Water System Master Plan

Town of Paradise

143084.00 ● Final Report ● March 2017

ISO 9001
Registered Company

Prepared for:

Prepared by:

CBCL LIMITED
Consulting Engineers

Final Report	G. Sheppard	3/13/17	G. Sheppard
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CBCL LIMITED
Consulting Engineers

CBCL LIMITED
Consulting Engineers

March 13, 2017

Mr. Ron Fleming, CET
Director of Public Works
Town of Paradise
28 McNamara Drive
Paradise, NL
A1L 0A6

Dear Mr. Fleming:

RE: Water System Master Plan

CBCL Limited is pleased to provide the Town of Paradise with our Final Report for the Water System Master Plan. We have enjoyed this interesting project and look forward to working with the Town in the implementation of the infrastructure recommendations.

Yours very truly,

CBCL Limited

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EXECUTIVE SUMMARY

The Town of Paradise retained CBCL Limited (CBCL) in August, 2014 to develop a water master plan (WMP). This WMP outlines short and long-term strategies for addressing existing and future water infrastructure requirements.

The study objectives are summarized as follows:

- To develop baseline water system information;
- To develop future water demand projections based on population projections and proposed land developments;
- To assess the ability of the water system to deliver water to all parts of the Town while meeting provincial guidelines;
- To evaluate operational procedures;
- To recommend system improvements; and
- To develop a prioritised capital works list.

Detailed analysis of the existing and future water demands, existing infrastructure and current operational procedures resulted in the following list of recommendations:

1. The existing 150 mm water main in the Evergreen Village Trailer Park should be abandoned and replaced, such that it is realigned within the road right-of-way.
2. A new storage tank should be constructed at the high point of Neils Pond Ridge to address the pressure deficiencies experienced at the higher elevations. Pressure zone PA-A should be removed from the Southlands Reservoir and serviced by the new tank. The proposed Karwood Drive development and Fairview Investments development should also be serviced by the new tank.
3. In addition to the new storage tank and proposed service areas, the Town should adopt a policy to limit the future development to the 180 m contour. This policy is only effective if the new tank and proposed service areas scheme are implemented.
4. The Town should check the isolation valves that define PA-H to ensure that they are closed. If they are all closed, the Town should check for leaks in PA-H.
5. The Town should, where possible, install water main connections that result in looping. The effort should focus on cul-de-sacs and unconnected streets.
6. The Town should consider completing a water audit.
7. The Town should consider implementing universal metering.

8. The Town should start a valve maintenance program and develop a valve record in GIS.
9. The Town should develop a GIS database to record information on the water infrastructure.
10. The Town should monitor ammonia, nitrate, nitrite, di- and tri-chloramines, and NDMA, which are undesirable by-products of the chloramination process.
11. The Town should continue with their residual chlorine monitoring program and conduct scheduled flushing activities to improve water age and hence water quality.

The capital costs associated with recommendations 1 and 2 above are presented in the following table:

Description	Cost
1. Evergreen Village Water Main Upgrades	$4,152,000
2. Neil's Pond Ridge Reservoir	$8,800,000

CHAPTER 1 INTRODUCTION

The Town of Paradise retained CBCL Limited (CBCL) in August, 2014 to develop a water master plan (WMP). This WMP outlines short and long-term strategies for addressing existing and future water infrastructure requirements.

1.1 Background

Located on the northeast Avalon, the Town of Paradise is one of the fastest growing municipalities in Newfoundland and Labrador. Incorporated in 1992, it comprises the former towns of Paradise and St. Thomas, as well as the developed areas of Three Island Pond, Topsail Pond, Evergreen Village and Elizabeth Park.

The rapid rise in population has resulted in increased pressure on Paradise’s municipal water system. In order to provide a reliable level of service to their existing customers and to make informed decisions regarding proposed developments, Paradise needs well-documented planning tools. The WMP will address this need with respect to the water system.

