Study on Water Quality and Demand on Public Water Supplies with Variable Flow Regimes and Water Demand

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Prepared for:

Newfoundland Labrador

Prepared by:

CBCL LIMITED
Consulting Engineers

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EXECUTIVE SUMMARY

This report is the main deliverable of the project entitled *'Study on Water Quality and Demand on Public Water Supplies with Variable Flow Regimes and Water Demand',* which was initiated by the Department of Environment and Conservation (ENVC) in the spring of 2010. The primary goal of the study was to determine the effects of variable water demands on water quality and use in a selection of communities that supply a large industrial user in addition to the usual assortment of residential, commercial, and institutional users. This was accomplished by assessing the design and operation of the water supply systems in light of historical water use records and water quality records.

The components of the water supply system (i.e., treatment, distribution mains and storage) must be sized to accommodate the demands of all users on the system effectively without compromising water quality or wasting land, energy, or chemicals. Industrial users often require large volumes of water, which increases the size of the various components of the water supply system.

The amount of water used by a community varies from hour to hour, day to day, and season to season. Over the course of the day, most communities follow a standard diurnal curve, with higher water use early in the morning and early evening and lower demand late at night and in the afternoon. Industrial, commercial, and institutional users can change the shape of the curve, often leading to less variation over the course of the day. Community demographics and cultural practices can also impact the shape of the curve.

Water use also varies by day and by season. For example, industrial users that operate only seasonally can introduce large variations in water demand over the course of the year. In Newfoundland and Labrador, this has often led to the installation of water supply systems designed to accommodate the large demands of the industrial user. This can result in operational problems and/or poor water quality during periods of low flow when the water retention time in the system increases.

The average daily water demand should be determined using historical water use records. It can also be approximated using assumed per capita flow rates and peaking factors, though this is not recommended as water use varies can vary significantly from one community to the next. A program encouraging the installation of flow meters/totalizers and careful record-keeping among system operators should be initiated. This will reduce designers' reliance on assumed per capita values.

The current design guidelines for the province of Newfoundland and Labrador recommend using a daily per capita demand of 340 Lpcd. Publications and regulations from across Canada suggest a range of default per capita demands ranging from less than 200 Lpcd to over 800 Lpcd. Three of the four of the case study communities in this study had average per capita flow rates above 1,000 Lpcd, suggesting that the province should consider increasing the recommended average daily per capita water use value in their design guidelines. If default average per capita demands are used to calculate average day demand, efforts should also be made to identify factors that might impact water demand in individual communities (leakage, winter water use, etc.).

The Atlantic Canada design guidelines recommend that water treatment processes be designed to provide the maximum day water demand of the community. Water storage is also sized based on MDD. Like the ADD, this value is usually determined by evaluating historical water use records. If these are not available, the ADD can be multiplied by a peaking factor to obtain an estimate of the MDD. These peaking factors vary inversely based on population size such that the smaller the community, the larger the peaking factor. This helps to account for the larger variation in day to day water use in small communities, where each user has more of an impact on total water use than they would in a larger community. Commonly used peaking factors were not adequate to describe the periodic increase in water use in many of the communities that participated in this study. As a result, the province may choose to recommend the use of larger peaking factors for small communities with large industrial users.

Peaking factors can also be used to determine the peak hour flow, which is used to size distribution mains and calculate disinfection compliance. As with maximum day peaking factors, peak hour peaking factors vary inversely with population. Many different calculation methods and empirical values can be used to determine the proper peak hour peaking factor. All have both advantages and disadvantages. The province currently relies on the Harmon formula, which was originally developed to size wastewater facilities, to calculate peak hourly demands. The results of the literature search conducted as part of this study suggest that the PRP-Gumble method (Zhang, 2005) is more appropriate for the design of water systems for populations above 1,000. Where feasible, communities with more than 25,000 inhabitants should be encouraged to develop water demand estimates for individual subdivisions. Most available peaking factor calculation methods are only valid for populations above 1,000, so smaller communities should be encouraged to rely on empirical values, such as those provided by the Ontario Ministry of the Environment (2008), to calculate peak hourly demands.

Water use is a function of the number and type of users in the community but is also influenced by the characteristics of the water treatment and distribution systems. Most water treatment processes rely on filters, which must be backwashed regularly. Leakage-related losses in some Canadian water distribution systems have been estimated at over 20% of the total volume of treated water. These losses add to the overall water demand of the system and must be taken into account during the design process.

Common problems associated with long water retention time (water age) include the formation of disinfection by-products (DBPs), low chlorine residuals, corrosion and solubilization of corrosion products, bacterial regrowth, and nitrification. Low chlorine residuals and bacterial regrowth can result in the initiation of a boil water advisory (BWA). Although some of these problems were identified in the participating communities, it was not possible to develop statistical relationships between historical water

quality and changing water demands because the ENVC water sampling schedules rarely corresponded to fish plant operation schedules. Descriptive analysis was used to draw connections between water age and the formation of DBPs and to establish whether BWAs were linked to fish plant operation.
The ENVC originally provided CBCL with a list of communities in the province thought to have operating fish plants. Twenty-four were chosen for the study, including twelve that fell within the 'small' and 'very small' categories defined in the ENVC's *Sustainable Options Report*. Historic water quality records, as-built drawings, and system maps were collected from the ENVC and Department of Municipal Affairs (DMA) for each of the communities (where available). Afterwards, all communities were contacted by fax and by phone to arrange for site visits.

During this initial contact phase, it was determined that the fish plants in many communities were no longer operating. These communities were removed from the list. In the end, fifteen communities were visited by CBCL staff in the summer and fall of 2010. During the site visits the staff member interviewed the system operator, toured the water system, and took pictures of important system components. Where possible, the staff member also toured the fish plant and interviewed the plant operator. Water use and chlorine residual records were also collected. All data, notes, and pictures were forwarded to CBCL engineers and assessed in detail. Numerous communities were contacted a second time to obtain further information and water use data.

Of these, four communities that were able to provide detailed information and records were identified as being representative of the overall group. The systems in these communities vary in size and sophistication. They are distributed fairly evenly throughout the province. Each was evaluated for average day, maximum day, peak hour, and minimum hour demands both with and without the fish plant operating. The results were compared to estimates using per capita water demands and peaking factors available in the provincial design guidelines and similar publications from other jurisdictions.

At the outset of the study it was decided that the water demand analysis would be limited to daily water use records. This decision was based on the anticipated difficulties associated with the installation of the portable flow meter required to monitor diurnal water use. Indeed, during the field portion of the project it was determined that accurate diurnal readings would only be obtainable in a small number of communities. This decision was reversed during subsequent meetings and additional site visits were conducted in two communities whose water systems were identified as being able to accommodate the installation of a temporary flow meter. In one of these, the flow meter was installed and allowed to record flow data for four days. The results were downloaded into Microsoft Excel and used to develop diurnal curves. The meter could not be installed in the second community so measurements were taken by hand over a period of approximately 22 hours. The water use patterns in the two communities were similar to the standard diurnal curve, though they had some distinct differences. Most obviously, water use peaked more dramatically in the morning than in the afternoon. No definite explanation for this finding was established.

Based on the findings of the study recommendations were made for: distribution system design; system operation, maintenance and monitoring; modifications to existing guidance documents; and public and industrial engagement in water conservation strategies. These are provided in Chapter 10 of this report.

LIST OF ACRONYMS

ACWWA	Atlantic Canada Water and Wastewater Association
ADD	ADD
AHD	Average hour demand
AWWA	American Water Works Association
BF	Baffling factor
BWA	Boil water advisory
DBP	Disinfection by-product
DMA	Department of Municipal Affairs (Newfoundland and Labrador)
DOC	Dissolved organic carbon
DVGW	Deutscher Verein des Gas und Wasserfaches (Germany)
EC	Environment Canada
ENVC	Department of Environment and Conservation (Newfoundland and Labrador)
FUS	Fire Underwriters Survey
GCDWQ	Guidelines for Canadian Drinking Water Quality
HAA	Haloacetic acid
HAAfp	Haloacetic acid formation potential
Lpcd	Litres per capita per day
MDD	Maximum day demand
MOE	Ministry of the Environment (Ontario)
NL	Newfoundland and Labrador
OETC	Operator Education, Training, and Certification Program (Newfoundland and Labrador)
PHF	Peak hour flow
PRP	Poisson rectangular pulse
RFF	Required fire flow
THM	Trihalomethane
THMfp	Trihalomethane formation potential
TOC	Total organic carbon
US EPA	United States Environmental Protection Agency
UV	Ultraviolet

CHAPTER 1 INTRODUCTION

1.1 Project Background

1.1.1 Purpose and Objectives

A formal request for proposals process was conducted in the spring of 2010 for a project entitled 'Study on Water Quality and Demand on Public Water Supplies with Variable Flow Regimes and Water Demand'. CBCL Limited was retained by the Department of Environment and Conservation (ENVC), Water Resources Management Division, to complete the study.

The main objectives of this study were to:

- Identify small communities in Newfoundland and Labrador with large industrial users;
- Determine whether the water demands of the large industrial user had impacts on the design, operation, and effectiveness of the water supply, treatment, and distribution systems;
- Develop strategies to minimize the negative impacts (if any) that the industrial demand is having on the quality of water delivered to residents in the participating communities;
- Identify what problems or issues have been experienced with these systems relating to industrial water use;
- Determine the water demand required by the industrial uses and the communities separately;
- Determine the effects of industrial uses on water demand and the sizing of distribution system infrastructure components;
- Develop daily water demand patterns for each community including when there is and is not seasonal demand on the system;
- Determine the effects of industrial uses and the sizing of distribution systems to accommodate industrial uses on water age and quality;
- Make recommendations for guidelines for the design, construction and operation of very small to medium sized public water supplies with seasonal industrial demands; and
- Make recommendations for guidelines for the design, construction and operation of small to medium sized public water supplies that are designed for fire flows and flushing operations.

1.1.2 Provincial Design Guidelines

The ENVC published the *Guidelines for the Design, Construction, and Operation of Water and Sewerage Systems* in 2005. This document outlines parameters of interest and recommendations for the design of water distribution systems. It specifically requires that systems be designed to accommodate fire flow and specifies a minimum pipe diameter of 150 mm for distribution and service mains providing fire protection.

1.1.3 Recent Provincial Water Quality Studies

It is well established in the water industry that common water quality issues such as low chlorine residuals, disinfection by-products, and lead are exacerbated by long retention times within the distribution system. Two reports published by the ENVC have identified a connection between some common water quality issues, such as Disinfection By-Products, in small communities and the age of the water within their distribution systems.

The first of these, *Sustainable Options for the Management of Drinking Water Quality in Small Water Systems*, discussed the impacts of pipe size and materials on water quality (ENVC, 2009). The author(s) noted that the common practice of over-sizing water distribution systems in an effort to accommodate industrial demands and fire flow requirements can result in long retention times within the system. The report identified a number of communities where water quality concerns might be linked to the presence and operation of a fish plant. Many of these communities have been included in the current study.

Best Management Practices for the Control of Disinfection By-Products in Drinking Water Systems in Newfoundland and Labrador, also published in 2009, included detailed analyses of six distribution systems in the province. Two of the communities who participated in the study have fish plants that draw water from the municipal supply system. Each system was assessed based on historical water quality and what flow data was available. The existing information was input into an EPANET water distribution system model. Values for unknown parameters were assumed based on typical values for small systems. Water age and chlorine decay were modelled for each community. The problems facing the systems were identified based on field visits and the results of the model and corrective measures were recommended.

1.2 Report Organization

This report represents the culmination of a ten-month study investigating the impacts that large industrial users have on the design and operation of water systems in small rural communities in Newfoundland and Labrador.

The report includes eight chapters and three appendices. The first chapter introduces the history, background, and objectives of the project. Chapter 2 is a discussion of the metrics commonly applied in the water industry to determine and evaluate municipal, commercial, and industrial water usage. Chapter 3 provides background information on the impacts of sporadic industrial demands on water quality. Chapter 4 discusses the project methodology while Chapter 5 presents summaries of the information gathered during the desktop and field portions of the study. Chapters 6 to 9 are four case studies developed based on detailed reviews of water use and chlorine residual data provided by four communities who participated in the study. Chapter 10 provides a summary of the findings of the study and a series of recommendations for system design, system operation, public and industrial engagement in water conservation strategies, and modifications to existing guidelines. Chapter 11 lists the references used in the report.

CHAPTER 2 QUANTIFYING WATER USE

2.1 Water Use in Canada

The Organisation for Economic Co-operation and Development has determined that Canada has the second highest average per capita water use rate in the developed world (OECD, 2010). Water use in Canada is regularly tracked and assessed by Environment Canada. Their latest report, released in 2010, presents water use data collected in 2006 from over 1,300 communities across the country. The results of the study highlight differences in per capita water demands between communities of different sizes. For example, it was found that smaller communities have higher average per capita water use rates than large communities. This pattern was apparent whether the average per capita use was calculated based on the total water demand or the residential water demand exclusively. Table 2.1 summarizes the average per capita water usage rates for different size communities in 2006.

Table 2.1 Water use rates in Canada, 2006 (adapted from Environment Canada, 2010)

Population	Total Average Daily Flow (Lpcd)	Average Daily Residential Flow (Lpcd)
< 1,000	923	433
1,000 to 2,000	677	431
2,000 to 5,000	884	496
5,000 to 50,000	693	423
50,000 to 500,000	534	298
> 500,000	569	294

The higher average water demands observed in smaller communities can be interpreted in a number of ways. Statistically, the average per capita water use rate in a small community can easily be skewed by sporadic peaks in demand. That is, when the population is small, the demand exerted by each individual user (residential, commercial, or industrial) has a greater impact on the average demand. Smaller communities are also less likely than larger communities to have implemented conservation measures, such as a volume-based rate structure, because users are rarely metered. The Environment Canada study found that communities where water users are metered have lower per capita water use rates than communities where residents are not metered.

The results of the study also illustrate the gap that exists between rural and urban areas in Canada with regards to basic services. Almost all of the residents of large cities in Canada have access to treated (99%), centrally distributed (99.5%) drinking water in their homes. In contrast, only 65.8% of the residents of communities with fewer than 2,000 inhabitants were served by a central water distribution system. Treated water was only available to 44.1% of those living in these small communities.

2.2 Water Use in Newfoundland and Labrador

A survey conducted by Environment Canada in 2006 found that, at 813 L/person/day (Lpcd), Newfoundland and Labrador had one of the highest per capita water use rates in Canada. When commercial, institutional, and industrial users were excluded, the average per capita water demand in the province dropped to 504 L/person/day. This is the highest average municipal per capita water use among all of the provinces (Environment Canada, 2010).

The high average per capita water use reported for Newfoundland and Labrador is not surprising given that a large proportion of the communities in the province have fewer than 5,000 inhabitants. As discussed previously, the Environment Canada study found that small rural communities had higher per capita water use rates than communities with larger, more urban populations. Other potential explanations for the high water usage rates include:

- Industrial demands (fish plants, etc.);
- Higher water use during winter months to prevent pipe freezing; and
- Leakage within the distribution system.

As well, the results for the province might be slightly skewed by the fact that Environment Canada gathered information from only 71 of the more than 600 communities in Newfoundland and Labrador. Consequently, the data presented in the report may not be representative of water use rates throughout the province.

Recent communications between the ENVC and Environment Canada indicated that the results of their most recent water use survey indicated that the average total and residential per capita water demands in the province had dropped to 804 Lpcd and 395 Lpcd, respectively (Environment Canada, 2011). The Environment Canada representative noted that the 24 communities who participated in the survey were some of the largest in the province. Some of these have instituted water metering for both commercial and residential users, which is generally accepted as a method of minimizing total and residential water demands. Some of the numbers used to calculate the average values were imputed based on the results of previous studies and others may represent overestimates (personal communication, November 21, 2011). Appendix I includes copies of these and the 2011 report along with an analysis of the results.

2.3 Water Users

Most water systems serve a mix of residential, industrial, institutional, and commercial users. Water use patterns amongst these different users in small communities are not as well-understood as those in large cities, partly because the water demands exerted by individual municipal, commercial, institutional, and industrial users have stronger impacts on their water use patterns. Consequently, it is difficult to look at the water demands in these communities without breaking out the individual demands of the different users on each system.

2.3.1 Residential Users

Residential users exert consumption demands (drinking, cooking), indoor non-consumption demands (showers, toilets, laundry), and outdoor non-consumption demands (irrigation, car washing). The total amount of water allotted to each of these is dependent on the culture, climate, and access to high-efficiency water fixtures (ex. low flow toilets and showerheads). The *Atlantic Canada Guidelines for the Supply, Treatment, Storage, Distribution, and Operation of Drinking Water Supply Systems* (AC Guidelines) suggest the following guidance values for single family residential households:

- 3 bedroom home – 1,000 L/day;
- 3 bedroom home with high use fixtures – 1,200 L/day;
- 4 bedroom home – 1,350 L; and
- 4 bedroom home with high use fixtures – 1,500 L.

The guidelines recommend that these values only be used in the absence of reliable water use records and/or provincial water use information.

2.3.2 Industrial Users

Industrial users often exert large water demands on a water system. These demands may be regular or sporadic depending on the operating schedule of the facility. For example, many of the municipal water distribution systems in small communities in Newfoundland and Labrador serve fish plants that operate for only part of the year. When the plant is in operation, the water demands exerted on the system are characterized by large, regular demands during each shift at the plant. When the fish plant is not operating, water use patterns are more characteristic of small, rural communities in Canada.

2.3.3 Commercial and Institutional Users

Commercial and institutional users also exert water demands. The volume, frequency, and schedule of their water demands vary based on the type of user. For example, schools and businesses are more likely to exert demands between 8 am and 6 pm while hospitals and retirement homes will use water throughout the day. Suggested allowances for commercial and institutional users are provided in the *Ontario Design Guidelines for Drinking Water Systems* (MOE Guidelines) and summarized in Table 2.2.

Table 2.2 Allowances for commercial and industrial users (adapted from MOE, 2008)

Commercial/Institutional User	Water Use Allowance	Units
Shopping centres	2,500 to 5,000 L	$L/m^2/day$
Hospitals	900 to 1,800 L	L/bed/day
Schools	70 to 140 L	L/student/day
Mobile home parks	1,000 L	L/space/day
Campgrounds	225 to 570 L	L/site/day
Motels	150 to 200 L	L/bed/day
Hotels	225 L	L/bed/day

2.4 Quantifying Water Demand

The amount of water used in a community will vary over the course of the day and throughout the year as a result of differences in instantaneous water use among users over time. For example, in residential areas, water use peaks in the morning and the early evening when most residents are preparing for work and/or meals. Some communities experience elevated water demands in the summer when agricultural and lawn watering water requirements are highest. Other communities, particularly in northern regions where distribution pipes tend to freeze, will have higher water demands in the winter when residents run their taps to prevent the pipes from bursting.

Water supply, treatment, and distribution infrastructure is designed to be able to provide sufficient water to meet peak demand as required. The main design parameters used for sizing disinfection and water treatment equipment are the average day demand (ADD) and the maximum day demand (MDD). The components of the distribution system are designed to accommodate the average hour demand (AHD), the peak hour demand (PHD), and the required fire flow (RFF).

2.4.1 Average Day Demand (ADD)

The ADD represents the total water demand that is exerted on the system by all users on a normal day. The AC Guidelines recommend that historical water use records be used to determine the ADD of a community. If insufficient data is available to develop a reasonable estimate of water use within the community, a published average per capita water demand may be used instead. This approach is not recommended, however, as published per capita demands often fail to take into account the individual water use patterns found in smaller communities, particularly those with large industrial users. Thus, they frequently under or overestimate the total daily water use. Per capita values and recommended demand assessment methods used in various Canadian jurisdictions are compared in Table 2.3.

Note that the reported water use can vary depending on the monitoring location. For example, monitoring at the inlet of the water treatment system will capture the total amount of water used by the community along with that lost in the treatment system (backwashing) and the distribution system (leakage). Conversely, relying entirely on user meter readings will underestimate the total water that must be removed from the source because it does not include treatment or distribution losses.

Table 2.3 Per capita water demands and demand assessment methods used in Canada

Jurisdiction	Year of Publication	Residential Lpcd	Industrial Lpcd	Total Lpcd	Other
NL Design Guidelines	2005	340			Use historical records if available. Recommended per capita water demand only applies to residential-only systems.
Atlantic Canada Design Guidelines	2004	refer to historical records	refer to historical records	refer to historical records	Small systems should use 1,000 to 1,500 L/unit/day for small systems (< 167 units); readers referred to other publications for standard per capita demands.
MOE Guidelines	2008	270 to 450			Note that residential demand can vary from 260 to 1,500 Lpcd.
Environment Canada Survey -(NL results)	2006	504	n/a	813	
	2010	395	n/a	804	
Quebec Design Guidelines	2006	n/a	n/a	465	A study conducted by Reseau (2000) found that per capita water use varied from 360 Lpcd to 1,103 Lpcd in the province.
Ten State Standards	2007	refer to historical records	refer to historical records	refer to historical records	
First Nations Design Standards	2006	refer to historical records	refer to historical records	refer to historical records	

2.4.2 Maximum Day Demand (MDD)

The MDD is the maximum water demand that can be expected from the community on any given day. Like ADD, it can be determined based on historical water use data. Most historical records will show changing water usage patterns over the course of each year. The MDD value should accurately represent the demand during these peak demand periods. Care must be taken to avoid choosing an MDD value that is associated with a unique, not to be repeated event such as a major main break or fire event.

If representative water use data is not available, peaking factors can be used to predict the MDD. As shown in Equation 2.1, the measured or assumed ADD is multiplied by a daily peaking factor (f_d) that can range from 1.5 to 4.0 and is based upon the size and characteristics of the community being analyzed.

$$MDD = f_d \times ADD$$ **Equation 2.1**

Smaller communities are assigned larger peaking factors because they are expected to have higher levels of flow variation due to the higher potential impact of individual events (ex. lawn watering, car washing) on the overall demand placed on the water distribution system.

Over the years, MDD peaking factors have been developed by industry professionals and academics to allow engineers and utilities to predict system demands in the absence of reliable historical water use data. Peaking factors for populations ranging from 30 to 150,000 people are provided in the MOE, NL, and AC guidelines. These have been developed empirically over time based on observation and analysis made by engineers and utilities of existing water treatment and distribution systems. A summary of the recommended maximum day peaking factors for small communities is provided in Table 2.2.

German researchers have developed equations that can be used to determine specific peaking factors that can be solved for any given population. A number of these have been summarized by Diao et. al. (2010). For example, according to Mutschmann and Stimmelmyer (2007), Equation 2.2 can be used to solve for the daily peaking factor ('P' represents population).

$$f_d = -0.1591(\ln P)+3.5488 \qquad \textbf{Equation 2.2}$$

Thus, the peaking factor is entirely dependent on the population (P). This equation was originally presented in design guidelines published by the German Technical and Scientific Association for Gas and Water (DVGW) in 2004. Equation 2.3 was further refined for the 2007 version of the guidelines to provide a more accurate prediction of the daily peaking factor that can be expected for different populations:

$$f_d = 3.9(P^{-0.0752}) \qquad \textbf{Equation 2.3}$$

Some examples of MDD peaking factors drawn from the aforementioned sources are provided in Table 2.4.

Table 2.4 MDD peaking factors

Population Size	MOE (2008)*	Mutschmann and Stimmelmayr (2007)	DVGW (2007)
150	4.90	2.75	2.68
300	3.60	2.64	2.54
500	2.90	2.56	2.44
500 – 1,000	2.75	2.45 – 2.56	2.32 – 2.44
1,001 – 2,000	2.50	2.34 – 2.45	2.20 – 2.32
2,001 – 3,000	2.25	2.27 – 2.34	2.14 – 2.20
3,001 – 10,000	2.00	2.08 – 2.27	1.95 – 2.14

*Also found on page 3-110 of the NL Guidelines (Table 3.7)

The peaking factors obtained using the equations provided by German researchers and government documents more or less match those available in the MOE design guidelines for populations above 1,000. When smaller populations are considered, however, the peaking factors derived from the German equations underestimate the variation in demand. This may reflect the fact that they are generally only used for demand forecasting in communities with more than 500 people.

The choice of peaking factor can impact water age and energy consumption. If the peaking factor is too low, the system will use more energy during periods of high demand. If the chosen peaking factor is too large, however, water age can increase during periods of low demand (Diao et al., 2010). The presence of industrial, commercial, and institutional water demands must also be considered, particularly in smaller communities, as they may impact demand only sporadically. This can lead to a need for water at volumes above that predicted using population-based peaking factors.

2.4.3 Projecting Future Water Demands

Communities inevitably change over time and many of these changes will impact the total quantity of water consumed. For example, the population of the town may increase or decrease depending on birth rates, death rates, immigration, and emigration. The percentage of the total demand represented by residential, industrial, institutional, and commercial users may change as services are added or removed from the community.

Proper infrastructure planning requires that engineers, planners, and utilities have reasonable estimates of the quantity of water that will be required in future years. This information is used to assess source water development options, size water treatment, storage, and distribution systems, and to establish appropriate water rates for users.

The accuracy of water demand projections depends on the availability of reliable population and water use data as well as an understanding of the distribution of different types of users within the community. In communities with limited data availability, a per capita method is used to calculate future water demand. This method depends on three parameters; the ADD, the current population, and the current rate of population growth (this can be negative). First, the average daily per capita water use is calculated by dividing the ADD by the current population. Next, the anticipated future population is determined by repeatedly solving an equation similar to the compound interest equation using the current population and growth rate. The anticipated population is multiplied by the average daily per capita water demand to determine the anticipated future ADD. The anticipated future MDD can then be calculated by multiplying the anticipated future ADD by the current peaking factor. A sample calculation using the per capita method is provided in Example 2.1.

Example 2.1

A small community has a population of 1,000 and is growing by 1.5% each year. The ADD in the community is 500,000 L/day. The daily peaking factor is 2.75. What will the ADD and MDD be after 20 years?

$$\text{Average Per Capita Water Demand} = \frac{500{,}000\ \text{L/day}}{1{,}000\ \text{people}} = \frac{500\frac{\text{L}}{\text{person}}}{\text{day}}$$

$$\text{Anticipated Future Population} = 1{,}000(1 + 0.015)^{20} = 1{,}346\ \text{people}$$

$$\text{Anticipated Future ADD} = 500\ \text{L/person/day} \times 1{,}346\ \text{people} = 673{,}000\ \text{L/day}$$

$$\text{Anticipated Future MDD} = 2.75 \times 673{,}000\ \text{L/day} = 1{,}850{,}750\ \text{L/day}$$

The per capita method relies on a number of assumptions that may be unrealistic for some communities. First, it assumes that each user is responsible for the same proportion of the total water demand and that they exert this same demand every day. Secondly, the per capita method assumes that the growth rate will remain constant over time. Finally, the method assumes that no new large industrial, institutional, or commercial users are added or removed over time. Despite these short-comings, this method is useful for communities with little access to more detailed information about land use and water demand in different parts of the community.

If the community has access to data showing water use rates and anticipated changes in demand for different types of users it may be possible to develop a disaggregate model of water usage. A disaggregate model takes into account the different patterns of demand observed for different types of water users. For example, if water use records are available for residential users and a fish plant, the anticipated future demand will be calculated using the population growth rate for the former and employment and operating time projections for the latter. This type of model is generally more accurate than the per capita method (AWWA, 2001). Where possible, this method has been used for water use projections in this study. Other, more complex models exist for water use projections. To be accurate, however, they require significantly more data input. These models will not be addressed as part of this study.

2.4.4 Peak Hour Demand (PHD)

The PHD of a community represents the maximum hourly demand experienced on an average day. Equation 2.4 can be used to calculate the PHD if the hourly peaking factor (f_h) and ADD are known.

$$PHD = f_h \times \frac{ADD}{24}$$ **Equation 2.4**

Like the MDD, the PHD of a given area can be determined based on historical water data. This is, however, rarely feasible unless the water users are metered. Thus, engineers and utilities frequently rely on hourly peaking factors. A summary of some common hourly peaking factors is provided in Table 2.5.

Table 2.5 Empirical f_h values (MOE, 2008*)

Population Size	Peaking Factor
150	7.40
300	5.40
500	4.30
500 – 1,000	4.13
1,001 – 2,000	3.75
2,001 – 3,000	3.38
3,001 – 10,000	3.00

*Values for communities with more than 500 residents can be found on page 3-110 of the NL Guidelines

Like the peaking factors used to calculate the MDD, f_h decrease as the population increases. This reflects the fact that as the population increases and, especially, diversifies, residents are less likely to follow similar schedules. This tends to flatten the demand curve, resulting in less obvious peaks over the course of the day.

The predicted peak flow has practical implications for the design and operation of water treatment, disinfection, and distribution systems. For example, treatment systems that are designed based on peaking factors that are too large may add more chemical than required while those designed using inappropriately small peaking factors may have trouble achieving treatment and disinfection compliance objectives during periods of high demand. Oversized distribution systems can contribute to excessive water age, while undersized ones can result in increased maintenance costs and pressure losses.

2.4.5 Peak Hour Peaking Factor Calculation Methods

2.4.5.1 Harmon Formula

The Harmon Formula, shown in Equation 2.5, is currently used by the ENVC to calculate peak flow rates for communities who do not have adequate water demand records to calculate average day, maximum day, and/or peak flow. 'P' stands for population.

$$f_h = \frac{18+\sqrt{P/1000}}{4+\sqrt{P/1000}}$$ **Equation 2.5**

The Harmon Formula was originally developed to explain variations in flow experienced in wastewater collection and treatment systems. It can also be used to calculate peak water flow rates in distribution systems if it is assumed that all water used in the community eventually makes its way back to the wastewater collection system. In practice, this often means that the peak demand calculated using the Harmon Formula does not account for water used for gardening or (most) other outdoor activities.

2.4.5.2 PRP-Gumbel Method

Zhang developed a new approach to calculating peaking factors using the Poisson Rectangular Pulse Model (PRP Model) and extreme value theory. The 'Gumbel' in the method title comes from a statistical distribution used to represent measurements with wide variations. The derivation method is complex but flexible and results in the simple relationship shown in Equation 2.6.

$$f_h = A+\frac{B}{\sqrt{N}}$$ **Equation 2.6**

A and B are coefficients that account for different levels of indoor vs. outdoor water use. Both can be calculated using additional equations provided in Zhang (2005). N represents the number of homes. If this is not known, an average occupancy can be assumed. Zhang assumes that there are 2.7 people per home to derive the following equations (P = population):

Indoor water use only:

$$f_h = 2.5+\frac{2.18}{\sqrt{P/1000}}$$ **Equation 2.7**

Indoor water use (90%) and outdoor water use (10%):

$$f_h = 3.02 + \frac{2.28}{\sqrt{P/_{1000}}}$$ **Equation 2.8**

Indoor water use (66%) and outdoor water use (33%):

$$f_h = 4.17 + \frac{2.46}{\sqrt{P/_{1000}}}$$ **Equation 2.9**

The 'indoor only' version of the PRP-Gumbel method is the only one that will be compared in detail to the other methods discussed in this section as most residential water use in Newfoundland and Labrador is indoor.

2.4.5.3 American Water Works Association (AWWA) Method

Zhang (2005) compared the PRP-Gumbel Method to a number of existing peaking factor calculation methods including that described by the American Water Works Association (AWWA). The equation used to calculate the peak hour peaking factor is shown in Equations 2.10. 'q' represents the average annual per capita demand per 1000 people and is calculated by converting the average daily per capita water usage from L/day to L/min and multiplying it by 1000.

$$f_h = \left(\frac{1095.31}{q}\right) P^{0.4}$$ **Equation 2.10**

The AWWA model was developed specifically for small communities and individual neighbourhoods and is only accurate for populations between 650 and 1,675. It is described in detail in the AWWA Manual M22, *Sizing Service Lines and Meters* (2004).

2.4.5.4 DVGW

The DVGW German Technical and Scientific Association for Gas and Water have also established a method for calculating peaking factors.

$$f_h = 18.1(P^{-0.1682})$$ **Equation 2.11**

English language information about the DVGW method was drawn from an article by Diao et al. (2010).

The six methods for determining peaking factors discussed in sections 2.4.4 and 2.4.5 are summarized in Table 2.6.

Table 2.6 Summary of peaking factor calculation methods

Method	Equation	Population Limits
MOE Guidelines (2008)	Empirical	None
Harmon Formula (1918)	$f_h = \frac{18+\sqrt{P/1000}}{4+\sqrt{P/1000}}$	1,000 ≤ P ≤ 1,000,000
PRP-Gumbel (2005) (indoor use only)	$f_h = 2.5+\frac{2.18}{\sqrt{P/1000}}$	1,000 ≤ P ≤ 25,000
AWWA (2004)	$f_h = \left(\frac{1095.31}{q}\right) P^{0.4}$	650 ≤ P ≤ 1,675
DVGW (2007)	$f_h = 18.1(P^{-0.1682})$	Unknown

*P = population; q = water demand per 1,000 people

The peak residential flows predicted by the six models in Table 2.4 for communities ranging from 0 to 100,000 users are shown in Figure 2.1. All calculations assume a daily per capita water usage of 395 L/min and that all water use is indoors. Note that MOE values were obtained by interpolating between the values provided in Table 2.5.

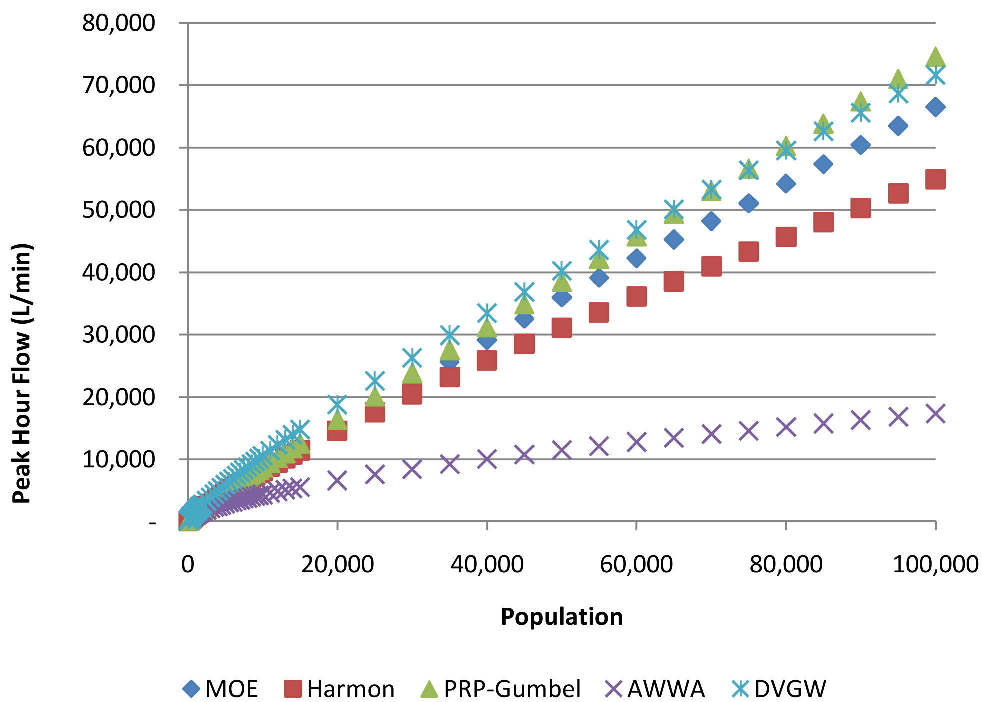


Figure 2.1 Peak flows predicted by different peaking factor models – 100 to 100,000 Users

Figure 2.1 shows that when the population is very small, the peak flows predicted by many of the calculation methods converge. As the population increases, however, they begin to deviate from one another. For smaller populations (approximately < 75,000) the DVGW method predicts the highest flows. The AWWA method predicts the lowest over the entire population range.

Many of the methods appear to align relatively well with the interpolated MOE empirical peaking factors. Figure 2.2 shows the peaking factors predicted by each of the methods plotted against the empirical factors listed in the MOE Guidelines.

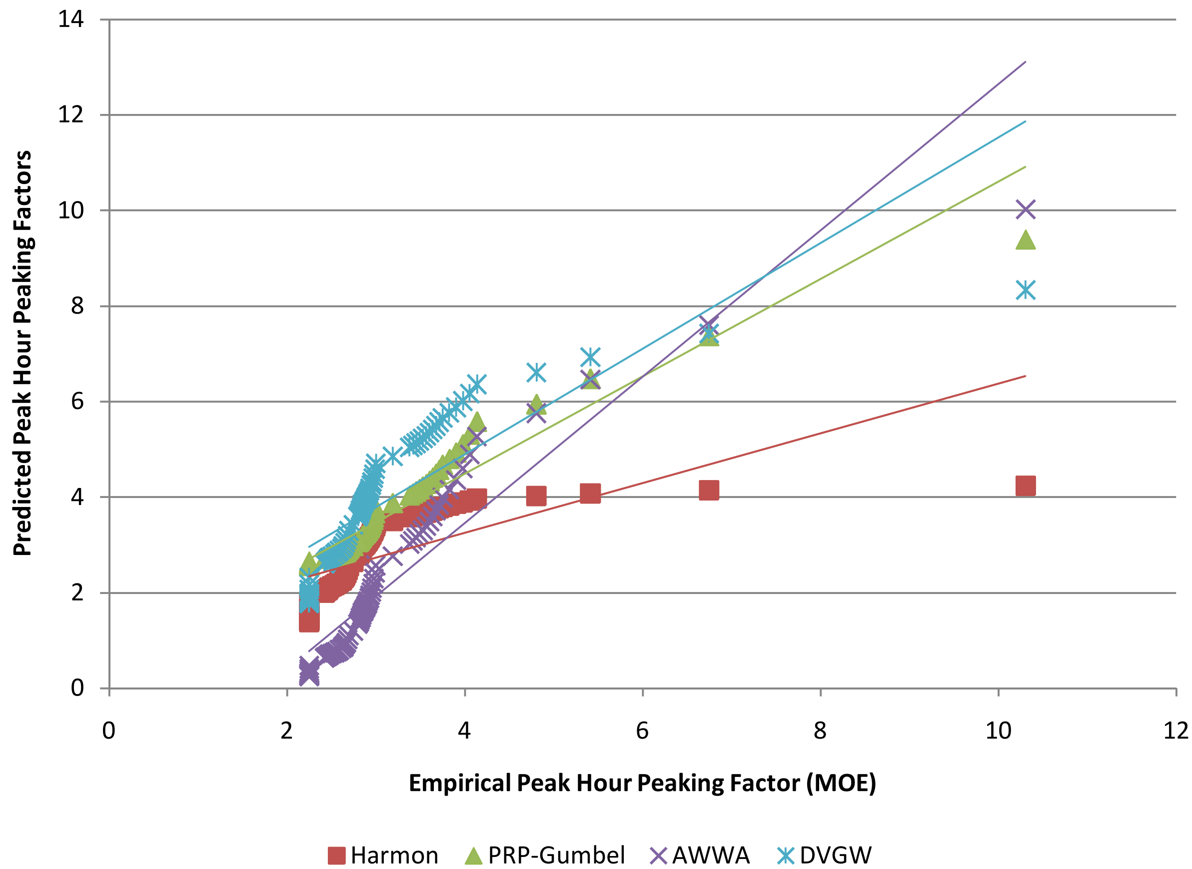


Figure 2.2 **Predicted vs. empirical peak hour peaking factors (f_h)**

The results of linear regressions performed on the datasets in Figure 2.2 are provided in Table 2.7. All relationships were significant at 95% confidence (i.e., $p < 0.05$).

Table 2.7 **Results of linear regressions performed on predicted and empirical peak hour peaking factors**

Method	Equation	r^2
Harmon (1918)	$y = 0.52x + 1.18$	0.47
PRP-Gumble (2005)	$y = 1.02x + 1.18$	0.93
AWWA (2008)	$y = 1.53x - 2.67$	0.88
DVGW (2007)	$y = 1.11x + 0.48$	0.69

Note that though both the PRP-Gumbel and AWWA methods have high r^2 values it is only the former that has a near 1 to 1 relationship with the interpolated MOE values.

Table 2.8 lists the advantages and disadvantages of each method.

Table 2.8 Advantages of Peaking Factor Methods

Method	Advantages	Disadvantages
MOE Guidelines (2008)	• Well established in Canada; • Cited in AC and NL Guidelines; and • No population limits.	• Empirical nature makes it difficult to implement on a large scale (i.e. database); and • No exact solution for populations that fall within ranges.
Harmon Formula (1918)	• Currently used by the ENVC; and • Applicable over a large population range.	• Usually used for wastewater (i.e., only accounts for water sent to sewer)
PRP-Gumbel (2005)	• Corresponds well to established empirical peaking factors; • Alternate versions of the equation (Zhang, 2005) can be used to solve for combined indoor and outdoor use; • Very accurate; and • Equation-based and thus easy to implement on a large scale (i.e., database).	• Limited applicability in very small communities; and • Large communities must be modelled as a series of individual subdivisions.
AWWA (2004)	• Well established method.	• Only appropriate within a specific population range; and • May underestimate peaking factors.
DVGW (2007)	• Corresponds well to established empirical peaking factors; and • No (known) population limits.	• Unknown limits and accuracy; and • May overestimate peaking factors.

2.5 Diurnal Water Demand Curves

2.5.1 Standard Diurnal Water Demand Curves

Water use can vary more over the course of any given day than it does day to day. Most communities will exhibit a repeated pattern of increased water consumption in the morning followed by a trough at midday and a larger peak in the early evening. Recent investigations in Nova Scotia have suggested that water use patterns have shifted somewhat in recent years and the highest water use now tends to occur in the morning rather than in the afternoon.

Figure 2.3 shows a standard municipal diurnal water demand curve.

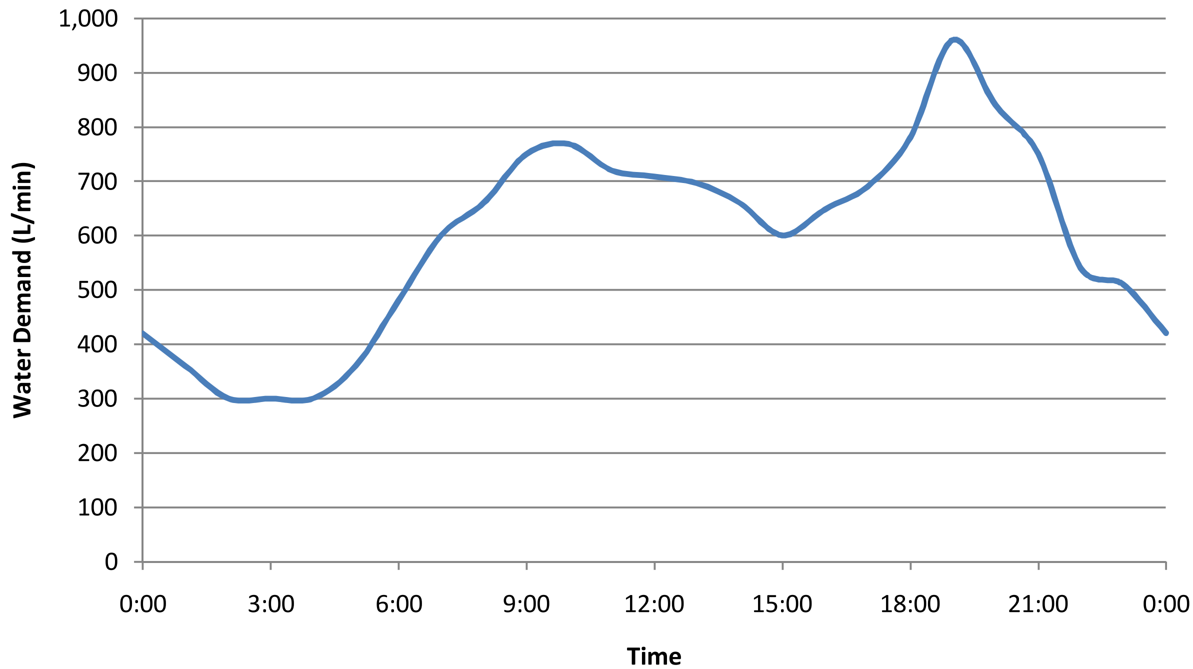


Figure 2.3 Standard diurnal water demand curve (adapted from AWWA, 2008)

The peaks on the water demand curve may be shifted or of different magnitudes in communities with different demographics (ex. urban vs. rural, low-income vs. high-income) or those with significant agricultural or industrial water demands. For example, a study conducted in Austin, Texas, found that diurnal water demand curves in low income residential areas were flatter than those in higher income residential areas. The authors suggested that this might reflect the different work schedules of residents in these two areas (Rhoades, 1995).

