the pipe wall.

7.8.6 Reconfiguring the Distribution System through Looping

There is limited potential for looping in the Burlington distribution system as it is a long linear system with a small number of dead ends. For this scenario, 2 additional pipes were included in the network incorporating 3 dead ends into loops. Further looping is not feasible.

Figure 91: Looping of the Burlington network

With the system looped, maximum water age at the end of the system is now 31 hours, an increase of 1 hour over existing conditions. The slight increase in water age also resulted in a slight lowering of chlorine levels at the end of the system. There was no observed benefit from looping the Burlington distribution network to try and reduce DBPs.

7.8.7 Combination of Corrective Measures/ Upgrade Distribution System

Potentially, no single corrective measure may meet all system criteria in order to be deemed successful. For Burlington, a combination of corrective measures was applied including:

- Moving the main chlorination system closer to the intake at node 6 with a dose of 5 mg/L
- Installing a chlorine booster at node 13 with a dose of 3.5 mg/L without flushing and 1 mg/L with flushing
- Replacing old 75 mm pipe at the end of the network with new 150 mm PVC pipe
- Flushing of the network at end node 15 at 1.5 L/s for 4 hours every 12 hours

Moving the main chlorination system closer to the intake increases the contact time at peak flow to 96 minutes. Under flushing conditions of 1.5 L/s at the end node, the contact time is 51 minutes. At no point in the system are chlorine levels above 4.0 mg/L, and at no point are they below 0.05 mg/L, with or without flushing. Pressure in the system is also above the minimum criteria. Water age without flushing is 48.5 hours, and with flushing decreases to 25.3 hours.

7.9 Impact of Modeled Corrective Measures

Of the 9 corrective measures identified in a previous section that could be modeled in EPANET in order to access their impact in terms of improving water quality (looking at chlorine, water age, and potential THM formation), three were grouped together with other related scenarios. In the case of Burlington, none of the scenarios modeled met all the required criteria in order to be deemed successful. Some scenarios did see a reduction in the overall chlorine dosage and a decrease in water age with the potential for lowering THM levels. The only way for the Burlington network to meet all required system criteria was to run a scenario that included a combination of different corrective measures. The following table highlights results from the scenarios modeled.

Table 68: Modeled scenarios for the Burlington network and their effectiveness

	Scenario Description	All Criteria Met	Comments
1	Optimize Disinfectant Dose	No	-Cl dose greater than 5.0 mg/L violates max Cl at 1st user of 4.0 mg/L -Secondary disinfection requirements not met at any reasonable Cl dosage
2	Install Chlorine Booster at Optimal Location	No	-potential to reduce overall chlorine use -required dosage at booster violates max Cl of 4 mg/L

3	Chlorine Dosage Control	No	-Greater Cl variability -No potential to reduce overall Cl dose -Secondary disinfection requirements not met
4	Regular System Flushing at Dead Ends/ Continuously Bleed System	No	-Water age decreased -Contact time, minimum pressure, primary and secondary disinfection criteria all violated
5	Replacing or Relining Mains/Downsizing Mains	No	-Contact time, minimum pressure, secondary disinfection criteria all violated
6	Reconfiguring Distribution System through Looping	No	-Water age increased slightly - Secondary disinfection requirements not met
7	Combination of CDM and RTM Corrective Measures/ Upgrade Distribution System	Yes	-Water age decreased -potential to reduce overall chlorine use -Contact time, pressure, primary and secondary disinfection criteria all met

Applied independently, none of the examined corrective measures met all criteria in order for a scenario to be deemed successful. Any corrective measures that did not meet the necessary criteria should be dropped from consideration and evaluated no further. A combination of CDM and RTM corrective measures applied together, however, did meet all system requirements, with potential for overall chlorine use and water age to be reduced; this combination will likely result in some reduction in THMs. Based on this assessment, the corrective measures (that met criteria) with the most potential for reducing THM formation is:

- A combination of CDM and RTM corrective measures
  - Relocating chlorination system
  - Installing a chlorine booster
  - Replacing pipe
  - Regular system flushing

7.10 Assessment of Corrective Measure Constraints for Burlington Network

The following table evaluates each remaining corrective measure for the Burlington water distribution system against identified solution constraints. The selection of the preferred solution(s) to water quality problems can be made based on the corrective measure(s) with the highest score(s).

Based on the resulting scores, there are three main tiers of possible solutions. The top three tiers in the decision matrix scoring system comprise the corrective measures that have the most potential for effectively optimizing chlorine dosage, reducing water age and lowering THMs.

The first tier, which scored 14, consists of the installation of a Potable Water Dispensing Unit. The second tier of solutions, which scored 13, consists of the general best management practice of improving system design. The third tier of corrective measures, which scored 12, consisted of “soft” solutions like watershed protection, training and adaptive policy to promote PWDU. It also included “hard” practices such as upgrading the distribution network through a combination of CDM and RTM measures, and relocating the intake to deeper water.

The selection of a preferred solution by the decision making body (town, engineering consultant, Department of Municipal Affaires) can be guided by this decision making framework. The next step in the process involves the implementation of the preferred solution, monitoring and review.

Table 69: Assessment of solution constraints for Burlington

Applicable Corrective Measures	Effectiveness	Cost	Time Scale for Implementation	Permanency of Solution	Adverse Hydraulic Impacts	Adverse WQ Impacts	Acceptable to Stakeholders	Meets Regulations	Total
Policy of POU/POE treatment	1	2	0	0	1	2	1	1	8
Advanced treatment	2	0	0	2	1	2	0	2	9
Alternative water source	1	0	0	2	1	2	1	2	9
High quality water storage and recovery	1	0	0	2	1	2	1	2	9
Policy to promote use of alternative disinfectants	1	2	0	1	1	1	1	2	9
Alternative disinfectants	2	1	1	1	1	1	1	2	10
Regionalization	1	1	1	2	1	1	1	2	10
Water treatment plants	2	0	0	2	1	2	1	2	10
Filtration	1	1	1	1	1	2	2	2	11
Point of use/entry treatment	2	2	2	0	1	2	1	1	11
System maintenance	1	2	2	0	1	2	1	2	11
Combination of corrective measures/ Upgrade distribution network	2	1	1	1	1	2	2	2	12
Policy to promote PWDU	1	2	0	2	1	2	2	2	12
Relocate intake in deeper water	1	1	1	2	1	2	2	2	12
Training	1	2	0	1	2	2	2	2	12
Watershed protection	0	2	1	2	1	2	2	2	12
Improved design of systems	1	2	0	2	2	2	2	2	13
Potable water dispensing unit	2	2	1	2	1	2	2	2	14

8.0 Ferryland Water Distribution System Model

Prior to 1988, the Ferryland water distribution system was fed by a series of interconnected, but poorly producing groundwater wells. The distribution network was mostly made up of 50 mm polyethelene (PE) pipe and was plagued with both water shortages and low pressures. A 12 Phase infrastructure plan was proposed in 1988 which included construction of a new gravity fed surface water source called Deep Cove Pond, and upgrades to existing infrastructure. To date, only up to Phase 3 (from the intake to midway along the distribution system (node 7) has been completed, leaving the Ferryland distribution network a somewhat haphazard mix of new and old piping of various size and material. The layout of the distribution network is also slightly irregular, due to the nature of older systems and their rather organic as opposed to planned development, and the availability of capital works funding for upgrades. With the current configuration of the Ferryland distribution network, water flows towards the end of the network, hooks around and flows back towards the middle of the network (in the southern end).

The Ferryland water distribution system is typical of many small towns in Newfoundland & Labrador– a long linear system with a surface water supply, with raw water displaying relatively high colour and dissolved organic carbon (DOC). The system has a screening chamber located in the chlorination building near the source intake. The liquid chlorination system is flow regulated. Upgrades to the Ferryland distribution system have been ongoing since 1988. A booster chlorination system was also installed in the middle of the system in early 2006.

Ferryland has had a problem with THM levels being over Canadian Drinking Water Quality Guidelines since 2001. According to town officials, there were problems in maintaining chlorine residuals towards the end of the distribution system and so the chlorine dose was kept high. Since the installation of the chlorine booster, overall chlorine use by the town has decreased and there has been some reduction in THM levels. The Ferryland distribution system can be classified as small and from the Eastern region of the province.

Figure 92: Ferryland water distribution system network

Descriptive data for the Ferryland water distribution system is detailed in following sections. This data was then input into the Ferryland EPANET hydraulic/water quality model. The next step involved calibrating the Ferryland model with system data also highlighted in the following sections. Different corrective measures and modeling scenarios were then selected based on observed problems with how the distribution system is currently operating. The potential effectiveness of the given solution or modeled scenario was then weighted against solution criteria and constraints.

8.1 Reservoir

The water supply for the town of Ferryland is Deep Cove Pond, located approximately 1.5 kilometres northwest of town. Deep Cove Pond has a catchment area of 1.17 km2. All previous groundwater sources have been phased out. The reservoir has a water level of 101.3 m. The intake is a 450 mm HDPE pipe located 60 m out into Deep Cove Pond in 3 m of water. A primary screening chamber exists in the chlorination building near the intake to help deal with solids, turbidity and colour problems.

Table 70: Average source water quality values for Ferryland

Water Quality Parameters	Average Values 1990-2006
Colour (TCU)	35.6
pH	6.2
Turbidity (NTU)	0.51
Bromide (mg/L)	0.02
Chloride (mg/L)	8.79
DOC (mg/L)	5.51

Temp (^oC)	12.3
Iron (mg/L)	0.08
Manganese (mg/L)	0.013

8.2 Pipes

The majority of pipes in the current Ferryland distribution system were installed since 1988. The intake pipe from the reservoir to the chlorination building is a 450 mm diameter HDPE pipe. The trunk main carrying water from the chlorination building to the community is 350 mm diameter HDPE. This pipe branches off to a 100 mm HDPE main near the school from the Southern Shore Highway and follows the abandoned railway track to bring water to the southern end of Ferryland. Other secondary pipes are a mix of old and new; pipes left over from the old distribution system (dating back to the 1960s or 1970s) are generally of PE and range from 25-40 mm in diameter. New pipes in the network range from 100-350 mm and are composed of HDPE, PVC and ductile iron (DI). In total there is approximately 6 km of trunk main laid down in the Ferryland distribution system.

The Hazen-Williams head loss formula was selected for this model in order to determine energy losses throughout the system. Roughness factors were selected based on pipe age: 150 for newer HDPE, and 140 for older HDPE, 150 for new PVC, and 130 for DI.

From information gathered on the system, line pressure has been estimated to range from 350-945 kPa (35-96.3 m). In addition, there are at least 13 fire hydrants located at different points on the distribution system.

8.3 Demand

The Ferryland distribution system has a flow meter located on the system at the point where the pipe from the source intake bends south to follow Highway 10 (Southern Shore Highway) into the community. Readings from the meter taken from Nov 21-29, 2000 indicate average daily consumption is 530 m^3/d or 6.13 L/s. Information gathered from earlier sources sets the available yield of the system at 114 L/s (9850 m^3/d). Types of water users and their number are summarized in the following table.

Table 71: Number and type of water user

Type of Water User	Number
Residential	169
School	1
Institutional (Municipal Hall/ Visitors Center)	2
Commercial (stores)	3

Residential demand was allocated to 20 different junctions throughout the distribution network based on housing density surrounding that junction. Non-residential demand is not significant on this system and so was equated to an equivalent number of residential properties.

With a population in 2001 of 607 residents, per capita demand is 873 L/p/d, based on an the average demand of 530 m^3/d.

Elevation of junctions ranged from 63 m (beginning of system) to 6 m above sea level along the coast.

Meter readings have not been taken at a frequency to establish a daily demand pattern for the Marystown distribution system. Peaks in the morning, noon and evening for domestic users are typical however. The following generic demand pattern was used in the Ferryland model for domestic water use.

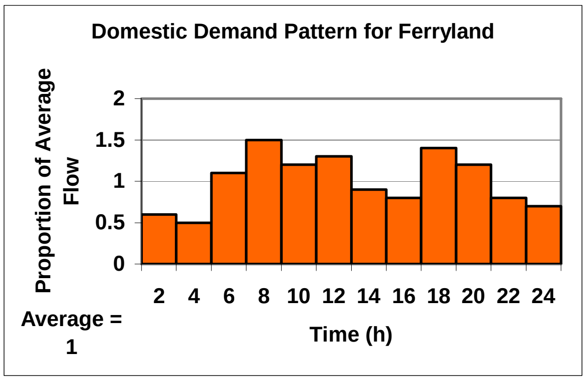


Figure 93: Typical domestic demand pattern

8.4 Chlorine Decay

Prior to 1988, Ferryland used gas chlorination for drinking water disinfection. After the switch to Deep Cove Pond, the town relied on two liquid (sodium hypochlorite) chlorination systems. One was located near the intake (node 2), and the other in the same building as the chlorine meter (node 3) where the main leading from the intake turns south to follow Highway 10. The chlorination systems each had one 200 L tank that was typically used over a 7-day period. The amount of chlorine injected into the distribution system was flow regulated for the chlorination system coupled with the flow meter. A chlorine dose of 1-3 ppm (1-3 mg/L) was typical.

Figure 94: Ferryland chlorination system at the intake (left) and at the flow meter (right)

In early 2006 a new chlorine booster was installed (at node 18) just before the 100 mm HDPE pipe runs inland and follows the abandoned railway track. This chlorination system has a 200 L chemical feed tank. The booster chlorination system is also flow controlled. With the installation of this booster, the chlorination system at the intake was decommissioned.

Both the main and booster chlorination systems are using liquid bleach (12% chlorine solution) for disinfection. The main chlorination system uses straight 12% solution sodium hypochlorite without any dilution. The 200 L chemical feed tank must be refilled approximately every 12 days in winter and every 7 days in summer. Depending on the flow conditions and chlorine demand, the initial chlorine dose can range from 3.78-10.66 mg/L. For modeling purposes an initial chlorine dose of 6.48 mg/L will be used. The booster chlorination system uses a diluted solution of sodium hypochlorite, assumed to be 3%. Depending on the flow conditions and chlorine demand, the booster chlorine dose can range from 0.71-1.46 mg/L. For modeling purposes a booster chlorine dose of 1.05 mg/L will be used.

Based on an average daily flow of 6.13 L/s, the available contact time is 242 minutes. The contact time at peak flow using the Harmon Formula is 63 minutes (a minimum of 20 minutes is required).

In lieu of a bulk chlorine decay test, a typical value for bulk decay coefficient was selected based on results from other decay tests on provincial surface water supplies. A default bulk chlorine decay coefficient of -1.5 d^{-1} was selected for the Ferryland model. A default wall decay coefficient of -1 m/day was also selected.

8.5 Chlorine and THM Data

Chlorine tests are regularly made by the Ferryland System Operator and by Department of Environment and Conservation staff. The following table summarizes average total and free chlorine, total THM, and BDCM results. Typically free and total chlorine are highest at the beginning of the system and lowest at the end, however, the distribution of data was unbalanced with hardly any readings from the middle of the system, readings from the beginning of the system from 2002 and earlier, and reading from the end of the system all from after mid-2002. Field readings taken on Feb 18, 2008 (in brackets) are felt to be more representative of the system and were used in calibration. There has been some improvement in average THM levels in the Ferryland system with the installation of the chlorine booster and a resulting reduction in overall chlorine use.

Table 72: Average chlorine, THM and BDCM (1998-2007) for Ferryland network

Location in Network	Junction	Free Chlorine - DOEC (mg/L)	Total Chlorine-DOEC (mg/L)	THM Total-DOEC (ug/L)	BDCM-DOEC (ug/L)
Beginning	11	0.19 (1.09)	0.23 (1.33)	53.4	2.1
Middle	9	0.02 (0.20)	0.08 (0.45)	109.4	8.2
End	16	0.41	0.53	249.3	10.8
End (after	16	0.14 (0.24)	0.25 (0.37)	201.3	7.48

booster installed)

The GCDWQ maximum acceptable concentration (MAC) for total THMs is 100 ug/L. As shown in the table, THM levels in Ferryland are over the limit in the middle and at the end of the distribution system.

8.6 Calibration of the Ferryland Model

In order to first calibrate the Ferryland hydraulic/water quality model, results were compared with flow, pressure and chlorine residual data gathered on the Ferryland distribution system. The collection of this data is outlined in previous sections.

Comparison of initial model results to calibration data is described in the following table, along with actions taken to compensate for any discrepancies, and final associated percentage errors found in the calibrated model. Average values from the model are taken for comparison once equilibrium or periodic behaviour from that parameter had been reached.

Table 73: Calibration of the Ferryland model

Issue	Percentage Error	Action	Percentage Error After Calibration
-average daily model flow of 6.13 L/s (daily range of 3.07-9.19 L/s) vs. average flow of 6.13 L/s	-0%	None- metered flow readings used for input, no comparison data available	
- node 20 (second highest elevation) model pressure of 29.7 m (range 20.7-38.6 m) vs. estimated min line pressure of 35 m -node 7 (lowest elevation) model pressure of 83 m (range 73.5- 92.5 m) vs. estimated max line pressure of 96.3 m	-13.8% -15%	None- model pressures display less than 16% error from estimated line pressures	
-node 11 (beginning of system) equilibrium Cl of 3.22 mg/L (range of 2.68-3.72 mg/L) vs. 1.09 mg/L	-195%	-decrease main Cl dose to 3.78 mg/L -increase booster Cl dose to 1.72 mg/L -increased bulk reaction rate from -1.5 d^{-1} to -2.2 d^{-1} -increase wall reaction rate from –1 m/d to –1.5 m/d	-45.4% (1.59 mg/L)
-node 9 (middle of	-130%	-decrease main Cl dose to 3.78	-12.5%

system) equilibrium Cl of 0.46 mg/L (range 0.13-0.79 mg/L) vs. 0.20 mg/L		mg/L -increase booster Cl dose to 1.72 mg/L -increased bulk reaction rate from –1.5 d^{-1} to –2.2 d^{-1} -increase wall reaction rate from –1 m/d to –1.5 m/d	(0.23 mg/L)
-node 16 (end of system) equilibrium Cl of 0.32 mg/L (range of 0.10-0.53 mg/L) vs. 0.24 mg/L	-33%	-decrease main Cl dose to 3.78 mg/L -increase booster Cl dose to 1.72 mg/L -increased bulk reaction rate from –1.5 d^{-1} to –2.2 d^{-1} -increase wall reaction rate from –1 m/d to –1.5 m/d	-41.7% (0.14 mg/L)

The calibration data set for Ferryland only covered basic elements, resulting in a rough calibration. Once results predicted by the model were felt to adequately reflect observed field data– matching pressures, flows, chlorine residuals– through the adjustment of certain network parameters, a baseline model was established. The different model scenarios will then be run on this baseline model, adjusting only selected network parameters.

The following graph shows mean observed versus mean simulated values of pressure for the Ferryalnd system. As can be seen in the graph and calibration table below, actual and modeled pressures match very closely.

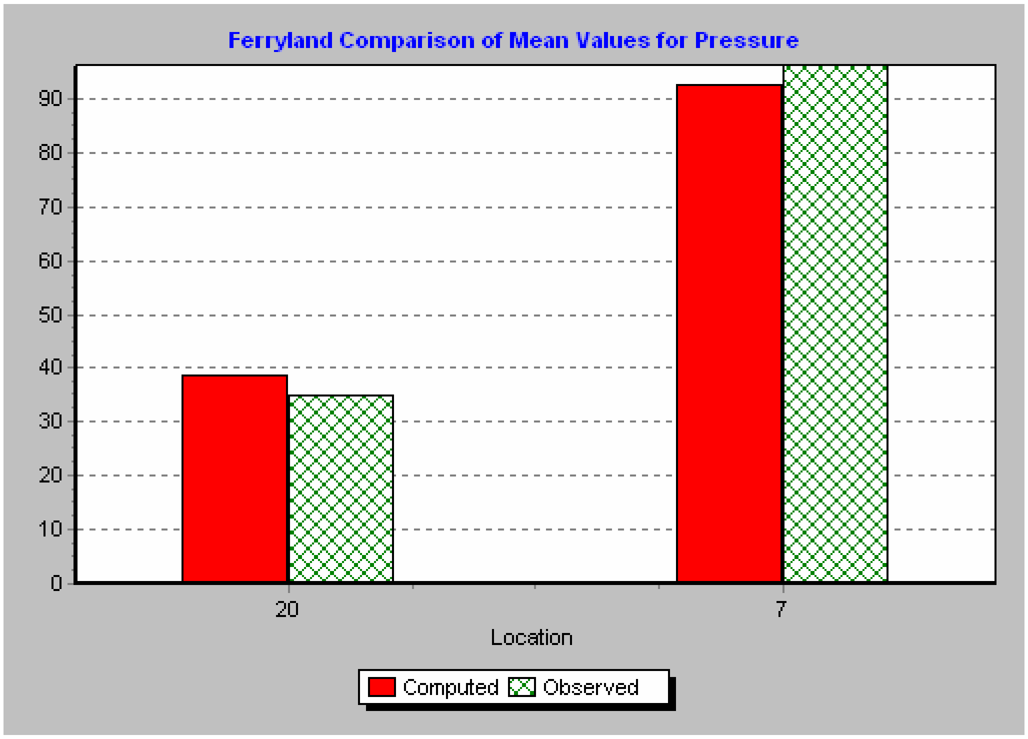


Figure 95: Mean observed and mean simulated values for pressure in Ferryland

Table 74: Calibration statistics for pressure

Location	Num Obs	Obs Mean	Comp Mean	Mean Error	RMS Error
15	1	35.00	38.61	3.612	3.612
2	1	96.30	92.44	3.856	3.856
Network	2	65.65	65.53	3.734	3.736

Correlation Between Means: 1.000

The following graph shows system flows over the 7-day simulation period. Given the domestic demand pattern used, the graph indicates expected variation from the average flow of 6.13 L/s.

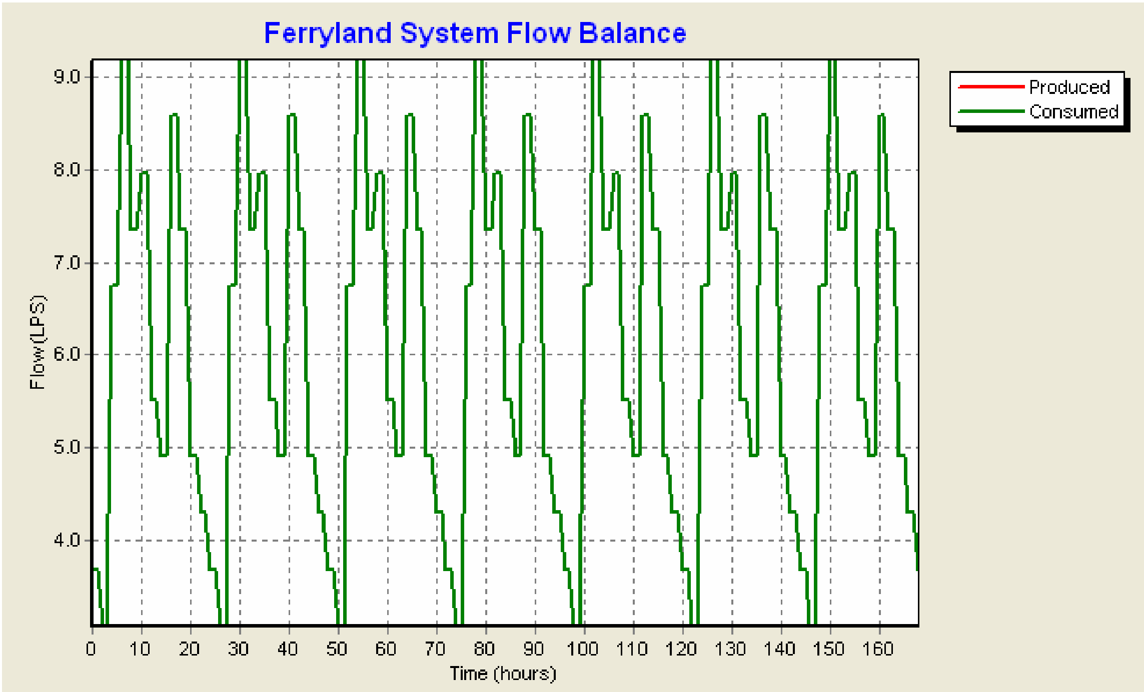


Figure 96: Ferryland system water demand

The following table and graph show calibration statistics for free chlorine residuals taken from three different points in the Ferryland distribution system. Observed chlorine readings taken from the field were assigned times after equilibrium had been reached for each node. Once chlorine reached equilibrium it still varied significantly with changes in system demand, particularly at locations downpipe of the chlorine booster. There was very good correlation between the observed field chlorine readings and model results.

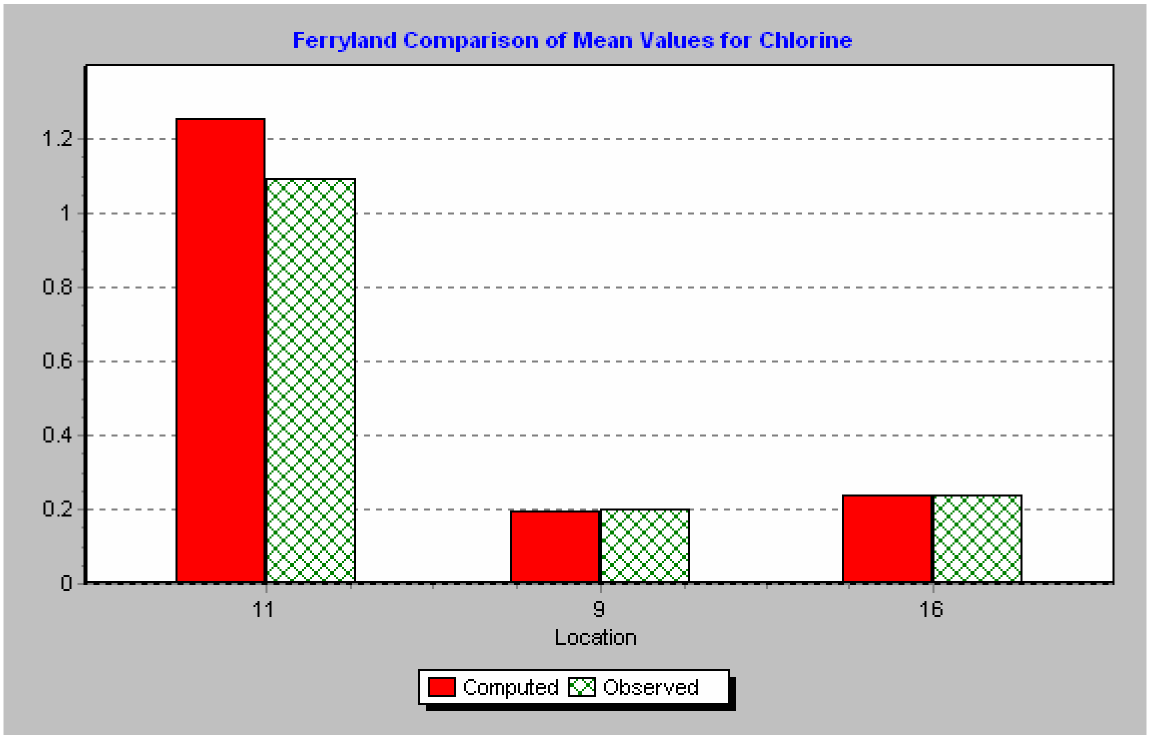


Figure 97: Mean observed and mean simulated value for chlorine residuals in Ferryland

Table 75: Ferryland calibration statistics for chlorine

Location	Num Obs	Obs Mean	Comp Mean	Mean Error	RMS Error
11	1	1.09	1.26	0.165	0.165
9	1	0.20	0.19	0.007	0.007
16	1	0.24	0.24	0.004	0.004
Network	3	0.51	0.56	0.059	0.096

Correlation Between Means: 1.000

8.7 Problems with the Ferryland Distribution System

By gathering detailed background information on the Ferryland water distribution system and establishing a calibrated baseline model, we were able to then identify problems with how the system operates normally. Several contributing factors were identified as contributing to the overall Ferryland THM problem as outlined in the following table.

Table 76: Problems contributing to high THMs in the Ferryland distribution system

	Causative Factors	Quantitative Value
5	High DOC in source water	5.51 mg/L
7	High chlorination dose	7.53 mg/L total main and booster dose
10	Excessive chlorine demand	-2.2 d-1 (bulk) -1.5 m/d (wall)
12	Long linear system	5.8 km intake to end total = 10.5 km

13	Branched system with multiple dead ends	at least 5 DE
14	Distance of chlorination system to first point of use	925 m contact time= 63 min CT = 79.4
15	Insufficient chlorination controls on system	flow proportional
18	Pipe material and age	>20 yrs
20	Large occasional demand on system	seasonal tourism
26	Poor O&M of system	Water Dist Class I
27	Multiple factors	-
28	Poor design of system	-
30	High per capita demand	873 L/p/d
31	Pressure problems	max = 96.3 m
32	Problems with chlorine residuals	0.01 mg/L @ end

The following figures illustrate some of the problems observed in the Ferryland distribution system including difficulty in maintaining adequate chlorine residuals at the end of the system, excessive chlorine demand, and a high rate of chlorine decay through the network.

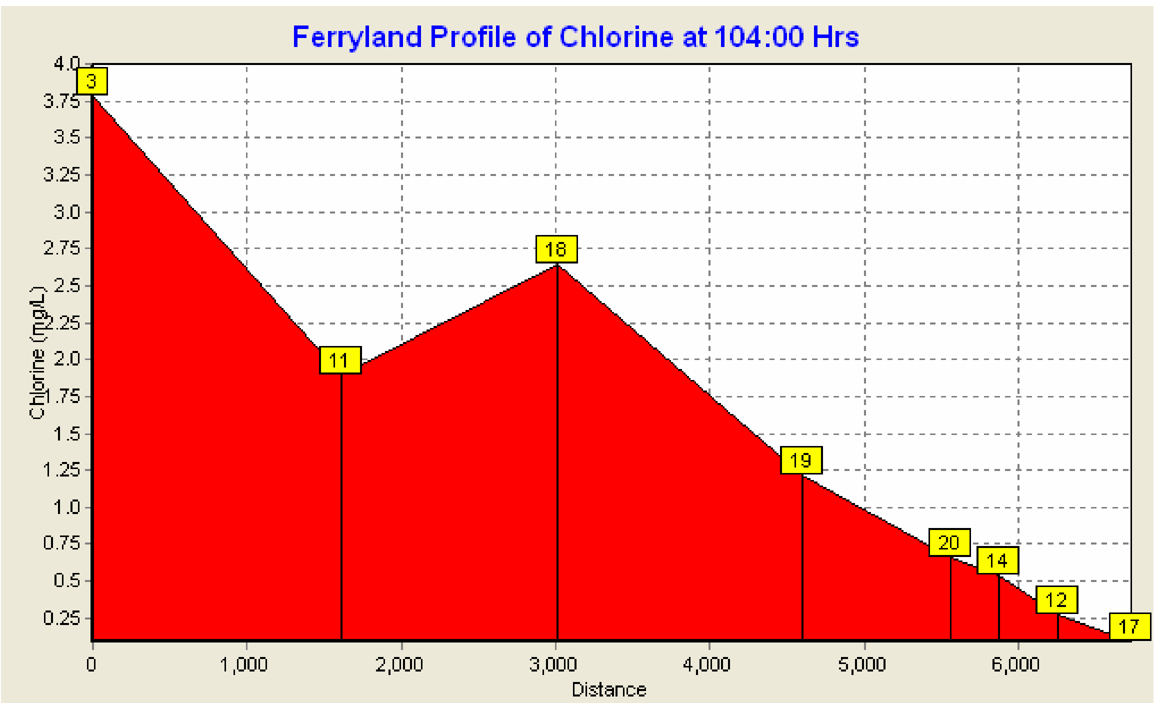


Figure 98: Chlorine decay profile through Ferryland distribution system

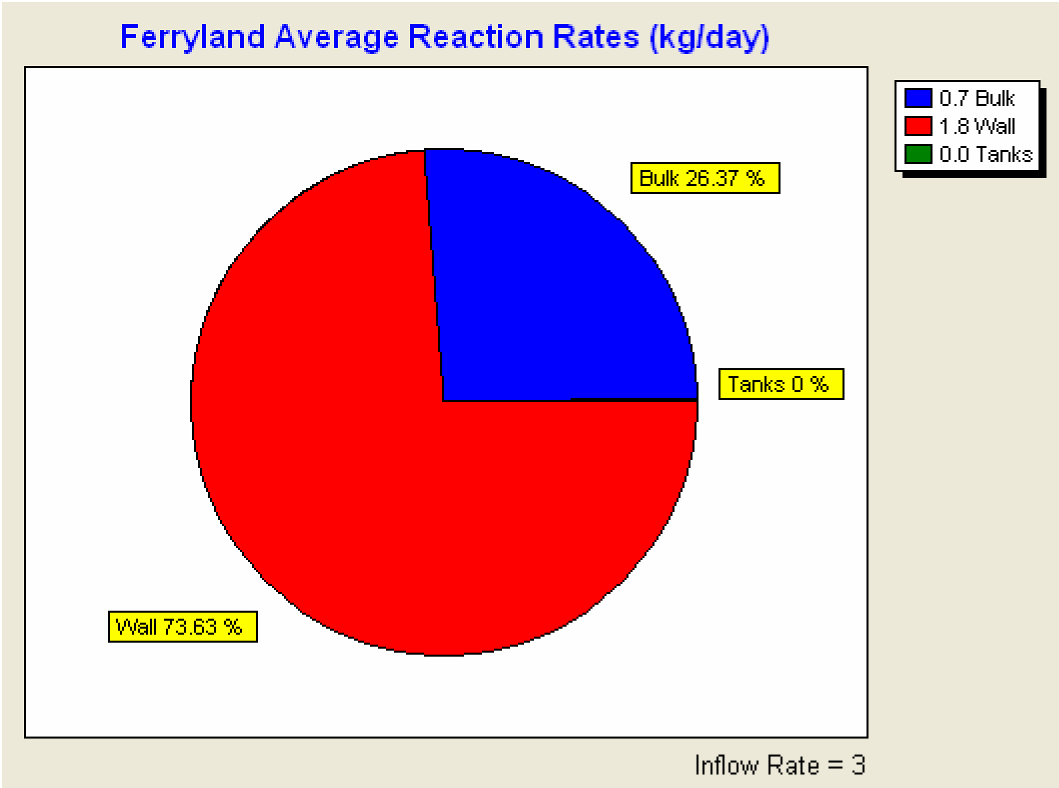


Figure 99: Chlorine decay contributions in the Ferryland distribution system

Solutions that might address the probable causes of high THM levels in the Ferryland distribution system are outlined in the following table. Those corrective measures highlighted in grey are the only solutions that can potentially be modeled.

Figure 100: Applicable THM corrective measures for Ferryland

Applicable Corrective Measures	Probable Causes Addressed
Policy of POU/POE treatment	All
Policy to promote use of alternative disinfectants	All
Policy to promote PWDU	All
Watershed protection	All
Alternative water sources	5
High quality water storage and recovery	All
System maintenance	All
Regionalization	All
Alternative disinfectants	All
Potable water dispensing unit	All
Point of use/entry treatment	All
Training	All
Improved design of systems	All
Optimize disinfectant dosage	5-7
Re-locate chlorination systems	5-7-14
Chlorine dose control	5-7-15
Regular system flushing at dead ends	5-12-13-20

Continuously bleed system at dead end	5-12-13-20
Abandoning or downsizing mains	5-12-13
Replace or reline pipe	18
Loop distribution network	13
Upgrade distribution network	18
Water treatment plants	5-7
Filtration	5-7
Advanced treatment	5-7
Combination of corrective measures	All

8.8 Results from the Ferryland Model

The next step was to model the different selected scenarios and see how the Ferryland distribution system responded. Given the ability of the baseline model to reflect current conditions accurately, a reasonable degree of confidence can be placed in the scenario results.

8.8.1 Optimize Chlorine Dosage

The Ferryland distribution system has a main hypo-chlorination system located where the main from Deep Cove Ponds hits Highway 10. The booster hypo-chlorination system is located 2.2 km downpipe from the main chlorination system. The main and booster chlorination system dosages are calculated to be 6.48 mg/L and 1.05 mg/L respectively. Both chlorination systems vary their dosage with flow, and calculated dosages are based on average flow. Primary disinfection requirements are met by the system; however, secondary disinfection requirements of maintaining a free chlorine residual of at least 0.05 mg/L at the end of the system are not met.

In EPANET we have chosen to model chlorine as a setpoint booster at nodes 3 and 18, which fixes the concentration of any flow leaving that node. This scenario looks at trying to optimize the chlorine dosage between the main and booster system. As stated in the objectives, the Ferryland system should have a 20 min contact time, contain a free chlorine residual of at least 0.3 mg/L at the first point of use, and maintain a free chlorine residual of 0.05-0.10 mg/L at the end of the distribution system. The following table summarizes the results of altering chlorine dosage.

Table 77: Altering chlorine dosage in Ferryland distribution system

Initial Chlorine Dose/ Booster Dose (mg/L)	CT Value at 1st User	Min Cl Residual at Start of System – node 11 (mg/L)	Max Cl Residual at Start of System – node 11(mg/L)	Cl Residual before Booster – node 18 (mg/L)	Max Cl Residual after Booster – node 18 (mg/L)	Min Cl Residual at End of System – node 17 (mg/L)
3.78/1.72	78.8	1.25	1.92	0.33-0.93	2.65	0.01
2.5/2.5	52.3	0.83	1.27	0.20-0.61	3.11	0.01
2.5/3.4	52.3	0.83	1.27	0.20-0.61	4.01	0.02
8.0/2.0	167	2.65	4.06	0.68-1.97	3.97	0.01

No combination of initial and booster chlorine dose examined met all criteria, as chlorine residual at the end of the system were always below 0.05 mg/L. Having a higher chlorine dose at the booster relative to the main chlorinator did result in a slight improvement in the chlorine residual at the end of the system.

8.8.2 Relocate Chlorination Systems

The main chlorination system for Ferryland is technically located in Calvert, the community north of Ferryland. There are no connections on the line until the school in Ferryland. The diameter (350 mm) and length (925 m) of pipe from the main chlorination system to the first user provides a contact time of 63 minutes at peak flow. As only a 20 minute contact time is required, there is potential to move the main chlorination system closer to the first user. Placing the chlorination system 450 m from the fist user reduces the contact time at peak flow to 30 minutes. At a dose of 5.5 mg/L, the maximum chlorine level at the first point of use is 4.0 mg/L.

A chlorine booster is a secondary chlorination system located on a water distribution system to boost chlorine residuals to appropriate levels in areas where they may have fallen below acceptable levels. The satellite chlorination system that was commissioned in 2006 is located approximately midway along the system at node 18, between the north and south ends of the distribution network. At this location, there are issues with maintaining adequate chlorine residuals at the end of the network. For this scenario, the location of the chlorine booster was changed to node 19, closer to the southern cluster of development in Ferryland. With a booster dose of 3 mg/L, adequate chlorine residuals are achieved throughout the system. The combined dose is now 8.5 mg/L as compared with 7.53 mg/L. With no reduction in overall chlorine use, there is little potential for a reduction in DBP levels. The further south on the system the chlorine booster is located, the higher the required chlorine dose at the beginning of the system in order to achieve acceptable residuals at dead ends on the north part of the distribution network.

8.8.3 Chlorine Dosage Control

The main and booster chlorination systems in Ferryland are both flow controlled, meaning the rate of chlorine solution injected into the water distribution system alters proportionally with flow. For calibration purposes, chlorine dosage was modeled as a constant dose as no information was available on typical fluctuations of the chlorine control system. Both flow and chlorine residual feedback controls can be used to manage the chlorine feed.

The chlorine dose was made to vary with time using two time patterns: one the same as that used for water demand, the other opposite to that used for water demand. Feedback control of chlorination systems typically function by increasing the chlorine dose when flows increase in order to maintain CT values at the first point of use. When demand is high, water moves at an increased rate through the distribution system, resulting in reduced water age, less time for chlorine decay, and higher chlorine residuals. The variation in chlorine will mimic the peaks and lows of flow throughout the system (for chlorinators that are flow controlled), only the lag time between peaks in flow and peaks in chlorine residuals will increase the further you get towards the end of the distribution system.

The following three graphs look at the variation in chlorine readings at different points in the network if the chlorine dose is constant, increases with flow, or decreases with flow. There is currently more variation in chlorine residuals down-pipe of the booster than up-pipe. Variation in chlorine residuals becomes more pronounced with flow control.

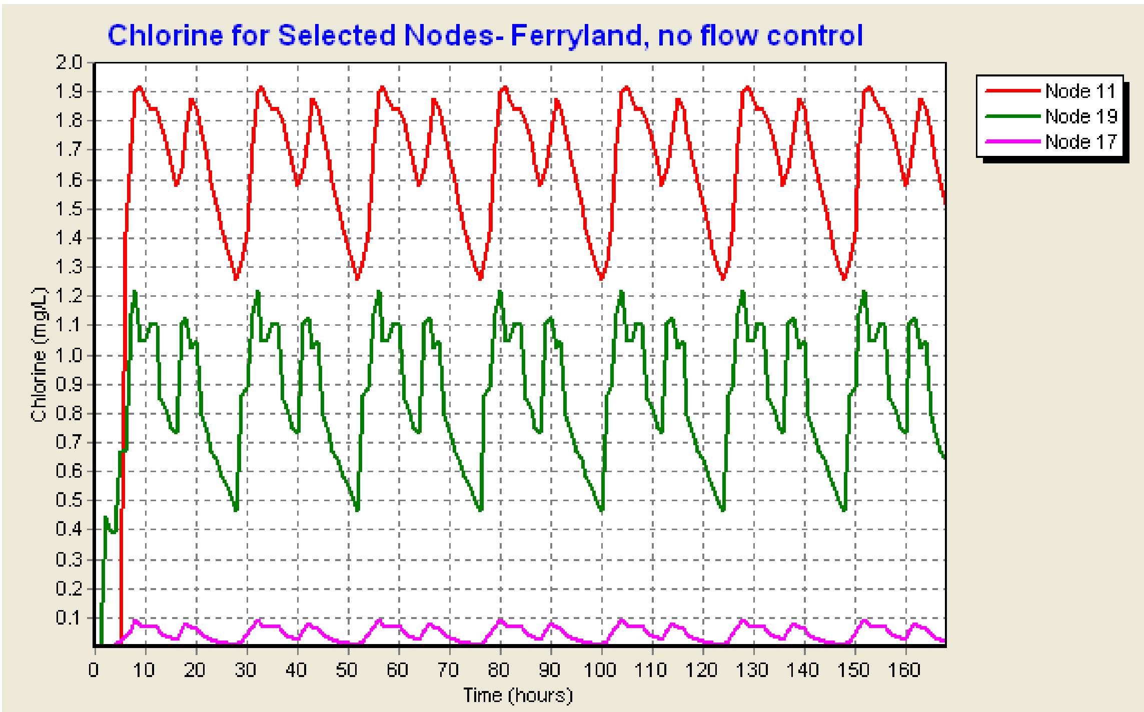


Figure 101: Chlorine levels with constant dosage, Ferryland

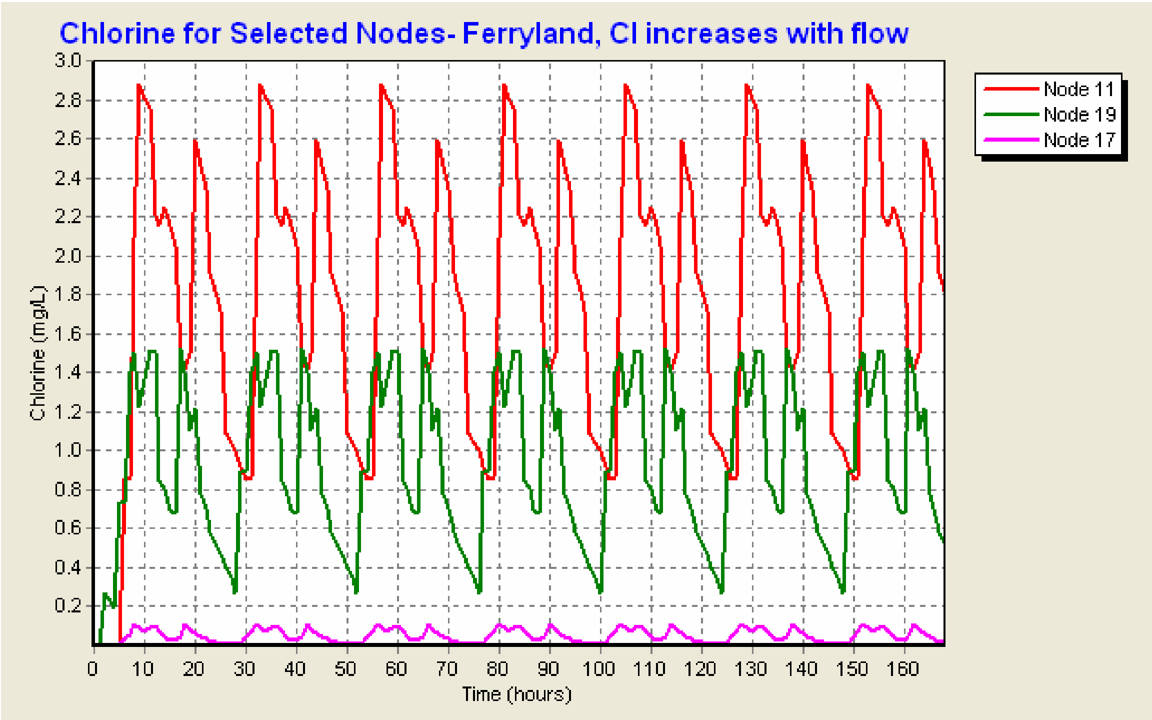


Figure 102: Chlorine levels with flow proportional chlorine control, Ferryland

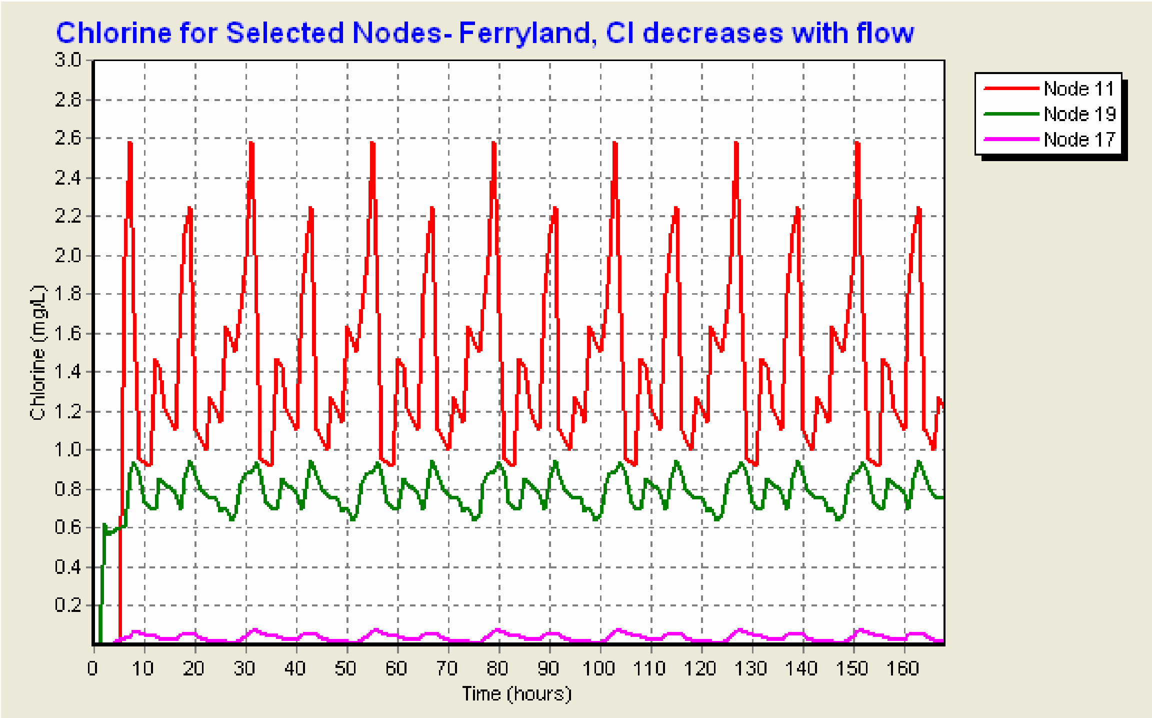


Figure 103: Chlorine levels with inverse flow proportional chlorine control, Ferryland

In the case of Ferryland, the lag time between the peak in flow and the corresponding peak in chlorine at the point of maximum residency in the network is 15 hours, indicating the difficulty in trying to optimize chlorine through flow control. With flow control, secondary disinfection requirement were still not met, and there was no option available to reduce the overall chlorine dosage. There are complicating factors involved with chlorine dosage control that make it effective only in select parts of the distribution system whether flow or residual controlled.