1.2 Study Objectives

The study objectives are summarized as follows:

- To develop baseline water system information;
- To develop future water demand projections based on population projections and proposed land developments;
- To assess the ability of the water system to deliver water to all parts of Town while meeting provincial guidelines;
- To evaluate operational procedures;
- To recommend system improvements; and
- To develop a prioritised capital works list.

CHAPTER 2 EXISTING WATER SYSTEM

Paradise obtains potable water from the Regional Water System (RWS), which is owned and operated by the City of St. John's. The RWS committee, made up of representatives from each municipality serviced by the RWS, regularly meet to discuss operational issues. Key system components include:

- Bay Bulls Big Pond (BBBP) water treatment plant (WTP).
- The Ruby Line, Kenmount and Paradise pumping stations.
- The Mundy Pond, Kenmount Hill, Southlands, Fowler's Road, Camrose Drive and Skinner's Road reservoirs. (The reservoirs are operated by the City of St. John's; however, they are owned by the municipalities in which they are located).
- Some of the transmission mains that connect the above-listed components.
- Meter and valve chambers that are used to measure municipal flow rates, isolate sections of the transmission mains, and control system pressures.

2.1 Source

Paradise's water distribution system is supplied by the BBBP WTP, which has a capacity of 85,000 m^3/d and services the municipalities of Conception Bay South (CBS), Portugal Cove – St. Philip's, Mount Pearl, and portions of St. John's, in addition to Paradise.

2.2 Piping

The Paradise water distribution system consists of approximately 104 km of transmission and distribution water main ranging in size from 150 mm to 600 mm diameter. The first pipelines were installed in 1982. Most of the water main material is ductile iron. The remaining pipe material is PVC. Table 2.1 presents a summary of the existing pipelines by size and material.

Table 2-1 - Existing Pipe Size and Material Type

Criterion	% of Total Pipeline Length
Size	
150mm diameter or less	8
200mm to 300mm diameter	52
300mm diameter or greater	40
Material Type	
Ductile iron	96
PVC	4

2.3 Pumping Stations

Two pumping stations service the Town, including the Paradise and Donna Road pumping stations (PS). The Paradise PS is part of the RWS and is located on Topsail Road. Its purpose is to transfer water from a transmission main in Topsail Road to the Camrose Drive reservoir. Therefore, operation of the pumps is controlled by the water level in Camrose Drive reservoir. The Donna Road PS is also located on Topsail Road approximately 80 m west of the Paradise station and provides constant pressure to an area above the 190 m contour between Trails End Drive and Clearview Heights. The pumping stations are further described in Table 2.2.

Table 2-2 - Existing Pumping Stations

Station ID	Location	Suction HGL (m)	Discharge HGL (m)	TDH (m)	Discharges to
P-1	Topsail Rd.	210.8	224.2	13.4	Camrose Drive Reservoir
P-2	Donna Rd.	213.2	253.4	42.3	PA-H

2.4 Storage Reservoirs

There are two storage reservoirs that feed the Paradise water system: the Southlands Reservoirs, located at Reservoir Road in St. John's, and the Camrose Drive reservoir on Camrose Drive. Both are part of the RWS. The reservoir storage volumes and configurations are summarized in Table 2.3.

Table 2-3 - Existing Storage Reservoirs

Reservoir ID	Reservoir Name	Volume (m^3)	Diameter (m)	TWL (m)	Floor Level (m)	Water Height (m)
R-1	West Southlands	10,500	36.3	218.0	207.9	10.1
	East Southlands	9,100	35.5	218.0	207.8	10.2
R-2	Camrose Drive	10,540	37.3	218.1	208.5	9.6

2.5 Pressure Reducing Valve Stations

There are six pressure reducing valve (PRV) stations in the Paradise system. Four are located on the 300 mm transmission main on St. Thomas Road, between Ashlen Crescent and Atlantica Drive. The two remaining PRV stations are installed in parallel on Duffs Crescent and Topsail Road and serve to limit the pressure in PA-C. The existing PRV stations are summarized in Table 2.4.