Diurnal water use patterns will also shift up and down depending on system losses. The base flow of water lost through leaks can vary seasonally, particularly in northern communities where water runs through the pipes at night to prevent them from bursting.

Note that monitoring the total amount of water entering (rather than leaving) a water storage volume will mask diurnal water use patterns.

2.5.2 Effect of Industrial Users on Diurnal Water Use Patterns

As described in Section 2.3.2, in many communities manufacturing and processing facilities rely on municipal infrastructure to fulfill their water needs. These industrial users can exert large water demands. Oftentimes, these occur on a set schedule. Figure 2.4 shows a standard municipal water demand curve accompanied by an industrial curve. In this example, the industrial user exerts a constant demand that represents 25% of the total municipal demand over the course of a 12 hour shift (6 am to 6 pm).

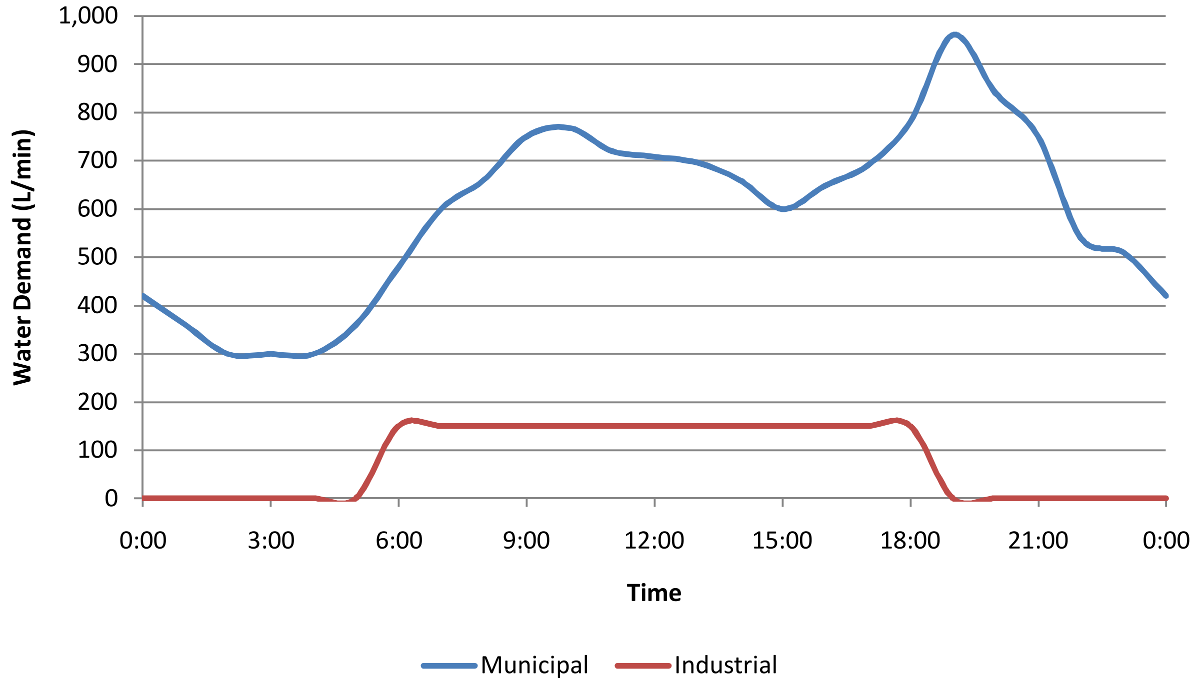


Figure 2.4 Municipal and industrial diurnal water demand curves

If the industrial demand is added to the municipal demand it can both increase the total magnitude and flatten out the peaks when the facility is operating, as shown in Figure 2.5.

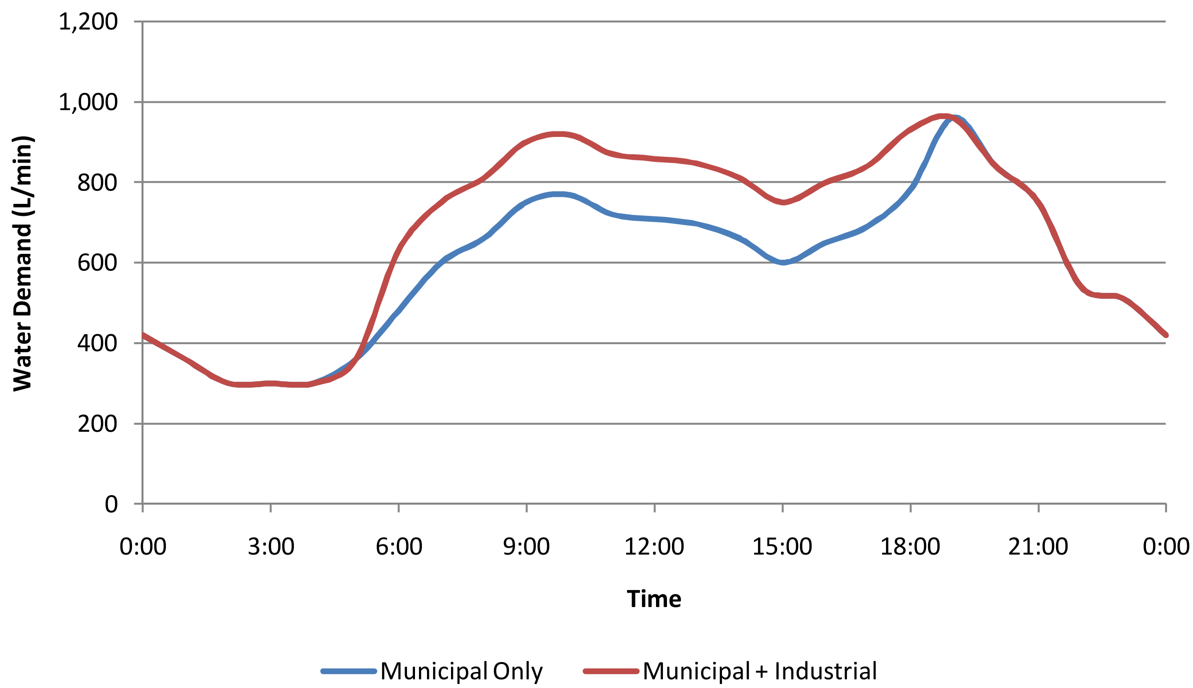


Figure 2.5 Effect of industrial demand on total demand

The size and nature of the facility will impact water demand, as will its operating schedule. Fish plants, which are the most common industrial users in rural Newfoundland and Labrador, usually operate for only part of the year and may operate over a single shift or multiple shifts. This will change the appearance of the industrial diurnal water demand curve.

2.6 Fire Flow

Fire flow is defined as the volume of water a system must be capable of providing, above and beyond the usual demands of the community, for the purpose of fighting fires. Fire flow requirements are usually set by insurance companies based on the type, size, and materials of construction of the buildings that characterize each part of the community. They are based on the total volume of water required to douse a fire of a certain type. The regulations specify the flow rate and time required to provide that volume.

2.6.1 Fire Flow Requirements

In most jurisdictions, communities are required to provide between 1,900 and 13,000 L/min (500 to 3,500 USgpm) of fire flow for a minimum of 2 hours. This amounts to between 228,000 L and 1,600,000 L of additional volume per day. The exact amount required can vary dramatically from community to community based on the number and types of buildings.

Numerous methods exist for the calculation of fire flow requirements. These include the Insurance Services Office (ISO), Iowa State University (ISU), National Fire Academy (NFA), and the Illinois Institute of Technology Research Institute (IITRI) methods. All four are described in detail in the *Manual of Water Supply Practices (M31) - Distribution System Requirements for Fire Protection* (AWWA, 2008).

In Canada, the Fire Underwriters Survey (FUS) provides guidance to communities for the design of municipal fire protection infrastructure in their publication, *Water Supply for Public Fire Protection* (CGI Inc., 2007). This includes a method for calculating required fire flow. A copy of the document has been provided in Appendix F of this report for further review.

The ISO method requires a careful assessment of each building that must be protected in the event of a fire. Industrial, institutional, commercial, multi-dwelling, and large (> 2 storey) residential buildings are classified according to construction, occupancy, exposure, and fire communication. Smaller residential buildings can also be classified based on the distance that separates them from their neighbours. Recommended fire flow ranges from 1,900 L/min (500 USgpm) for houses located more than 30.5 m from one another to 5,300 L/min for those separated by fewer than 3.4 m.

At its simplest, the NFA method relies entirely on the floor space of the affected building and the percentage of it involved in a fire. For example, a 186 m^2 (2,000 ft^2) house that is 100% affected would require a fire flow of 2,520 L/min (670 USgpm). Additional fire flow must be provided for multi-floor buildings and those that are located close to other buildings.

The ISU method calculates the required fire flow based on the volume of the space being doused. Thus, a 186 m^2 (2,000 ft^2) house with a height of 6 m (19.9 ft) would require 1,490 L/min (394 USgpm) of fire flow.

The IITRI method was developed empirically based on historical records from the City of Chicago. The following equations are used for residential and non-residential buildings, respectively:

Residential Building: $Q_{ff} = \left(9 \times 10^{-5}\right) A^2 + \left(50 \times 10^{-2}\right) A$ **Equation 2.12**

Non-residential Building: $Q_{ff} = \left(-1.3 \times 10^{-5}\right) A^2 + \left(42 \times 10^{-2}\right) A$ **Equation 2.13**

Q_{ff} is equal to the required fire flow and A represents the area of the fire, which is usually that of the affected floorspace in a the affected building (length x width). Note that these equations are only valid with US units (ft^2 and USgpm). A house with an area of 186 m^2 (2,000 ft^2) would require approximately 5,000 L/min of fire flow.

The FUS method relies on Equation 2.14.

$Q_{ff} = 220C\sqrt{A}$ **Equation 2.14**

'C' is a constant related to the construction of the building. It can range from 1.5 for wood frame construction to 0.6 for fire-resistive construction. 'A' refers to the total floor-space of the building (i.e., sum of floor-space of each level). The equation predicts that a wood-framed building with 186 m^2 of floor-space would require approximately 4,500 L/min.

The flow determined using Equation 2.14 is then modified based on:

- Number of floors;
- Size of adjoining floors;
- Basement level (% below grade);
- Level of fire hazard posed by building contents;
- Availability of automatic sprinkler system; and
- Exposure to other buildings.

The fire flow required for one or two storey single-family residential buildings can alternatively be taken from a table provided in the FUS document. The fire flow allowance for a wood-framed single-family home ranges from 2,000 L/min for buildings separated by more than 30 m to 4,000 L/min for those separated by between 3 and 10 m. Buildings that are less than 3 m apart are treated as one and must be evaluated using the more complex method summarized previously.

The fire flow requirements predicted for a wood-framed 186 m^2 house by each method are summarized in Table 2.9.

Table 2.9 Summary of results of different fire flow calculation methods

Method	Fire Flow Required
Insurance Services Office (ISO)	1,900 – 5,000 L/min
National Fire Academy (NFA)	2,520 L/min
Iowa State University (ISU)	1,490 L/min
Illinois Institute of Technology Research Institute (IITRI)	5,000 L/min
Fire Underwriters Survey (FUS)	2,000 – 4,500 L/min

A list of historical fire flow requirements used in small communities is provided in the 2008 version of the MOE Guidelines and summarized in Table 2.10. These values represent rough estimates and cannot take the place of fire flow requirements determined using the calculation methods listed previously.

Table 2.10 Fire flow requirements for small communities (Adapted from MOE, 2008)

Population	Suggested Fire Flow (L/min)	Suggested Duration (h)	Suggested Volume (L)
500 to 1,000	2, 280	2	273,600
1,000	3,840	2	460,800
1,500	4,740	2	568,800
2,000	5,700	2	684,000
3,000	6,600	2	792,000
4,000	7,500	2	900,000
5,000	8.640	2	1,036,800
6,000	9,540	3	1,717,200
10,000	11,340	3	2,041,200

The Ontario guidelines suggest that industrial and commercial users be treated as 'equivalent' populations. The fire flow requirements of these equivalent populations should be included in the total fire flow required for the community.

2.6.2 Fire Hydrants

Fire flow is provided through fire hydrants, which are installed at specified intervals throughout serviced areas. According to the AWWA, hydrants should only be installed on pipes with diameters above 150 mm that have been designed to provide fire flow. Spacing will usually range from 100 m to 175 m (ENVC, 2005), but will be specific to each community because of limitations imposed by available firefighting equipment (AWWA, 2008).

The AC Guidelines, which were compiled in 2004, recommend that hydrants be installed at intersections and designated locations based on the recommendations of the Insurance Advisory Organization, which has since been replaced by the FUS (see Section 2.6.1). The FUS recommends that hydrant spacing be based on required fire flow, as summarized in Table 2.11.

Table 2.11 Area per hydrant (adapted from *Water Supply for Public Fire Protection*, CGI Inc, 2007)

Fire Flow L/min	Average Area per Hydrant m^2
2,000	16,000
4,000	15,000
6,000	14,000
8,000	13,000
10,000	12,000

The FUS also recommends that the distance between fire hydrants be less than 90 m in commercial, industrial, institutional, and multi-family residential neighbourhoods. Single-family residential areas are permitted to have 180 m between hydrants.

Hydrants should be tested and maintained regularly by operators to ensure that they are able to provide water at the proper flow and pressure in the event of an emergency. The FUS suggests that inspections occur a minimum of twice a year, preferably in the spring and fall.

2.7 Impacts of Water Demands on Water Treatment, Disinfection, Storage, and Distribution

2.7.1 Treatment

The AC Guidelines recommend that the maximum daily demand expected within the next 20 to 25 years (in addition to fire flow, where applicable) be used as the design flow for water treatment equipment. As discussed in Section 2.4, this can be calculated using the per capita method, a disaggregate water use model, or more complex water use projection models. Sufficient water must also be provided for treatment system cleaning (backwashing, etc.) and to account for known leakage in the distribution system.

The design flow must be determined carefully to avoid under- or over-sizing the treatment system components. A treatment system with undersized components will have elevated energy costs and may experience contaminant carryover, inadequate contact time for chemical processes (pH, coagulation, oxidation, disinfection, etc.), and excessive wear on equipment. Over-sized equipment can result in high capital costs.

2.7.2 Disinfection

The total number of microorganisms (bacteria, protozoa, and viruses) destroyed in a disinfection system is difficult to quantify as it is a function not only of the number of microorganisms present in the raw water but also of the characteristics of the disinfection process. The log reduction and CT concepts were developed to help engineers, operators, and regulators quickly evaluate the effectiveness of a disinfection system.

The log reduction concept is discussed in detail in *Study on Pathogen Inactivation in Drinking Water Systems in Newfoundland and Labrador* (submitted by CBCL Limited to ENVC in August 2011) and will not be discussed in detail in this report.

The *Standards for Bacteriological Water Quality in Newfoundland and Labrador* require that communities achieve a chlorine contact time of 20 minutes with a free chlorine residual of 0.3 mg/L or a CT of 6. This represents a 2-log reduction of viruses at worst-case conditions (T = 0.5^{o}C, pH = 8) using free chlorine.

CT for systems using chemical disinfectants is calculated based on the chlorine residual at the end of the contact volume, the contact time allowed between the water and the disinfectant within the contact volume, and a baffling factor to take into account the amount of mixing in the contact volume, as shown in Equation 2.13.

$$CT_{achieved} = C_{HOCl} \times T \times BF \qquad \textbf{Equation 2.13}$$

Where:
C_{HOCl} = Free chlorine residual at the outlet (mg/L)
T = Time (minutes)
BF = Baffling factor

The contact time is calculated by dividing the total contact volume (L) by the design flow rate (L/min). As the design flow rate increases, the contact time decreases. Note that, depending on the disinfection strategy used, the design flow rate may refer to the maximum flow rate through a chlorine reaction tank, the maximum flow between the point of disinfection and a storage volume used to buffer variations in water demand, or the peak hourly demand in communities that rely on the central transmission main between the point of chlorination and the first user for disinfection.

If the disinfection system is designed using an inflated flow rate, the actual amount of contact time between the chemical disinfectant and the water will be higher than anticipated. This might result in the formation of DBPs. On the other hand, if the assumed flow is less than that which actually occurs in the system, there will be insufficient contact time between the water and the disinfectant, which may reduce the ability of the disinfection process to inactivate pathogens.

2.7.3 Distribution

Distribution systems are designed to provide enough water to satisfy the peak hour demands of all of the users served by the system. All distribution systems will include the following:

- Transmission mains;
- Distribution mains; and
- Service lines (laterals).

The AC and NL guidelines suggest minimum sizes for different types of pipes throughout the system. These are summarized in Table 2.12.

Table 2.12 Recommended pipe sizes within the distribution system (Adapted from ACWWA, 2004)

Type	Minimum Nominal Diameter
Primary distribution mains	200 mm
Distribution mains	150 mm
Service lines providing fire protection	150 mm
Service lines not providing fire protection	100 mm

These recommendations are also provided in the NL Guidelines.

Both the AC and NL guidelines recommend that systems be designed to ensure a maximum velocity of 1.5 m/s under peak demand conditions and 3.0 m/s under fire flow conditions. A minimum residual pressure of 275 kPa (40 psi) should be maintained under normal flow conditions. Thus, the size of the various water mains in the distribution system is directly related to the water demands exerted by community as well as the required fire flow. If the peak demand or required fire flow are overestimated, the water mains will be oversized, which can lead to increased water age and water quality concerns. If they are underestimated, the water mains will be undersized, resulting in excessive pressure loss within the system, particularly during periods of high demand.

2.7.3.1 Special Note – Calculating Pressure Losses in Pipes

In Canada, pressure losses in pipes are usually calculated using one of two formulae: the Darcy-Weisbach equation (1845) or the Hazen-Williams equation (1920). Due to the nature of the equations, there is some variation between resulting head losses.

Darcy-Weisbach Equation

The Darcy-Weisbach equation is a general formula for all pipe flow applications. Although it was originally developed based on empirical observations, it can be derived from Chezy's equation (1769) which balances the drag and motivating forces on moving water. The Darcy-Weisbach equation for frictional head loss is expressed in terms of a dimensionless frictional factor (f), the length of pipe (L), the internal diameter of the pipe (d) and the mean velocity of flow in the pipe (V). For a given flow velocity, pipe size and pipe length variation in material and material roughness determines the frictional factor and, consequently, the pipe's head losses. The equation is expressed below, wherein head loss is denoted as 'h_f' and gravitational acceleration as 'g'.

$$h_f = f\left(\frac{L}{d}\right)\left(\frac{V^2}{2g}\right)$$ **Equation 2.14**

The Darcy-Weisbach frictional factor can be determined using the Colebrook-White equation (1939), the Jain equation (1976), or graphically using the Moody diagram (1944). In all cases, the frictional factor is dependent on a pipe's relative roughness coefficient (ε/d) and the state of flow as classified by the Reynolds number (Re). The Reynolds number is a function of mean pipe flow velocity, internal diameter, and the temperature-dependent kinematic viscosity of the fluid.

Hazen-Williams Equation

The Hazen-Williams equation is another formula commonly used to determine head loss in pipes. It is used almost exclusively in water supply engineering. The Hazen-Williams equation is expressed in metric units in Equation 2.15.

$$V = 0.849CR^{0.63}S^{0.54} \qquad \text{Equation 2.15}$$

The equation is expressed in terms of mean velocity of flow (V), the Hazen-Williams coefficient of roughness (C), the hydraulic radius of the pipe (R) and the slope of the energy gradient (S or h_f/L). The Hazen-Williams roughness coefficient is dependent only upon pipe material. The coefficient makes no adjustments for pipe diameter, velocity or viscosity, all of which impact frictional losses.

The equation's accuracy is also limited to a certain range of diameters and frictional slopes, as the multiplying factor ('1.318' imperial, or '0.849' metric) is held constant. In spite of the equation's limitations, the Hazen-Williams formula has a wide engineering application and is applied indiscriminantly to pipe design. This is primarily due to its' simplicity.

Hydraulic Calculation Recommendations

It is recommended by this report that frictional head losses for pipe system design and existing system analysis be completed using the Darcy-Weishbach equation. In light of common engineering practice, however, all head loss calculations within this report have been completed using both Darcy-Weisbach and Hazen-Williams.

2.7.4 Storage

Storage facilities are included in some distribution systems to improve the operation of the system. For example, the presence of a water storage facility allows the water utility to operate treatment equipment on a set schedule, which can lead to energy savings and allow the operator to work regular hours. Storage facilities also act as buffers against the effects of changing daily and hourly demands on the water system through demand equalization. Elevated storage facilities can help to maintain pressure within the distribution system (MOE, 2008). Finally, the provision of water storage in key locations in the community can reduce the size of mains and service lines required throughout the distribution system. For example, if storage is provided close to an industrial user, the distribution lines in the rest of the community do not have to be sized to accommodate its water demands.

Water storage facilities are sized to include balancing, fire, and emergency storage. The AC Guidelines recommend that the following allowances be made:

- Fire storage equal to the required fire flow (see Section 2.4);
- Balancing storage is equal to 25% of the MDD; and
- Emergency storage is equal to 25% of the sum of fire storage and balancing storage OR 15% of the projected average day design flow.

Thus:

Total Storage Volume = Fire Storage + Balancing Storage + Emergency Storage **Equation 2.15**

These guidelines were developed based on similar recommendations provided in the 1985 version of the MOE Guidelines. They have been directly copied into the NL Guidelines.

Some water storage facilities include additional volume that is mainly used to maintain adequate pressure in the distribution system. This is commonly referred to as 'dead storage'. Depending on the size of the storage facility, dead storage can contribute to excessive retention time and water age in the distribution system. The AC Guidelines and the NL Guidelines recommend that, where possible, water storage facilities should not include dead storage. Where the inclusion of dead storage cannot be avoided mixing should be provided to maintain water quality.

In cases where storage is used to maintain adequate pressure in the distribution system, its ability to do so will decrease during periods of high demand as the water level decreases. The FUS only counts the amount of water that can be provided at a residual pressure of 150 kPa (22 psi) as available fire flow. That is, once the level in the storage volume has fallen below that required to maintain the proper residual pressure in the system the remaining water is not considered to be available for fire protection. Storage systems that rely on pumps to maintain pressure are not limited in this way.

2.8 Special Design Considerations for Communities in Newfoundland and Labrador

2.8.1 Alternative Strategies for Managing Variable Industrial Demands and Fire Flows

Industrial, institutional, and commercial users can impact the diurnal water demand curve, particularly in smaller communities. For example, a fish plant that draws a constant flow of water from the system may operate for only a set number of hours per day. If these hours coincide with periods of high demand from residential areas, the peak hour flow will be higher than predicted using industry values for peaking factors and per capita water use rates.

Providing adequate fire flow can be difficult for small communities because the required volume (i.e., required flow x duration) can be equal to or greater than the ADD. This can result in the design and construction of oversized water treatment, storage, and distribution components that in turn lead to excessive retention time and other operational difficulties. These difficulties are exacerbated in communities with industrial users.

Some of the problems associated with providing for variable industrial demands and fire flow in small communities can be avoided by modifying or expanding the distribution system. This might include the design and installation of general or dedicated water storage or the provision of a secondary distribution system. The community may also require that owners provide automatic sprinkler systems for some or all buildings to aid with fire protection.

2.8.1.1 Storage

Communities can minimize the impacts of variable water demands (fire flow or seasonal industrial demands) by providing adequate storage at the beginning of the distribution system. Alternatively or in addition to this, dedicated storage can be provided to supply the needs of the industrial user, thus minimizing its impacts on day to day and diurnal variations in water demand. If storage is provided at the beginning of the system it should be sized as described in Section 2.7.4 and care should be taken to provide adequate mixing and minimize dead storage.

2.8.1.2 Secondary Water Systems

In some communities, secondary (or dual) water supplies and distribution systems are used to provide water for fire protection and/or industrial users. The secondary supply may be a surface water source, reclaimed water, or salt water.

The cost of developing a secondary water system is often similar to that for a potable water system. Costs include those associated with:

- Water supply development;
- Intake design and installation;
- Treatment (if required);
- Storage; and
- Design and installation of distribution system infrastructure.

Secondary water systems need not be entirely separate from their potable water systems. For example, a secondary main can be laid next to a potable water main but only used during periods of high flow. The two mains can be independent or interconnected. If the two distribution systems are interconnected and one regularly carries untreated water, care must be taken to ensure that this does not contaminate the primary potable water distribution system.

The main advantage of secondary distribution systems is that they allow designers to reduce pipe sizes in the primary potable water system. This minimizes water retention time (i.e., water age). Smaller pipes are also less expensive than larger ones, however, any cost savings accrued through pipe size reductions in the primary potable water system are likely to be less than those associated with constructing a secondary water supply and/or distribution system.

2.8.1.3 Automatic Sprinkler Systems

Automatic sprinkler systems minimize fire flow requirements by providing water to douse the fire within the building itself. Buildings with sprinkler systems still require fire protection through fire flow allowances, however, they require much less than buildings without sprinklers. Sprinklers can be set-up to use non-potable water. This minimizes the amount of water that must be made available through the potable water distribution system. Detailed information about the design of automatic sprinkler systems is provided in *Distribution System Requirements for Fire Protection – Manual of Water Supply Practices – M31* (AWWA, 2008).

2.8.2 Declining Populations

Population decline has been a reality in parts of Newfoundland and Labrador for many years. Between 2001 and 2006 the province’s population declined by 1.5% while that of Canada grew by 5.0% (Stats Canada, 2006). Some communities, most notably those surrounding St. John’s and Corner Brook, have seen increases in population, but these are exceptions. Between 2001 and 2006 the populations of many small communities in the declined by between 5% and 15% (Stats Canada, 2006).

It is generally not advised that designers develop population projections using negative growth rates. Not only will this result in system components that are too small for the current needs of the community, but it may limit the future growth of the community and/or complicate the provision of services if there is a rebound in population. There are, however, ways to adjust the design of treatment, storage, and distribution system components to account for an anticipated drop in overall water demand.

For example, water treatment systems can be designed to treat the current maximum day flow with allowances made for the future increases in capacity. The water treatment building can be constructed with sufficient space to accommodate future equipment installation. As well, redundant filters (or other system components) can be installed that operate for a set amount of time each a day at current flows but which can be run for longer periods of time should water demands increase.

With regards to distribution, it may be necessary to limit system extensions in some communities with declining populations. Instead, small groups of homes may be better served by small local water supplies and treatment systems (ex. wells).

2.8.3 Water Shortages and Conservation

A number of communities in Newfoundland and Labrador have experienced water shortages in recent years. As a result, many communities are beginning to look into their options for water conservation. The ENVC has identified a number of causes including:

- Increasing water demands in growing communities;
- High industrial (i.e., fish plant) water demands;
- Dry summers;
- Insufficient water available from the water supply (groundwater and spring-fed supplies);
- Undersized reservoirs and/or dams; and
- Leakage in the distribution system.

Though the populations of most communities in the province are declining some, particularly on the Avalon Peninsula, are growing. This has resulted in increasing water demands and, in some cases, water shortages. To fulfill the water needs of these growing populations it will be necessary to develop new water supplies, treatment plants, and distribution systems.

The total water demand can also be decreased by metering water users and charging them based on usage (rather than a flat rate) as well as by encouraging the use of low flow fixtures. Other municipalities have encouraged the use of low flow fixtures through rebates, renovations, and public outreach programs. Similar water conservation tactics can be used to prevent water shortages related to the other factors identified by the ENVC. Additional conservation methods might also be appropriate in certain communities.

Weather patterns and climate change can impact the amount of water used as well as the amount of water available. Hot, dry weather is uncommon in most areas of Newfoundland and Labrador but communities can minimize its effects on water supplies and water use by instituting source water protection to ensure that water levels are kept as high as possible and limiting outdoor water use by encouraging the planting of drought resistant species.

Distribution systems in Canada, particularly the older ones, lose an average of 12.8% of the total available volume of treated water through leakage (EC, 2010). Leak detection and repair is the most effective way to minimize the total amount of water that must be withdrawn from the water supply. It also increases the volume of treated water available to users. Operators in Newfoundland and Labrador are trained on leak detection through the Operator Education, Training, and Certification (OETC) program.

One factor that has not yet been implicated in any reported water shortages in the province is the common practice of running household taps through the winter to prevent freezing. This can result in dramatic increases in total water use in a community and should be discouraged (where feasible). To minimize problems related to frozen pipes residents and other water users could be encouraged to insulate their pipes, possibly through a government awareness or rebate program.

CHAPTER 3 WATER QUALITY

3.1 Water Age

3.1.1 What is water age?

The term 'water age' refers to the amount of time that a unit of treated water spends within a distribution system. It is analogous to the retention time. Water age is used as a proxy for water quality because many potentially harmful chemical and biological processes, such as chlorine decay, corrosion, disinfection by-product formation, and pathogen re-growth, are time dependent. The risks posed by these parameters increase as water ages within the distribution system.

Where feasible, water storage and distribution systems are designed to minimize water age. For example, water age can be exacerbated by over-sized storage and distribution system components. The velocity of the water travelling through the mains will decrease when actual demands fail to match up with those projected using historical data or peaking factors. The result is a longer detention time for each unit of water, which can lead to deterioration in water quality.

Improperly mixed storage tanks and dead ends within the distribution system can also act as nodes for the development of excessive water age. The age of the water within storage tanks can be minimized by designing and/or operating the system to avoid stagnation within the storage volume. This may include: strategic placement of inlet and outlet piping; inclusion of baffles or diffusers; and/or a multi-cell design (MOE, 2008). Excessive water age within the distribution mains can be avoided by minimizing the number of dead-ends in the system. This can be accomplished by looping sections of the distribution system. Water age can also be minimized by flushing the distribution system regularly to remove old or stagnant water.

3.1.2 Calculating Water Age

Water age is difficult to calculate because of the inherent complexity of water distribution systems. Instead, it is often inferred based on the size of major system components and the known flow. Water age is not always an accurate proxy for water quality. Most chemical and biological processes are dependent on reaction time but are also functions of pre-existing water quality and the state of the distribution system components. For example, an uncommonly low free chlorine residual may be indicative of excessive water age or may be related to a higher-than-normal concentrations of chlorine consuming species in the bulk water or the walls of the distribution mains.

Hydraulic models and/or tracer studies can be used to develop more accurate estimates of water age. Studies conducted on water systems in Canada and the United States have found that water age varies between one and three days within most distribution systems, although in some communities, the age of water at dead end locations was estimated to be between 12 and 25 days (US EPA, 2002).

3.2 Effects on Water Quality

3.2.1 Disinfection and Disinfectant Decay

Advances in disinfection technology have improved human health throughout the developed world. In modern water treatment systems pathogens are removed (filtration, etc.), inactivated (ultraviolet, or UV, radiation), and/or killed (chemical disinfectants). Usually, a chemical disinfectant is also added to the finished water as it travels into the distribution system to prevent bacterial regrowth and re-infection.

Primary disinfection refers to disinfection that takes place within the treatment plant. The goal of primary disinfection is to remove, kill, or inactivate a set number of pathogens. For example, the Guidelines for Canadian Drinking Water Quality (GCDWQ) suggest that drinking water treatment systems be designed to reduce the total number of *Cryptosporidium* and *Giardia* cysts by 99.9% and viruses by 99.99% (Health Canada, 2008). Primary disinfection can be achieved through engineered filtration, chemical disinfection (chlorine, ozone, etc.), or UV radiation.

The goal of secondary disinfection is to establish a disinfectant residual within the distribution system. This prevents re-infection of the water and the development of biofilms on the inside surfaces of the distribution mains. Free chlorine and monochloramine (often measured as combined chlorine) are the two most common secondary disinfectants. The former is ubiquitous in Newfoundland and Labrador.

Most jurisdictions have established minimum secondary disinfectant residual levels that must be maintained throughout the system. The current design guidelines in Newfoundland and Labrador require that a detectable chlorine residual be maintained throughout the distribution system (ENVC, 2005). In Nova Scotia, a minimum residual of 0.2 mg/L of free chlorine must be present in the distribution system (NSE, 2002). The new draft guidelines for that province suggest a minimum combined chlorine residual of 1.0 mg/L when monochloramine is being used for secondary disinfection (NSE, 2010 – Draft). The Ten States Standards for Water Works also recommends a minimum residual of 0.2 mg/L for free chlorine and 1.0 mg/L for combined chlorine, depending on the disinfectant employed (SPPHEM, 2007).

The decay of free chlorine is usually modeled as a first-order or pseudo-first-order reaction dependent on initial chlorine dosage and total time elapsed as shown in Equation 3.1.

$$C_t = C_O e^{-kt} \qquad \textbf{Equation 3.1}$$

In practice, however, the decay of chlorine within the distribution system is affected by the concentration of various chlorine-consuming species in the bulk water and on the inner surface of the pipe. The first order model is only able to describe the behaviour of chlorine in the bulk water when chlorine is the limiting reagent. Although the first order model is often adequate for modelling purposes (Powell et al., 2000a), it is unable to explain phenomena observed in the minutes immediately following chemical application and has limited applicability to systems where the bulk water does not exert a strong chlorine demand. This has led researchers (Warton et al., 2006; Jonkergouw et al., 2009) to develop more complex reaction models to describe the behaviour of chlorine in the distribution system. The details of these models are beyond the scope of this study but can be found in the aforementioned references.

3.2.2 Disinfection By-products

Disinfection by-products (DBPs) such as trihalomethanes (THMs) and haloacetic acids (HAAs) are compounds formed through the reaction of chlorine with naturally occurring organic matter (NOM) present in the bulk water. DBPs can also be formed when NOM comes into contact with monochloramine, however, the rate of this reaction is slower. Factors that can affect the rate of DBP formation include the applied chlorine dose, temperature, pH, contact time, and the concentration of NOM.

The term 'NOM' does not refer to a specific chemical species but rather to an array of heterogeneous molecules that share a common source. These include proteins, hydrophobic humic and fulvic acids, hydrophilic acids, and lignins. Although these molecules tend to share many physical and chemical characteristics, they differ in reactivity. The heterogeneous nature of NOM makes it difficult to predict the formation of DBPs using common water quality parameters. NOM is usually measured as total organic carbon (TOC), dissolved organic carbon (DOC), or using UV absorbance (e.g. UV254). The first two parameters do not differentiate between reactive and non-reactive NOM molecules. UV absorbance at specific wavelengths, in particular 254 nm, has been found to be correlated to the concentration of THM precursors (Edzwald et al., 1985), however, the relationship is not exact. Other methods, including HPSEC and resin fractionation, have been used to isolate different NOM fractions based on their size and reactivity. These methods are complex and onerous, making them of limited use, especially as different surface water sources have distinct NOM distributions and often vary seasonally.

Smaller communities served by surface water sources and lacking water treatment processes optimized to remove NOM are at risk of developing high levels of DBPs, particularly in dead ends, storage tanks, and other locations with increased water age. In Newfoundland and Labrador, these communities often have elevated concentrations of NOM in their raw and treated water. When the water is disinfected the NOM exerts a strong chlorine demand, which in turn results in the need to apply large amounts of chlorine to ensure disinfection and achieve required residual levels. It also results in the formation of DBPs. As the water ages in the distribution system, the chlorine and NOM react to form even more DBPs, resulting in high THM and HAA readings throughout the community.

Some THMs and HAAs have been linked to cancers and reproductive issues in animals and are therefore considered possible carcinogens in humans (Health Canada, 2006, Health Canada, 2008). Consequently, the US EPA and many Canadian provinces have set legal limits on the levels of THMs and HAAs permitted in drinking water. The GCDWQ recommends a limit of 100 µg/L for THMs and 80 µg/L. Many additional DBPs have been identified in treated drinking water, some of which are believed to pose a greater risk to human health than those that are currently regulated (Karanfil et al., 2008).

The province of Newfoundland and Labrador has not formally adopted enforceable legal limits on any DBPs, however, information and mitigation strategies are available on the government website and in government-published reports. The ENVC samples for THMs and HAAs in communities across the province quarterly and publishes the results on their website.

3.2.3 Corrosion

Corrosion is defined as "the deterioration of a material, usually a metal, which results from a reaction with its environment" (NACE International, 2000 – cited in Health Canada, 2009). Corrosion is a concern for utilities because it can compromise the integrity of distribution system components, reduce the effectiveness of secondary disinfection, and lead to the formation of corrosion products. The latter can include lead, copper, and iron species. For example, tuberculation of iron pipes (cast iron, ductile iron, steel, etc.) is a well-known phenomenon that can eventually lead to flow restrictions, aesthetic water quality concerns, and bacterial regrowth.

Electrochemical corrosion occurs when a metal is oxidized to produce metal ions and electrons as shown in the following reaction where 'M' represents the metal species:

$$M \underset{t}{\rightarrow} M^{n+} + ne^{-}$$ **Equation 3.2**

(Health Canada, 2009)

Electrochemical corrosion is not the only mechanism that can result in the deterioration of distribution system components and the release of corrosion products. For example, concrete and cement-lined pipes can also degrade over time, releasing calcium hydroxide and occasionally, asbestos and aluminium and even PVC pipes are not immune from corrosion (Health Canada, 2009).

The following parameters are known to influence corrosion and the transport of corrosion products in distribution systems:

- Distribution system component material;
- Age of the system;
- Disinfectant type;
- Water quality; and
- Water age.

As discussed previously, components of the distribution system that are constructed of metal, concrete, or PVC are all subject to some form of corrosion. As the system components age, corrosion rates can change, increasing or decreasing depending on the type of pipe, the degree of bacterial re-growth, and the water quality. It has been shown that copper release decreases with increasing pipe age (Lytle and Schock, 2000). Iron release increases as the system ages, often leading to 'red water' complaints from residents.

The bulk water quality and the choice of secondary disinfectant can also influence corrosion rates. Low pH, low alkalinity, high alkalinity, high conductivity, and the presence of natural organic matter have all been shown to impact corrosion rates in distribution systems (Lee et al., 1989; Edwards and Dodrill, 1995). Some corrosion products are soluble and will be carried in the bulk water to users' taps, while others will react with other species in the water or with the surface of the pipe to form scales. These scales can eventually be sloughed off during flushing events or dissolved as a result of changes in pH, disinfectant type, or concentration.

The switch to chloramines is known to increase lead concentrations in drinking water (Edwards and Dudi, 2004). This may be related to the dissolution of deposits previously formed in the distribution systems and/or service lines through the interactions of free chlorine and pipes and fixtures that contain lead (Xie et al., 2010). Chloramination may also impact lead dissolution by increasing nitrification, and consequently lowering pH (Health Canada, 2009).

A complex relationship exists between water age, the rate of corrosion, and the formation and transport of corrosion products. A number of researchers, as summarized by Schock (1996), have shown that lead levels increase exponentially in the bulk water over time until a certain point when the rate becomes constant. Schock hypothesized that this plateau represented the amount of time required for the lead dissolution reaction to reach equilibrium.

Some metals have been shown to react differently to long stagnation periods. For example, Sorg et al. (1999) showed an increase in lead dissolution with increased stagnation time but also observed that copper concentrations in the bulk water initially increased but subsequently decreased over time. The results of these studies, however, are only valid for a given range of water quality, specific pipe size, material, and age. Under different conditions, different phenomena may be observed. For example, in the aforementioned study, the stagnation profiles observed over time for raw water were different than those observed for water softened in an ion-exchange unit (Sorg et al., 1999).

Water age can also impact corrosion rates by reducing the effectiveness of corrosion control strategies. The chemicals used to stabilize pH (lime, soda ash, etc.) or prevent corrosion (various phosphate-based corrosion inhibitors) decay as water ages in the distribution system, leaving pipes and other components vulnerable to corrosion reactions (US EPA, 2002).

Corrosion is of particular concern for small communities that alternate between periods of high and low water use. During periods of high water use, the velocity of the water within the distribution system increases. This can result in a higher rate of corrosion (Hanson et al., 1987). When less water is used by the community, the velocity decreases and stagnation can result. Stagnation provides greater contact time between the bulk water and the pipe walls, which can lead to increased solubilisation of corrosion products (Lytle and Schock, 2000). When the system returns to high flow conditions, these corrosion products are transported to users' taps. Regular flushing during periods of low flow can help to mitigate this.

3.2.4 Bacterial Regrowth

The availability of biodegradable organic matter, inorganic carbon, and nutrients (nitrogen, phosphorous) contribute to the extent of bacterial regrowth within the distribution system, as does the pipe material used (Clement, 2004). Bacterial regrowth is also closely tied to water age. Areas of stagnant water, such as dead ends and poorly mixed storage facilities, can contribute to increased bacterial regrowth. The low water velocities found in these locations minimizes the sloughing off of bacterial biofilms, which can encourage the formation of thick, resilient biofilms that will later resist sloughing even under higher flow conditions.

Bacteria in the distribution system can be suspended in the bulk water (planktonic) or attached to pipe walls (sessile). Although the former are more likely to be transported to users' taps, the latter are more likely to contribute to the long-term deterioration of water quality. As the attached bacteria grow, they exude an extracellular biofilm that acts as a protective barrier between themselves and the bulk water. Attached bacteria have been implicated in taste and odour complaints, increased corrosion, depletion of disinfectant residuals, and increased concentrations of planktonic bacteria in the bulk water.

Bacteria protected by biofilms are more difficult to inactivate than those in the bulk water. LeChevallier et al. (1988) showed that bacteria protected by biofilms were 150 to 3,000 times more resistant to free chlorine and 2 to 100 times more resistant to chloramines than were free-floating planktonic bacteria. They suggested that this was because it was difficult for the disinfectants to penetrate the biofilms. Biofilms can also exert a strong disinfectant demand, which can make it difficult to maintain an adequate residual within the water system.

Pipe corrosion often precedes the growth of biofilms. Corrosion can give rise to pitting or tuberculation of the pipe surface. The resulting irregularities provide a safe harbour for bacteria, including some that contribute to the deterioration of water quality over time.

Heterotrophic bacteria, or bacteria that feed on organic carbon, are endemic in the environment, the bulk water, and in biofilms in water distribution systems. Most of the species that occur in the distribution system pose minimal risk to human health, however, their presence in water samples can indicate that conditions amenable to the growth of more dangerous microorganisms exist.

3.2.5 Nitrification

Nitrification occurs when autotrophic 'nitrifying' bacteria in the distribution system convert ammonia to nitrite and nitrate. Nitrifying bacteria are those that are able to gain energy through the oxidation of reduced inorganic nitrogen species instead of organic (bound) nitrogen. Two subgroups of nitrifying bacteria exist: ammonia-oxidizing bacteria, which convert ammonia to nitrite (Equation 3.3); and nitrite-oxidizing bacteria, which convert nitrite to nitrate (Equation 3.4).

Ammonia to Nitrite (Nitrification)

$$NH_4^+ + \frac{3}{2}O_2 \rightarrow NO_2^- + H_2O + 2H^+ \qquad \textbf{Equation 3.3}$$

Nitrite to Nitrate (Nitrafication)

$$NO_2^- + \frac{1}{2}O_2 \rightarrow NO_3^- \qquad \textbf{Equation 3.4}$$

(Vaccari et. al. 2006)

Common ammonia-oxidizing bacteria include *Nitrosomonas*, *Nitrosoccocus*, and *Nitrisospira.* Nitrite-oxidizing bacteria include *Nitrobacter, Nitrospina, Nitrococcus,* and *Nitrospira*.