8.8.4 Regular System Flushing/ Continuous Bleed at Dead Ends

The current Ferryland water distribution network was originally two separate distribution systems that were later joined when water quality from the groundwater wells serving the southern end of town became an issue. Both the north and south ends of the current distribution network contain a combination of new, larger pipe and old, smaller diameter pipe. Due to the abrupt pipe diameter changes in the network, you get a combination of fast and slow moving water in the network. The maximum retention time in the Ferryland distribution system is 27.1 hours. Any flushing program, therefore, must occur at a time period of less than this in order to achieve any improvement in water age, ideally at half the current return period or approximately every 13.5 hours. For this corrective measure, two scenarios were examined: flushing twice a day at 3 dead ends, and continuous bleeding of selected dead ends (nodes 7, 17, 10).

The average daily flow rate (demand) on the system is 6.13 L/s, and flushing rates will be some multiple of this. Maximum pressure is violated at low elevations, but this is not considered a major issue. Negative system pressures are experienced with an increase of average flow of only 12 % at node 17. Under average flow conditions, the maximum velocity reached in the network is 1.24 m/s, well above required flushing velocities.

For the scenario where flushing occurs twice a day at dead ends (nodes 7, 17 and 10), base demand at these nodes was increased by a factor of 5 resulting in an additional 3.92 L/s instantaneous demand on the entire system for 4 hours at 12 hour intervals. Chorine residual improved slightly at the end of the distribution network from 0.01 to 0.02 mg/L. Water age at the end of the network was reduced from 27.1 to 19.6 hours. Contact time requirements at peak flow are not met with this scenario at 19 minutes.

For the continuous bleed scenario base demand at nodes 7, 17, and 10 was increased by a factor of 4 resulting in an additional continuous 2.94 L/s to average demand. With more demand at dead ends of the system, water moves faster through the distribution network. Minimum chlorine readings at the end of the system increase to 0.05 mg/L, and there is an increase in chlorine levels throughout the network. Maximum water age in the network is also reduced from 27.1 to 14.2 hours. There is no opportunity to reduce overall chlorine dosage with this scenario. Both maximum and minimum pressure criteria are violated with continuous bleeds on the system. Contact time at peak flow with continuous bleeds on the system is reduced to 21 minutes, still within criteria range.

System flushing is more appropriate on distribution systems that are over-designed with excess capacity. Over capacity is not an issue on the Ferryland water distribution network with the large number of small diameter pipes. Increases in demand through flushing or bleeding cause pressure problems, violations of contact time, do not meet secondary disinfection criteria, and offer no opportunity to reduce the overall chlorine dose.

8.8.5 Upgrading the Ferryland Distribution Network- Reconfiguring, Abandoning and Replacing Mains

The Ferryland distribution network is a combination of old pipe from the 60's or 70's, and new pipe from 1988 or later. When the north and south ends of the network were combined on the Deep Cove Pond source, the plan was for additional phases of infrastructure development that would link up the 350 mm sections of pipe in the north and south ends of town, creating a new trunk main, so that the old 100 mm main running along the old railway track could be abandoned. This scenario looks at upgrading the Ferryland distribution network as planned, but never completed due to a lack of funding.

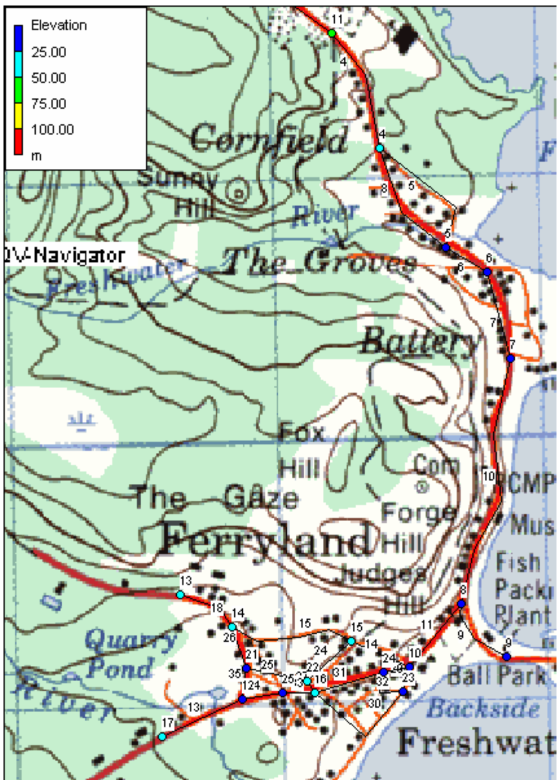


Figure 104: Upgraded Ferryland distribution network

The new 350 mm trunk main now runs along Highway 10 up to node 16. The remaining section of 40 mm pipe running along Highway 10 south of node 16 was replaced with 150 mm pipe. Additional 150 mm laterals have been added to the south end of the network to reduce the number of dead ends (number of dead ends reduced to 3). The 100 mm mains that ran inland have been removed from the network. Older 25, 40 and 100 mm laterals were left on the network.

With the current configuration of the Ferryland distribution network, water flows towards the end of the network, hooks around and flows back towards the middle of the network (in the southern end). With the upgraded configuration, water flows consecutively from the beginning to the end of the network. Even with a chlorine dose of 8 mg/L at the main chlorinator, secondary disinfection requirements cannot be met in most of the southern end of town. The booster chlorination system was located on one of the old abandoned lines and on the upgraded network was relocated to node 8. With a main chlorine dose of 3.5 mg/L and a booster chlorine dose of 3 mg/L, primary and secondary disinfection requirements on the system are met. The combined chlorine dose of 6.5 mg/L is less than the currant dose of 7.53 mg/L. Water age at the end of the network is increased from 27.1 to 42.2 hours with the network upgraded. Maximum pressure requirements are violated, but this is not a major issue. Maximum water velocity in the network has been reduced to 0.49 m/s with the removal of many smaller diameter pipes.

8.9 Impact of Modeled Corrective Measures

Of the 9 corrective measures identified in a previous section that could be modeled in EPANET in order to access their impact in terms of improving water quality (looking at chlorine, water age, and potential THM formation), five were grouped together with other related scenarios. Not all scenarios met the required criteria in order to be deemed successful. Any scenario that saw a reduction in the overall chlorine dosage and a decrease in water age has potential for lowering THM levels. The following table highlights which scenarios had a positive impact on water quality.

Table 78: Modeled scenarios for the Ferryland network and their effectiveness

	Scenario Description	All Criteria Met	Comments
1	Optimizing Chlorine Dosage	No	-Secondary disinfection requirements not met
2	Relocate Chlorination Systems	Yes	-Primary and secondary disinfection requirements met -No potential to reduce overall Cl dose
3	Chlorine Dosage Control	No	-Secondary disinfection requirements not met -No potential to reduce overall Cl dose
4	Regular System Flushing at Dead Ends/ Continuously Bleed System	No	-Pressure, contact time, secondary disinfection requirements violated
5	Upgrading the Ferryland Distribution Network- Reconfiguring, Replacing, Abandoning Mains	Yes	-Primary and secondary disinfection requirements met -Water age increases, but total Cl dose reduced

Any corrective measures that did not meet the necessary criteria should be dropped from consideration and evaluated no further. Scenarios that saw potential for overall chlorine use to be reduced and water age in the distribution system lowered will be the most effective in terms of lowering THMs. Based on this assessment, the corrective measures (that met criteria) with the most potential for reducing THM formation are:

- Optimizing location of chlorination systems

- Upgrading the Ferryland distribution network- reconfiguring, replacing, abandoning mains

8.10 Assessment of Corrective Measure Constraints for Ferryland Network

The following table evaluates each of the remaining corrective measures for the Ferryland water distribution system against identified solution constraints. The selection of the preferred solution(s) to water quality problems can be made based on the corrective measure(s) with the highest score(s).

Based on the resulting scores, there are three main tiers of possible solutions. The top three tiers in the decision matrix scoring system comprise the corrective measures that have the most potential for effectively optimizing chlorine dosage, reducing water age and lowering THMs.

The first tier, which scored 14, consisted of installing a Potable Water Dispensing Unit and upgrading the distribution system network. The second tier of solutions, which scored 13, consisted of the general best management practice of improving distribution system design. The third tier of corrective measures, which scored 12, consisted of “soft” solutions such as watershed protection, operator training and adaptive policy to promote PWDUs.

The selection of a preferred solution by the decision making body (town, engineering consultant, Department of Municipal Affaires) can be guided by this decision making framework. The next step in the process involves the implementation of the preferred solution, monitoring and review.

Table 79: Assessment of solution constraints for Ferryland

Applicable Corrective Measures	Effectiveness	Cost	Time Scale for Implementation	Permanency of Solution	Adverse Hydraulic Impacts	Adverse WQ Impacts	Acceptable to Stakeholders	Meets Regulations	Total
Policy of POU/POE treatment	1	2	0	0	1	2	1	1	8
Advanced treatment	2	0	0	2	1	2	0	2	9
Alternative water source	1	0	0	2	1	2	1	2	9
High quality water storage and recovery	1	0	0	2	1	2	1	2	9
Policy to promote use of alternative disinfectants	1	2	0	1	1	1	1	2	9
Alternative disinfectants	2	1	1	1	1	1	1	2	10
Regionalization	1	1	1	2	1	1	1	2	10
Water treatment plants	2	0	0	2	1	2	1	2	10
Combination of corrective measures	2	0	1	2	1	2	1	2	11
Filtration	1	1	1	1	1	2	2	2	11
Relocate chlorination system	0	2	1	1	1	2	2	2	11
Point of use/entry treatment	2	2	2	0	1	2	1	1	11
System maintenance	1	2	2	0	1	2	1	2	11
Policy to promote PWDU	1	2	0	2	1	2	2	2	12
Training	1	2	0	1	2	2	2	2	12
Watershed protection	0	2	1	2	1	2	2	2	12
Improved design of systems	1	2	0	2	2	2	2	2	13
Potable water dispensing unit	2	2	1	2	1	2	2	2	14
Upgrade distribution network	2	1	1	2	2	2	2	2	14

9.0 Cartwright Water Distribution System Model

The Cartwright water distribution system is typical of many small towns in Newfoundland & Labrador– a long linear system with a surface water supply whose raw water displays high colour. The Cartwright system is gravity fed, and in addition to supplying water to the community, also supplies the local fish plant operated by Labrador Shrimp Co. Ltd. The fish plant is typically in operation from June to Oct, and uses chlorinated municipal water for washing fish and making ice. The system receives primary screening at the source, chlorine disinfection and pH adjustment. There is a main powder hypo-chlorination system near the source, and a new satellite powder hypo-chlorination system was installed in the middle of the distribution system in July of 2004. Maintaining chlorine residuals throughout Cartwright's long linear system, particularly at dead ends, is especially problematic during times when the fish plant is not operating. The Cartwright distribution system can be classified as small and from the Labrador Region of the province.

Figure 105: Cartwright towards the end of the water distribution system (fish plant- middle foreground)

Descriptive data for the Cartwright water distribution system is detailed in following sections. This data was then input into the Cartwright EPANET hydraulic/water quality model. The next step involved calibrating the Cartwright model with system data also highlighted in the following sections. Different corrective measures and modeling scenarios were then selected based on observed problems with how the distribution system is currently operating. The potential effectiveness of the given solution or modeled scenario was then weighted against solution criteria and constraints.

9.1 Reservoir

The water supply for the town of Cartwright is Burdett's Pond, located approximately half a kilometre south of town off of the Airport Road. Burdett's Pond has a catchment area of 12.9 km^2, reservoir storage of 246, 052 m^3, and mean monthly runoff of 757, 082 m^3. The intake is a 350 mm pipe located approximately 45 m into the pond. Primary screening and a wet well exist at the very beginning of the distribution system to help deal with solids, turbidity and colour problems. A dam at the northeastern shore of the Burdett Pond system, approximately a kilometer away from the intake helps to maintain water levels. Significant amounts of vegetation were flooded during the time of construction of the holding dam. The reservoir is at an elevation of 66.5 m. Average

DOC levels for Burlington are in the 3rd quartile or the highest 25% of average DOC values in source waters in the province.

Table 80: Average source water quality values for Cartwright

Water Quality Parameters	Average Values 1992-2006
Colour (TCU)	96.7
pH	5.59
Turbidity (NTU)	0.89
Bromide (mg/L)	0.01
Chloride (mg/L)	4.57
DOC (mg/L)	9.13
Temp (°C)	12.5
Iron (mg/L)	0.44
Manganese (mg/L)	0.0089

Figure 106: Burdett's Pond

9.2 Pipes

The pipes in the current Cartwright distribution system were installed over 7 phases (to date) beginning in 1984, with some older metal pipes still on the network. All mains in the system are composed of high-density polyethylene pipe (HDPE). The intake pipe is 350 mm and the major trunk main that extends to almost the end of the community is 200 mm, reducing to150 mm, 100 mm, 75 mm and 50 mm for various lateral mains. In total there is over 7 km of trunk main laid down in the Cartwright distribution system.

The Hazen-Williams head loss formula was selected for this model in order to determine energy losses throughout the system. Roughness factors were selected for the pipes based on pipe age: 150 for HDPE pipes laid prior to 1984, 155 for HDPE pipes laid after 1984, and 130 for very old DI pipe (assumed).

The maximum estimated distribution system operating pressure is given as 90 PSI or 63.3 m. In addition, there are at least 5 fire hydrants located at different points on the distribution system.

9.3 Demand

The Cartwright distribution system is metered near the source in the chlorination building located just before the distribution main starts following Airport Road into town.

According to most recent information, average flow (taken from meter readings) into the Cartwright distribution system is 504 m^3/d or 5.83 L/s. Typical instantaneous flow readings observed in the morning by the system operator range from 5.05-7.57 L/s. Types of water users and their number are summarized in the following table.

Table 81: Type and number of water users on the Cartwright network

Type of Water User	Number
Residential	217
Fish Plant	1
School	1
Medical Institution	1
Industry (other than Fish Plant)	2
Commercial (one hotel)	14

Non-residential water usage in Cartwright is estimated at 10 %. Subtracting this portion from the total average demand and based on a census population in 2001 of 629 people, average daily residential water demand is 721 L/person/day. The 10 % of total demand that is non-residential equates to 50.4 m3/d or 0.583 L/s.

Residential demand was allocated to 26 different junctions throughout the distribution network based on housing density surrounding that junction. Non-residential demand was allotted to 3 different junctions to account for fish plant and hotel water usage. Institutional and commercial demand was equated to an equivalent number of residential properties.

Based on fish plant water use statistics from 1986, total annual freshwater use for 219 fish plants in the province was 4,920,000 m^3/yr (1993, DOEL). This works out to 61.6 m^3/d or 0.71 L/s for each fish plant. Latest fish plant water use from Cartwright is roughly 4416.82 m^3/season, with the 2007 season only lasting approximately two months from June to August. This equates to 71.2 m^3/d or 0.82 L/s. The fish plant typically operates from 7 am to 7 pm during the season. Fish plant demand was included in the calibrated model.

Information gathered from earlier sources sets the available yield of the system at 25.2 L/s and total average demand on the system at 7.89 L/s plus fish plant consumption. Based on these numbers, per capita consumption is 1084 L/p/d.

After reviewing all of the demand information, an average base demand of 5.83 L/s was selected with a fish plant demand of 0.82 L/s for input into the model

Elevation of junctions with assigned demands ranged from 20 m (hotel- first user on system) to 0.6 m above sea level.

Meter readings have not been taken at a frequency to establish a daily demand pattern for the Cartwright distribution system. Peaks in the morning, noon and evening for domestic users are typical however. The following two generic demand patterns were used in the Cartwright model- one for domestic water use, and one for fish plant demand.

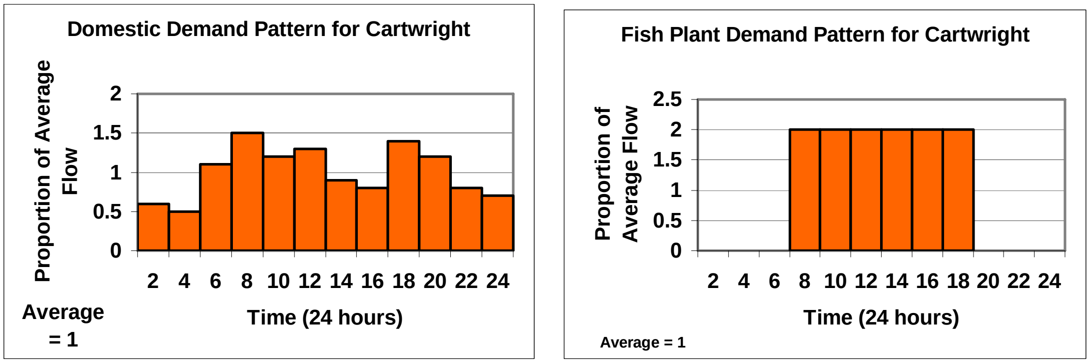


Figure 107: Typical domestic and fish plant demand patterns

9.4 Chlorine Decay

As of July 2004, the Cartwright water distribution network receives disinfection from a hypo-chlorination system located near the intake (node 2), coupled with a hypo-chlorination booster (node 7) midway along the distribution network. Both chlorination systems use a powdered high test calcium hypochlorite (HTH) powder mixed with water to form a chlorine solution. The main chlorination system has a 190 L tank and chlorine pump that can deliver up to 13 L/h of chlorine solution. The main chlorinator is flow controlled, accepting pulse input from the water meter for operation.

Based on the amount of HTH used in Cartwright (approximately 9 L HTH per 190 L of water), the percent dilution of the chlorine solution is 2.6%. The chlorine dose at the main chlorinator near the intake is 4.9 mg/L, while the chlorine dose at the satellite booster station is 4.1 mg/L.

According to gathered information, there are difficulties in maintaining adequate chlorine residuals at dead ends on the system. Based on an average daily flow of 504 m^3/d, the available contact time at the first point of use is 39 minutes. The contact time for peak flows using the Harmon Formula is 27 minutes (a minimum of 20 minutes is required). Special attention will have to be made to contact times during periods of high demand when the fish plant is operational.

In lieu of a bulk chlorine decay test, a typical value for the bulk chlorine decay coefficient was selected based on results from other decay tests on provincial surface water supplies. A bulk decay coefficient of –0.5 d^{-1} was selected for the Cartwright model. A default wall decay coefficient of –1 m/day was also selected.

9.5 Chlorine and THM Data Gathering

Chlorine tests are regularly made by the Cartwright System Operator and by Department of Environment staff. The following table summarizes average chlorine, total THM, and BDCM results. At the first point of use, chlorine readings are above the maximum value portable Hach Chlorine Test Kits can read. A value of 4.0 mg/L will be used for analysis purposes. Towards the middle and end of the system, chlorine readings drop significantly. Only one sample was ever taken from the beginning of the system. Data prior to the commissioning of the chlorine booster system was not included in the averages for the middle and end of the system.

Table 82: Average chlorine, THM and BDCM (2000-2007) readings on Cartwright network

Location in Network	Junction	Free Chlorine-DOEC (mg/L)	THM Total-DOEC (ug/L)	BDCM – DOEC (ug/L)
Beginning	3	>2.20 (4.00)	220	2.3
Middle	12	1.18	248	4.1
End	19	0.24	270	3.9

The CCME maximum acceptable concentration (MAC) for total THMs is 100 ug/L. As shown in the table, THM levels in Cartwright are well over the limit.

9.6 Calibration of the Cartwright Model

In order to first calibrate the Cartwright hydraulic/water quality model, results were compared with flow, pressure and chlorine residual data gathered on the Cartwright distribution system. The collection of this data is outlined in previous sections.

Comparison of initial model results to calibration data is described in the following table, along with actions taken to compensate for any discrepancies, and final associated percentage errors found in the calibrated model. Average values from the model are taken for comparison once equilibrium or periodic behaviour from that parameter had been reached.

Table 83: Calibration of Cartwright model

Issue	Percentage Error	Action	Percentage Error After Calibration
-average daily model flow of 6.24 L/s (daily range of 2.91-9.56 L/s) vs. average flow of 5.83 L/s	-7.0%	None	
- node 9 model pressure ranges from 63.3-65.7m vs. max estimated system operating pressure of 63.3m	-1.9%	None	
-node 25 model pressure ranges from 63.3-65.8m vs. max estimated system operating pressure of	-2.0%	None	

63.3m			
-node 27 model pressure ranges from 63.1-65.1m vs. max estimated system operating pressure of 63.3m	-1.3%	None	
-node 3 equilibrium (after 7hr) Cl of 3.75 mg/L vs. 4.0 mg/L	-6.3%	-increase bulk reaction rate from -0.5 to -0.8 -increased wall reaction rate from –1 to –2.0	-13.1% (3.48mg/L)
-node 12 equilibrium (after 12hr) Cl of 1.95 mg/L vs. 1.18 mg/L	-65.3%	-increase bulk reaction rate from -0.5 to -0.8 -increased wall reaction rate from –1 to –2.0	-19.9% (1.42mg/L)
-node 19 equilibrium (after 25hr) Cl of 0.41 mg/L vs. observed average of 0.24 mg/L	-70.8%	-increase bulk reaction rate from -0.5 to -0.8 -increased wall reaction rate from –1 to –2.0	-8.3% (0.22mg/L)

The calibration data set for Cartwright only covered basic elements, resulting in a rough calibration. Once results predicted by the model were felt to adequately reflect observed field data– matching pressures, flows, chlorine residuals– through the adjustment of certain network parameters, a baseline model was established. The different model scenarios will then be run on this baseline model, adjusting only selected network parameters.

The following graph shows mean observed verses mean simulated values of pressure for the Cartwright system. As can be seen in the graph below, actual and modeled pressures match almost exactly.

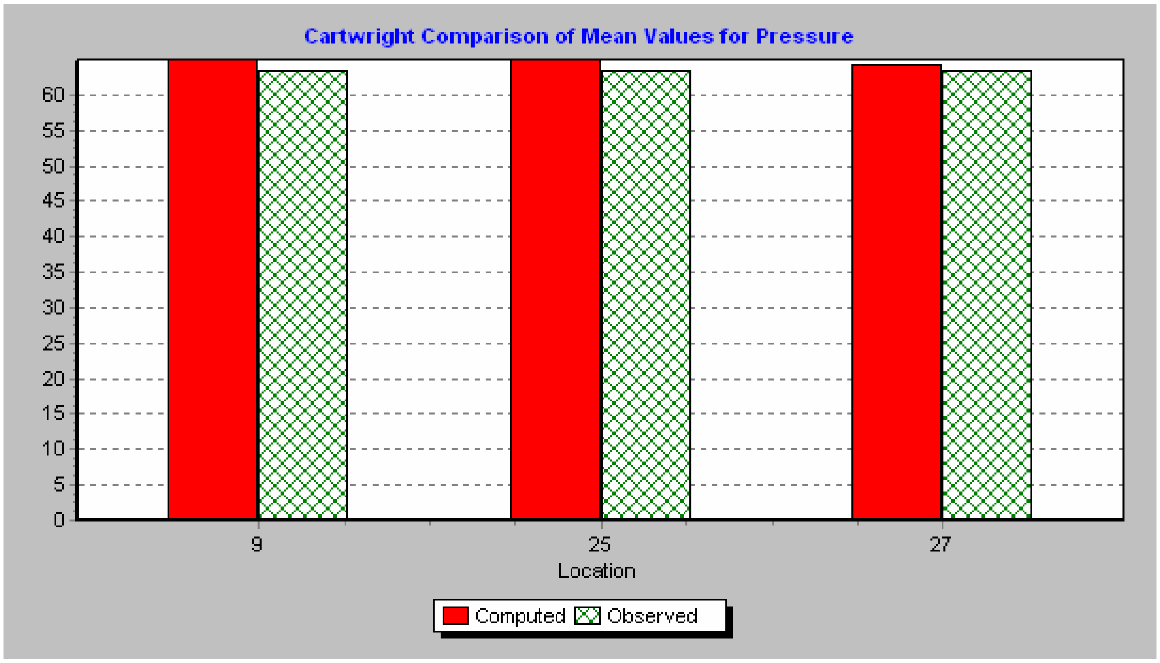


Figure 108: Mean observed and mean simulated value for pressure in Cartwright

The following graph shows flow leaving the reservoir over the 7-day simulation period. The instantaneous flow range of 5.1- 7.6 L/s observed in the field is within the range simulated by the model.

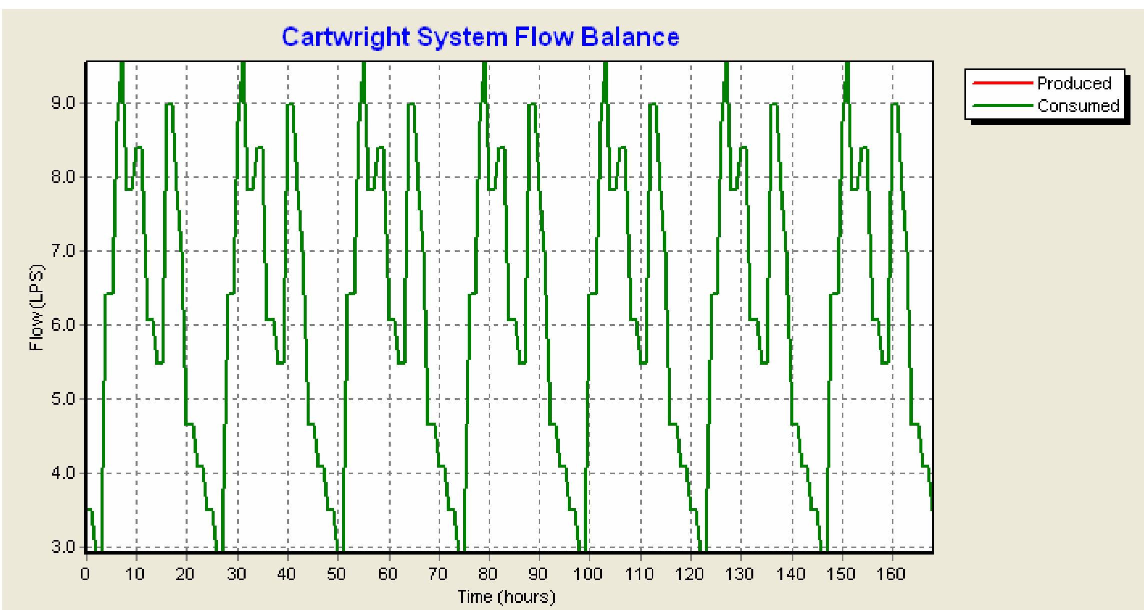


Figure 109: Cartwright system water demand

The following table and graph show calibration statistics for free chlorine residuals taken from three different points in the Cartwright distribution system. Observed chlorine readings taken from the field were assigned times after equilibiram had been reached for each node. Once chlorine reached equilibrium, it still varied significantly with changes in system demand. A median point along this chlorine pulse cycle was used to compare simulated to observed results. There was little error observed between field and modeled chlorine residuals indicating a near perfect correlation.

Table 84: Cartwright calibration statistics for chlorine

Location	Num Obs	Obs Mean	Comp Mean	Mean Error	RMS Error
3	1	4.00	3.66	0.342	0.342
12	1	1.18	1.13	0.051	0.051
19	1	0.24	0.20	0.039	0.039
	3	1.81	1.66	0.144	0.201

Correlation Between Means: 1.000

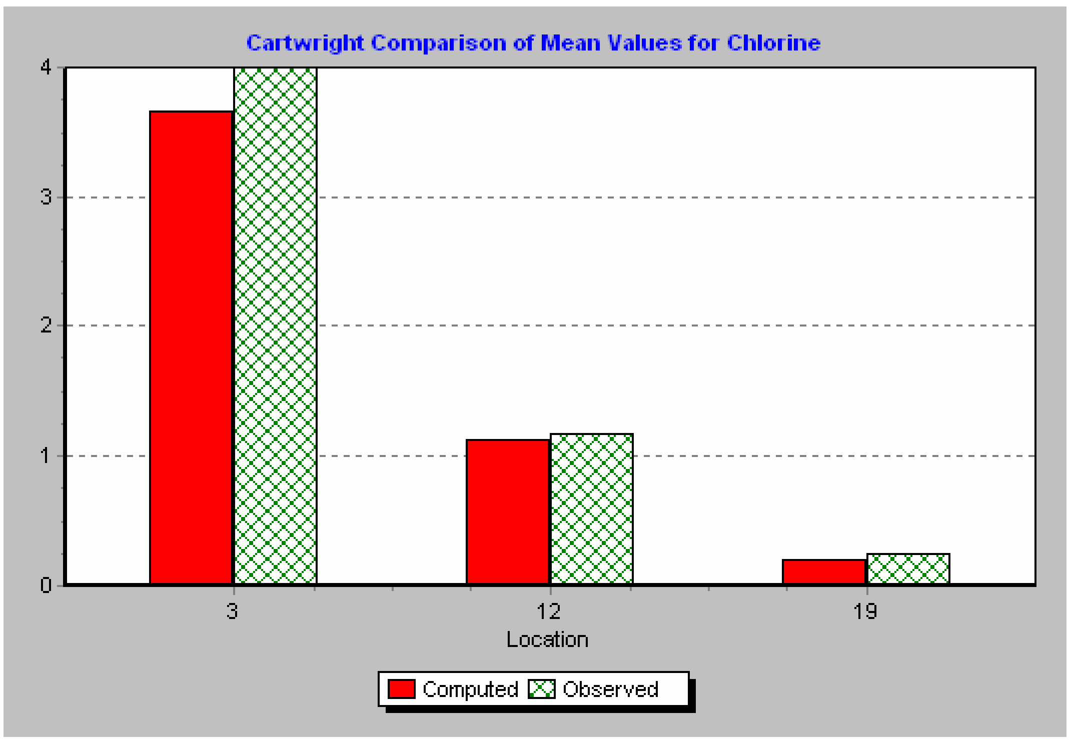


Figure 110: Mean observed and mean simulated value for chlorine residuals in Cartwright

9.7 Problems with the Cartwright Distribution System and Appropriate Corrective Measures

By gathering detailed background information on the Cartwright water distribution system and establishing a calibrated baseline model, we were able to identify problems with how the system operates normally. Several contributing factors were identified as contributing to the overall Cartwright THM problem as outlined in the following table.

Table 85: Problems contributing to high THMs in the Cartwright distribution system

	Causative Factors	Quantitative Value
1	Reservoir contains flooded vegetation	Yes
5	High DOC in source water	9.13 mg/L

7	High chlorine dose	4.82 mg/L max after booster
9	Higher chlorine use with booster system	yes
10	Excessive chlorine demand	-0.8 d-1 (bulk) -2.0 m/d (wall)
12	Long linear system	5.9 km intake to end total = 10.6 km
13	Branched system with multiple dead ends	at least 7 DE
15	Insufficient chlorination controls on system	flow proportional
16	System is oversized	0.01-0.30 m/s 200-75 mm Q_{avg} = 5.83 L/s
18	Pipe material and age	>25 years
20	Large occasional demand on system	yes
26	Poor O&M of system	Water Dist- Class I
27	Multiple factors	-
28	Poor design of system	-
29	High iron and manganese	Fe = 0.44 mg/L
30	High per capita demand	721 L/p/d
32	Problems with chlorine residuals	0.04 mg/L @ end 4.82 mg/L after booster

The following figures illustrate some of the problems observed in the Cartwright distribution system.

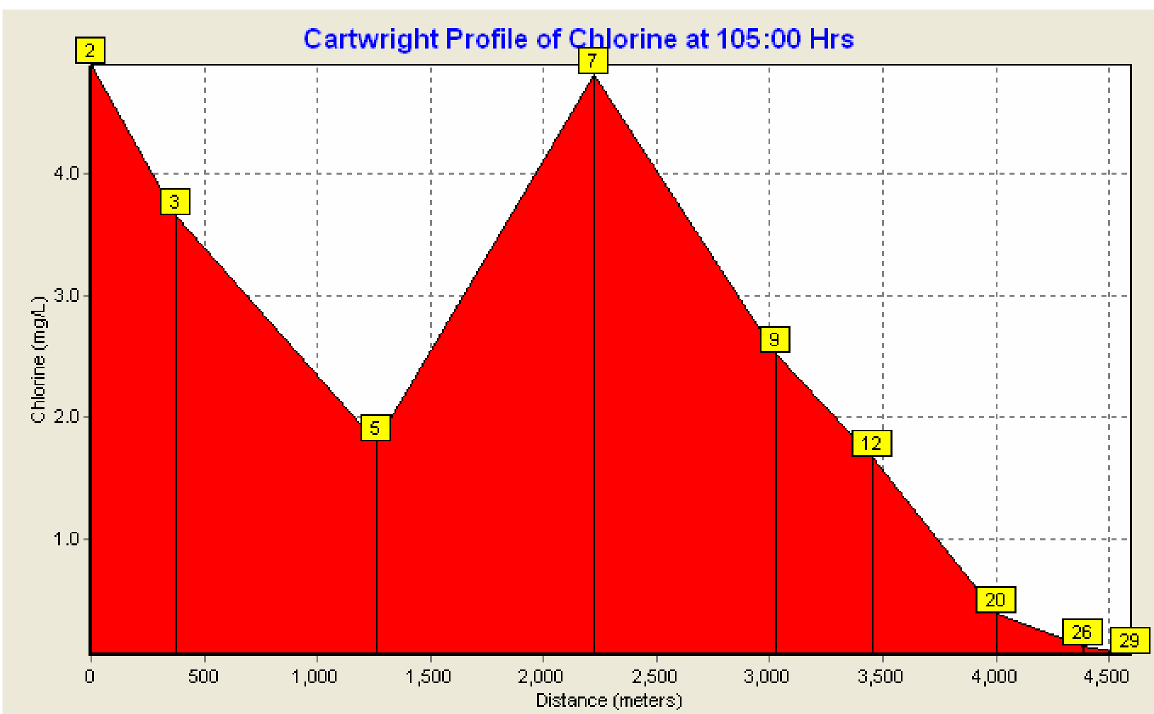


Figure 111: Chlorine decay profile through Cartwright distribution system

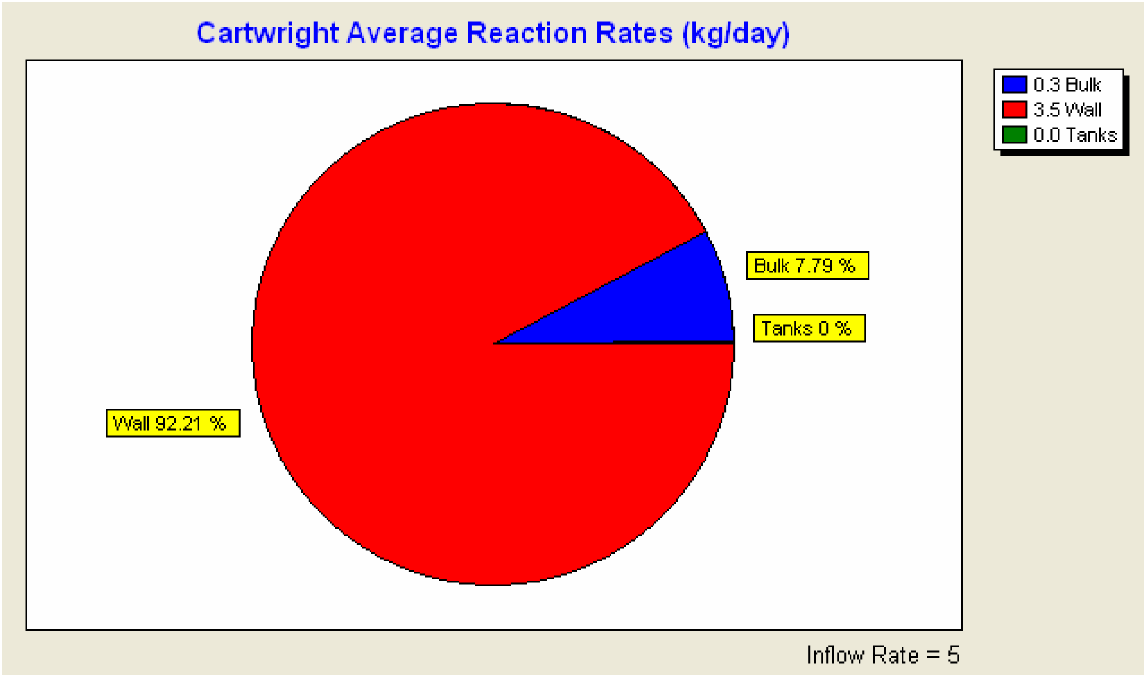


Figure 112: Chlorine decay contributions in Cartwright distribution system

Solutions that might address the probable causes of high THM levels in the Cartwright distribution system are outlined in the following table. Those corrective measures highlighted in grey are the only solutions that can potentially be modeled.

Table 86: Applicable THM corrective measures for Cartwright

Applicable Corrective Measures	Probable Causes Addressed
Policy of POU/POE treatment	All
Policy to promote use of alternative disinfectants	All
Policy to promote PWDU	All
Watershed protection	All
High quality water storage and recovery	All
Regionalization	All
Alternative disinfectants	All
System maintenance	All
Potable water dispensing unit	All
Point of use/entry treatment	All
Training	All
Alternative water sources	1-5
Remove submerged vegetation	1-5
Optimize disinfectant dosage	1-5-7-9
Optimize location of chlorine booster	7-10
Chlorine dose control	1-5-7-9
Regular system flushing at dead ends	1-5-12-13-16-20
Continuously bleed system at dead end	1-5-12-13-16-20
Downsizing mains	1-5-12-13-16
Replace or reline pipe	18

Loop distribution network	13
Water treatment plants	5-7-9
Filtration	5-7-9
Iron and manganese removal	1-7-10-29
Advanced treatment	3-5-6
Improved design of system	28
Combination of corrective measures	All

9.8 Results from the Cartwright Modeling

The next step was to model the different selected corrective measures and see how the Cartwright distribution system responded. Given the ability of the baseline model to reflect current conditions accurately, a reasonable degree of confidence can be placed in the scenario results.

9.8.1 Optimize Chlorine Dosage

The Cartwright distribution network has a main hypo-chlorination system located in the chlorination building just off Airport Road on the Burdett’s Pond access road. The satellite hypo-chlorination building was located 2.9 km downpipe of the main chlorination system. The main and booster chlorination system dosages are calculated to be 4.9 mg/L and 4.1 mg/L respectively. Both chlorination systems vary their dosage with flow, so dosages are based on average flow. Primary disinfection requirements for the system are met (contact time, CT value), but there is potential for reducing the initial chlorine dosage. Secondary disinfection requirements are borderline (ie. at maintaining a free chlorine residual throughout the network). Chlorine residuals after the booster chlorination system exceed 4.0 mg/L, indicating the dosage should be reduced. Chlorine dosages in the province typically range from 5-15 mg/L.

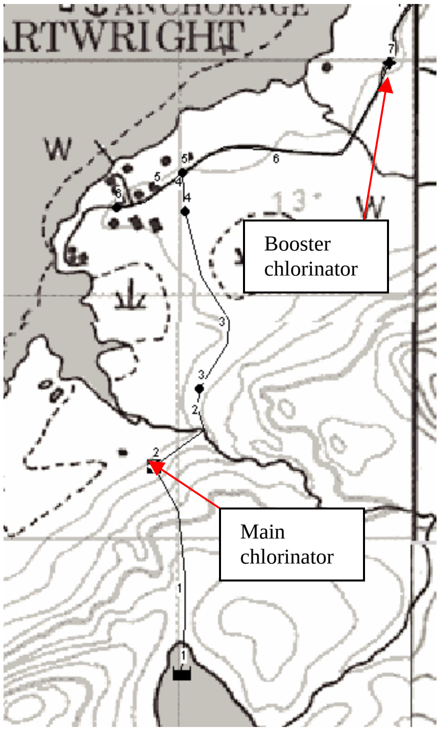


Figure 113: Location of main and booster chlorinator for Cartwright

In EPANET we have chosen to model chlorine as a setpoint booster at nodes 2 and 7, which fixes the concentration of any flow leaving that node. As stated in the objectives, the Cartwright system should have a 20 min contact time, contain a free chlorine residual of at least 0.3 mg/L at the first point of use (or equivalent CT value), and maintain a free chlorine residual of 0.05-0.10 mg/L at the end of the distribution system. The following table summarizes the results of altering chlorine dosage.

Table 87: Altering chlorine dosage in Cartwright distribution system

Initial Chlorine Dose/ Booster Dose (mg/L)	CT Value at 1st User	Min Cl Residual at Start of System – node 3 (mg/L)	Max Cl Residual at Start of System – node 3 (mg/L)	Cl Residual before Booster – node 7 (mg/L)	Max Cl Residual after Booster – node 7 (mg/L)	Min Cl Residual at End of System – node 29 (mg/L)
4.9/4.1	85.4	3.32	3.75	0.37-0.72	4.82	0.04
3.0/3.0	52.6	1.97	2.30	0.23-0.43	3.44	0.03
2.0/2.0	34.7	1.30	1.53	0.14-0.29	2.29	0.02
2.0/3.5	34.7	1.30	1.53	0.14-0.29	3.79	0.04
0.7/4.0	12.3	0.46	0.54	0.05-0.10	4.10	0.04

No combination of initial and booster chlorine dose examined met all criteria, as chlorine residuals at the very end of the system were always just shy of objective values. If a chlorine residual of 0.04 mg/L is deemed acceptable, there is potential to almost halve the total chlorine dose.

9.8.2 Optimize Chlorine Booster Location

A chlorine booster is a secondary chlorination system located on a water distribution system to boost chlorine residuals to appropriate levels in areas where they may have fallen below a set objective. The satellite chlorination system that was commissioned in 2004 is located midway along the system at node 7. At this location, there still appears to be difficulties in maintaining adequate free chlorine residuals at the very end of the system.

As an alternative, the satellite chlorination station was placed closer to the main concentration of water users toward the end of the system at node 9 (3.7 km from primary chlorination system). With an initial chlorine dosage of 2.5 mg/L, the minimum equilibrium chlorine residual just before node 9 is 0.08 mg/L, within our secondary disinfection criteria range of 0.05-0.10 mg/L. Chlorine residuals at the first point of use range from 1.64-1.91 mg/L, with a CT value of 43.8, thus primary disinfection requirements are also met. A booster chlorination dose of 2.5 mg/L leaving node 9 is sufficient to provide a minimum chlorine residual of 0.05 mg/L at the end of the system, which meets secondary disinfection criteria.