Table 2-4 - Existing Pressure Reducing Valve Stations

Station ID	Location	Main Valve Size (mm)	Main Valve Centreline Elevation (m)	Inlet HGL (m)	Outlet HGL (m)
PRV-1	Duffs Crescent	200	127.8	218	153
PRV-2	Topsail Road	300	123.0	218	153
PRV-3	233 St. Thomas Line	300	144.7	218	218
PRV-4	377 St. Thomas Line	300	116.8	218	158
PRV-5	442 St. Thomas Line	300	85.0	158	158
PRV-6	555 St. Thomas Line	300	56.5	158	105

2.6 System Operation

Paradise is a part of the BBBP service area. Water from the BBBP WTP is pumped to the Ruby Line clearwell and from there it is pumped in two directions: north toward St. John's and west toward Paradise, Mount Pearl, CBS and Portugal Cove-St. Philip's. The Southlands and Camrose Drive Reservoirs provide storage for Paradise. Existing pumping stations, storage reservoirs, PRV stations and pressure zones are shown on Figure 2.1.

The Southlands Reservoir feeds pressure zone PA-A, which includes Elizabeth Park, Karwood Drive and adjacent streets. Water is pumped to the Camrose Drive Reservoir by the Paradise PS. The Camrose Drive Reservoir feeds pressure zones PA-B to PA-G as well as Portugal Cove-St. Philip's. PA-C through PA-G are separated by pressure reducing valves (described in Section 2.4). The Donna Road pumping station provides pressure to zone PA-H.