Ammonia found in the distribution system can be naturally occurring in the raw water, added to the water to form chloramines for secondary disinfection, or may be formed through the interaction of nitrates with pipe materials (Zhang *et al.*, 2009). Nitrification can result in taste and odour issues, increased corrosion, and the consumption of alkalinity.

For example, nitrifying bacteria tend to form biofilms on the surfaces of distribution system components. As the biofilm becomes thicker and more stable it is more difficult for oxidants, including oxygen, to penetrate. This can result in the formation of an anaerobic layer at the surface of the pipe that provides an ideal environment for anaerobic sulphate-reducing bacteria. When these bacteria come into contact with sulphates and the iron present in some distribution system components (cast iron or steel pipes, etc.), the iron is oxidized to ferric oxide and the sulphur is reduced to form iron sulfide (Vaccari *et al.*, 2006). This results in additional decay of the pipe wall.

Ammonia and nitrite can affect disinfection effectiveness by reducing the concentration of free chlorine. Ammonia reacts with chlorine to form chloramines (Vaccari *et al.*, 2006). Though used in some communities for secondary disinfection, chloramines (monochloramine, dichloramine, and trichloramine) are less effective disinfectants than free chlorine, particularly when their speciation is not properly controlled. Nitrite also reacts with chlorine, resulting in the formation of nitrate and chloride, neither of which has any disinfecting power (Vaccari *et al.*, 2006). In either case, the distribution system is left vulnerable to bacterial regrowth.

Zhang and Edwards (2010) recently published a comprehensive study that identified conditions that can stimulate or inhibit nitrification in distribution systems. Phosphorous and inorganic carbon (as $CaCO_3$) were found to be essential to the activity of nitrifying bacteria while high pH, zinc, and copper were found to inhibit nitrification. Numerous interactions between these parameters were also observed. Ammonia concentrations tend to increase as water ages, particularly in systems that employ chloramines for secondary disinfection. This increases the likelihood of nitrification in the distribution system, particularly in dead ends and regions of low flow.

CHAPTER 4 PROJECT METHODOLOGY

4.1 Desktop Study

4.1.1 Preliminary Research

At the beginning of the study numerous academic and governmental publications were used to complete a preliminary literature search. The findings were used to establish the goals and information collection requirements of the study. Historical water quality records were obtained from the ENVC for all of the participating communities and as-built drawings and system maps were obtained from the Department of Municipal Affairs (DMA) where available.

4.1.2 Development of Information Collection Sheets

In order to streamline the collection of data from municipal and industrial operators during the site visits, information collection sheets (surveys) were developed ahead of the field portion of the study. The surveys were developed using an online software program, SurveyMonkey, which allows technicians to input data from the field. Hard copies of the information collection sheets were sent to participating communities ahead of the technician to allow the operator(s) to prepare for the site visit ahead of time.

The surveys included a questionnaire for the technician to fill in with the help of the municipal and/or industrial operator. Most of the questions included a checklist of potential answers along with a space to record additional information. Some questions, particularly those requesting design parameters or operating data, were left completely open. Details on the following were requested from the municipal and/or industrial operators:

Municipal

- Operator identification, education, and experience;
- Municipal rate structure;
- Water supply;
- Water treatment (where applicable);
- Disinfection equipment;
- Storage facilities (where applicable);
- Size, extent, and materials of the distribution system components;
- Availability of water use/flow records;
- Operation and maintenance schedules;

- Fire flow requirements, hydrants, and flushing procedures; and
- Wastewater disposal.

Industrial (Fish Plant)

- Water use patterns;
- Record-keeping;
- Infrastructure components (storage, water main, additional treatment, etc.);
- Plant production schedules;
- Water quality concerns;
- Industrial rate structure; and
- Wastewater disposal.

The information collection sheet also included a list of system components to be photographed and examples of flow records to be obtained from the operator(s) where possible. A copy of the information collection sheet is provided in Appendix A.

4.1.3 Identification of Communities

At the initiation of the study, the ENVC provided CBCL Limited with a list of communities in Newfoundland and Labrador believed to have an operational fish plant on site. The communities were categorized based on population size and region. An initial list of twenty four very small, small, and medium sized communities (as defined in the *Sustainable Options Report*, ENVC, 2009) was selected to participate in the study. The list was approved by the ENVC.

During initial phone conversation with representatives from the initial community list several communities indicated that they did not have operational fish plants. In an attempt to keep 24 communities in the study, additional communities were added to the list. A total of 27 communities were initially contacted. Throughout the course of the study, it was discovered that only 19 had operating fish plants.

4.1.4 Water Quality Assessment

Once the list of participating communities was established, CBCL Limited requested raw and tap water quality data for each community from the ENVC. The data provided was entered into spreadsheets for later evaluation.

4.2 Field Study

4.2.1 Site Visits

Site visits were conducted by field technicians between September and December 2010. The date and time of each site visit were arranged by the technician through a phone call. At each site visit, the technician toured the system with the operator and filled out the information collection sheet. The technician also collected flow records wherever they were available. Where possible, the technician visited the fish plant and spoke to a representative of the company. Of the 19 communities visited during the study, only 15 filled out the information collection sheets provided by CBCL staff.

4.2.2 Water Use Records

As discussed previously municipal and industrial water use records were collected from each community if they were available. In many cases, however, insufficient data was collected during the initial site visit and CBCL was required to contact the community again to attempt to gather the necessary information. Occasionally, this second request also failed to produce useful flow data, usually because the operator(s) did not have access to municipal or industrial flow records. In other communities, the owners of the fish plant were not willing to provide water use records. By March of 2011 only six of the participating communities had produced water use and/or chlorine residual monitoring records. Four of these were judged to be of sufficient quality to justify further investigation.

4.3 Analysis and Report Development

The answers from the questionnaires and the water use and chlorine residual monitoring records collected during the field visits were entered into Microsoft Excel spreadsheets and integrated with the water quality data evaluated during the desktop study. The four communities that were able to provide historical water use and free chlorine residual monitoring records were chosen as case studies. These were evaluated in detail for the following:

- Average day water demand (with and without fish plant);
- Maximum day water demand (with and without fish plant);
- Fire flow requirements;
- Peaking factors;
- Design (diameter, velocity, pressure loss) of the central distribution main and any water storage volumes on the distribution system;
- Disinfection effectiveness as measured by CT achieved;
- Free chlorine residuals measured at different locations throughout the town;
- Boil water advisories vs. fish plant operation;
- Historical DBP levels;
- Fire hydrant spacing; and
- Various demographic indicators.

Diurnal water use data was available for two of the case study communities. This was assessed against the standard diurnal curves discussed in Chapter 2.

The remaining communities were evaluated based on the amount of information available for each. The following assumptions were used as required:

- Average residential per capita water demand = 395 Lpcd
- Average total per capita water demand = 804 Lpcd
- Maximum day peaking factor based on MOE recommendations;
- Peak hour peaking factor calculated using the PRP-Gumbel method;
- The pipe diameter reported by the operator represented the inner diameter (ID);
- The flow through the pipe is not obstructed by tuberculation or other forms of corrosion;
- Roughness factors (‘C’ for Hazen-Williams and ‘ks’ for Darcy-Weisbach) were chosen based on the pipe material indicated by the operator; and
- Ductile iron was assumed to be cement-lined, as this is the default design in most of Canada.

Note that communities with pipes with smaller IDs or with significant levels of corrosion would be expected to experience pressure losses in excess of those predicted in this study. Also, the results of this study only address the impacts of fire flow demands on the design of the central distribution main and major storage volumes on the distribution system. Other factors and components, including intake pump flow rating, the diameter and characteristics of the raw water transmission main, and the size of the distribution pumps (if not the same as the intake pumps) can also impact fire protection effectiveness in a community.

The results of the analyses were used to recommend infrastructure and operational improvements for specific communities and for the province as a whole.

CHAPTER 5 RESULTS

5.1 Introduction

This section presents the analyses conducted on the information gathered during the field portion of the study.

As discussed in Chapter 4, 15 of the 19 communities identified at the beginning of the field program provided responses to the questions on the information collection sheets. The responses varied in quality, making it necessary to incorporate information from numerous additional sources. This included:

- Information collection sheets and site visit reports from this study and two other ENVC water quality studies conducted concurrently;
- OETC community summary reports;
- ENVC publications;
- As-built drawings;
- Statistics Canada publications; and
- Environment Canada publications.

The demographics, record-keeping practices, water demands, storage and water distribution infrastructure, approximate water retention time, fish plant operation schedules, historical and observed water quality, disinfection effectiveness, boil water advisories, fire protection, and flushing practices of each community have been summarized and assessed in light of common industry practices. As required, assumptions have been made to allow for comparison amongst communities.

Of the 15 communities who responded to the survey only four were able to provide complete water use and free chlorine residual monitoring records. More detailed assessments of the water systems in these communities are provided in chapters 6 to 9 of this report.

5.2 Users

Water use is influenced by population size, land use, various socioeconomic factors, and the number and distribution of residential, commercial, industrial, and institutional users. Table 5.1 shows the total population as determined through the 2006 Census, the percentage change in population between 2001 and 2006, the median age, and serviced population (2010) in each of the participating communities. The percentage of the total population that is served by the community's potable water system is also indicated.

Table 5.1 Total and serviced populations in participating communities (Statistics Canada, 2006; ENVC, 2010)

Community	Total Population 2006	Population Change 2001-2006	Serviced Population 2010	% Serviced
	Statistics Canada	Statistics Canada	ENVC Records and Stats Canada	Calculated
Community A	3,764	-6.4%	3,764	100%
Community B	794	1.3%	659	83%
Community C	1,607	-9.8%	1,607	100%
Community D	552	-12.2%	552	100%
Community E	451	-11.6%	451	100%
Community F	1,877	-9.7%	1,877	100%
Community G	407	-10.2%	407	100%
Community H	417	-7.3%	417	100%
Community I	5,436	-8.0%	5,436	100%
Community J	978	-2.4%	978	100%
Community K	444	-6.9%	387	87%
Community L	1,539	-14.4%	1,481	96%
Community M	1,029	-6.6%	1,029	100%
Community N	2,448	-6.2%	2,448	100%
Community O	355	-3.0%	355	100%
Newfoundland and Labrador (2006)	**505,469**	**-1.5%**	**406,364***	**80%**
Canada (2006)	**31,612,897**	**5.0%**	**27,856,304***	**88%**

*from *Survey of Drinking Water Plants 2005-2007* (Statistics Canada, 2009B)

In 2006, the populations of the participating communities varied from 355 to 5,436. All but one experienced negative population growth between 2001 and 2006. Overall, these rates of population decline were higher than that of the province as a whole. The one community that grew during that period is located on the Avalon Peninsula close to St. John's. Most of the residents in the participating communities are served by centralized water supply systems.

Socioeconomic factors have been shown to have an impact on water use patterns. Table 5.2 shows the median age, unemployment rate, and median earnings in each of the participating communities as listed in the results of the 2006 Census.

Table 5.2 Median age and employment statistics in participating communities (Statistics Canada, 2006)

Community	Median Age	Unemployment*	Median Earnings
	2006	2005	2006
	Statistics Canada	Statistics Canada	Statistics Canada
Community A	45	28.4%	$12,839
Community B	45	14.9%	$12,520
Community C	48	39.8%	$14,095
Community D	41	50.0%	$10,112
Community E	47	18.6%	$8,044
Community F	40	29.9%	$11,683
Community G	47	53.1%	$11,871
Community H	37	58.7%	$10,080
Community I	40	24.0%	$18,802
Community J	44	29.1%	$8,740
Community K	38	13.7%	$10,883
Community L	46	23.4%	$11,680
Community M	40	32.7%	$12,574
Community N	48	23.6%	$10,226
Community O	49	13.3%	$13,272
Newfoundland and Labrador (2006)	**42**	**14.4%**	**$20,500**
Canada (2006)	**39**	**6.3%**	**$26,500**

The values in Table 5.2 show that the participating communities are, for the most part, characterized by higher median ages, higher unemployment rates, and lower median earnings than the province or Canada as a whole. Note that the unemployment rate is determined by dividing the number of people who report having been unemployed (i.e., of age and looking for work) in the week prior to the census by the total labour force. This method may have resulted in an exaggerated unemployment figure in communities with large seasonal industries.

Nonetheless, these socioeconomic indicators suggest that water use in the participating communities may not follow patterns observed in other Canadian communities. For example, Rhoades (1995) found that low-income neighbourhoods in Austin, Texas used less water overall than higher income neighbourhoods. The low-income areas also exhibited a flattened diurnal water curve compared to higher income areas, suggesting that residents were not operating on the 'standard' nine-to-five work day schedule. Based on their demographics, the communities that participated in this study might be expected to use less water, to use it to fulfill different needs, and to have flattened diurnal water use curves.

The number of dwellings, people per dwelling, land area, and population density will all have subtle effects on water use, retention time in the distribution system, and resulting water quality. These statistics are provided in Table 5.3 for each of the communities.

Table 5.3 **Land area and population density in participating communities (Statistics Canada, 2006)**

Community	# Dwellings	# People per Dwelling	Land Area	Population Density
	2006	2006	km^2	2006
Community A	1,485	2.5	31.5	119
Community B	315	2.5	11.6	69
Community C	645	2.5	31.3	51
Community D	225	2.5	3.3	167
Community E	180	2.5	29.8	15
Community F	645	2.9	13.7	137
Community G	155	2.6	26.7	15
Community H	135	3.1	38.2	11
Community I	2,060	2.6	62.0	88
Community J	395	2.5	12.1	81
Community K	165	2.7	14.6	30
Community L	1,539	1.0	14.3	108
Community M	385	2.7	7.6	136
Community N	1,005	2.4	25.7	95
Community O	155	2.3	2.9	122

Communities with more individual dwellings and fewer people per dwelling are expected to use more water than those with fewer homes and higher occupancy rates. This is because many domestic tasks that result in water demands occur irrespective of the number of people in the house (ex. dish-washing, gardening, running taps in the winter, etc.). This may not hold true in all cases, however, and may be difficult to differentiate from the many other factors that impact water demand. The total land occupied by a community will dictate the size of its distribution system while the population density will affect the expected water demand at different points in the system. Large distribution systems with few users are more likely to have long retention times and, consequently, water quality concerns related to water age.

The number and distribution of users will also impact the water demand and retention time in the system. The total number of residential, industrial, commercial, and institutional users in each community (as reported by system operators) is provided in Table 5.4. Some cells in the table have been left blank because the system operators in some communities were unable or unwilling to provide information about the number and types of users on their distribution systems.

Table 5.4 Residential, commercial, industrial, and institutional users in participating communities

Community	Total Users on System	Residential	Industrial	Commercial	Institutional	% Residential
Community A	2,001	1,800	1	200	-	90%
Community B	642	630	1	11	-	98%
Community C	743	729	1	9	5	98%
Community D	294	273	1	18	2	93%
Community E						
Community F	721	682	2	34	3	95%
Community G	196	189	1	4	3	96%
Community H	171	154	1	13	3	90%
Community I	2,828	2,403	2	420	3	85%
Community J	572	500	1	70	1	87%
Community K						
Community L	800	797	2	-	1	100%
Community M	412	412	1	-	1	100%
Community N	1,160	1,159	1	52	3	95%
Community O	250	222	1	25	2	89%

The communities participating in this study have relatively small populations and are mostly residential. Exceptions include communities A and I, which are larger and act as hubs for surrounding communities. The distribution of users will impact the shape of the diurnal curve (Section 2.5) as well as the overall water demand. Communities that are more than 95% residential with only one industrial user can likely be characterized using a standard residential diurnal curve and residential per capita water use values when the facility is offline. Those with a more complex mix of users are less likely to conform to the standard residential diurnal curve and may have higher per capita water demands throughout the year.

5.3 Record-keeping Practices

Water system operators in Newfoundland and Labrador are encouraged to keep records of the amount of water used in the community. Most who choose to do so input total water use values into daily or monthly log sheets. Some also record the instantaneous and average flow rate (L/min or USgpm) along with the total number of hours that their distribution pumps operate.

The flow monitoring location chosen during the design of the system will impact the usefulness of the data collected. A flow meter located at the intake will measure all of the water used in the treatment system (backwashing etc.), lost in the distribution system, and consumed by users. One located between the point of chlorination and a large storage volume will record the amount of water sent to the storage volume but will not provide an accurate estimate of the rate of water consumption by users from hour to hour. A summary of the location(s) of the flow meters used in the participating communities is provided in Table 5.5.

Table 5.5 Record-keeping practices in participating communities ('x' indicates communities with written water use records)

Community	Record Keeping				
	Intake	Treatment System Outlet	Municipal Transmission Main	Industrial User	Pump Operation
Community A		x		x	
Community B		x			
Community C		x			x
Community D	x				
Community E					x
Community F		x		x	
Community G	x			x	
Community H			x	x	x
Community I		x		x	x
Community J	x		x		x
Community K		x			x
Community L	x	x			x
Community M			x	x	x
Community N	x				x
Community O		x			

The results of the survey suggest that all of the participating communities keep some form of water use records. Unfortunately, many of the operators who reported having the records were unable or unwilling to provide them to CBCL during the site visits or after follow-up phone calls.

Most of the participating communities who reported monitoring water use do so at the outlet of the treatment system or by recording pump operating details. Only six reported that they monitored the industrial user's water use.

5.4 Predicted Water Demands

Adequate water use records were only provided by four of the 15 communities that participated in the study. In the absence of actual data, most design standards and guidelines in Canada recommend using average per capita water use values and peaking factors to estimate the average day, maximum day, and peak hour demands of a community (see Section 2.4). These demand values can then be used to design or evaluate the water supply system.

The ADD, MDD, and PHD calculations were performed for all 15 of the participating communities, including those that provided water use records. The calculations were performed using three different average per capita water use values. The first (340 Lpcd) was drawn from the NL Guidelines. It represents residential water use only. The other two values (395 Lpcd and 804 Lpcd) were provided to the ENVC by Environment Canada. The smaller value represents the average per capita water demand for residential users in the province while the larger one represents the total average per capita value (i.e., includes commercial, industrial, and institutional water demands).The results are summarized in Tables 5.6 through 5.8.

Table 5.6 Average day water demands predicted using different per capita water use values

Community	Population	Calculated (340 Lpcd)	Calculated (395 Lpcd)	Calculated (804 Lpcd)
		L/day	L/day	L/day
Community A	3,764	1,279,760	1,486,780	3,026,256
Community B	6 59	224,060	260,305	529,836
Community C	1,607	546,380	634,765	1,292,028
Community D	552	187,680	218,040	443,808
Community E	451	153,340	178,145	362,604
Community F	1,877	638,180	741,415	1,509,108
Community G	407	138,380	160,765	327,228
Community H	417	141,780	164,715	335,268
Community I	5,436	1,848,240	2,147,220	4,370,544
Community J	978	332,520	386,310	786,312
Community K	387	131,580	152,865	311,148
Community L	1,481	503,540	584,995	1,190,724
Community M	1,029	349,860	406,455	827,316
Community N	2,448	832,320	966,960	1,968,192
Community O	355	120,700	140,225	285,420

Table 5.7 Predicted maximum day peaking factors for participating communities (MOE 2008, DVGW, 2007)

Community	Population	MOE*	DVGW
Community A	3,764	2.00	2.10
Community B	659	2.75	2.39
Community C	1,607	2.50	2.24
Community D	552	2.75	2.43
Community E	451	3.60	2.46
Community F	1,877	2.50	2.21
Community G	407	3.60	2.48
Community H	417	3.60	2.48
Community I	5,436	2.00	2.04
Community J	978	2.75	2.32
Community K	387	3.60	2.49
Community L	1,481	2.50	2.25
Community M	1,029	2.50	2.31
Community N	2,448	2.25	2.17
Community O	355	3.60	2.51

*Maximum day peaking factors for communities with more than 500 residents are also provided in the NL Guidelines

Table 5.8 Predicted peak hour peaking factors for participating communities

Community	Population	MOE*	Harmon	PRP-Gumbel	AWWA	DVGW
Community A	3,764	3.0	3.4	3.6	2.3	4.6
Community B	659	4.0	3.9	5.2	4.7	6.1
Community C	1,607	3.5	3.7	4.2	3.3	5.3
Community D	552	4.1	4.0	5.4	5.1	6.3
Community E	451	4.5	4.0	5.7	5.5	6.5
Community F	1,877	3.4	3.6	4.1	3.1	5.1
Community G	407	4.8	4.0	5.9	5.7	6.6
Community H	417	4.7	4.0	5.9	5.7	6.6
Community I	5,436	3.0	3.2	3.4	2.0	4.3
Community J	978	3.8	3.8	4.7	4.0	5.7
Community K	387	4.9	4.0	6.0	5.8	6.7
Community L	1,481	3.6	3.7	4.3	3.4	5.3
Community M	1,029	3.7	3.8	4.6	3.9	5.7
Community N	2,448	3.2	3.5	3.9	2.8	4.9
Community O	355	5.1	4.0	6.2	6.0	6.8

*Peak hour peaking factors for communities with more than 500 residents are also provided in the NL Guidelines

The actual ADD, MDD, and PHD values for each of the four case study communities that provided water use records are discussed at length in Chapter 6. All further analyses conducted as part of Chapter 5 use the water demand values estimated using a per capita demand of 395 Lpcd, MOE maximum day peaking factors, and peak hour peaking factors calculated using the PRP-Gumbel method.

Recommended fire protection allowances were determined based on the fire flow and duration recommendations for small communities provided in the *Ontario Guidelines for Drinking Water Systems* (MOE, 2008).

Table 5.9 Recommended fire flow, fire flow duration, and fire protection storage in participating communities

Community	Population	Recommended Fire Flow	Recommended Duration	Total Volume
		L/min	hours	m^3
Community A	3,764	7,500	2	900
Community B	659	2,280	2	274
Community C	1,607	4,740	2	569
Community D	552	2,280	2	274
Community E	451	2,280	2	274
Community F	1,877	5,700	2	684
Community G	407	2,280	2	274
Community H	417	2,280	2	274
Community I	5,436	8,640	2	1,037

Community	Population	Recommended Fire Flow	Recommended Duration	Total Volume
		L/min	hours	m^3
Community J	978	2,280	2	274
Community K	387	2,280	2	274
Community L	1,481	4,740	2	569
Community M	1,029	3,840	2	461
Community N	2,448	5,700	2	684
Community O	355	2,280	2	274

The fire protection allowances shown in Table 5.9 were used to determine recommended storage volumes and fire hydrant spacing. These are discussed in sections 5.5 and 5.10, respectively.

5.5 Infrastructure

Many of the participating communities were able to provide at least some information about the size of the major components of their distribution systems. Each operator was specifically asked to provide details about the raw water transmission main, central distribution main, and any storage volumes on the distribution system. In some cases, the information collected did not match the information provided to CBCL by the ENVC and/or the findings of the two other water quality studies conducted by CBCL for the ENVC concurrently with this one. In such cases, a value was chosen that was believed to be the most accurate based on the source of the information. Table 5.10 lists some of the characteristics of the raw water transmission mains in the participating communities.

Table 5.10 Characteristics of raw water transmission mains in participating communities

Community	Diameter	Length	Volume	Material
	mm	m	m^3	
Community A	350	-	-	Asbestos cement
Community B	350	500	48	Ductile iron
Community C	450	1,524	242	Ductile iron
Community D	200	500	16	PVC
Community E	-	-	-	
Community F	300	-	-	PVC
Community G	150	-	-	
Community H	250	-	-	Ductile iron
Community I	350	335	32	Ductile iron
Community J	400	-	-	Ductile iron
Community K	300	3,000	212	
Community L	400	-	-	
Community M	450	91	14	HDPE
Community N	450	50	8	PVC
Community O	300	91	6	Ductile iron

The diameter and length of the raw water transmission mains are likely based on the population (i.e., residential flow required), the number and type of other water users, and the distance between the community and its water source. Long, large diameter pipes will have longer retention times, particularly during periods of low demand. For the most part, this is unlikely to impact the quality of the water delivered to customers because it occurs before chlorination. In some cases, however, it may result in increased corrosion and increased dissolution of corrosion products as well as the growth of microorganisms. Where possible, it is preferable to size the raw water transmission main appropriately so that retention time is minimized to prevent water quality deterioration and corrosion.

The characteristics of the central distribution mains in each of the participating communities are provided in Table 5.11. For the purposes of this study the central distribution main is defined as the largest main between the point of chlorination and the first user.

Table 5.11 Characteristics of central distribution mains in participating communities

Community	Diameter	Length	Volume	Material*
	mm	m	m^3	
Community A	350	2,500	241	Cast iron
Community B	350	610	59	Ductile iron
Community C**	450	5,275	839	Ductile iron
Community D	200	305	10	Ductile iron
Community E	250	1,000	49	PVC
Community F	300	3,000	212	
Community G	150	300	5	
Community H	250	1,500	74	Ductile iron
Community I	350	1,500	144	Ductile iron
Community J	400	500	63	
Community K	250	400	20	Ductile iron
Community L	400	1,500	188	
Community M	250	800	39	PVC
Community N	400	1,000	126	HDPE
Community O	250	183	9	Ductile iron

*some information obtained from other ENVC studies
**distribution main length includes a large section between the first user and the fish plant

The diameter, length, and material of the central distribution main can all affect the quality of the water delivered to the user. The characteristics of the main will also impact the residual pressure available in the system, the retention time (i.e., water age) and the water velocity. The impacts of main size and material on pressure loss in the distribution system are discussed for each of the case study communities in Chapter 6. The average retention time, water velocity at average day flow, and velocity at average day flow + fire flow for each community are shown in Table 5.12. Note that the ADD (L/day) and flow (L/min) used for the calculations are based on an average per capita water demand of 395 Lpcd. Retention time is calculated by dividing the volume of the pipe by the flow through it. Using a larger per capita value decreases the calculated retention time and increase the calculated water velocity.

Table 5.12 Average retention time and velocity in distribution mains in participating communities

Community	Average Retention Time	Velocity	
		Average Day	Average Day + Fire Flow
	hours	m/s	m/s
Community A	3.9	0.18	1.48
Community B	5.4	0.03	0.43
Community C	31.7	0.05	0.54
Community D	1.1	0.08	1.29
Community E	6.6	0.04	0.82
Community F	6.9	0.12	1.47
Community G	0.8	0.11	2.26
Community H	10.7	0.04	0.81
Community I	1.6	0.26	1.76
Community J	3.9	0.04	0.34
Community K	3.1	0.04	0.81
Community L	7.7	0.05	0.68
Community M	2.3	0.10	1.40
Community N	3.1	0.09	0.85
Community O	1.5	0.03	0.81

The average retention time between the point of chlorination and the first user varies from approximately 1 hour to over 10 hours. The amount of retention time in the rest of the system will depend on the size of the system, water demands in individual areas, and the size of distribution system components (pipes, storage, etc.) in those areas.

Depending on the quality of the water being chlorinated and the amount of chlorine added to the water the communities with higher retention times would be more likely to record high levels of THMs and HAAs. This will be discussed at greater length in Section 5.7 and the case studies in Chapter 6.

The velocity of water in the central distribution mains varied from a low of 0.03 m/s to a high of 0.26 m/s under normal (average day) flow conditions. This is well below the maximum velocity of 1.5 m/s recommended in the AC and NL guidelines. When fire flow is included, the estimated velocity of the water in the mains increases. In some communities, the increase is quite large. Though intuitively one might assume that this increase would be more dramatic in smaller communities where the required fire flow can be double the average day flow, it is the large communities that experience large increases in velocity, as shown in Figure 5.1.

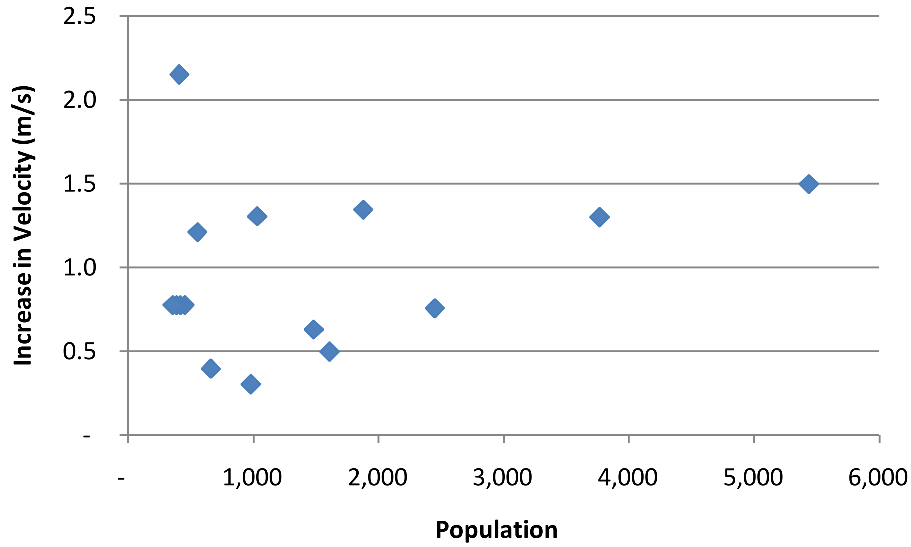


Figure 5.1 Relationship between increased velocity under fire flow conditions and population

This relationship occurs because at lower populations the recommended fire flow increases at a rate greater than the increase in average flow at 395 Lpcd.

The maximum recommended velocity under fire flow conditions is 3 m/s. High velocities are associated with increased pressure loss and should be avoided if possible. Though the estimated velocity of water in the transmission mains in the participating communities all appear to be below this value, it should be kept in mind that the estimates are based on an average per capita water demand of 395 Lpcd. The results of the case studies presented in Chapter 6 suggest that some communities in the study have average per capita water use rates well above this standard value. It is likely that water velocities in these communities exceed those recommended in the AC and NL guidelines at least periodically, resulting in excessive pressure loss.

Many (but not all) of the participating communities have some form of water storage infrastructure. The volume, location, and shape of these are provided in Table 5.13.

Table 5.13 Characteristics of water storage infrastructure in participating communities

Community	Storage Volume	Location	Shape
	m^3		
Community A	946 2,839	After first user After first user	Sphere Cylindrical
Community B			
Community C	1,825	After chlorination, before first user	
Community D			
Community E*	Not indicated	After first user	Standpipe
Community F			
Community G	6.4	Before chlorination	Wet well
Community H			

Community	Storage Volume	Location	Shape
	m^3		
Community I*	34,065	Before chlorination	Pond w concrete dams
Community J**	19	Not indicated	Square
Community K			
Community L			
Community M	255	After chlorination, before first user	Round
Community N	1,125	After chlorination, before first user	Round
Community O***	949	After chlorination, before first user	Round

*A 2,000 m^3 storage tank is being built after the new water treatment system in Community I
**Not listed in the OETC Community Reports or the ENVC Water Storage Tank Database
***Storage volume listed as 'round' assumed to be spherical

A raw water reservoir located ahead of chlorination is unlikely to contribute to the formation of THMs and HAAs unless it experiences fluctuations in NOM levels. Storage located after the point of chlorination will contribute to chlorine decay and the formation of THMs and HAAs because it will increase the total retention time of the water in the system as shown in Table 5.14.

Table 5.14 Retention time in storage volumes located after chlorination in participating communities

Community	Storage Volume	ADD (at 395 Lpcd)	Retention Time
	m^3	m^3/day	hours
Community A*	3,785	1,487	61.1
Community C	1,825	635	69.0
Community E	Not indicated	178	Unknown
Community M	255	406	15.1
Community N	1,125	967	27.9
Community O	949	140	162.4

*total volume of two separate storage volumes

Communities with small water demands but large storage volumes are the most likely to experience long retention times.

Storage is recommended for most communities because it can be used to buffer variations in demand, provide fire flows, and minimize some of the problems caused by large water demands associated with industrial users. Guidelines for sizing storage volumes are provided in Section 2.7.4. The recommended storage volume calculated for each community based on these guidelines is provided in Table 5.15. Maximum day water demand has been calculated based on an average per capita water demand of 395 Lpcd and peaking factors from the MOE Guidelines.

Table 5.15 Recommended storage volumes for participating communities

Community	Maximum Day (L/day at 395 Lpcd)	Balancing Storage	Fire Protection Storage	Emergency Storage	Total Recommended Storage	Actual Storage
	MOE	m^3	m^3	m^3	m^3	m^3
Community A	2,973,560	743	900	411	2,054	3,785
Community B	715,839	179	274	113	566	-
Community C	1,586,913	397	569	241	1,207	1,825
Community D	599,610	150	274	106	529	-
Community E	641,322	160	274	108	542	Not indicated
Community F	1,853,538	463	684	287	1,434	-
Community G	578,754	145	274	105	523	6.4
Community H	592,974	148	274	105	527	-
Community I	4,294,440	1,074	1,037	528	2,638	34,065
Community J	1,062,353	266	274	135	674	19
Community K	550,314	138	274	103	514	-
Community L	1,462,488	366	569	234	1,168	-
Community M	1,016,138	254	461	179	894	255
Community N	2,175,660	544	684	307	1,535	1,125
Community O	504,810	126	274	100	500	949

Of those communities who do have water storage capacity, half have more than the volume recommended by the AC and NL guidelines. Where this storage is located after chlorination it may be contributing to chlorine decay and the formation of DBPs. Of the four remaining communities, only three were able to provide CBCL with information about the (approximate) volume of their storage components. These are all smaller than recommended and unlikely to be able to provide for fire protection and/or balance variations in demand.

Once again, the maximum day values used for these preliminary analyses are estimates and do not include industrial demands. Communities that hope to provide water to industrial users should have even more storage. Alternatively, dedicated storage can be provided for or by the industrial user. The latter approach is recommended as it balances the need for additional storage with water quality concerns arising from long retention times.

5.6 Fish Plant Operation

The fish plants in most small communities in Newfoundland and Labrador operate only periodically. This often leads to large variations in industrial and total water demands over the course of the year, which can result in water quality deterioration during periods of low demand and operational challenges when demands are higher.

During the field visits, community and fish plant operators were asked to indicate what times of the year the fish plants were operational. As shown in Figure 5.2, one third of the respondents were not able to provide any information.

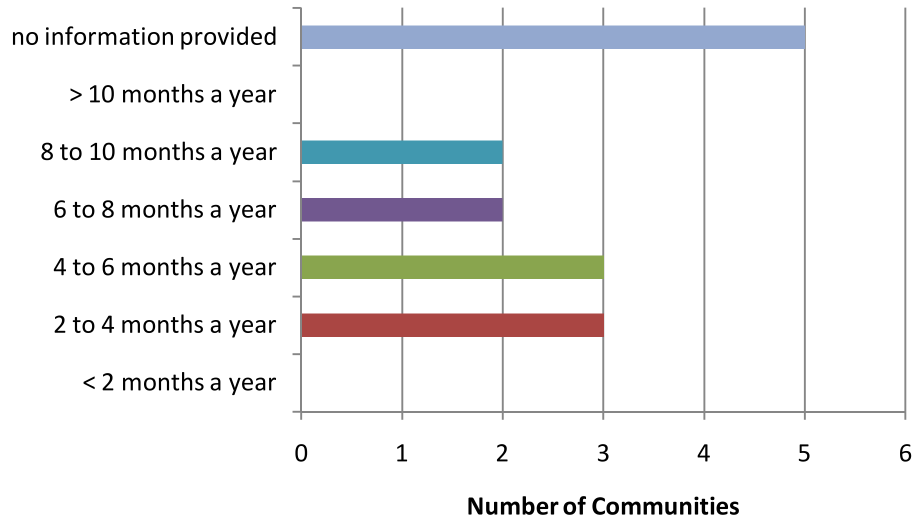


Figure 5.2 Reported duration of fish plant operation in participating communities

Of the communities that did provide a response to the question, three indicated that their fish plant operated for between two and four months each year. None of the operators reported year-long fish plant operation.

The impacts of fish plant operation on total water demand varied from community to community. In some communities, the fish plant's water demand represented a large portion of the design flow used to size the distribution system components. Intuitively, these would be expected to experience more water quality and operational difficulties due to periodic plant operation than those where the fish plant demand represents only a small proportion of the total water demand.

5.7 Water Quality

Only limited water quality information was available for most of the participating communities, which made it difficult to assess the impacts of fish plant operation on water quality. The impacts of fish plant operation on water quality and system operation are discussed in greater detail in the case studies presented in Chapters 6 to 9.

Nonetheless, measured THM and HAA values were compared to a few variables to determine which factors were most likely to impact DBP formation. First, the average measured THM and HAA concentrations for each community were compared to their average DOC levels (Figure 5.3).

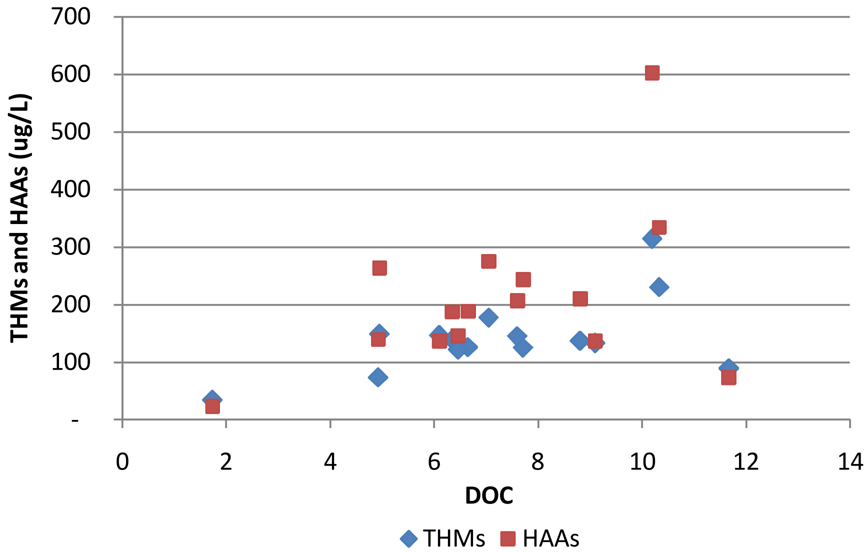


Figure 5.3 **Average THMs and HAAs vs. average DOC (ENVC, 2000 to 2009)**

Visually, both DBPs appear to be positively correlated to DOC, but only the relationship between THMs and DOC was found to be significant at a 95% confidence level ($p < 0.05$). Though the relationship between DOC, which is a proxy for NOM, and both THMs and HAAs is well documented (Section 3.2.2), any important correlations between the variables were likely obscured by a number of other factors. These include the chlorine dose, the distribution of different types of NOM in each water source, water temperature, pH, level of water treatment provided, and retention time within the distribution system.

THM and HAA concentrations were also compared to the retention time in the central distribution mains calculated in Section 5.5. This is shown in Figure 5.4.

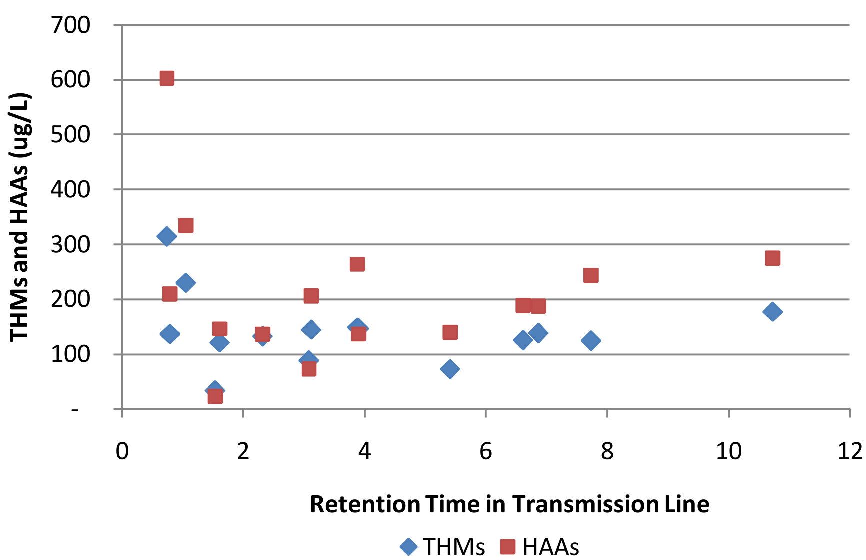


Figure 5.4 **Average THMs and HAAs vs. calculated retention time in the central distribution main**

No apparent relationships existed between the retention time in the central distribution main in each community and the average THMs and HAAs measured in their tap water.

The results were similar when the average THM and HAA concentrations were compared to the total land area in each community, which should be at least loosely related to the size of the water distribution system and total system retention time (Figure 5.5).

Some correlation between the variables is apparent in Figure 5.5, but numerous outliers are apparent. Once again, given the large number of factors that impact the formation of THMs and HAAs, the lack of apparent relationship is not surprising. It is expected that under controlled conditions THMs and HAAs would be found to be dependent on retention time.

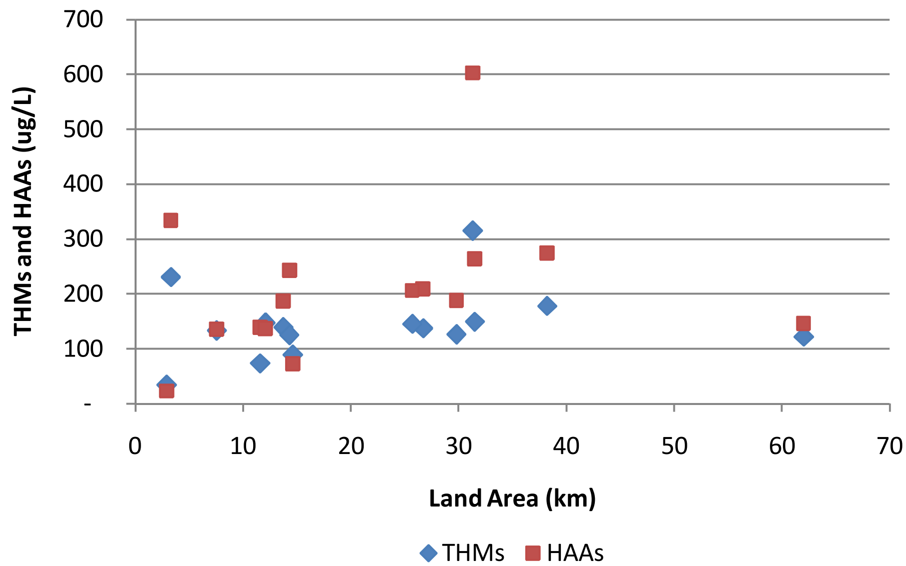


Figure 5.5 **Average THMs and HAAs vs. land area of community**

The levels of corrosion products, such as iron and lead, in tap water can also be affected by retention time (Section 3.2.3). Average iron and lead concentrations in the tap water from each of the participating communities are summarized in Table 5.16.

Table 5.16 **Average lead and iron concentrations in the tap water in participating communities (ENVC, 2000 to 2009)**

Community	Average Lead ug/L	Average Iron ug/L
Community A	3.0	262.5
Community B	3.0	82.5
Community C	1.5	325.8
Community D	0.6	532.5
Community E	0.9	120.4
Community F	0.4	89.3
Community G	0.6	298.2
Community H	0.5	276.9
Community I	1.6	217.8

Community	Average Lead	Average Iron
	ug/L	ug/L
Community J	0.6	49.7
Community K	3.7	404.8
Community L	2.4	343.7
Community M	0.5	79.4
Community N	1.2	198.9
Community O	1.0	56.8

On average, none of the participating communities were found to have average historical lead levels above the health-based limit recommended by Health Canada (10 ug/L). Four of the communities were found to have average historical iron levels above the recommended aesthetic objective of 300 ug/L. The high iron levels could be related to corrosion within the distribution system, background levels in the water source, or a combination of both.

Concerns about poor water quality are not restricted to residential users. Fish plant operators in many communities reported difficulties related to low chlorine, high chlorine, low pressure, and bacteria and/or viruses, as shown in Figure 5.6.