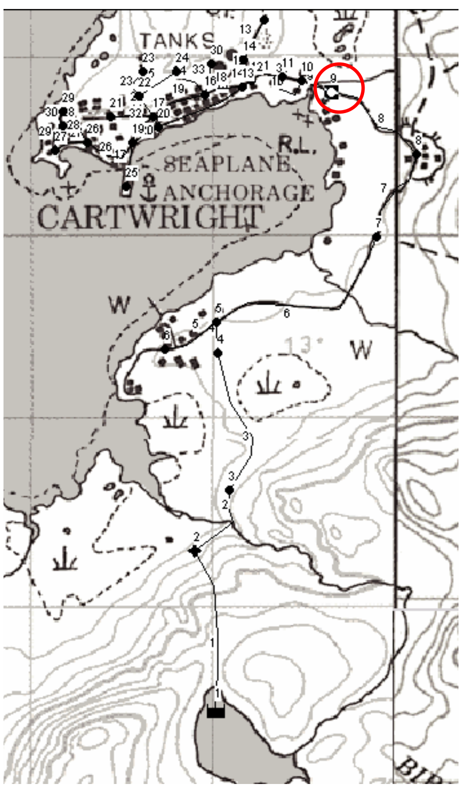


Figure 114: Optimal location for chlorine booster in Cartwright

The combined chlorine dose with the booster chlorination system location optimized is almost half that of the current total dosage. A primary dosage of 2.5 mg/L and booster dosage of 2.5 mg/L at node 9 provide adequate system results while minimizing chlorine usage, which will in turn reduce potential THM formation.

9.8.3 Chlorine Dosage Control

The main and booster chlorination systems in Cartwright are both flow controlled, meaning the rate of chlorine solution injected into the water distribution system alters proportionally with flow. For calibration purposes, chlorine dosage was modeled as a constant dose as no information was available on typical fluctuations of the chlorine control. Water quantity (flow) and/or quality (chlorine residual) feedback controls can be used to manage the chlorine feed.

The chlorine dose was made to vary with time using two time patterns: one the same as that used for water demand, the other opposite to that used for water demand. Feedback control on chlorination systems typically function by increasing the chlorine dose when flows increase in order to maintain CT values at the first point of use. However, when demand is high, water moves at an increased rate through the distribution system, resulting in reduced water age, less time for chlorine decay, and higher chlorine residuals. The variation in chlorine will mimic the peaks and lows of flow throughout the system (for chlorinators that are flow controlled), only the lag time between peaks in flow and peaks in chlorine residuals will increase the further you get towards the end of the distribution system.

The following three graphs look at the variation in chlorine readings at three different points in the network if the chlorine dose is constant, increases with flow, decreases with flow. Variation in chlorine residuals increases significantly with flow control.

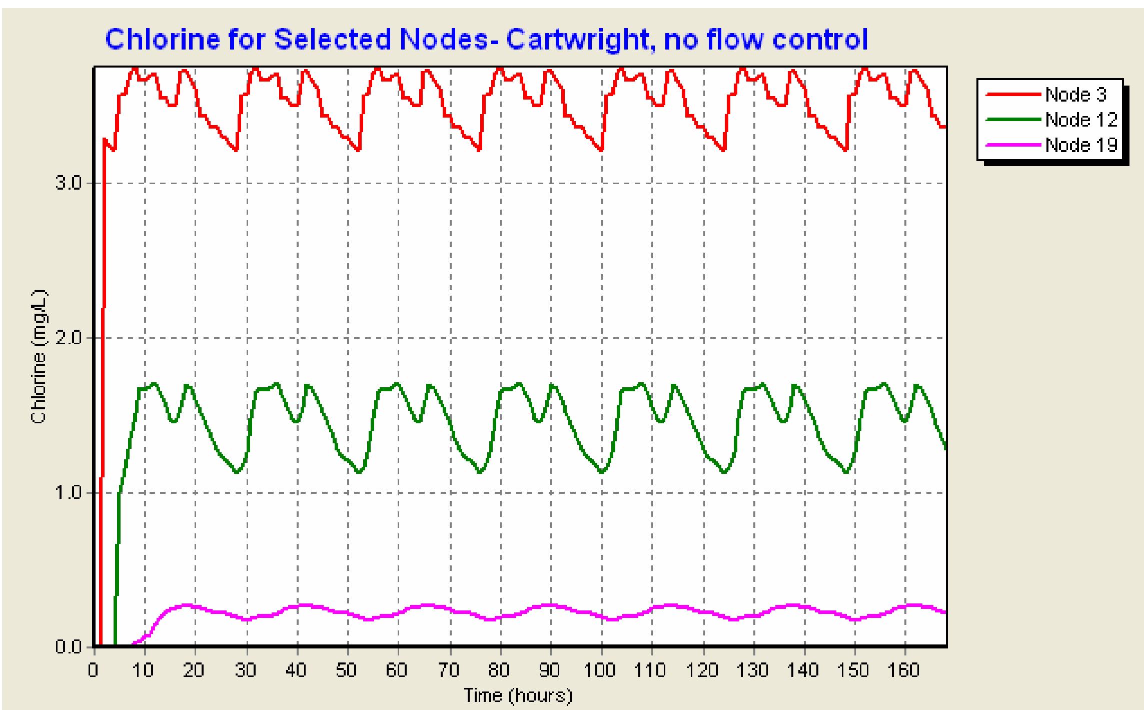


Figure 115: Chlorine levels with constant dosage, Cartwright

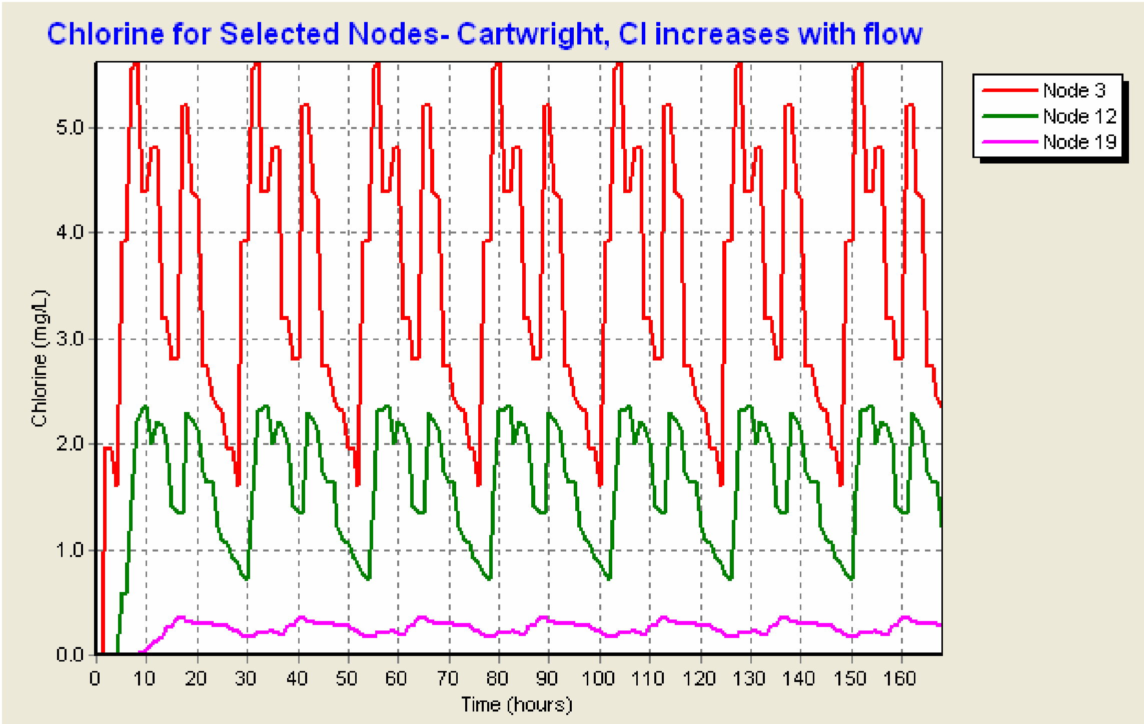


Figure 116: Chlorine levels with chlorine control proportional to flow, Cartwright

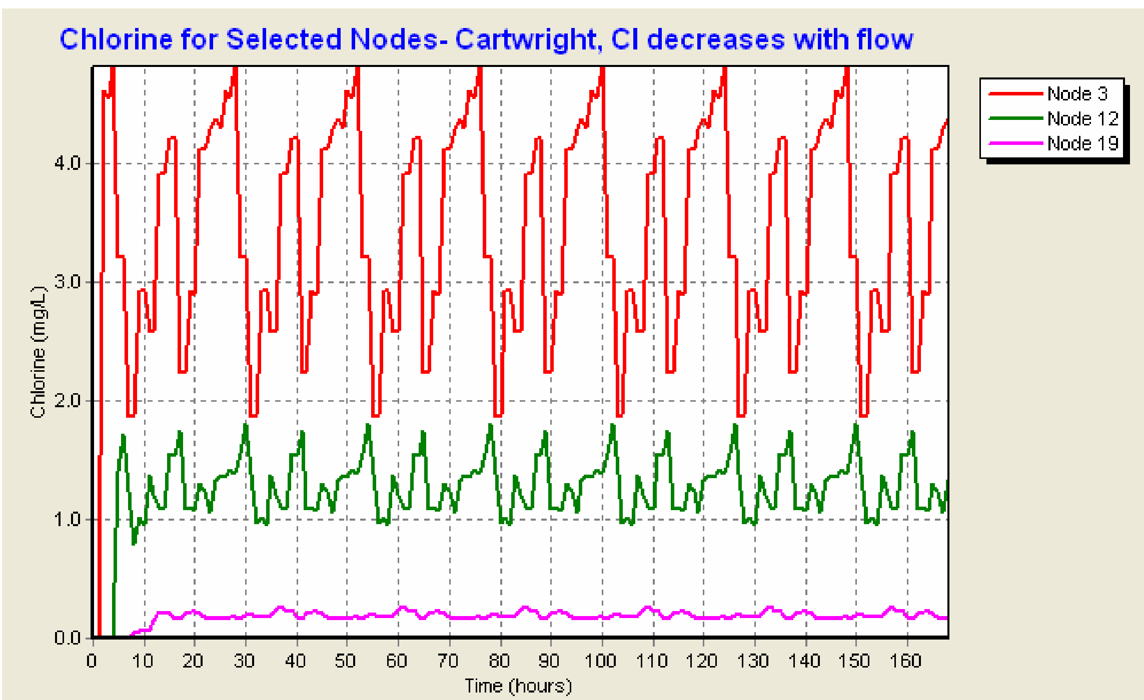


Figure 117: Chlorine levels with chlorine control inversely proportional to flow, Cartwright

In the case of Cartwright, the lag time between the peak in flow and the corresponding peak in chlorine residual at the end of the system is 18 hours, indicating the difficulty in trying to optimize chlorine through flow control. With flow control, extremes in chlorine residual values also exceeded the maximum criteria value of 4.0 mg/L, and there was no option to reduce the overall chlorine dosage. There are complicating factors involved with chlorine dosage control that make it effective only in parts of the distribution system whether flow or residual controlled.

9.8.4 Regular System Flushing/ Continuous Bleed at System Ends

The Cartwright distribution system was designed to accommodate the water demands of the local fish plant. Fish plant demand is modeled as a demand for 0.82 L/s for 12 hours every day. Without fish plant demand water movement in the distribution system is slower, providing more time for chlorine in the water to decay, and conversely, a higher contact time at the first point of use. The maximum retention time in the Cartwright water distribution system is 24 hours while the fish plant is operational. Without the fish plant, maximum retention time increases to 27 hours. Any flushing program, therefore, must occur at a time period of less than 24 hours in order to achieve any improvement in water age, ideally at half the current return period or every 12 hours. For this corrective measure, two scenarios were looked at: flushing twice a day at each of the six dead ends, and continuous bleeding of selected dead ends (nodes 6, 15, 30, 23, 29).

The average daily flow rate (demand) in the network is 5.83 L/s, and flushing rates will be some multiple of this. Pressure throughout the distribution system is within guideline range at current demand levels. Negative system pressures are experienced at approximately 2.3 times the average daily flow rate. Even at the maximum flushing rate the system is capable of, it is impossible to reach a flushing velocity of 0.75 m/s.

For the scenario where flushing occurs twice a day at each of the six dead ends, base demand at each dead end node was increased by a factor of 5 resulting in an additional 4.6 L/s instantaneous demand on the entire system for 4 hours at 12 hour intervals. Maximum water age at dead end nodes was reduced from between 1.7 to 3 hours. Chlorine residuals improved slightly throughout the distribution network, increasing by 0.02 to 0.08 mg/L depending on the node. There was slightly less variation in chlorine residuals for this scenario.

For the continuous bleed scenario, an additional demand of 1 L/s was placed on nodes 6, 15, 30, 23, 29, effectively doubling demand. With more demand at dead ends of the system, water moves faster through the distribution network. Minimum chlorine readings at the end of the system increase to 0.10 mg/L (from 0.04 mg/L), and there is an increase in chlorine residuals throughout the middle and end portions of the system. Chlorine levels after the booster are above 4 mg/L. Maximum water age is also reduced throughout the system (to 14 hours at end node 29). With the above continuous bleeds on the system, the main chlorine dose can be reduced from 4.9 mg/L to 2.5 mg/L at the initial point of disinfection and from 4.1 mg/L to 2.5 mg/L at the booster, and adequate chlorine residuals still be maintained throughout the network. Contact time requirements at peak flow are not met; however, the CT value is adequate.

Manual flushing once a day (or more) at multiple dead ends in the Cartwright distribution system may not be a practical use of resources; however, use of automatic hydrant flushing units could help. A flushing program with a flushing frequency of more than once a day is also not practical. Continuously bleeding a system is wasteful of resources (water, energy costs, chlorine) and may harm the receiving environment. The benefits of each of the option must be examined in the context of its various disadvantages in the case of flushing corrective measures.

9.8.5 Replacing or Relining Pipe/Downsizing Mains

The Cartwright distribution network is oversized for the demand placed on the system, particularly when the fish plant is not in operation. Pipe sizes range from 200-75 mm with the majority of pipe in the network sized at 150 mm or greater in order to fit fire hydrants. The maximum observed water velocity in the system is 0.30 m/s, observed in the section of pipe leading from the intake.

The pipe in the Cartwright distribution network ranges in age with some sections installed prior to 1984. For the first scenario, all pipes in the network were modeled as brand new, reflected in the input pipe roughness coefficient value. All pipes were given a Hazen-Williams C value of 155 for new HDPE pipe. The model results indicated a very slight improvement in chlorine residuals towards the end of the network (increase of 0.01 mg/L at end node 29). Chlorine residuals after the booster exceeded 4.0 mg/L.

For the second scenario, each pipe was resized so as to achieve a peak velocity of approximately 0.4 m/s or a minimum pipe size of 40 mm. Under these criteria, pipe sizes in the Cartwright distribution system now range from 40-175 mm. The resulting maximum water age in the system is now 10 hours (reduced from 24 hours). Pressures throughout the network have decreased slightly with minimum pressure in the system down from 45.8 m to 37.7 m, but still within acceptable range. Chlorine readings at the end of the system reach 0 mg/L and are lower throughout the entire system. With reduced pipe diameter we end up with a new contact time of 20 minutes at peak flow, and an equivalent CT value of 64.

The rate of reaction of chlorine at the pipe wall is inversely related to pipe diameter, so the smaller the pipe diameter, the greater the pipe wall reaction rate and the greater the amount of chlorine consumed at the pipe wall. Even through there is a significant decrease in water age throughout the system, the overall chlorine dosage would have to be increased in order to achieve adequate residuals at the end of the system, which could potentially offset any reduction in DBPs.

9.8.6 Reconfiguring the Distribution System through Looping

While a long linear system, there is a fair degree of looping already in the Cartwright distribution system. For this scenario, 4 additional pipes were included in the network to incorporate dead ends into loops, excepting only for the lateral running to the fish plant. With water moving through the distribution system so slowly, particularly along these dead ends, there is plenty of time for chlorine to decay and for DBPs to form.

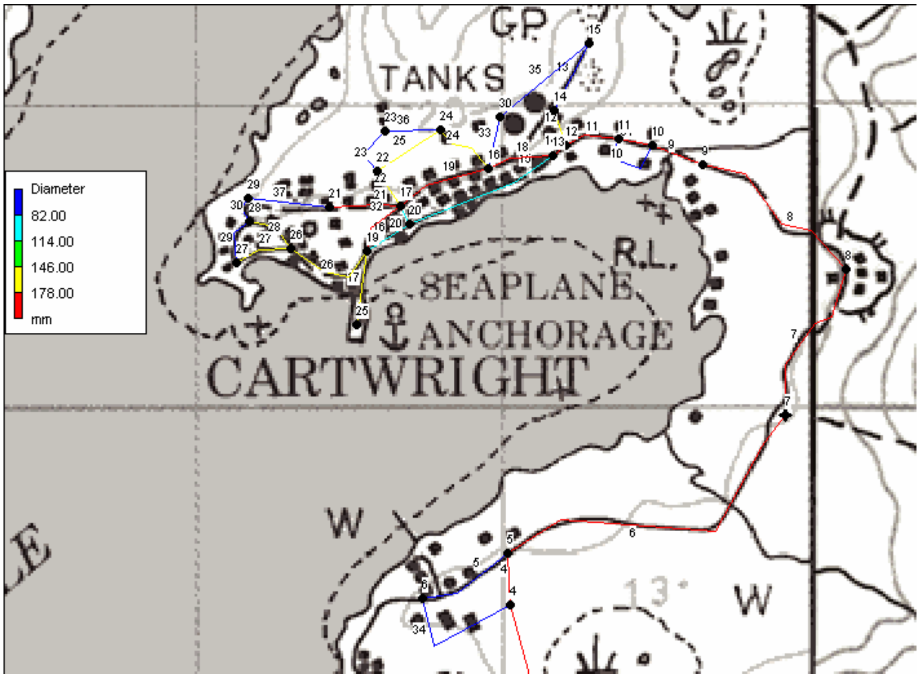


Figure 118: Looping in the Cartwright distribution network

With the system looped, average water age at the very end of the system fell from 24 hours to 22 hours. There was improvement in chlorine residuals throughout the system, increasing from 0.04 to 0.08 mg/L at the end node 29. With this increase in residuals at the end of the system it is possible to lower the booster chlorine dosage to 2.5 mg/L and avoid chlorine levels above 4.0 mg/L.

9.9 Impact of Modeled Corrective Measures

Of the 8 corrective measures identified in a previous section that could be modeled in EPANET in order to access their impact in terms of improving water quality (looking at chlorine, water age, and potential THM formation), two were grouped together with other related scenarios. Not all scenarios met the required criteria in order to be deemed successful. Any scenario that saw a reduction in the overall chlorine dosage and a decrease in water age has potential for lowering THM levels. The following table highlights which scenarios had a positive impact on water quality.

Table 88: Modeled scenarios for the Cartwright network and their effectiveness

	Scenario Description	All Criteria Met	Comments
1	Optimize Chlorine Dosage	No	-Potential to reduce overall Cl dose by 3.5 mg/L -minimum Cl at end of system just below criteria of 0.04 mg/L
2	Optimize Chlorine Booster Location	Yes	-Potential to reduce overall Cl dose by 4.0 mg/L
3	Chlorine Dosage Control	No	- the maximum criteria value for

			Cl of 4.0 mg/L exceeded
4	Regular System Flushing/ Continuously Bleed System at Dead Ends	Yes/No	-Regular flushing slight improvement in water age -Continuous bleeding saw main/booster dose reduced to 2.5/2.5 mg/L, but contact time not met
5	Replacing or Relining Pipe/ Downsizing Mains	No	-Minimal improvement in Cl residuals with replacement/ relining, max Cl of 4.0 mg/L exceeded -With downsizing water age will decrease, but higher Cl dose required to maintain end residual
6	Reconfiguring the Distribution System through Looping	Yes	-Water age decreased slightly -Improvement in Cl residuals allowing for a reduction in booster Cl dose

Any corrective measures that did not meet the necessary criteria should be dropped from consideration and evaluated no further. Scenarios that saw potential for the overall chlorine use to be reduced and water age in the distribution system lowered will be the most effective in terms of lowering THMs. Based on this assessment, the corrective measures (that met criteria) with the most potential for reducing THM formation are:

- Optimizing the chlorine booster location
- Regular system flushing at dead ends/ continuously bleed system
- Looping the distribution network to eliminate dead ends

9.10 Assessment of Corrective Measure Constraints for Brighton Network

The following table evaluates each remaining corrective measure for the Cartwright water distribution system against identified solution constraints. The selection of the preferred solution(s) to water quality problems can be made based on the corrective measure(s) with the highest score(s).

Based on the resulting scores, there are three main tiers of possible solutions. The top three tiers in the decision matrix scoring system comprise the corrective measures that have the most potential for effectively optimizing chlorine dosage, reducing water age and lowering THMs.

The first tier, which scored 14, consists of installation of a Potable Water Dispensing Unit and looping of the distribution network. The second tier of solutions, which scored 13, consists of the general best management practice of improving system design. The third tier of corrective measures, which scored 13, consists of “soft” solutions such as watershed protection, system operator training, and adaptive policy to promote PWDUs, and the “hard” solution of regular system flushing.

The selection of a preferred solution by the decision making body (town, engineering consultant, Department of Municipal Affaires) can be guided by this decision making framework. The next step in the process involves the implementation of the preferred solution, monitoring and review.

Table 89: Assessment of solution constraints for Cartwright

Applicable Corrective Measures	Effectiveness	Cost	Time Scale for Implementation	Permanency of Solution	Adverse Hydraulic Impacts	Adverse WQ Impacts	Acceptable to Stakeholders	Meets Regulations	Total
Policy of POU/POE treatment	1	2	0	0	1	2	1	1	8
Advanced treatment	2	0	0	2	1	2	0	2	9
Alternative water sources	1	0	0	2	1	2	1	2	9
Combination of corrective measures	2	0	0	2	1	1	1	2	9
High quality water storage and recovery	1	0	0	2	1	2	1	2	9
Policy to promote use of alternative disinfectants	1	2	0	1	1	1	1	2	9
Alternative disinfectants	2	1	1	1	1	1	1	2	10
Regionalization	1	1	1	2	1	1	1	2	10
Water treatment plants	2	0	0	2	1	2	1	2	10
Filtration	1	1	1	1	1	2	2	2	11
Iron and manganese removal	1	1	1	1	1	2	2	2	11
Point of use/entry treatment	2	2	2	0	1	2	1	1	11
Remove submerged vegetation	1	0	1	2	1	2	2	2	11
System maintenance	1	2	2	0	1	2	1	2	11
Install chlorine booster at optimal location	2	2	1	1	1	1	2	2	12
Policy to promote PWDU	1	2	0	2	1	2	2	2	12
Regular system flushing at dead ends	1	2	1	1	2	2	2	1	12
Training	1	2	0	1	2	2	2	2	12
Watershed protection	0	2	1	2	1	2	2	2	12
Improved design of systems	1	2	0	2	2	2	2	2	13
Loop distribution network	2	1	1	2	2	2	2	2	14
Potable water dispensing unit	2	2	1	2	1	2	2	2	14

10.0 St. Paul's Water Distribution System Model

The St. Paul's water distribution system is typical of many small towns in Newfoundland and Labrador– a long linear system with a storage tank. The surface water supply displays high colour and dissolved organic carbon (DOC), both precursors in DBP formation. The system has an infiltration gallery and screening chamber at the beginning of the network on the source intake and in the nearby pump house. The network currently has a flow regulated gas chlorination system located next to the tank. As water inflows into the tank it is dosed with chlorine. Upgrades to St. Paul's water distribution system have been ongoing since 2002 including the construction of the infiltration gallery around the intake and the relocation of the point of chlorination from the pump house to the tank. The majority of the St. Paul's water distribution network was initially laid out in 1978. The distribution network is mostly made up of 250-100 mm ductile iron (DI) pipe. Water from Two Mile Pond is pumped uphill to a storage tank, which then feeds the community. Water levels in the tank control pump activity at the intake.

St. Paul's has had a problem with THM levels being over GCDWQ since THM data was first gathered in 1998. According to town officials, there are major problems with source water quality, in maintaining chlorine residuals towards the end of the distribution system, and with high chlorine at the beginning of the system. Frequent power outages have also interrupted the operation of the chlorination system. The St. Paul's distribution system can be classified as very small and from the Western region of the province.

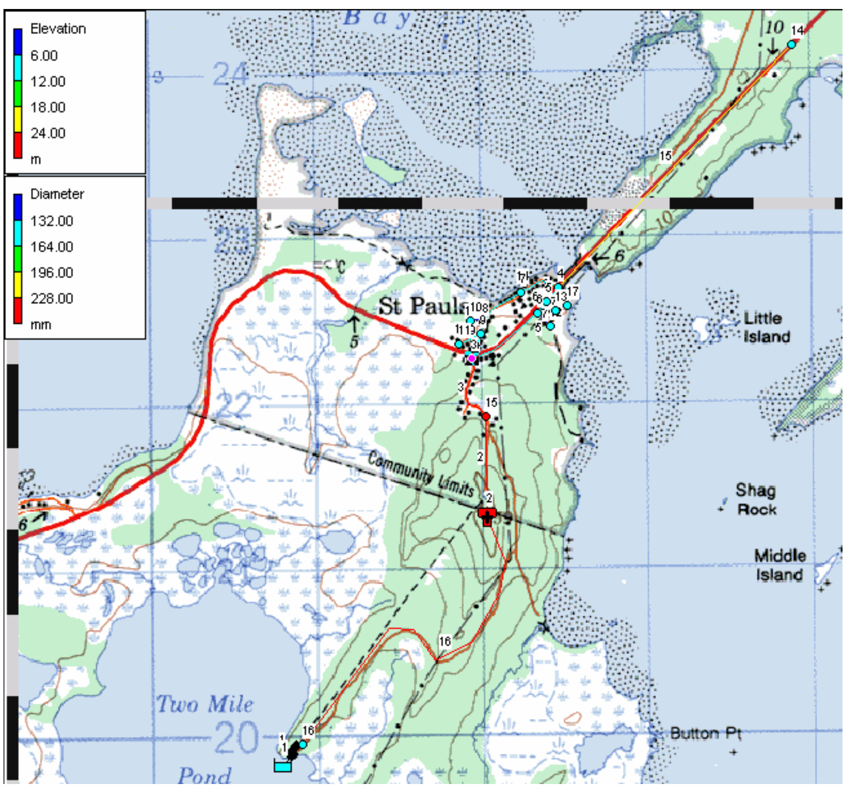


Figure 119: St. Paul's water distribution network

Descriptive data for the St. Paul's water distribution system is detailed in following sections. This data was then input into the St. Paul's EPANET hydraulic/water quality model. The next step involved calibrating the St. Paul's model with system data also highlighted in the following sections. Different corrective measures and modeling scenarios were then selected based on observed problems with how the distribution system is currently operated. The potential effectiveness of the given solution or modeled scenarios was then weighted against solution criteria and constraints.

10.1 Reservoir

The water supply for the town of St. Paul's is Two Mile Pond, located approximately 2 kilometers southwest of town. Two Mile Pond is situated on a costal flat surrounded by bog. The exposed location is only 1.1 km from open ocean. The shallowness of the pond, combined with exposure to strong winds, drives wave action that disturbs bottom sediment and leads to severe turbidity problems. An infiltration gallery of various sized filter material was recently constructed around the intake to try and reduce colour and turbidity problems with the source water, but became plugged after only 6 months. A screen is also located on the end of the intake to deal with large solids. The reservoir has a water level of 9.5 m and the intake extends out 40 m into the pond.

Table 90: Average source water quality values for St. Paul's

Water Quality Parameters	Average Values 1988-2005
Colour (TCU)	81.8
pH	7.3
Turbidity (NTU)	12.9
Bromide (mg/L)	0.03
Chloride (mg/L)	32.6
DOC (mg/L)	6.16
Temp (^{o}C)	19.6
Iron (mg/L)	0.36
Manganese (mg/L)	0.023

10.2 Pumps

There are two 1-1/2 AC pumps operating on the St. Paul's distribution system. They are configured in parallel with only one pump operating at a time in relay. In times of high water demand and low water storage level in the tank, both pumps will be operational. The pumps cut in and out on an automated basis, controlled by the water level in the reservoir through a pressure transducer. If the system is operating normally, the pumps operate 4 hours on and 4 hours off.

The following table displays performance information for the two pumps in use in the St. Paul's distribution system.

Table 91: St. Paul's pump performance information

Pump Type	Power	Rpm	Flow (L/s)	Static Head (m)
1-1/2 AC	5.52 KW	3600	5.36	58.5

10.3 Tank

The St. Paul's storage reservoir is a rectangular underground cement tank configured as in the diagram below with separate inlet and outlet, overflow, and surface chlorination building. When the pump is in operation, water flows into the tank from the inlet pipe located 1.5 m from the bottom of the tank. When the pump is on or off, the tank supplies water to the community directly from the outlet pipe located 0.6 m from the bottom of the tank. It is assumed that water is well mixed within the tank for modeling purposes, however the close location of the inlet and outlet probably creates a short circuit within the tank. The total tank volume (418 m^3) differs from the total potential active tank volume (382 m^3) because of the elevation of the outlet pipe.

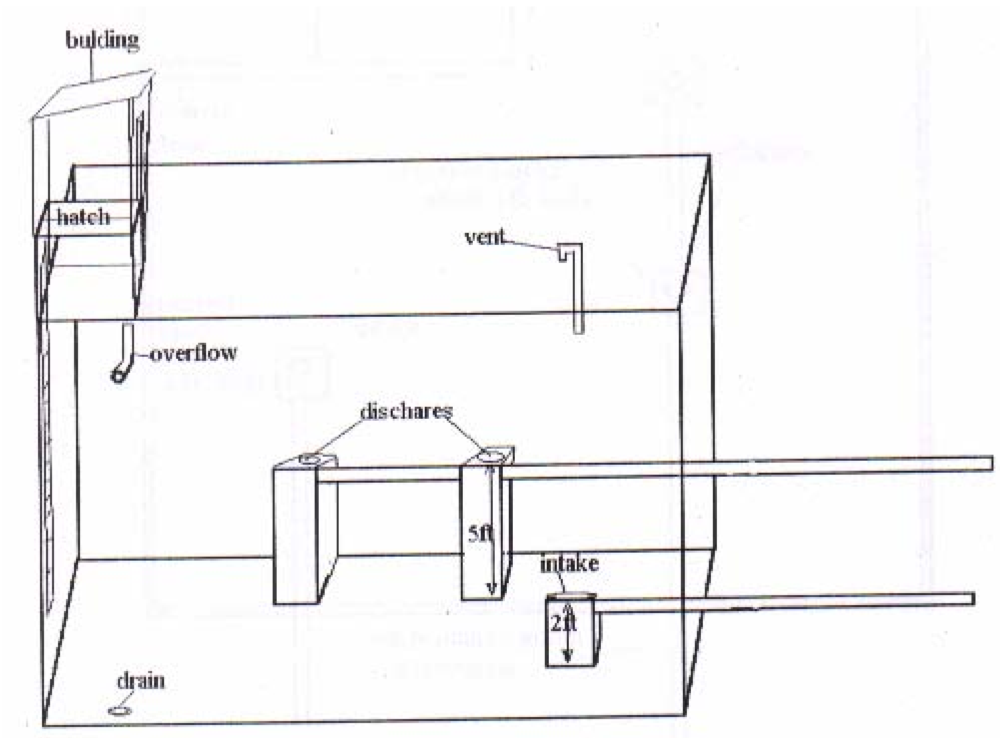


Figure 120: St. Paul's tank configuration

Water levels in the tank direct the operation of the pumps as previously mentioned. An overflow pipe siphons off water from the top of the tank once the tank is full. Exact water levels for tank and pump operation had to be estimated from available information. If the tank control system is operating properly, the maximum water level in the tank reaches approximately 5.2 m and the water level fluctuates approximately 1 m. The tank fills for approximately 4 hours, every 4 hours. The following table provides tank characteristics.

Table 92: St. Paul's tank characteristics

Bottom Elevation	Height	Width	Length	Equivalent Diameter	Volume	Max Water Level	Min Water Level	Active Tank Volume
53.9 m	7 m	4.9 m	12.2 m	8.72 m	418 m^3	5.2 m	4.2 m	14.3%

10.4 Pipes

The majority of pipes in the St. Paul's distribution system were installed in 1978 and are of ductile iron. The intake pipe from the reservoir to the pump house, tank and then Highway 430 is 250 mm in diameter. The trunk main, which loops around the main part of the community, is 150 mm in diameter. The majority of side mains are 100 mm in diameter. The extension of the system across the bridge spanning St. Paul's inlet to Gros Morne Resort is comprised of 200 mm high-density polyethylene (HDPE) pipe. In total there is approximately 7.2 km of pipe laid down in the St. Paul's distribution system.

The Hazen-Williams head loss formula was selected for this model in order to determine energy losses throughout the system. Roughness factors were selected based on pipe age: 155 for newer HDPE, and 90 for older DI.

From information gathered on the system, line pressure is known to range from 207-241 kPa (21.1-24.6m) at the beginning of the system (Fox Road) and 517-552 kPa (52.7-56.2m) in the middle of the system. In addition, there are at least 13 fire hydrants located at different points on the distribution system.

10.5 Demand

The St. Paul's distribution system does have a flow meter located in the pumphouse. Average daily consumption is estimated to range from 189-220 m^3/d or 2.19-2.55 L/s. Average water use during the period from Sept 27 to Dec 1, 2005 was 1.29 L/s. For modeling purposes a demand of 2.55 L/s was used. Types of water users and their number are summarized in the following table.

Table 93: Water users in St. Paul's

Type of Water User	Number
Residential	136
Hotel	1
Institution (Municipal Hall/ Visitors Center, School)	2
Commercial (stores)	2

Residential demand was allocated to 14 different junctions throughout the distribution network based on housing density surrounding that junction. Non-residential demand is not significant on this system and so was equated to an equivalent number of residential properties.

With a population in 2001 of 330 residents, per capita demand is 338-667 L/p/d, based on average demand ranges. It is thought that meter readings are not very accurate at low flows, and thus total flow is underestimated. For the model, higher flow ranges were used (ie. 667 L/p/d). The only demand at the end of the distribution network (node 14) comes from the Gros Morne Resort, a gas bar and an 18-hole golf course. Demand at this location varies seasonally, and is expected to peak during the summer at the height of the tourist season. Off-season and on-season water demands of 0.02 L/s and 1.5 L/s were used respectively for node 14. Calibration was performed using off-season demand;

however, modeled scenarios were run using on-season demand. With on-season demand at the end node, overall average demand increases to 4.03 L/s.

Elevation of junctions ranged from 60 m (at the tank) to 7 m above sea level along the coast.

Meter readings have not been taken at a frequency to establish a daily demand pattern for the St. Paul's distribution system. Peaks in the morning, noon and evening for domestic users are typical however. The following generic demand pattern was used in the St. Paul's model for domestic water use.

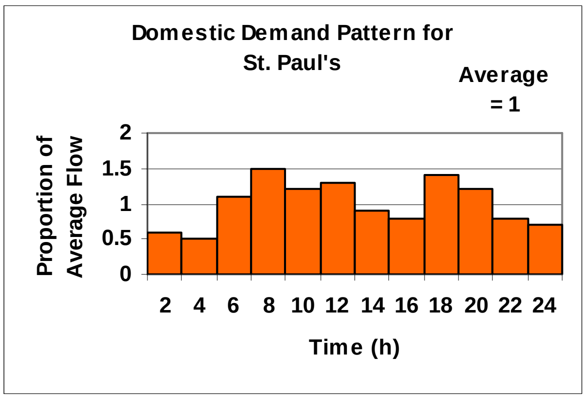


Figure 121: Typical domestic demand pattern

10.6 Chlorine Decay

The St. Paul's water distribution network currently has a gas chlorination system located next to the storage reservoir. This new system became operational in November 2005. Prior to this, chlorine was added to the system at the pump house. The current chlorination system uses two alternating 150 lb chlorine gas tanks typically used over a 3-4 week period. Chlorine is injected directly into the tank and doses at different rates with automated control. The chlorination system is controlled by a chlorine residual analyzer in the storage tank that adjusts the chlorine feed rate so as to maintain a minimum residual in the tank of 1.9 mg/L, and the system was similarly modeled to achieve this. The analyzer measures the chlorine residual every 5 minutes and adjusts the chlorine feed rate automatically. Instantaneous chlorine feed rates are typically around 4 lbs/day. Based on the range of flows and chlorine usage observed in St. Paul's, the chlorine dosage can range from 11 to 17.1 mg/L (5-15 mg/L being typical). For modeling purposes, a chlorine dosage of 12.6 mg/L was used. With the old chlorination system located at the pump house near the intake, the chlorine dosage used to range from 16-48 mg/L.

According to gathered information, there are difficulties in maintaining adequate chlorine residuals at the far end of the system. Based on an average daily flow of 2.55 L/s, the available contact time at the first point of use is 2214 minutes (a minimum of 20 minutes is required). The contact time for peak flow, using the Harmon Formula for peak flow is 545 minutes. Based on these calculations, obtaining an adequate contact time on the St.

Paul's distribution system is not a problem thanks to the retention time provided by the storage tank.

A default bulk chlorine decay coefficient of -1.5 d^{-1} was selected for the St. Paul's model. A default wall decay coefficient of -1 m/day was also selected. A fairly high bulk decay rate was selected due to the high level of colour and organic material in the source water.

10.7 Chlorine and THM Data Gathering

Chlorine tests are regularly made by the St. Paul's System Operator and by Department of Environment and Conservation staff. The following table summarizes average total and free chlorine, total THM, and BDCM results taken by the Department of Environment and Conservation. Average field readings taken from Sept 27-Dec 1, 2005 (in brackets) by the System Operator will be used in calibration as they most accurately reflect the current status of the system.

Table 94: Average chlorine, THM, BDCM (2000-2007) readings for St. Paul's

Location in Network	Junction	Free Chlorine-DOEC (mg/L)	Total Chlorine - DOEC (mg/L)	THM Total-DOEC (ug/L)	BDCM (ug/L)
Beginning	15	>2.20 (1.02)	>2.20	-	
Middle-25%	3	0.78 (0.91)	1.05	288	21.7
Middle-75%	6	0.51 (0.68)	0.71	280	21.7
End	14	0.04 (0.03)	0.12	309	21.4

The CCME maximum acceptable concentration (MAC) for total THMs is 100 ug/L. As shown in the table, THM levels in St. Paul's are well over the limit throughout the system. BDCMs are also over the guideline of 16 ug/L throughout the St. Paul's network.

10.8 Calibration of the St. Paul's Model

In order to first calibrate the St. Paul's hydraulic/water quality model, results were compared with flow, pressure and chlorine residual data gathered on the St. Paul's distribution system. The collection of this data is outlined in previous sections.

Comparison of initial model results to calibration data is described in the following table, along with actions taken to compensate for any discrepancies, and final associated percentage errors found in the calibrated model. Average values from the model are taken for comparison once equilibrium or periodic behaviour from that parameter had been reached.

Table 95: Calibration of St. Paul's model

Issue	Percentage Error	Action	Percentage Error After Calibration
-average daily model	-1%	None	

flow of 2.57 L/s (daily range of 1.3-3.85 L/s) vs. average flow of 2.55 L/s			
- node 15 model pressure of 30.2 m (range of 29.7-30.6 m) vs. recorded line pressure of 24.6 m	-22.8%	None	
-node 6 model pressure of 50.1 m (range of 49.6-50.5 m) vs. recorded line pressure of 52.7 m	-5.0%	None	
-link 1 pump operation of 5.6 hrs on/ 6.1 hrs off vs. observed pump operation of 4 hrs on/ 4 hrs off	-40/53%	None	
-node 2 (tank) minimum Cl of 3.00 mg/L vs. 1.90 mg/L	-57.9%	-increased the bulk reaction rate from -1.5 to -2.5 d^{-1} -increased the wall reaction rate from -1 to -2.5 m/d	-7.4% (1.76 mg/L)
-node 15 (start of system) equilibrium (after 19hr) Cl of 3.42 mg/L (range of 3.00-3.84 mg/L) vs. 1.02 mg/L	-235%	-increased the bulk reaction rate from -1.5 to -2.5 d^{-1} -increased the wall reaction rate from -1 to -2.5 m/d	-121% (2.25 mg/L)
-node 3 (middle 25% of system) equilibrium (after 27hr) Cl of 1.98 mg/L (range of 1.60-2.36 mg/L) vs. 0.91 mg/L	-118%	-increased the bulk reaction rate from -1.5 to -2.5 d^{-1} -increased the wall reaction rate from -1 to -2.5 m/d	-21.4% (1.11 mg/L)
-node 6 (middle 75% of system) equilibrium (after 32hr) Cl of 0.37 mg/L (range of 0.64-1.17 mg/L) vs. 0.68 mg/L	-33.1%	-increased the bulk reaction rate from -1.5 to -2.5 d^{-1} -increased the wall reaction rate from -1 to -2.5 m/d	-30.9% (0.47 mg/L)
-node 14 (end of system) Cl of 0 mg/L (range of 0 mg/L) vs. 0.03 mg/L	-100%	-increased the bulk reaction rate from -1.5 to -2.5 d^{-1} -increased the wall reaction rate from -1 to -2.5 m/d	-100% (0mg/L)

A site visit was undertaken in February 2006 in order to gather further information on the distribution network. The calibration data set for St. Paul's was not complete, only covering basic elements, resulting in a rough calibration. Once results predicted by the model were felt to adequately reflect observed field data– matching pressures, flows, chlorine residuals, tank behaviour– through the adjustment of certain network parameters, a baseline model was established. The different model scenarios will then be run on this baseline model, adjusting only selected network parameters.

The following graph shows mean observed system pressure verses mean simulated values of pressure for nodes 15 and 6 (highest and lowest elevation) on the St. Paul's distribution system. As can be seen in the graph and calibration table below, actual and modeled pressures correlate very well.

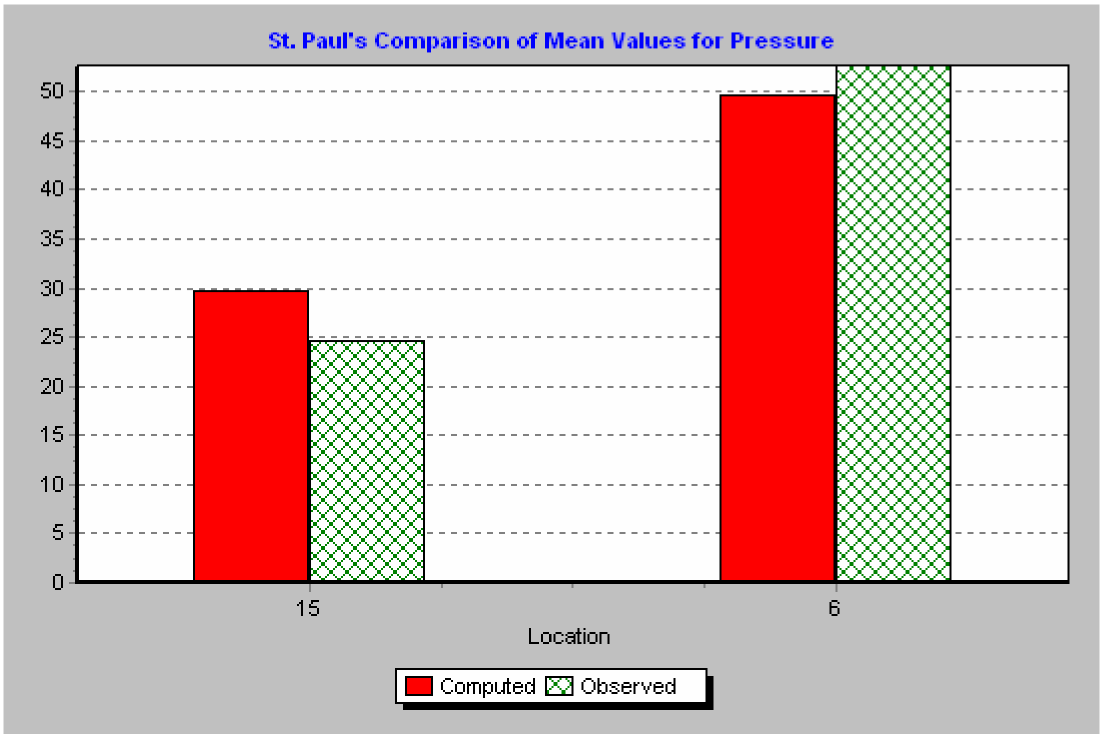


Figure 122: Mean observed and mean simulated value for system pressure in St. Paul's

Table 96: Calibration statistics for pressure

Location	Num Obs	Obs Mean	Comp Mean	Mean Error	RMS Error
15	1	24.60	29.81	5.210	5.210
6	1	52.70	49.75	2.950	2.950
Network	2	38.65	39.78	4.080	4.233

Correlation Between Means: 1.000

The following graph shows tank water level variation over the 7 day simulation period. It indicates the tank is on an average 5.6 hour filling to a 6.1 hour emptying cycle, similar to observed tank operation of a 4 hour filling/ 4 hour emptying cycle.