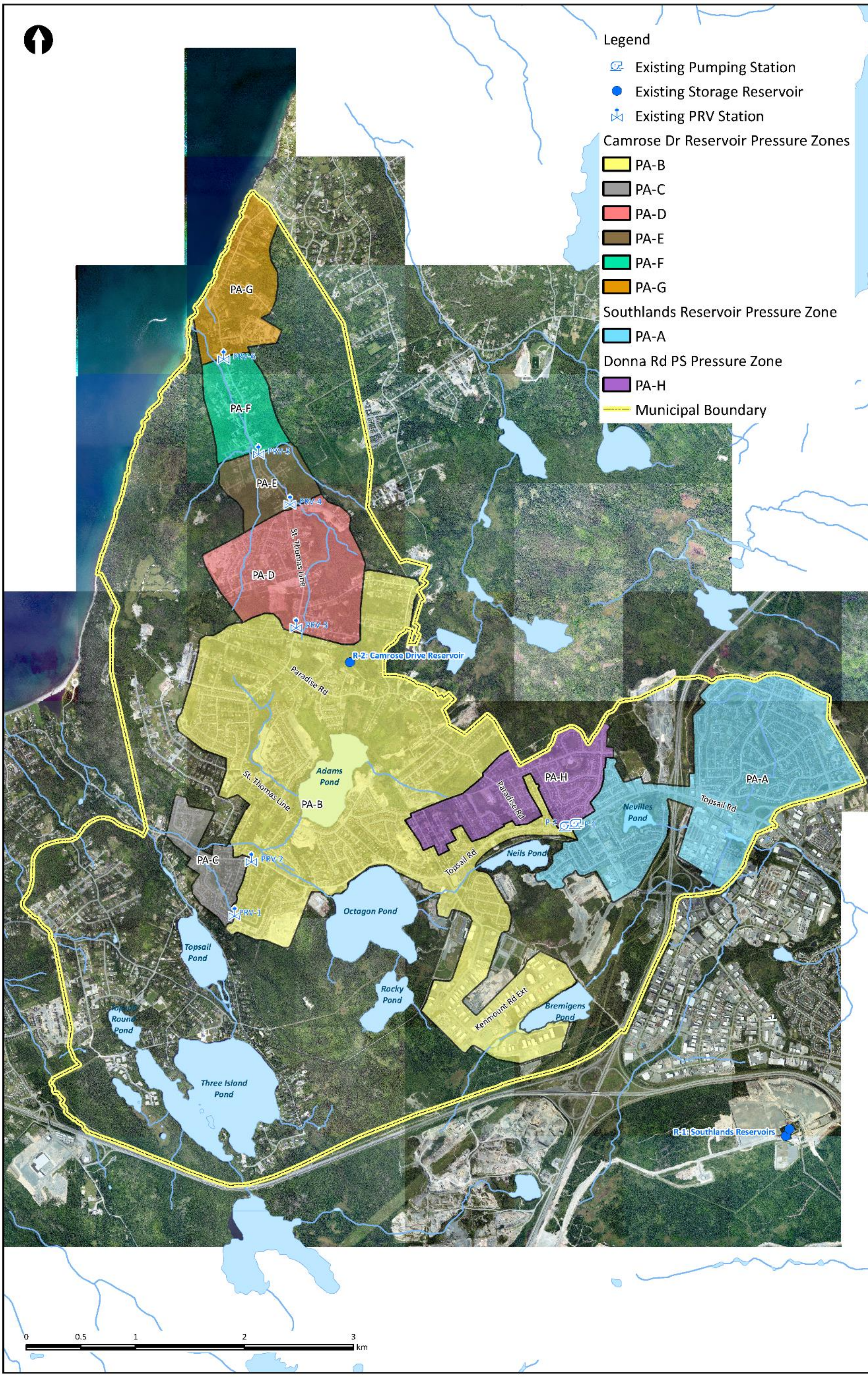


Figure 2-1 - Existing System

CHAPTER 3 WATER DEMANDS

3.1 Historical Water Use

The Town of Paradise is serviced by the Bay Bulls Big Pond (BBBP) Water Treatment Plant (WTP). Measured water flows for 2010-2014 were obtained from the City of St. John's (City). Total daily demands for BBBP were reviewed in consultation with City officials and the average and maximum day demands for Paradise were summarized.

- *Average Day Demand*: Total system water use for one year, divided by 365 days.
- *Maximum Day Demand:* Water use over the 24-hour period (midnight to midnight) with the highest demand during the year.

The average day and maximum day demands along with the average day per capita water consumption rates are presented in Table 3.1. These consumption rates are similar to jurisdictions that do not have universal metering. Table 3.2 shows consumption rates for other local municipalities.

Table 3-1 - Measured Water Flows

<table>
<tr><th>2011 Population</th><th rowspan="2" colspan="2">Serviced Population</th><th colspan="2">Measured Flow - m³/d</th><th rowspan="2">Max. Day Factor</th><th>Average Day Water Consumption Rate</th></tr>
<tr><th>(From Population Projections, Sec. 3.2)</th><th>Average Day Demand</th><th>Max. Day Demand</th><th>L/person/d</th></tr>
<tr><td>17,550</td><td>13,163</td><td>75%</td><td>9,689</td><td>12,243</td><td>1.3</td><td>736</td></tr>
</table>

Table 3-2 - Comparison of Average Daily Consumption Rates

Region	Average Daily Consumption Rates (L/person/d)
St. John's	790
Mount Pearl	977
Portugal Cove – St. Philip's	823
Conception Bay South	697
Happy Valley - Goose Bay	687
Newfoundland and Labrador	804*
Canada	510**

*Consumption rates are for total water usage (includes residential, commercial, industrial and institutional usage).

**2011 Environment Canada Municipal Water Use Report.

3.2 Population Projections

As part of the *"St. John's Regional Drinking Water Study",* an integrated demographic analysis was used to forecast housing demands and associated populations in the St. John's Urban Region, for 10, 25 and 35-year periods. The analysis takes into account the current age structure in the regions and includes allowances for migration. Low, median and high growth models were used, as follows:

- *Low Growth*: population with decreasing birth rates and increasing longevity, requiring positive net-migration to sustain and grow its existing population numbers;
- *Median Growth*: population with higher birth rates and increasing longevity and sustains existing population numbers; and
- *High Growth*: population with high birth rates and high death rates.