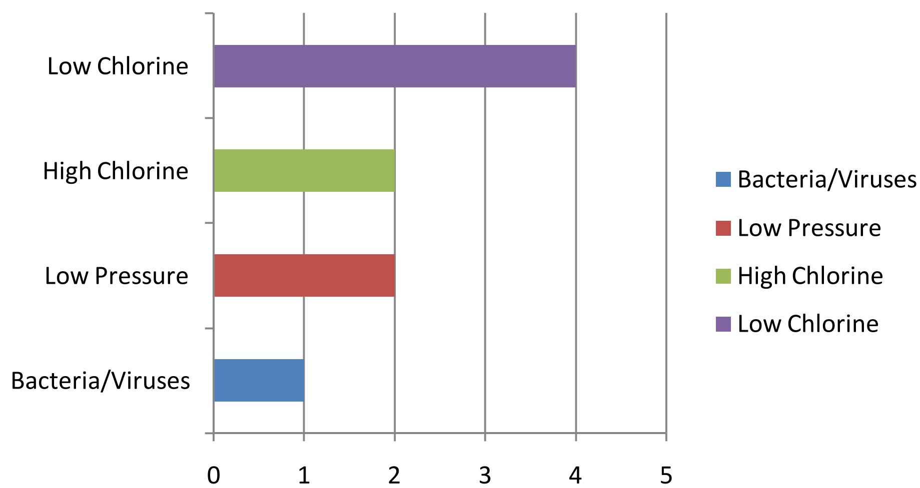


Figure 5.6 Water quality concerns reported by fish plant operators

The impacts of variable water demands on water quality in a selection of case study communities is examined in greater detail in Chapters 6 to 9 of this report.

5.8 Disinfection

Provincial disinfection requirements are described in the ENVC document *Standards for Bacteriological Drinking Water Quality*. Communities that rely on chemical disinfection are required to achieve 20 minutes of chlorine contact time with a free chlorine residual of 0.3 mg/L.

Disinfection is quantified using the log reduction and CT concepts. Log reduction refers to the percentage of microorganisms removed or inactivated in a water treatment system. 90% reduction of microorganisms corresponds to 1-log reduction. The amount of log reduction achieved in a disinfection system is determined based on the level of treatment provided.

Different types of filters are assigned log reduction credits based on the amount of microorganisms they are able to remove. Microorganism inactivation in chemical/physical disinfection processes is dependent both on the amount of disinfectant applied (dose) and the amount of time that the microorganisms spend in contact with the disinfectant (effective contact time). Effective contact time is calculated by dividing the volume available for contact by the flow through the system. The former is determined by multiplying the total available volume by a baffling factor (0.1 to 1) to account for mixing. Large tanks with a single inlet and single outlet are usually assigned a baffling factor of 0.3 while contact pipes are assigned a baffling factor of 1.

Effective contact volumes were estimated for each community based on the infrastructure details presented in Section 5.5. These are presented in Table 5.17 along with chlorine residual results obtained during field visits.

Table 5.17 Effective contact volume, chlorine residual, and additional disinfection infrastructure in participating communities

Community	Effective Contact Volume	Chlorine Residual	Other Disinfection
	m^3	mg/L	
Community A	241	0.99	
Community B	59	0.95	
Community C*	567	0.06	Ozone for oxidation of NOM and metals
Community D	10	1.7	
Community E	49	0.3	
Community F	212	0.06	
Community G	5	0.7	
Community H	74	0.8	
Community I	144	2.2	New WTP under construction (2011/2012)
Community J	63	0.41	
Community K	20	0.03	
Community L	188	0	
Community M	116	0.3	
Community N	463	0.9	
Community O	294	0.07	

*contact volume includes piping between treatment plant and storage tank and the storage volume

Percent inactivation (i.e., log reduction) can be estimated by multiplying the dose by the amount of contact time and comparing the result to values listed in 'CT tables'. Temperature and pH can also influence disinfection effectiveness and have also been incorporated into the tables. The effects of chemical and physical disinfection vary depending on the organism being inactivated. CT tables have been developed for viruses, *Giardia*, and *Cryptosporidium*.

A system that complies with the provincial disinfection requirements in Newfoundland will achieve a CT of 6. This represents between 2- and 4-log inactivation of viruses, depending on the pH and temperature of the water. The effective contact time in each of the communities at ADD (calculated based on 395 Lpcd) is listed in Table 5.18 along with the resulting CT values. Note that two communities were able to provide the average flow rate between the point of chlorination and a contact volume ahead of the first user. In these two cases the reported flow rate has been included in the table and used to calculate CT.

Table 5.18 Effective contact time and CT achieved at calculated ADD

Community	Effective Contact Time	CT Achieved
	min	
Community A	233	231
Community B	325	308
Community C	428	26
Community D	63	108
Community E	397	119
Community F	412	25
Community G	47	33
Community H	644	515
Community I	97	213
Community J	234	96
Community K	185	6
Community L	464	-
Community M	410	123
Community N	690	621
Community O	518	36

Almost all of the participating communities are likely to meet provincial disinfection requirements at calculated ADDs. The lack of water use records from most of the communities made it impossible to determine whether disinfection requirements are being met during periods of higher demand. Nonetheless, CT values were determined for the peak hour flows predicted by the peak hour peaking factors presented in Table 5.8 (PRP-Gumbel method). The effective contact time and CT predicted at these flows are presented in Table 5.19.

Table 5.19 Effective contact time and CT values predicted at calculated peak hour flow

Community	Effective Contact Time	CT Achieved
	min	
Community A	64	64
Community B	63	59
Community C	428	26
Community D	12	20
Community E	69	21

Community	Effective Contact Time	CT Achieved
	min	
Community F	101	6
Community G	8	6
Community H	110	88
Community I	28	62
Community J	50	20
Community K	31	1
Community L	108	-
Community M	88	26
Community N	177	159
Community O	518	36

At calculated peak flows some of the communities will fail to achieve a CT of 6 and/or meet provincial disinfectant requirements. Many others will comply but only just. This is of concern in light of the average and MDDs measured in some of the case study communities (Chapter 6), which are higher than the peak hour demands used to estimate the effective contact time and CT values presented in Table 5.19.

The ENVC may consider putting some communities on precautionary boil water advisories during periods of high water demand. This should be done cautiously, however, as boil water advisories can undermine user confidence. Instead, communities and industrial users should be encouraged to adopt design and operational strategies that will minimize the effects of periodic high water demands on disinfection effectiveness.

5.9 Fire Protection

Fire flow requirements are usually calculated for individual buildings using methods developed by the insurance industry (Section 2.6). Smaller communities often lack the resources to determine or provide the fire flows required for individual buildings and opt instead to develop their infrastructure based on estimated values. The recommended fire flow was determined for each community based on population and recommendations provided in the MOE Guidelines.

The amount of fire flow available during a fire event can also be impacted by the space between individual fire hydrants. The FUS recommends hydrant spacing based on population. Recommended fire flow and hydrant spacing for each participating community are listed in Table 5.20.

Individual systems' ability to meet fire flow requirements were not evaluated in detail for most of the participating communities because of a lack of reliable water use data. The four communities that did provide sufficient data are discussed in detail in chapters 6 to 9.

Fire hydrant spacing was estimated for each community based on the number of hydrants reported by the operator and the land area of each community as listed in records from Statistics Canada. The results are presented in Table 5.21.

Table 5.20 Recommended fire flow (MOE, 2008) and fire hydrant spacing (CGI Inc., 1999)

Community	Population	Recommended Fire Flow	Recommended Area/Hydrant
		L/min	m^2
Community A	3,764	7,500	13,000
Community B	659	2,280	16,000
Community C	1,607	4,740	15,000
Community D	552	2,280	16,000
Community E	451	2,280	16,000
Community F	1,877	5,700	14,000
Community G	407	2,280	16,000
Community H	417	2,280	16,000
Community I	5,436	8,640	13,000
Community J	978	2,280	16,000
Community K	387	2,280	16,000
Community L	1,481	4,740	15,000
Community M	1,029	3,840	15,000
Community N	2,448	5,700	14,000
Community O	355	2,280	16,000

Table 5.21 Land area, number of hydrants, and approximate hydrant spacing in participating communities

Community	Land Area	Number of Hydrants	Average Area/Hydrant	Diagnosis
	km		m^2	
Community A	31.5	250	12,600	Adequate
Community B	11.6	39	29,667	Inadequate
Community C*	31.3	48	65,208	Inadequate
Community D	3.3	8		
Community E	29.8	37	80,541	Inadequate
Community F	13.7			
Community G	26.7	15	178,000	Inadequate
Community H	38.2	22	173,636	Inadequate
Community I	62.0			
Community J	12.1	140	8,629	Adequate
Community K	14.6	25	58,400	Inadequate
Community L	14.3	75	19,067	Inadequate
Community M	7.6	37	20,405	Inadequate
Community N	25.7	153	16,797	Inadequate
Community O	2.9	24	12,125	Adequate

The results in Table 5.21 suggest that hydrant spacing in many of the participating communities is adequate (under the stated assumptions). Those who were found to have excessive space between hydrants may benefit from the installation of additional hydrants. The actual space between hydrants should be measured before installing more as the land area provided by Statistics Canada is likely to be an overestimate of the size of the water distribution system.

5.10 Flushing Programs

Communities with fewer than 500 people were asked to indicate whether or not they had formal flushing programs in place and, if possible, to provide details. During the field program, the majority of the participating communities filled out this section of the information collection sheet, irrespective of their population. The results are summarized in Table 5.22. Non-responses (i.e., blanks) can likely be attributed to improper completion of the information collection sheets.

Table 5.22 Summary of flushing programs in participating communities

Community	Flushing program in place?	Frequency	Details
Community A			
Community B	Yes	As required	One hydrant at the playground is kept open during summer to keep the chlorine residual up.
Community C	Yes	Twice a year	
Community D	Yes		All hydrants are opened for about 5hrs.
Community E	Yes	As required	Hydrants opened one at a time.
Community F			
Community G	Yes	Monthly	Hydrants are opened once a month for 20 minutes.
Community H	Yes	Twice a year	
Community I			
Community J	Yes	Twice a year	All hydrants are opened for 10 to 15 minutes. The entire process takes about a week.
Community K	Yes		Fire hydrants are opened.
Community L	Yes	Twice a year	
Community M	Yes	Twice a year	All hydrants are flushed.
Community N	Yes	Annually	Various hydrants are opened until water runs clean.
Community O	Yes	Twice a year	Every hydrant is opened for 20 minutes.

Some communities provided copies of their flushing programs. These are provided in Appendix G.

CHAPTER 6 CASE STUDY: COMMUNITY A

6.1 Introduction

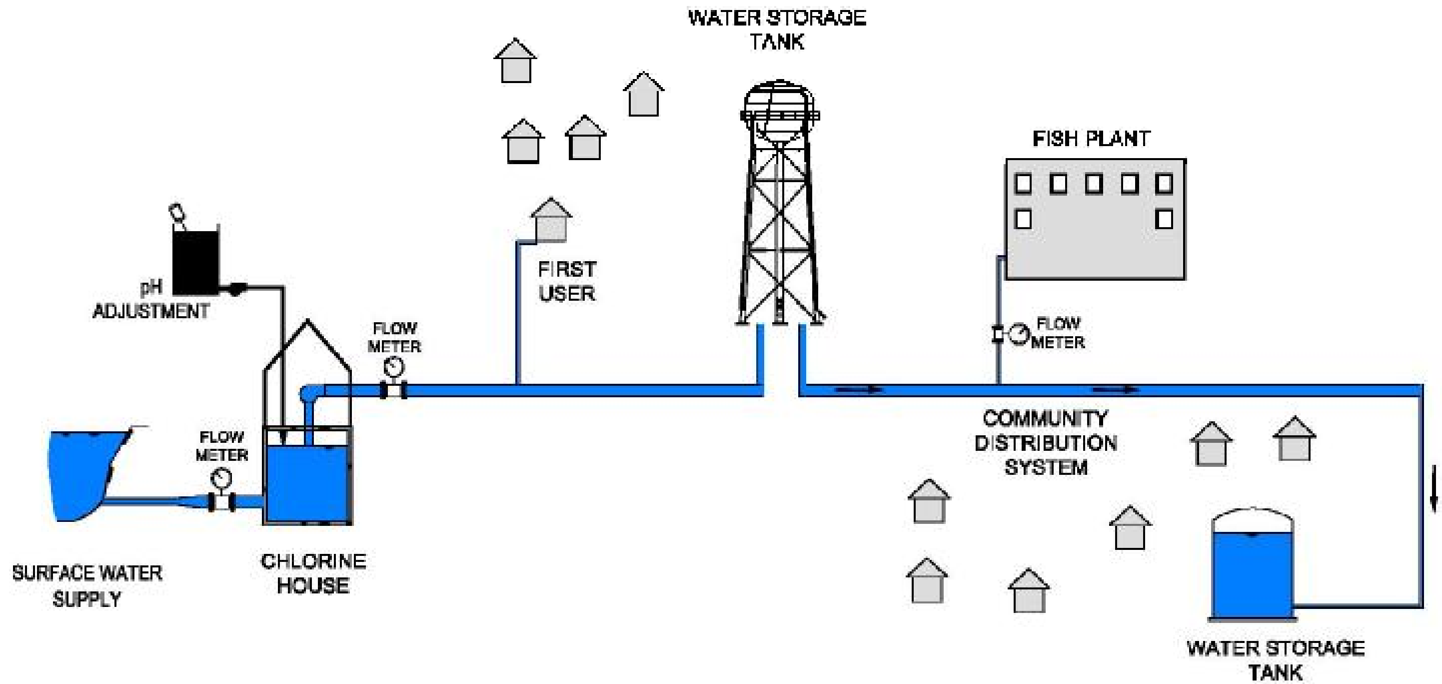
Community A is located in the Eastern Region of Newfoundland and Labrador. It has an approximate land area of 31.5 km^2. Drinking water is drawn from a nearby pond, disinfected with chlorine, and pH adjusted before being sent to a 946 m^3 water storage tower. Flow is measured both ahead of and after chlorination. Water flows through the distribution system through gravity, eventually gathering in a secondary storage tank (2,839 m^3). Additional system design details are provided in various parts of Chapter 5 and in the subsections that follow. A basic schematic of Community A's water supply system is provided in Figure 6.1.

6.2 Users

Table 6.1 presents a selection of demographic data gathered from Statistics Canada, ENVC records, Community A's website, and provided by the system operator during the site visit.

Table 6.1 Demographics – Community A

Parameter	Value	Source
Total Population	3,764	Stats Canada, 2006 Census
Serviced Population	3,764	ENVC
Population Change (2001 - 2006)	-6.4%	Stats Canada, 2006 Census
Total Users on System		
Residential	1,800	Reported
Industrial	1	Reported
Commercial	n/a	Reported
Institutional	n/a	Town website
Total	2,000	Calculated
% Residential	90%	Calculated
Population Density (per km^2)	119	Calculated
Unemployment	28%	Stats Canada, 2006 Census
Median Earnings	$12,839	Stats Canada, 2006 Census
Median Age	45	Stats Canada, 2006 Census



Date	Scale	Designed	Drawn	Checked	Approved	Contract
11/09/12	N.T.S.	SG	CG	SG	GS	103046.00

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Community A Schematic

Figure
6.1

The most recent published census data indicates that approximately 3,764 people lived in Community A in 2006. The entire population is serviced by the drinking water system. Between 2001 and 2006 the population shrank by 6.4%. A reduction in population leads to reduced water demand and can result in increased water age.

During a follow-up phone call, the system operator reported that the community has approximately 1,800 residential users (i.e.. homes), 1 industrial user, and a total of 2,000 users. The number of commercial and institutional users was not indicated but the town's website lists an elementary school, a high school, a branch of the College of the North Atlantic, an adult education collegiate, a public library, two community health centres, and a retirement home in addition to numerous businesses. Nonetheless, it can be said that the majority of the users in the community fall into the residential category (90%). This is a smaller percentage than that found in many of the smaller communities that participated in the study (Section 5.2); reflecting the greater user diversity found in larger communities. Greater user diversity leads to variation in water use patterns, increased water demand, and a higher calculated per capita water demand.

The population density in Community A is approximately 119 people per km^2, which is higher than in many of the smaller communities that participated in the study. It suggests that water system users are generally clustered close together, reducing the size of the distribution system and potentially minimizing some of the water quality concerns related to stagnation and high water age.

The unemployment level, median earnings, and median age of the community are characteristic of rural Newfoundland. In particular, communities with seasonal industries tend to have high reported unemployment levels due to the data collection method employed by Statistics Canada (Section 5.2). Even so, these economic indicators suggest that water demands and use patterns in Community A may differ from those in larger communities in the province and in Canada.

6.3 Water Demands

Daily water use records from 2009 were provided by the Community A system operator. The town uses two totalizers to measure the amount of water used by residential and commercial users as well as that used by the fish plant when it's operating. The records provided to CBCL Limited were entered into spreadsheets and used to estimate the daily volume of water measured by each of the totalizers.

During the site visit, the operator indicated that the fish plant operated from April until sometime in August with occasional short-term operations in September and October. Upon closer inspection it was determined that the fish plant records provided by the town were incomplete as they only covered April and June of 2009. Thus, all of the analyses performed for the study were based exclusively on the records from the totalizer located between the point of chlorination and the water storage tower at the beginning of the distribution system.

These numbers provide only an approximation of the community's water use as they represent the amount of water provided to some of the community as well as that entering the tower to make up for the amount drawn out by users. The tower is small (946 m^2), so it can be assumed that days when high volumes of water flowed into the tower were preceded by a period (hours or days) of higher-than-average water demand in the community.

Figure 6.2 shows the calculated daily water use measured by the totalizer over the course of 2009. The blue shaded area indicates when the fish plant was in operation.

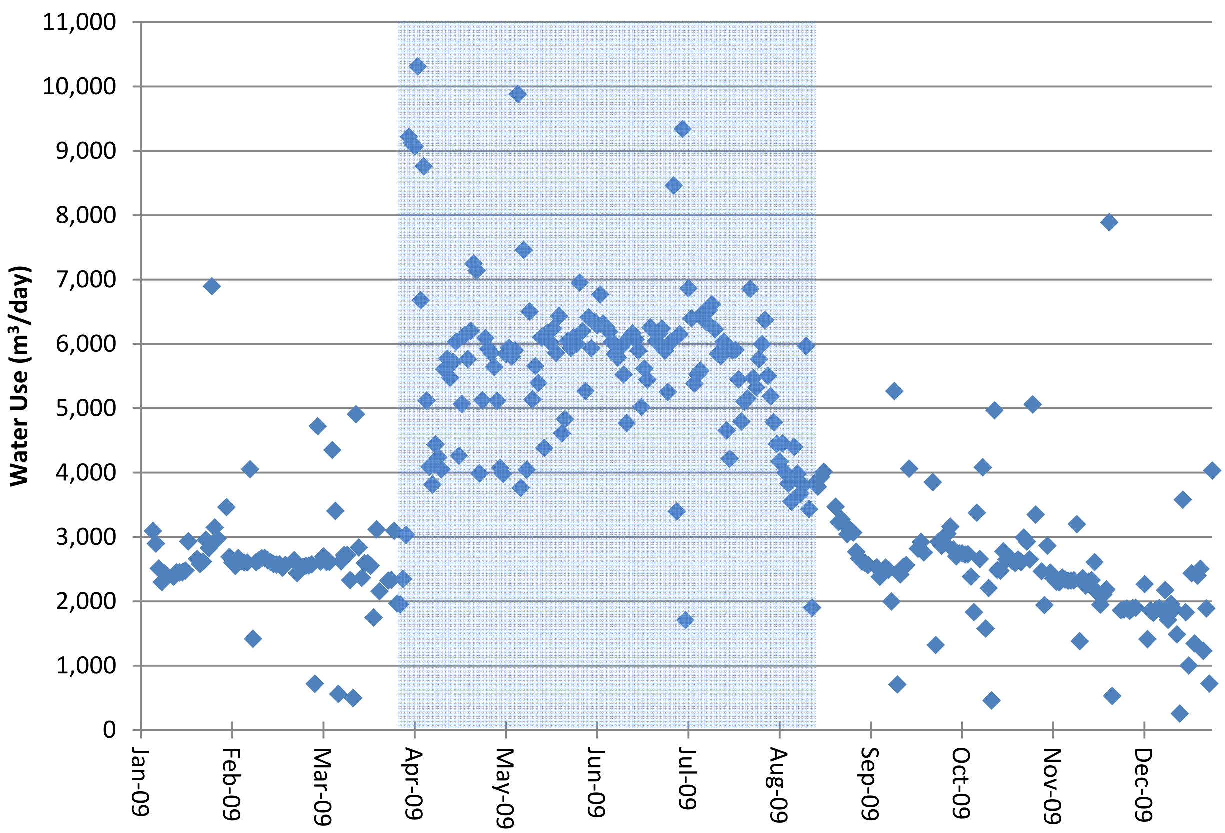


Figure 6.2 Water demand in Community A (2009)

The data presented in Figure 6.2 clearly shows the increase in total water demand that occurs when the fish plant begins operating in April. Indeed, when it is online it essentially doubles the total demand in the system. The average and maximum day water demands calculated for periods when the fish plant is or is not operating are summarized in Table 6.2.

Table 6.2 Average day, maximum day, and per capita water demands in Community A

	Units	Fish Plant Online	Fish Plant Offline	Total
Average Day	L/day	5,622,586	2,567,199	3,863,699
Maximum Day	L/day	10,314,274	7,891,725	10,314,274
Per Capita	Lpcd	1,494	682	1,026
Max Day Peaking Factor		1.5	3.1	2.7

Table 6.2 also shows the average per capita water demand when the fish plant is online, when it's offline, and overall for the year. Not surprisingly, the per capita demand is higher when the fish plant is

online. It is higher than the average provincial total per capita water demand value reported by Environment Canada (804 Lpcd). As will be discussed in later sections, this high demand can impact disinfection effectiveness and the operation of the system.

The per capita water demand during periods when the fish plant is offline is 682 Lpcd. The province currently uses a per capita demand of 340 Lpcd to calculate disinfection compliance when water use records are unavailable. The water use records in Community A suggest that, for some communities, this is likely too low.

Maximum day peaking factors were calculated for periods when the fish plant is online, when it is offline, and overall for the year. The MOE Guidelines suggest that a maximum day peaking factor of 2 be employed for communities with populations between 3,000 and 10,000 people. Overall and when the fish plant is offline, the maximum day peaking factor is higher than would be expected for the population. When the fish plant is online it is lower than would be expected. These results highlight how important it is to use historical water use records instead of assumed peaking factors when designing water treatment processes and/or water distribution system components.

No information was provided about fire flow requirements or allowances in the community. The MOE Guidelines recommend making an allowance of 7,500 L/min for 2 hours in communities with populations of approximately 4,000. This adds an additional 900,000 L/day to the total water demand of the community. This will not impact the average or MDDs but should be taken into account during the design and operation of the water system.

6.4 Assessment of Infrastructure and Fire Protection

The water demands calculated in Section 6.3 were used to evaluate the design of various water distribution system components. Design details were obtained from the system operator during the site visit and, in some cases, confirmed by consulting ENVC records (ex. Water Storage Tank Database).

Most communities in Newfoundland and Labrador rely on the transmission main between the point of chlorination and the first user for chlorine contact, making it a particularly important component of the distribution system. The characteristics of the main are summarized in Table 6.3.

Table 6.3 Characteristics of the transmission main between the point of chlorination and the first user

Item	Value/description	Units
Pipe material	**Asbestos cement**	
Pipe length	2,500	m
Pipe diameter	0.35	m
Pipe radius	0.175	m
Pipe area	0.096	m^2
Pipe volume	240	m^3

The size of the main, along with other characteristics such as pipe material, system elevations, and flow rates; will impact the pressure loss through the pipe. Topographical and system elevation information was not available but a unit rate of pressure loss was calculated for the transmission main using both the Hazen-Williams and Darcy-Weisbach methods.

A number of assumptions were made for these calculations. These have been described at length in Section 4.3 of this report. The results are summarized in tables 6.4 and 6.5.

Table 6.4 Unit rate of pressure loss calculated using Hazen-Williams

Parameter	Value	Units
C	140	
Pressure loss		
ADD	0.4	psi/km
ADD + fish plant	1.7	psi/km
ADD + fire flow	8.5	psi/km
Total	12.5	psi/km

Table 6.5 Unit rate of pressure loss calculated using Darcy-Weisbach

Parameter	Value	Units
Temperature	5	^{o}C
Kinetic viscosity	0.00000151	m^2/s
ks	0.12	mm
ks/D	0.0003	
Pressure loss		
ADD	0.4	psi/km
ADD + fish plant	1.7	psi/km
ADD + fire flow	9.1	psi/km
Total	12.8	psi/km

Both the Hazen-Williams and Darcy-Weisbach calculation methods predict large pressure losses in the central distribution under fire flow conditions; particularly when the fish plant is operating. The community may consider installing a larger water main or, preferably, a secondary main dedicated to fire protection. It should be noted that the central distribution is only one of many components that contribute to the system's ability to provide for fire flows. Other important components include distribution pumps (or secondary fire pumps), raw water transmission mains, smaller mains within the system, and water storage volumes.

The distribution system's storage capacity was assessed based on the recommendations of the AC and NL guidelines. The results are provided in Table 6.6.

Table 6.6 Evaluation of Community A's storage capacity

Parameter	Value	Units
MDD (including fish plant demand))	10,314,274	L
25% of MDD	2,578,569	L
Fire Flow	900,000	L
25% of MDD and Fire Flow	869,642	L
Total	4,348,211	
	4,348	m^3
Actual size:		
Middle of distribution system	946	
End of distribution system	2,839	
Total	3,785	m^3

The two water storage volumes in Community A provide a total volume of 3,785 m^3. This is less than the total amount of storage recommended by the AC and NL guidelines based on the MDD and required fire flow, however, it is likely that the system is generally able to operate normally during most fire events.

6.5 Disinfection

Chemical disinfection effectiveness is dependent on the amount of chemical added and the amount of contact time permitted between the disinfectant and target microorganisms. Community A relies on the transmission main between the point of chlorination and the first user for chlorine contact. Disinfection calculations are described in detail in Section 5.8 of this report.

Effective contact time and CT were calculated under average flow and peak flow conditions. The 'average flow' value represents the average daily water demand in 2009. Peak flow was calculated using the PRP-Gumbel peaking factor. CT was calculated using the chlorine residual measured at the first user during the CBCL site visit. The results are summarized in Table 6.7.

Table 6.7 CT calculated at average and peak flow in Community A

Parameter	Units	Average Flow	Peak Flow
Flow rate	L/min	2,683	9,723
Effective contact time	min	90	25
CT		89	25

The water system in Community A meets the province's disinfection requirements at average day and peak hour flows. In the latter case, the system achieves only 25 minutes of contact time and a CT of 25. The contact time and CT values calculated using actual water demands are lower than those calculated using assumed per capita water demands (Table 5.18 and Table 5.19). This highlights the importance of using actual, vs. assumed, water demands when evaluating disinfection compliance.

Note that the predicted peak flow may underestimate the absolute peak flow because it was calculated using a peaking factor developed for residential water use. Thus, if fish plant demands are particularly high there is some risk that the system will not meet disinfection requirements. This could be remedied by adding a contact volume ahead of the transmission main that would provide the required amount of chlorine contact under all flow conditions. Ideally, such a system would consist of a series of interconnected volumes that could be used or not used depending on the water demands in the distribution system (i.e., mostly used in the summer).

Variable water demands can make it difficult to maintain chlorine residuals throughout the distribution system. Chlorine residual results from the first seven months in 2009 are presented along with total water demand in Figure 6.3. Note that the town office is located close to the point of chlorination.

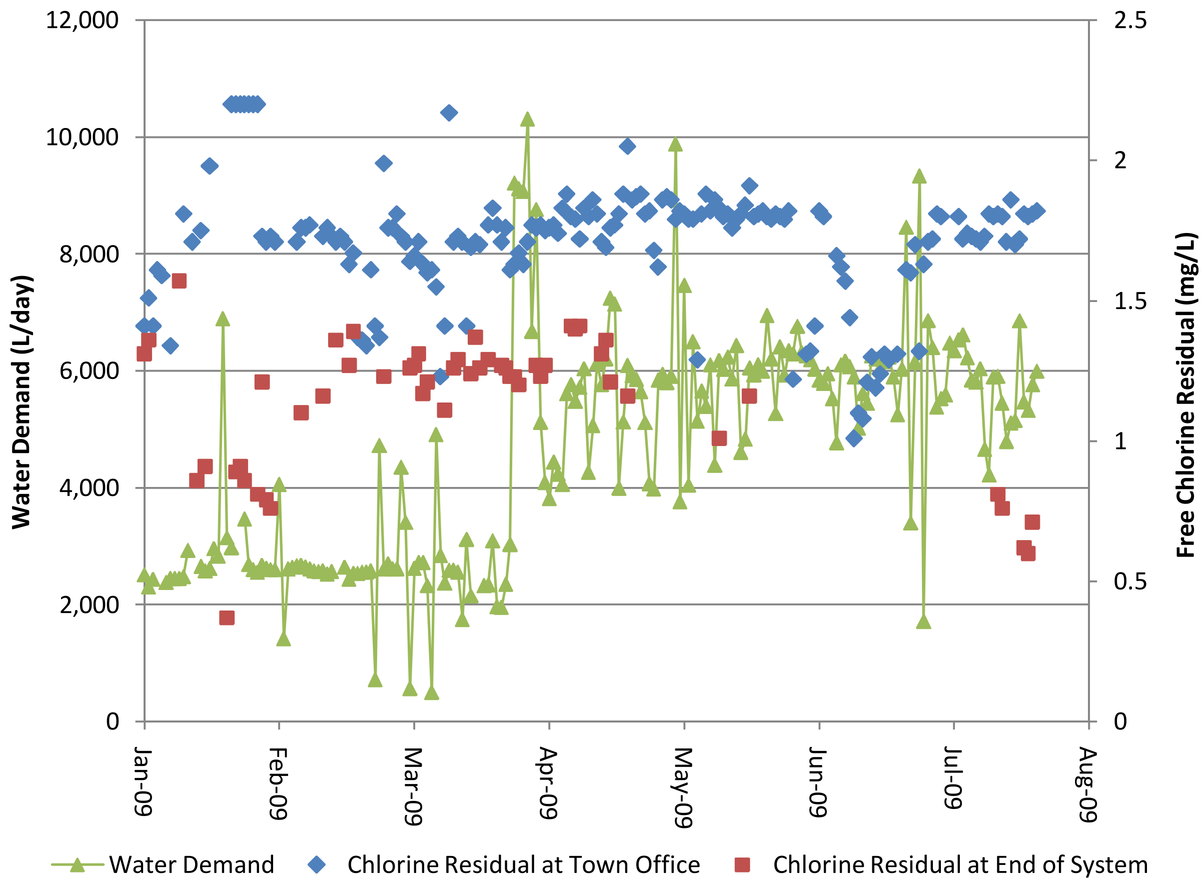


Figure 6.3 Reported chlorine residuals and water demands in Community A (2009)

For the most part, chlorine residuals were maintained at a consistent level throughout much of the year; particularly at the town office. During a follow up phone call the operator indicated that more chlorine is added when the fish plant is online. Therefore it is not surprising that the chlorine residuals do not appear to have been negatively affected by changes in water demand during the months for which data was provided.

Community A had two boil water advisories in 2009. The first occurred in late May and was related to distribution system maintenance. The second occurred in early September and had reason code 'E2', which corresponds to a lack of free chlorine residual in the distribution system. Fish plant operations were winding down or ended by this point in the year so it is unlikely that the two are connected.

6.6 Water Quality

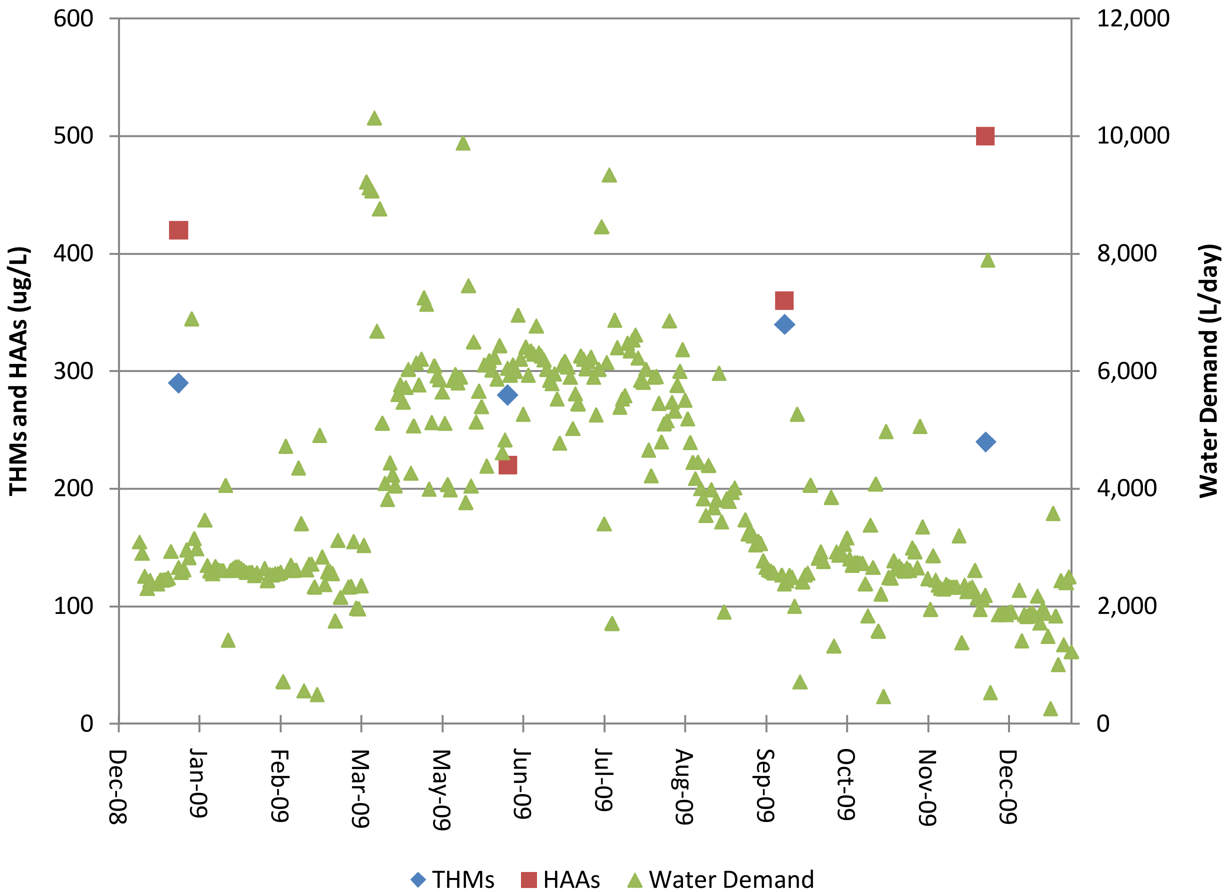


Figure 6.4 THMs and HAAs vs. water demand in Community A (2009)

Increased water age is known to contribute to the formation of DBPs such as THMs and HAAs. The ENVC measures these two parameters four times a year in Community A. The results are presented along with the measured water use for 2009 in Figure 6.4.

Both THM and HAA levels appear to vary inversely with water demand, which would be expected if DBP formation was closely related to water age in the distribution system. Unfortunately, it would be unwise to draw any definite conclusions based on only one year of water use data and four THM and HAA measurements. The community might, however, choose to look into minimizing water age in the system during periods when the fish plant is not operating to minimize the formation of THMs and HAAs. This could be accomplished by more frequent flushing.

DBP formation is also related the concentration of reactants. That is, the amount of chlorine added and the concentration of NOM (often measured as DOC) in the raw water. THM, HAA, and DOC results from Community A are provided in Figure 6.5.

Both THMs and HAAs have regularly been above the provincial recommended values of 100 µg/L and 80 µg/L, respectively. Linear regressions performed on both datasets suggest the existence of weak but significant positive relationships between DOC and both THMs and HAAs at a 95% significance level (r^2 = 0.36 and r^2 = 0.45, respectively). This suggests that if the total amount of NOM in the water was reduced through water treatment (coagulation, nanofiltration, etc.), THM and HAA levels would be likely to decrease.

The amount of chlorine available for reaction could be reduced in much of the distribution system by adding a chlorine booster station at the outlet of the second storage tank located at the end of the distribution system and subsequently lowering the chlorine dose added at the beginning of the system.

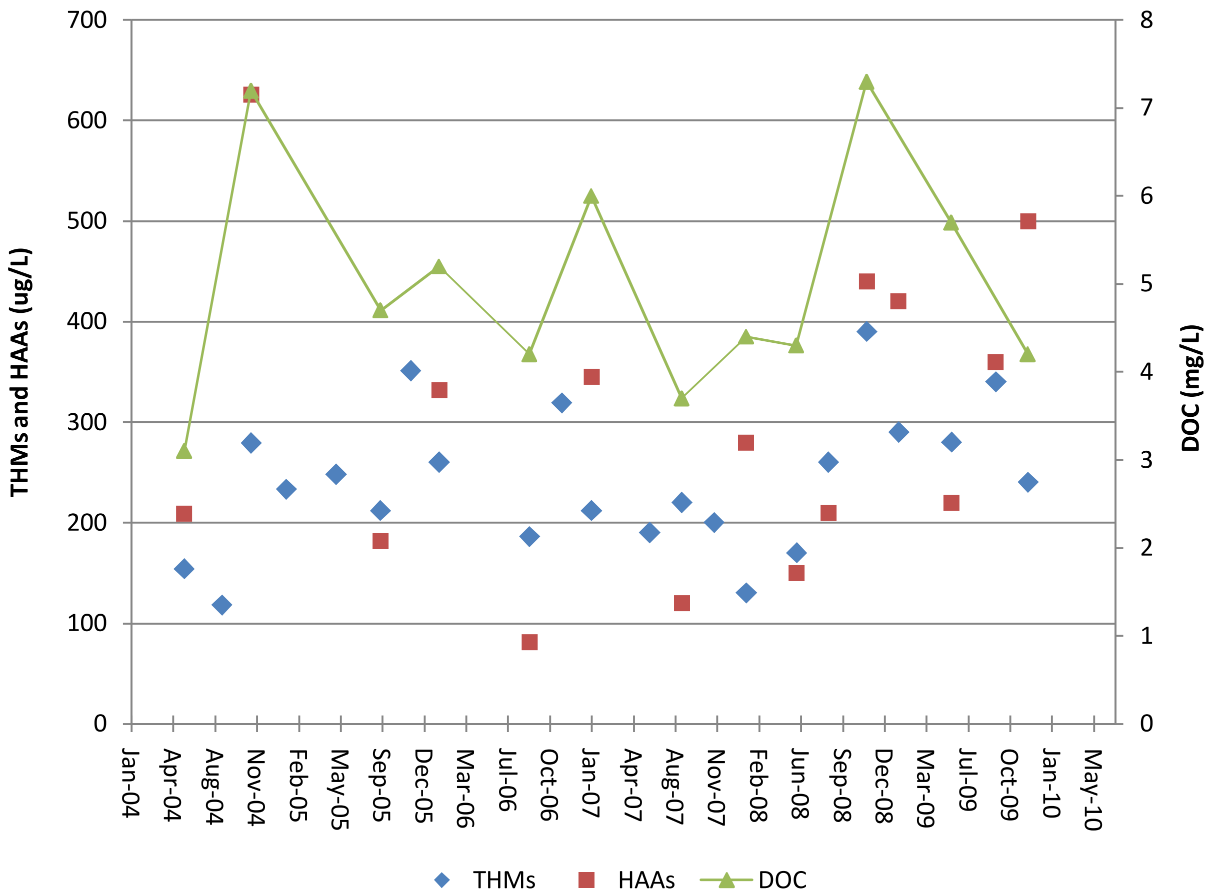


Figure 6.6 THMs, HAAs, and DOC measured in the tap water of Community A (2004-2009)

Most likely, the formation of THMs and HAAs in the distribution system in Community A is related to all three identified risk factors. Thus, addressing only one of them may not result in a large enough reduction in DBP formation to meet provincial recommendations.

6.7 Summary of Recommendations

A number of design and operational changes could be made to the water system in Community A to reduce the impacts of variable water demands on water quality, disinfection effectiveness, and system operation. Pressure loss, insufficient chlorine contact time, and other problems related to excessively high flows during periods of fish plant operation or during fire flow events could be minimized by:

- Adding a secondary main for the fish plant;
- Adding dedicated storage for the fish plant;
- Encouraging the adoption of alternative fire protection strategies such as automatic sprinklers in individual homes and businesses to minimize fire flow requirements; and
- Adding a chlorine contact volume ahead of the first user to ensure disinfection compliance when water demands are high.

The formation of THMs and HAAs due to high water age and excessive concentrations of reactants (chlorine and NOM) could be addressed by flushing the distribution system more frequently during periods of low demand, by minimizing chlorine application at the beginning of the distribution system by adding a booster station at the end of the system, and/or by removing NOM through water treatment. Adopting only one of these solutions may not be sufficient to prevent the formation of DBPs as the formation of THMs is related to all three identified risk factors.

CHAPTER 7 CASE STUDY: COMMUNITY C

7.1 Introduction

Community C is located on the south coast of Newfoundland east of Channel-Port aux Basques. The community recently constructed a new water treatment plant that uses ozone and filtration to reduce THM and HAA formation and remove pathogens. Water is transferred from a local pond to the town through a 450 mm (18-inch) ductile iron pipe. The pipe that runs between the intake and the water treatment plant is approximately 1,500 m in length. A standpipe is located at the beginning of the distribution system between the water treatment plant and the distribution system. The standpipe was constructed in 2008 and has a volume of 1,825 m^3. A rough schematic of the water supply system is provided in Figure 7.1.

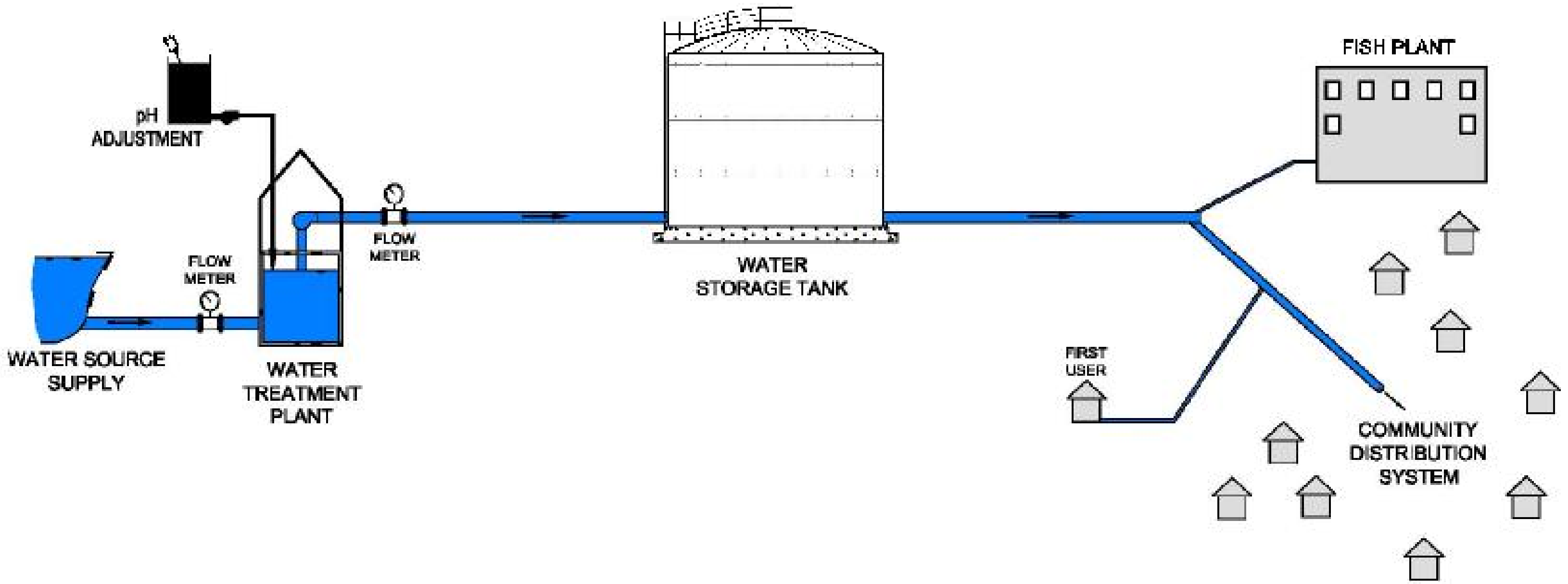
The distribution system is branched and spreads out east and west from the trunk main. The community has 48 fire hydrants and 26 dead ends. There is a water main flushing program which is carried out twice a year over a 9 day period. Most of the discharge from the flushing program is drained into the sewer system.