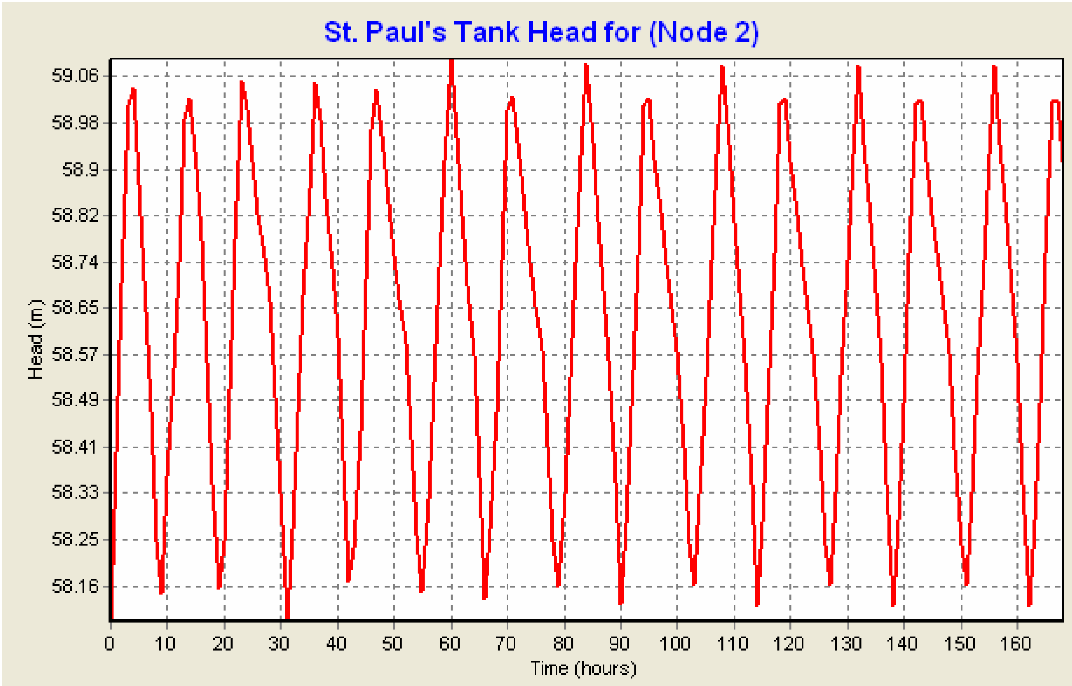


Figure 123: Water level variation in St. Paul's tank

The following graph shows system flows over the 7-day simulation period. Water is pumped at a rate of 6.53 L/s for 5.6 hours from the source to the storage tank approximately every 6 hours. Community demand is met from water in the storage tank based on the 24 hour demand pattern with an average demand of 2.55 L/s. The pump operation and tank filling cycle was felt to adequately reflect the St. Paul's system given the information available.

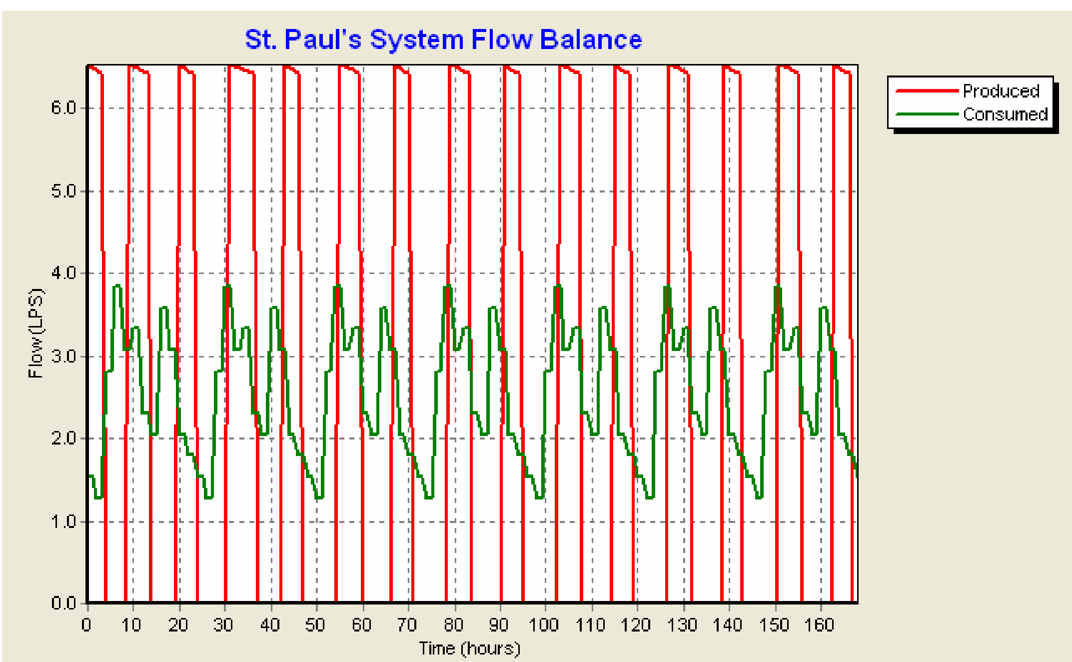


Figure 124: St. Paul's pumped flow and system demand

The following graphs and table show calibration statistics for free chlorine residuals taken from five different points in the St. Paul's distribution system. Observed chlorine readings taken from the field were assigned times after equilibrium had been reached for each node. Once chlorine reached equilibrium, it still varied with changes in system demand. A median point along the chlorine pulse cycle was used to compare simulated to observed results. There was a fairly good correlation observed between field and modeled chlorine residuals throughout the system.

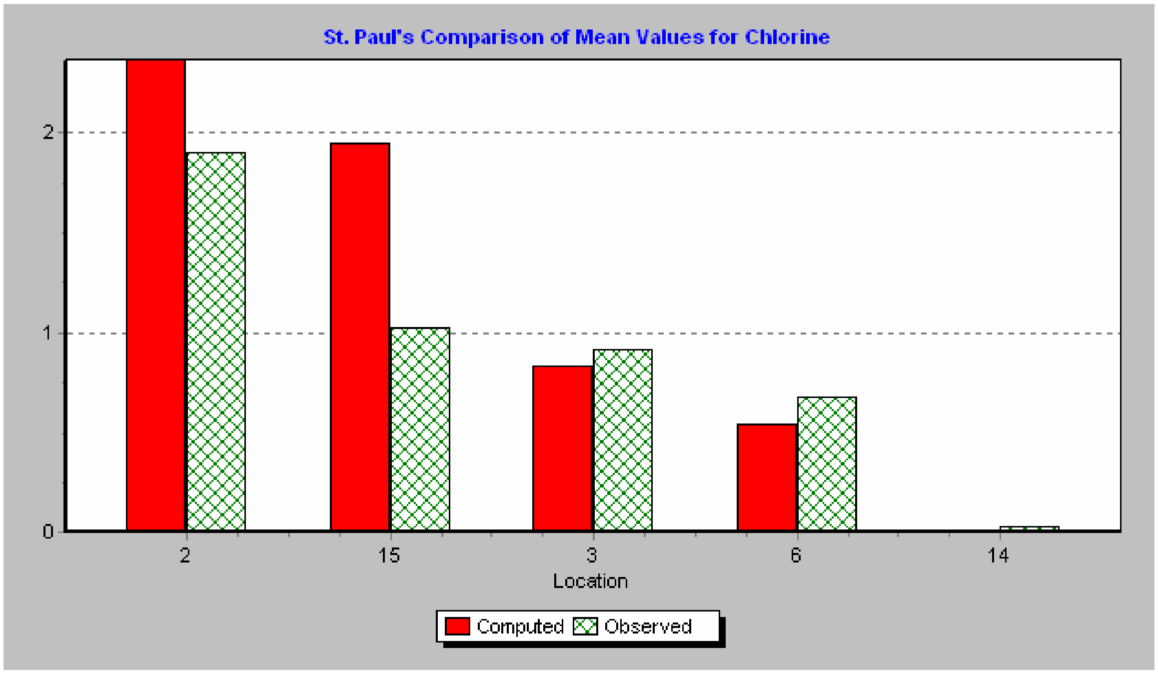


Figure 125: Mean observed and mean simulated value for chlorine residuals in St. Paul's

Table 97: St. Paul's calibration statistics for chlorine

Location	Num Obs	Obs Mean	Comp Mean	Mean Error	RMS Error
2	1	1.90	2.37	0.467	0.467
15	1	1.02	1.95	0.929	0.929
3	1	0.91	0.83	0.079	0.079
6	1	0.68	0.54	0.143	0.143
14	1	0.03	0.00	0.030	0.030
Network	5	0.91	1.14	0.330	0.471

Correlation Between Means: 0.916

10.9 Problems with the St. Paul's Distribution System

By gathering detailed background information on the St. Paul's water distribution system and establishing a calibrated baseline model, we were able to identify problems with how the system operates normally. According to the model results, chlorine residuals, while high at the beginning of the system, are inadequate by the end of the system. Several contributing factors were identified as contributing to the overall St. Paul's THM problem as outlined in the following table.

Table 98: Problems contributing to high THMs in the St. Paul's distribution system

	Causative Factors	Quantitative Value
2	Shallow intake pond with long exposed fetch length	yes
3	Surface water source exposed to saltwater influence	1.1 km (NW)
5	High DOC in source water	6.16 mg/L
6	High levels of bromide in source water	0.03 mg/L
7	High chlorine dose	12.6 mg/L
10	Excessive chlorine demand	-2.5 d-1 (bulk) -2.5 m/d (wall)
11	High pH	7.3
12	Long linear system	6.0 km
13	Branched system with multiple dead ends	at least 6 DE
14	Distance of chlorination system to first point of use	582 m contact time= 545 min CT = 491
16	System is oversized	0.0-0.13 m/s 250-100 mm Q_{avg} = 2.55 L/s
17	High retention time in network	59+ hrs
18	Pipe material and age	>25 yrs
20	Large occasional demand on system	Hotel/golf course
21	Tank location	beginning
22	Balance between pumped supply and demand not optimized with storage	4 hr to fill/ 4 hrs to empty
23	High retention time in tank	47 hrs
24	Dead zones/ poor mixing in tank	Inlet/outlet close
25	Little variation in water levels/ turnover in tank	86 % inactive volume 1 m
26	Poor O&M of system	Water Dist Class I
27	Multiple factors	-
28	Poor design of system	-
29	High iron	0.36 mg/L
30	High per capita demand	667 L/s
31	Pressure problems	min = 21.1 m
32	Problems with chlorine residuals	0 mg/L @ end

The following figures illustrate some of the problems observed in the St. Paul's distribution system.

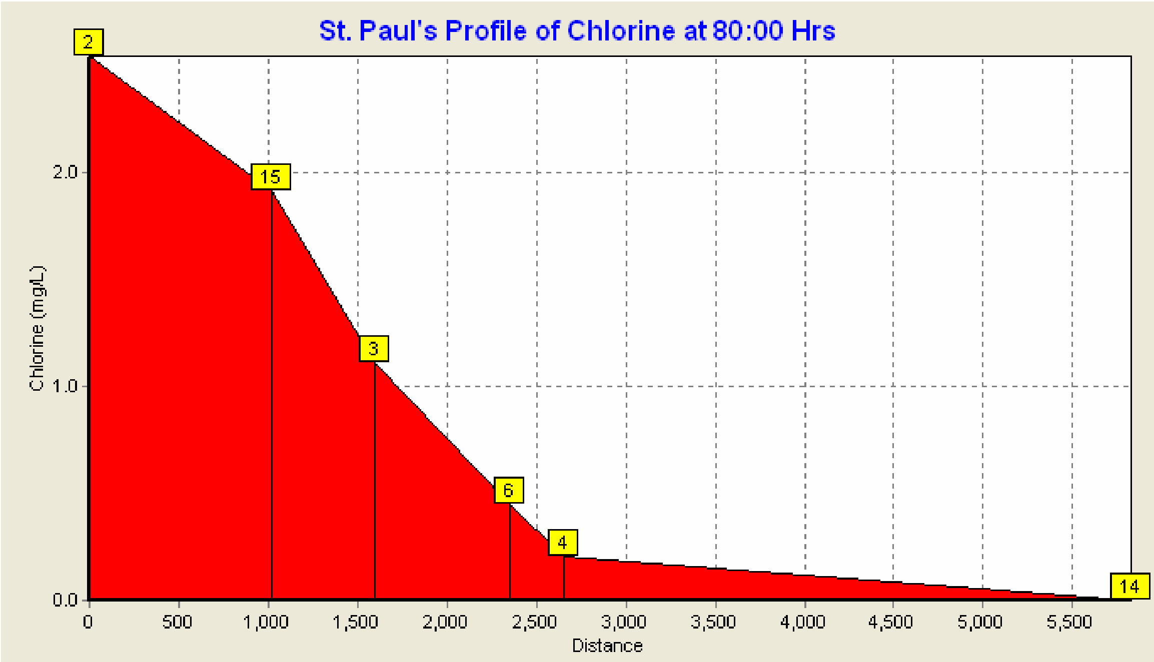


Figure 126: Chlorine decay profile through St. Paul's distribution system

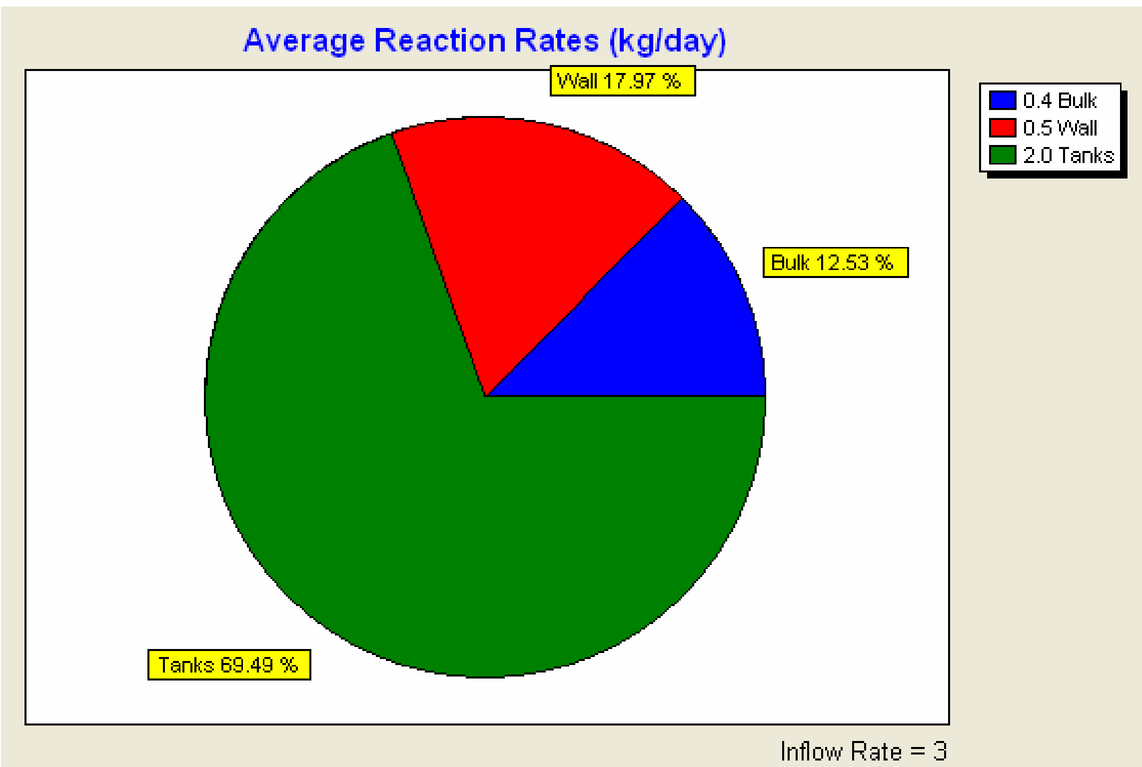


Figure 127: Chlorine decay contributions in the St. Paul's distribution system

Other issues with the St. Paul's distribution system include the difficulty in maintaining adequate chlorine residuals at the end of the system due to the seasonal lack of demand. Solutions that might address the probable causes of high THM levels in the St. Paul's distribution system are outlined in the following table. Those corrective measures highlighted in grey are the only solutions that can potentially be modeled.

Table 99: Applicable THM corrective measures for St. Paul's

Applicable Corrective Measures	Probable Causes Addressed
Policy of POU/POE treatment	All
Policy to promote use of alternative disinfectants	All
Policy to promote PWDU	All
Watershed protection	All
Alternative water source	3-5-6
Wind breaks around exposed costal water sources	3-6
High quality water storage and recovery	All
Alternative disinfectants	All
System maintenance	All
Potable water dispensing unit	All
Point of use/entry treatment	All
Training	All
Improved design of systems	All
Regionalization	All
Re-locate chlorination system	2-3-5-6-14-21
Install chlorine booster at optimal location	7-10
Reduce storage capacity/ adjust pump schedule	17-20-23-24-25
Increase mixing in tank	17-23-24-25
Regular system flushing at dead ends	2-5-12-13-16-17-20
Continuously bleed system at dead end	2-5-12-13-16-17-20
Downsizing mains	2-5-12-13-16
Replace or reline pipe	18
Loop distribution network	13
Water treatment plants	5-7
Filtration	5-7
Iron removal	7-10-29
Advanced treatment	3-5-6-7
Combination of corrective measures	All

10.10 Results from the St. Paul's Model Scenarios

The next step was to model the different selected scenarios and see how the St. Paul's distribution system responded. Given the ability of the baseline model to reflect current conditions fairly accurately, a reasonable degree of confidence can be placed in the scenario results.

10.10.1 Relocate Chlorination System

In EPANET we have chosen to model chlorine injection at node 18 with the chlorine source as a setpoint booster, which fixes the concentration of any flow leaving that node. If the primary chlorination system is located at the outlet of the tank rather than the inlet, the contact time at peak flow drops from 354 minutes to 30 minutes, using an average flow of 4.03 L/s. The requirement of a contact time of 20 minutes is therefore met.

With the chlorination system located at the outlet of the tank, a chlorine dose of 10 mg/L will achieve an adequate chlorine residual of 0.05 mg/L at the end of the system (node 14). However, the chlorine residual at the first point of use (node 15) averages around 6.0 mg/L, well above the maximum chlorine residual disinfectant criteria of 4 mg/L. At a chlorine dose of 4.9 mg/L, the maximum chlorine level at the first point of use falls below 4 mg/L, while the chlorine at the end of the system hits a minimum of 0.03 mg/L.

With the chlorination system located at the outlet of the tank, a chlorine dose of 1.9 mg/L is sufficient to achieve adequate chlorine residuals of 0.10 mg/L or greater in the main part of the system (up to node 4 or just before the bridge). There is no trace of chlorine evident at the end of the system (node 14) at this dosage level.

10.10.2 Install Chlorine Booster

A chlorine booster is a secondary chlorination system located on a water distribution system to boost chlorine residuals to appropriate levels in areas where they may have fallen below acceptable levels. For this scenario, a chlorine booster station (node 19) was located three quarters of the way along the pipe connecting the Gros Morne resort to the rest of the St. Paul's distribution network, 3.0 km from the main chlorination system.

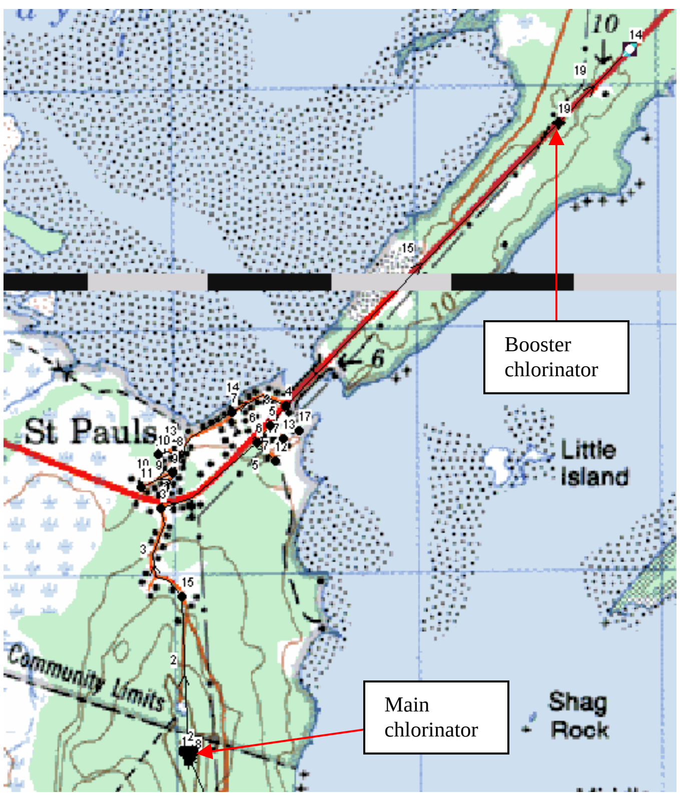


Figure 128: Optimal location of main and booster chlorinator in St. Paul's

With an initial chlorine dosage of 6 mg/L, chlorine residuals in the main part of the system up to node 4 are adequate at 0.10 mg/L or above. A booster chlorination dose of 1.0 mg/L leaving node 19 is sufficient to provide a minimum chlorine residual of 0.05

mg/L at the end of the system at a demand of 0.15 L/s or greater. If demand at the end of the system is less than this, adequate secondary disinfection requirements cannot be met.

If seasonal achievement of criteria is acceptable (ie. when demand is sufficiently high at the end of the network), the option of a chlorine booster towards the end of the system reduces the overall chlorine dosage from 12.6 to 7 mg/L. Minimizing chlorine usage has the potential to reduce THM formation.

10.10.3 Reducing Tank Storage Capacity/ Adjusting Pump Schedule

Reducing the tank storage capacity and adjusting the pump schedule are modeled in the same way. The water levels in the tank are set to automatically trigger pump operation. As the system is currently set, approximately 14.3% of the total tank volume is being actively used; the pump turns off once water levels reach 5.2 m and turn on once the water level drops to 4.2 m. Due to the location of the tank outlet (0.61 m from the bottom of the tank), 8.6 % of tank capacity is permanently inactive. Water quality degrades as a result of long residence times in storage tanks; chlorine residuals decrease with increased residence times, while disinfection by-products such as THMs increase. The maximum water age in the St. Paul's storage tank is approximately 48 hrs under off-season demand conditions (maximum water age is 31.3 hrs for on-season demand conditions).

For this corrective measure, the active storage volume in the tank was altered under three slightly different variations. The tank is always 8.6% full, but this volume is considered dead. Also, the top 1.8 m of the tank is never used under current conditions, accounting for an additional 25.7% dead volume. The first variation altered the active volume in the tank keeping the maximum water level at 5.2 m, as with current tank operation. The second variation altered the active volume in the tank by keeping the maximum water level at 7 m. This scenario produces the largest inactive tank volume. The third variation altered the active volume in the tank by keeping the minimum water level at 0.62 m. This scenario produces the larges dead tank volume and no inactive volume.

The following table summarizes the results from the scenario variations examined. Under the first variation with a third of the tank volume dead, the system performs optimally with between 14 and 25% of the tank volume active. With the full capacity of the tank potentially active, optimum results in terms of water age and end chlorine residual are attained with decreasing active tank volumes, with the best results at 25% of the tank volume active. The most effective system results were observed when the inactive volume in the tank was kept at 0%. The lowest observed water age and highest end chlorine residuals were observed with the active tank volume at only 25%.

Table 100: Effect of varying water levels in St. Paul's tank

Active Tank Volume (%)	Inactive Tank Volume (%)	Dead Volume (%)	Max Water Level (m)	Min Water Level (m)	Max Water Age in Tank (hrs)	Max Water Age at End of System: On-season Demand (hrs)	Min Chlorine at End of System (mg/L)	Chlorine Pulse Cycle (times/ day)
5	60.5	34.5	5.2	4.85	32.0	49.0	0.03	5.4
14.3	51.2	34.5	5.2	4.2	31.3	46.8	0.03	2

25	40.5	34.5	5.2	3.45	30.9	47.2	0.03	1.0
50.1	15.5	34.5	5.2	1.7	32.8	47.7	0.02	0.61
65.5	0	34.5	5.2	0.62	34.5	51.4	0.01	0.46
91.2	0	8.7	7	0.62	44.8	62.8	0.01	0.33
75.1	16.2	8.7	7	1.75	41.0	58.0	0.01	0.46
50.1	41.2	8.7	7	3.5	41.6	58.1	0.01	0.61
25	66.2	8.7	7	5.25	37.9	54.6	0.02	1.1
25	0	75	2.36	0.62	22.1	39.1	0.04	1.0
50	0	50	4.11	0.62	29.7	43.8	0.02	0.61
75	0	25	5.86	0.62	38.7	55.74	0.01	0.43

To summarize, system performance is optimized at around 25% active volume, regardless of the amount of inactive or dead volume. Overall water age is reduced (by 7.7 hours from existing conditions) and end chlorine residuals slightly increased when the inactive volume is kept at 0% and the active volume at 25%. Chlorine demand in the tank is also decreased from 69.5 to 36.3%. The worst results were observed when the entire volume of the tank was potentially active. Pump activity with an active volume of around 25% involves a longer working and resting period, approximately double the currant 4 hours on/ 4 hours off cycle. It appears that tank operation has a significant effect on chlorine residuals throughout the system, and by extension plays a significant role in THM formation.

10.10.4 Increase Mixing in Tank

When using EPANET to model hydraulic and water quality behaviour, the assumption was made that tanks behave as continuously stirred tank reactors (CSTR) where there is complete mixing. Complete mixing is an idealized assumption of the St. Paul's tank behaviour and in reality the tank probably functions more on the principle of two compartment mixing as there is a dead zone in the bottom 0.61m of the tank. Different tank-mixing scenarios were examined to determine if there were any major differences in system behaviour.

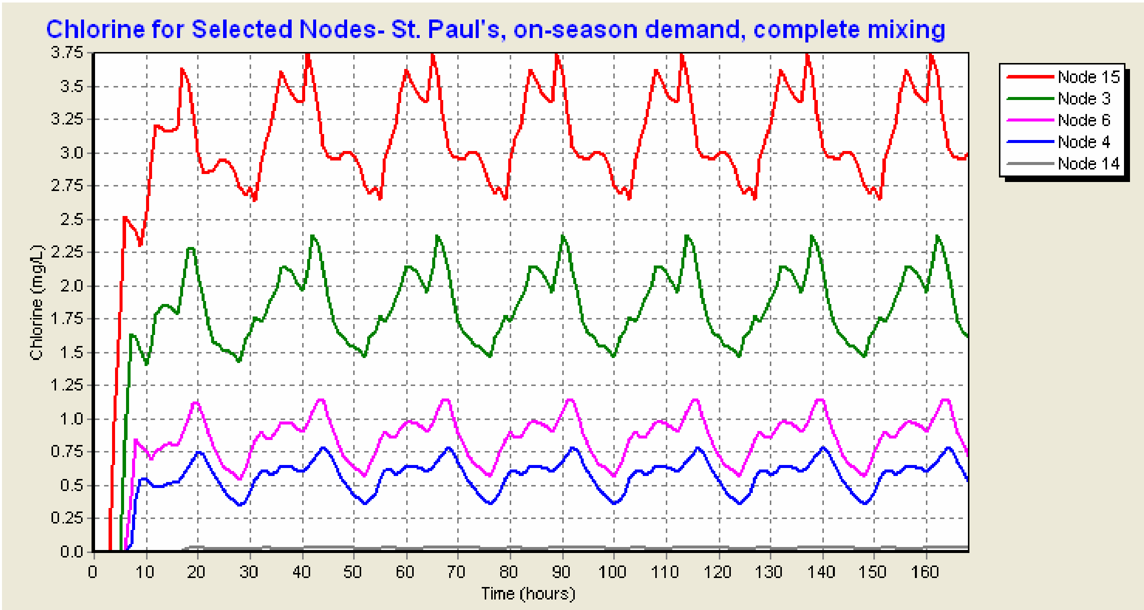


Figure 129: St. Paul’s tank as completely mixed tank model

For 2-compartment mixing tank models, a parameter representing the fraction of the total tank volume devoted to the mixed compartment must be input. The mixed compartment simulates short-circuiting between inflow and outflow, while the second compartment represents a dead zone in the tank. For this scenario a mixing fraction of 0.88 (based on maximum tank height of 5.2 m) was used.

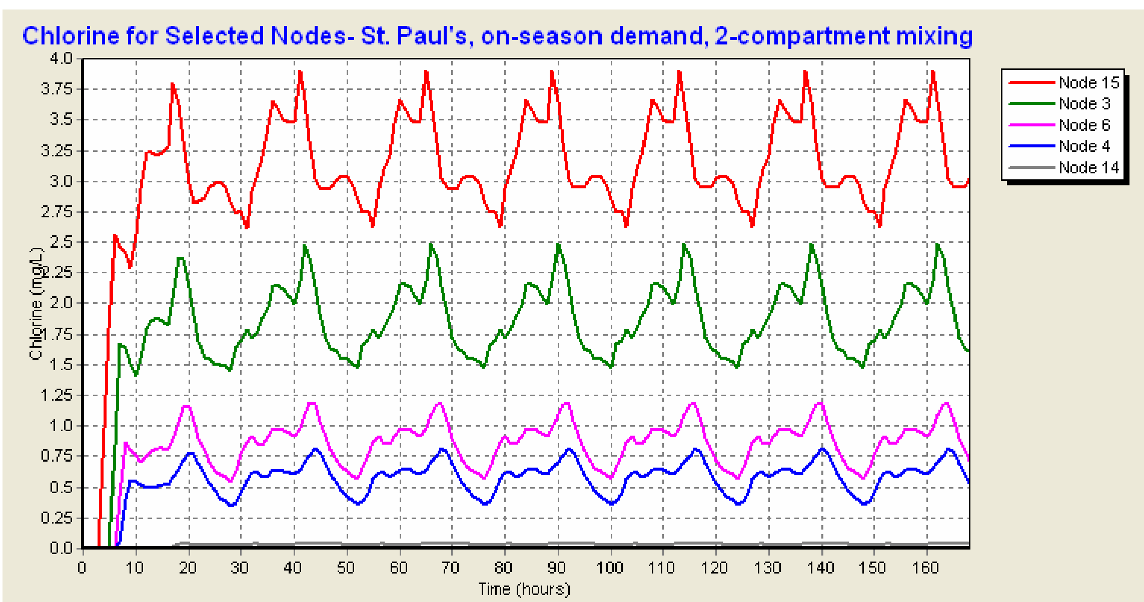


Figure 130: St. Paul’s tank as 2-compartment mixed tank model

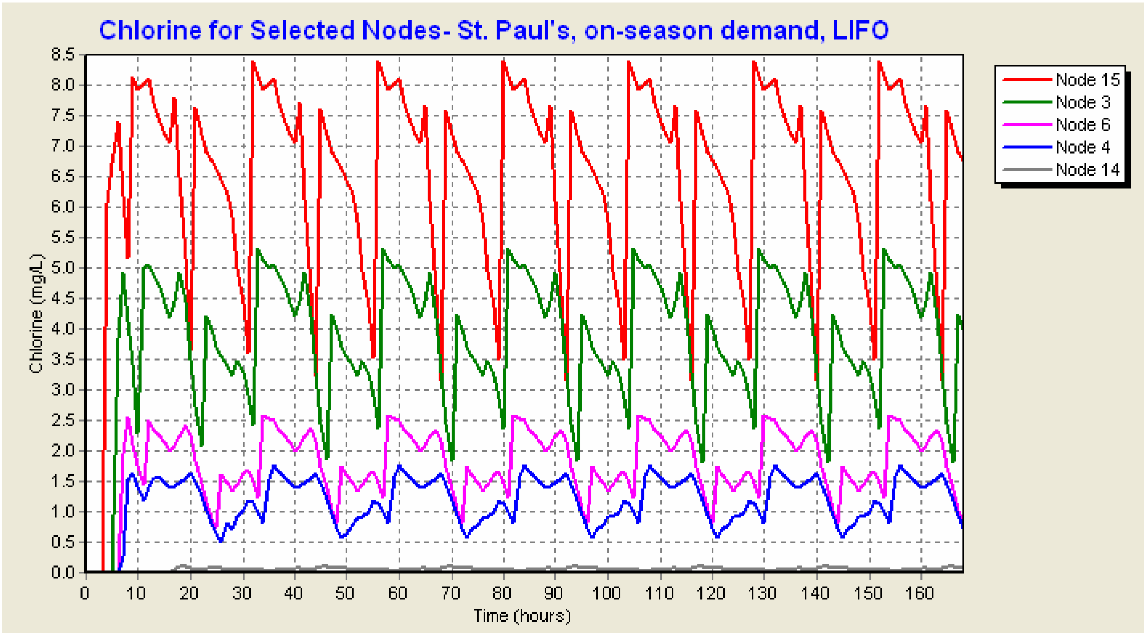


Figure 131: St. Paul's tank is as last in-first out tank mixing model

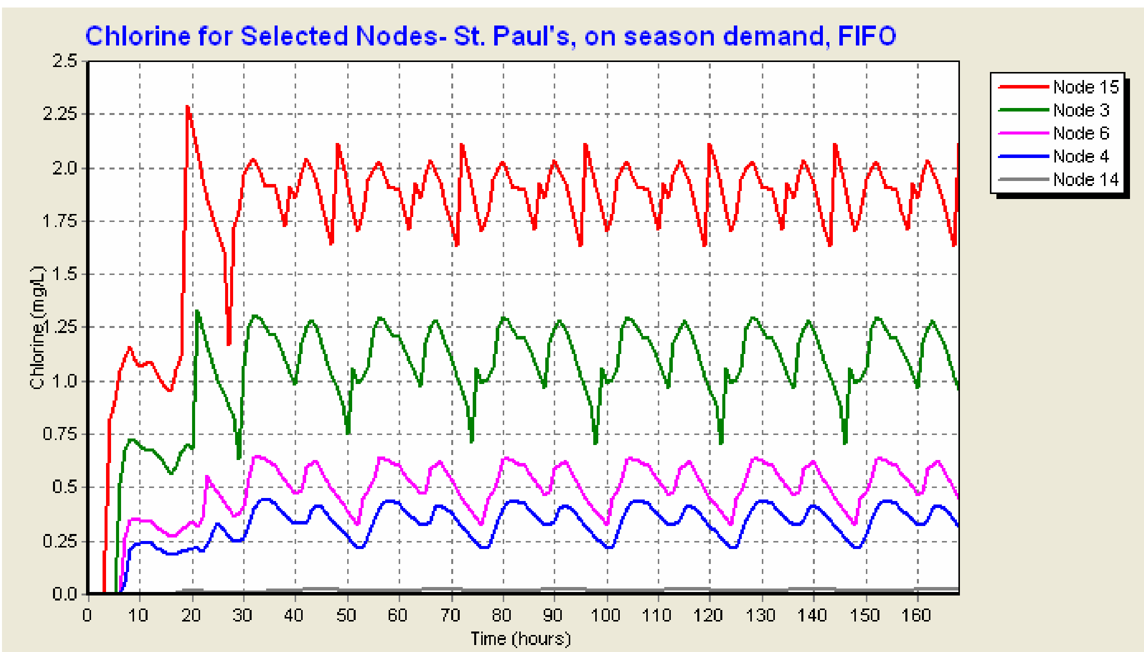


Figure 132: St. Paul's tank as first in-first out tank mixing model

The behaviour of chlorine throughout the system can change significantly based on the tank mixing models selected. Chlorine residuals increased above 4.0 mg/L at the first user, and to a minimum of 0.04 mg/L with LIFO mixing. There was less variation in chlorine residuals with FIFO mixing, however, chlorine residuals at the end of the system fell to 0.01 mg/L. There was hardly any difference between complete mixing and 2-compartment mixing with end chlorine residuals falling to 0.03 mg/L.

Increased mixing in the tank can also be achieved by forcing greater turnover in the tank. This type of simulation was performed in the previous section where the active volume in the tank was increased at the expense of the inactive volume.

10.10.5 Regular System Flushing at Dead Ends/ Continuous Bleed at System End

The maximum retention time in the St. Paul's water distribution system with off-season demand at the end node is 59 plus hours. With little water demand at the end of the system, water age in pipe 15 keeps increasing linearly with time. With on-season demand of 1.5 mg/L at the end node, water age at the end of the system is only 46.8 hours. Chlorine residuals increase throughout the network, and are still less than 4.0 mg/L at the first point of use.

Any flushing program must occur at a time period of less than 46.8 hours in order to achieve any improvement in water age, ideally at least half the current return period or every 23.4 hours. For this corrective measure, two scenarios were looked at: flushing twice a day at the end node, a continuous bleed at the end node.

The average daily flow rate is 2.55 L/s for off-season demand and 4.03 L/s for on-season demand. Flushing rates will be some multiple of this. Neither pressure nor contact time criteria are violated by either flushing or continuous bleed scenarios. However, even at the maximum average demand the system is capable of supplying without pressures becoming negative (7.03 L/s), it is impossible to reach a flushing velocity of 0.75 m/s. The maximum velocity that can be achieved in the system is only 0.21 m/s.

For the scenario where flushing occurs twice a day at the end node, base demand at node 14 was increased to 2.55 L/s (from 0.02 L/s off-season demand and 1.5 L/s on-season demand) for 4 hours at 12 hour interval. During on-season demand, maximum water age at the end node was reduced to 23.4 hours. Chlorine residuals increase throughout the network, slightly over 4 mg/L at the first user and to a minimum of 0.06 mg/L at the end of the network. At a chlorine dosage of 11 mg/L, chlorine residuals are below 4 mg/L at the first point of use and slightly above 0.05 mg/L at the end of the network. During off-season demand, maximum water age at the end node is 53.9 hours. The minimum chlorine residual at the end of the network was only 0.01 mg/L.

For the continuous bleed scenario, an additional constant demand of 2.55 L/s was placed on node 14, effectively doubling average flow. With more demand at the end of the system, water moves faster through the distribution network and maximum water age is reduced to 23.4 hours. Chlorine residuals increase throughout the network, over 4 mg/L at the first user and to a minimum of 0.08 mg/L at the end of the system. With a continuous bleed of 2.55 L/s on the system, the chlorine dose can be reduced from 12.6 to 8 mg/L while maintaining a chlorine residual below 4.0 mg/L at the first point of use and above 0.05 mg/L at the end of the system.

Manual flushing once a day or more at the end of the St. Paul's distribution network may not be a practical use of resources; however, automatic hydrant flushing units could be used. Continuously bleeding the system is wasteful of resources (water, energy costs, chlorine) and may harm the receiving environment. However, both options offer positive potential for the reduction of DBPs through decreased water age and chlorine dose. The benefits of either option must be examined in the context of its various disadvantages in the case of flushing corrective measures.

10.10.6 Replacing or Relining Pipe/ Downsizing Mains

The St. Paul’s distribution network is oversized for the demand placed on the system. Pipe size ranges from 250 to 100 mm with the majority of pipe in the network sized at 150 mm or greater in order to fit fire hydrants. The maximum observed velocity in the system is 0.13 m/s observed in the section of pipe leading from the intake to the tank.

The DI pipe in the St. Paul’s network dates to 1978, while the HDPE section extending to the Gros Morne Resort dates to 2002. For the first scenario, all old pipes in the network were modeled as brand new PVC, reflected in the input pipe roughness coefficient value. This resulted in an increase in the Hazen-Williams C value from 90 to 150. The model results indicated no change in chlorine residuals or water age.

For the second scenario, each pipe was resized so as to achieve a peak velocity of approximately 0.4 m/s or a minimum pipe size of 40 mm. Using these criteria, pipe sizes in the St. Paul’s distribution system now range from 40-125 mm. The resulting maximum water age at the end of the system is now 26.9 hours (reduced from 46.8 hours). Pressure in the network has decreased slightly but is still within criteria range. Chlorine residuals at the end of the system have dropped from 0.03 to 0.01 mg/L, and are slightly lower throughout the entire system. Due to the residence time available in the tank, contact time requirements are not an issue. The rate of reaction of chlorine at the pipe wall is inversely related to pipe diameter, so the smaller the pipe diameter, the greater the pipe wall reaction rate and the greater the amount of chlorine consumed at the pipe wall (wall chlorine demand increases from 78% to 36.8%). Even with a significant decrease in water age in the system, the chlorine dose would have to be increased in order to achieve adequate residuals at the end of the system, potentially offsetting any reduction in DBPs.

10.10.7 Reconfiguring the Distribution Network through Looping

The St. Paul’s water distribution network is fairly compact, except for the line that extends for almost 2 km across St. Paul’s inlet to the Gros Morne Resort. There is the potential for increased looping of the system in the main residential portion of the network. For this scenario, 4 additional pipes were included in the network to incorporate dead ends into loops. The line extending to the resort cannot be feasibly looped. With water moving through the distribution system so slowly, particularly at dead ends, there is plenty of time for chlorine to decay and for DBPs to form.

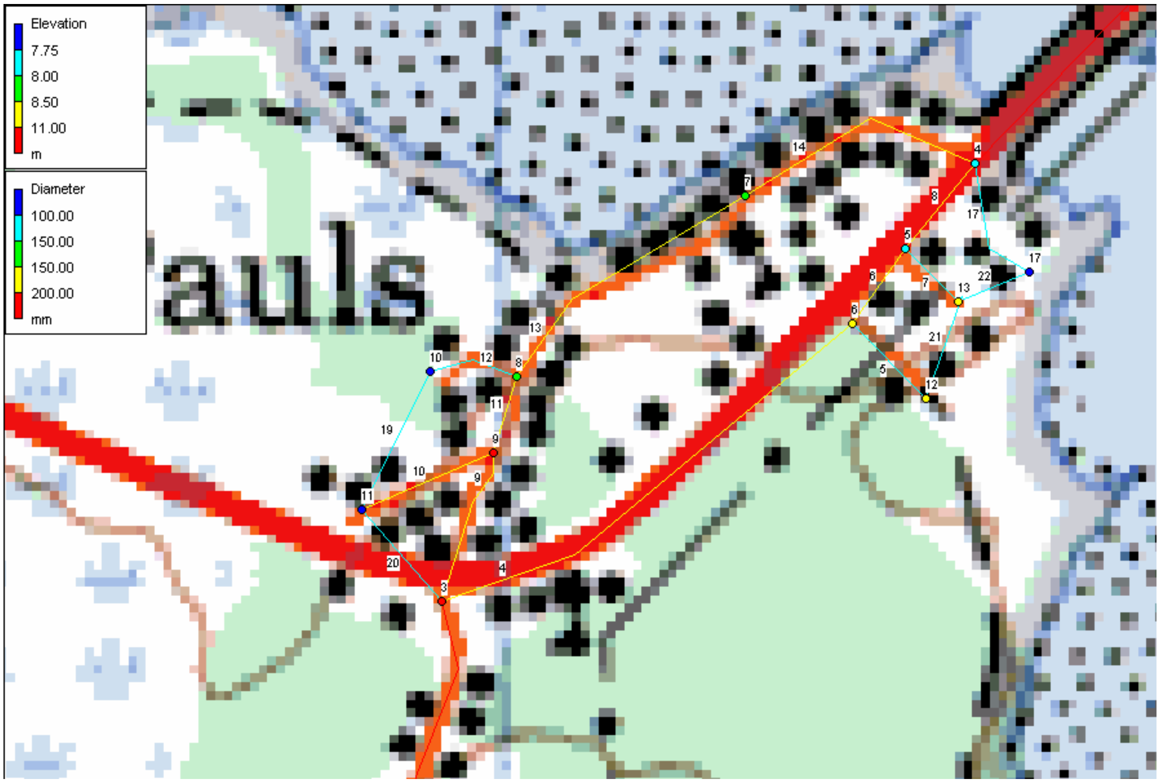


Figure 133: Looping of the St. Paul's network

With the system looped, maximum water age at the end of the system was 47.8 hours, an increase of 1 hour over the network as is. This slight increase in water age also resulted in a slight lowering of minimum observed chlorine residuals at the end of the system from 0.03 to 0.02 mg/L. There is no benefit to looping the St. Paul's distribution network to try and reduce DBPs.

10.11 Impact of Modeled Corrective Measures

Of the 9 corrective measures identified in a previous section that could be modeled in EPANET in order to access their impact in terms of improving water quality (looking at chlorine, water age, and potential THM formation), two were grouped together with other related scenarios. Not all scenarios met the required criteria in order to be deemed successful. Any scenario that saw a reduction in the overall chlorine dosage and a decrease in water age has potential for lowering THM levels. The following table highlights which scenarios had a positive impact on water quality.

Table 101: Modeled scenarios for the St. Paul's network and their effectiveness

	Scenario Description	All Criteria Met	Comments
1	Relocate Chlorination System After Tank	No	-Cl dose set to achieve secondary disinfection violates max Cl of 4 mg/L -Cl dose set to keep Cl below 4 mg/L at 1st point of use violates

			secondary disinfection requirements
2	Install Chlorine Booster	Yes/No	-chlorine dosage reduced by 5.6 mg/L -secondary disinfection requirements not met with off-season demand at end node
3	Reducing Tank Storage Capacity/ Adjusting Pump Schedule	Yes/No	-water age decreased while Cl residuals increased slightly -for 25% active, 0% inactive, 75% dead volume scenario -if end node residual of 0.04 mg/L is deemed acceptable
4	Increase Mixing in Tank	No	-secondary disinfection requirements not met -Cl levels exceed 4.0 mg/L
5	Regular System Flushing at Dead Ends/ Continuously Bleed System	Yes	-water age decreased while Cl residuals increased -overall Cl dose can be reduced
6	Replacing or Relining Mains/Downsizing Mains	No	-secondary disinfection requirements not met -water age will decrease, but higher Cl dose required
7	Reconfiguring Distribution System through Looping	No	-no improvement in Cl residuals

Any corrective measures that did not meet the necessary criteria should be dropped from consideration and evaluated no further. Scenarios that saw potential for the overall chlorine use to be reduced and water age in the distribution system lowered will be the most effective in terms of lowering THMs. Based on this assessment, the corrective measures (that met criteria) with the most potential for reducing THM formation are:

- Regular system flushing at dead ends/ continuously bleed system
- Install a chlorine booster
- Reduce tank storage capacity

10.12 Assessment of Corrective Measure Constraints for St. Paul's Network

The following table evaluates each remaining corrective measure for the St. Paul's water distribution system against identified solution constraints. The selection of the preferred solution(s) to water quality problems can be made based on the corrective measure(s) with the highest score(s).