In order to classify the growth potential of a population, estimates were carried out for population age cohort groups. (Cohort: a group of persons sharing demographic characteristics). For example; the younger cohort groups for the ages 0-4 to 25-34 represent the future population potential of a region. The forecast population is an integration of a starting point (2011), births (fertility by age), deaths by age, and in and out migration trends. After the population is forecasted for a time period, housing demand estimates are made based on the forecast population by age cohort and historical occupancy statistics from Statistics Canada.

Table 3.3 presents a summary of the population projections for the median growth scenario for Paradise.

Table 3-3 - Populations Projections – From St. John's Regional Drinking Water Study

Census Population	Observed Population	Projected Population - Median Growth		
2011	**2011**	**2021**	**2036**	**2046**
17,695	17,550	20,261	22,662	23,671

In Table 3.3, the census population figures for 2011 include long-term care facility residents, whereas the observed population figures for 2011 do not included these persons. The observed population figures are used as the basis for the population projections. Discounting of the long-term care facility residents does not significantly impact the projections because the number of long-term care facility residents remains fairly consistent (i.e. the number of long-term care facility residents will only increase if additional facilities are built).

Through discussions with Town officials, the population projections in Table 3.3 appear to be low, as they have estimated the current population to be roughly 21,000. However, Town officials agree with the estimated percent increases in population. Therefore, the population projections were edited to include the Town's estimate of current population, and are presented in Table 3.4. The population projections in Table 3.4 have been used in this study. According to Statistics Canada, the next census of population will be conducted in 2016. The Town can then confirm the current population estimate with the census.

Table 3-4 - Populations Projections – Based on Town of Paradise Estimates

Census Population	Observed Population	Town's Estimate of Current Population	Projected Population - Median Growth		
2011	2011	2014	2021	2036	2046
17,695	17,550	21,000	24,360	27,090	28,350

3.3 Water Demand Projections

The population forecasts, summarized in Table 3.4, were used to estimate the future maximum day water demands for 2026, 2036, and 2046. Table 3.5 presents a summary of the projected maximum day water demands. For the purposes of this study, the following assumptions were made in calculating the estimated demands:

- Average daily residential water consumption is 500 L/person/d.
- Residential population density of 40 people/ha.
- People per household of 3.0.

Approximately 390 ha have been identified for future development (note that this approximation does not include land above the 180m contour in the Picco Ridge area). Assuming full build out by 2046, the increase in population between 2014 and 2046 is 7,350 people, which equates to approximately 19 people/ha. A density of 19 people/ha was assumed to be too low since parts of the proposed developments will include commercial areas as well as multi-unit residences. Therefore, an evaluation of existing residential developments, for example the Trails End and Elizabeth Park subdivisions, was conducted. Taking small sample areas, considering the number of single family homes in each sample area, and assuming 3 people per home indicates that a population density of 40 people per hectare is a reasonable assumption.

In addition to the projected population increases, allowances for servicing existing unserviced populations in Paradise were included. Based on input provided by the Town, it was estimated that approximately 200 currently unserviced houses will be connected, all by 2036.

The water demand projections presented in Table 3.5 were taken from the *St. John's Regional Drinking Water Study*. As mentioned in Section 3.2, the population projections for Paradise have been changed from those used in the *St. John's Regional Drinking Water Study* to include the Town's estimate of the current population. However, Town officials agreed that the percent increase in population is reasonable and, since the current maximum day demand is based on actual measured flow from BBBP, the estimated future maximum day demands presented in Table 3.5 are considered reasonable.