Water is delivered to the fish plant through a 450 mm diameter ductile iron pipe located near the beginning of the main distribution system. The fish plant's location at the beginning of the distribution system results in a large amount of water being removed prior to the majority of the residential users. Based on conversations with the system operator, this creates problems with chlorine residual maintenance throughout the system. No additional water treatment is provided prior to use at the fish processing plant.

The local fish plant has historically operated between 10 to 15 weeks per year for 12 hours per day. In 2010 operation was decreased to four weeks. No operational data was available from the fish plant for this study.

7.2 Users

According to the 2006 census, the total population of Community C is 1,607, which is a 9.8% decrease from 2001.The distribution system in Community C includes a total of 744 service connections. Residential, commercial, industrial and institutional users all pay different water rates. The type and number of each class of users are summarized in Table 7.1 along with other relevant demographic information.



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Community C Schematic

Figure
7.1

Table 7.1 Demographics – Community C

Parameter	Value	Source
Total Population	1,607	Stats Canada, 2006 Census
Serviced Population	1,607	ENVC
Population Change (2001 - 2006)	-9.8%	Stats Canada, 2006 Census
Total Users on System		
Residential	729	Reported
Industrial	1	Reported
Commercial	9	Reported
Institutional	5	Reported
Total	744	Calculated
% Residential	98%	Calculated
Population Density (per km^2)	51	Calculated
Unemployment	39.8%	Stats Canada, 2006 Census
Median Earnings	$14,095	Stats Canada, 2006 Census
Median Age	48	Stats Canada, 2006 Census

The majority (98%) of the users in Community C are residential, which suggests that water use patterns in the community should follow established residential standards (i.e., per capita water demand, diurnal water use curves, etc.).

Statistics Canada lists the total land area of Community C as 31.3 km^2, which results in a population density of 51 people per km^2; indicating a fairly large distribution system. Note that the majority of users are clustered in a smaller area of approximately 4 km^2. The population density calculated using this value is approximately 400 people per km^2, which suggests that the distribution system is actually more concentrated than would be predicted using the Statistics Canada value. In theory, a more compact distribution system should minimize some of the problems associated with water stagnation and excessive water age.

The unemployment level in Community C was approximately 40% when the last census was conducted. This is high compared to provincial and national levels, however, it is fairly standard for small rural communities in the province. Census results also indicated that the median age in the community was 48, which is also higher than provincial and national results. These demographic indicators suggest that the water use patterns in Community C are likely to differ from those found in larger communities. For example, residents may be more likely to exert residential demands throughout the day rather than during specific peak hours.

7.3 Water Demands

Flow is monitored automatically at the outlet of the existing water treatment system before the water storage tower. Daily totalizer values are recorded by the water system operator. The fish plant operator indicated that the incoming flow and total daily water volume are monitored and recorded, however, the records were not provided to CBCL. The water use data provided by the municipal operator is shown in Figure 7.2.

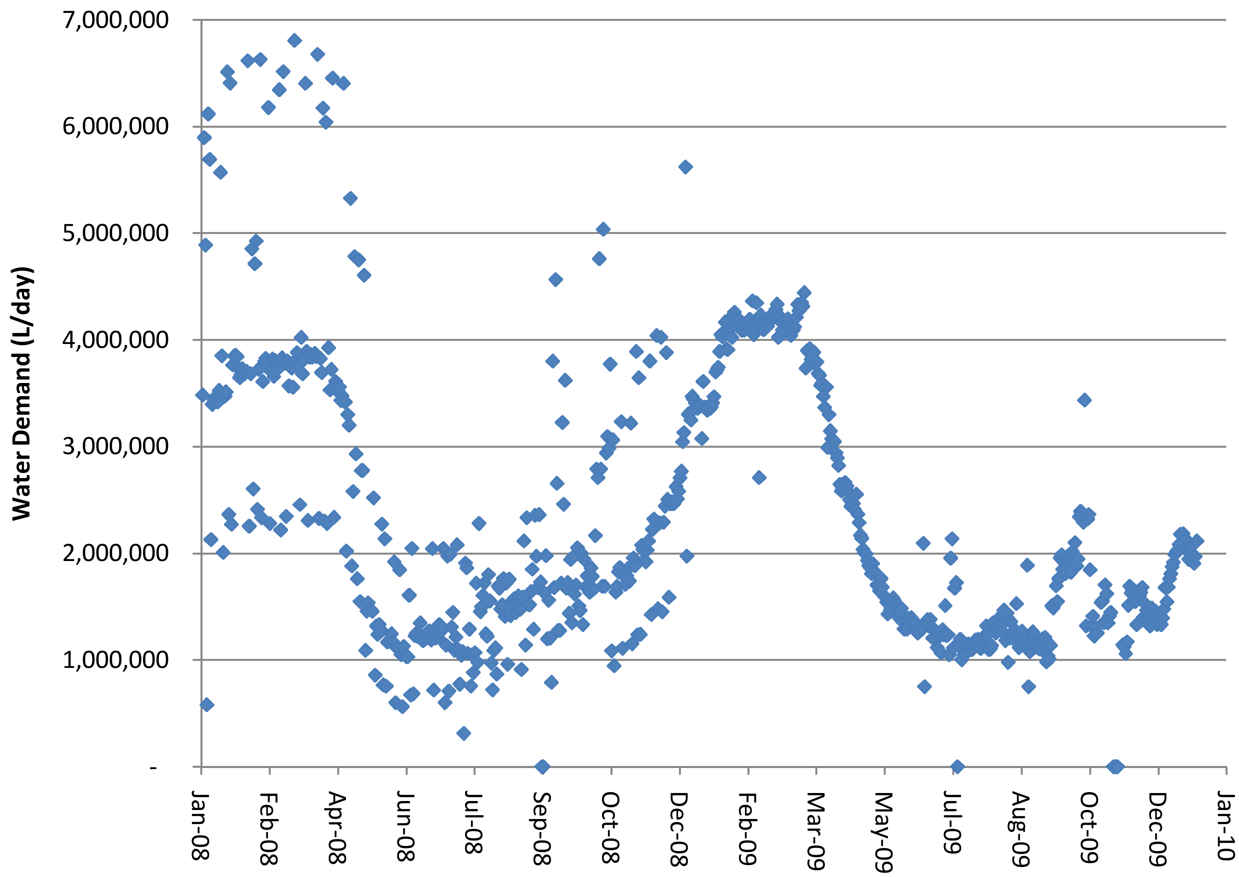


Figure 7.2 Water use measured at the outlet of the treatment system in Community C (2008 and 2009)

The values shown in Figure 7.2 do not represent actual day to day water demands. Rather, they provide a record of the amount of water that entered the tower each day. Despite this, rough water use patterns can be inferred from the data because the amount of water entering the tower each day reflects the amount of water being used in the community. Note that the flow records indicate that the flow to the tower ranges from 760 L/min (200 GPM) to 3,000 L/min (800 GPM).

Two obvious peaks exist on the graph. These periods of high water use were not related to fish plant operation but rather occurred during the winter months. Smaller peaks in the late summer and early fall correspond to periods of fish plant operation. The lowest water demands occurred in the spring and early summer.

The average, maximum day, and average per capita water demands for 2008 and 2009 are presented in Table 7.2 along with the maximum day peaking factors.

Table 7.2 Water demand values for Community C

	2008	2009	Total
Average	2,472,663	2,203,081	2,338,995
Maximum Day	6,804,209	4,441,598	6,804,209
Per Capita Demand	1,539	1,371	1,456
Maximum Day Peaking Factor	2.75	2.02	2.91

The average day water demand ranged from approximately 2,500,000 L/day in 2008 to 2,200,000 L/day in 2009. Maximum day water demands were between 2.02 and 2.75 times the ADD when assessed on an annual basis. These peaking factors correspond well to that suggested by the MOE in the MOE Guidelines for a population of this size (2.5). The calculated average per capita water use is much higher than that used by the ENVC (340 Lpcd) or those established during Environment Canada water use surveys.

As discussed in previous sections, the community's fish plant operates for only a short period of time each year and the majority of the users in Community C are residential. As a result, one would expect that standard residential water demands and water use patterns would predominate. Contrary to expectations, however, in 2008 and 2009 water use peaked in the winter and average per capita water use was approximately four times higher than would be predicted for a mostly residential community. During a follow up phone call, the town mayor noted that most of the residents are in the habit of running their taps throughout the winter to prevent their pipes from freezing.

The average water demand calculated for 2008 and 2009 for periods when the fish plant was offline (i.e., municipal demands only) was 2,535,930 L/day. The peaking factor was 2.7 and the average per capita demand was 1,578 Lpcd. The latter is higher than any of those shown in Table 7.2 and likely reflects the water use patterns described by the town mayor. It also confirms that the high water use in Community C is related to residential, rather than industrial, water demands.

Water use could be dramatically reduced by finding other ways to protect the pipes in winter. A significant reduction in winter water use might allow the community to minimize the size of some of the distribution system components, reduce water age, and use less chlorine for secondary disinfection. Tap running could be reduced by developing a public education campaign to teach residents about water conservation and/or by introducing an incentive program to help them insulate their pipes more effectively.

7.4 Assessment of Infrastructure and Fire Protection

The central distribution main in Community C is 450 mm in diameter and travels between the storage tank and the fish plant/town. The first user is located along the transmission main and the CT calculations described in later sections are only based on the distance between this user and the storage tank. Table 7.3 summarizes the characteristics of the central distribution line as described by the system operator and the town mayor. Note that different main lengths were quoted by each of the interviewees. The most conservative of these (i.e., largest) is listed in Table 7.3.

Table 7.3 Characteristics of the central distribution main in Community C

Item	Value/description	Units
Pipe material	Concrete-lined steel	
Pipe length	5,275	m
Pipe diameter	0.45	m
Pipe radius	0.175	m
Pipe area	0.096	m^2
Pipe volume	279	m^3

Pipe construction information was combined with the water demands calculated in Section 7.3 to estimate the unit rate of pressure loss (psi/km) in the central distribution main. The results obtained using the Hazen-Williams method are shown in Table 7.4 while those calculated using the Darcy-Weisbach method are provided in Table 7.5.

Table 7.4 Unit rate of pressure loss calculated for the central distribution main using the Hazen-Williams method

Item	Value/description	Units
C	140	
Pressure loss		
ADD	0.100	psi/km
ADD + fish plant	0.100	psi/km
ADD + fire flow	1.250	psi/km
Total	1.250	psi/km

Table 7.5 Unit rate of pressure loss calculated for the central distribution main using the Darcy-Weisbach method

Item	Value/description	Units
Temperature	5	^oC
Kinetic viscosity	0.00000151	m^2/s
ks	0.12	mm
ks/D	0.0003	
Pressure loss		
ADD	0.107	psi/km
ADD + fish plant	0.107	psi/km
ADD + fire flow	1.290	psi/km
Total	1.290	psi/km

The results of both sets of calculations suggest that, at current flow rates, the pressure loss in the central distribution line should not be excessive.

Table 7.6 compares the storage capacity recommended for the community based on the AC and NL guidelines to that which currently exists in the community.

Table 7.6 Evaluation of storage capacity in Community C

Parameter	Value	Units
MDD (includes fish plant)	6,804,209	L
25% of MDD	1,701,052	L
Fire flow	568,800	L
25% of MDD + fire flow	567,463	L
Total	2,837,315	L
	2,837	m^3
Actual size	1,825	m^3

The results presented in Table 7.6 show that the volume of the existing storage tower is smaller than that recommended by the AC and NL guidelines for a system with the community's measured water demands. This should not be a problem during the summer months, but in the winter (when demands are higher) the town may be hard pressed to provide enough water during an extended fire event.

Note that the water supply in Community C is dammed and provides additional storage that could theoretically be used to supply fire flow or fish plant demands.

7.5 Disinfection

Like in most communities, the operator in Community C regularly measures the free chlorine residual at designated locations throughout the distribution system. Residuals are nearly always acceptable at the beginning and middle of the distribution system but low at the end. The municipal operator and the mayor both reported that it is difficult to maintain adequate free chlorine residuals when the fish plant was online. No mention was made of difficulties during periods of high water usage in the winter. The chlorine residuals measured at the town hall by the operator in 2008 and 2009 are compared to the water use records in Figure 7.3.

The data in Figure 7.3 confirms the operator's statement about the difficulty of maintaining a free chlorine residual throughout the system when the fish plant is operating (late summer, early fall). Free chlorine levels appear to have remained normal throughout the winters. One possible explanation is that when the fish plant operates it exerts a strong water demand at the very beginning of the distribution system and what water remains after that point likely stagnates in the oversized distribution system, losing its free chlorine residual in the process. NOM levels are also highest during the summer and early fall, which may impact the rate of chlorine decay in the distribution system, particularly as the water ages.

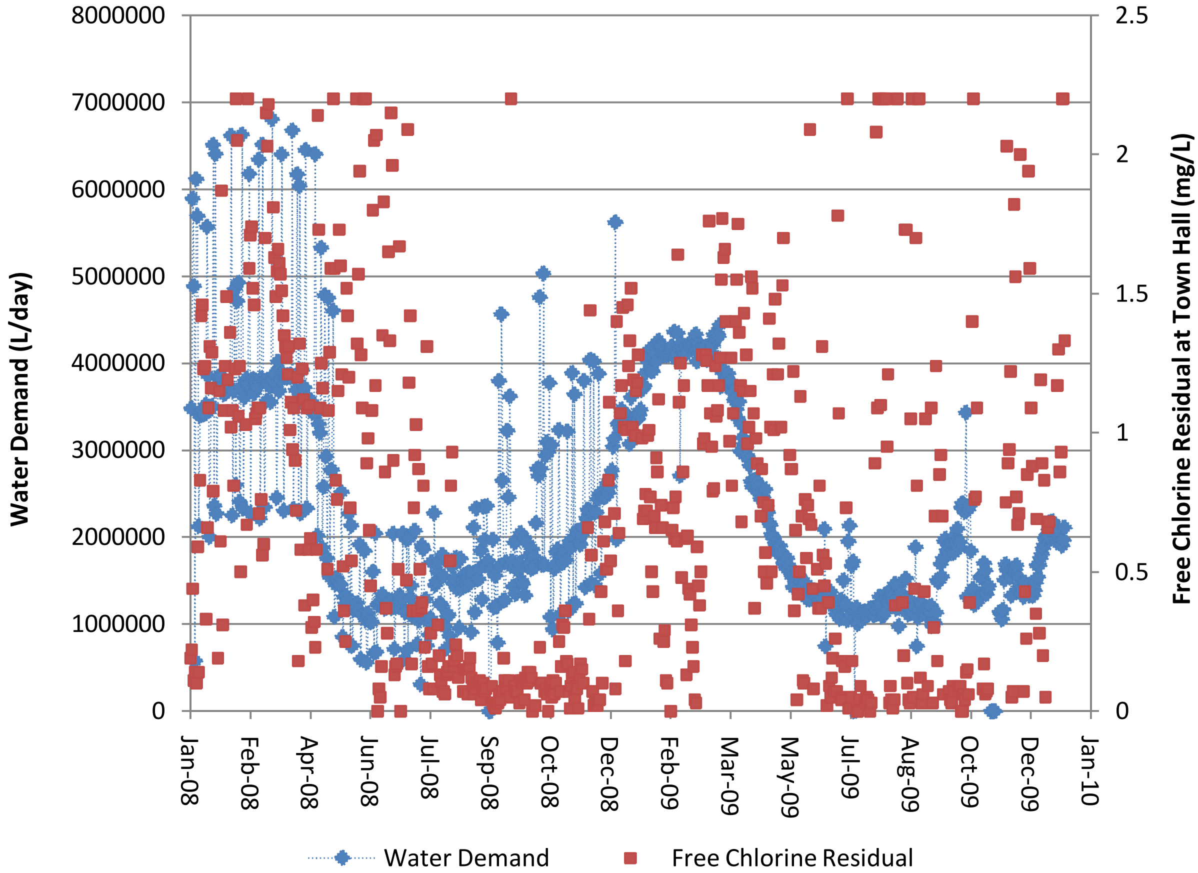


Figure 7.3 Free chlorine residual measured at the town hall (2008 and 2009)

Between 2006 and 2010 Community C experienced 5 boil water advisories (BWAs). Three of the advisories have occurred in May and the other two both occurred in September while the fish plant was operating. The community is currently on a long-term BWA related to its inability to maintain adequate chlorine residuals throughout the distribution system.

In Community C, chlorine contact occurs in the pipe that connects the treatment plant to the water storage tower, in the tower itself, as well as in the transmission main between the tower and the first user. If a baffling factor of 0.3 is assumed for the storage tower, the effective contact volume works out to approximately 570 m^3. The free chlorine residual detected at the first user during the CBCL site visit was 0.06 mg/L.

The effective chlorine contact time and CT calculated using the measured average flow (see Section 7.3) and chlorine residual are presented in Table 7.7. The disinfection compliance indicators calculated at predicted peak flow (peaking factor calculated using PRP-Gumbel method) and at an assumed free chlorine residual of 0.3 mg/L are also shown.

Table 7.7 Effective contact time and CT calculated for Community C under average day and peak flow conditions

Parameter	Units	Average Day Flow	Peak Flow
Flow rate	L/min	760	3,000
Effective contact time	min	749	187
CT (measured residual)		56	11
CT (residual = 0.3 mg/L)		225	45

The large chlorine contact time should provide enough contact time to meet provincial disinfection requirements, even at a low chlorine residual. If a larger residual is assumed, CT increases accordingly.

The new water treatment plant includes ozone and filtration. It is expected that both processes will achieve some level of pathogen inactivation. The exact amount should be verified before the community makes costly adjustments to their existing water system and/or adopts a precautionary BWA during periods of high water demand.

7.6 Water Quality

Chlorine levels, NOM quantity and quality, pH, temperature, and water age can all contribute to the rate of DBP formation. Community C's water has historically been high in colour and DOC, which are indicators of THM and HAA precursors. This is typical of many of the water supplies throughout the province. Until recently, the community did not provide any formal water treatment besides chlorination, which resulted in the formation of high levels of THMs and HAAs. Historical DOC, THM, and HAA results for the tap water in Community C are shown in Figure 7.4.

Linear regressions performed on paired THM, HAA, and DOC results did not find any correlations between the variables at a 95% confidence level. The lack of relationships may mean that DBP formation is more strongly related to water age and/or chlorine dosing strategies than to NOM levels. It is likely, however, that NOM levels do play some role and that the lack of correlations reflects the small sampling size and infrequent sampling events.

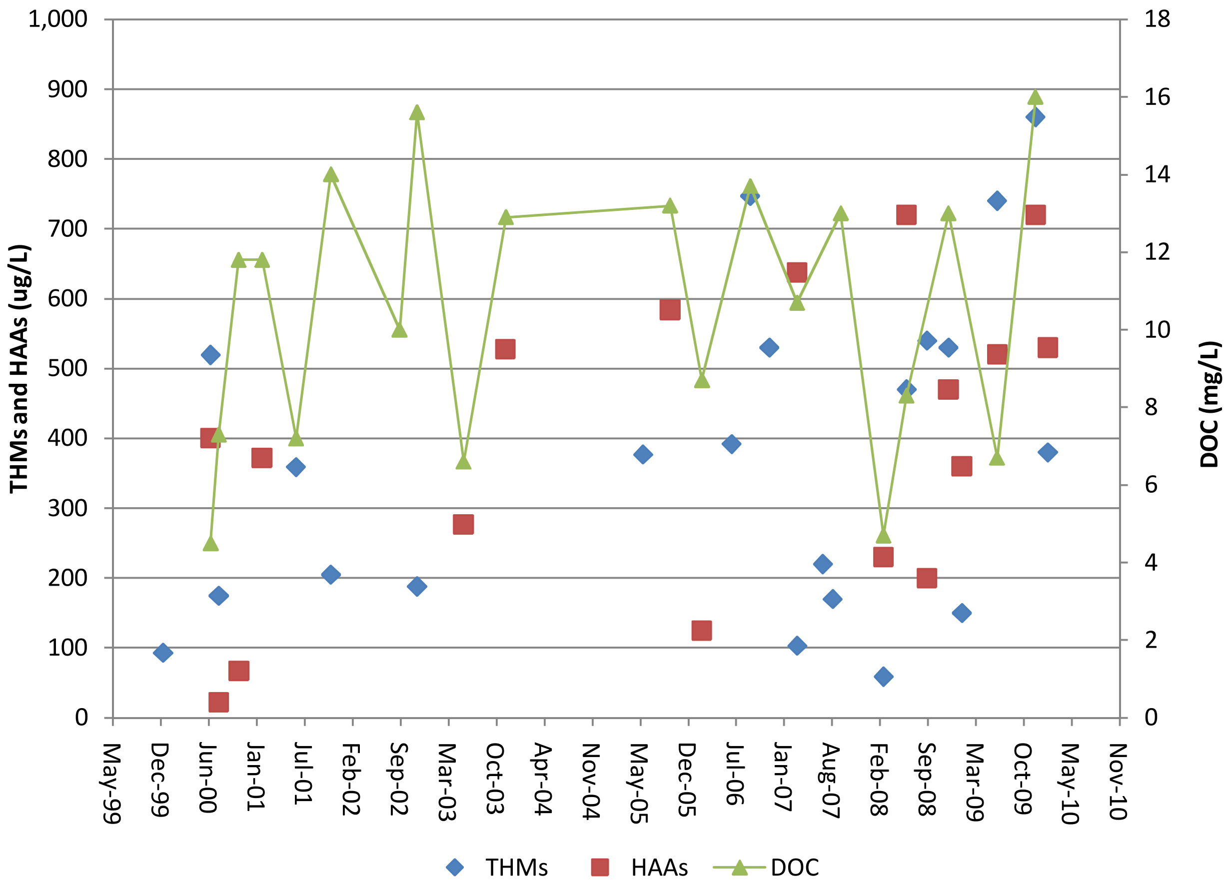


Figure 7.4 **DOC, THMs, and HAAs in the tap water in Community C (1999 to 2010)**

THMs and HAAs are compared to the total water demand in Figure 7.5. Only eight data points are available for each of the DBPs over the two year period shown in the graph, however, there does appear to be some relationship between water demand and measured DBP levels. THM and HAA levels are low in the winter when water demand is high (low water age) and elevated in the spring, summer, and early fall months when demands are lower (high water age). DBP levels were especially high in the summer and fall of 2009, corresponding to the period when the fish plant was operating, possibly confirming the hypothesis that fish plant operation results in increased water age and its attendant water quality issues in the community's water distribution system.

It should be kept in mind, however, that NOM quantity and reactivity tends to be lower in the winter and higher in the summer and fall. Though the limited available water quality records failed to indicate a correlation between DOC and DBPs, it is likely that changing levels of NOM in the water also influence the formation of THMs and HAAs.

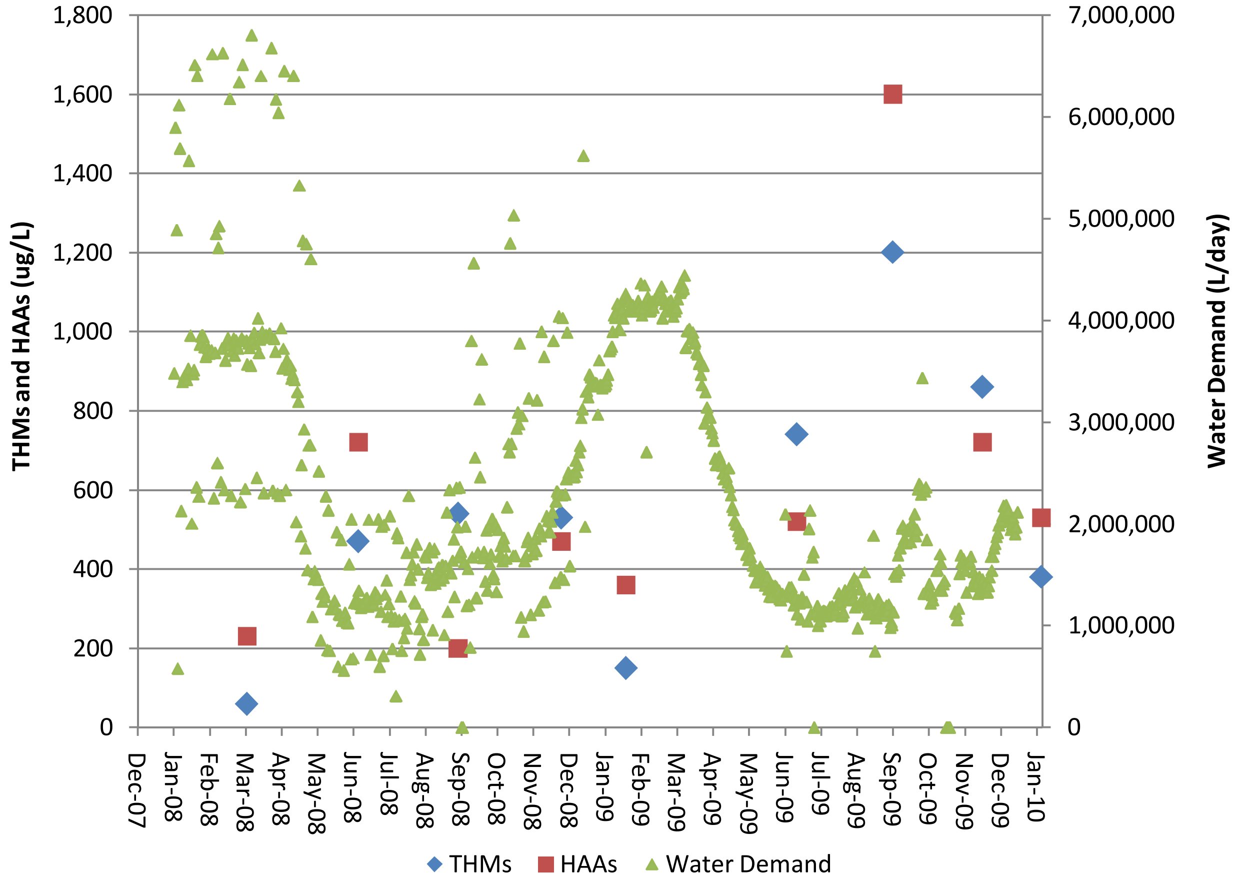


Figure 7.5 **THMs, HAAs, and water demands in Community C (2008 and 2009)**

Based on the results shown in Figure 7.5, the town may wish to consider flushing the system more frequently during periods of low flow and/or when the fish plant is operating to reduce water age within the community distribution system. Long-term it would be wise to institute water conservation practices, particularly in the winter months, with the eventual goal of reducing the size of some distribution system components to prevent water stagnation.

7.7 Summary of Recommendations

Community C has recently constructed a new WTP that includes ozone, filtration, and pH control. This process is expected to remove colour, iron, and manganese and reduce the formation of THMs and HAAs. The water use and quality records evaluated for this study predate the WTP and treated water quality data was not available at the time of this study. Some of the challenges identified in this chapter, particularly with regards to DBPs, may have since been addressed.

The average per capita water demand in Community C is approximately four times that expected for a primarily residential community in Newfoundland and Labrador. Though some of this demand can be attributed to the fish plant that operates in the community, a large proportion of it is related to the common winter practice of running water taps to prevent pipes from freezing. The community and/or provincial government should consider a public education campaign to teach residents about water conservation and/or develop an incentive program to help residents insulate the pipes in their homes. Reducing water use will reduce costs associated with treatment (i.e., energy costs for ozone).

Both the water storage standpipe and the fish plant are located at the beginning of the water distribution system. When the fish plant is operating it uses large amounts of water, which might reduce the amount of water available and/or increase water stagnation throughout the rest of the distribution system. The 12-hour operation schedule at the plant may also contribute to the system operator's difficulty maintaining adequate chlorine residual in the system when the plant is operating. To normalize demands and avoid these difficulties the community and fish plant could consider establishing a dedicated storage volume for the fish plant.

Other recommendations include:

- Distribution system flushing should be performed more frequently, particularly during periods of lower water demand and/or when the fish plant is operating; and
- New distribution system piping should be constructed using in a looped, rather than branched, configuration to reduce water age.

CHAPTER 8 CASE STUDY: COMMUNITY G

8.1 Introduction

Community G is a small community located in Notre Dame Bay. Statistics Canada lists the town's total land area as 26.7 km^2, but in reality the majority of the population is clustered close to the coast in an area of approximately 2.5 km^2.

The water system in Community G is straightforward; raw water is drawn from a nearby pond and passes through a set of rough screens. It then travels for approximately 1.5 km through a 150 mm (6") ductile iron pipe to the pumphouse, where chlorine is added. After chlorination the central main splits in two, with one branch extending southwest towards the mainland portion of the town and the other crossing the causeway to the north to service the fish plant and the island portion of the town. No additional water storage is provided in the water distribution system.

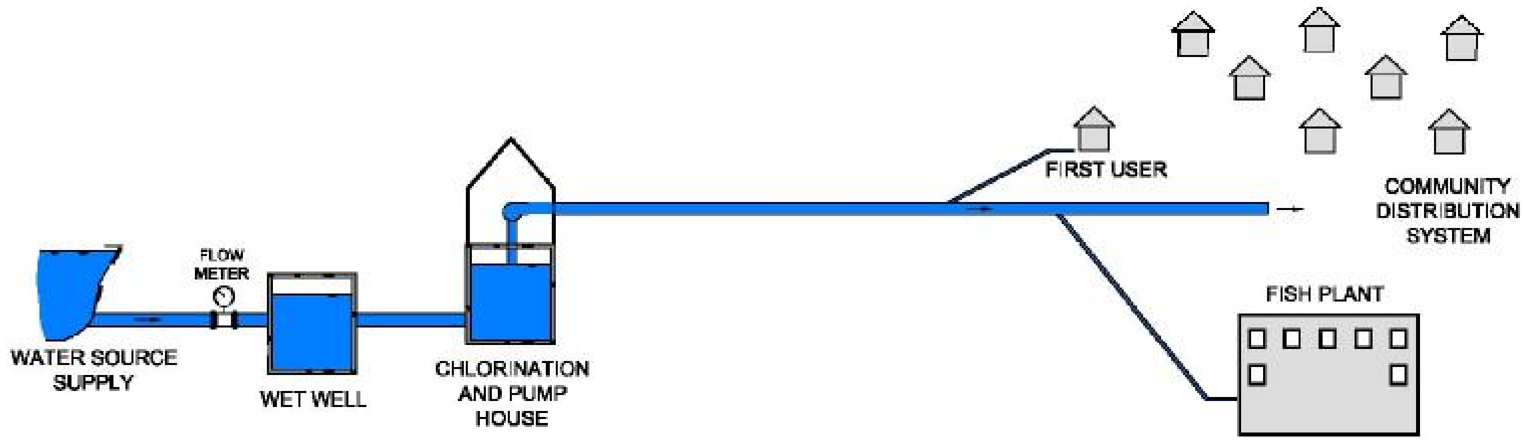
Technically, the fish plant is the first user on the system. The fish plant has not operated for the past year, however, and the operator did not provide any details about the new first user. A rough schematic of the water supply system in Community G is provided in Figure 8.1.

8.2 Users

In 2006 Community G reported 407 residents, which represented a decrease of 10.2% from 2001. There are a total of 189 residential users (households), four commercial users, three institutional users and one industrial user. The community is primarily residential (96%). Most of these users are clustered close to the coast.

Like many other communities of this size in rural Newfoundland and Labrador the residents of Community G are more likely to be employed seasonally, leading to an inflated unemployment value and low overall earnings for the year. Though many residents are employed for at least part of the year, during the periods when they are not it is likely that they use water at different times and for different tasks than users in larger centres. The per capita water demand values and water use patterns assumed by most design engineers were developed for larger communities with different demographics. They may not be appropriate to describe water use patterns in Community G.

Table 8.1 summarizes some of the information discussed in this section.



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Community G Schematic

Figure
8.1

Table 8.1 Demographics – Community G

Parameter	Value	Source
Total Population	407	Stats Canada, 2006 Census
Serviced Population	407	ENVC
Population Change (2001 - 2006)	-10.20%	Stats Canada, 2006 Census
Total Users on System		
Residential	189	Reported
Industrial	1	Reported
Commercial	4	Reported
Institutional	3	Reported
Total	197	Calculated
% Residential	96%	Calculated
Population Density (per km^2)*	15	Calculated
Unemployment	53.1%	Stats Canada, 2006 Census
Median Earnings	$11,871	Stats Canada, 2006 Census
Median Age	47	Stats Canada, 2006 Census

*Actual density is approximately 163 people/km^2

8.3 Water Demands

The water operator in Community G provided CBCL with five years of daily totalizer readings. The totalizer is located ahead of the pumphouse but gives a good estimate of daily water use because there is no significant buffering (i.e., storage) provided between the intake and the distribution system. The records were entered into a spreadsheet and used to calculate approximate average day, maximum day, and per capita water demands. These are summarized in Table 8.2.

Table 8.2 Total water demands in Community G (2006 to 2010)

	Year				
	2006	2007	2008	2009	2010
Average Day	272,570	397,371	365,048	195,941	171,429
Maximum Day	456,238	552,960	971,077	397,636	646,048
Average Per Capita	670	976	897	481	421
Maximum Day Peaking Factor	1.7	1.4	2.7	2.0	3.8

Total and per capita average day water demands have decreased over time while MDDs have varied from a high of 971,000 L/day in 2008 to 397,636 L/day in 2009. As average day water use has decreased the difference between average and MDDs has increased, resulting in a higher maximum day peaking factor. The MOE Guidelines recommend that designers use a peaking factor between 2.9 and 3.6 for a community with 400 people. The peaking factors from 2008 and 2010 straddle this range while those from other years are well below it, emphasizing the difficulty in predicting water use patterns without the use of detailed historical water use records.

Individual daily water use values are shown in Figure 8.2, which illustrates how much water use has decreased over time, particularly after the summer of 2008. Given the dramatic reduction in water demands after this point it seemed prudent to use more recent average and MDD values to evaluate pressure loss, storage capacity, and disinfection compliance, all of which are discussed in later sections.

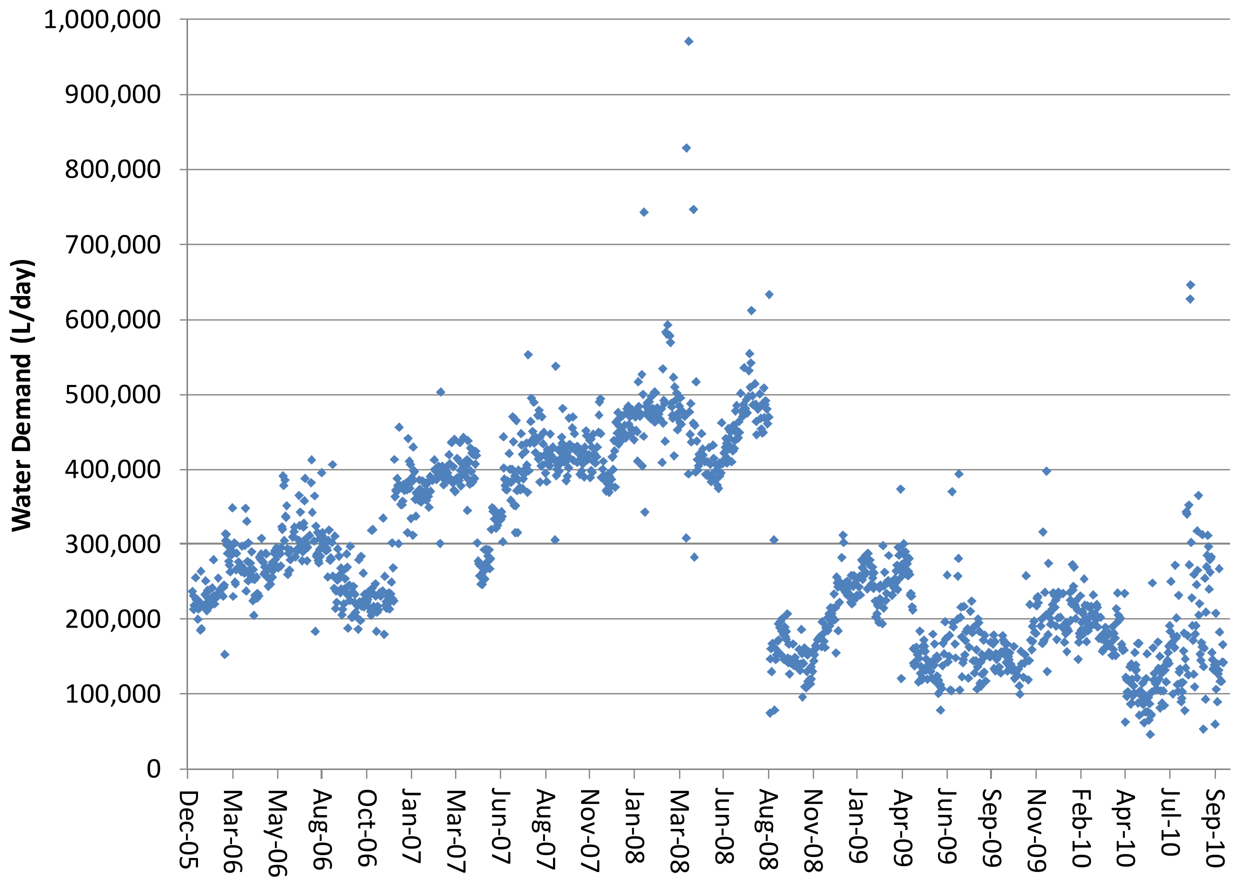


Figure 8.2 Daily water demand in Community G (2005 to 2010)

In most of the years for which data is available there are two distinct annual peaks in water demand; one in the winter and another in the summer. The exact dates of the peaks vary somewhat from year to year, but they do not appear to be related to fish plant operation. During the site visit, the operator of the fish plant reported that in 2007, 2008, and 2009 the fish plant operated from approximately March until September. This period of time corresponds to some of the peaks in total water demand, but also to some of the troughs. In fact, as shown in Table 8.3, average and maximum day water demands were lower when the fish plant was online than when it was offline, suggesting that additional factors were responsible for some or all of the peaks in water demand.

It may be that, like in Community C, the residents in Community G are running their taps during the winter months to prevent their pipes from freezing. The town may also be experiencing more frequent pipe breaks during the winter, increasing total demand. Elevated summer demands may be related to outdoor water use or to fish plant operation.

Table 8.3 **Comparison of water demands with the fish plant on and offline (2007 to 2009)**

	Year		
	2007	**2008**	**2009**
Fish Plant Online			
Average Day	388,271	400,226	185,824
Maximum Day	552,960	971,077	393,859
Average Per Capita	954	983	457
Maximum Day Peaking Factor	1.4	2.4	3.5
Fish Plant Offline			
Average Day	408,479	331,287	209,863
Maximum Day	502,986	742,892	397,636
Average Per Capita	1,004	814	516
Maximum Day Peaking Factor	1.2	2.2	1.9

CBCL was able to measure instantaneous water use in Community G over a three day period in July of 2011. The resulting diurnal water use curves are shown in Figure 8.3. Note that the blue line represents the water use measured in Community G while the red line indicates the standard water use that would be expected for the community based on the calculated average day water demand (2009) and a standard diurnal water use curve (AWWA, 2008).

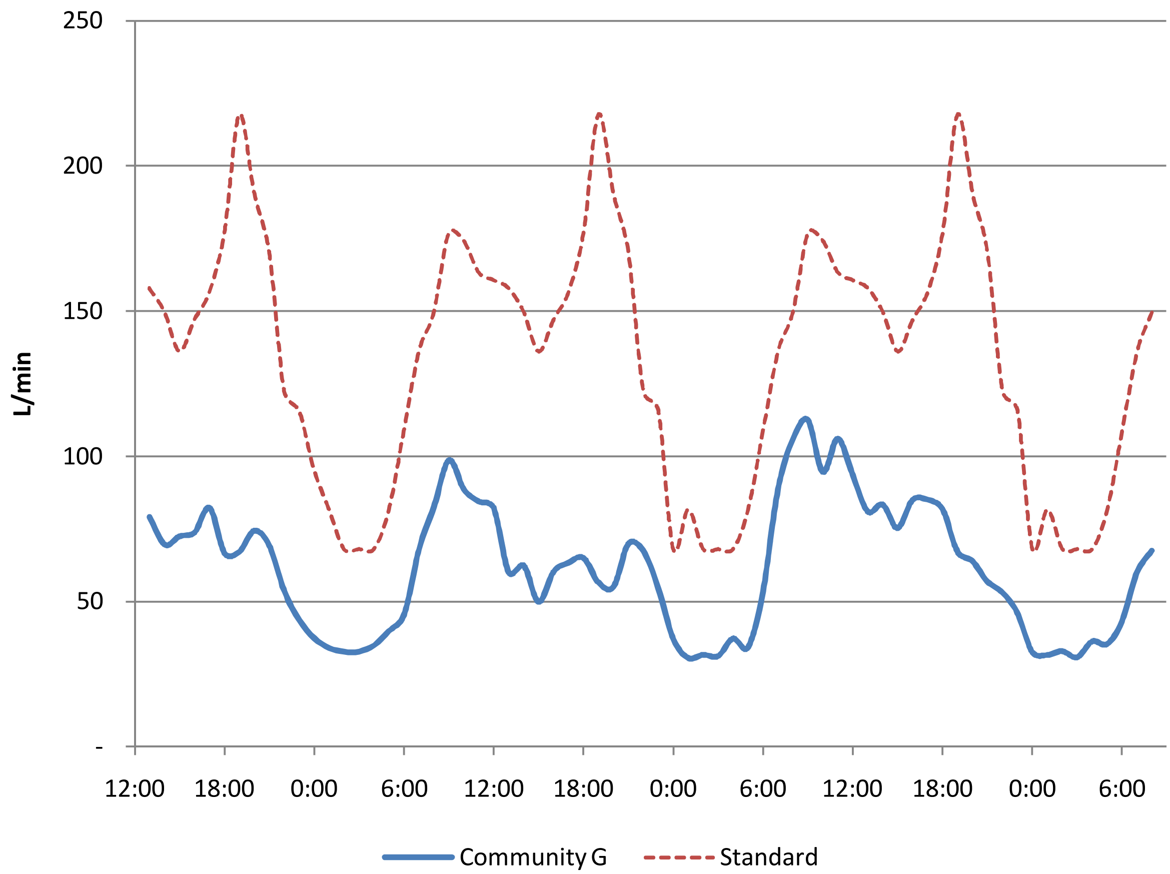


Figure 8.3 **Diurnal water use in Community G (July 13 to 16, 2011)**

The magnitude of the blue curve is less than that of the checked red curve, indicating that water use in Community G was lower than average during this second site visit. The shape of the curve is also different. Both curves have morning and evening peaks, but while the standard curve has its highest peak in the evening, the curve from Community G peaks in the morning and decreases gradually over the course of the day. A second, smaller peak is obvious in the evening but it is not as distinct as that in the standard curve.