Based on the resulting scores, there are three main tiers of possible solutions. The top three tiers in the decision matrix scoring system comprise the corrective measures that have the most potential for effectively optimizing chlorine dosage, reducing water age and lowering THMs.

The first tier, which scored 14, consists of installing a Potable Water Dispensing Unit. The second tier of solutions, which scored 13, consists of the general best management practice of improving system design, and regular system flushing at dead ends. The third tier of corrective measures, which scored 12, consists of “soft” solutions such as watershed protection, adaptive policy changes to promote PWDUs, and operator education and training. The more technical or “hard” solution in the third tier consists of reducing storage capacity in the tank.

The selection of a preferred solution by the decision making body (town, engineering consultant, Department of Municipal Affaires) can be guided by this decision making framework. The next step in the process involves the implementation of the preferred solution, monitoring and review.

Table 102: Assessment of solution constraints for St. Paul's

Applicable Corrective Measures	Effectiveness	Cost	Time Scale for Implementation	Permanency of Solution	Adverse Hydraulic Impacts	Adverse WQ Impacts	Acceptable to Stakeholders	Meets Regulations	Total
Policy of POU/POE treatment	1	2	0	0	1	2	1	1	8
Advanced treatment	2	0	0	2	1	2	0	2	9
Alternative water source	1	0	0	2	1	2	1	2	9
High quality water storage and recovery	1	0	0	2	1	2	1	2	9
Policy to promote use of alternative disinfectants	1	2	0	1	1	1	1	2	9
Combination of corrective measures	2	0	0	2	1	1	1	2	9
Alternative disinfectants	2	1	1	1	1	1	1	2	10
Install chlorine booster at optimal location	1	1	1	1	1	1	2	2	10
Regionalization	1	1	1	2	1	1	1	2	10
Water treatment plants	2	0	0	2	1	2	1	2	10
Wind breaks around exposed costal water sources	0	2	0	2	1	1	2	2	10
Continuously bleed system at dead end	2	1	1	1	2	2	1	1	11
Filtration	1	1	1	1	1	2	2	2	11
Iron and manganese removal	1	1	1	1	1	2	2	2	11
Point of use/entry treatment	2	2	2	0	1	2	1	1	11
System maintenance	1	2	2	0	1	2	1	2	11
Policy to promote PWDU	1	2	0	2	1	2	2	2	12
Reduce storage capacity/ adjust pump schedule	1	2	1	1	1	2	2	2	12
Training	1	2	0	1	2	2	2	2	12
Watershed protection	0	2	1	2	1	2	2	2	12
Improved design of systems	1	2	0	2	2	2	2	2	13
Regular system flushing at dead ends	2	2	1	1	2	2	2	1	13
Potable water dispensing unit	2	2	1	2	1	2	2	2	14

11.0 Hawke's Bay Water Distribution System Model

The Hawke's Bay water distribution system ranges in age with some sections 25 years or older. The network is fairly compact, but branched with several dead ends. Water is pumped from the Torrent River to an elevated storage tank to provide pressure to the system. Water levels in the tank control pump activity at the intake and subsequently chlorine injection at the pump house. The pump feeds the town and tank when running and when off, the tank feeds the town. The surface water supply is on a large watershed (616 km^2) that experiences fairly large variation in annual flow and above average colour. The Hawke's Bay distribution system can be classified as very small and from the Western Region of the province. Hawke's Bay has been having an issue with THM levels frequently being over Canadian Drinking Water Quality Guidelines.

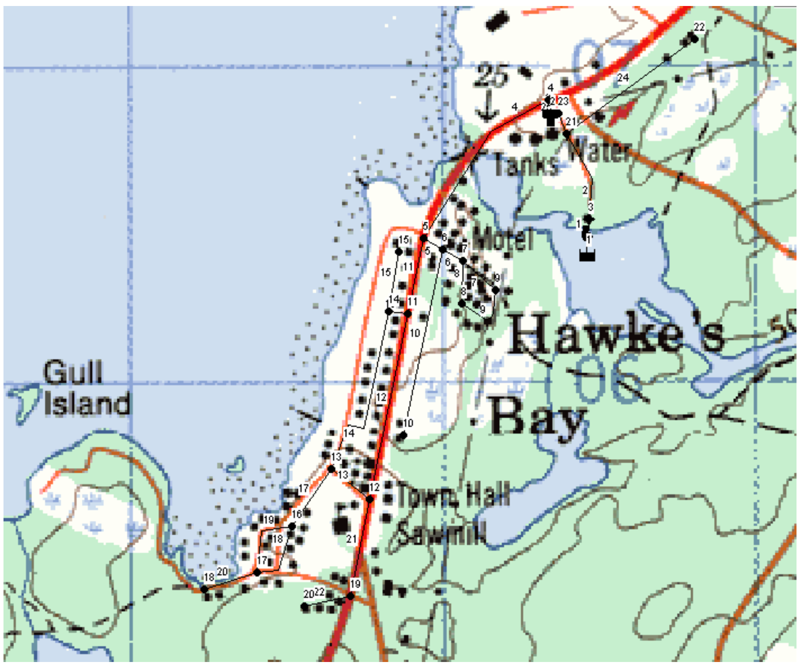


Figure 134: Hawke's Bay water distribution system network

Descriptive data for the Hawke's Bay water distribution system is detailed in following sections. This data was then input into the Hawke's Bay EPANET hydraulic/water quality model. The next step involved calibrating the Hawke's Bay model with system data also highlighted in the following sections. Different corrective measures and modeling scenarios were then selected based on observed problems with how the distribution system is currently operated. The potential effectiveness of the given solution or modeled scenarios was then weighted against solution criteria and constraints.

11.1 Reservoir

The water supply for the town of Hawke's Bay is the Torrent River with a watershed area of 616 km^2. The intake is located approximately 500 m inland from Route 430 (Viking Trail). The water level elevation at the intake location is 4.6 m, although water levels do vary throughout the year. During summer months when water levels are low, the town has experienced turbidity problems, which might be alleviated if the intake was extended out into deeper water.

Table 103: Average source water quality values for Hawke's Bay

Water Quality Parameters	Average Values 1997-2005
Colour (TCU)	48
pH	7.0
Turbidity (NTU)	0.54
Bromide (mg/L)	0.024
Chloride (mg/L)	4.4
DOC (mg/L)	5.1
Temp (^oC)	10.8

Figure 135: Torrent River at Hawke's Bay intake location

11.2 Pumps

There are two pumps operating in tandem on the Hawke's Bay water distribution system. One pump at a time feeds the town and tank when running and when off, the tank feeds the town. The pumps cut in automatically when system pressure drops to 42 psi (290 kPa) and cuts out when system pressure hits 62 psi (427 kPa) based on the readings of an altitude valve at the base of the tank. When operational, the pumps produce a steady flow rate of 12.1 L/s. The alternating pumps will run for approximately 6 hours and then off for 6 hours. No pump performance curves were available for Hawke's Bay however, pump power is 11 kW or 12.7 L/s at 63 m TDH.

Figure 136: Hawke's Bay pump configuration

11.3 Tank

The Hawke's Bay elevated tank is a Horton Watersphere that was installed in 1974. The tank is there to provide pressure to the distribution system in a relatively flat coastal area where the source is at a lower elevation than most of the demand points. The tank is located off a small T-branch approximately 360 m down-pipe of the intake. The height to the spherical tank bottom is 30.5 m with a total volume of 284 m^3 and a diameter of 8.15 m. When the pump is operational, water is sent to both the tank and the distribution system, and when the pump is off the tank feeds water to the entire system. The tank takes approximately 6 hours to fill to approximately ¾ of the sphere's height and 6 hours to empty. The inlet and outlet of the tank are located at the same opening in the base of the tank. The following height to volume curve was derived for the spherical Hawke's Bay elevated water storage tank.

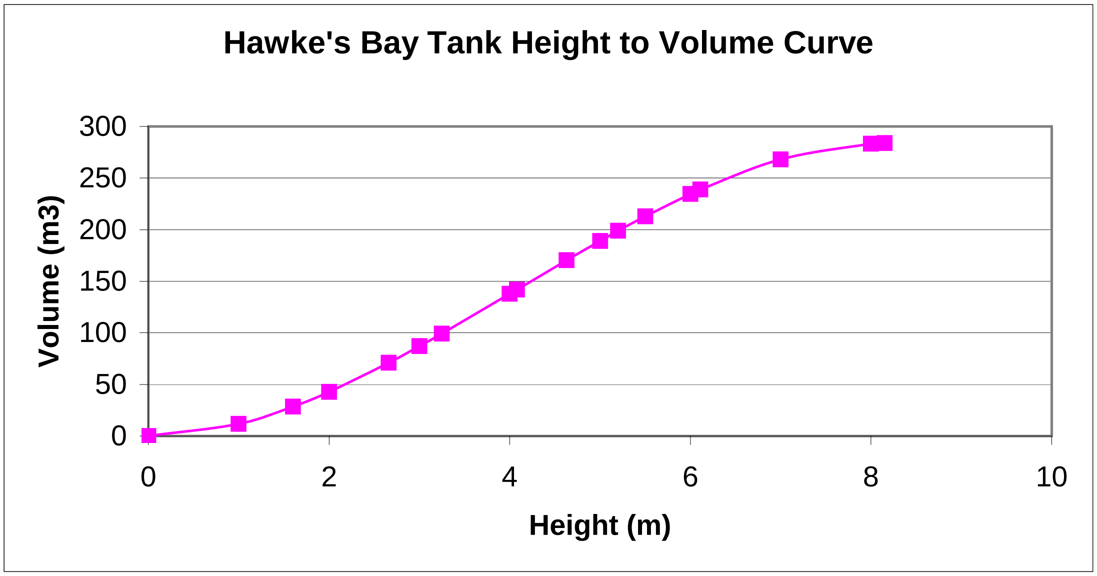


Figure 137: Hawke's Bay tank height to volume curve

Figure 138: Hawke's Bay water storage tank

11.4 Pipes

The Hawke's Bay water distribution system ranges in age with some sections 25 years or older. The network is fairly compact, but branched with several dead ends. Pipes in the distribution system range from 250 mm PVC coming from the intake to 50 mm PVC lateral lines. There is approximately 2.4 km of trunk main laid down from the intake to the end of the network in the Hawke's Bay distribution system (approximately 6.0 km of pipe in total). Line pressure in the system is estimated at 345 kPa.

The Hazen-Williams head loss formula was selected for this model in order to determine energy losses throughout the system. Roughness factors were selected based on the age of the pipe, ranging from 130 for older pipes to 150 for newer pipe.

11.5 Demand

From meter readings taken from the pump house over the period from March 28-April 28, 2006, an average daily demand of 571 m^3/d (6.61 L/s) was determined for the town of Hawke's Bay. An instantaneous flow reading of 12.1 L/s was observed in the pump house during a site visit. With a census population in 2001 of 445 people this equates to a per capita average demand of 1283 L/person/day. This is an excessively high water demand (typical demand in the province ranges from 350-650 L/p/d). The following breakdown of the type of water users does little to explain this excessive demand either.

Table 104: Type of water user in Hawke's Bay

Type of Connection	Number
Domestic	165-181
Commercial	25
Industrial	1
Institutional	1

The overall demand was then attributed to 18 different junctions throughout the distribution network based on building density surrounding that junction. Elevation of the junctions with assigned demand ranged from 2.3 m to 22.7 m above sea level. The generic daily demand pattern used in all the models was also used for Hawke's Bay.

11.6 Chlorine Decay

The Hawke's Bay distribution system has a gas chlorination system. Chlorine is only injected into the system when the pump is running, and provides a constant dose proportional to flow. From readings of the chlorine cylinder weigh scales over the period from Jan 1- May 30, 2006; average daily chlorine consumption is 6.4 lb/day. Based on water and chlorine use a chlorine dose of 5.1 mg/L was calculated.

According to the town, they are experiencing difficulties in maintaining detectable free chlorine residuals in all areas of the distribution system. The first water user on the Hawke's Bay network is on a 50 mm line that attaches to the main coming from the intake, prior to the tank. At an average flow of 571 m^3/d in the trunk main running to the first user, the contact time at peak flow is only 10 minutes. Chlorine residuals vary widely at this first user ranging from 0.03 to over 2.2 mg/L. Under worst-case conditions, the CT factor value is 0.3, well below the required value of 6. Reconnecting this lateral down-pipe of the storage tank may correct this design problem.

A bulk chlorine decay coefficient value of –2.5 d^{-1}, and a wall decay coefficient of –2.5 m/day was assumed for the model.

Figure 139: Hawke's Bay chlorination system

11.7 Chlorine and THM Data Gathering

Chlorine readings are regularly made by the Hawke's Bay System Operator and by Department of Environment and Conservation staff. The following table summarizes average chlorine and total THM results. The free chlorine readings from the town were collected during the month of April 2006.

Table 105: Average chlorine, THM and BDCM (DOEC averages from 1999-2005) readings from the Hawke's Bay network

Location in Network	Junction	Free Chlorine-Town (mg/L)	THM Total-DOEC (ug/L)	BDCM-DOEC (ug/L)
Beginning	5	0.78	114	4.0
Middle	12	0.17	116	5.6
End	17	0.09		
End	19	0.14		

The CCME maximum acceptable concentration (MAC) for total THMs is 100 ug/L. As shown in the table, average THM levels in Hawke's Bay are slightly over the limit.

11.8 Calibration of the Hawke's Bay Model

In order to calibrate the Hawke's Bay hydraulic/water quality model, results were compared with flow, pressure, tank filing/emptying cycles and chlorine residual data collected from the Hawke's Bay distribution system. The collection of this data is outlined in previous sections.

Comparison of initial model results to calibration data is described in the following table along with actions taken to compensate for any discrepancies, and final associated percentage errors found in the calibrated model. Average values from the model are taken for comparison once equilibrium or periodic behaviour from that parameter had been reached.

Table 106: Calibration of Hawke's Bay model

Issue	Percentage Error	Action	Percentage Error After Calibration
-17.7 L/s model flow during tank filling vs. observed instantaneous flow of 12.1 L/s	46%	-adjusted pump power down from 11 kW to 5.5kW	7.4% (13L/s)
-node 4 model pressure ranges from 31-37m vs. tank altitude valve pressure of 30-44 m	-5.5%	- adjusted pump power down from 11 kW to 5.5kW	-5.5% (31-37m)
-tank 6 hr filling/ 9.5hr emptying cycle vs. observed approximate 6.5 hr filling/ 6.5 hr emptying cycle	19.2%	-adjusted pump power down from 11 kW to 5.5kW	54% (11 hrs filling/9 hrs emptying)
-node 5 equilibrium (after 7hr) Cl of 1.53 mg/L vs. observed average of 0.78 mg/L	-96.2%	-adjusted pump power down from 11 kW to 5.5kW -decreased chlorine dosage to 4.0 mg/L	-44.9% (0.13mg/L)
-node 12 equilibrium (after 8hr) Cl of 0.53 mg/L vs. 0.17 mg/L	-211%	-adjusted pump power down from 11 kW to 5.5kW -decreased chlorine dosage to 4.0 mg/L	-117% (0.39mg/L)
-node 19 equilibrium (after 16hr) Cl of 0.18 mg/L vs. observed average of 0.14 mg/L	-28.6%	-adjusted pump power down from 11 kW to 5.5kW -decreased chlorine dosage to 4.0 mg/L	-7.1% (0.13mg/L)
-node 17 equilibrium (after 15hrs) Cl of 0.16 mg/L vs. 0.09 mg/L	-72.2%	-adjusted pump power down from 11 kW to 5.5kW -decreased chlorine dosage to 4.0 mg/L	-22% (0.11mg/L)

Once results predicted by the model were felt to adequately reflect observed field data– matching pressures, tank filling/empting cycles, flows, chlorine residuals– through the adjustment of certain network parameters, a baseline model was established. The different model scenarios will then be run on this baseline model, adjusting only selected network parameters.

The following graph shows tank water level variation over a 14-day simulation period. It indicates the tank is on an 11 hour filling/ 9 hour emptying cycle, somewhat similar to observed tank operation.

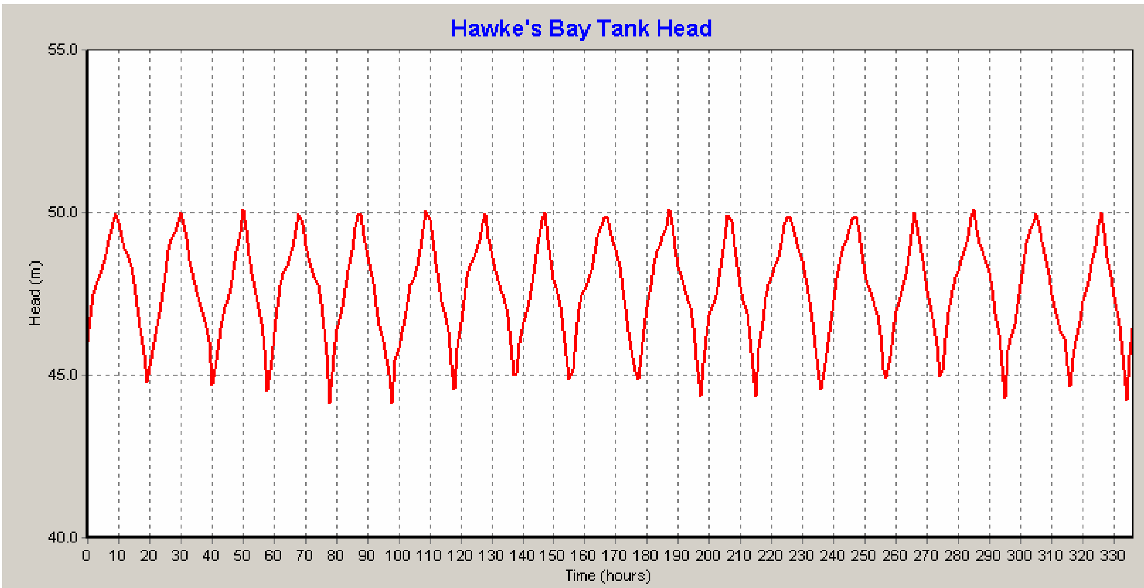


Figure 140: Hawke's Bay tank operation

The following graph shows flow coming from the pump over a 7-day simulation period and consumer water use. The pump operation mirrors that of the tank filling/emptying cycle. The steady instantaneous flow rate of 12.1 L/s observed in the field from the pump house meter is matched by simulation results.

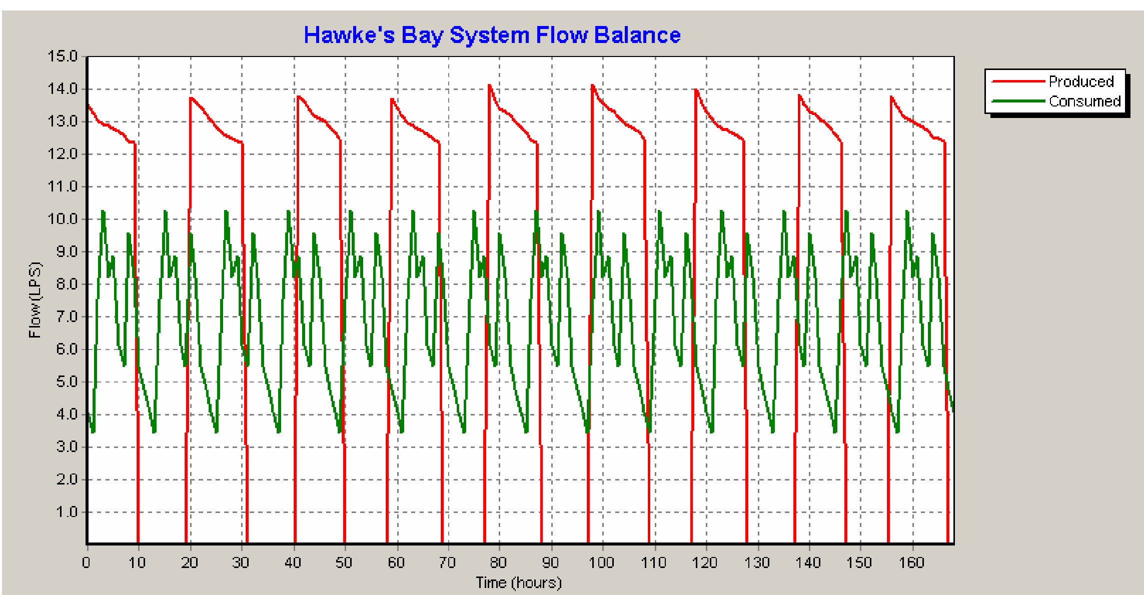


Figure 141: Hawke's Bay pumped flow and system demand

The following table and graph show calibration statistics for free chlorine residuals taken from four different points in the Hawke's Bay distribution system. Observed chlorine readings taken from the field were assigned an arbitrary time (80 hrs) after equilibrium had been reached for each node. Once chlorine reached equilibrium, it still varied significantly, pulsing with pump/tank operation. A median point along this chlorine pulse cycle was used to compare simulated to observed results. All observed results were within range of the simulated variation at each individual node. There was little error between field and modeled chlorine residuals, indicating a very good correlation.

Table 107: Hawke's Bay calibration statistics for chlorine

Location	Num Obs	Obs Mean	Comp Mean	Mean Error	RMS Error
5	1	0.78	1.02	0.245	0.245
12	1	0.17	0.17	0.004	0.004
19	1	0.14	0.19	0.054	0.054
17	1	0.09	0.08	0.010	0.010
Network	4	0.30	0.37	0.078	0.125

Correlation Between Means: 0.998

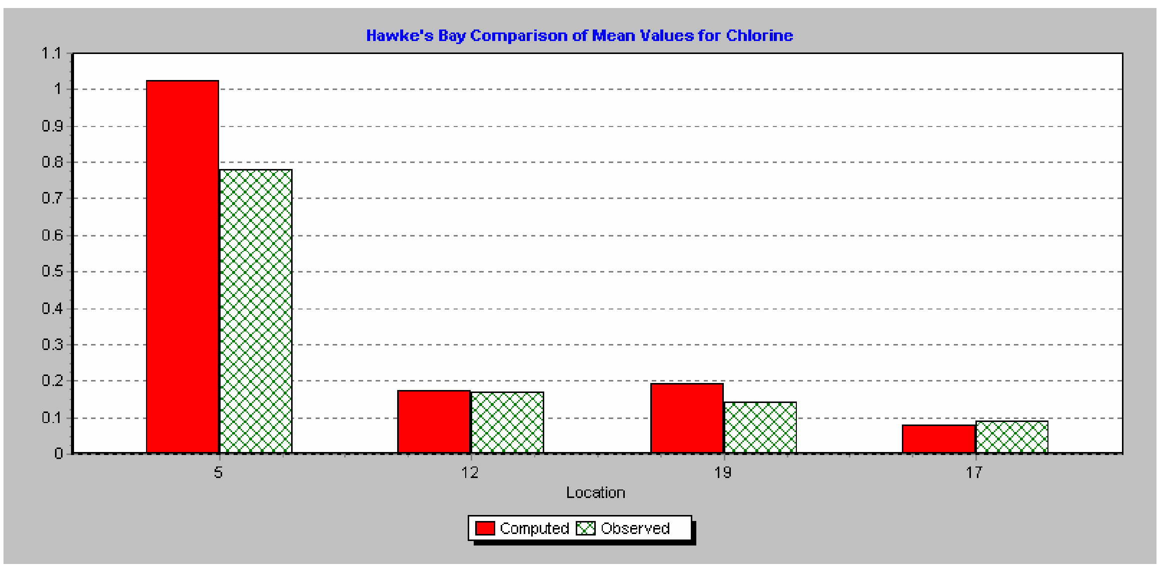


Figure 142: Mean observed and mean simulated value for chlorine residuals in Hawke's Bay

11.9 Problems with the Hawke's Bay Distribution System and Appropriate Corrective Measures

By gathering detailed background information on the Hawke's Bay distribution system and establishing a calibrated baseline model, we were able to identify problems with how the system operates normally. The source water appears highly productive for THM formation with fairly high colour, turbidity and DOC. Low water levels in the Torrent River in the summer have also given rise to turbidity and other water quality problems.

The town appears to be using 3-times as much water per capita than is normal. The usual range of water consumption in the province is from 350-650 L/p/d; Hawke's Bay is using

1283 L/p/d, assuming the meter is working correctly. There is a lack of water demand in parts of the distribution system, evident by the slow water velocities observed throughout the system. Oversized pipes used to fit fire hydrants are also contributing to the slow water velocities. Water age in the distribution system and tank, however, is not excessive, a maximum of 29 and 17 hours respectively.

The Hawke's Bay distribution network is fairly compact, however the network is highly branched with several dead ends and old section of pipe. System pressures in higher elevations at the southern end of town are very low according to the model, at times falling below minimum system pressure requirements.

According to the model results, there are problems achieving adequate chlorine residuals at the end of the system, corroborating field data collected by both the Dept of Environment and the Town of Hawke's Bay. The chlorine dosage at the beginning of the system is currently a little low in order to maintain adequate residuals in the system when the pump is not operating. Rapid chlorine decay was observed at the beginning of the system along with excessive chlorine decay throughout the distribution system and in the tank. The chlorine CT value and free residual level requirements at the first user are both violated. At peak flow the contact time is 49 minutes but the CT value is only 1.5. A minimum free chlorine residual of 0.30 mg/L at the first point of use cannot be maintained. There is also a wide variation in chlorine residuals during the tank filling/ emptying cycle.

The following table outlines several contributing factors that were identified as contributing to the overall THM problem in Hawke's Bay.

Table 108: Problems contributing to high THMs in the Hawke's Bay distribution system

	Causative Factors	Quantitative Value
2	Shallow intake	yes
3	Surface water source exposed to saltwater influence	900 m (NW)
5	High DOC in source water	5.1 mg/L
6	High levels of bromide in source water	0.024 mg/L
10	Excessive chlorine demand	-2.5 d-1 (bulk) -2.5 m/d (wall)
11	High pH	7.0
13	Branched system with multiple dead ends	at least 5 DE
14	Distance of chlorination system to first point of use	880 m contact time= 49 min CT = 1.5
15	Insufficient chlorination controls on system	manual
16	System is oversized	0.01-0.29 m/s 250-150 mm Q_{avg} = 6.61 L/s
18	Pipe material and age	>25 yrs
21	Tank location	beginning

22	Balance between pumped supply and demand not optimized with storage	6.5 hr to fill/ 6.5 hrs to empty
26	Poor O&M of system	Water Dist Class I
27	Multiple factors	-
28	Poor design of system	-
30	High per capita demand	1283 L/p/d
31	Pressure problems	min = 21.3 m
32	Problems with chlorine residuals	0.02 mg/L @ end

The following figures help illustrate some of the problems observed in the Hawke's Bay distribution system.

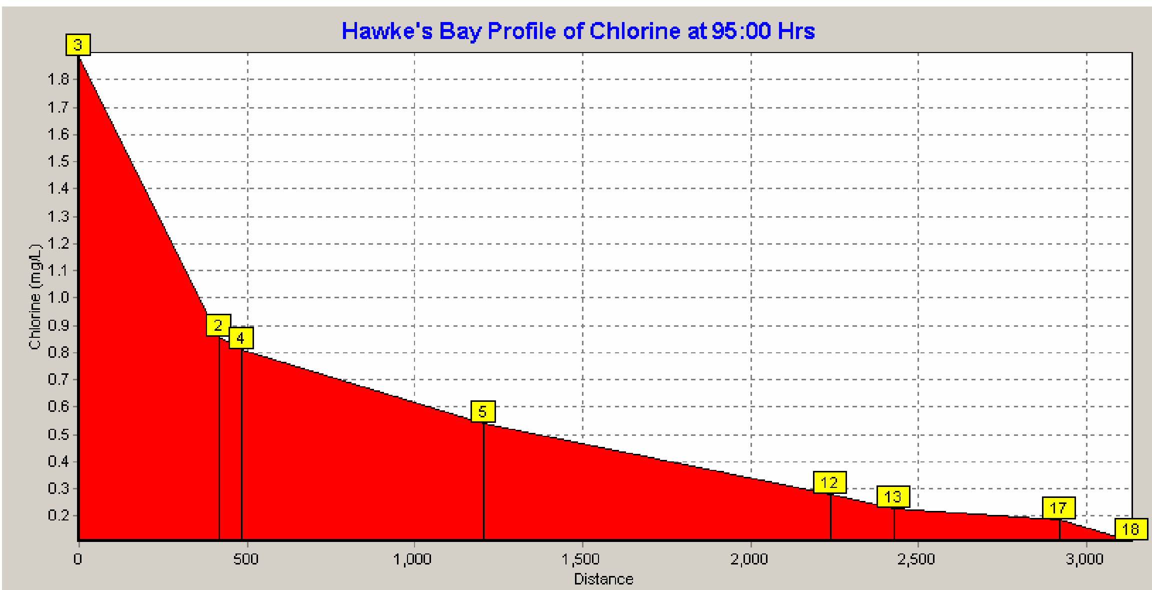


Figure 143: Chlorine decay profile through Hawke's Bay distribution system

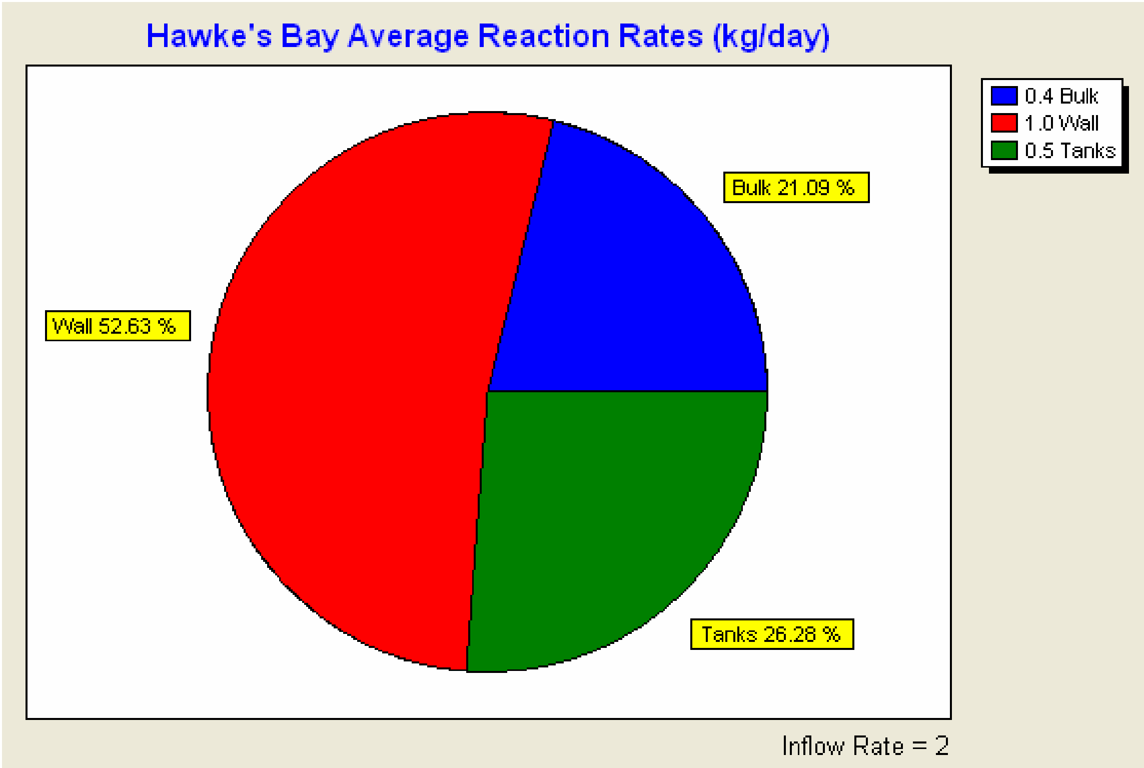


Figure 144: Chlorine decay contributions in Hawke's Bay distribution system

Solutions that might address the probable causes of high THM levels in the Hawke's Bay distribution system are outlined in the following table. Those corrective measures highlighted in grey are the only solutions that can potentially be modeled.

Table 109: Applicable THM corrective measures for Hawke's Bay

Applicable Corrective Measures	Probable Causes Addressed
Policy of POU/POE treatment	All
Policy to promote use of alternative disinfectants	All
Policy to promote PWDU	All
Watershed protection	All
High quality water storage and recovery	All
Alternative disinfectants	All
System maintenance	All
Potable water dispensing unit	All
Point of use/entry treatment	All
Training	All
Improved design of system	All
Regionalization	All
Alternative water sources	3-5-6
Relocate intake to deeper water	2
Wind breaks around exposed coastal sources	3-6
Re-locate chlorination system	2-3-5-6-14-21
Chlorine dose control	3-5-6
Tank location	21

Loop distribution network	13
Reduce storage capacity/adjust pump schedule	22
Replace or reline pipe	18
Regular system flushing at dead ends	2-5-16
Continuously bleed system at dead end	2-5-16
Downsizing mains	2-5-12-16
Water treatment plants	5
Filtration	5
Advanced treatment	3-5-6
Combination of corrective measures	All

11.10 Results from the Hawke's Bay Model Scenarios

The next step was to model the different selected corrective measures and see how the Hawke's Bay distribution system responded. Given the ability of the baseline model to reflect current conditions fairly accurately, a reasonable degree of confidence can be placed in the scenario results.

11.10.1 Relocate Chlorination System After Tank

Relocating the chlorination system after the tank also requires reconfiguring the distribution network so that the lateral line located before the storage tank is connected after the tank.

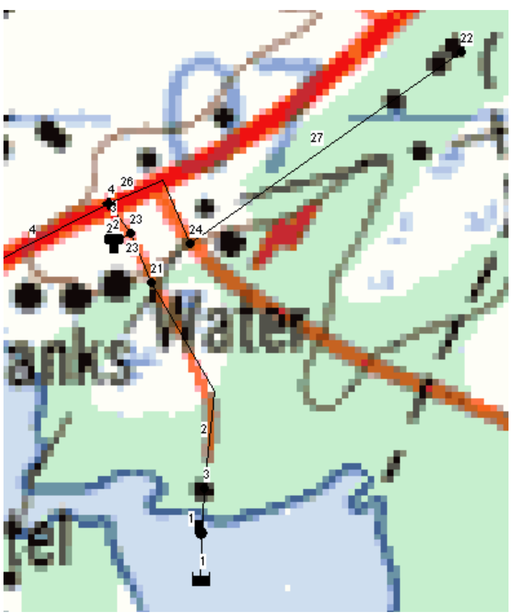


Figure 145: Network configuration with lateral down-pipe of tank

Using a chlorine dosage of 3 mg/L at node 4, adequate residuals were observed in all parts of the system. The contact time at the first user at peak daily flow was only 14 minutes, however, the CT factor value was 14, which is adequate. If the reconfigured lateral line is resized to 75 mm, both the contact time (31 min) and CT (31) value are adequate. There was also much less variation observed in chlorine residuals. By bypassing the tank, the average time available for THM formation in the Hawke's Bay system is cut by 8.2 (range 0.5-17) hours.

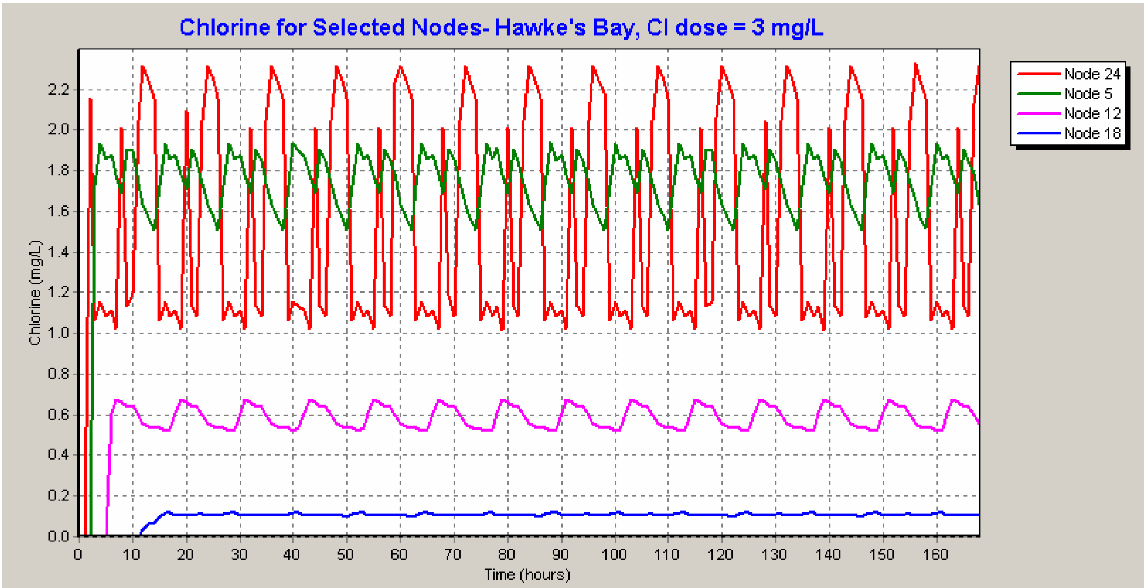


Figure 146: Chlorine residuals in Hawke's Bay Network with chlorination system moved after tank

11.10.2 Chlorine Dosage Control

The chlorination system only functions when the pump is operational, injecting a constant chlorine dose (of 5.1 mg/L) with a constant flow of 12.1 L/s for the 6.5 hours the pump is typically operational. As there is currently no variation in flow from the pump, a manual constant chlorine dose is adequate for the current set up. Water quantity (flow) and quality (chlorine residual) feedback controls could be used to manage the chlorine dosage if the existing chlorination system was upgraded or the location moved. EPANET can only be used to model flow control of the chlorine dosage.

For this simulation, the chlorination system was located down-pipe of the tank and the network configuration was altered the same as for the previous scenario. The chlorine dose was varied using a time pattern so that the chlorine dosage increased with flow, and then decreased with flow. Chlorine values mimic the peaks and lows of flow values, but there is usually a lag time between the peaks and troughs which increases the further you get towards the end of the distribution system. The following two graphs look at chlorine readings throughout the network if chlorine dosage is controlled increasing proportional to flow and decreasing proportional to flow. There is much greater variation in chlorine residuals with flow control when compared with a constant chlorine dosage.

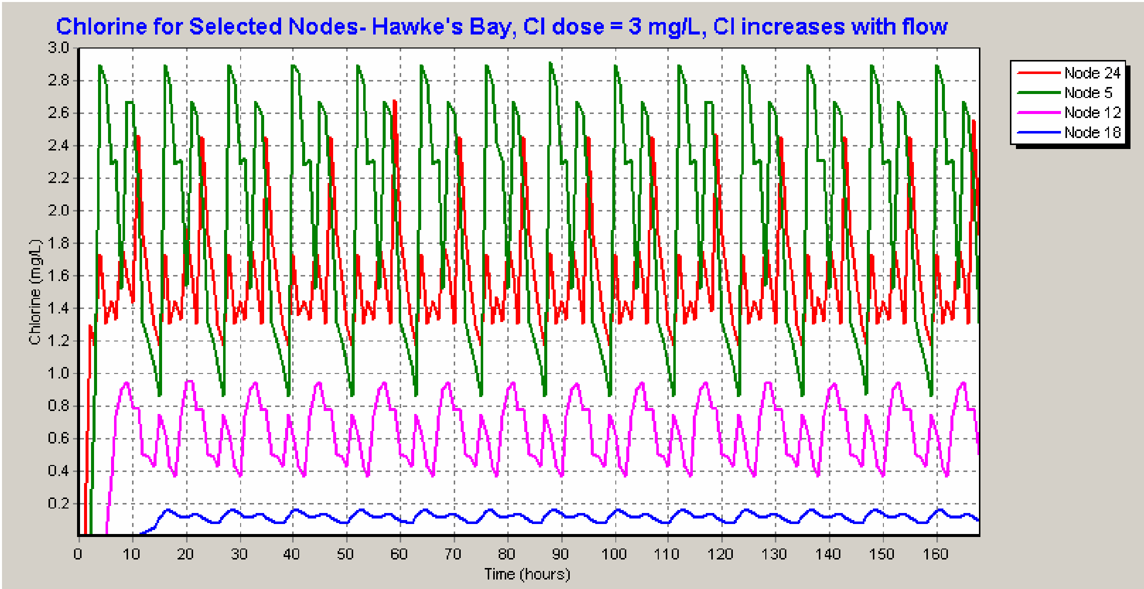


Figure 147: Chlorine control proportional to flow in Hawke's Bay network

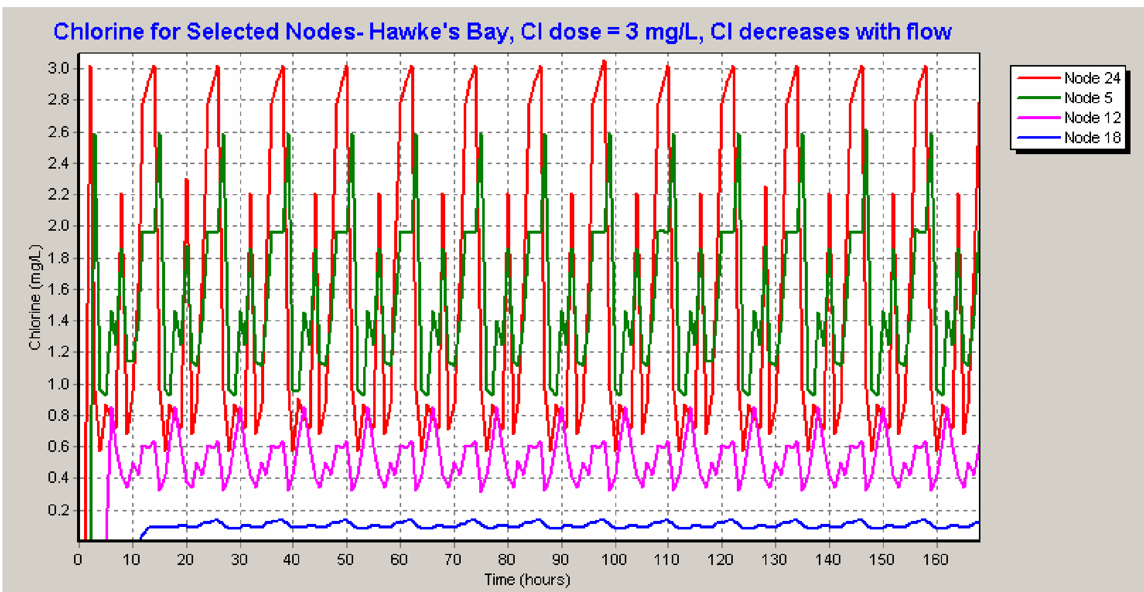


Figure 148: Chlorine control inversely proportional to flow in Hawke's Bay network

For Hawke's Bay, the lag time between peaks in flow and the corresponding peak in chlorine was on the range of 1-13 hours from the beginning to the end of the system. The lag time is the same for a constant dose as for increasing chlorine proportional to flow, only the peaks and lows in the chlorine residual are more extreme. As the lag time varies from one point to the next in the distribution system, there is no optimal time pattern that can bring about any reduction in the chlorine dosage. Primary and secondary disinfection requirements were met with both flow proportional and inversely proportional control of the chlorine dosage.

11.10.3 Tank Location

For this scenario the existing tank was relocated towards a high point off of node 19 towards the end of the system. This new location provides an increase in tank elevation of 25 meters. The calibrated pump power of 5kW was insufficient for this new system configuration so the pump power was increased to 8 kW (the pump is rated to 11 kW). At this pump capacity, instantaneous flow is kept at around 12 L/s and the tank is on a 12-hour filling/ 8 hour emptying cycle.

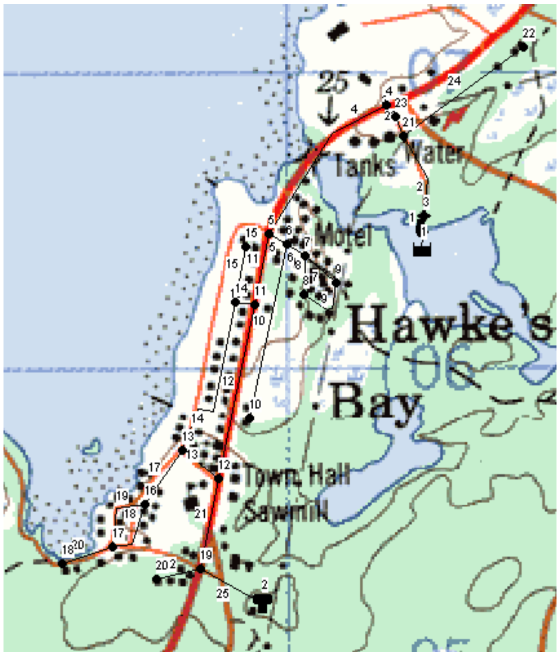


Figure 149: System reconfigured with tank at end of system

With the tank at the end of the system and keeping the chlorine dose at 4 mg/L, chlorine residuals vary wildly, and fall below 0.05 mg/L in most parts of the system when the tank is feeding the network. Even increasing the chlorine dosage to 15 mg/L is not enough to provide sufficient chlorine residuals. Wall chlorine decay increases significantly, while chlorine decay in the tank decreases significantly. With the tank at the end of the system, the maximum water age in the tank increases to 31 hours (up from 17 hours), while the maximum water age in the system becomes 37 hours (up from 29 hours). There is an increase in water velocity by approximately 0.05 m/s in the truck main leading up to the tank, and system pressures are increased ranging from 46-72 m (up from 21-44 m), which is slightly over the acceptable maximum level.