Table 3-5 - Projected Water Demands

2014 Total Max. Day Demand*	Projection Year					
	2021		2036		2046	
	Percent Increase	2021 Total Max. Day Demand	Percent Increase	2036 Total Max. Day Demand	Percent Increase	2046 Total Max. Day Demand
m^3/d	2014 to 2021	m^3/d	2014 to 2036	m^3/d	2014 to 2046	m^3/d
12,243	16%	14,165	29%	15,767	35%	16,524

*Measured maximum day demand for 2014

The future demands were allocated for known developments in the Octagon Pond (Fairview Investments), Karwood Drive, and Picco Ridge areas. These areas are identified on Figure 3.1. It was assumed that 90% of the population increase would occur in these new developments, while the remaining 10% would take place in infill areas, particularly around Adams Pond and Neil's Pond.

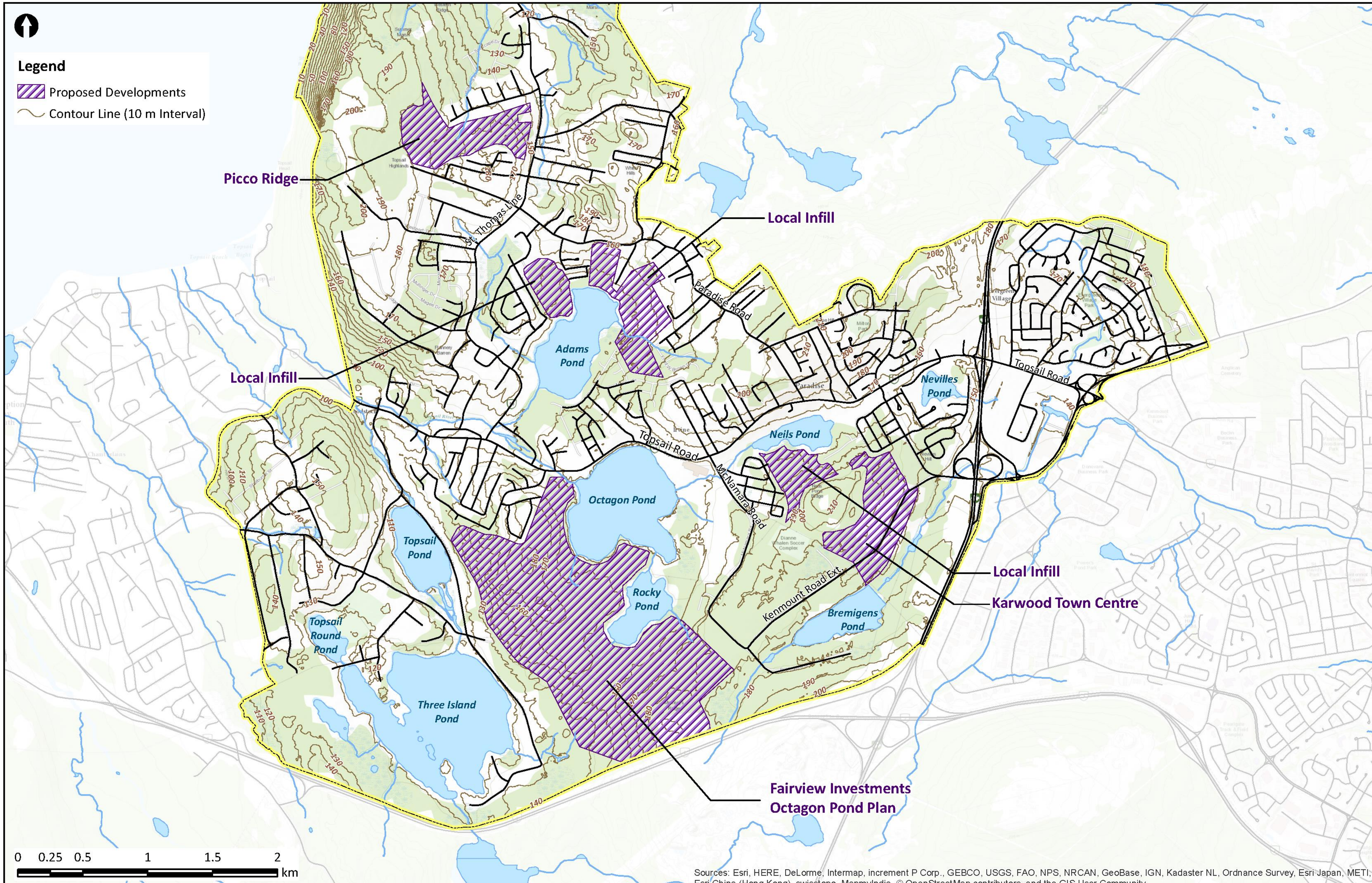


Figure 3-1 - Proposed Developments

CHAPTER 4 HYDRAULIC ANALYSIS

4.1 Approach

The distribution system analysis included assessing the existing pumping stations, transmission and distribution mains and storage tanks under the existing and future demand conditions presented in Section 3.3. Fire flow and storage requirements were also assessed. Proposed system upgrades were developed for present and anticipated future deficiencies in the current distribution system.

4.2 Design Criteria

The design criteria used in the hydraulic analysis are listed below. Additional design parameters, including pipe roughness coefficients, were sourced from the *"Guidelines for the Design, Construction and Operation of Water and Sewerage Systems"* (Guidelines) published by the provincial Department of Environment and Conservation, and other literature references.

- Minimum design pressure: 28 metres (40 psi)
- Maximum design pressure: 63 metres (90 psi) in accordance with the Guidelines
- Maximum day flows: From flow data (Section 3)
- Maximum day factor: From flow data (Section 3) and compared to the Guidelines
- Fire flow pressure requirements:
    - Demand: Maximum day flows plus fire flow
    - Residual pressure: 15.5 metres (22 psi)
- Maximum design distribution main velocity: 1.5 m/s
- Maximum design distribution main velocity (for fire flows): 3.0 m/s
- Transmission main velocity range: 0.3 to 0.6 m/s
- Minimum suction pressure at booster pumps: 21 metres (30 psi)

Distribution storage was assessed using the following equation:

$$S = A + B + C$$

Where, S = Storage (m^3)