This atypical diurnal water use curve may have been specific to the three day measuring period, but may also point to non-standard water use patterns by users in Community G. As discussed in Section 8.2, Community G has a socioeconomic profile similar to that in many other small rural communities in the Newfoundland and Labrador. Residents are older and more likely to be unemployed or seasonally employed than those in other parts of the province or Canada as a whole. Consequently, they may be more likely use water to fulfill residential needs throughout the day instead of concentrating these activities in the early morning and late afternoon / early evening. This is similar to results obtained by Rhoades (1995).

8.4 Assessment of Infrastructure and Fire Protection

The water supply and distribution system in Community G was described briefly in Section 8.1. The community's central distribution line begins at the pumphouse, which is also the point of chlorination, and continues towards the causeway that connects the island portion of the community to the mainland. Before it reaches the causeway, it splits in two, with one section continuing north and the other west. The main itself is 150 mm (6") in diameter and made of ductile iron (assumed to be cement-lined).

During the site visit the operator reported that the first user on the central distribution main is located only 300 m away from the pump house. The total volume of pipe between the point of chlorination (pump house) and the fish plant is 5.3 m^3.

Table 8.4 Characteristics of the central distribution main in Community G

Item	Value/description	Units
Pipe material	Ductile iron	
Pipe length*	300	m
Pipe diameter	0.15	m
Pipe radius	0.075	m
Pipe area	0.018	m^2
Pipe volume	5.30	m^3

*Distance between point of chlorination and first user

The MOE Guidelines recommend using a fire flow rate of 2,280 L/min over a period of two hours for communities with fewer than 1,000 people. In Community G, the instantaneous fire flow represents more than 15 times the average instantaneous flow rate (136 L/min). As a result, fire flow events are expected to have a greater impact on the operation of the system than they would in a larger community. For example, as shown in tables 8.5 and 8.6, the unit rate of pressure loss in the central distribution main is expected to increase dramatically when fire flow is added to average day water demands.

Table 8.5 Unit rate of pressure loss in the central distribution main in Community G calculated using the Hazen-Williams method

Item	Value/description	Units
C	140	
Pressure loss		
ADD	0.214	psi/km
ADD + fish plant	0.214	psi/km
ADD + fire flow	43.901	psi/km
Total	43.901	psi/km

Table 8.6 Unit rate of pressure loss in the central distribution main in Community G calculated using the Darcy-Weisbach method

Item	Value/description	Units
Temperature	5	^{o}C
Kinetic viscosity	0.00000151	m^2/s
ks	0.26	mm
ks/D	0.0017	
Pressure loss		
ADD	0.248	psi/km
ADD + fish plant	0.248	psi/km
ADD + fire flow	52.201	psi/km
Total	52.201	psi/km

The operator did not report having any difficulties with pressure maintenance during fire events, however, it appears unlikely that the system was designed to provide for fire flows.

The total fire flow that should be available for the two hour period (273,000 L) is approximately 40% greater than the ADD from 2009 (196,000 L/day). The community currently lacks a storage tank, but should they choose to build on in the future they should take the additional volume required for fire protection into consideration when sizing it. The total volume of water storage recommended by the AC and NL guidelines is provided in Table 8.7.

Table 8.7 Evaluation of water storage capacity in Community G

Parameter	Value	Units
MDD (includes fish plant)	397,636	L
25% of MDD	99,409	L
Fire flow	273,600	L
25% of MDD + fire flow	93,252	L
Total	466,261	L
	466	m^3
Actual size	0	m^3

Community G should consider installing separate storage and/or mains for demand management and fire protection. They could also enlarge the transmission main to minimize pressure concerns associated with fire flows, however, this would likely result in increased water age and associated water quality problems.

8.5 Disinfection

Table 8.8 provides a summary of the results of contact time and CT calculations conducted for the water system in Community G.

Table 8.8 Effective contact time and CT calculated using measured free chlorine residual and average and peak day flows

Parameter	Units	Average Day Flow*	Peak Flow
Flow rate	L/min	136	805
Effective contact time	min	39.0	6.6
CT (measured residual)		27.3	4.6

The total effective volume available for chlorine contact between the point of chlorination (pump house) and the first user is 5.3 m^3. At the 2009 average day flow (136 L/min), this results in a total chlorine contact time of 39 minutes. At peak flow (calculated using a peak hour peaking factor of 5.9 as predicted using the PRP-Gumbel method), the system achieves only 6.6 minutes of chlorine contact and is therefore out of compliance with provincial disinfection requirements.

During the site visit, CBCL staff measured the free chlorine residual at the first user in Community G. This value (0.7 mg/L) was used to calculate the CT achieved by the water system. At average day flow, the system achieves a CT of 27.3, which is above provincial requirements. At peak flow it only achieves a CT of 4.6.

The most effective way to increase chlorine contact time would be to construct a dedicated chlorine contact volume. This should include baffling to encourage effective mixing and minimize short-circuiting. The community could add filtration and/or UV to improve disinfection and reduce reliance on chlorine. Note that UV disinfection does not work correctly unless the water has a transmittance above 75%. If the transmittance of the water is below this, UV should only be installed along with turbidity and/or colour removal treatment processes.

The operator in Community G also provided CBCL with chlorine residual monitoring records. These are presented in Figure 8.4.

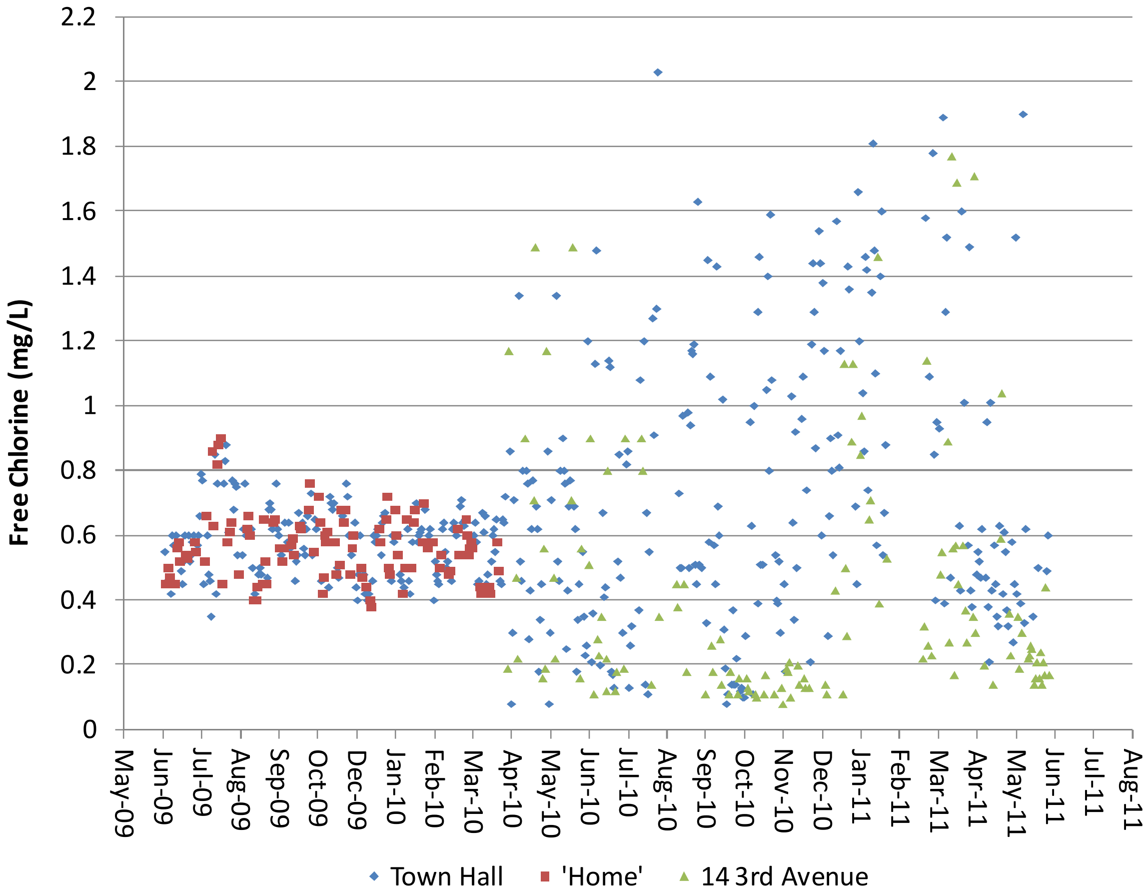


Figure 8.4 Free chlorine residuals measured by Community G system operators (2009 to 2011)

Before April of 2010 free and total chlorine were measured at the town hall and at the operator's home. In May of 2010 a new operator began working for the town. He now samples at the town hall and 14 3rd Avenue. The records show that the free chlorine residual is always above 0.1 mg/L and occasionally climbs to over 2 mg/L. The residual tends to be higher at the town hall than at 14 3rd Avenue.

Community G had two boil water advisories in 2009. Both occurred while the fish plant was operating and were called when total coliforms or *E.Coli* were detected in the distribution system (code = F2). Neither was associated with abnormally high or low total daily water demand.

8.6 Water Quality

Like many other communities in Newfoundland and Labrador, Community G relies on a raw water source that is high in NOM. When the NOM comes in contact with chlorine it forms THMs and HAAs. The ENVC measures total THMs and HAAs in Community A between two and four times a year. THM levels have frequently been measured at levels above the 100 µg/L limit recommended by the provincial government, while HAAs have always been above the recommended limit of 80 µg/L.

Numerous factors, including NOM level and type, disinfection strategy, chlorine dose, water age, temperature, and pH, can impact the rate of THM and HAA formation. THMs and HAAs are graphed against total water demand and DOC concentration in Figures 8.5 and 8.6, respectively.

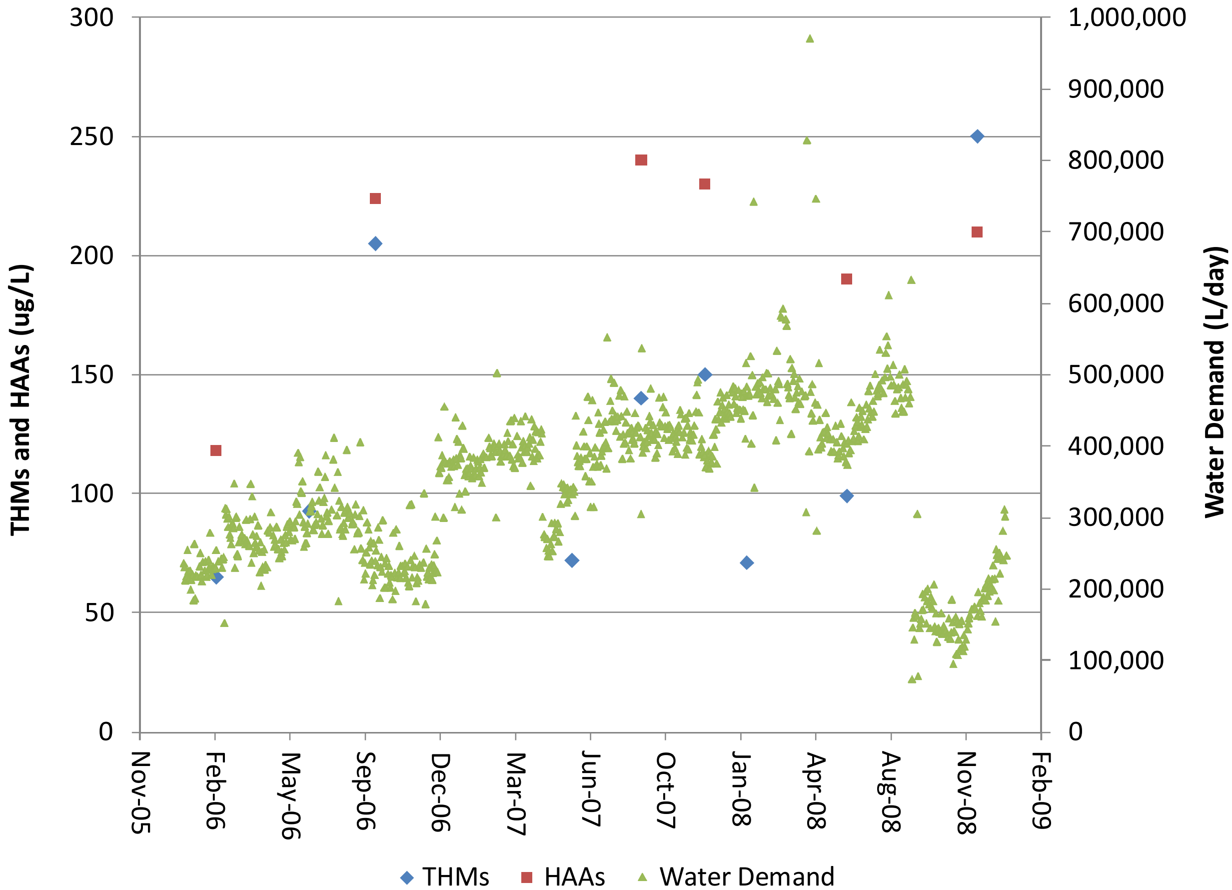


Figure 8.5 THMs and HAAs vs. water demand in Community G (2005 to 2009)

In general, the distribution system in Community G would be less likely to experience high water age because it consists of small pipes and lacks storage capacity. Thus, it is not surprising that neither THMs nor HAAs appear to be connected to the average daily water demand. This does not mean that they are totally unrelated to water age, however, it seems likely that this is not the only driver for DBP formation.

No relationship appears to exist between DOC and DBP formation either (Figure 8.6). This is surprising, however, there are very few data points available for analysis and those that are available are not always paired.

The lack of any clear relationship between DBP formation and water age or DOC suggests that DBP formation in the Community G distribution system is influenced by numerous factors. This makes it difficult to establish any one strategy for minimizing DBP formation. The most effective would be removal of DBP precursors (NOM) combined with careful optimization of the disinfection process. Unfortunately, this remedy is likely to be costly to design, build, and operate.

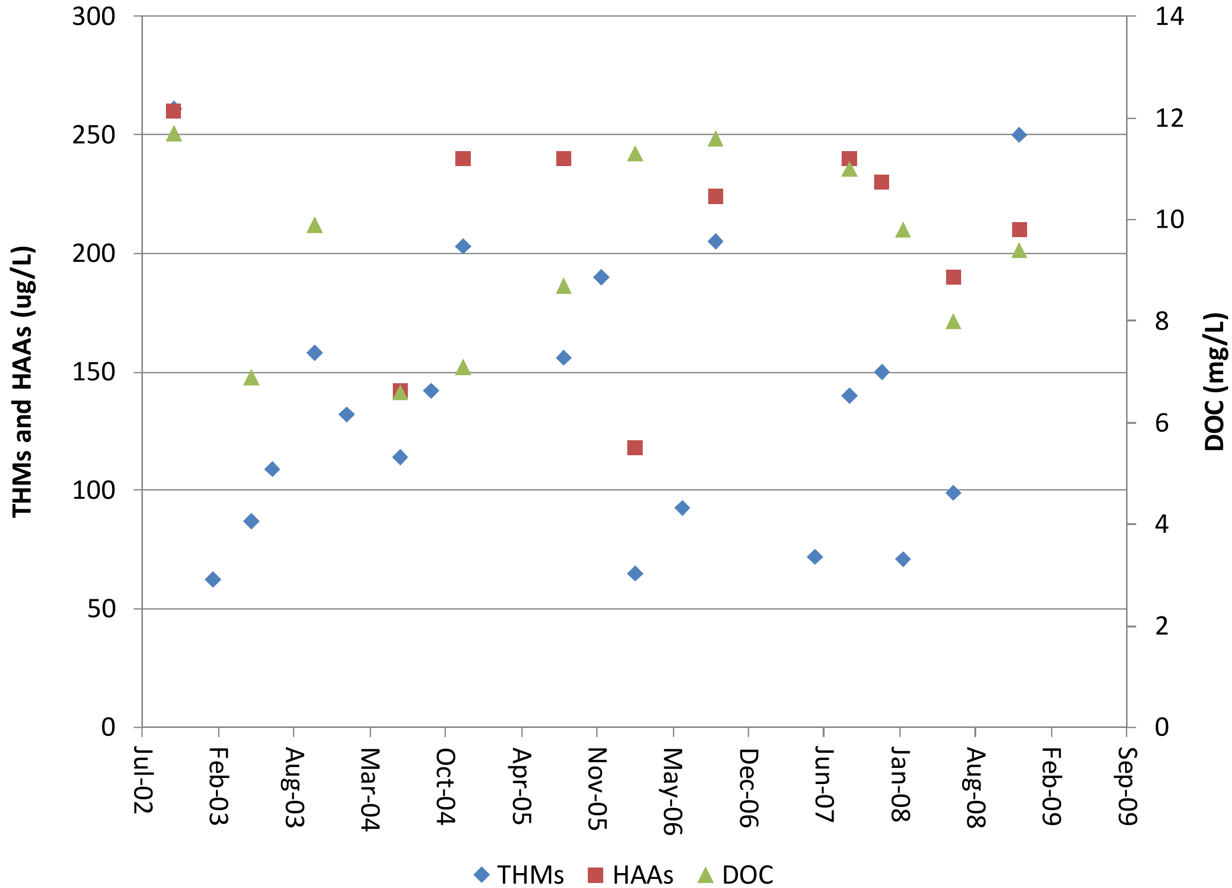


Figure 8.6 **THMs and HAAs vs. DOC in Community G (2002 to 2009)**

8.7 Summary of Recommendations

Community G has recently been approved for funding to build a potable water dispensing unit. This small-scale system will provide residents with clean and safe water to fulfill consumption needs such as drinking, cooking, and tooth brushing. Once this system has been installed there will be less focus on water quality maintenance in the full-scale distribution system (i.e., minimize water age). At that point it may be more feasible to construct a large water storage tank to buffer variations in water demands over the course of the day and provide fire protection. The community should also consider installing larger or, preferably, secondary mains for fire protection. This will minimize pressure loss problems during fire events. If this option is determined to be too costly, the community may choose to explore alternative fire protection strategies.

CHAPTER 9 CASE STUDY: COMMUNITY I

9.1 Introduction

Community I's water supply consists of a pond that feeds into a dammed reservoir. The reservoir has a volume of 34,070 m^3 (9,000,000 Usgal). Water travels from the reservoir to a screen house and then towards the chlorination building through a 350-mm diameter, cement-lined ductile iron pipe. At this point, the flow is split into two separate pipes, both of which are monitored by flow totalizers. The water is then chlorinated with chlorine gas prior to travelling through the distribution system. A rough schematic of the water system is provided in Figure 9.1.

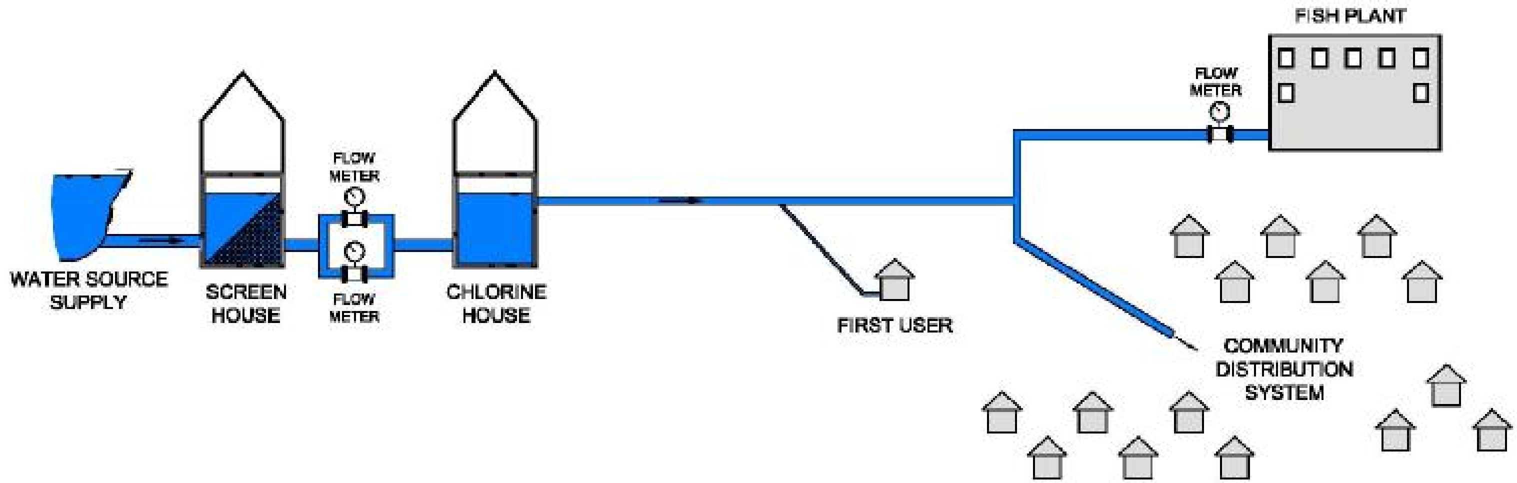
The town's fish plant is connected to the distribution system through a 250-mm diameter ductile iron pipe that extends approximately 1,000 meters from the central distribution main leaving the chlorination building via a tee. During the site visit the municipal operator indicated that in recent years the fish plant has operated for approximately 6 to 8 months per year for 8 hours a day. Water use records suggest that plant operation and/or water use actually continued throughout the year in 2008 and 2010. In the summer of 2011 the fish plant was taken offline because of a dispute between various stakeholders.

The town's new WTP was under construction when CBCL staff visited the community. The plant will consist of microfiltration with coagulation pre-treatment and disinfection and is expected to come online in 2012. A 2,000 m^3 storage tank will be built along with the new WTP.

9.2 Users

Community I is the largest community that participated in this study. It has a population of 5,436 based on the 2006 census and is located on the Burin Peninsula. The population decreased by 8% between 2001 and 2006. These and other relevant demographic indicators are summarized in Table 9.1.

Residential users account for 85% of all system users. This is a smaller percentage than that found in many of the smaller communities. There are two industrial users; the local fish plant and a shipyard. Payments for water usage vary for different users, some users pay annual lump sum, there are different rates for specific users and some users are covered in their taxes. The fish plant pays an annual rate based on the amount of water used.



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Community I Schematic

Figure
9.1

Table 9.1 Demographics – Community I

Parameter	Value	Source
Total Population	5,436	Stats Canada, 2006 Census
Serviced Population	5,436	ENVC
Population Change (2001 - 2006)	-8.0%	Stats Canada, 2006 Census
Total Users on System		
Residential	2,403	Reported
Industrial	2	Reported
Commercial	420	Reported
Institutional	3	Reported
Total	2828	Calculated
% Residential	85%	Calculated
Population Density (per km^2)	88	Calculated
Unemployment	24.0%	Stats Canada, 2006 Census
Median Earnings	$18,802	Stats Canada, 2006 Census
Median Age	40	Stats Canada, 2006 Census

The median age and unemployment level in Community I are lower than those found in many of the other communities that participated in the study. This reflects the greater diversity of educational and commercial opportunities found in the town, which acts as a hub for many of the smaller communities on the Burin Peninsula.

The demographics of Community I suggest two opposing important drivers for water demand and water use patterns. The lower unemployment and higher median earnings enjoyed by residents compared to smaller communities in the province increases the likelihood that most employees follow a standard 9 to 5 schedule. This would tend to result in a standard diurnal water demand curve with peaks early in the morning and late in the afternoon. The (relatively) low percentage of residential users might negate this, however, and may also result in a higher total per capita water demand.

9.3 Water Demands

The town clerk in Community I was able to provide CBCL with approximately two and a half years of water use records. Though total water use is monitored continuously in the chlorination building, the town's water system operator only records water use once a month. Thus, daily water demands were calculated by dividing the monthly totals by the number of days between readings. This inevitably led to some inaccuracy, particularly with regards to the MDD. The approximate water demands exerted by the community, the fish plant, and in total are summarized in Table 9.2.

Table 9.2 Average day, maximum day, and average per capita water demands in Community I

	Units	Municipal	Fish Plant	Total
ADD	L/day	5,952,559	757,163	6,709,722
MDD	L/day	8,404,583	1,770,956	9,027,790
Peaking Factor		1.4	2.3	1.3
Per Capita	Lpcd	1,095	139	1,234

During the period for which water use records are available the average total per capita water demand in Community I was 1,095 Lpcd without the fish plant and 1,234 Lpcd with the fish plant demand included. This is well above the average residential and total water demands reported by Environment Canada (395 Lpcd and 804 Lpcd, respectively). The high municipal demand may indicate that residents are using larger than average volumes for domestic tasks (running taps, gardening, etc.) or that the distribution system is experiencing significant leakage. Water conservation initiatives and metering may help to minimize residential water demand. Leak detection efforts should also be stepped up to minimize water loss, which represents wasted energy and chemicals in addition to compromised distribution system components.

The individual average day water demand values calculated from the monthly totals are presented in Figure 9.2.

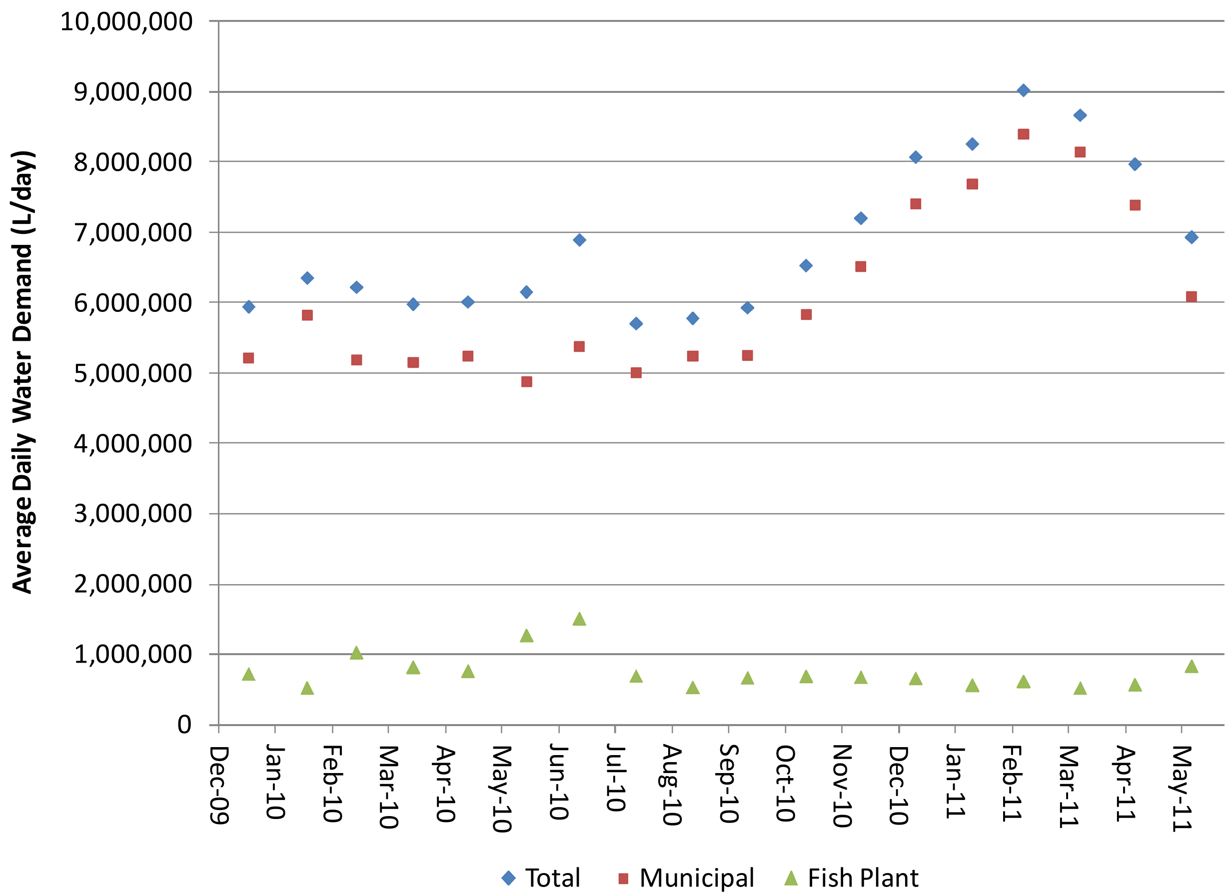

Figure 9.2 Average day municipal, fish plant, and total water demands in Community I (2009 to 2011)]

During one of the site visits the water system operator indicated that the uptick in water demand in early 2011 was related to a leak in the northern part of the town. If the results from 2011 are ignored, the calculated average day water demand drops from approximately 6,700,000 L/day to 6,500,000. The average total per capita water demand drops from 1,234 Lpcd to 1,190 Lpcd. This is still well in excess of the total and residential per capita water demand values provided by Environment Canada.

Though Community I was visited by CBCL in the fall of 2010, a second site visit was conducted in June of 2011 to measure water use over the course of a day. Initial communications with system operators suggested that it would be possible to connect a temporary flow meter to the central distribution main and leave it for a week to collect instantaneous flow data. Upon arrival it was determined that the outside of the pipe was too corroded to obtain an accurate measurement and that the pipe was located in an unsafe location. Instead, CBCL staff recorded the total and instantaneous water use approximately every fifteen minutes from 12 pm to 9 pm on June 22nd and from 6 am to 8 am. The results are shown in Figure 9.3 alongside the standard diurnal water demand curve that would be expected for Community I based on the calculated average day water demand in Table 9.2.

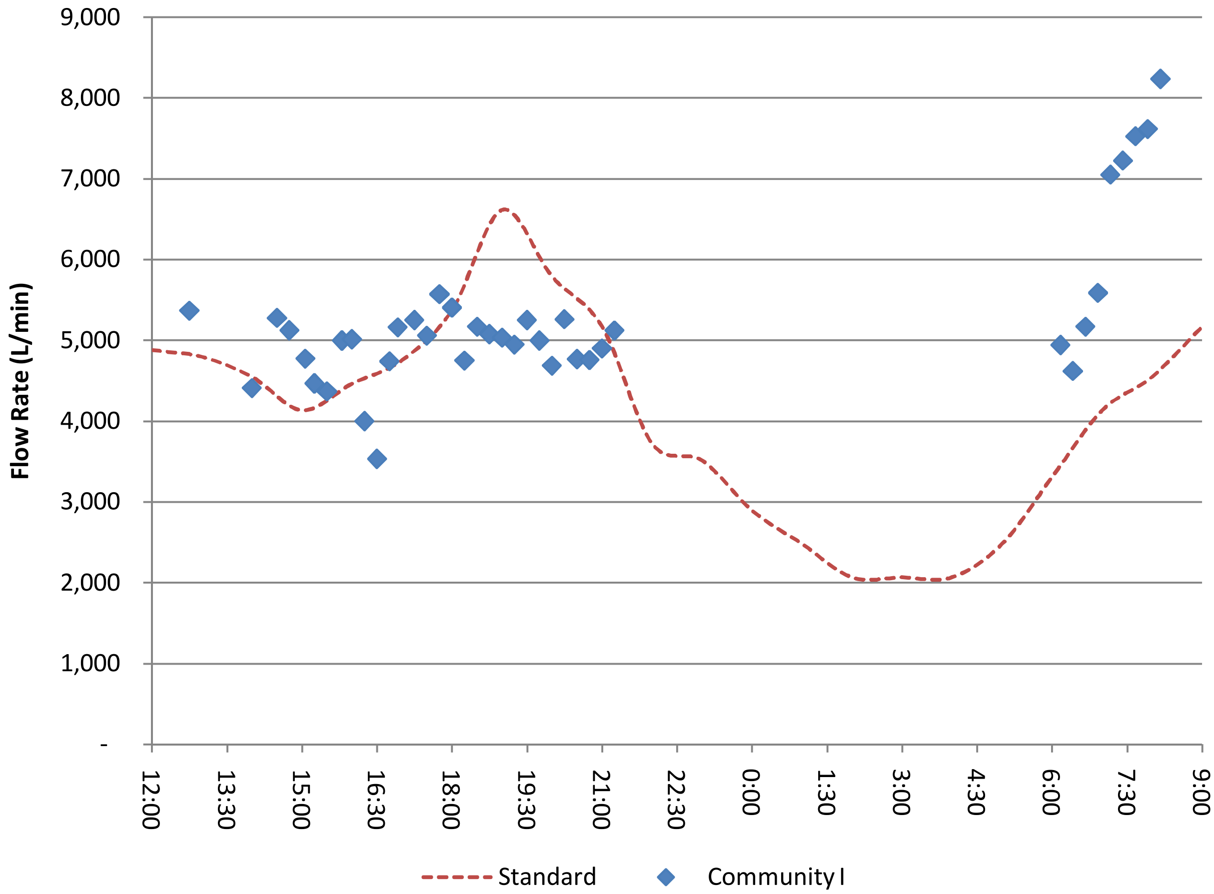

Figure 9.3 **Results of water use monitoring in Community I compared to a standard diurnal water demand curve**

Despite the small number of data points available, the results in Figure 9.3 suggest that water use in Community I did not follow a standard diurnal water use curve on the day of the site visit. This may have been a single occurrence, but it may also hint at underlying water use patterns in the community and the impact of fish plant demand.

In most communities and subdivisions, water use peaks in the early evening; coinciding with the time when most residents return home from work or school. This peak was not apparent in Community I on the day of the site visit. This may be because the fish plant exerted water demands throughout the afternoon and masked the expected drop in demand at this time. It may also be that most residents in Community I shower in the morning rather than in the evening. The more dramatic peak that is suggested on the morning of June 23rd may confirm the latter and/or may be related to high water demands from the fish plant as it begins to operate.

9.4 Assessment of Infrastructure and Fire Protection

During the site visit the system operator reported that the central distribution main that carries water from the chlorination building to users has a diameter of 350 mm and is constructed of cement-lined ductile iron. He also reported that the first user is located 1,500 m from the point of chlorination. This results in a pipe volume of 144 m^3 between the point of chlorination and the first user, as shown in Table 9.3.

Table 9.3 Characteristics of the central distribution main in Community I

Item	Value/description	Units
Pipe material	Cement-lined ductile iron	
Pipe length*	1,500	m
Pipe diameter	0.35	m
Pipe radius	0.175	m
Pipe area	0.096	m^2
Pipe volume	144	m^3

Note that the central distribution main splits in two directions a short distance from the point of chlorination. One portion services the main part of the town while the other travels north towards the fish plant, where it also serves a handful of residences.

The MOE Guidelines recommend that communities with 5,000 to 6,000 residents should have the capacity to provide between 8,640 L/min and 9,540 L/min of fire flow for a minimum of two hours. Community I has approximately 5,500 residents so a fire flow of 9,000 L/min has been adopted for the purposes of this study.

The unit rate of pressure loss in the central distribution main was calculated using the Hazen-Williams and Darcy-Weisbach methods under ADDs with and without the fish plant and with fire flows. The results are summarized in Tables 9.4 and 9.5.

Table 9.4 Unit rate of pressure loss in the central distribution calculated using the Hazen-Williams method

Item	Value/description	Units
C	140	
Pressure loss		
ADD	1.914	psi/km
ADD + fish plant	2.388	psi/km
ADD + fire flow	16.450	psi/km
Total	17.681	psi/km

Table 9.5 Unit rate of pressure loss in the central distribution calculated using the Darcy-Weisbach method

Item	Value/description	Units
Temperature	5	^oC
Kinetic viscosity	0.00000151	m^2/s
ks	0.12	mm
ks/D	0.0003	
Pressure loss		
ADD	1.997	psi/km
ADD + fish plant	2.470	psi/km
ADD + fire flow	18.370	psi/km
Total	19.742	psi/km

The results in Tables 9.4 and 9.5 suggest that when fire flows are added to the total flow in the central distribution main significant pressure losses are likely (under the stated assumptions). This could be remedied by increasing the pipe diameter (i.e., installing a larger pipe) or by adding a secondary main for fire protection.

Table 9.6 presents the results of an assessment of the water storage capacity available in Community I.

Table 9.6 Assessment of storage capacity in Community I

Parameter	Value	Units
MDD (includes fish plant)	9,027,790	L
25% of MDD	2,256,948	L
Fire flow	1,090,800	L
25% of MDD + fire flow	836,937	L
Total	4,184,684	L
	4,185	m^3
Raw water reservoir	34,605	m^3
New storage tank	2,050	m^3

A water storage tank designed based on the recommendations provided in the AC and NL guidelines would have a volume of approximately 4,185 m^3. The raw water reservoir used as the town's water supply provides approximately nine times this amount of storage. The storage tank that is being constructed concurrently with the new WTP will have a volume of 2,050 m^3, which is only half of that recommended by the AC and NL guidelines. The town may wish to consider installing additional water storage to ensure that sufficient treated water is available at all times to meet fire flow requirements.

9.5 Disinfection

Currently, all disinfection in Community I occurs by chlorination. The central distribution main between the point of chlorination and the first user acts as the chlorine contact volume. As shown in Table 9.3, this amounts to 144 m^3. As shown in Table 9.7, at the ADD calculated based on the monthly totalizer records the system should achieve 31 minutes of chlorine contact.

Table 9.7 Effective chlorine contact time and CT achieved in Community I

Parameter	Units	Average Day Flow	Peak Flow
Flow rate	L/min	4,660	16,006
Effective contact time	min	31	9
CT (measured residual)		68	20

Peak flow was estimated by multiplying the ADD by the peak hour peaking factor predicted using the PRP-Gumbel method (3.4). At this flow rate the system only achieves 9 minutes of chlorine contact. This is less than required by the province's disinfection requirements.

During the initial site visit CBCL staff measured the free chlorine residual at the first user and found it to be 2.2 mg/L, which is the maximum concentration that can be measured by most hand-held chlorine analyzers. With a chlorination residual of 2.2 mg/L the disinfection system in Community I can achieve a CT of 68 at average flow and 20 at peak flow.

In addition to water use records, the town clerk provided CBCL with records of chlorine residual levels measured by system operators at different points within the distribution system. These were entered into Microsoft Excel spreadsheets and assessed based on the distance between the sampling locations and the point of chlorination (Figure 9.4) and the average day water demand (Figure 9.5 and Figure 9.6).

Figure 9.4 shows how the average free chlorine residual measured in the distribution system decreases as the sampling locations become further and further from the point of chlorination. This result is expected as free chlorine is known to decay over time. Outliers in Figure 9.4 may represent locations where water age is not only dependent on the distance from the point of chlorination. For example, one part of the system may have multiple dead ends despite its proximity to the point of chlorination.

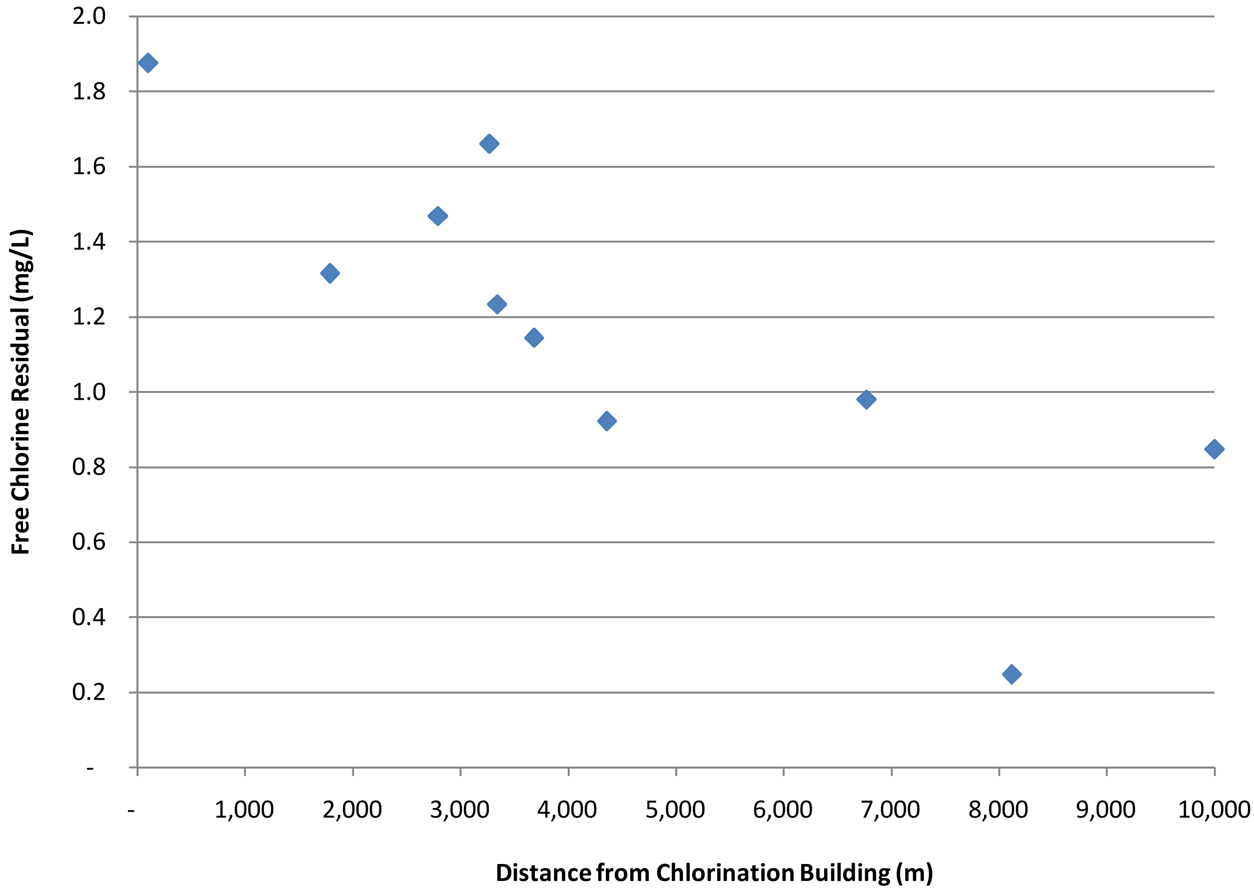

Figure 9.4 **Average free chlorine residual vs. distance from chlorination building in Community I**

The results in Figure 9.5, which show individual free chlorine residual measurements plotted against ADD, suggest that water demand does not have any obvious impact on chlorine residual. It should be kept in mind, however, that the water demand values represent monthly averages and are not paired to individual chlorine residual data points. Note that Location B is located close to the point of chlorination while locations A and C are further along the distribution system.