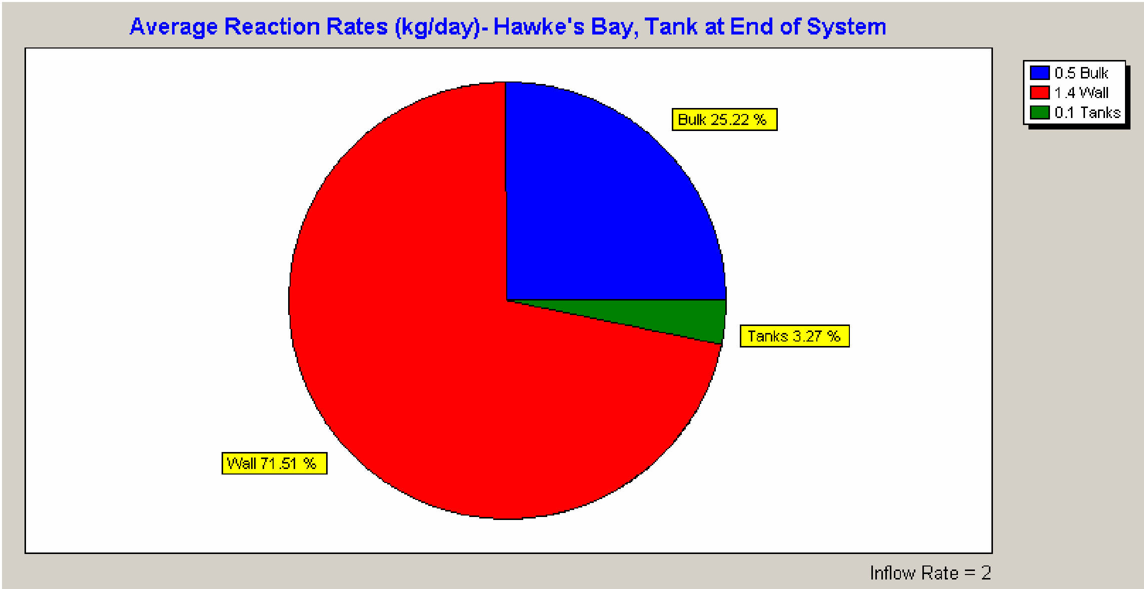


Figure 150: Chlorine decay rates with Hawke's Bay tank at end of system

11.10.4 Reducing Tank Storage Capacity/Adjusting Pump Schedule

Water quality degrades as a result of long residence times in storage tanks; chlorine residuals decrease with increased residence times, while disinfection by-products (DBPs) such as THMs increase. The turnover rate of water in the Hawke's Bay tank is high with the average water age in the tank approximately 7 hrs. The full volume of the tank is not currently being used to allow for water expansion in the case of freezing in winter. Of the volume that is being used (approximately 84% of the total tank volume), it is 100% active allowing for complete mixing and turnover. 30% of chlorine decay occurs in the storage tank.

In this scenario, the actual storage volume being used in the tank was reduced to see what affect this had on chlorine residuals and water age throughout the system. The chlorine dosage was kept at 4 mg/L. From the model scenarios, decreasing the active tank volume being used will significantly reduce water age in the tank and throughout the system. Even minimum chlorine residuals at the first point of use and at the end of they system showed some improvement, indicating potential to reduce the overall chlorine dose. Decreasing the active volume of the tank means that the pumps will be starting up and stopping more frequently, however.

Table 110: Effect of varying water levels in Hawke's Bay tank

Active Tank Volume (%)	Dead Tank Volume (%)	Max Water Level (m)	Average Water Age in Tank (hrs)	Average Water Age at End of System (hrs)	Pump/ Chlorine Pulse Cycle (times/day)	Min Cl at Node 22/18 (mg/L)
10	90	1.6	2.8	14.5	8	0.18/0.08
25	75	2.7	3.9	15.8	5	0.15/0.07
35	65	3.25	4.2	16.2	4	0.12/0.06

50	50	4.1	5.8	16.2	3	0.07/0.05
60	40	4.6	6.8	17.2	2	0.06/0.04
70	30	5.2	6.7	18	2	0.08/0.04
84	16	6.1	7	24	2	0.05/0.03

Even under current operating conditions, system pressures at high points in the distribution network fall below design guidelines. Reducing the effective tank volume for storage will only aggravate current pressure problems.

11.10.5 Replacing or Relining Pipe/ Downsizing Mains

The Hawke's Bay distribution network is oversized for the demand placed on the system. Pipe sizes range from 250-50 mm with the majority of laterals sized at 150 mm in order to fit fire hydrants. The maximum observed water velocity in the system is 0.29 m/s, observed in the section of pipe from the intake to the tank.

The pipe in the Hawke's Bay distribution network ranges in age with some sections over 25 years. For the first scenario, all pipes in the network were modeled as brand new, reflected in the input pipe roughness coefficient value. All pipes were given a Hazen-Williams C value of 150 for new PVC pipe. The model results indicated a very slight improvement in chlorine residuals throughout the network (increase of 0.01-0.02 mg/L).

For the second scenario, each pipe was resized so as to achieve a peak velocity of approximately 0.4 m/s or a minimum pipe size of 40 mm. Under these criteria, pipe sizes in the Hawke's Bay distribution system now range from 40-200 mm. The resulting maximum water age in the system now becomes 18 hours (reduced from 29 hours). Pressures throughout the network have decreased with minimum pressure in the system down from 21.3 m to 14.4 m. Chlorine readings at the end of the system fall below 0.05 mg/L and are lower then for the existing system. With reduced pipe diameter we end up with a new contact time of 31 minutes, and an equivalent CT value of 0.6.

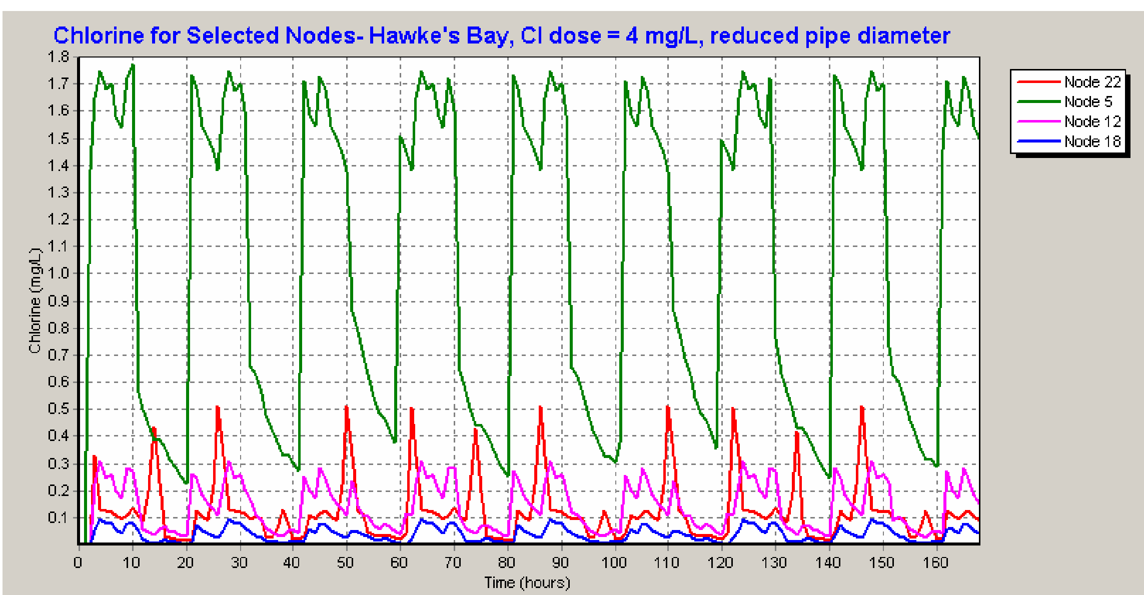


Figure 151: Chlorine levels in Hawke's Bay distribution network with reduced pipe diameter

The rate of reaction of chlorine at the pipe wall is inversely related to pipe diameter, so the smaller the pipe diameter, the greater the pipe wall reaction rate and the greater the amount of chlorine consumed at the pipe wall. Even though there is a significant decrease in water age throughout the system, the overall chlorine dosage would have to be increased in order to achieve adequate residuals at the end of the system, which could potentially offset any reduction in DBPs.

11.10.6 Reconfiguring the Distribution System through Looping

For this scenario, 4 additional pipes were included in the network to incorporate dead ends into loops. With water moving through the distribution system so slowly, particularly along these dead ends, there is plenty of time for chlorine to decay and for DBPs to form.

Figure 152: Looping of the Hawke's Bay distribution network

With the system looped, average water age at the very end of the system fell from 29 hours to 22 hours. There was no discernable improvement in chlorine residuals, however.

11.10.7 Regular System Flushing/ Continuously Bleed at System End

As the maximum water age in the distribution system is 29 hours, system flushing would have to take place at a frequency less than this in order to achieve any improvement in water age, ideally at half the current return period or every 14.5 hours. For this corrective measure, four scenarios were looked at: flushing once a day at the end of the system, flushing once a day at each dead end, flushing twice a day at each dead end, and a continuous bleed at the end of the system. The average daily flow rate (demand) in the network is 6.61 L/s, and flushing rates will be some multiple of this. Pressure throughout the distribution system is barely adequate at current demand levels, and negative system pressures are experienced at approximately 2 times the average daily flow rate at current pump capacity. Even at the maximum flushing rate the system is capable of, it is impossible to reach a flushing velocity of 0.75 m/s.

Flushing once a day at the end of the system was achieved by placing a large demand (9.5 L/s) at the end of the system (node 18) for 4 hours every night. Average water age at the end of the system was reduced to 12 hours where the flushing occurred. However, this reduction in water age with flushing did not occur throughout the distribution system due to the branched nature of the network. Chlorine residuals tended to increase slightly, but again, only in the direct path of the flushing.

A scenario where flushing occurs once a day at each of the five dead ends was also examined. Base demand at each dead end node was increased by a factor of 10 resulting in an additional 12 L/s instantaneous demand on the entire system for 4 hours every night. Average water age at dead end nodes was reduced from between 2 to 10.5 hours. Chlorine residuals improved throughout the distribution network increasing by 0.03 to 0.39 mg/L depending on the node. Improvements were most pronounced towards the middle and end of the system.

Alternatively, a scenario where flushing occurs twice a day at each of the five dead ends was examined. Base demand at each dead end node was increased by a factor of 5 resulting in an additional 6 L/s instantaneous demand on the entire system for 4 hours at 12 hour intervals. Average water age at dead end nodes was reduced from between 3 to 10.5 hours. Chlorine residuals improved throughout the distribution network increasing by 0.03 to 0.31 mg/L depending on the node. There was noticeably less variation in chlorine residuals for this scenario.

For the continuous bleed scenario, an additional constant demand of 6 L/s was placed on the end node (node 18). With more demand at the end of the system, water moves faster through the distribution network and the tank filling/emptying cycle changes to 10 hours for filling, 7 hours for emptying. Minimum chlorine readings at the end of the system increase to 0.20 mg/L (from 0.02 mg/L), and there is an improvement in chlorine residuals throughout the middle and end portions of the system. Maximum water age is also reduced throughout the system (to 11 hours at node 18). With a continuous bleed, the chlorine dose can be reduced from 4 mg/L to 2.5 mg/L and adequate chlorine residuals still be maintained throughout the network.

Manual flushing once a day (or more) at multiple dead ends in the Hawke's Bay distribution system may not be a practical use of resources; however, use of automatic hydrant flushing units could help. A flushing program with a flushing frequency of more than once a day is also not practical. Continuously bleeding a system is wasteful of resources (water, energy costs, chlorine) and may harm the receiving environment. The benefits of each of the option must be examined in the context of its various disadvantages in the case of flushing corrective measures.

11.11 Impact of Modeled Corrective Measures

Of the 9 corrective measures identified in a previous section that could be modeled in EPANET in order to access their impact in terms of improving water quality (looking at chlorine, water age, and potential THM formation), two were grouped together with other related scenarios. Not all scenarios met the required criteria in order to be deemed successful. Any scenario that saw a reduction in the overall chlorine dosage and a decrease in water age has potential for lowering THM levels. The following table highlights which scenarios had a positive impact on water quality.

Table 111: Modeled scenarios for the Hawke's Bay network and their effectiveness

	Scenario Description	All Criteria Met	Comments
1	Relocate Chlorination System After Tank	Yes	-Potential to reduce overall Cl dose slightly -lateral line to 1st user must be reconfigured
2	Chlorine Dosage Control	Yes	-Greater Cl variability -No potential to reduce overall Cl dose
3	Tank Location	No	-Secondary disinfection criteria not met
4	Reducing Tank Storage Capacity/ Adjusting Pump Schedule	No	-Aggravates existing pressure problems in system -Reducing active tank volume significantly lowers water age and potential for lower Cl dose
5	Replacing or Relining Mains/Downsizing Mains	No	-Minimal improvement in Cl residuals with replacement/ relining -Downsizing mains aggravates existing pressure problems -Water age will decrease, but higher Cl dose required
6	Reconfiguring Distribution System through Looping	Yes	-Water age decreased slightly -No improvement in Cl residuals
7	Regular System Flushing at Dead Ends/ Continuously Bleed System	Yes	-Water age decreased while Cl residuals increased -Overall Cl dose can be reduced

Any corrective measures that did not meet the necessary criteria should be dropped from consideration and evaluated no further. Scenarios that saw potential for the overall chlorine use to be reduced and water age in the distribution system lowered will be the most effective in terms of lowering THMs. Based on this assessment, the corrective measures (that met criteria) with the most potential for reducing THM formation are:

- Relocating the chlorination system after the tank
- Regular system flushing at dead ends/ continuously bleed system
- Reconfiguring the distribution system through looping

It must be stated that no alternative meets the requirement for 20 minute contact time at peak flow, or maintains acceptable free chlorine residuals (or equivalent CT value) at the first point of use unless the lateral line to the first user is reconfigured and resized.

11.12 Assessment of Corrective Measure Constraints for Hawke's Bay Network

The following table evaluates each remaining corrective measure for the Hawke's Bay water distribution system against identified solution constraints. The selection of the preferred solution(s) to water quality problems can be made based on the corrective measure(s) with the highest score(s).

Based on the resulting scores, there are three main tiers of possible solutions. The top three tiers in the decision matrix scoring system comprise the corrective measures that have the most potential for effectively optimizing chlorine dosage, reducing water age and lowering THMs.

The first tier, which scored 14, consists of installing a Potable Water Dispensing Unit. The second tier of solutions, which scored 13, consists of the general best management practice of improving system design, and the "hard" solution of looping the distribution network. The third tier of corrective measures, which scored 13, consists a mix of "soft" practices such as operator education and training, adaptive policy change to promote use of PWDUs, and watershed protection, and more technical or "hard" solutions such as regular system flushing at dead ends, relocation of the chlorination system, and relocating the intake in deeper water.

The selection of a preferred solution by the decision making body (town, engineering consultant, Department of Municipal Affaires) can be guided by this decision matrix. The next step in the process involves the implementation of the preferred solution, monitoring and review.

Table 112: Assessment of solution constraints for Hawke's Bay

Applicable Corrective Measures	Effectiveness	Cost	Time Scale for Implementation	Permanency of Solution	Adverse Hydraulic Impacts	Adverse WQ Impacts	Acceptable to Stakeholders	Meets Regulations	Total
Policy of POU/POE treatment	1	2	0	0	1	2	1	1	8
Advanced treatment	2	0	0	2	1	2	0	2	9
Alternative water sources	1	0	0	2	1	2	1	2	9
Combination of corrective measures	2	0	0	2	1	1	1	2	9
High quality water storage and recovery	1	0	0	2	1	2	1	2	9
Policy to promote use of alternative disinfectants	1	2	0	1	1	1	1	2	9
Alternative disinfectants	2	1	1	1	1	1	1	2	10
Chlorine dose control	0	2	1	1	1	1	2	2	10
Regionalization	1	1	1	2	1	1	1	2	10
Water treatment plants	2	0	0	2	1	2	1	2	10
Wind breaks around exposed sources	0	2	0	2	1	1	2	2	10
Continuously bleed system at dead end	1	2	1	1	2	2	1	1	11
Filtration	1	1	1	1	1	2	2	2	11
Point of use/entry treatment	2	2	2	0	1	2	1	1	11
System maintenance	1	2	2	0	1	2	1	2	11
Policy to promote PWDU	1	2	0	2	1	2	2	2	12
Regular system flushing at dead ends	1	2	1	1	2	2	2	1	12
Relocate chlorination system	1	2	1	1	1	2	2	2	12
Relocate intake in deeper water	1	1	1	2	1	2	2	2	12
Training	1	2	0	1	2	2	2	2	12
Watershed protection	0	2	1	2	1	2	2	2	12
Improved design of systems	1	2	0	2	2	2	2	2	13
Loop distribution network	1	1	1	2	2	2	2	2	13
Potable water dispensing unit	2	2	1	2	1	2	2	2	14

12.0 Summary of Modeled Water Distribution Systems

The purpose of developing several water distribution system models using EPANET was to predict how generic types of systems with THM issues responded to certain corrective measures. The generic types of systems that tend to display THM issues can be identified from source water factors, DBP precursor levels, system design, water demand variability, presence of storage tanks, and operation and maintenance practices. Only a subset of identified corrective measures could actually be modeled in EPANET including chlorine demand management, retention time management, water demand management, and operational and infrastructural measures.

Table 113 highlights the 6 communities that were modeled and provides a comparison of system characteristics that can be used to highlight probable causes that may be contributing to high THM levels. As is evident by the system characteristics, each community water system is unique and relying on generic assumptions can be problematic. The probable causes of high THMs are not always evident from a cursory review of the distribution system, but the list developed provides a fairly comprehensive assessment.

With the help of the 6 models that were developed, quantifiers have been identified for each probable cause (where applicable) at which threshold THM problems are more likely to develop. These quantifiers can be used to assess likely causes of high THMs in other communities.

Probable causes that were common to all 6 modeled communities included: high DOC in source water, excessive chlorine demand, inadequate operation and maintenance of the distribution network, unsuitable system design, and multiple contributing factors. Other identified probable causes were more dependent on the site-specific characteristics of the individual water distribution network.

Through modeling, the response of the water distribution system to different corrective measures can be evaluated without actual implementation. Again, the success of a corrective measure is very much dependant on the site-specific characteristics of the network. In some cases, most single corrective measures examined offered some improvement; in others, multiple corrective measures resulting in extensive changes to the network were required.

After modeling helped weed out ineffective corrective measures, an assessment of remaining solutions against 8 identified constraints was made. This assessment was made to better match corrective measures with the needs of the community. Corrective measures that consistently placed in the top three tiers included instillation of a PWDU, operator education and training, watershed protection, and improved design of water distribution systems. The later three corrective measures are more generic and can be classified as “soft” solutions, without any technological or infrastructural requirements.

Table 113: Summary of issues for modeled water distribution systems

	Issue	Brighton	Burlington	Ferryland	Cartwright	St. Paul's	Hawke's Bay
1	Reservoir contains flooded vegetation	yes	-	-	yes	-	-
2	Shallow intake	? m into water ? m below surface	? m into water 1 m of water	50 m into water 3 m of water	45 m into water ? m below surface	40 m into water ? m below surface	? m into water yes- ? m below surface
3	Surface water source exposed to ocean salt spray	300 m (NW)	3.2 km (S)	2.3 km (E)	1.5 km (NW)	1.1 km (NW)	900 m (NW)
4	Mixing of high DOC surface water with groundwater	-	-	-	-	-	-
5	High DOC in source water (>2 mg/L)	6.4 mg/L	8.19 mg/L	5.51 mg/L	9.13 mg/L	6.16 mg/L	5.1 mg/L
6	High levels of bromide in source water (>0.02mg/L)	0.027 mg/L	0.02 mg/L	0.02 mg/L	0.010 mg/L	0.03 mg/L	0.024 mg/L
7	High chlorine dose (over 7 mg/L or over 4 mg/L at first point of use)	Cl dose = 6.28 mg/L 2.93 mg/L max @ 1st user	Cl dose = 12.2 mg/L 4.88 mg/L max @ 1st user	Cl dose = 6.48 mg/L 1.92 mg/L max @ 1st user 2.65 mg/L after booster	Cl dose = 4.9 mg/L 3.75 mg/L max @ 1st user 4.82 mg/L after booster	Cl dose = 12.6 mg/L 2.66 mg/L max @ 1st user	Cl dose = 5.1 mg/L 1.37 mg/L max @ 1st user
8	Point of Cl application in WTP	-	-	-	-	-	-
9	Higher chlorine use with booster system	-	-	no Cl booster dose = 1.05 mg/L	yes Cl booster dose = 4.1 mg/L	-	-
10	Excessive chlorine demand	0.4 d-1 (bulk) 1.5 m/d (wall)	2.0 d-1 (bulk) 1.5 m/d (wall)	2.2 d-1 (bulk) 1.5 m/d (wall)	0.8 d-1 (bulk) 2.0 m/d (wall)	2.5 d-1 (bulk) 2.5 m/d (wall)	2.5 d-1 (bulk) 2.5 m/d (wall)
11	High pH	6.9	6.1	6.2	5.6	7.3	7
12	Long linear system	3.1 km intake to end total = 3.1 km	4.6 km intake to end total = 4.9 km	5.8 km intake to end total = 10.5 km	5.9 km intake to end total = 10.6 km	6.0 km intake to end total = 7.2 km	2.5 km intake to end total pipe = 6.0 km
13	Branched system with multiple dead ends	at least 1 DE	at least 4 DE	at least 5 DE	at least 7 DE	at least 6 DE	at least 5 DE
14	Distance of chlorination system to first point of use	1 km contact time (PD) = 145 min CT = 11.6	210 m contact time (PD) = 20 min CT = 81.6	925 m contact time (PD) = 63 min CT = 79.4	440 m contact time (PD) = 27 min CT = 85.4	582 m contact time (PD) = 545 min CT = 1003	880 m contact time (PD) = 10 min CT = 0.3
15	No chlorination controls on system	manual	manual	flow proportional	flow proportional	residual analyzer	manual
16	System is oversized	0.01-0.12 m/s	0.02-0.18 m/s	0-1.24 m/s	0.01-0.30 m/s	0-0.13 m/s	0.01-0.29 m/s

		250-150 mm pipe Q(average) = 1.07 L/s Q(instantaneous) = 7.15 L/s	300-75 mm pipe Q(average) = 6.13 L/s -	450-25 mm pipe Q(average) = 6.13 L/s -	200-75 mm pipe Q(average) = 5.83 L/s Q(instantaneous) = 7.57 L/s	250-100 mm pipe Q(average) = 2.55 L/s -	250-50 mm pipe Q(average) = 6.61 L/s Q(instantaneous) = 12.1 L/s
17	High retention time in network	max = 102 hrs	max = 30 hrs	max = 27.1 hrs	max = 24 hrs	max = 59+ hrs	max = 29 (34) hrs
18	Pipe material and age (>25yrs)	1986 and younger DI, PVC	1980 HDPE, PVC	older than 1988 HDPE, PVC, DI	older than 1984 HDPE, DI	1978 DI, HDPE	older than 1980
19	Water treatment plant is undersized	-	-	-	-	-	-
20	Large occasional demand on system	-	-	seasonal torism	fish plant	hotel/golf course	-
21	Tank location	beginning	-	-	-	close to beginning	beginning
22	Balance between pumped supply and demand not optimized with storage	6 hrs to fill/ 30 hrs to empty	-	-	-	4 hours to fill/ 4 hours to empty	6.5 hrs to fill/ 6.5 hrs to empty
23	High retention time in tank	max = 57 hrs	-	-	-	max = 48 hrs	max = 17 hrs
24	Dead zones/ poor mixing in tank	inlet/outlet same	-	-	-	inlet/ outlet close	inlet/outlet same
25	Little variation in water levels/ turnover in tank	25% inactive volume 3.7 m	-	-	-	85.7% inactive volume 1 m	0% inactive volume 6.1 m
26	Poor operation and maintenance of system (flushing)	Water Dist- Class I	Water Dist- Class I	Water Dist- Class I	Water Dist- Class I	Water Dist- Class I	Water Dist- Class I
27	Multiple of the above factors	-	-	-	-	-	-
28	Poor design of system	-	-	-	-	-	-
29	Iron and Manganese	Fe = 0.1mg/l Mn = 0.02 mg/L	Fe = 0.19 mg/L Mn = 0.008 mg/L	Fe = 0.08 mg/L Mn = 0.013 mg/L	Fe = 0.44 mg/L Mn = 0.009 mg/L	Fe = 0.36 mg/L Mn = 0.023 mg/L	Fe = 0.1mg/l Mn = 0.006 mg/L
30	Per capita demand	398 L/p/d	450 L/p/d	873 L/p/d	721 L/p/d	667 L/p/d	1283 L/p/d
31	System pressure at demand nodes	min = 57 m max = 63 m	min = 35.2 m max = 59.2 m	min = 35.0 m max = 96.3 m	min = 45.8 m max = 65.8 m	min = 21.1 m max = 56.2 m	min = 21.3 m max = 44 m
32	Chlorine residuals at system end	0.08 - 0.26 mg/L	0-0.02 mg/L	0.01-0.09 mg/L	0.04-0.11 mg/L	0 mg/L	0.02 - 0.12 mg/L
	Chlorine residuals at 1st user	1.2-3.0 mg/L	4.08-4.88 mg/L	1.26-1.92 mg/L	3.20-3.75 mg/L	1.84-2.66 mg/L	0.03-1.4 mg/L
	Chlorine residual after booster	-	-	2.04-2.65 mg/L	4.46-4.82 mg/L	-	-
	Watershed size	0.44 km2	13.1 km2	1.17 km2	12.9 km2	7.44 km2	616 km2
	Pumped or gravity	pumped	gravity	gravity	gravity	pumped	pumped
	Population serviced	203	309	529	552	376	391
	Fire hydrants	14	11	at least 13	at least 5	13	alt least 9

12.1 Lessons Learned from Distribution System Modeling

With each model that was developed, and each corrective measure tested out in that model, new understanding, both site-specific and generic, of the behaviour of water distribution systems was developed. The following items are some of the lessons gathered from the modeling exercise.

- There is much greater variation in chlorine residuals with flow control when compared with a constant chlorine dosage.
- Chlorine wall decay is excessive and the leading contributor to overall chlorine decay in the distribution system.
- Overall water age is decreased by having the water storage tank at the beginning of the system.
- In reducing the pipe diameter, the pipe wall reaction rate increases, and a greater amount of chlorine is consumed at the pipe wall.
- Looping produces better system improvements on networks that do not display overcapacity or excessive water age.
- Manual or automated flushing should be targeted preferentially towards dead ends with low demand.
- A manual flushing program with a flushing frequency of more than once a day is not practical.
- Continuous bleeds are most appropriate on linear systems or systems with overcapacity.
- System flushing is more appropriate on distribution systems that are over-designed with excess capacity.
- Flushing or bleeding the system is not practical where the distribution network has a contact time at peak flow close to 20 minutes.
- Every distribution system is unique and responds differently to different possible corrective measures.
- Some distribution systems do not respond positively in terms of meeting required system criteria to either Chlorine Demand Management (CDM) or Retention Time Management (RTM) corrective measures.
- On long distribution systems, chlorine boosters should be located relatively close to population clusters or more densely populated areas.
- Even with the corrective measures applied to the distribution system, the response may not be positive enough to completely correct DBP issues.
- Water distribution systems have improved water quality with increased dead volume, decreasing inactive water volume, and decreasing active water volume in the water storage tank.
- Probable causes of high THMs are not always evident from a cursory review of the distribution system.

12.2 Draft BMPs to Reduce Disinfection by Products for New, Upgrading and Existing Water Distribution Systems

One of the main focuses of this report has been to develop a set of BMPs that can be used to help reduce THMs and other DBPs for new, upgrading and existing water distribution systems. These BMPs have been shaped by the understanding developed of THM characteristics and behaviour, the assessment of various corrective measures, and through modeling of water distribution systems. A draft of these BMPs for the control of DPBs is located in Appendix B.

13.0 Integrated Decision Making Framework for Selecting DBP Corrective Measures

There is no standard solution that will address the issue of high DBP levels in drinking water for all communities. There are numerous probable causes that may be contributing to the formation of DBPs as identified in this report, just as there are numerous potential corrective actions that can be taken to address the problem. The difficulty lies in selecting the most appropriate corrective measure in light of what might be contributing to DBP levels. The selected corrective measure must address the issue of DBPs, but it must also fit the community involved in terms of available resources and other solution constraints. Once a preferred corrective measure is selected and implemented, further monitoring and review is required to make sure that the DBP problem has been corrected by the action taken. The following figure outlines the decision making process for the selection of DBP corrective measures using THMs as an example. Each step in the process will be discussed further in this section.

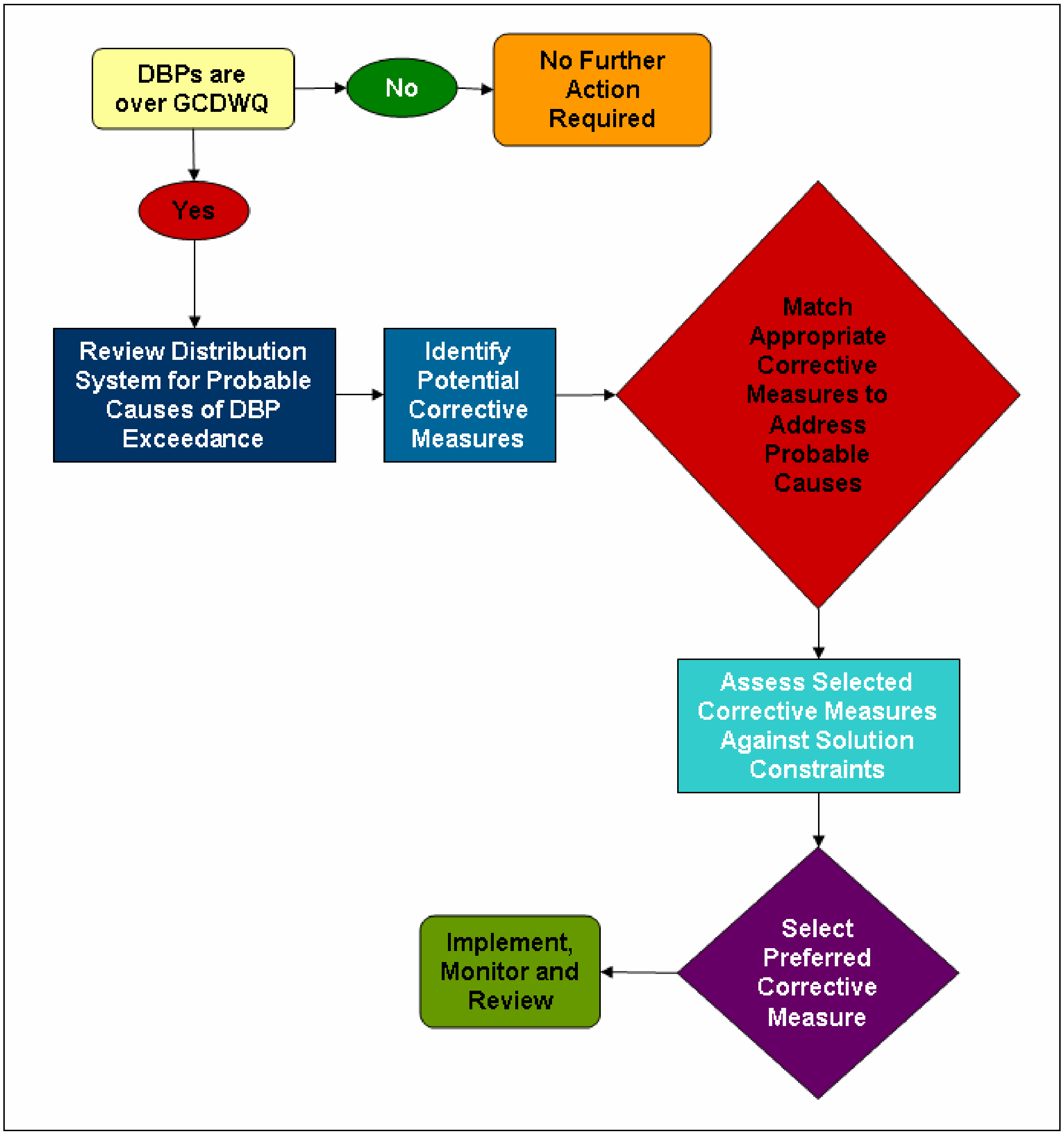


Figure 153: Decision making framework for the selection of DBP corrective measures (using THMs)

The development of the decision making framework for addressing DBP issues has been an iterative process based on known DBP formation behaviour and best management practices used to deal with DBPs in other jurisdictions; assessment of DBP characteristics and response to existing corrective measures in Newfoundland and Labrador; and through modeling of several water distribution systems that are experiencing DBP problems in the province. An expanded decision making framework is located in Appendix C. Although the framework developed has been tailored towards addressing THM issues, the approach is a holistic one that can be used to mitigate issues with other DBPs.

13.1 Review Distribution System for Probable Causes of THM Exceedance

There are numerous factors that can potentially be contributing to the formation of THMs at levels above the guideline value. In most cases it will be some combination of factors that is resulting in high THMs. The main contributing factors are outlined in the following tables and include:

- Source water factors
- Presence of DBP precursors
- Distribution system characteristics
- Water demand
- Water storage tanks
- Operation and maintenance practices
- Other

Table 114: Probable causes of high THMs relating to source water factors

Factor	Probable Cause	Qualifier
Source	1. Reservoir contains flooded vegetation	
	2. Shallow intake or shallow intake pond with long exposed fetch length	• Less than 1 m of water
	3. Surface water source exposed to ocean spray or other salt water influence	• Less than 1 km to ocean

Table 115: Probable causes of high THMs relating to DBP precursors

Factor	Probable Cause	Qualifier
DBP Precursors	4. Mixing of high DOC surface water with groundwater	
	5. High levels of DOC in source water	• Greater than 4.2 mg/L
	6. High levels of bromide in source water	• Greater than 0.02 mg/L
	7. High chlorine dose	• Total dose over 7 mg/L • Greater than 4.0 mg/L at first point of use
	8. Point of chlorine application in WTP	• Pre-chlorination
	9. Higher chlorine use with booster chlorination system	
	10. Excessive chlorine demand	• Less than -0.5 d^{-1} • Less than -1 m/d
	11. High pH	• Greater than 7

Table 116: Probable causes of high THMs relating to the distribution system design

Factor	Probable Cause	Qualifier
Distribution System Characteristics	12. Long liner system	• Greater than 3 km
	13. Branched system with multiple dead ends	• More than 3 dead ends
	14. Distance of chlorination system to	• Greater than 500 m

	first point of use	• Contact time greater than 40 minutes
	15. Insufficient chlorine controls on system	
	16. System is oversized	• Velocity in all pipes less than 0.4 m/s
	17. High retention time in network	• Greater than 48 hours
	18. Pipe material and age	• Cast iron • Greater than 25 years
	19. Water treatment plant is undersized	

Table 117: Probable causes of high THMs relating to demand on the distribution system

Factor	Probable Cause	Qualifier
Demand	20. Large occasional demand on system	

Table 118: Probable causes of high THMs relating to tanks on the distribution system

Factor	Probable Cause	Qualifier
Tanks	21. Tank location/ configuration	
	22. Balance between pumped supply and demand not optimized with storage	
	23. High retention time in tank	• Greater than 24 hours
	24. Dead zones/ poor mixing in tank	
	25. Little variation in water levels/ turnover in tank	

Table 119: Probable causes of high THMs relating to operation and maintenance factors

Factor	Probable Cause	Qualifier
Operation and Maintenance	26. Poor operation and maintenance of distribution system	

Table 120: Other probable causes of high THMs

Factor	Probable Cause	Qualifier
Other	27. Multiple factors listed	
	28. Poor design of distribution system	
	29. High iron and manganese	• Iron greater than 0.3 mg/L • Manganese greater than 0.05 mg/L
	30. High per capita demand	• Greater than 500 L/p/d
	31. Pressure problems	• Greater than 66 m • Less than 28 m
	32. Problems with chlorine residuals	• Less than 0.05 mg/L at end of system •Greater than 4.0 mg/L anywhere in system

For each water distribution system that experiences THM issues, this list of probable causes should be reviewed to see what factors are possibly triggering the problem. With probable causes identified, it is then easier to identify potential corrective measures.

13.2 Identify Potential Corrective Measures and Match to Appropriate Probable Causes

Corrective measures to deal with THM problems were reviewed in depth in a previous Section 4 of this report. Identified broad-based corrective measure categories are outlined in the following tables and include:

- Policy measures
- Source based control measures
- Chlorine demand management (CDM)
- Retention time management (RTM)
- Water demand management (WDM)
- Water distribution system operational and infrastructural measures
- Alternative disinfectants
- Source water treatment
- Point of use/point of entry measures
- Operator education and training
- Water system design measures

Matching appropriate corrective measures to probable causes of high THMs requires an assessment of which corrective measures can adequately address the specific problems involved. The effectiveness of potential corrective measures ranges from low to high, depending on the measure involved and site-specific conditions. Water treatment provides the best blanket option for reducing THMs, while chlorine demand management options (for example) may be effective in some communities but not in others.

Table 121: Policy related corrective measures and the probable causes of high THMs they address

Category	Corrective Measure	Probable Causes Addressed
Policy	Policy of POU/POE treatment	All
	Policy to promote use of alternative disinfectants	All
	Policy to promote PWDU	All

Table 122: Source based corrective measures and the probable causes of high THMs they address

Category	Corrective Measure	Probable Causes Addressed
Water Source	Watershed protection	All
	Alternative water sources: • groundwater • surface water sources with DOC less than 4.2 mg/L • avoid shallow ponds with long exposed fetch lengths	1-3-4-5-6
	Stop mixing groundwater with surface water in the	4

	distribution system	
	Reservoir flooding: • avoid flooding vegetated areas • remove vegetation before flooding • remove submerged vegetation	1-4-5
	Wind breaks around exposed coastal water sources (with BDCM greater than 16 µg/L)	3-6
	Relocate intake to deeper water	2
	High quality water storage and recovery	All

Table 123: Chlorine demand management measures and the probable causes of high THMs they address

Category	Corrective Measure	Probable Causes Addressed
CDM	Optimize disinfectant dosage if: • Cl greater than 4.0 mg/L in system • Cl regularly greater than 0.20 mg/L at end of system • booster on system	1-2-3-4-5-6-7-9
	Re-locate chlorination system: • closer to first user • down-pipe of storage tank	1-2-3-4-5-6-7-9-14-21
	Install chlorine booster at optimal location if: • combined chlorine dose less than single chlorine dose • chlorine greater than 4.0 mg/L at first user	7-10
	Chlorine dose control: • automated flow or residual control • dedicated and certified system operator	1-2-3-4-5-6-7-9-15

Table 124: Retention time management measures and the probable causes of high THMs they address

Category	Corrective Measure	Probable Causes Addressed
RTM	Tank location and type: • at beginning of system • multiple smaller tanks • elevated storage	21-23-24-25
	Adjusting pump schedules to: • optimize balance between demand and supply • force turnover of water in tank • increase velocity of inflow into tank	17-22-23-24-25
	Reduce storage capacity by: • taking tank offline • reduce maximum water level in tank	17-20-23-24-25
	Increase mixing in tank: • separate inlet/outlet • baffles, location, orientation of inlet	17-23-24-25

	• smaller diameter inlet, duckbill valve at inlet • mechanical mixing devise • avoid stratification in tank • increase active volume in tank	
	Tank aeration	17-23-24-25

Table 125: Water demand management measures and the probable causes of high THMs they address

Category	Corrective Measure	Probable Causes Addressed
Water Demand Management	Regular system flushing at dead ends: • automated flushing devise • manual flushing	1-2-5-12-13-16-17-20
	Continuously bleed system at dead ends	1-2-5-12-13-16-17-20
	Increase demand with new water connections	1-2-5-12-13-16-17-20

Table 126: Operational and infrastructural measures and the probable causes of high THMs they address

Category	Corrective Measure	Probable Causes Addressed
Operational and Infrastructural	Optimize valve arrangement: • minimize number of shut valves • locate shut valves in areas of high demand	17
	Re-routing of flows in the system through valving	17
	Pumping to re-circulate water in the distribution system	17
	Abandoning or downsizing mains	1-2-5-12-13-16
	Clean, replace or reline: • old pipe • cast iron pipe	18
	Loop distribution network	13
	System maintenance: • flushing, reservoir cleaning • swabbing or pigging • pump, flowmeter maintenance	All (26)
	Increase capacity of WTP	19
	Regionalization: • regional systems • regional operators	All

Table 127: Alternative disinfection measures and the probable causes of high THMs they address

Category	Corrective Measure	Probable Causes Addressed
Alternative Disinfectants	Chloramines	All
	Ozone	All
	Chlorine Dioxide	-

	UV	All
	MIOX	All

Table 128: Water treatment measures and the probable causes of high THMs they address

Category	Corrective Measure	Probable Causes Addressed
Water Treatment	Water treatment plant: • conventional WTP • ultrafiltration	4-5-7-9
	Point of chlorine application in WTP: • use alternative pre-disinfectant • no pre-chlorination	8
	Filtration: • ultrafiltration or nanofiltration •appropriately sized and maintained	4-5-7-9
	pH adjustment	11
	Iron and manganese removal	7-9-10-29
	Advanced treatment for large systems: • EC, RO, GAC for natural organic material • RO, EDR and IX for bromide	3-4-5-6-7-9
	Potable water dispensing unit (PWDU): • community support	All

Table 129: Other corrective measures and the probable causes of high THMs they address

Category	Corrective Measure	Probable Causes Addressed
Point of Use	POU/POE treatment	All
Training	Operator education and training	All (26)
	Operator certifiction	All (26)
Design	Improve water distribution system design	28
Combined	Combination of corrective measures	All

Specific water distribution system design improvements are discussed in the section on BMPs to reduce disinfection by-products for new, upgrading and existing water distribution systems (Section 12.2), and outlined in Appendix B and in Section 4.16.

Depending on site-specific conditions, a number of corrective measures may have the potential to reduce THM levels in a community's water distribution system. To assess which corrective measure is best suited to a community's needs, an assessment of each potential solution against a set of decision making constraints is required.

13.3 Assessment of Selected Corrective Measures Against Solution Constraints

In order to assess the suitability of potential corrective measures to address THM problems in specific communities, a scoring mechanism is recommended that will weigh each corrective measure against a fixed set of solution constraints. The scoring mechanism is meant to identify which corrective measures will best address THM issues against the following constraints:

- Effectiveness of the corrective measure to reduce THMs
- Cost
- Time scale for implementation
- Permanency of solution
- Adverse hydraulic effects
- Adverse impact on water quality
- Acceptability to stakeholders
- Meets all necessary regulations

The scoring of potential corrective measures is based on the scoring mechanism outlined in the following table. The highest a corrective measure can possibly score is 16. Further detail on the scoring mechanism is provided in the following sections.

Table 130: Scoring mechanism for assessment of corrective measures

Constraint	Score Zero (0)	Score One (1)	Score Two (2)
Effectiveness	Low- no substantial reduction	Moderate- near guideline	High- below guideline
Cost	High- >$500,000	Moderate- $150,000-$500,000	Low- <$150,000
Time scale for implementation	Long- over a year	Moderate- 1 to 12 months	Short- within a month
Permanency of solution	Short term- temporary	Moderate term- useful life < 15 years	Long term- useful life > 15 years
Adverse hydraulic effects	Adverse effect- violates criteria	Questionable or no impact	Positive effect- system closer to ideal
Adverse impact on water quality	WQ deteriorates- other WQ parameters deteriorate	Questionable or no impact	WQ improves- other WQ parameters improve
Acceptability to stakeholders	Against- known or perceived issues	Questionable or indifferent	Support- no issue
Meets regulations	Violates	Borderline- minor, temporary, insignificant violation	Meets

13.3.1 Effectiveness

The effectiveness of a corrective measure to reduce THMs is a matter of scale depending on how high THM levels are in the community, and by how much they can potentially be reduced. If THM levels can be brought below the guideline level of an annual running average of 100 μg/L, the corrective measure should be scored high. If THM levels are brought close to (within 25 μg/L) of the guideline value or are reduced by more than 50%, the corrective measure should be scored moderate. If there is no substantial reduction in THM levels, the corrective measure should be scored low. In evaluating effectiveness of a corrective measure, the best case scenario will be considered.

13.3.2 Cost

In order to determine the cost of various corrective measures, generic costs were determined for each. Appendix D provides a comparison of costs for each identified corrective measure. Cost levels were derived according to the classification system in the following table. Associated costs should be considered as best estimates only. The costs listed are for the most expensive option. The cost range associated with some corrective measures (CDM, RTM, alternative disinfectants) are in some cases quite considerable, and an average cost will be considered in the decision matrix.

Table 131: Cost range of potential corrective measures

Estimated Cost (CD$)	Cost Level
Less than $150,000	Low
$150,000-$500,000	Moderate
Greater than $500,000	High

Costly corrective measures may not be suitable in different parts of the province due to the lack of available resources. This factor must be taken into consideration in making any kind of management decision for the long-term effectiveness and sustainability of any corrective measure to improve drinking water quality.