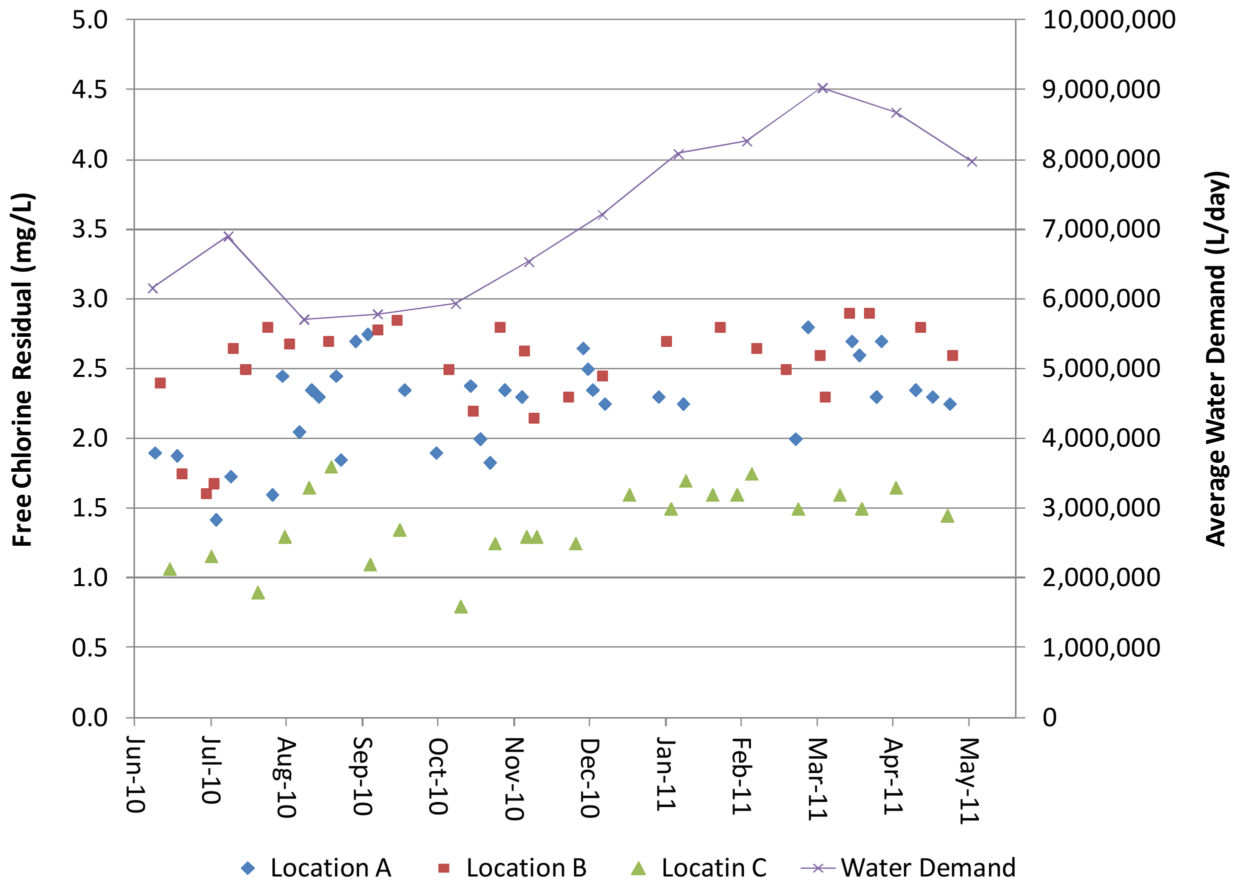

Figure 9.5 Free chlorine residuals measured at three locations in Community I vs. total water demand (June 2010 to May 2011)

9.6 Water Quality

Like most of the other communities that participated in the study, Community I has struggled with high THM and HAA levels throughout their distribution system. THMs and HAAs form when chlorine reacts with NOM, so chlorine dose, NOM concentration and character, and retention time can all contribute to high levels of these DBPs in the distribution system. Bromine levels, temperature, and pH can also contribute.

No significant linear relationship was found between DOC (i.e., NOM quantity) and THMs or HAAs at a 95% confidence level. This does not mean that DBP formation is unrelated to NOM levels; first because there are very few data points available for analysis and secondly because different NOM species are more reactive than others, a fact that is not reflected in the DOC measurement. It is unclear whether the new treatment plant will provide sufficient NOM removal to impact THM and HAA formation in the distribution system.

The results presented in Section 9.5, and in particular Figure 9.6, show that chlorine levels in the Community I distribution system are generally above 1.0 mg/L and often in excess of the 2 mg/L. Though this indicates that the system is well-protected from bacterial regrowth, it also means that an excess of free chlorine is available in the bulk water to react to form THMs and HAAs. The town may wish to consider adding slightly less chlorine to minimize DBP formation. Note that care must be taken to ensure disinfection compliance if the chlorine dose is reduced.

Retention time also drives DBP formation. High retention time (water age) occurs in the distribution system for numerous reasons:

- Low water demand;
- Oversized system components;
- Insufficient mixing (storage tanks); and
- Dead ends.

THM and HAA levels measured by the ENVC in 2007 and 2008 are shown plotted against time along with average water demand in Figure 9.6.

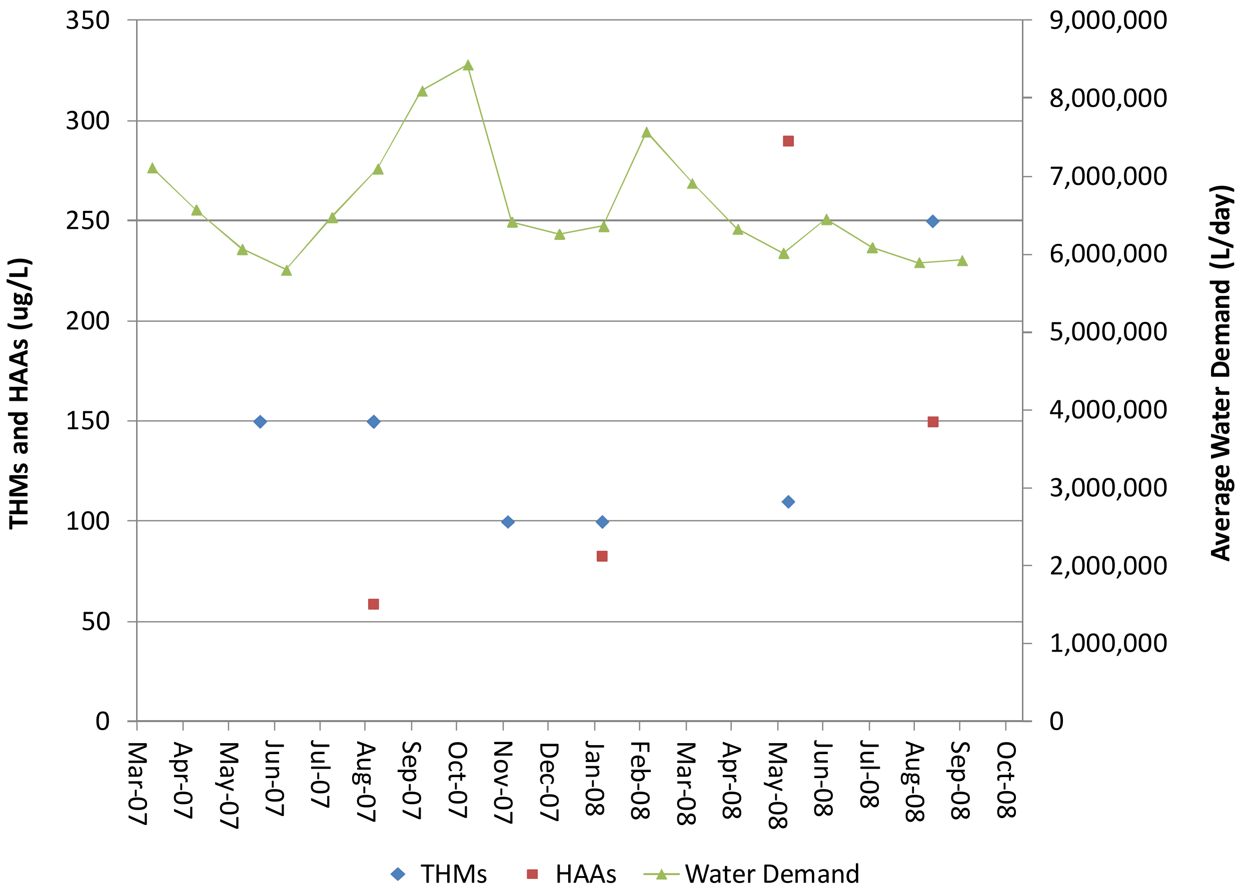

Figure 9.6 THMs and HAAs plotted with average day water demand (2007 and 2008)

The limited dataset presented in Figure 9.6 does not suggest a relationship between average day water demand and THM or HAA formation in Community I.

For approximately three years between 1998 and 2002 the ENVC measured THMs levels in four locations in Community I four times each year. The results are shown in Figure 9.7.

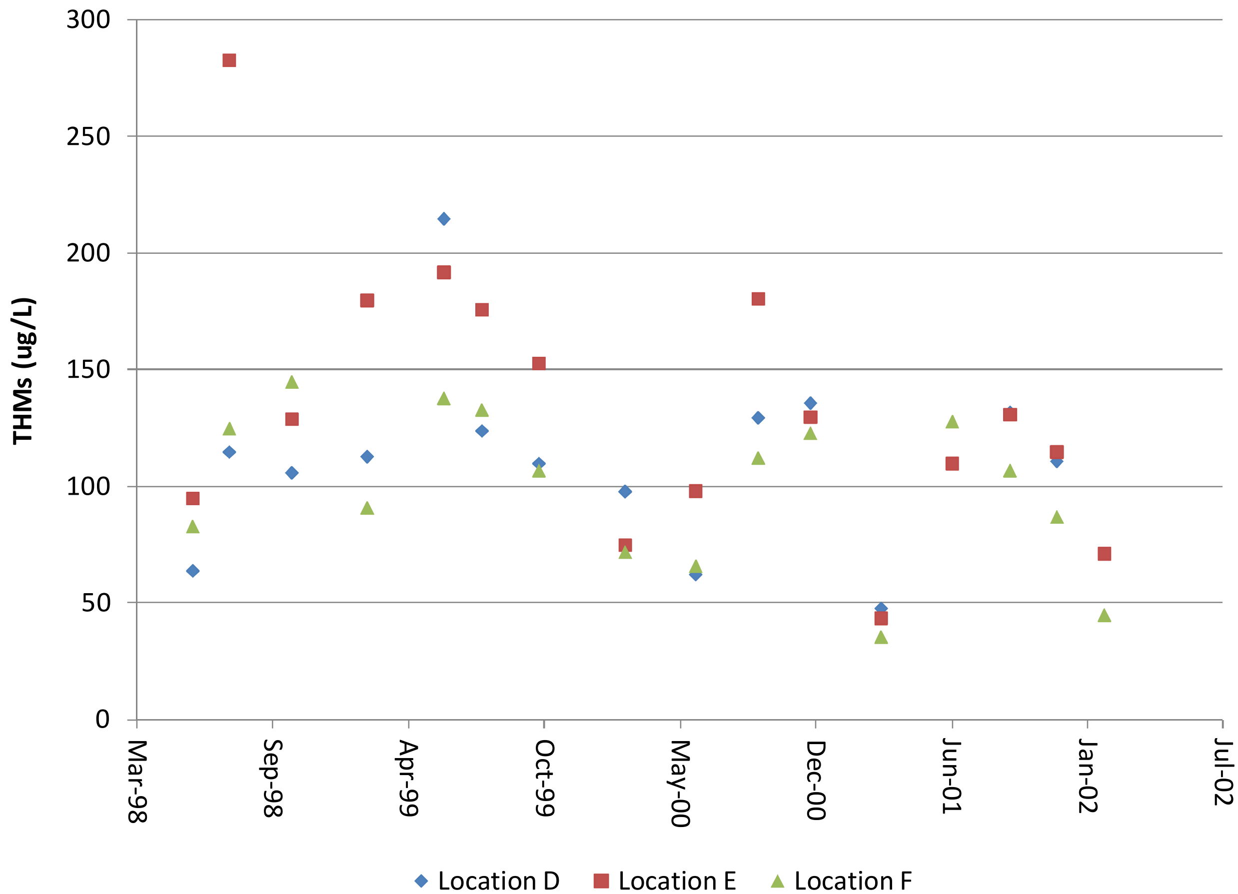

Figure 9.7 THMs measured at four locations in Community I (1998 to 2002)

Location D is furthest from the point of chlorination while Location F is the closest. Location E represents the fish plant, which is northeast of the town. Though it is closer to the point of chlorination than Location D it was frequently found to have higher levels of THMs. This may be because a factor other than distance is increasing the water age in this section of the distribution system.

9.7 Recommendations

Community I can make a number of improvements to the design, operation, and management of their water system to better serve residents and other water users. The town has already taken steps in this direction by constructing a new WTP and water storage tank. The first is expected to provide pathogen removal; minimizing the town's reliance on chlorine contact in the transmission main for disinfection. The new treatment system is also expected to reduce NOM to some degree, though possibly not enough to reduce THM and HAA levels below those recommended by the province.

In addition to the new WTP and storage tank the town may opt to make additional design improvements including:

- Optimize water treatment for NOM removal ahead of chlorination;
- Increase total treated water storage for fire protection;

- Install a larger central distribution main to minimize pressure losses during periods of high demand and/or construct a secondary water main for fire protection; and
- Boost chlorine at the outskirts of the community if necessary.

These design projects should be accompanied by operational improvements including:
- Reduced chlorine dose to minimize THM and HAA formation;
- Perform more frequent leak detection programs to reduce water loss in the distribution system;
- Improve record keeping practices (water use); and
- Focus chlorine residual monitoring on a selection of representative locations (< 5).

The town should also aim to foster a culture of water conservation by initiating programs to encourage residents to minimize water use. This may include metering, incentive programs for low flow water fixtures, and/or educational programs.

CHAPTER 10 SUMMARY AND RECOMMENDATIONS

10.1 Summary

Participating Communities

Choosing a set of representative communities was difficult because many of those identified by the ENVC at the outset of the study were found to no longer have operating fish plants. Most of the fish plants that are still open operate for only part of the year; usually for between 2 and 6 months.

The majority of the participating communities are more than 90% residential, suggesting that residential water use patterns and per capita assumptions should be valid. Most have declining populations, making it difficult to design, or justify the cost of building, new infrastructure.

System Monitoring and Record Keeping

Overall, record keeping (water use and chlorine residual) was found to be inadequate in many communities. The operators contacted for the study regularly monitored chlorine residuals but some lacked a coherent sampling strategy. For example, in two of the case study communities operators had over ten different sampling sites, only one or two of which were sampled on any given day. This made it very difficult to assess disinfection effectiveness over the long term or draw any conclusions about the effects of water demands or overall water quality on chlorine residual maintenance in the distribution system.

Many operators were unable or unwilling to provide detailed information about the distribution system and/or water use records. The same was found for the fish plant operators.

Water Demands

Water demand records were only available for a few of the participating communities. As a result, only four communities were subjected to a detailed evaluation process. The results are summarized in chapters 6 to 9 of this report. The remaining communities were assessed as a group in Chapter 5.

Average and maximum day water demands in the case study communities were higher than would be expected based on their population, as shown in Table 10.1.

Table 10.1 Summary of water demands in case study communities

	Community			
	A	C	G	I
Population	3,764	1,607	407	5,436
Total ADD (L/day)	3,863,699	2,338,995	185,824	6,709,722
Total MDD (L/day)	10,314,274	6,804,209	393,859	9,027,790
Per Capita Demand				
Expected Residential Only	395	395	395	395
*Measured Municipal Only**	682	1,578	814	1,095
Expected Total	804	804	804	804
Measured Total	1,026	1,456	457	1,234
Maximum Day Peaking Factor				
Expected Value	2.0	2.5	2.9 - 3.6	2.0
*Measured Municipal Only**	3.1	2.7	1.9	1.4
Measured Total	2.7	2.9	3.5	1.3

*Defined as the total demand (residential + commercial + institutional) when the fish plant is offline OR water use calculated from dedicated municipal water meter records

All four case study communities were found to have average per capita water demands above those used by the ENVC for disinfection compliance calculations. Total peaking factors increased with decreasing population but per capita demands were not related to population.

Industrial demands were not always to blame for large variations in water demand throughout the year. For example, communities C and G were found to have a higher municipal water demand when the fish plant was offline than when it was online. System operators and community representatives who were contacted to provide comment on this phenomenon indicated that residents in these communities regularly run their taps to prevent their pipes from bursting during the cold winter months. Winter water demands were more important than fish plant demands in two of the four case study communities. This highlights the need for water conservation programs in smaller communities and shows that it is difficult to predict the effect of an industrial user on total water demand.

Diurnal water curves developed for two of the case study communities suggest that the residents in these communities have different water use patterns than those commonly assumed in the water industry. This may be related to industrial water use, community demographics or cultural habits

Infrastructure and Fire Protection

CBCL was unable to obtain record or system drawings for most communities and many system operators were not familiar with basic components of the distribution systems in their communities. This made it difficult to assess the infrastructure in most of the participating communities. The lack of detailed understanding among system operators highlights the need for more targeted operator education and the importance of creating accurate record drawings during the design and construction of new water treatment systems. The ENVC and DMA may also consider initiating a program to develop record drawings and/or system records for existing systems that currently lack them.

Most of the case study communities have distribution systems that are oversized for average day flows, particularly during periods when the fish plant is offline. Given that most of the plants only operate for a few months each year, this might be leading to high water retention times throughout most of the year. Two of the case study communities also experience high winter water demands. This reduces retention time in the winter.

Fire flow can represent over 50% of total flow during a fire event in small communities. Systems in these communities may not be sized to accommodate these flows. Most of the participating communities do not have any or adequate fire protection storage (as recommended in the AC and NL guidelines). Those who do tend to have oversized storage tanks with excessive retention times.

Disinfection

All of the communities who participated in the study are meeting disinfection requirements at assumed average day flows (population x 395 Lpcd). Two are not be able to do so at peak flows as calculated using assumed per capita water demands and peaking factors. The more detailed evaluations conducted for the case study communities suggest that many of them have per capita demands above the assumed value and are unlikely to be able to meet contact time or CT requirements under peak flow conditions.

Water Quality

Excessive THM and HAA levels are a common problem in the participating communities. Preliminary comparisons of THM and HAA records to DOC records from the ENVC indicate that DOC levels are related to THM and HAA formation. More detailed evaluations conducted for the case study communities showed that DOC, chlorine dose, and retention time all play a role in DBP formation. The relative importance of each factor was specific to each community. Note that DBP and DOC datasets were limited (2 to 4 results per year) and that retention time estimates were implied based on overall system area and central distribution main volume.

10.2 Recommendations

The information gathered during the study of the report will assist in the design and operation of water treatment and distribution system components in small and medium municipalities in Newfoundland and Labrador. There are a number of general recommendations that could be implemented to mitigate some of the issues that have been identified in the survey and case study analyses. This list does not represent a complete list of recommendations but provides direction for future projects.

Improvements to System Design

A number of design improvements could be implemented in the participating communities to minimize the effects of variable water demands on water use and water quality. These include:

- Secondary distribution system components (mains, storage, etc.) should be considered to fulfill industrial and/or fire flow demands;

- Water meters should be installed at the intake, after chlorination, and after any large storage facilities;
- Pipe insulation should be installed on municipal and residential pipes to prevent bursting. This should result in reduced water use during winter months;
- Chlorine booster stations should be installed in communities with large distribution systems, those that have a large industrial user located at the beginning of the distribution system, or in areas that are having difficulty maintaining a free chlorine residual;
- New sections of the distribution system should be looped where possible to reduce dead ends;
- Chlorine dosing should be automated and paced based on the flow rate through the chlorine contact volume; and
- In small communities with intractable water quality problems a potable water dispensing unit should be constructed to supply users with safe and clean potable water for their consumption needs.

Most of the recommended design improvements carry a significant capital cost and thus may not be appropriate for all communities.

Improvements to System Operation and Maintenance

System operation and maintenance plans must be well planned yet remain flexible in communities with variable water demands. Some suggestions include:

- Operator training modules focused on record keeping and variable flow management should be developed;
- Daily water use records should be maintained in paper or easily accessible electronic form for both the municipal water distribution and any major industrial users;
- Chlorine residual monitoring should be conducted at a set number of representative locations throughout the distribution system;
- Chlorine dosing should be carefully monitored and adjusted based on the results of residual monitoring throughout the distribution system;
- Maintenance programs designed to reduce water stagnation/water age (ex. main flushing) should be conducted during periods of lower water demand (as established by historical water use records); and
- Leak detection programs should be conducted regularly to minimize water loss through leaks in the distribution system.

Communities may also consider issuing a precautionary boil water advisory during periods of high water demand. Boil water advisories can reduce user confidence in the water system and should only be issued in a precautionary fashion if:

- The chlorination system is not flow-paced;
- High water demands have historically been associated with low chlorine residuals and/or detection of microorganisms; and
- High water demands are reliably connected to a scheduled event (ex. fish plant operation).

Public and Industrial Engagement

A number of initiatives have been introduced in other small communities in Canada to encourage water conservation among residential and industrial users. Water conservation will minimize the energy and chemical inputs required to treat the water and can help to minimize large variations in water demand. These include**:**

- Develop educational programs to reduce water use, particularly during the winter months (Appendix E);
- Offer rebates for water conserving fixtures;
- Renovate public buildings to minimize total water use;
- Encourage industries to work and/or draw water at off-peak times (Appendix E);
- Develop better coordination between the municipality and fish processing plant operator regarding flow data and water quality issues experienced in both systems; and
- Where feasible, institute water metering and usage-based pricing for residential, commercial, and industrial users.

Existing Guideline Documents

The existing NL design guidelines include numerous short sections with recommendations for the design of water distribution system components. Most of these are drawn from the guideline documents prepared by other jurisdictions. Each section should be reviewed in light of the results of this study and adjusted if necessary. These should be presented in a clear and concise manner in a defined section of the guidelines. Some examples of potential additions and improvements follow.

The ENVC and associated departments should strongly encourage system designers to use historical water use records to design system components. Designers could be required to monitor water use for a specified period of time (i.e., one year) before completing the design of a new treatment or disinfection system. Consultation with industrial water users during the design process should be required to establish reasonable water use estimates if historical industrial water use records are not available.

Where historical records are not available, maximum day peaking factors should continue to be chosen based on population as described in the existing NL design guidelines or calculated using the DVGW method. Peak hour peaking factors can be determined using a number of different calculation methods or by referring to the table of values presented in the existing guidelines. The former may be of more use to small communities and/or communities with populations that fall within the population ranges described in the table.

Per capita water use rates should only be used when historical water use records are unreliable or unavailable. The ENVC estimate of 340 Lpcd is well below that measured in all four case study communities and is likely to underestimate the total amount of water required, particularly for communities with high industrial and/or winter demands. Note that the Environment Canada values of 395 Lpcd and 804 Lpcd for residential and total water use, respectively, are also below those calculated for most of the case study communities.

If the ENVC does choose to maintain a default per capita water demand value the following should be kept in mind:

- The influence of population and monitoring on water use (Section 2.1);
- The impacts of different types of water users on overall and diurnal water demand (Section 2.2);
- The source and accuracy of existing per capita water demand estimates (Section 2.4.1 and Appendix I);
- The source and accuracy of existing maximum day and hourly peaking factor calculation methods (Section 2.4.2 and Section 2.4.3);
- Variations in water demand among different communities evaluated in this study (Chapter 10);
- The impact of the chosen per capita demand factor on disinfection calculations, water age, and fire protection (Section 2.7); and
- The feasibility of using the chosen value and demand projection method for the design of water system components in small rural communities with declining populations (Section 2.8.2).

Separate per capita values or multipliers could be established for communities with industrial users and/or significant winter demands. These should result in a per capita water demand value above 340 Lpcd (a list of potential alternatives is presented in Appendix I). Alternatively designers could be required to monitor industrial and commercial water separately and add these to the estimate of per capita residential water demand. The latter option is preferred.

A method for predicting future water demands should be provided in the design guidelines. This should include the preferred period of projection (ex. 20 years), equations, and instructions for choosing an appropriate growth rate. Though many communities in Newfoundland and Labrador continue to experience population decline, the growth rate should not be set at a negative value. Rather, a low growth rate should be assumed and system components should be designed with flexibility in mind to ensure that water quality can be optimized at different flow rates.

Minimum fire flow requirements should be published for small communities to be used during the design of water infrastructure. The MOE Guidelines provide estimates, but they are specific to Ontario and may not apply to other provinces. Sections of the Fire Underwriters Survey should be included in the sections describing the determination of appropriate fire flow requirements for communities in Newfoundland and Labrador. These should take into account the average size, construction, and spacing of buildings in the province. A copy of the most recent Fire Underwriters Survey guidance document is provided in Appendix F.

CHAPTER 11 **REFERENCES**

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APPENDIX A

Information Collection Sheets

1. Municipal Operator

1. Community name:

[ ]

2. How do users pay for water? (check all that apply)

- [ ] Annual lump sum amount
- [ ] Monthly lump sum amount
- [ ] Annual rate based on usage
- [ ] Monthly rate based on usage
- [ ] Different rates for residents, institutions, industry, etc.
- [ ] Included in taxes
- [ ] Users do not pay for water
- [ ] Unknown

3. How many service connections are there on the distribution system?

[ ]

4. How many of these service connections can be defined as:

Residential	
Industrial (factories, etc.)	
Commercial (businesses)	
Institutional (hospitals, schools, etc.)	

5. Is flow monitored automatically within the distribution system? (check all that apply)

- [ ] Yes, at the intake
- [ ] Yes, at the outlet of the treatment system
- [ ] Yes, on the transmission main that serves the large industrial user
- [ ] Yes, within the municipal system
- [ ] No

Other (please specify)

[ ]

6. Are flow records kept for the following?

	Paper	Digital	Not applicable
Intake	○	○	○
Outlet of treatment system	○	○	○
Transmission main to large user	○	○	○
Municipal distribution system	○	○	○
Other	○	○	○

Be sure to obtain a copy (paper or digital) of any flow records the operator has access to.

7. Do you keep records of distribution pump operation? (on, off, etc.)

- ○ Yes
- ○ No
- ○ Not Applicable

8. Does your community have a reservoir (storage tank, tower, etc.)?

- ○ Yes
- ○ No

9. If yes, what are its dimensions? (please fill out all that apply)

Shape ________

Height (depth) ________

Length ________

Width ________

Diameter ________

Elevation from the ground ________

10. What design and operational strategies are used to ensure that water in the reservoir does not become stagnant? (check all that apply)

- ☐ Inlet/outlet design
- ☐ Internal baffling
- ☐ Mechanical mixing
- ☐ None
- ☐ Unknown
- ☐ Not applicable (no reservoir)

Other (please specify)

____

11. What is the flow rate into the reservoir?

12. Where are samples taken for the following parameters:

Chlorine residual

Coliform count

THMs and HAAs

Lead

13. How frequently are the following parameters monitored?

	Once a month	Every second month	Quarterly	Twice a year	Annually	Not monitored
Chlorine residual (handheld/operator)	◯	◯	◯	◯	◯	◯
Chlorine residual (ENVC)	◯	◯	◯	◯	◯	◯
Coliforms	◯	◯	◯	◯	◯	◯
THMs	◯	◯	◯	◯	◯	◯
HAAs	◯	◯	◯	◯	◯	◯
Lead	◯	◯	◯	◯	◯	◯
Iron	◯	◯	◯	◯	◯	◯
Manganese	◯	◯	◯	◯	◯	◯

14. What is the approximate distance between the sampling location(s) and the point of disinfection?

Chlorine residual

Coliform count

THMs and HAAs

Lead

15. Please describe the intake pipe that carries water from the source to the treatment system:

Diameter

Length

Material

16. Where does the transmission main that serves the large industrial user diverge from the municipal distribution system? (Please indicate it on the map).

17. Describe the transmission main that carries water from the treatment system to the large industrial user:

Diameter

Length

Material

18. How many dead ends are there in the distribution system? (please indicate them on the map).

Please indicate the other major components of the municipal distribution system on the map provided.

19. Where possible, please describe the other major components of the municipal distribution system.

Diameter

Length

Material

20. How often do you perform the following distribution system maintenance activities?

	Once a month	Every two months	Quarterly	Twice annually	Annually	As required	Never	Not applicable
Check chlorine residuals	○	○	○	○	○	○	○	○
Sample for THMs and HAAs	○	○	○	○	○	○	○	○
Sample for coliforms	○	○	○	○	○	○	○	○
Sample for lead	○	○	○	○	○	○	○	○
Preventative maintenance of system components	○	○	○	○	○	○	○	○
Flush the distribution system	○	○	○	○	○	○	○	○
Drain/pump hydrants	○	○	○	○	○	○	○	○
Collect flow data	○	○	○	○	○	○	○	○
Record pump operating data	○	○	○	○	○	○	○	○
Inspect reservoir	○	○	○	○	○	○	○	○
Clean/maintain reservoir	○	○	○	○	○	○	○	○

21. How do you dispose of municipal wastewater?

- ☐ Collected and sent to a wastewater treatment facility
- ☐ Collected and disposed of in the ocean
- ☐ Collected and disposed of in a fresh water body
- ☐ Managed by individual users (septic systems, etc.)

Other (please specify)

2. Industry Representative

1. Name of the facility:

2. Contact person

Name

Position

Phone number

Email address

3. How frequently does the facility operate?

- < 2 months/year
- 2 to 4 months a year
- 4 to 6 months a year
- 6 to 8 months a year
- 8 to 10 months a year
- > 10 months a year

4. Please provide the dates when you started and finished production in the following years:

	MM	DD	YYYY
2007 start	/	/	
2007 finish	/	/	
2008 start	/	/	
2008 finish	/	/	
2009 start	/	/	
2009 finish	/	/	

5. What hours of the day does the facility operate?

6. Which of the following are monitored?

- [ ] Inlet water flow
- [ ] Total daily water volume (in)
- [ ] Outlet wastewater flow
- [ ] Total daily wastewater volume (out)
- [ ] Inlet pressure
- [ ] Incoming water quality

Other (please specify)

7. Are records kept of the following?

- [ ] Inlet water flow
- [ ] Total daily water volume (in)
- [ ] Outlet wastewater flow
- [ ] Total daily wastewater volume (out)
- [ ] Inlet water pressure
- [ ] Incoming water quality

Other (please specify)

8. How much water does your facility use?

Average day

Maximum day

Average hour

Maximum hour

9. Describe the main pipe that provides water to your facility:

Diameter

Length

Material

10. Do you provide additional treatment to incoming water?

- ( ) Yes
- ( ) No
- ( ) Unknown

11. Have any of the following parameters affected the operation of your facility or the quality of your products?

- [ ] High chlorine
- [ ] Low chlorine
- [ ] Presence of bacteria, viruses, etc.
- [ ] Turbidity
- [ ] Iron
- [ ] Manganese
- [ ] Lead
- [ ] Low pH
- [ ] TDS (salt)
- [ ] Alkalinity/hardness
- [ ] Low pressure

12. Does your facility have water storage capacity? (reservoirs, storage tanks, etc.)

- ○ Yes
- ○ No
- ○ Unknown

13. How do you pay for the water used at the facility?

- ○ Monthly lump sum
- ○ Annual lump sum
- ○ Monthly rate based on usage
- ○ Annual rate based on usage
- ○ Water is not paid for
- ○ Unknown

Other (please specify)

____

14. How do you dispose of wastewater from the facility?

____

3. Fire flows / flushing operations

Note: This section should only be completed for communities with 1,500 or fewer residents.

1. Does your community have fire hydrants?

- ◯ Yes
- ◯ No

Other (please specify)

[ ]

2. If yes, how many are there in the community?

Total number of hydrants [ ]

Please indicate all hydrants on the map provided by the field technician.

3. Does your community have a flushing program for its water distribution system?

- ◯ Yes
- ◯ No
- ◯ Unknown

4. If you answered 'yes' to the previous question, please describe the flushing program in as much detail as possible. (ie. which hydrants are opened and for how long, etc.)

[ ]

5. How often is the flushing program carried out?

- ◯ Quarterly
- ◯ Twice a year
- ◯ Annually
- ◯ As required
- ◯ Unknown
- ◯ Not applicable (no flushing program is in place)

Other (please specify)

[ ]

4. Flow testing

1. Measured flow rates

Municipal intake	
Post-chlorination	
Industrial inlet	
Other	

5. Pictures

1. Please try to take pictures of as many of these as possible. (Photo labels are always helpful!)

- ☐ Industrial facility
- ☐ Representative residential building
- ☐ Storage tank
- ☐ Intake
- ☐ Flowmeters
- ☐ Fire hydrants
- ☐ Pumps at treatment facility
- ☐ Booster pumps

Other (please specify)

2. Industry Representative

1. Name of the facility:

2. Contact person

Name

Position

Phone number

Email address

3. How frequently does the facility operate?

- < 2 months/year
- 2 to 4 months a year
- 4 to 6 months a year
- 6 to 8 months a year
- 8 to 10 months a year
- > 10 months a year

4. Please provide the dates when you started and finished production in the following years:

	MM		DD		YYYY
2007 start		/		/	
2007 finish		/		/	
2008 start		/		/	
2008 finish		/		/	
2009 start		/		/	
2009 finish		/		/	

5. What hours of the day does the facility operate?

6. Which of the following are monitored?

- ☐ Inlet water flow
- ☐ Total daily water volume (in)
- ☐ Outlet wastewater flow
- ☐ Total daily wastewater volume (out)
- ☐ Inlet pressure
- ☐ Incoming water quality

Other (please specify)

____

7. Are records kept of the following?

- ☐ Inlet water flow
- ☐ Total daily water volume (in)
- ☐ Outlet wastewater flow
- ☐ Total daily wastewater volume (out)
- ☐ Inlet water pressure
- ☐ Incoming water quality

Other (please specify)

____

8. How much water does your facility use?

Average day	
Maximum day	
Average hour	
Maximum hour	

9. Describe the main pipe that provides water to your facility:

Diameter	
Length	
Material	

10. Do you provide additional treatment to incoming water?

- ○ Yes
- ○ No
- ○ Unknown

11. Have any of the following parameters affected the operation of your facility or the quality of your products?

- ☐ High chlorine
- ☐ Low chlorine
- ☐ Presence of bacteria, viruses, etc.
- ☐ Turbidity
- ☐ Iron
- ☐ Manganese
- ☐ Lead
- ☐ Low pH
- ☐ TDS (salt)
- ☐ Alkalinity/hardness
- ☐ Low pressure

12. Does your facility have water storage capacity? (reservoirs, storage tanks, etc.)

- ○ Yes
- ○ No
- ○ Unknown

13. How do you pay for the water used at the facility?

- ○ Monthly lump sum
- ○ Annual lump sum
- ○ Monthly rate based on usage
- ○ Annual rate based on usage
- ○ Water is not paid for
- ○ Unknown

Other (please specify)

14. How do you dispose of wastewater from the facility?

APPENDIX B

Completed Information Collection Sheets

(digital)

APPENDIX C

Case Study Community Water Demand Data

(digital)

APPENDIX D

Standard Record Keeping Sheets

(Includes blank sheets for distribution and completed examples)

Location 1: 187 Yale St.
Location 2: 50 Kaye St.
Location 3: 30 Water St. (Town Hall)
Units: mg/L

Month	Date	Time	Location 1		Notes	Time	Location 2		Notes	Time	Location 3		Notes
			Total Chlorine	Free Chlorine			Total Chlorine	Free Chlorine			Total Chlorine	Free Chlorine	
January	1	8:00	1.5	1.2		8:00	1.5	1.2		8:00	1.5	1.2	
	2	13:00	2	2		13:00	2	2		13:00	2	2	
	3	9:00	1.4	1		9:00	1.4	1		9:00	1.4	1	
	4	9:30	2	1		9:30	2	1		9:30	2	1	
	5	9:00	0.4	0.2		9:00	0.4	0.2		9:00	0.4	0.2	
	6	8:00	0.4	nd	low free chlorin	8:00	1.1	1		8:00	0.4	0.1	
	7	13:00	1.5	1.2		13:00	1.5	1.2		13:00	1.5	1.2	
	8	9:00	2	2		9:00	2	2		9:00	2	2	
	9	9:30	1.4	1		9:30	1.4	1		9:30	1.4	1	
	10	9:00	2	1		9:00	2	1		9:00	2	1	
	11	8:00	0.4	0.2		8:00	0.4	0.2		8:00	0.4	0.2	
	12	13:00	1.5	1.2		13:00	1.5	1.2		13:00	1.5	1.2	
	13	9:00	2	2		9:00	2	2		9:00	2	2	
	14	9:30	1.4	1		9:30	1.4	1		9:30	1.4	1	
	15	9:00	2	1		9:00	2	1		9:00	2	1	
	16	8:00	0.4	0.2		8:00	0.4	0.2		8:00	nd	nd	low residual
	17	13:00	1.5	1.2		13:00	1.5	1.2		13:00	1.5	1.2	
	18	9:00	2	2		9:00	2	2		9:00	2	2	
	19	9:30	1.4	1		9:30	1.4	1		9:30	1.4	1	
	20	9:00	2	1		9:00	2	1		9:00	2	1	
	21	8:00	0.4	0.2		8:00	0.4	0.2		8:00	nd	nd	low residual
	22	13:00	1.5	1.2		13:00	1.5	1.2		13:00	1.5	1.2	
	23	9:00	2	2		9:00	2	2		9:00	2	2	
	24	9:30	1.4	1		9:30	1.4	1		9:30	1.4	1	
	25	9:00	2	1		9:00	2	1		9:00	2	1	
	26	8:00	0.4	0.2		8:00	0.4	nd	low free chlorin	8:00	0.4	0.2	
	27	13:00	1.5	1.2		13:00	1.5	1.2		13:00	1.5	1.2	
	28	9:00	2	2		9:00	2	2		9:00	2	2	
	29	9:30	1.4	1		9:30	1.4	1		9:30	1.4	1	
	30	9:00	2	1		9:00	2	1		9:00	2	1	
	31	8:00	0.4	0.2		8:00	0.4	0.2		8:00	0.4	0.2	
February	1	13:00	1.5	1.2		13:00	1.5	1.2		13:00	1.5	1.2	
	2	9:00	2	2		9:00	2	2		9:00	2	2	
	3	9:30	1.4	1		9:30	1.4	1		9:30	1.4	1	
	4	9:00	2	1		9:00	2	1		9:00	2	1	
	5	8:00	0.4	0.2		8:00	0.4	0.2		8:00	0.4	0.2	
	6	13:00	1.5	1.2		13:00	1.5	1.2		13:00	1.5	1.2	
	7	9:00	2	2		9:00	2	2		9:00	2	2	
	8	9:30	1.4	1		9:30	1.4	1		9:30	1.4	1	
	9	9:00	2	1		9:00	2	1		9:00	2	1	
	10	8:00	0.4	0.2		8:00	0.4	0.2		8:00	0.4	0.2	
	11	13:00	1.5	1.2		13:00	1.5	1.2		13:00	1.5	1.2	
	12	9:00	2	2		9:00	2	2		9:00	2	2	

Location 1:
Location 2:
Location 3:
Units: mg/L

Month	Date	Time	Location 1		Notes	Time	Location 2		Notes	Time	Location 3		Notes
			Total Chlorine	Free Chlorine			Total Chlorine	Free Chlorine			Total Chlorine	Free Chlorine	

Location: Distribution Pump #1
Units: US gallons/minute (gpm)

Month	Date	Time	Pump Flow Rate	Pump Hours of Operation	Notes
January	1	8:00	208	8	
	2	13:00	250	10	
	3	9:00	220	8.5	
	4	9:30	240	9	
	5	9:00	200	7	
	6	8:00	208	8	
	7	13:00	250	10	Fixed distribution pump #2
	8	9:00	220	8.5	
	9	9:30	240	9	
	10	9:00	200	7	
	11	8:00	208	8	
	12	13:00	250	10	
	13	9:00	220	8.5	
	14	9:30	240	9	
	15	9:00	200	7	
	16	8:00	208	8	
	17	13:00	250	10	
	18	9:00	220	8.5	
	19	9:30	240	9	
	20	9:00	200	7	
	21	8:00	208	8	
	22	13:00	250	10	
	23	9:00	220	8.5	
	24	9:30	240	9	
	25	9:00	200	7	
	26	8:00	208	8	
	27	13:00	250	10	
	28	9:00	220	8.5	
	29	9:30	240	9	
	30	9:00	200	7	
	31	8:00	208	8	
February	1	13:00	250	10	
	2	9:00	220	8.5	
	3	9:30	240	9	
	4	9:00	200	7	
	5	8:00	208	8	
	6	13:00	250	10	
	7	9:00	220	8.5	
	8	9:30	240	9	
	9	9:00	200	7	
	10	8:00	208	8	
	11	13:00	250	10	
	12	9:00	220	8.5	
	13	9:30	240	9	
	14	9:00	200	7	
	15	8:00	208	8	
	16	13:00	250	10	
	17	9:00	220	8.5	
	18	9:30	240	9	
	19	9:00	200	7	
	20	8:00	208	8	
	21	13:00	250	10	
	22	9:00	220	8.5	
	23	9:30	240	9	
	24	9:00	200	7	
	25	8:00	208	8	
	26	13:00	250	10	
	27	9:00	220	8.5	

Location:

Units:

Month	Date	Time	Pump Flow Rate	Pump Hours of Operation	Notes

Location: Outlet of storage volume
Units: m
Storage Volume (L): 250,000
Conversion: Litres / meter
*only feasible if you know how much water is entering the storage volume each day

Month	Date	Time	Level
January	1	8:00	10
	2	13:00	12
	3	9:00	11
	4	9:30	11.5
	5	9:00	12
	6	8:00	12.5
	7	13:00	10
	8	9:00	10
	9	9:30	12
	10	9:00	11
	11	8:00	11.5
	12	13:00	12
	13	9:00	12.5
	14	9:30	10
	15	9:00	10
	16	8:00	12
	17	13:00	11
	18	9:00	11.5
	19	9:30	12
	20	9:00	12.5
	21	8:00	10
	22	13:00	10
	23	9:00	12
	24	9:30	11
	25	9:00	11.5
	26	8:00	12
	27	13:00	12.5
	28	9:00	10
	29	9:30	10
	30	9:00	12
	31	8:00	11
February	1	13:00	11.5
	2	9:00	12
	3	9:30	12.5
	4	9:00	10
	5	8:00	10
	6	13:00	12
	7	9:00	11
	8	9:30	11.5
	9	9:00	12
	10	8:00	12.5
	11	13:00	10
	12	9:00	10
	13	9:30	12
	14	9:00	11
	15	8:00	11.5
	16	13:00	12
	17	9:00	12.5
	18	9:30	10
	19	9:00	10
	20	8:00	12
	21	13:00	11
	22	9:00	11.5
	23	9:30	12
	24	9:00	12.5
	25	8:00	10

Location:
Units:
Storage Volume (L):

Month	Date	Time	Level

Location: Outlet of WTP
Units: 1000 US gallons

Month	Date	Time	Total Volume	Notes
January	1	8:00	-	
	2	13:00	100	
	3	9:00	200	
	4	9:30	300	
	5	9:00	400	
	6	8:00	500	
	7	13:00	600	
	8	9:00	700	
	9	9:30	800	
	10	9:00	900	Fixed leak at 210 Terrace Dr.
	11	8:00	1,000	
	12	13:00	1,100	
	13	9:00	1,200	
	14	9:30	1,300	
	15	9:00	1,400	
	16	8:00	1,500	
	17	13:00	1,600	
	18	9:00	1,700	
	19	9:30	1,800	
	20	9:00	1,900	
	21	8:00	2,000	
	22	13:00	2,100	
	23	9:00	2,200	
	24	9:30	2,300	
	25	9:00	2,400	
	26	8:00	2,500	
	27	13:00	2,600	
	28	9:00	2,700	
	29	9:30	2,800	
	30	9:00	2,900	
	31	8:00	3,000	
February	1	13:00	3,100	
	2	9:00	3,200	
	3	9:30	3,300	
	4	9:00	3,400	
	5	8:00	3,500	Low chlorine residual at outlet
	6	13:00	3,600	
	7	9:00	3,700	
	8	9:30	3,800	
	9	9:00	3,900	
	10	8:00	4,000	
	11	13:00	4,100	
	12	9:00	4,200	
	13	9:30	4,300	
	14	9:00	4,400	
	15	8:00	4,500	
	16	13:00	4,600	
	17	9:00	4,700	
	18	9:30	4,800	
	19	9:00	4,900	
	20	8:00	5,000	
	21	13:00	200	Reset totalizer
	22	9:00	300	
	23	9:30	400	
	24	9:00	500	
	25	8:00	600	
	26	13:00	700	
	27	9:00	800	
	28	9:30	900	

Month	Date	Time	Total Volume	Notes

APPENDIX E

Background Information on Water Conservation Strategies

Policy Research Initiative | Projet de recherche sur les politiques

Sustainable Development
Briefing Note

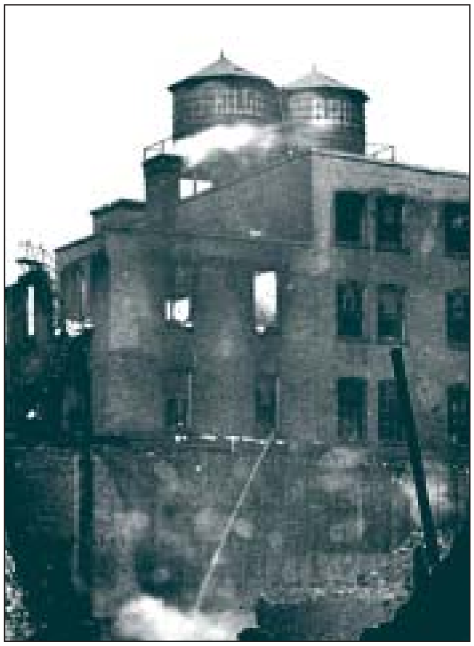
Wet Industry: An Opportunity for Strategic Municipal Water Demand Management

Highlights

- Municipal water systems must satisfy peak demand. This can lead to wasted capacity in off-peak hours and increase the per-unit cost of water services.
- Leamington, Ontario, manages its water services jointly with wet industry to shift industrial demand from peak to off-peak hours. Flattening the demand curve, this reduces per unit costs and the need to expand infrastructure.
- Raising water prices may also reduce peak demand, but industry may then relocate to other sites where costs are lower.