13.3.3 Time Scale for Implementation

The time scale for implementation of a corrective measure refers to how long it will take to implement that specific measure. If there is a drinking water quality issue with THMs, the faster that this problem can be corrected, the better. A short time frame is any corrective measure that can be implemented almost immediately (within a month). A moderate time frame is any corrective measure that can be implemented within 1-12 months. A long time frame is anything that takes over a year to implement. The following table provides typical time frames, from conception to completion, for different types of water infrastructure projects in Newfoundland and Labrador.

Table 132: Time frame for implementation of water infrastructure

Type of Project	Time Frame
New surface water source	1.5 years
New groundwater source	9 months
Water treatment plant	2 years
Trunk mains	9 months
Pumping stations- new	9 months
Pumping Stations- replace	3 months
Single treatment method (chlorine, pH, in line filter, etc.)- new	9 months

Single treatment method (chlorine, pH, in line filter, etc.)- replace	3 months

13.3.4 Permanency of Solution

Most water infrastructure in Newfoundland and Labrador is designed around a 25 year life cycle. The useful life of a component will vary according to the materials, environment (climate, water and soil characteristics, etc.), and maintenance practices.
The following table provides an approximation of the useful life of various water infrastructure components (USEPA, 2002). Experience indicates that the useful life of many water infrastructure components in the province is considerably less than that indicated (as low as 2 years for gas chlorination systems). However, with proper maintenance, water infrastructure components in the province can live out their useful life.

Table 133: Useful life of water infrastructure

Component	Years
Reservoirs and Dams	50-80
Treatment Plants- concrete structures	60-70
Treatment Plants- mechanical and electrical	15-25
Trunk mains	65-95
Pumping Stations- concrete structures	60-70
Pumping Stations- mechanical and electrical	25
Distribution	65-95
Water Storage Tank	40-100

The permanency of a solution can be deemed short term, if the corrective action is planned as or is likely to turn out as temporary measure. If a corrective measure is planned as a permanent solution, but is unlikely to live out its useful life, it can be deemed moderate term solution. Any corrective measure that is likely to live out its useful life can be deemed a long term solution.

13.3.5 Adverse Hydraulic or Water Quality Effects

There is potential for certain corrective measures to cause adverse hydraulic and/ or water quality effects. Adverse hydraulic effects are likely to cause problems with pressure, flow, fire flow, water velocity, water levels in tanks, pump capacity, water retention times, etc.

Adverse water quality effects may happen when a corrective measure improves the THM problem, but causes deterioration in other water quality parameters. With some corrective measures, impacts are likely to be questionable as with the use of alternative disinfectants that have the potential to form alternative DBPs.

13.3.6 Acceptability to Stakeholders

The most important stakeholder in the drinking water quality sector is the water consumer. Other stakeholders include the municipal government as supplier, provincial government as regulator, consultants, manufacturers, etc. If there is no issue with a potential corrective measure in terms of its acceptability, stakeholder support will be from across the board. If there is a known or perceived issue with a corrective measure, stakeholder support is likely to be weak. This could include anything from concerns over cost, reluctance to regionalize with a neighbouring town, known health issues, lack of availability of materials and equipment, lack of convenience, ability of a community to operate, environmental concerns, etc. Where stakeholder attitudes towards a corrective measure are indifferent, or split either for or against, it can be a bit of a grey area particularly with respect to perceived issues.

13.3.7 Meets Regulations, Guidelines, Standards

Corrective measures must meet required regulations, guidelines and standards including:

- Guidelines for the Design, Construction and Operation of Water and Sewerage Systems
- Standards for Chemical and Physical Monitoring of Drinking Water
- Standards for Bacteriological Quality of Drinking Water
- Well Drilling Regulations
- Environmental Control Water and Sewer Regulations

Of particular interest is if the corrective measure violates any of the modeling objectives criteria laid out in Section 5.1 of this report such as primary and secondary disinfection requirements, maximum chlorine levels, acceptable pressure ranges, and maximum retention time in storage tanks. A borderline classification can be used if the violation is deemed minor (of small magnitude), temporary (of small duration) or not of significance.

13.4 Implement, Monitor and Review

After assessing the potential corrective measures against solution constraints, it should now be easy to identify which measure is best suited to address DBP issues and meet the needs of the community involved. The preferred solution can be selected from the highest scoring corrective measure or measures. The next step is to implement the preferred solution and to monitor its success. If issues with DBPs remain, the solution should be reviewed to see if possible improvements in performance can be made. If not, implementation of secondary corrective measures should be examined.

13.5 Decision Making Framework Application for Select Communities with High THMs

One of the objectives of the report was to produce a decision making framework to help communities figure out how best to deal with DBP issues. The six communities examined in some detail in this report are only a small sample of the total number of communities in the province experiencing problems with high levels of DBPs. In order to test out the application of the framework derived as part of this report, three additional communities from different regions of the province will be used as trial cases. The application of the framework will only proceed as far as reviewing the system for probable causes of high THMs and matching of appropriate corrective measures. Further assessment of possible solutions is not considered necessary at this stage.

In order to guide decision making, a checklist of information on community water distribution systems is required. A checklist form was developed as part of this report and can be found in Appendix E to help assist in the decision making process.

13.5.1Summerford

THM levels in Summerford are frequently over the guideline level, averaging 239 µg/L (at sites downpipe of site 1). Summerford also has the occasional BDCM exceedance over the 16 µg/L guideline. Using information provided by the town and from several Department of Environment and Conservation databases (Drinking Water Quality Database, GIS Database, OETC database) the checklist of information on community water distribution systems was compiled for Summerford and can be found in Appendix E. A review of the Summerford water distribution system for probable causes of high THMs is summarized in the following table.

Table 134: Problems contributing to high THMs in the Summerford distribution system

	Causative Factors	Quantitative Value
3	Surface water source exposed to saltwater influence	800 m (NW)
5	High DOC in source water	7.35 mg/L
10	Excessive chlorine demand	? d-1 (bulk) ? m/d (wall)
11	High pH	7.48
12	Long linear system	6 km intake to end total = ? km
13	Branched system with multiple dead ends	~6 DE
14	Distance of chlorination system to 1st user	1500 m contact time (p) = 38-41 min
15	Insufficient chlorination controls on system	flow proportional
16	System is oversized	0.19-0.27 m/s 250 mm Q_{avg} = 8.03-13.3 L/s
17	High retention time in network	25 hrs
18	Pipe material and age	1984
20	Large occasional demand on system	Crab plant Q = 332 m3/d
26	Poor O&M of system	Water Dist- Class I
27	Multiple factors	-
28	Poor design of system	-
30	High per capita demand	710 L/p/d
32	Problems with chlorine residuals	0.02-1.6 mg/L @ end

The main contributing factors to the DBP issue appear to be the source water quality, the length of the distribution system and number of dead ends, high chlorine demand, excessive contact time, overcapacity and high retention time in the distribution system, age of pipe infrastructure, and large occasional demand from the crab plant in Cottlesville. Although there were numerous triggers for DBP formation identified for Summerford, most were barely over the threshold to be considered a trigger, and there was no easily correctible major probable cause that stood out. The following table outlines which corrective measures may be appropriate for the problems contributing to high THMs observed in the Summerford distribution network.

Table 135: Applicable THM corrective measures for Summerford

Applicable Corrective Measures	Probable Causes Addressed
Policy of POU/POE treatment	All
Policy to promote use of alternative disinfectants	All
Policy to promote PWDU	All
Watershed protection	All
High quality water storage and recovery	All
Alternative disinfectants	All
System maintenance	All
Potable water dispensing unit	All
Point of use/entry treatment	All
Training	All
Improved design of system	All
Regionalization	All
Combination of corrective measures	All
Alternative water sources	3-5
Wind breaks around exposed coastal sources	3
Optimize disinfectant dose	3-5
Re-locate chlorination system	3-5-14
Regular system flushing at dead ends	5-12-13-16-17-20
Continuously bleed system at dead end	5-12-13-16-17-20
Downsizing mains	5-12-13-16
Replace or reline pipe	18
Loop distribution network	13
Water treatment plants	5
Filtration	5
Advanced treatment	3-5

As no single issue seems to predominate DBP problems in Summerford, CDM and RTM corrective measures may not be that effective. The best option for Summerford may therefore be a water treatment system or switching to alternative disinfectants. Further assessment of possible corrective measure will not be undertaken at this stage. Before any action is decided upon, further evaluation of possible corrective measures is recommended in order to identify the most suitable option.

13.5.2 Port Saunders

THM levels in Port Saunders are frequently over the guideline level, averaging 138 µg/L since 2000. Port Saunders also has the occasional BDCM exceedance over the 16 µg/L guideline. A field visit to the town of Port Saunders by DOEC staff was made on June

30, 2007, after the town communicated concerns over high levels of THMs. Using information from several Department of Environment and Conservation databases (Drinking Water Quality Database, GIS Database, OETC database) the checklist of information on community water distribution systems was compiled for Port Saunders and can be found in Appendix E. A review of the Port Saunders water distribution system for probable causes of high THMs is summarized in the following table.

Table 136: Problems contributing to high THMs in the Port Saunders distribution system

	Causative Factors	Quantitative Value
3	Surface water source exposed to saltwater influence	600 m (W)
5	High DOC in source water	6.03 mg/L
6	High level of bromide in source water	0.024 mg/L
7	High chlorine dose	2.67-7.41 mg/L
10	Excessive chlorine demand	? d-1 (bulk) ? m/d (wall)
11	High pH	7.9
12	Long linear system	5 km intake to end total = 10 km
13	Branched system with multiple dead ends	at least 12 DE
14	Distance of chlorination system to 1st user	750 m contact time (p) = 1704-2132 min
15	Insufficient chlorination controls on system	manual
16	System is oversized	0.12-0.61 m/s 200-150 mm Q_{avg} = 3.89-10.8 L/s
17	High retention time in network	28.4-35.5 hrs
18	Pipe material and age	1990 and older
20	Large occasional demand on system	Shrimp plant and ice plants
22	Balance between pumped supply and demand not optimized with storage	1 hr to fill/ 1 hr to empty
23	High retention time in tank	24.3 hrs
24	Dead zones/ poor mixing in tank	Inlet/ outlet close- 0.6 m
25	Little variation in water levels/ turnover in tank	37-83% inactive volume
26	Poor O&M of system	Water Dist- Class I
27	Multiple factors	-
28	Poor design of system	-
29	High iron	Fe = 0.05 mg/L
30	High per capita demand	450-1249 L/p/d
31	Pressure problems	Low at N end of town
32	Problems with chlorine residuals	0.1 mg/L @ end 0.3-0.98 mg/L @ 1st user

The main contributing factors to the DBP issue appear to be the source water quality, the storage tank, the large number of dead ends, high chlorine demand, insufficient chlorine control, excessive contact time and overcapacity in the distribution system. The large number of identified probable causes of high THMs in the Port Saunders distribution system means there are plenty of potential corrective measures that may be applicable. The following table outlines which corrective measures may be appropriate for the problems contributing to high THMs observed in the Port Saunders distribution network.

Table 137: Applicable THM corrective measures for Port Saunders

Applicable Corrective Measures	Probable Causes Addressed
Policy of POU/POE treatment	All
Policy to promote use of alternative disinfectants	All
Policy to promote PWDU	All
Watershed protection	All
High quality water storage and recovery	All
Alternative disinfectants	All
System maintenance	All
Potable water dispensing unit	All
Point of use/entry treatment	All
Training	All
Improved design of system	All
Regionalization	All
Combination of corrective measures	All
Alternative water sources	3-5-6
Wind breaks around exposed coastal sources	3-6
Re-locate chlorination system	3-5-6-7-14
Chlorine dose control	3-5-6-7-15
Tank location	23-24-25
Reduce storage capacity/adjust pump schedule	17-22-23-24-25
Increase mixing in tank	17-23-24-25
Regular system flushing at dead ends	5-12-13-16-17-20
Continuously bleed system at dead end	5-12-13-16-17-20
Downsizing mains	5-12-13-16
Replace or reline pipe	18
Loop distribution network	13
Water treatment plants	5-7
Filtration	5-7
pH adjustment	11
Iron and manganese removal	7-10-29
Advanced treatment	3-5-6-7

The water storage tank would seem to be the major contributing factor to high DBPs in the Port Saunders distribution system. RTM and CDM corrective measures may prove sufficient to lower DBP levels below guidelines. Further assessment of possible corrective measure will not be undertaken at this stage. Before any action is decided upon, further evaluation of possible corrective measures is recommended in order to identify the most suitable option.

13.5.3 Arnold's Cove

Using information from several Department of Environment and Conservation databases (Drinking Water Quality Database, GIS Database, OETC database) the checklist of information on community water distribution systems was compiled for Arnold's Cove and can be found in Appendix E. Arnold's Cove has only been displaying above guideline THM values since 2005. Since 2005, average THMs on this system have been 142 µg/L. BDCMs do not appear to be an issue. A review of the Arnold's Cove water distribution system for probable causes of high THMs is summarized in the following table.

Table 138: Problems contributing to high THMs in the Arnold's Cove distribution system

	Causative Factors	Quantitative Value
1	Reservoir contains flooded vegetation	yes
5	High DOC in source water	5.3 mg/L
10	Excessive chlorine demand	? d-1 (bulk) ? m/d (wall)
12	Long linear system	9.4 km intake to end total = ? km
13	Branched system with multiple dead ends	~3 DE
14	Distance of chlorination system to 1st user	1400 m contact time (p) = 40-179 min
15	Insufficient chlorination controls on system	manual
16	System is oversized	0.13-0.72 m/s 150-350 mm Q_{avg} = 12.6-69.4 L/s
17	High retention time in network	9.8-53 hrs
18	Pipe material and age	DI ?
20	Large occasional demand on system	Fish plant Q = 56.8 L/s
26	Poor O&M of system	Not certified
27	Multiple factors	-
28	Poor design of system	-
30	High per capita demand	1085 L/p/d

It is not immediately clear what has triggered the recent exceedances in THMs in Arnold's Cove. One possibility is disruption of normal operations with the fish plant resulting in reduced water demand. Another possibility is the switch from automatic flow proportional control of the chlorine dose to manual control some time around 2004. Free chlorine residuals in the Arnold's Cove distribution system meet both primary and secondary disinfection requirements. It is interesting to note that free chlorine residuals were actually higher (0.66 mg/L at site 3) before 2005 than after (0.54 mg/L at site 3), when THMs became an issue. Other contributing factors to the DBP issue appear to be source water quality, length of the distribution system, high chlorine demand, overcapacity in the system and excessive contact time due to the fish plant demand, high retention time without fish plant demand, and lack of a certified operator.

The number of identified probable causes of high THMs in the Arnold's Cove distribution system is relatively small. The following table outlines which corrective measures may be appropriate for the problems contributing to high THMs observed in the Arnold's Cove distribution network.

Table 139: Applicable THM corrective measures for Arnold's Cove

Applicable Corrective Measures	Probable Causes Addressed
Policy of POU/POE treatment	All
Policy to promote use of alternative disinfectants	All
Policy to promote PWDU	All
Watershed protection	All
High quality water storage and recovery	All
Alternative disinfectants	All
System maintenance	All
Potable water dispensing unit	All
Point of use/entry treatment	All
Training	All
Improved design of system	All
Regionalization	All
Combination of corrective measures	All
Alternative water sources	1-5
Remove submerged vegetation	1-5
Optimize disinfectant dose	1-5
Re-locate chlorination system	1-5-14
Chlorine dose control	1-5-15
Regular system flushing at dead ends	1-5-12-13-16-17-20
Continuously bleed system at dead end	1-5-12-13-16-17-20
Downsizing mains	1-5-12-13-16
Loop distribution network	13
Water treatment plants	5
Filtration	5
Advanced treatment	3-5-6-7

The solutions which address the largest number of issues are demand management corrective measures. Flushing devises that increase demand when the fish plant is not operational are the best option. Better chlorine demand management is also a likely option for Arnold's Cove. This can be achieved through reducing contact time by having a secondary chlorination system located closer to the first user that can be used when the fish plant is not operating, reducing the disinfectant dose and non-manual chlorine dosage control. Further assessment of possible corrective measure will not be undertaken at this stage. Before any action is decided upon, further evaluation of possible corrective measures is recommended in order to identify the most suitable option.

14.0 Conclusions and Recommendations

The problem of high disinfection by products in drinking water systems affects approximately a third of public drinking water systems and up to half the population of the province. The seriousness of DBP issues range from minor to very major, but to date only limited action has been taken to address the issue. This report is intended to provide a comprehensive overview of the extent of the DBP problem, factors contributing to the problem, possible solutions and their effectiveness, and how to determine the most appropriate solutions to fix DBP issues on individual community drinking water systems.

The source of DBP problems is unique to each drinking water system, but underlying causes can usually be identified. The Decision Making Framework for Selecting DBP Corrective Measures (Appendix B) is an attempt to streamline the management decision making process in order to select the most suitable corrective measure based on the probable causes of high DBPs and measured against different solution constraints such as affordability.

The analysis performed and methodology derived in this report is based on the best information available at the time, and should be taken as a starting point for further and more in-depth assessments.

Key messages of this report include the following:

- The majority of communities with DBP issues are very small (pop < 501) and small (pop 501-1500) towns in rural Newfoundland and Labrador.
- A one size fits all solution to DBP issues in the province will not work. Each water system has its own unique characteristics, and so each solution must be likewise unique.
- For the majority of very small drinking water systems with significant DBP issues and where other corrective measures will not work or are not financially feasible, PWDU are the most appropriate corrective measure.
- DOC is the most significant available predictor of THM and HAA formation potential in drinking water systems in the province. Chlorine is the second most significant predictor of DBP formation potential.
- Chlorine dosages used to disinfect drinking water in some systems in the province are in the same range as those required to treat wastewater.
- THMs and HAAs have been identified as the two largest classes of DBPs detected on a weight basis in chlorinated finished water.
- There is a risk trade-off with drinking water disinfection between microbial pathogens and disinfection by-products. The general consensus amongst water quality and health experts is that the risk posed by consuming water that hasn't been disinfected is much greater than that of consuming disinfected water containing DBPs.
- Even with certain corrective measures implemented, the response of the water distribution system may not be positive enough to completely correct DBP issues.

- Corrective measures to address DBP issues must not fix one problem only to create a dozen more (ie. bankrupt town, introduce other DBPs, violate existing guidelines, violate disinfection requirements, etc.).

The key recommendations of this report include the following:

- Drinking water system information is spread over numerous sources, jurisdictions, databases, government departments, individuals, etc. Every effort should be made for proper record keeping and sharing of data.
- The province must keep up to date on new and emerging DBPs associated with all forms of disinfectants used in the province and be prepared for sampling of such DBPs.
- A thorough examination of the validity of provincial drinking water quality data should be made in light of errors observed, use of different labs, different less than detect protocols, data gaps, new parameters, new drinking water quality guidelines, lack of meta data, uneven data distribution, etc.
- The use of alternative disinfectants should be encouraged in the province as long as their use does not create additional problems or issues.
- Drinking water systems in Newfoundland and Labrador using chlorine to disinfect should be selectively sampled for iodomethanes to determine if there is an issue with this emerging type of hazardous THM species.
- More study is required to fully understand the dynamics of THM and HAA formation in the province's drinking water systems.
- Any potential new water source that is to be disinfected with chlorine should have a chlorine decay rate test and THM formation potential test performed at an accredited laboratory prior to the final selection, development and commissioning of the new source. If THM formation potential under reasonable worst case scenario conditions based on known system conditions (temperature, pH, DOC, time, chlorine) is greater than 150 µg/L, consideration should be given to abandoning the source if a more viable source is available, or treatment options or alternative disinfectants made a requirement for that drinking water system.
- All drinking water systems should be regularly tested for UVA_{245}.
- Drinking water systems in Newfoundland and Labrador should be selectively tested for THM formation potential to determine THM formation rates and likely THM levels under worst case conditions (temperature, pH, DOC, time, chlorine).
- As long as water demand from all potential users can be met, a surface water source from a smaller sized drainage area should be selected over a surface water source from a larger sized drainage area for all new source water supplies for ease of management of the watershed area.
- More research is required to have a better understanding of provincial surface water behaviour and dynamics including seasonal effects, water quality and temperature depth profiles, wind and wave inter-action, sediment dynamics, etc.
- The province should adopt a maximum residual disinfectant level for chlorine of 4.0 mg/L for all water consumers based on the USEPA guideline. Chlorine residuals above this level can cause known or expected health risks such as eye nose irritation and stomach discomfort.

- In Newfoundland and Labrador, design guidelines for fire flows, fire storage and other fire fighting requirements are established by the Insurance Advisory Organization and the Fire Commissioners Office. The justification for such requirements is not well documented and should be investigated more comprehensively. Generally, the requirements for fire flows, particularly in small communities, result in oversized water distribution systems.
- Consideration should be given to a design guideline requiring the achievement of a daily peak water velocity for all pipes in a distribution system in the range of 0.2-0.4 m/s.
- It should be mandatory that drinking water systems using chloramines for disinfection test regularly for cyanogen chloride and N-Nitrosodimethylamine (NDMA).
- Further evaluation of the performance and effectiveness of water treatment infrastructure in the province is required. Such evaluations should be used to identify methods that work and could be transferred to other systems (eg. MIOX, iron and manganese removal, filtration). Such evaluations should also be used to identify treatment methods that are not functioning as expected, the causes of such underperformance, and options to improve performance.
- Operator certification should be made mandatory.
- Distribution system modelling should be expanded to include DBP growth scenarios using laboratory derived growth rates and ultimate formation levels.
- The use of CFD modeling to develop guidelines for water storage tank design and configuration, investigate problems with existing water storage tanks and recommend modifications should be investigated.
- Collect SCADA flow data from water distribution systems across the province in order to develop a generic provincial daily water demand pattern.
- EPANET or other distribution modeling software packages should be used to model mixed source water distribution systems in the province to better understand their dynamics and behaviour.
- The OETC database should include information outlined in the checklist of information on community water distribution systems including: flowrate, length and size of pipe from chlorinator to first user, tank volume, chlorine usage rate (gas), dilution ratio of water to liquid chlorine, % chlorine solution used, frequency chlorination tank is refilled (liquid), chlorine dosage, contact time, CT factor, etc.
- Communities with major DBP issues should be preferentially targeted for more aggressive corrective measures.
- Training should be provided to consultants involved in the design of water infrastructure to apprise them of changes to design guidelines, new concerns, scientific knowledge, methods and innovations.
- The OETC program should design a module on managing DBPs for incorporation into their training curriculum.
- The Decision Making Framework for the Selection of DBP Corrective Measures should be used as a tool for evaluating any community water distribution system that displays DBP issues.

- The BMPs for the Control of Disinfection By Products should be made official policy, incorporated into existing design guidelines, and promoted by the Department of Environment and Conservation.

15.0 References

AWWA Research Foundation, 1994, *Natural Organic Matter in Drinking Water: Origin, Characterization and Removal*, AWWA, USA.

AWWA Research Foundation, 2006, *Managing Distribution Retention Time to Improve Water Quality- Phase II: Guidance Manual*, AWWA, Denver, USA.

AWWA Research Foundation, 2006, *DBP Control in High Bromide Water While Using Free Chlorine During Distribution*, AWWA, Denver, USA.

AWWA Research Foundation, 2003, *Influence of Distribution System Infrastructure on Bacterial Regrowth*, AWWA, Denver, USA.

AWWA Research Foundation, 2003, *Feedback Control of Booster Chlorination Systems*, AWWA, Denver, USA.

AWWA Research Foundation, 2005, *Effects of Blending on Distribution System Water Quality*, AWWA, Denver, USA.

AWWA Research Foundation, 2004, *Use of Chlorine Dioxide and Ozone for Control of Disinfection By-Products*, AWWA, Denver, USA.

AWWA Research Foundation, 2004, *Optimizing Chloramine Treatment*, AWWA, Denver, USA.

AWWA Research Foundation, 2006, *Assessment of Chloramine and Chlorine Residual Decay in the Distribution System*, AWWA, Denver, USA.

AWWA Research Foundation, 2006, *Formation and Decay of Disinfection By-Products in the Distribution System*, AWWA, Denver, USA.

Bureau of Reclamation, 2001, *Water Treatment Primer for Communities in Need: Report # 68*, United States Department of the Interior, Denver, Co.

CCME, March 2007, *Guidelines for Canadian Drinking Water Quality*, Winnipeg, MB.

Clark, Robert M., Grayman, W., M., 1998, *Modeling Water Quality in Drinking Water Distribution Systems*, American Water Works Association, Denver, Colorado.

CWWA, 2006, *CWWA Bulletin*, Ottawa, 20(9):1-8.

Dawe, P., 2006, *A statistical evaluation of water quality trends in selected water bodies of Newfoundland and Labrador*, Journal of Environmental Engineering and Science, 5(1): 59-73.

Department of Environment, 2000, *Trihalomethane Levels in Public Water supplies of Newfoundland and Labrador*, Government of Newfoundland and Labrador, St. John's.

Department of Environment and Conservation, 2005, *Guidelines for the Design, Construction and Operation of Water and Sewerage Systems*, Government of Newfoundland and Labrador, St. John's.

Department of Environment and Conservation, Department of Municipal Affairs, 2006, *Chlorination Equipment Selection Guidelines*, Government of Newfoundland and Labrador, St. John's.

Department of Environment and Conservation, 2008, *Sustainable Options for the Management of Drinking Water Quality in Small Water Systems*, Government of Newfoundland and Labrador, St. John's.

Department of Environment and Lands, Water Resources Division, 1993, *State of Water Resources of Newfoundland*, Government of Newfoundland and Labrador, St. John's.

Environnent Canada, 2006-07-28, *The Climate of Newfoundland*, http://atlantic-web1.ns.ec.gc.ca/climatecentre/default.asp?lang=En&n=83846147-1

Hach, 2003, *Pocket Colorimeter II Analysis Systems Instruction Manual Chlorine*, Hach Company, USA.

Health Canada, October 2007, *Guidelines for Canadian Drinking Water Quality: Guideline Technical Document for Haloacetic Acids*, Health Canada, Ottawa, Ontario.

Khan, Amir A., Khan, H., Goebel, M., 2002, *Chlorine Demand Management for THMs Control in Newfoundland and Labrador*, CWRA 55th Annual Conference, Winnipeg, MB.

LaVerda, Nancy, 2001, *Toxicologic Evaluation of Trihalomethanes (THMs)*, Applied Occupational and Environmental Health.

McGhee, J. Terence, 1991, Water Supply and Sewerage, 6th Edition, McGraw-Hill Inc., New York.

Nicol, Keith, Nicol, Heather, January 8, 2008, *Fit to Drink: Researchers tapping into locals' passion for spring water*, Telegram, St. John's.

Park, A., 2006, *Potable Water Dispensing Units In Newfoundland and Labrador*, Department of Environment and Conservation, Corner Brook.

Poole, N., 2006, *Predicting THM Levels in Newfoundland & Labrador's Public Water Supplies*, Department of Environment and Conservation, Corner Brook.

Rossman, L.A., 2000, *EPANET 2 Users Manual*, US Environmental Protection Agency, Cincinnati, OH

Reuters, November 6, 2006, *Filters best way to clean tap water: study*.

Sadiq, R., Rodriguez, M.J., 2004, *Disinfection by products (DBPs) in drinking water and predictive models for their occurrence: a review*, Science of the Total Environment, 321: 21-46.

Singer, P. C., 2006, *DBPs in Drinking Water: Additional Scientific and Policy Consideration for Public Health Protection*, AWWA Journal, 98(10): 73-80.

Singer, P. C., 1999, *Formation and Control of Disinfection By-Products in Drinking Water*, AWWA, Denver, USA.

Stauffer, Julie, 2004, *The Water You Drink: Safe or Suspect*, New Society Publishers, Canada.

Uber, J.G., 2003, *Maintaining Distribution System Residuals Through Booster Chlorination*, AwwaRF, Denver, Colorado.

USEPA, Sept 2002, The *Clean Water and Drinking Water Infrastructure Gap Analysis*, Office of Water, US.

USEPA, Sept 2007, *Stage 2 Disinfectants and Disinfection Byproducts Rule*, Office of Water, US.

Walfoort, C., Messina, M.J., Miner, D., 2008, *Storage Tank Aeration Eliminates Trihalomethanes*, Opflow, 34(5): 28-29.

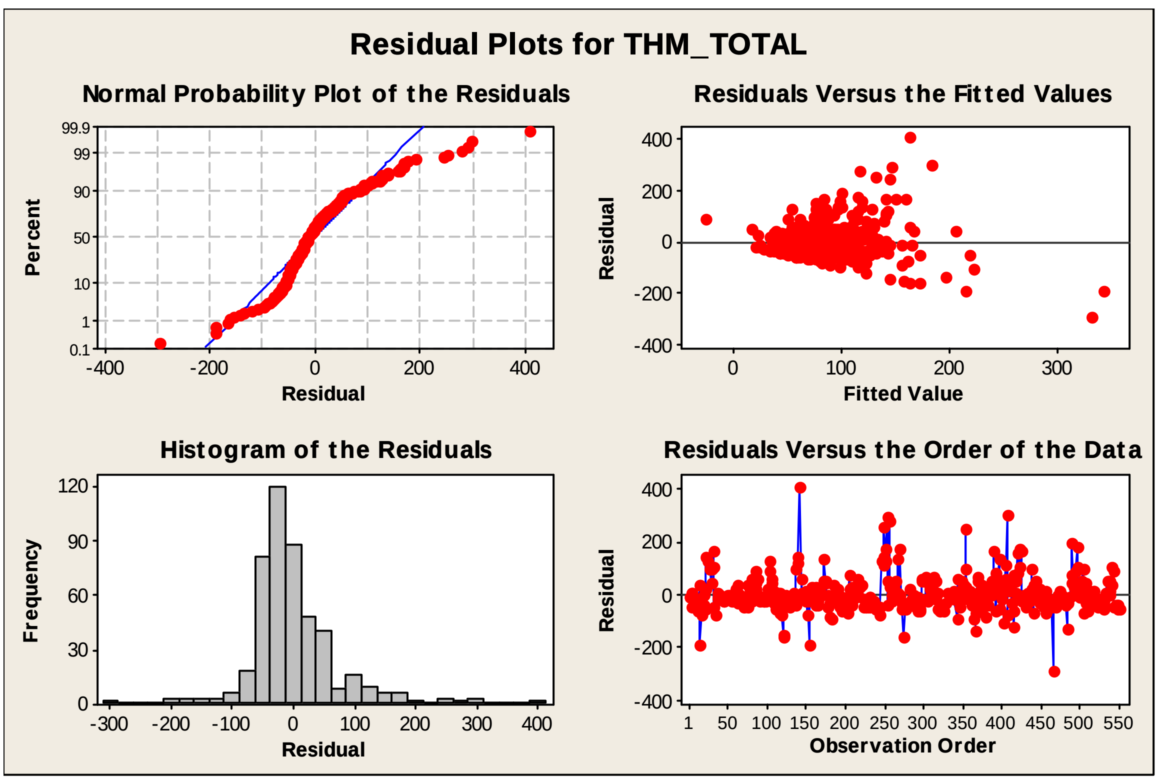
Appendix A: THM Regression Analysis

Multiple Linear Regression: Eastern Region

Response is THM_TOTAL

457 cases used, 94 cases contain missing values

Vars	R-Sq	R-Sq(adj)	Mallows C-p	S	SITE_NUM	FREE_CHLORINE	TEMPERATURE_TEMP	PH_LEVEL	BROMIDE	DOC	PRESS	Max VIF
1	17.9	17.7	32.9	69.345						X	2512180	-
1	2.5	2.3	123.8	75.553					X		3078124	-
2	20.6	20.2	19.0	68.276		X				X	2461582	1.0
2	18.8	18.4	29.7	69.044					X	X	2513451	1.0
3	22.1	21.6	11.9	67.682		X		X		X	2453521	1.1
3	21.4	20.8	16.3	68.007		X			X	X	2463389	1.0
4	23.2	22.6	7.2	67.268		X		X	X	X	2448560	1.2
4	22.8	22.1	9.6	67.443		X	X	X		X	2150827	1.1
5	23.8	23.0	5.9	67.094		X	X	X	X	X	2150068	1.1**
5	23.4	22.5	8.4	67.282	X	X		X	X	X	2440102	1.2
6	23.9	22.9	7.0	67.102	X	X	X	X	X	X	2155719	1.1

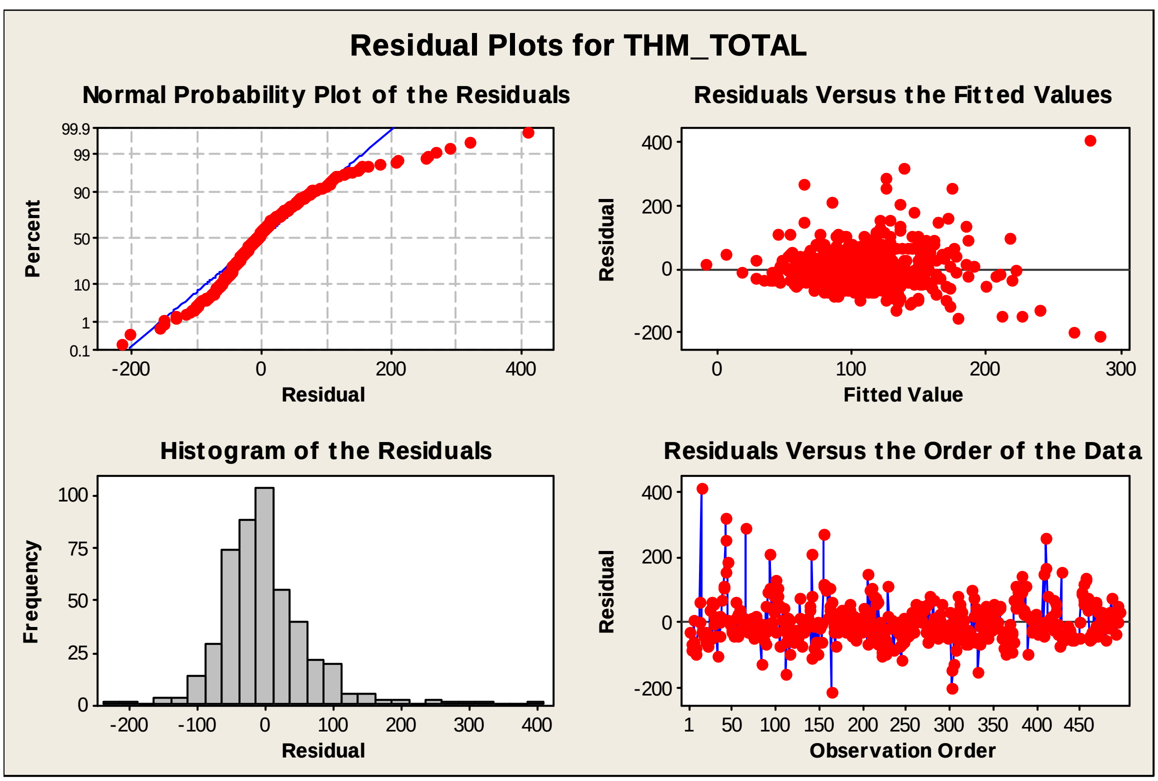


Multiple Linear Regression: Central Region

Response is THM_TOTAL

466 cases used, 30 cases contain missing values

Vars	R-Sq	R-Sq(adj)	Mallows C-p	S	PH_LEVEL	BROMIDE	DOC	Temp	SITE_NUM	FREE_CHLORINE	PRESS	Max VIF
1	10.4	10.2	93.7	73.260			X				2527187	-
1	8.6	8.4	104.5	73.964						X	2655327	-
2	18.3	18.0	46.3	70.004			X			X	2369941	1.0
2	15.8	15.4	62.1	71.088	X		X				2393333	1.1
3	24.2	23.7	11.9	67.512	X		X			X	2217274	1.1
3	19.5	18.9	41.3	69.594		X	X			X	2346433	1.0
4	25.6	25.0	5.1	66.950	X	X	X			X	2182292	1.1
4	24.8	24.1	10.5	67.340	X		X		X	X	2215570	1.1
5	25.9	25.1	5.3	66.892	X	X	X		X	X	2188068	1.1**
5	25.7	24.9	6.8	66.998	X	X	X	X		X	2184185	1.1
6	26.0	25.0	7.0	66.942	X	X	X	X	X	X	2190331	1.1

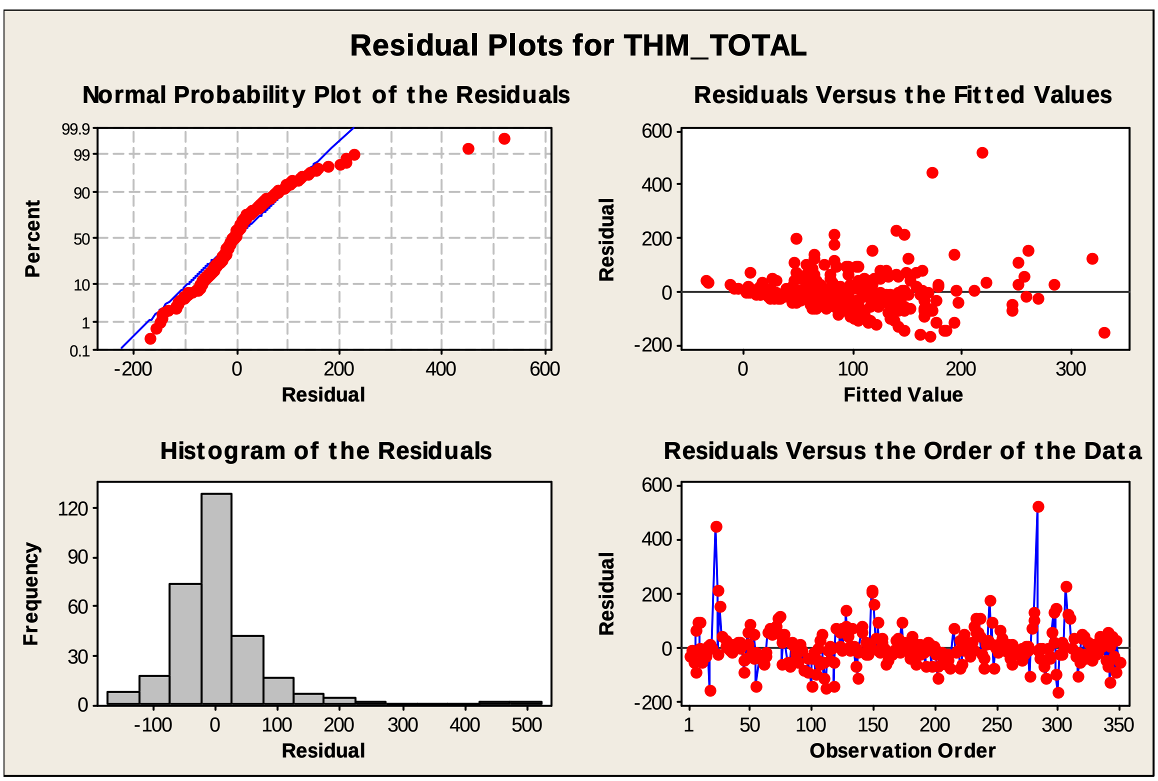


Multiple Linear Regression: Western Region

Response is THM_TOTAL

296 cases used, 54 cases contain missing values

Vars	R-Sq	R-Sq(adj)	Mallows C-p	S	PH_LEVEL	BROMIDE	DOC	Temp Tap	FREE_CHLORINE	SITE_NUM_1	PRESS	Max VIF
1	21.6	21.3	87.1	84.114			X				2573801	-
1	15.7	15.4	115.4	87.195					X		2682124	-
2	34.4	33.9	27.2	77.065			X		X		2233973	1.0
2	26.9	26.4	63.4	81.342			X	X			1983737	1.0
3	38.7	38.1	8.1	74.592			X	X	X		1691992	1.0
3	35.5	34.9	23.7	76.523	X		X		X		2233229	1.3
4	39.8	39.0	5.0	74.075	X		X	X	X		1669881	1.4
4	39.3	38.5	7.4	74.380			X	X	X	X	1688551	1.0
5	40.2	39.2	5.0	73.943	X		X	X	X	X	1669794	1.4**
5	39.8	38.8	7.0	74.201	X	X	X	X	X		1673384	1.4
6	40.2	39.0	7.0	74.069	X	X	X	X	X	X	1671922	1.4

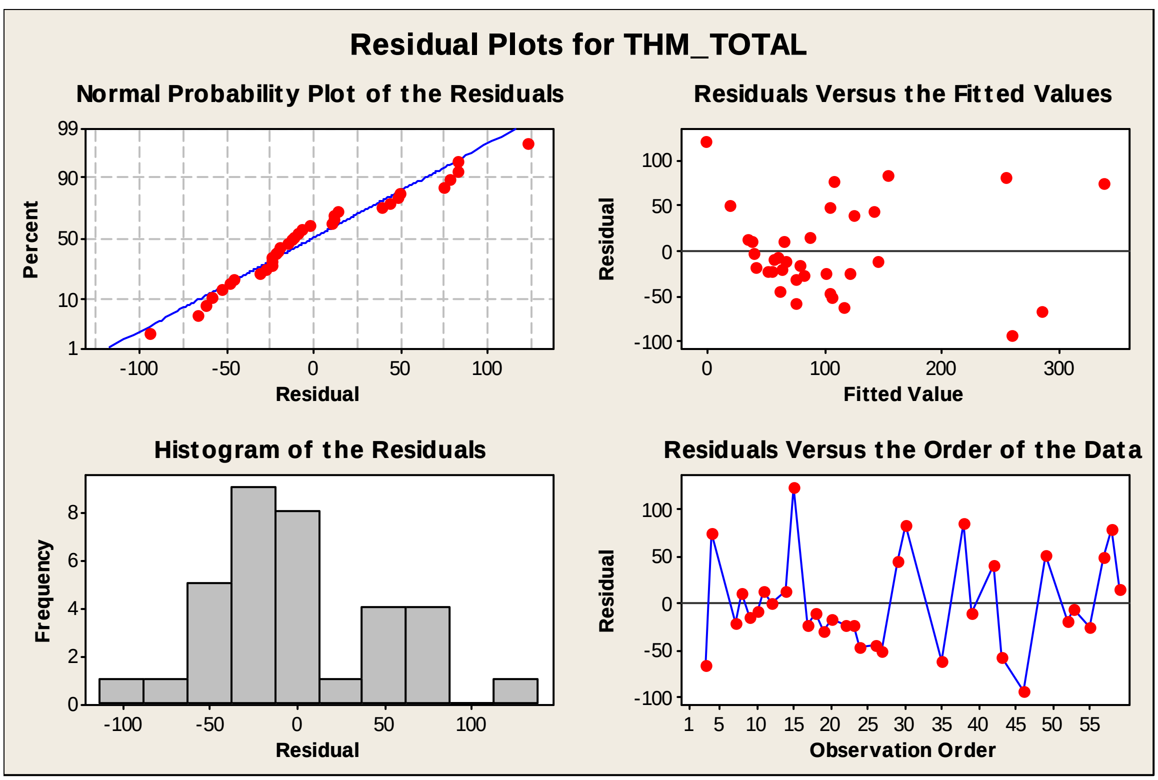


Multiple Linear Regression: Labrador Region

Response is THM_TOTAL

34 cases used, 25 cases contain missing values

Vars	R-Sq	R-Sq(adj)	Mallows C-p	S	SITE_NUM	FREE_CHLORINE	TEMPERATURE_TAP	PH_LEVEL	BROMIDE	DOC	PRESS	Max VIF
1	60.2	59.0	7.2	59.067						X	411838	-
1	35.2	33.2	30.6	75.393		X					470739	-
2	67.4	65.3	2.4	54.311		X				X	343695	1.2
2	65.6	63.4	4.2	55.848			X			X	146315	1.1
3	70.5	67.5	1.6	52.570		X	X			X	147274	1.3**
3	67.6	64.4	4.3	55.058	X	X				X	354429	1.2
4	71.1	67.1	3.0	52.900		X	X		X	X	160506	1.3
4	70.5	66.5	3.5	53.407	X	X	X			X	158041	1.3
5	71.1	66.0	5.0	53.816		X	X	X	X	X	172957	1.6
5	71.1	65.9	5.0	53.836	X	X	X		X	X	176632	1.4
6	71.1	64.7	7.0	54.802	X	X	X	X	X	X	203399	1.6



Appendix B: BMPs for the Control of Disinfection By-Products

BMPs for the Control of Disinfection By-Products

Issue:

Long-term exposure to disinfection by-products in drinking water (and possibly short term exposures in pregnancies) may pose a health risk to the population of Newfoundland and Labrador. Mitigative measures can be taken to help reduce disinfection by-products to below guideline levels for new, upgrading and existing water distribution systems.

Background:

Provincial guidelines require that water supplies be disinfected and maintain a disinfectant residual in the water distribution system in order to ensure the destruction of potentially harmful pathogens. Chlorine is the most commonly used form of disinfectant in the province. Disinfection by-products (DBP) are chemical compounds formed by the reaction of a water disinfectant with a precursor in a water supply system. DBPs are undesirable in drinking water as there is some evidence that long-term exposure may cause health risks. While minimizing disinfection by-products is important, the risks of not disinfecting water far outweigh the risks created by disinfection by-products. There is a wide array of mitigative options available to deal with DBP issues, and any action taken to reduce one type of DBP is likely to help reduce other forms as well. The main DBPs of concern in Newfoundland and Labrador are trihalomethanes (THMs), bromodichloromethane (BDCMs), and haloacetic acids (HAAs).

THMs and HAAs have been identified as the two largest classes of DBPs detected on a weight basis in chlorinated finished water. THM and HAA levels tend to peak in the fall for most water distribution systems in the province. DOC is the most significant available predictor of THM and HAA formation potential in drinking water in the province followed by chlorine dosage. Reaction kinetics in the formation of DBPs are higher at warmer temperatures. The rate of formation of THMs is fastest in the initial hours after chlorine has been added and then slows down. THM formation can proceed for several days in a distribution system as long as there is free chlorine residual. BDCMs are more likely to occur in surface water sources with high bromide levels in exposed costal areas. The majority of drinking water systems in the province that display high HAAs also display pH levels below the minimum guideline level of 6.5.

Best Management Practices:

BMPs for the reduction of DBPs in new, upgrading and existing drinking water systems may apply only selectively. The following are BMPs for the control of disinfection by-products in drinking water systems in Newfoundland and Labrador:

Policy Measures

- It should remain the mandate of any community with a centralized water distribution system to provide adequate quality drinking water to users; the onus for providing potable water meeting GCDWQ should not be placed on the water consumer.
- In very small and small communities with DBP levels significantly above the guideline value, a policy of point of use household treatment devices can be

implemented as a temporary or emergency measure. A temporary measure should be considered as lasting three months or less.
- More diversity in water disinfection methods should be promoted in the province.

Source Based Control Measures
- All existing, new and potential surface and groundwater supplies should be designation as Protected Public Water Supply Areas.
- Water source options and recommendations are conditional on water availability.
- Waters sources and source water intakes should be located as far as possible from the coastline and prevailing coastal winds. Water sources should be sited in locations sheltered (by trees, differences in elevations, berms, fences, etc.) from ocean salt-water spray, and prevailing westerly and coastal winds.
- The lower the level of DOC in surface water sources, the lower the formation potential for DBPs. Any source water with DOC greater than 2 mg/L can produce unacceptably high levels of DBPs with the addition of chlorine for disinfection. As a guideline, surface waters with a DOC less than 4.2 mg/L should be used as new source water supplies to minimize DBP formation potential. DOC levels between 0 and 4.2 mg/L represent the 1st quartile of the range of DOC levels in surface water sources across the province.
- Reservoirs filled by small streams/springs and groundwater sources are the preferable source water type when trying to maintain DBPs within guideline levels.
- Groundwater and surface waters should not be mixed in the same distribution system if the only source of treatment is disinfection through chlorination as this significantly contributes to the formation of THM species (BDCMs). Mixing should only be allowed if there is either significant removal of natural organic material, bromide or both.
- Where a land area is to be flooded to create a surface water reservoir, vegetation must be removed from the area prior to inundation as per permit requirements. Where a vegetated area has already been flooded to create a source water reservoir, water levels should be lowered and vegetation removed if DBP levels warrant. Alternatively, methods to remove vegetation without lowering water levels can be investigated.
- Shallow ponds with long fetch lengths in the direction of prevailing winds should be avoided as water sources.
- The optimal type of surface water intake is one that permits varying the depth of water withdrawal to alternate with seasonal changes.
- The intake should be located off the bottom of the waterbody to ensure conduit openings are not clogged by bed-load deposits (silt, sand, gravel, debris), and deep enough below the water surface to ensure submersion during extreme low water events.
- The optimal depth for an intake structure is below the summer thermocline, typically in deeper water, but not at the lowest level in the waterbody.
- Horizontal intake filtration berms have a negligible effect on reducing DBP precursors.

- Where a high quality drinking water source is available either as a primary, secondary, or emergency supply, use of this source should be made to lessen the formation potential of DBPs, especially during periods of maximum DBP formation potential, typically summer and fall.
- Any potential new water source that is to be disinfected with chlorine should have a chlorine decay rate test and THM formation potential test performed at an accredited laboratory prior to final selection, development and commissioning of the new source. If THM formation potential under reasonable worst case scenario conditions (temperature, pH, DOC, time, chlorine) based on known system conditions is greater than 150 µg/L, consideration should be given to abandoning the source if a more viable source is available, or treatment options or alternative disinfectants made a requirement for that drinking water system.

Chlorine Demand Management

- The maximum residual disinfectant level in any drinking water system should not exceed 4.0 mg/L. Chlorine residuals above this level can cause known or expected health risks such as eye nose irritation and stomach discomfort.
- A detectable free chlorine residual should be considered anything greater than or equal to 0.05 mg/L unless accompanied and confirmed by a total residual chlorine test. A free chlorine residual of 0.02 mg/L may be acceptable if total chlorine residual confirms presence and removes the possibility of tester error.
- A contact time or CT factor value for inactivation of *giardia* should only be used when the distribution system has experienced a previous *giardia* contamination event and relies on chlorine disinfection as its only form of treatment.
- The chlorine dosage should be kept as low as possible while still maintaining required primary and secondary disinfection objectives. If chlorine residuals at all points (particularly end points) in the distribution system are typically over 0.1 mg/L, there is potential to reduce the chlorine dosage to achieve “detectable” levels. Typical chlorine dosages for drinking water disinfection in the province range between 2-15 mg/L. High chlorine demand results in a high chlorine dose.
- The application point of the chlorine dose should be as close to the first user as possible while still achieving primary and secondary disinfection objectives.
- A buffer above the minimum contact time and CT value should be incorporated into the required primary disinfection objectives for chlorine to take into account future developments either down-pipe or up-pipe of the design *First User*. The buffer should not exceed 2-10 times the minimum contact time or CT value.
- Chlorination systems should be located down-pipe of water storage tanks in systems where a sufficient contact time or CT value is available. This may increase system maintenance requirements. The placement down-pipe of the tank depends on system hydraulics and the location of the tank.
- Once an optimal point of chlorination has been identified based on an established *First User* location, future residential, commercial, institutional or industrial development up-pipe of this *First User* site should be restricted, or provision for the relocation of the chlorination system made.

- Calculation of CT factor values and contact time is important for system design purposes and should be reviewed regularly with each season and with any new developments on a distribution system.
- For calculation of the CT factor value, worst-case scenario conditions should be evaluated: the contact time at peak daily flow should be used, and the minimum observed chlorine residual (within the normal observed range) at the first point of use for the period of interest.
- THMs in the province tend to peak during the fall and are relatively high during the spring and summer in response to peaks in THM precursors. THMs are at their lowest during the winter. Chlorine demand is at its highest during the spring and at its lowest during the winter. Adjusting chlorine dosage, or targeting the use of other specific corrective measures (flushing, bleeding system, not mixing groundwater with surface water, use of deeper intakes, reducing tank storage capacity, use of POE/POU devices, etc.) during periods of highest THM formation potential or highest chlorine demand may help reduce DBP formation.
- Where removal of DBP precursors is not possible, practical or affordable, lowering the chlorine dosage (while still maintaining required primary and secondary disinfection objectives) can be used as a first response to high DBP levels.
- Chlorine boosters have limited application for reducing DBPs, and should only be used for this purpose where the initial chlorine dose is high or when the free chlorine residual reading at the first point of use is over 4.0 mg/L. The only potential a chlorine booster has for reducing DBPs is if the total combined chlorine dose from primary and booster chlorination systems is less than the chlorine dose from a single primary chlorination system.
- Water distribution systems with existing booster chlorination systems need to optimize their chlorine dosages so as to minimize overall chlorine use.
- On long distribution systems, chlorine boosters should be located relatively close to more densely populated areas.
- All communities using chlorine for disinfection should be equipped with at least two field chlorine test meter. Manual chlorine residual readings should be collected from multiple points on the distribution system on a daily basis as per Permit to Operate requirements. Values should be recorded and archived.
- All water distribution systems should be equipped with a flow meter. Communities should take regular flow meter readings (at least once a week), with values recorded and archived. Flow meters should be properly sized, sited, installed, maintained and calibrated.
- As a minimum, all communities disinfecting with chlorine should use flow meter readings and manual chlorine residual readings in order to make decisions concerning chlorine dosage control.
- Combined automated flow and residual analyzer control of chlorine dosage should only be considered for large communities or communities with dedicated and well-trained water system operators and well-maintained distribution systems.
- Chlorine residual feedback controls have limited application for reducing DBPs.

- Chlorine control using a fixed location residual analyzer can only optimize chlorine levels at a specific point, with mixed results (greater variation in chlorine residuals) elsewhere on the system.
- Automated flow and/or residual analyzer controls should not be installed with the expectation that they can replace water distribution system operators, or negate the need for manual chlorine residual readings.

Retention Time Management

- Water storage tanks contribute significantly to DBP levels in a distribution system due to dead zones, low water turnover rates, and poor circulation. These effects can generally be reduced by proper design and operation of storage facilities, such as appropriate tank sizing, inlet/outlet configuration, mixing, and operational schedule.
- Storage tank volumes should be minimized to avoid unnecessary storage. Stored water volumes should be optimized to meet requirements for storage, pressure and volume for fire fighting.
- Where the main purpose of a water storage tank is to provide pressure to the water distribution system, elevated storage tanks should be used as opposed to standpipe tanks.
- Tanks located at the beginning of the distribution system tend to reduce overall water age in the tank and distribution network, and reduce variability in chlorine residuals.
- The balance between supply from the pumps and network demand should be optimized in order to reduce the volume of storage required.
- Variation in water level in the tank should be maximized to force turnover of water in the tank.
- Systems with variable speed pumps or multi-pump installations can be configured to increase the pumping rate for a short period each emptying/filling cycle so as to increase the velocity at the tank inlet and improve mixing.
- When there are no issues involved (with supply, pressure or CT value), absolute storage capacity on a distribution system can be reduced by taking storage tanks off line or reducing the maximum water level in a tank.
- Tank design must incorporate the need for greater mixing through replacing a common inlet/outlet with separate pipes, installing baffles, moving the location or orientation of the inlet, increasing the distance between the inlet and outlet, reducing the diameter of the inlet, installing a duckbill valve to increase the velocity of the inlet jet, or installing a paddle or impellor devices to improve mixing within the tank.
- Water temperature stratification is an issue with above ground standpipe tanks.
- Water retention times in storage tanks should be minimized.
- Communities with slower DBP growth rates should be preferentially targeted for retention time management corrective measures.
- For water storage tanks with long residence times, aeration systems can be used to strip volatile DBP compounds from the water. With the installation of a water

storage tank aeration system, consideration must be given to the resulting loss of chlorine residuals.

Water Demand Management

- Effort should be made to locate new water connections, and manual and automated flushing sites on areas of the distribution network with high retention times so that demand is increased in these areas.
- Manual or automatic flushing for the control of DBPs must occur so that the period between flushing is less than the maximum retention time in the distribution system. A manual flushing program with a flushing frequency of more than once a day is not practical. System flushing is most appropriate on distribution systems with excess capacity.
- A distribution system can be bled continuously in order to lower retention times under certain conditions. Continuous bleeds are most appropriate on linear systems or systems with excess capacity.
- Flushing or bleeding the system is not practical where the distribution network has a contact time at peak flow close to 20 minutes.

Water Distribution system Operational and Infrastructural Measures

- Distribution system flushing can be used as a first response measure to water quality failures, including high levels of DBPs. One time flushing, however, can only be considered a short term response.
- Minimizing the number of shut valves required to produce a hydraulic boundary, and locating valves in areas with relatively high demand on either side of the shut valve can reduce retention times. Shut valves can be used in a network to re-route flows through parts of a system with low demand and high retention times. This may only be appropriate for larger water distribution systems.
- Continual system flushing (manual, automated or through a continuous bleed) and reducing overall system capacity (abandoning mains, downsizing mains) offers positive potential for reducing DBP levels, but must be weighed against water conservation needs, and contact time or CT factor requirements.
- Pumping water from one zone in a distribution system to another in order to re-circulate water can be used to reduce overall peak retention times.
- Decay of chlorine at the pipe wall is the leading contributor to overall chlorine decay in the distribution system. Pipe wall decay of chlorine can be reduced by regular system flushing, chemical flushing, swabbing, pigging or relining pipe. Chemically assisted flushing programs should be targeted to communities that are unable to achieve flushing velocities of 0.75 m/s without encountering negative pressures in the distribution system.
- Pipes greater than 25 years old, particularly unlined cast iron pipes, should either be replaced or relined if known to be contributing to water quality problems. Unlined and cast iron pipe should only be used if there is no reasonable alternative.
- New development in communities should be controlled so as to promote optimal water distribution system layout. Networks should be designed to avoid

branching, to minimize the number of dead ends, and to maximize looping of the system.
- Looping of the distribution system is optimal on networks that do not display overcapacity or excessive water age.
- The design of water distribution systems needs to reflect current long term declining population trends in the province when estimating future water demand.
- Pipe size should be optimized to meet required hydraulic conditions.
- Consideration should be given to a design guideline requiring the achievement of a daily peak water velocity for all pipes in a distribution system in the range of 0.2-0.4 m/s.
- Centralized or regional drinking water systems are most appropriate in high population, high population density areas that are relatively flat with a deep soil profile for the laying of extensive pipeline.
- Centralized or regional drinking water systems should include a water treatment plant if the population being serviced is medium to very large in size.
- Centralized or regional drinking water systems require support from the communities involved and should have well trained, full time operators.
- Only NSF approved chemicals and materials should be used in water disinfection and treatment.

Alternative Disinfectants
- Alternative disinfectants such as ozone, chloramines, UV and MIOX can significantly reduce the production of chlorinated DBPs.
- In order to provide a disinfectant residual in the distribution system, ozone and UV must be paired with a disinfectant that does leave a residual, such as chlorine.
- All disinfection methods, except for UV, will produce some form and level of DBPs.

Source Water Treatment
- Source water treatment for the targeted removal of DBP precursors provides the best assurance that DBPs will not form.
- Natural organic material can be removed to varying degrees using conventional, standard, and advanced treatment processes. Bromide removal requires advanced treatment processes.
- A water treatment plant (WTP) on a distribution system will not necessarily reduce THM levels if the WTP has not been designed specifically to remove DBP precursors or if the treatment system has not been adequately designed. WTPs in communities with DPB issues must be designed for the removal of DBP precursors.
- The practice of continuous pre-chlorination prior to any other form of treatment in the WTP should be discontinued. Depending on the treatment train, chlorine may be added before filtration, but never before coagulation and sedimentation. Pre-chlorination in conventional treatment plants may be necessary on a periodic cycling basis to deal with in-plant vectors such as algae growth and odour conditions.

- The most successful forms of treatment to reduce THM formation are chemical treatment (coagulation and flocculation, GAC), multi-media filtration, membrane (micro to nano) filtration, and reverse osmosis.
- Stand-alone pre-filtration systems (of pour size >10 µm) have no significant effect on reducing DBPs.
- To be effective in reducing DBPs, filtration systems (granular) must be in combination with chemical treatment, they must be appropriately sized and maintained (all types), or they must be of sufficiently small pore size (ultra-filtration, nano-filtration).
- pH adjustment has a limited effect on reducing DBPs. pH adjustment should be optimized for each individual system and should occur post chlorination.
- Iron and manganese removal (preferably through the use of permanganate) offers positive potential for the reduction of DBPs through reducing chlorine demand and required chlorine dosage, and the oxidation of DBP precursors. Primary disinfection requirements must still be met with any reduction in chlorine dosage.
- Large scale advanced water treatment processes are not appropriate for very small and small sized water distribution systems in the province.

Point of Use/Point of Entry Measures

- Advanced water treatment technology may be appropriate in very small and small sized communities on a small scale in the form of Potable Water Dispensing Units (PWDUs).
- Collection of drinking water in containers from a centralized location (roadside springs, stores) is common practice in many communities of the province. Roadside springs are not reliable sources of safe drinking water and their use should be discouraged.
- PWDUs should not replace regular water distribution systems and should not replace regular water disinfection or treatment systems.
- There should be demonstrated community support for the installation of a PWDU.
- Household Point of Use and Point of Entry (POU/POE) treatment systems must be used and maintained properly by the consumer including cleaning, replacement of parts, and proper storage of treated water.
- POU/POE control measures may be applicable for very small communities that cannot afford any water treatment, as an interim solution to water quality problems while a more permanent solution is being sought, for situations where DBPs may be high for certain periods of the year, for houses located on parts of the distribution system that have extremely high residence times and known DBP issues. POU/POE devices are only effective if properly maintained.

Water System Design Measures

- The design of water distribution systems and water treatment plants is not static. New concerns, scientific knowledge, methods and innovations occur over time and those who design drinking water distribution and treatment systems must be flexible and knowledgeable enough to incorporate such changes.

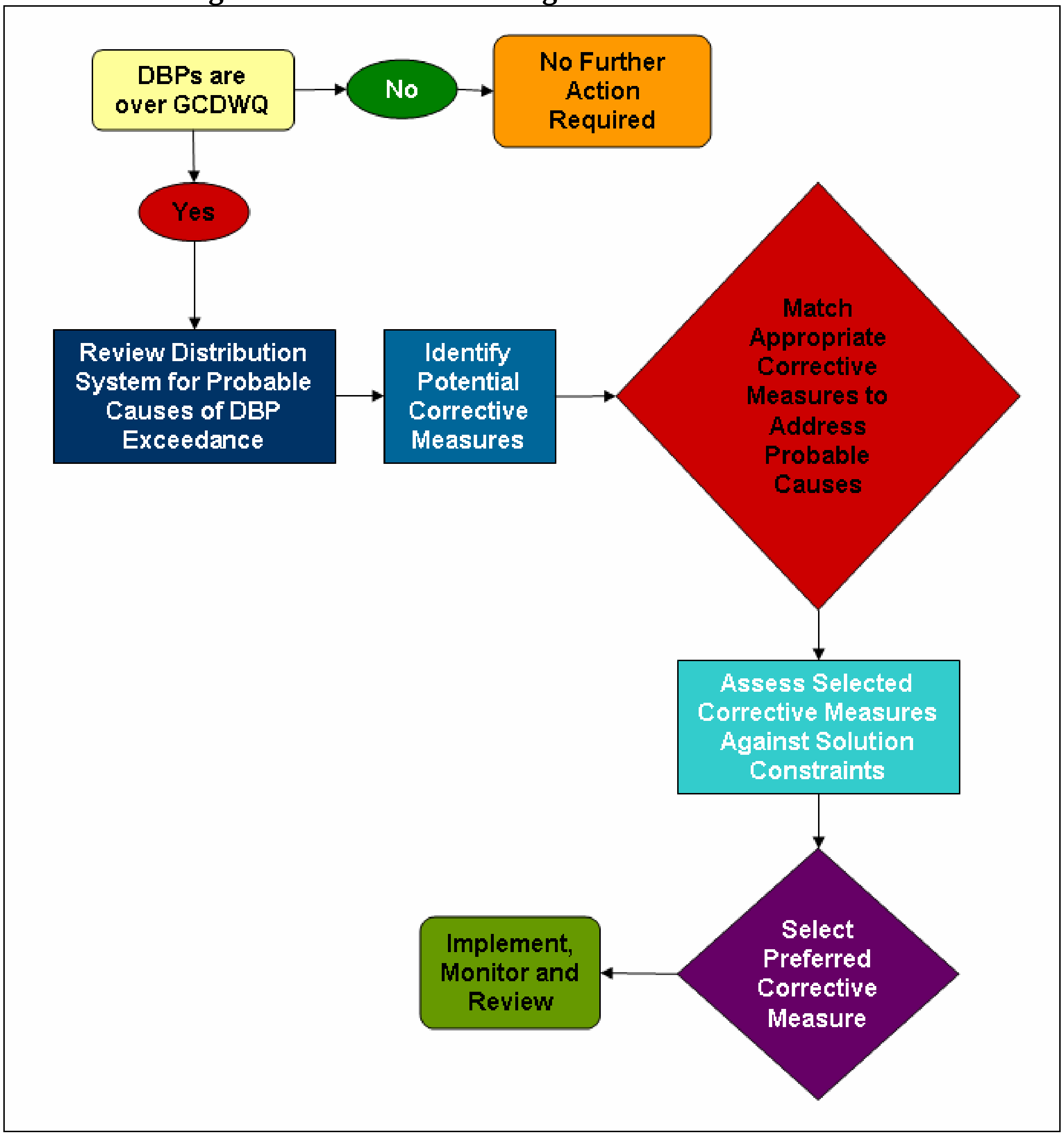
- The NL *Guidelines for the Design, Construction and Operation of Water and Sewerage Systems* should be updated at least every 10 years.
- Distribution system modeling and water treatment plant modeling should be used as a tool in the design of water distribution and water treatment systems.

Operator Education and Training

- Operator education and training is an essential component of any DBP control methodology.
- Communities should require that their water system operators be certified.

Appendix C: Decision Making Framework for Selecting DBP Corrective Measures

Decision Making Framework for Selecting DBP Corrective Measures



Expanded Decision Making Framework for Selecting DBP Corrective Measures

DBPs are over Canadian Drinking Water Quality Guidelines → No

↓ Yes

Review Existing Distribution System for Probable Causes of DBPs → **Corrective Measures (CM)** → **Matching Appropriate DBP Corrective Measures to Probable Causes** → **Assessment of Solution Constraints** (CMs scored from 0-16) → **Select Preferred Solution(s) from Highest Scoring Corrective Measures** → **Implement, Monitor and Review**

Review Existing Distribution System for Probable Causes of DBPs

Source

1. Reservoir contains flooded vegetation
2. Shallow intake (< 1 m of water) or shallow intake pond with long exposed fetch length
3. Surface water source exposed to saltwater influence (<1 km to ocean)

DBP Precursors

4. Mixing of high DOC surface water with groundwater
5. High DOC in source water (> 4.2 mg/L)
6. High levels of bromide in source water (> 0.02 mg/L)
7. High chlorine dose (total dose over 7 mg/L or over 4 mg/L at first point of use)
8. Point of chlorine application in WTP (pre-chlorination)
9. Higher chlorine use with booster system
10. Excessive chlorine demand (bulk < -0.5 d^{-1}, wall < -1 m/d)
11. Extremes of pH (<6.5, > 7.5)

System Characteristics

12. Long linear system (> 3 km)
13. Branched system with multiple dead ends (> 3 DE)
14. Distance of chlorination system to first point of use (> 500 m or contact time > 40 min)
15. Insufficient chlorination controls on system
16. System is oversized (max pipe v < 0.4 m/s)
17. High retention time in network (> 48 hrs)
18. Pipe material and age (> 25 yrs, cast iron)
19. Water treatment plant is undersized

Demand

20. Large occasional demand on system

Tanks

21. Tank location/ configuration
22. Balance between pumped supply and demand not optimized with storage
23. High retention time in tank (> 24 hrs)
24. Dead zones/ poor mixing in tank
25. Little variation in water levels or turnover in tank

O & M

26. Poor operation and maintenance of system

Other

27. Multiple factors listed
28. Poor design of system
29. High iron and manganese (Fe > 0.3 mg/L, Mn > 0.05 mg/L)
30. High per capita demand (> 500 L/p/d)
31. Pressure problems (< 28 m, > 66 m)
32. Problems with chlorine residuals (< 0.05 mg/L, > 4.0 mg/L)

Corrective Measures (CM) / Matching Appropriate DBP Corrective Measures to Probable Causes

Policy	Policy
Policy of point of use/point of entry treatment	All
Policy to promote use of alternative disinfectants	All
Policy to Promote Potable Water Dispensing Units	All

Source	Source
Watershed protection	All
Alternative water sources •groundwater •surface water sources with DOC < 4.2 mg/L •avoid shallow ponds with long exposed fetch lengths	1-3-4-5-6
Stop mixing groundwater with surface water	4
Reservoir flooding: •avoid flooding vegetated areas •remove vegetation before flooding •remove submerged vegetation	1-4-5
Wind breaks around exposed coastal water sources with high DBPs	3-6
Relocate intake to deeper water	2
High quality water storage and recovery	All

Chlorine Demand Management	CDM
Optimize disinfectant dosage if: •Cl > 4.0 mg/L @ first user •Cl > 0.2 mg/L (regularly) @ last user •chlorine booster on system	1-2-3-4-5-6-7-9
Re-locate chlorination system: •closer to first user •downpipe of storage tank	1-2-3-4-5-6-7-9-14-21
Install chlorine booster at optimal location if: •combined Cl dose < single Cl dose •Cl > 4.0 mg/L @ first user	7-10
Chlorine dose control •automated flow or chlorine residual control •dedicated and certified system operator	1-2-3-4-5-6-7-9-15

Retention Time Management	RTM
Tank location and type •at beginning of system •multiple smaller tanks •elevated storage	21-23-24-25
Adjust pump schedule to: •optimize balance between demand and supply •force turnover of water in tank •increase velocity of inflow into tank	17-22-23-24-25
Reduce storage capacity •take tank offline •reduce maximum water level in tank	17-20-23-24-25
Increase mixing in tank: •separate inlet/outlet •baffles, location, orientation of inlet •smaller diameter inlet, duckbill valve at inlet •mechanical mixing device •avoid stratification in tank •increase active volume in tank	17-23-24-25
Tank aeration	17-23-24-25

Demand Management	Demand Mgmt
Regular system flushing at dead ends •automated flushing device or manual flushing	1-2-5-12-13-16-17-20
Continuously bleed system at dead ends	1-2-5-12-13-16-17-20
Increase demand with new water connections	1-2-5-12-13-16-17-20

Operational and Infrastructure	O & I
Optimize valve arrangement •minimize number of shut valves •locate shut valves in areas of high demand	17
Re-routing of flows in the system through valving	17
Pumping to recirculate water in the distribution system	17
Abandoning or downsizing mains	1-2-5-12-13-16
Clean, replace or reline: •old pipe, cast iron pipe	18
Loop distribution network	13
Upgrade distribution system •reconfigure, replace, abandon pipe •new intake	2-18
System maintenance •flushing, reservoir cleaning •swabbing or pigging •pump, flowmeter maintenance	All
Increase capacity of water treatment plant	19
Regionalization •regional system or regional operator	All

Alternative Disinfectants	Alt Disinfectant
Disinfection with chloramines	All
Disinfection with ozone	All
Disinfection with UV	All
Disinfection with MIOX	All

Treatment	Treatment
Water treatment plants •conventional WTP •targeted removal of precursors during seasonal extremes (DOC in summer/fall)	4-5-7-9
Point of application of chlorine in WTP •use alternative pre-disinfectant •no pre-chlorination	8
Filtration •ultrafiltration or nanofiltration •appropriately sized and maintained	4-5-7-9
pH adjustment	11
Iron/Manganese removal •oxidation and filtration	7-9-10-29
Advanced treatment for large systems •EC, RO, GAC, DAF for natural organic material •RO, EDR, IX for bromide	3-4-5-6-7-9
Potable Water Dispensing Units (PWDUs) •community support	All

Point of Use	Point of Use
Point of use/point of entry treatment	All

Training	Training
Operator education and training	All-26
Operator certification	All-26

Design	Design
Improved design of water distribution and treatment systems •modeling • *BMPs for the Control of Disinfection By-Products*	28

Combination	Combination
Combination of corrective measures •where a single corrective measure is ineffective	All

Assessment of Solution Constraints

Effectiveness
- low (0)- no substantial reduction in DBPs
- moderate (1)- DBPs near guideline
- high (2)- DBPs below guideline

Cost
- high (0)- >$500,000
- moderate (1)- $150,000-$500,000
- low (2)- <$150,000

Time Scale for Implementation
- long (0)- >1 year
- moderate (1)- 1 to 12 months
- short (2)- within a month

Permanency of Solution
- short (0)- temporary
- moderate (1)- useful life < 15 years
- permanent (2)- useful life > 15 years

Adverse Hydraulic Effects
- adverse effect (0)- violates criteria
- no impact (1)
- positive effect (2)- system closer to ideal

Adverse Impact on Water Quality
- WQ deteriorates (0)- parameters show deterioration
- no impact (1)
- WQ improves (2)- parameters show improvement

Acceptable to Stakeholders
- against (0)- known or perceived issues
- indifferent (1)- grey area
- support (2)- no issues

Meets all Necessary Regulations
- violates regulations (0)
- borderline (1)- minor, temporary, insignificant violation
- meets regulations (2)

Appendix D: Resource Intensity of Selected Corrective Measures

Corrective Measure	Cost Estimate (CD$)	Comments	Resource Intensity (H-M-L)
Air stripper	-		H
Distillation	$2,588,000		H
Alternative water source	$2,000,000	surface water, groundwater	H
Chlorine dioxide	$1,800,000		H
Reverse osmosis	$1,578,555		H
Activated carbon filters	$1,344,695		H
Electrodialysis reversal	$1,333,002		H
Conventional treatment	$824,357		H
Oxidation (O3, Cl2,CO2)	$800,000		H
Ozone	$800,000		H
Microfiltration	$672,348		H
Acitvated alumina	$549,571		H
Coagulation/filtration	$537,878		H
Retention time management	$500,000	proper tank location/configuration	H
High quality water storage and recovery	$500,000		H
Increase capacity of water treatment plant	$500,000		H
Enhanced coagulation and filtration	$466,551		M
Adsorption/filtration	$456,027		M
Ion exchange	$429,718		M
UV	$400,000		M
Chloramines	$400,000		M
Lime softening	$397,562		M
Direct filtration	$317,465		M
Greensand filtration	$315,126		M
Filtration and disinfection	$305,772		M
Chlorine- gas	$200,000		M
MIOX	$150,000		M
Relocate intake to deeper water	$150,000		M
PWDU	$110,000		L
Chlorine- hypo	$100,000		L
Install chlorine booster	$100,000		L
Re-locate chlorination system	$100,000		L
Pumping to recirculate water in system	$90,000	per pump	L
Clean pipes	$85,000	swabbing or pigging	L
pH adjustment	$75,000		L
Corrosion Inhibitors	$75,000		L
Chlorine dose control	$75,000		L
Increase mixing in tank	$75,000		L
Regular system flushing	$75,000	per flushing devise	L
Regionalization	$65,000	per operator	L
Adjusting pump schedule	$25,000		L
Reduce storage capacity	$10,000		L
Watershed protection	$5,114	designation fee, annual costs	L
Optimize valving	$5,000	per gate valve	L
Increase demand with new water connections	$1,500	per connection	L

Water user treatment	$950	POE or POU devices (per household per year)	L
Reservoir flooding	$250	per hecter for clearing	L
New pipe- downsize, replace, reline, loop	$70	per m of pipe	L
Education and training	$50	per certification test	L
Wind breaks around water sources	$50	per m of fence	L
Improved design	-		L
Policy	-		L
Continuously bleed system	-		L
System maintenance	-		L

[*]Equipment and O&M costs based on water use of 500,000 USGal/d (Bureau of Reclamation, 2001)

[±]Costs do not include general sitework, building, external pumps/piping, pretreatment, or sludge disposal (Bureau of Reclamation, 2001)

[§]Cost based on Newfoundland and Labrador Capital Works Funding Requests

[♣]Cost is a factor of chlorination system cost

[€]Other sources

[∞]Costs are for most expensive option

[¥]US inflation since Sept 2001 as of Dec 14, 2007 :16.93%

[Ω]1 US$: 0.994431 CD$ as of Dec 14, 2007

[♠]Demand = 45,425m3/d or 450 L/p/d for 100,945 people

Appendix E: Checklist of Information on Community Drinking Water Distribution Systems

Government of Newfoundland and Labrador
Department of Environment and Conservation
Water Resources Management Division

Checklist of Community Information for DBP Management

	Data	Qualifier
Community		
LPG Number		
Source Name		
Water Supply Number		
Population		
Average THMs at end of system		ug/L
Average BDCM at end of system		ug/L
Water treatment		
Any major changes to water system		
Watershed size- surface water source		km2
Reservoir contains flooded vegetation		Yes/No, area flooded km2
Length of intake into water		m
Depth of intake below water surface		m
Distance from nearest edge of surface water source to ocean		m
Direction of ocean to surface water source		N, S, E, W
Surface water and groundwater mixed in distribution system		Yes/No
Average DOC in source water		mg/L
Average Bromide in source water		mg/L
Average pH of source water		
Average iron in source water		mg/L
Average manganese in source water		mg/L
Pumped or gravity distribution system		pumped, gravity
Amount of Chlorine gas used per day		kg, lb
% solution of liquid chlorine		%
Volume of liquid chlorine tank		L
Volume of liquid chlorine to water used to fill tank		L:L
Frequency chlorine tank refilled		days
Chlorine dose		mg/L
Max chlorine residual at 1st user		mg/L
Min chlorine residual at 1st user		mg/L
Max chlorine residual at last user		mg/L
Min chlorine residual at last user		mg/L
Max chlorine residual after chlorine booster		mg/L
Min chlorine residual after chlorine booster		mg/L
Point of application of chlorine in WTP		start, middle, end
Increase in total chlorine use with a chlorine booster		Yes/No
Bulk chlorine demand		1/d
Wall chlorine demand		m/d
Chlorination control system		manual, flow proportional, residual analyser
Length of longest run of pipe in distribution system- intake to end		km
Total length of pipe in distribution system		km
Number of dead ends in distributrion system		
Distance of main chlorination system to first point of use		m
Contact time at peak flow		min
CT value at 1st user		
Meter on distribution system		Yes/No
Velocity range in mains		m/s
Pipe size range		mm
Average daily flow		L/s, m3/d, Gal/d
Max peak flow observed		L/s, Gal/s
Retention time in network		hours
Pipe material		DI, CI, PVC, HDPE
Year oldest pipe on distribution system installed		
Water treatment plant is undersized		Yes/No
Large occational demand on system		Fishplant, Tourism, Golf course, No
Non-residential demand (fishplant)		m3/d
Per capita demand		L/p/d
Maximum water pressure in distribution system		m
Minimum water pressure in distribution system		m
Number of fire hydrants on distribution system		
Pump operation		with demand, tank levels, with pressure
Type of tank		standpipe, elevated, in ground, on ground
Tank location		beginning, middle, end
Tank volume		m3
Tank dimensions		L-W-H, D-H, D
Maximum water height in tank		m
Water level variation in tank		m
Time to fill tank: Time to empty tank		hours: hours
Retention time in tank		hours
Inlet/outlet in tank the same		Yes/No
Location of inlet/outlet- height		bottom, middle, top
Length and height between inlet/outlet		m
Percent inactive volume in tank		%
Percent dead volume in tank		%
Percent active volume in tank		%
Frequency tank is cleaned		times/yr, every x yrs
Does all water spend time in storage tank		Yes/No
Level of operator certification		I, II, III, IV
Frequency distribution system is flushed		per year

Government of Newfoundland and Labrador
Department of Environment and Conservation
Water Resources Management Division

Checklist of Community Information for DBP Management

	Data	Qualifier
Community	Summerford	
LPG Number	4975	
Source Name	Rushy Cove Pond	
Water Supply Number	WS-S-0721	
Population	976	
Average THMs at end of system	239	ug/L
Average BDCM at end of system	12.6	ug/L
Water treatment	pH adjustment	
Any major changes to water system	No	
Watershed size- surface water source	0.51	km2
Reservoir contains flooded vegetation	No	Yes/No, area flooded
Length of intake into water	30	m
Depth of intake below water surface	>1	m
Distance from nearest edge of surface water source to ocean	800	m
Direction of ocean to surface water source	NW	N, S, E, W
Surface water and groundwater mixed in distribution system	No	Yes/No
Average DOC in source water	7.35	mg/L
Average Bromide in source water	0.015	mg/L
Average pH of source water	7.48	
Average iron in source water	0.05	mg/L
Average manganese in source water	0.014	mg/L
Pumped or gravity distribution system	pumped	pumped, gravity
Amount of Chlorine gas used per day	10 lb	kg, lb
% solution of liquid chlorine		%
Volume of liquid chlorine tank		L
Volume of liquid chlorine to water used to fill tank		L:L
Frequency chlorine tank refilled		days
Chlorine dose	3.95- 5.55	mg/L
Max chlorine residual at 1st user	3.1	mg/L
Min chlorine residual at 1st user	0.5	mg/L
Max chlorine residual at last user	0.02	mg/L
Min chlorine residual at last user	1.6	mg/L
Max chlorine residual after chlorine booster		mg/L
Min chlorine residual after chlorine booster		mg/L
Point of application of chlorine in WTP		start, middle, end
Increase in total chlorine use with a chlorine booster		Yes/No
Bulk chlorine demand	?	1/d
Wall chlorine demand	?	m/d
Chlorination control system	flow proportional	manual, flow proportional, residual analyser
Length of longest run of pipe in distribution system- intake to end	6	km
Total length of pipe in distribution system	?	km
Number of dead ends in distributrion system	~6	
Distance of main chlorination system to first point of use	1.5	m
Contact time at peak flow	38-41	min
CT value at 1st user	19-21	
Meter on distribution system	Yes	Yes/No
Velocity range in mains	0.19-0.27	m/s
Pipe size range	250	mm
Average daily flow	816-1148 m3/d, 9.44-13.3 L/s	L/s, m3/d, Gal/d
Max peak flow observed		L/s, Gal/s
Retention time in network	24.8-25.2	hours
Pipe material	DI, PVC	DI, CI, PVC, HDPE
Year oldest pipe on distribution system installed	1984	
Water treatment plant is undersized		Yes/No
Large occational demand on system	fishplant in Cottlesville	Fishplant, Tourism, Golf course, No
Non-residential demand (fishplant)	332	m3/d
Per capita demand	710	L/p/d
Maximum water pressure in distribution system		m
Minimum water pressure in distribution system		m
Number of fire hydrants on distribution system	56	
Pump operation	pressure pump	constant, with demand, tank levels, booster
Type of tank		standpipe, elevated, in ground, on ground
Tank location		beginning, middle, end
Tank volume		m3
Tank dimensions		L-W-H, D-H, D
Maximum water height in tank		m
Water level variation in tank		m
Time to fill tank: Time to empty tank		hours: hours
Retention time in tank		hours
Inlet/outlet in tank the same		Yes/No
Location of inlet/outlet- height		bottom, middle, top
Length and height between inlet/outlet		m
Percent inactive volume in tank		%
Percent dead volume in tank		%
Percent active volume in tank		%
Frequency tank is cleaned		times/yr, every x yrs
Does all water spend time in storage tank		Yes/No
Level of operator certification	I	I, II, III, IV, none
Frequency distribution system is flushed	2 time/ yr	

Government of Newfoundland and Labrador
Department of Environment and Conservation
Water Resources Management Division

Checklist of Community Information for DBP Management

	Data	Qualifier
Community	Port Saunders	
LPG Number	3975	
Source Name	Tom Taylor's Pond	
Water Supply Number	WS-S-0589	
Population	747	
Average THMs at end of system	138 since 2000	ug/L
Average BDCM at end of system	10.7 since 2000	ug/L
Water treatment	No	
Any major changes to water system		
Watershed size- surface water source	13.3	km2
Reservoir contains flooded vegetation	No	Yes/No, area flooded km2
Length of intake into water		m
Depth of intake below water surface		m
Distance from nearest edge of surface water source to ocean	600	m
Direction of ocean to surface water source	W	N, S, E, W
Surface water and groundwater mixed in distribution system	No	Yes/No
Average DOC in source water	6.03	mg/L
Average Bromide in source water	0.024	mg/L
Average pH of source water	7.9	
Average iron in source water	0.05	mg/L
Average manganese in source water	0.0188	mg/L
Pumped or gravity distribution system	pumped	pumped, gravity
Amount of Chlorine gas used per day	5.5 lb	kg, lb
% solution of liquid chlorine		%
Volume of liquid chlorine tank		L
Volume of liquid chlorine to water used to fill tank		L:L
Frequency chlorine tank refilled		days
Chlorine dose	2.67-7.41	mg/L
Max chlorine residual at 1st user	0.98	mg/L
Min chlorine residual at 1st user	0.08	mg/L
Max chlorine residual at last user	?	mg/L
Min chlorine residual at last user	0.1	mg/L
Max chlorine residual after chlorine booster		mg/L
Min chlorine residual after chlorine booster		mg/L
Point of application of chlorine in WTP		start, middle, end
Increase in total chlorine use with a chlorine booster		Yes/No
Bulk chlorine demand	?	1/d
Wall chlorine demand	?	m/d
Chlorination control system	manual	manual, flow proportional, residual analyser
Length of longest run of pipe in distribution system- intake to end	5	km
Total length of pipe in distribution system	10	km
Number of dead ends in distributrion system	12	
Distance of main chlorination system to first point of use	750	m
Contact time at peak flow	1704-2132	min
CT value at 1st user	1363	
Meter on distribution system	No	Yes/No
Velocity range in mains	0.12-0.61	m/s
Pipe size range	150-200	mm
Average daily flow	3.89-10.8 L/s	L/s, m3/d, Gal/d
Max peak flow observed		L/s, Gal/s
Retention time in network	28.4-35.5	hours
Pipe material	DI, PVC	DI, CI, PVC, HDPE
Year oldest pipe on distribution system installed	older than 1990	
Water treatment plant is undersized		Yes/No
Large occational demand on system	Fishplant	Fishplant, Tourism, Golf course, No
Non-residential demand (fishplant)		m3/d
Per capita demand	450-1249	L/p/d
Maximum water pressure in distribution system		m
Minimum water pressure in distribution system		m
Number of fire hydrants on distribution system	yes	
Pump operation	tank level control	with demand, tank levels, with pressure
Type of tank	in ground	standpipe, elevated, in ground, on ground
Tank location	beginning	beginning, middle, end
Tank volume	560	m3
Tank dimensions	17-9-3.7 m	L-W-H, D-H, D
Maximum water height in tank	3.05	m
Water level variation in tank	0.61	m
Time to fill tank: Time to empty tank	1:1	hours: hours
Retention time in tank	24.3	hours
Inlet/outlet in tank the same	No	Yes/No
Location of inlet/outlet- height	bottom	bottom, middle, top
Length and height between inlet/outlet	h-0.15 m, l-0.61m	m
Percent inactive volume in tank	17	%
Percent dead volume in tank	17	%
Percent active volume in tank	67	%
Frequency tank is cleaned	2	times/yr, every x yrs
Does all water spend time in storage tank	Yes	Yes/No
Level of operator certification	I	I, II, III, IV
Frequency distribution system is flushed	2	per year

Government of Newfoundland and Labrador
Department of Environment and Conservation
Water Resources Management Division

Checklist of Community Information for DBP Management

	Data	Qualifier
Community	Arnold's Cove	
LPG Number	110	
Source Name	Steve's Pond	
Water Supply Number	WS-S-0006	
Population	1003	
Average THMs at end of system	68	ug/L
Average BDCM at end of system	2.4	ug/L
Water treatment	pH adjustment	
Any major changes to water system	intake moved b/f 1991, switched to manual Cl control ~2004	
Watershed size- surface water source	3.81	km2
Reservoir contains flooded vegetation	dam on pond- possible	Yes/No, area flooded km2
Length of intake into water		m
Depth of intake below water surface		m
Distance from nearest edge of surface water source to ocean	4500	m
Direction of ocean to surface water source	SW	N, S, E, W
Surface water and groundwater mixed in distribution system	No	Yes/No
Average DOC in source water	5.3	mg/L
Average Bromide in source water	0.02	mg/L
Average pH of source water	6.42	
Average iron in source water	0.1	mg/L
Average manganese in source water	0.026	mg/L
Pumped or gravity distribution system	gravity	pumped, gravity
Amount of Chlorine gas used per day	10 lbs	kg, lb
% solution of liquid chlorine		%
Volume of liquid chlorine tank		L
Volume of liquid chlorine to water used to fill tank		L:L
Frequency chlorine tank refilled		days
Chlorine dose	4.17	mg/L
Max chlorine residual at 1st user	1.63	mg/L
Min chlorine residual at 1st user	0.79	mg/L
Max chlorine residual at last user	1.36	mg/L
Min chlorine residual at last user	0.06	mg/L
Max chlorine residual after chlorine booster		mg/L
Min chlorine residual after chlorine booster		mg/L
Point of application of chlorine in WTP		start, middle, end
Increase in total chlorine use with a chlorine booster		Yes/No
Bulk chlorine demand	?	1/d
Wall chlorine demand	?	m/d
Chlorination control system	manual (active), flow prop	manual, flow proportional, residual analyser
Length of longest run of pipe in distribution system- intake to end	9.4	km
Total length of pipe in distribution system		km
Number of dead ends in distributrion system	~3	
Distance of main chlorination system to first point of use	1400	m
Contact time at peak flow	179 w/o fishplant, 40 w fishplant	min
CT value at 1st user	31.6	
Meter on distribution system	Yes	Yes/No
Velocity range in mains	0.13-0.72	m/s
Pipe size range	150-350	mm
Average daily flow	1088.6 m3/d, 12.6 L/s	L/s, m3/d, Gal/d
Max peak flow observed		L/s, Gal/s
Retention time in network	53 w/o fishplant, 9.8 w fish plant	hours
Pipe material	DI, PVC, Poly Pipe	DI, CI, PVC, HDPE
Year oldest pipe on distribution system installed		
Water treatment plant is undersized		Yes/No
Large occational demand on system	fishplant	Fishplant, Tourism, Golf course, No
Non-residential demand (fishplant)	4907	m3/d
Per capita demand	1085	L/p/d
Maximum water pressure in distribution system	70.3	m
Minimum water pressure in distribution system	29.5	m
Number of fire hydrants on distribution system	50	
Pump operation	pressure booster pump	with demand, tank levels, with pressure
Type of tank		standpipe, elevated, in ground, on ground
Tank location		beginning, middle, end
Tank volume		m3
Tank dimensions		L-W-H, D-H, D
Maximum water height in tank		m
Water level variation in tank		m
Time to fill tank: Time to empty tank		hours: hours
Retention time in tank		hours
Inlet/outlet in tank the same		Yes/No
Location of inlet/outlet- height		bottom, middle, top
Length and height between inlet/outlet		m
Percent inactive volume in tank		%
Percent dead volume in tank		%
Percent active volume in tank		%
Frequency tank is cleaned		times/yr, every x yrs
Does all water spend time in storage tank		Yes/No
Level of operator certification	not certified	I, II, III, IV
Frequency distribution system is flushed	1	per year