Background

Before 1962, water pressure in many Ontario municipal water systems fell during peak demand hours. Industries that needed water built their own water towers that filled overnight from municipal systems. This began to change in 1962, with the passage of the *Ontario Water Resources Act* (OWRA). Municipal water towers were built to balance system pressure, and reservoirs and water systems developed to ensure the ability to meet peak demand, for peak period use, and for fire safety. Industrial water towers disappeared as the municipalities assumed the cost of ensuring water availability and pressure during peak hours.



The water towers atop the Kilgour Brothers' paper and box factory marked the eastern reach of a blaze that leveled Toronto's wholesale district in 1904. Such industrial water towers have become rare in Ontario since the development of municipal water towers in the 1960s.

However, municipal water and sewer systems that focus on peak demand have a dilemma. High volume, underutilized systems add a fixed cost to water rates. Flattening the demand curve can solve this, but the residential demand curve is resistant to flattening as it involves the consumption habits of a large number of small consumers. Large industrial water users may be more tractable.

Some Approaches to Managing Industrial Demand for Municipal Water

In the early 1950s, the town of Exeter, Ontario, disproportionately assigned the cost of a municipal waterworks expansion to the local food processor. The plant closed and residential ratepayers picked up the slack until another processor was lured to the site several years later.

Canada

Similarly, in the early 1990s, municipalities in the Netherlands and Germany often disconnected services to industrial customers to meet conservation targets. Industry, especially the water-intensive food industry, responded with consolidation, resulting in unemployment, and because most of the cost of water service is fixed, there were runaway rate increases for residential ratepayers (Dick, 1999).

Toronto, Ontario, charges relatively high rates to all users, including wet industry. Although Toronto's food industry has grown, most of the large-scale food processors are gone, and the industry has cited high water prices as contributing to the decision to leave, close, or consolidate. Less water intensive, small, ethnic, and specialty food "assemblers" have replaced the larger employers.

The City of Sacramento, California, rewards companies that install water-efficient equipment with connection fee reductions of as much as 75 percent. Water and sewer efficiency reduces the manufacturer's cost of production, and in the long term, a company that begins with a culture of conservation may have a lower draw on all municipal services.

Hamilton, Ontario, treats water overnight and pumps it to uphill reservoirs above the Niagara Escarpment to supply the daytime needs of the city. This strategy shifts production and distribution costs to off-peak hours, but requires large high-elevation storage capacity that may not be available in flatter terrain.

Back to the Future: Leamington's Time Shifting Approach to Demand Management

Municipal Wet Industry Water Demand Management

There are three strategies for efficient water demand management (WDM):

- leak reduction;
- conservation; and
- peak load and trough management.

Each strategy plays a different role, and can target different classes of users.

Factors contributing to leakage (beyond old decayed infrastructure) may include a lack of measurement control and elevated system pressure due to dead-end lines or low off-peak demand. Lack of measurement control, and system decay can be remedied with metering and line maintenance. Elevated system pressure due to dead-end lines is a potential threat to public safety as bacteria can multiply in stagnant lines.

Many conservation programs, particularly those targeting wet industry, are ultimately inefficient, because reducing use without addressing time of use can lead to reducing off-peak use with little or no impact on peak use, which drives system capacity demands.

The Leamington Story

Leamington, Ontario, has one water treatment facility that was built in partnership with the H.J. Heinz Company of Canada, and which is owned by the municipalities of Leamington, Kingsville, Essex, and Lakeshore. In 1999, Leamington had approximately 7,150 residential accounts. Fifty-one percent of system capacity is allocated to non-residential users who account for 73 percent of all water use (Stantec, 1999).

Until the 1970s, food processing employed about 50 percent of Leamington's labour force. Food processing remains a leading employment sector, but is followed closely by the agriculture and the automotive sectors. This shift in employment patterns is crucial to Leamington's water strategy.

Leamington's WDM strategy has an hour-by-hour focus that measures production and distribution with an eye to flattening the entire demand curve. Leamington's water-treatment system capacity is 40 percent more efficient than Toronto's, despite having lower user fees (water and sewer combined rates of \$0.6248/m^3 vs. \$1.1599/m^3).

In the 1970s and 1980s, when the Leamington system reached its first capacity hurdles, Heinz limited itself to 20 percent of hourly system capacity. In the late 1990s, Leamington reached its next capacity hurdle, because of the rapid expansion of the greenhouse vegetable industry, now larger than the entire US greenhouse vegetable industry. This industry grew by 360 percent from 1996 to 2000, delivering more than \$200 million worth of investment, and representing one quarter of Canada's greenhouse industry. This time, an industry-wide demand management solution was implemented to flatten Leamington's demand curve. This has since led to significant and voluntary water conservation action by agricultural ratepayers (Stantec, 1999).

Capacity Utilization

The Leamington system seeks to optimize capacity use by its wet industry customers to reduce the fixed costs of water production. Industry is encouraged (in some cases required) to install flow control and water storage equipment in new construction. Reservoir retrofits cost \$100 to \$125 per cubic metre. Greenhouse operations can manage their water load using a 24-hour draw with an engineered reservoir that holds 60 percent of the capacity of their daily requirements.

Water conservation is inherent to this type of system. With water recycling technology, a greenhouse farm can expand without impacting the municipal system. In Leamington, wet industry has shifted to drawing water 24/7, storing it during low use times for use during high use times. This has had the substantial side benefit of shifting the electricity load from peak rate times to low rate times. Leakage has also been reduced, as system pressures no longer rise during off-peak hours. These benefits reduce costs for all ratepayers.

Leamington's Water Demand Management Strategy

New municipal water demand from greenhouse or field irrigation expansion is controlled through the following strategy.

1. An \$800 per ha water system access fee is charged for new greenhouse developments.
2. Reservoir installation reduces per ha flow requirements from up to 2.4 m^3/hour to 0.8 m^3/hour. Reservoir installation costs a one-time \$100 to \$120 per m^3.
3. Differential rate pricing for 2004 comes to \$0.40/m^3 for compliant greenhouse operations. After 2004, non-compliant greenhouse operations will be charged \$1.60/m^3.
4. Universal metering and alternating-day residential outdoor irrigation bans manage residential water use that is difficult to control under even optimum circumstances.
5. Water safety improvements include fully looped water mains to eliminate dead-end pressure and turbidity, applied to all industrial and agricultural connections. A carbon filtration unit at the Heinz intake assures water quality and has made Leamington the first municipality in Ontario to deliver water that meets Hazard Analysis and Critical Control Point (HACCP) (a food safety protocol) standards.

Future developments may include the following.

6. Current estimates indicate only 15 to 20 percent of hydroponic greenhouses are currently recycling their water; recycling and other stewardship initiatives could potentially enable the greenhouse industry to treble its size.

7. The current demand for field irrigation ranges between 10 and 20 percent of Leamington's peak demand. The development of private and parallel raw water systems for field irrigation could deliver 40 million litres per day for field irrigation at a significantly lower cost than treated water.

Conclusion

Water demand management requires accurate measurement, efficient production, and a balanced approach to conservation. Universal metering is essential for a municipal water system to measure and thus manage demand. As the largest single water users in many municipalities, wet industries are an obvious ally and target for water demand management. This group of high water-use ratepayers can be effectively managed to shift demand and increase system efficiency. The need for new water infrastructure projects could be deferred in many municipalities by flattening the demand curve, as done in Leamington.

Further reading:

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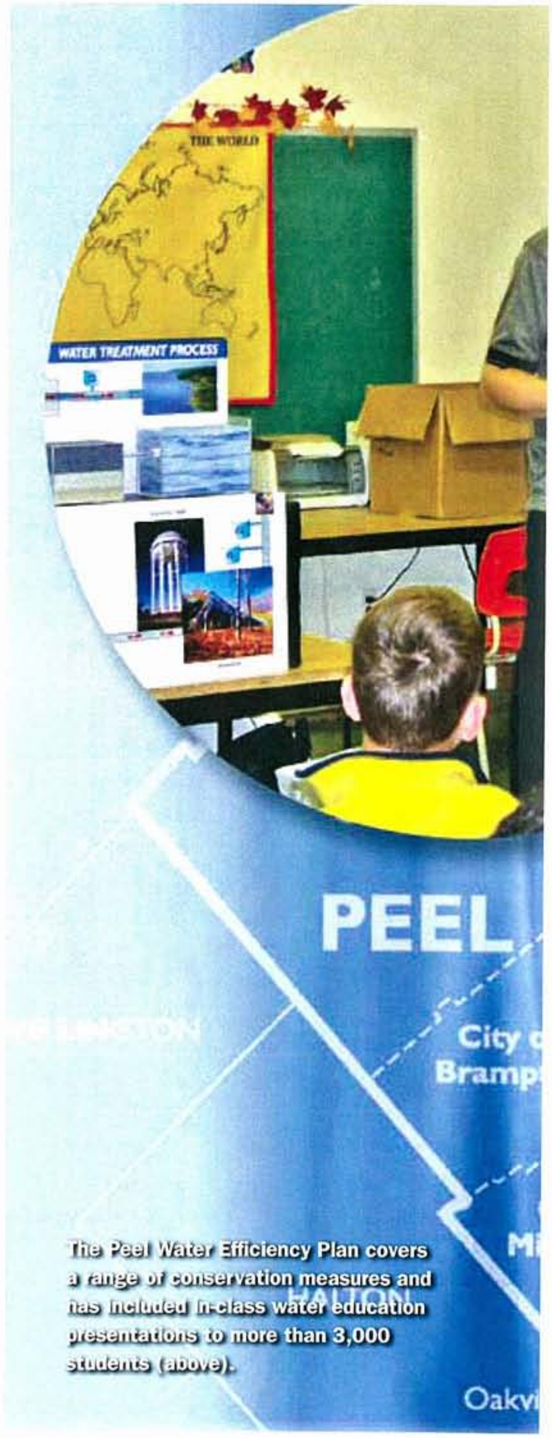
The author, Phil Dick, is an Investment Development Officer with the Province of Ontario. He can be contacted at:

Food Industry Division,
Ontario Ministry of Agriculture & Food,
1 Stone Road West, Guelph, ON N1G 4Y2,
tel: 519 826.4385
email: phil.dick@omaf.gov.on.ca

Water Smart Peel
Greater Awareness Today Means More Water Tomorrow



The Peel Water Efficiency Plan covers a range of conservation measures and has included in-class water education presentations to more than 3,000 students (above).

To stem the tide of growing demand on its water and wastewater infrastructure, the Region of Peel implemented a comprehensive water-efficiency program to inspire residents to get involved and save water. **BY JOE VIEIRA**

WATER SMART PEEL IS the culmination of technical and outreach programs that share one objective—to inspire children, residents, and businesses of the Region of Peel, Ontario, to help protect the environment and drinking water sources and to reduce water use. One of Canada's fastest-growing regions, the Regional Municipality of Peel serves more than 1 million residents in Mississauga, Brampton, and Caledon. Officials expect the region's current population to increase by 230,000—23 percent—by 2015.

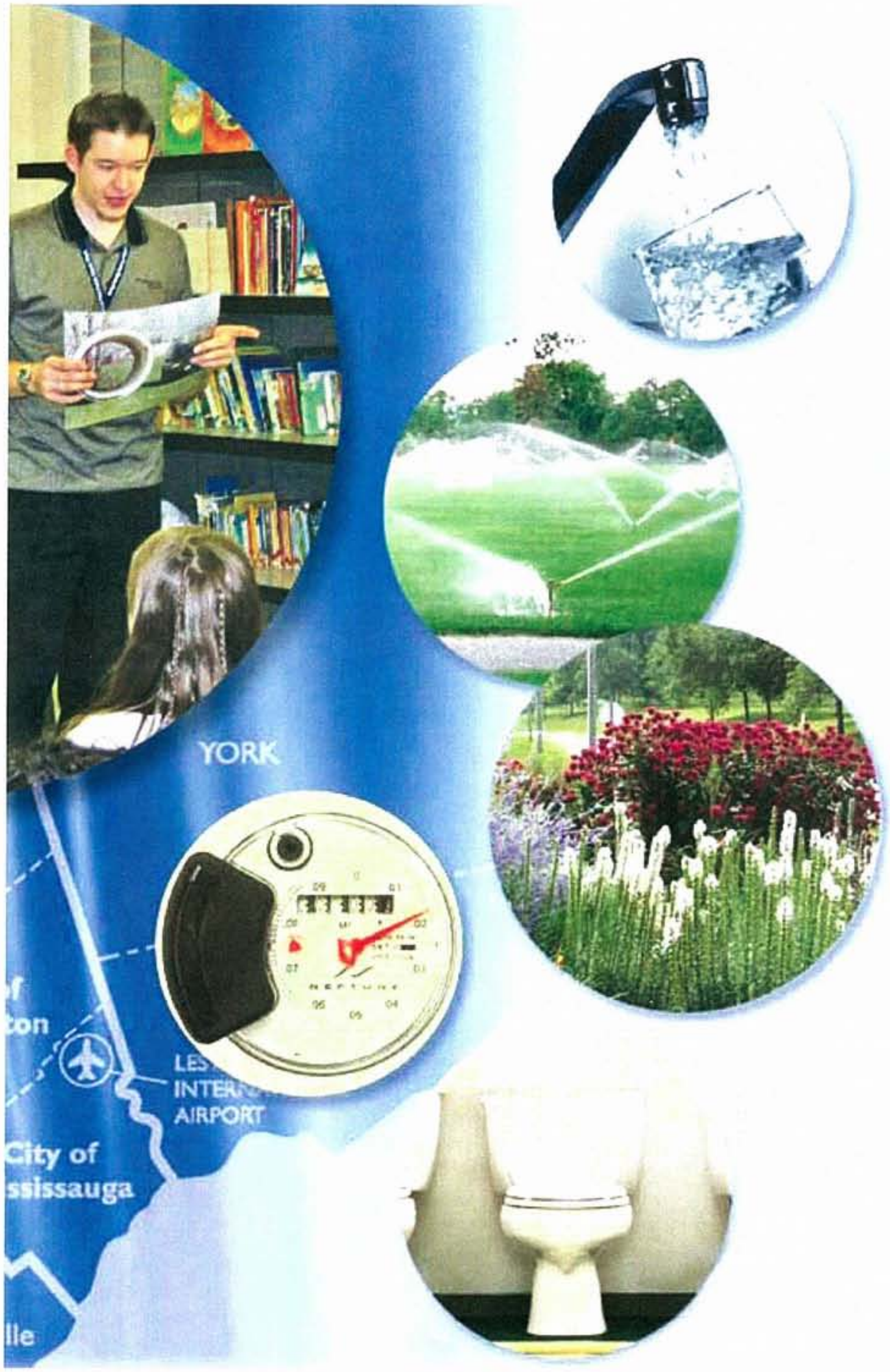
The services and programs delivered by Water Smart Peel are aimed at increasing local awareness of water- and wastewater-related issues through educational initiatives that encourage students, residents, and businesses to protect water and reduce daily water use. Incentives are also provided for Peel's citizens to adopt proven water efficiency practices.

PLAN FORMULATION AND IMPLEMENTATION

To facilitate Water Smart Peel and respond to ever-growing demands on the water supply and wastewater system, the Region of Peel developed a Water Efficiency Plan to promote more effective water use without negatively affecting service levels.

Water Demand Profile. Water use, water supply, and wastewater treatment system data were collected and analyzed to develop the WEP. Currently, when all water demands are calculated, including residential and industrial, commercial, and institutional (ICI), the overall average demand rate in the region is about 500 L/d per capita, which is equivalent to 132 gpcd. Residential demands currently range between 280 and 300 L/d per capita (74–79 gpcd), or about 58 percent of the region's total demand.

Peak Day Demand. During the warm summer months, per capita water demands increase by about 20 percent, primarily because of outdoor water use. In addition to a seasonal demand increase, peak day demand—the one day in the year when water demand exceeds average daily demand by 50 percent or more—must be accommodated. These increases are of particular concern because water treatment plants must be designed and built to meet far greater demands than are required for most days of the year. The WEP reduces summer and peak day demands.



Joe Vieira is acting supervisor, Public Education and Outreach, Regional Municipality of Peel (www.region.peel.on.ca), Brampton, Ontario, Canada.

Identifying Efficiency Measures. Because not all water-use reduction measures apply to the regional municipality, potential water efficiency measures were subject to the following screening criteria:

- Technical feasibility: Measures must be based on proven technology and experience and must reduce water demands.
- Applicability: Measures must address the region's inefficient water demands and fall within the region's jurisdiction.
- Social acceptability: Measures must satisfy the community's values and priorities.
- Cost-effectiveness: Measures must cost less to implement than infrastructure expansion to meet the same water demand, generally determined by the level of rebates offered and water savings per fixture, appliance, and action.

Recommended water efficiency measures were scheduled for implementation during the entire planning period from 2004 to 2015. Some measures were initially rolled out as pilot programs to help clarify various implementation aspects, such as participation rates and public acceptance, before moving ahead with a full program. Other measures were implemented as full programs from the onset.

PHOTOGRAPHS: REGION OF PEEL

Projected Demand Increases. Based on current projections for population growth and projected demand without the WEP, Peel anticipates:

- The average annual day demand (AADD) will increase to more than 613 mil L (162 mg), or about 18 percent during the next 10 yr from the 2003 level of 518 mil L/d (137 mgd).
- The peak day demand will increase to 932 mil L/d (246 mgd) from a 2003 level of 780 mil L/d (206 mgd), an increase of nearly 20 percent.

Goals and Objectives. The WEP goal is to reduce water use, specifically

- to reduce capital costs for new water supply and wastewater facilities by implementing water efficiency measures to reduce AADD, peak day demands, and wastewater flows.
- by 2015, to reduce AADD by 8–10 percent, from 50–60 mil L/d (13–16 mgd), reduce peak day demand by 8–10 percent, from 75–90 mil L/d (20–24 mgd), and reduce wastewater flows by 5–7 percent, from 30–40 mil L/d (8–11 mgd).
- to sustain water-use reductions for the long term and to adjust the plan as needed to achieve maximum water savings.

WEP INITIATIVES

Reducing excessive water use makes good fiscal and environmental sense, and implementing the WEP will reduce the necessary costs of water and wastewater infrastructure expansion. Regional officials estimate a cost of $112 million for infrastructure expansion to provide the equivalent supply of water and wastewater treatment that will be saved by WEP implementation (compared to $33 million estimated for implementing the plan). The following WEP measures have been put into effect.

Toilet Replacement Program. One of the greatest potential water-saving initiatives is replacing nonefficient toilets with toilets that use no more than 6 L (1.6 gal) of water to flush. The program offers rebates to residents and businesses for replacing old toilets with new, ultra-low-flow (ULF) models that flush with 6 L of water or high-efficiency toilets (HET) that flush with a maximum of 4.8 L (1.3 gal). To be eligible for rebates, applicants must be located in the Region of Peel and must purchase a water-efficient toilet from an approved list and install it within the region's jurisdiction. Approved models perform well, meet customer satisfaction, and achieve water savings.

Since November 2005, the region has offered \$60 rebates for purchasing and installing ULF toilets and \$100 rebates for HETs, which include dual-flush models. In 2006, the region issued 8,691 rebates to residents or property owners of single-family and multifamily dwellings. It's estimated that this program has saved 347 mil L/yr (92 mg/yr) of water.

In March 2007, the region launched the program for ICI customers, offering rebates of \$60, \$100, and \$140 to businesses and property owners. The \$140 rebate is reserved for customers who replace flush-valve toilets, which cost significantly more than gravity-flush toilets and are frequently used in commercial applications.

Retrofit Project. The region's Water Efficiency Retrofit Project evolved from a 2004 toilet replacement pilot project involving three nonprofit housing properties operated by the region. The substantial water savings realized in the pilot project encouraged the staff to continue retrofitting nonprofit buildings built before 1996. Sixteen buildings with 1,867 residential units were retrofitted with water efficiency fixtures in phase one, and 13 buildings with 1,680 residential units were retrofitted in phase two.


Toilets line the hall of a residential building during a retrofit project.


Regional staff provide a guided tour of Water Wise Gardens during the Lawn and Garden Consultation Program.

The retrofit project yielded an average water savings of 298 L/d/unit (79 gd/unit), or a total of 1,152 m^3/day (1,507 ft^3/day). About 59 percent of the savings results from eliminating toilet, showerhead, and faucet leakage. The remaining 41 percent of the savings represents a collective reduction in flush and flow volumes associated with installed water-efficient fixtures.

Water Audit–Capacity Buy-Back Program. The Indoor Water Audit–Capacity Buy-Back Program identifies areas of potential water savings within ICI facilities that may be achieved through permanent process changes. Starting with the 10 largest facilities, staff members are assessing these sites to identify specific water-saving opportunities. Water demand data will help staff members recommend viable measures to facility managers and owners, such as reusing water or switching to air-cooled equipment. All actual water savings achieved will be verified when the recommended measures are implemented. Water savings related to process changes must be permanent to ensure that water savings attributable to the program will benefit regional customers.

Each participating facility is eligible for a one-time rebate of \$0.25 L/d of water saved up to a maximum of \$250,000, or half the cost to implement the process change, whichever is less. This rebate pays for a portion of implementation costs and serves as an incentive for the cost–benefit analysis. In addition to incentives, participating ICI facilities continue to save money through lower water bills and utility and technical costs.

Outdoor Water Education Program. The Lawn and Garden Consultation Program addresses increasing residential outdoor water demand. During the summer, per capita water demands increase by about 20 percent, due primarily to outdoor water use for lawns and gardens.

Emphasizing behavioral change, lawn and garden consultations focus on awareness and education. During the 2006 summer campaign, two-person teams of university-trained students visited homeowners to assess the homeowners' lawns and gardens and reviewed their water use and landscape maintenance practices. The students provided advice and techniques specific to each homeowner's property to maintain a healthy lawn and garden without overwatering or applying excessive amounts of chemicals. Information packages, including outdoor water efficiency kits and literature on water efficiency practices and pesticide awareness, were provided to each home visited.

The Region of Peel developed a Water Efficiency Plan to promote more effective water use without negatively affecting service levels.

The teams completed 503 consultations, and residents provided feedback about the program's quality, particularly the level of detail and information provided. A survey of all participating households revealed that all participants found the consultations helpful.

Building on the 2006 campaign's success, 600 additional consultations were provided during summer 2007. The program was again favorably received and garnered compliments and media attention. Residents continue to be placed on a waiting list for the 2008 program.

The region also hosted successful Lawn and Garden Workshops in the summers, complementing the consultation program by helping interested residents develop beautiful lawns and gardens without excessive water and pesticide use. The workshops included guest speakers, interactive demonstrations, and tours of the region's Water Wise Gardens.

Restaurant Spray-Valve Replacement Program. Most restaurants use a spray valve to rinse food from dishes before putting them into automated dishwashers. By installing a low-flow spray valve, facilities can reduce water and natural gas consumption, as well as reduce operating costs. In October 2006, the Region of Peel and a gas utility provider launched the Spray-N-Save: Pre-Rinse Spray Valve Replacement Program. A total of 115 pre-rinse spray valves were installed in Peel-area restaurants in 2006 for a water savings of about 12.6 mil L/yr (3.3 mg/yr).

PUBLIC EDUCATION AND OUTREACH

Education and outreach programs are helping the Region of Peel involve children and adults in hands-on water experiences.

Water Treatment Facility Tours. Free guided tours of the Lorne Park Drinking Water Treatment Facility are offered to local community groups and students in grades 8 and above. Staff members lead visitors through Peel's underground drinking water treatment facility. By observing firsthand various stages of the drinking water treatment process, visitors better understand water quality and wise water use. In 2007, the staff conducted 60 tours for approximately 1,400 visitors.

Students of all ages learn about water conservation at the Peel Children's Water Festival.

Peel Children's Water Festival. Since 1999, the Peel Children's Water Festival has provided an environmental educational experience to more than 60,000 Peel elementary school students and teachers. This week-long festival has more than 50 interactive activity centers to educate students in grades 2–5 about the water cycle, water conservation, the link between water and human health, source water protection, human–water interaction, and resource planning. Developed by water specialists and educators, these activities combine Ontario curriculum requirements with hands-on learning. In 2007, 9,414 people—students, teachers, chaperones, volunteers, and members of the general public—attended the festival.

School Presentations. Free presentations complementing the Ontario curriculum are offered to Peel-area school teachers. These fun and interactive sessions address several areas of water education, including water treatment and distribution, water quality and health, wastewater collection and treatment, surface and groundwater protection, and water efficiency. In 2007, about 3,000 students in 150 classes participated in classroom presentations.

Peel Water Story. A multimedia resource, the Peel Water Story also complements the Ontario curriculum. As a cross-curricular resource consisting of a book, CD-ROM, and Web site, PWS supports existing water-focused programming for all grades and subject areas.

FUTURE SUCCESS

The long-term effectiveness and success of Water Smart Peel depend largely on water user participation and establishing sustainable behavior in favor of water efficiency and protection of drinking water sources. To date, the program's success is based on effective marketing, piloting some water efficiency measures prior to full implementation, monetary incentives, public education, and community and stakeholder collaboration. This approach will help Peel reach its water savings goals and help the region continue to meet public expectations.

APPENDIX F

Fire Underwriters Survey Guidance Document

(digital)

APPENDIX I

Environment Canada Municipal Water and Wastewater Survey

(2009 data and 2011 report provided in digital form)

Appendix I

Review and Analysis of the Results of the 2009 Municipal Water and Wastewater Survey

Introduction

The results of the *2009 Municipal Water and Wastewater Survey* (2009 MWWS) are available online in Excel format as well as in a summary report entitled *2011 Municipal Water Use Report – Municipal Water use 2009 Statistics*. Both were released to the public in October of 2011 and as a result were not used during the development of the preliminary drafts of this report. In an effort to respond to concerns raised by some reviewers, a detailed evaluation of the survey results was undertaken by CBCL. The results are summarized in this appendix. Additional information can be found in the Environment Canada summary report and Excel spreadsheets, which are provided in digital format on the data CD included with this report.

Accuracy and Data Validation

A preliminary review of the data available online from the 2009 MWWS revealed a number of potential inaccuracies related to entries from Newfoundland and Labrador. These include a reported total per capita water demand of 17,684 Lpcd for the community of Cow Head and identical population and total per capita water demand values for Baine Harbour. The community of Kippens is only included in the residential water use dataset, where it is assigned a demand of 0 though its population is included in the calculation of the average total per capita water demand.

Environment Canada was contacted and asked to provide comment on these potential inaccuracies. According to their representative, the water use value from Cow Head, which was carried forward from 2006, was confirmed during the validation process that followed the data collection phase of the MWWS (personal communication, November 21, 2011). No comment was provided on Baine Harbour or Kippens. A follow-up call was made to the town clerk in Cow Head, who reported that the town's flow meter has not been working correctly for a number of years and is in the process of being replaced. She also indicated that she knew of no reason why the per capita water use value would be so high as the town does not use an excessive amount of water (personal communication, November 22, 2011).

The full dataset for Newfoundland and Labrador is presented in Table I.1. Table I.2 is a corrected version of the same dataset that does not include Cow Head or Kippens. Removing Baine Harbour from the analysis did not affect the average total or residential per capita water demand values so the town was kept in the table.

Note that only two communities that submitted flow data to CBCL for the *Study on Water Quality and Demand on Public Water Supplies with Variable Flow Regimes and Water Demand* also submitted information for the 2009 MWWS. The per capita water demands reported in the MWWS do not match up with those calculated during this study using multiple years of water use records. This highlights the difficulties inherent in setting a default per capita water demand value for system designers. Nonetheless, the *2009 Municipal Water and Wastewater Survey* includes reasonable water use estimates from 23 communities in Newfoundland and Labrador and, at present, represents the best publically available information on water use in the province.

Calculation of Per Capita Water Demands

On the 2009 MWWS survey communities were asked to provide an estimate of the total amount of water used in the past year and to describe the estimation method used to come to that value. The average total daily per capita water demand reported in the 2011 summary report for Newfoundland and Labrador was calculated by dividing the sum of the total water use values reported by all of the communities by the total surveyed population.

$$\text{Average Day Per Capita Water Demand (Lpcd)} = \left[\frac{\sum_{i=1}^{n} \text{Total Annual Water Use}_i (m^3)}{\sum_{i=1}^{n} \text{Population}_i}\right] \times 1000 \left(\frac{L}{m^3}\right) \times \frac{1}{365 \left(\frac{\text{days}}{\text{year}}\right)}$$

The resulting value represents the average water use per person in the province (ie. population = total number of individual persons living in the 24 participating communities). Thus, it predicts the most likely water demand exerted by any given resident of the province. In Table I.1 the average total per capita water demand for the province is 804 Lpcd. This value may not accurately reflect the per capita water use in smaller communities as the inputs from the City of St. John's, which has a large population, water meters, and a relatively low per capita water demand, overwhelm the inputs from other communities.

Figure I.1 is a histogram that shows the likelihood that an individual person is exerting a given water demand based on the results of the 2009 MWWS. Note that the values have been divided into bins (0 – 250, 250 – 500, etc.) and all values that fall below the top limit of the bin are included in the column that rises from that value on the graph.

Individual per capita water demand values are also provided in the 2009 MWWS for each of the responding communities. The average of these individual values provides an estimate of the per capita water use of the surveyed communities (ie. population = 24).

$$\text{Average Day Per Capita Water Demand (Lpcd)} = \left[\frac{\sum_{i=1}^{n} \text{Per Capita Water Use}_i \text{ (Lpcd)}}{\text{Number of Communities}}\right]$$

Figure I.2 is a histogram that shows the number of communities that reported per capita water demands falling into a series of bins between 0 and 3,000 Lpcd.

The average of the individual per capita values obtained during the 2009 MWWS is 1,120 Lpcd (Table I.2). This value overestimates the total per capita water demand exerted by any given resident of the province but may provide a good estimate of the total per capita water demand that can be expected in any given community.

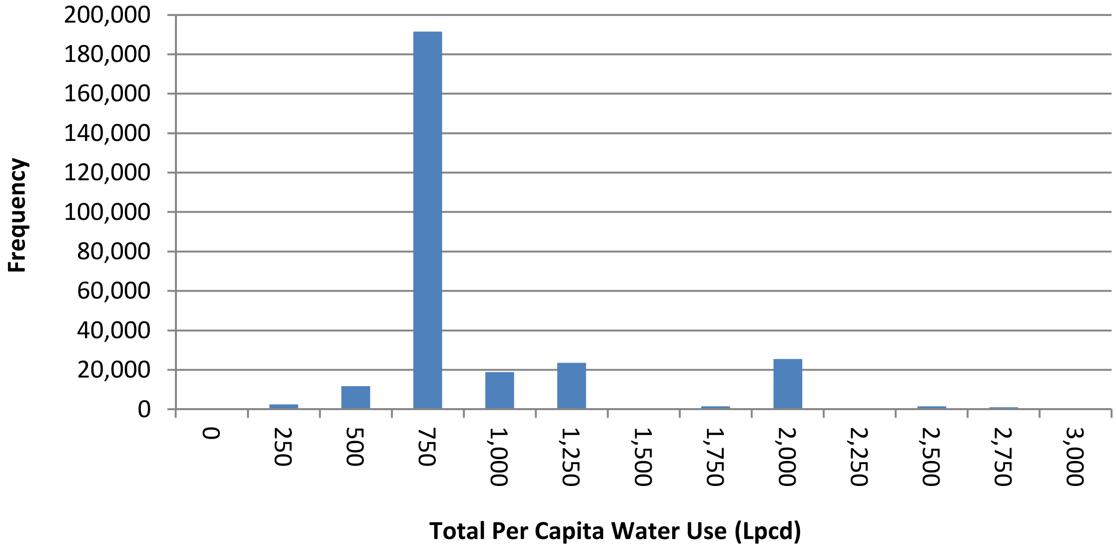


Figure I.1 **Histogram showing the likelihood that a given person is exerting a given water demand**

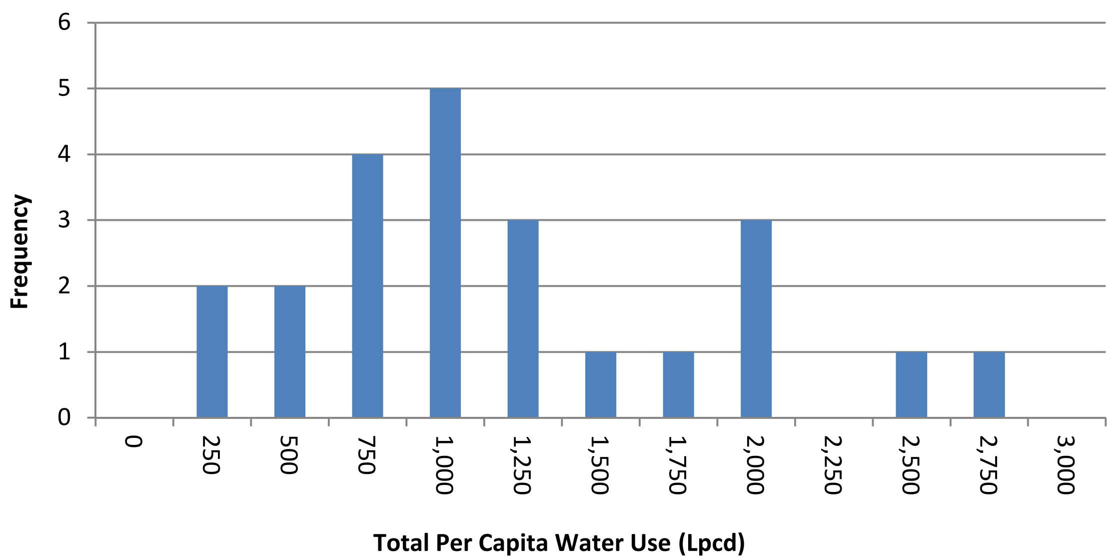


Figure I.2 **Histogram showing the likelihood that a given community has a given per capita water demand**

Residential Demands

The 'residential only' per capita values provided in the 2009 MWWS summary report were calculated by multiplying the reported total annual water use by the percentage of total water use reported to be 'residential'. As shown in Table I.1, the average residential per capita water use value for the province of Newfoundland and Labrador is 395 Lpcd. The lowest reported value is 126 Lpcd and the highest is 1,698 Lpcd. The corrected table (I.2) yields an average value of 398 Lpcd, which is essentially equal to that from Table I.1. Neither value is much higher than the current ENVC-recommended residential per capita water demand value of 340 Lpcd.

Recommendations

The ENVC may choose to adopt different per capita water demand values for communities of different sizes and/or those with different distributions of users (ie. industrial vs. residential only). Preferably, however, designers should be required to monitor flow before sizing system components. This is especially true for communities with industrial users. Potential per capita values are summarized in Table I.3.

Table I.3 Summary of potential per capita water use values

Value	Source	Advantages	Disadvantages
340 Lpcd	Current NL Design Guidelines	• In use	• Original source unknown
450 Lpcd	Previous NL CT Calculator	• Previously in use to account for winter demands	• Original source unknown
395 Lpcd	2009 MWWS – Residential Only	• Based on recent water use estimates from 20+ communities	• Biased towards larger communities • Data accuracy and validation are non-ideal
398 Lpcd	2009 MWWS – Res. Only – Corrected*	• Based on recent water use estimates from 20+ communities	• Biased towards larger communities • Data accuracy and validation are non-ideal
651 Lpcd	2009 MWWS – Res. Only – Corrected**	• Based on recent water use estimates from 20+ communities	• Data accuracy and validation are non-ideal • Biased towards smaller communities
804 Lpcd	2009 MWWS – Total	• Based on recent water use estimates from 20+ communities	• Biased towards larger communities • Data accuracy and validation are non-ideal
774 Lpcd	2009 MWWS – Total – Corrected*	• Based on recent water use estimates from 20+ communities	• Biased towards larger communities • Data accuracy and validation are non-ideal
1,120 Lpcd	2009 MWWS – Total – Corrected**	• Based on recent water use estimates from 20+ communities	• Data accuracy and validation are non-ideal • Biased towards smaller communities

*Corrected by CBCL Limited November 2011

**Per capita value calculated by averaging per capita values from 23 communities

Additional Information

Additional information, including the questionnaire used for the 2009 MWWS, can be found at:

http://ec.gc.ca/eau-water/default.asp?lang=En&n=ED0E12D7-1

The summary report, *2011 Municipal Water Use Report – Municipal Water use 2009 Statistics,* and the associated Excel file have been provided on the data CD that accompanies this report.

Table I.1

	Total Water Use				Residential Water Use				
Community	Source	Responding Population	Annual Volume	Per Capita	Community	Source	Responding Population	Annual Volume	Per Capita
			m^3/year	Lpcd				m^3/year	Lpcd
St. John's	MWWS2009	180,000	35,500,000	540	St. John's	MWWS2006	99640	5,818,095	160
Corner Brook	MWWS2009	20,050	14,293,017	1,953	Corner Brook	MWWS2009	20050	7,146,509	977
Grand Falls-Windsor	MWWS2009	13,616	5,308,000	1,068	Grand Falls-Windsor	MWWS2009	13616	3,184,800	641
Gander	MWWS2009	10,000	2,731,518	748	Gander	MWWS2009	10000	1,447,705	397
Paradise	MWWS2009	11,550	1,897,161	450	Paradise	MWWS2009	11550	1,328,013	315
Happy Valley-Goose Bay	MWWS2006	7,572	**3,250,000**	1,176	Happy Valley-Goose Bay	MWWS2006	7572	1,800,622	652
Stephenville	MWWS2009	7,500	2,731,741	998	Stephenville	MWWS2009	7500	2,185,393	798
Marystown	MWWS2006	5,382	**1,909,320**	972					
Deer Lake	MWWS2009	4,782	1,672,597	958	Deer Lake	MWWS2009	4782	1,087,188	623
Carbonear	MWWS2009	4,818	3,385,019	1,925					
Wabana	MWWS2009	2,389	189,271	217	Wabana	MWWS2009	2389	123,026	141
Twillingate	MWWS2009	2,341	991,048	1,160	Twillingate	MWWS2009	2341	416,240	487
Burgeo	MWWS2009	1,537	970,495	1,730	Burgeo	MWWS2009	1537	776,396	1,384
Baie Verte	MWWS2009	1,206	322,572	733	Baie Verte	MWWS2009	1206	177,415	403
Harbour Main-Chapel's Cove	MWWS2009	466	133,575	785	Harbour Main-Chapel's Cove-La	MWWS2009	466	83,339	490
Rocky Harbour	MWWS2009	978	954,679	2,674	Rocky Harbour	MWWS2009	978	429,606	1,203
Carmanville	MWWS2009	678	202,824	819	Carmanville	MWWS2009	678	192,683	778
Old Perlican	MWWS2009	660	454,609	1,886	Old Perlican	MWWS2009	660	409,148	1,698
Cow Head	MWWS2009*	493	**3,182,200**	17,684					
Trinity Bay North	MWWS2009	1,506	1,324,894	2,410	Trinity Bay North	MWWS2009	1506	463,713	844
St. Pauls	MWWS2009	297	68,218	629	St. Pauls	MWWS2009	297	47,753	441
Baine Harbour**	MWWS2006	126	**5,808**	126	Baine Harbour	MWWS2006	126	5,808	126
Sunnyside	MWWS2009	411	198,961	1,325	Sunnyside	MWWS2009	411	96,695	644
Brighton	MWWS2009	191	33,186	476	Brighton	MWWS2009	191	32,523	467
					Kippens	MWWS2009	1739	-	-
Average		11,606	3,404,613	1,810			8,602	1,238,758	621
Total		278,549	81,710,713	**804**			189,235	27,252,668	**395**
Maximum		180,000	35,500,000	17,684			99,640	7,146,509	1,698
Minimum		126	5,808	126			126	-	-
Number		24	24	24			22	22	22

**I think this should be 2006*
***identical pop and average water use seems questionable*

Published Value

Calculated Value

Table I.2

	Total Water Use				Residential Water Use				
Community	Source	Responding Population	Annual Volume	Per Capita	Community	Source	Responding Population	Annual Volume	Per Capita
			m^3/year	Lpcd				m^3/year	Lpcd
St. John's	MWWS2009	180,000	35,500,000	540	St. John's	MWWS2006	99,640	5,818,095	160
Corner Brook	MWWS2009	20,050	14,293,017	1,953	Corner Brook	MWWS2009	20,050	7,146,509	977
Grand Falls-Windsor	MWWS2009	13,616	5,308,000	1,068	Grand Falls-Windsor	MWWS2009	13,616	3,184,800	641
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Paradise	MWWS2009	11,550	1,897,161	450	Paradise	MWWS2009	11,550	1,328,013	315
Happy Valley-Goose Bay	MWWS2006	7,572	**3,250,000**	1,176	Happy Valley-Goose Bay	MWWS2006	7,572	1,800,622	652
Stephenville	MWWS2009	7,500	2,731,741	998	Stephenville	MWWS2009	7,500	2,185,393	798
Marystown	MWWS2006	5,382	**1,909,320**	972					
Deer Lake	MWWS2009	4,782	1,672,597	958	Deer Lake	MWWS2009	4,782	1,087,188	623
Carbonear	MWWS2009	4,818	3,385,019	1,925					
Wabana	MWWS2009	2,389	189,271	217	Wabana	MWWS2009	2,389	123,026	141
Twillingate	MWWS2009	2,341	991,048	1,160	Twillingate	MWWS2009	2,341	416,240	487
Burgeo	MWWS2009	1,537	970,495	1,730	Burgeo	MWWS2009	1,537	776,396	1,384
Baie Verte	MWWS2009	1,206	322,572	733	Baie Verte	MWWS2009	1,206	177,415	403
Harbour Main-Chapel's Cove	MWWS2009	466	133,575	785	Harbour Main-Chapel's Cove-La	MWWS2009	466	83,339	490
Rocky Harbour	MWWS2009	978	954,679	2,674	Rocky Harbour	MWWS2009	978	429,606	1,203
Carmanville	MWWS2009	678	202,824	819	Carmanville	MWWS2009	678	192,683	778
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Trinity Bay North	MWWS2009	1,506	1,324,894	2,410	Trinity Bay North	MWWS2009	1,506	463,713	844
St. Pauls	MWWS2009	297	68,218	629	St. Pauls	MWWS2009	297	47,753	441
Baine Harbour**	MWWS2006	126	**5,808**	126	Baine Harbour**	MWWS2006	126	5,808	126
Sunnyside	MWWS2009	411	198,961	1,325	Sunnyside	MWWS2009	411	96,695	644
Brighton	MWWS2009	191	33,186	476	Brighton	MWWS2009	191	32,523	467
Average		12,089	3,414,283	1,120			8,928	1,297,746	651
Total		278,056	78,528,513	**774**			187,496	27,252,668	**398**
Maximum		180,000	35,500,000	2,674			99,640	7,146,509	1,698
Minimum		126	5,808	126			126	5,808	126
Number		23	23	23			21	21	21

***identical pop and average water use seems questionable*

Published Value

Calculated Value

APPENDIX J

Photographs

(digital)

APPENDIX K

Chlorine Residual Records

(digital)