A = Fire storage (fire flow over specified duration – dependent on type of construction)

B = Peak balancing storage (25% of maximum day demand)

C = Emergency and maintenance storage (25% of A + B)

The key considerations for assessing distribution storage opportunities include:

- Size and locate new storage reservoirs that can provide service to multiple pressure zones.
- Locate a sufficient number of new reservoirs so that the combination of new and existing tanks adequately services, by gravity, the largest area feasible.
- Ensure tanks have sufficient water turnover capability to address potential water quality problems.
- Expand service areas to include adjacent areas (where hydraulically feasible) if the existing tank water volumes have excess capacity for the present areas serviced.

4.3 Computer Water Modelling

To facilitate the analysis, the water distribution system was modeled using Innovyze's InfoWater software. InfoWater is a water distribution system modeling and management software which uses a geographic information system (GIS) interface. A water system model is developed by inputting distribution system components (pipes, reservoirs/tanks, pumps, PRVs, etc.) and demands into the software. Additional steps, including calibration and sensitivity analyses, are often carried out to establish confidence in the model. Several simulation scenarios are usually required to assess the performance of a water system.

For this project, computer model development included the following steps:

- The water model developed for the St. John's Regional Drinking Water Study was expanded to include all distribution mains in Paradise. The Town provided a water system map to CBCL, and where available, as-built drawings which describe the pipe size and material.
- Measured water flows for Paradise were obtained from the RWS (refer to Section 3.1). Using the total flow, water demands were apportioned to each pipe node based on a review of topographic and aerial maps of the areas and assessing the housing density and the amount of commercial/industrial land that would be tributary to the node.
- Future demands were developed using population projections and applied to the model.

The computer model was used to evaluate existing and future conditions under the following scenarios:

- Maximum day demand.
- Maximum day demand + fire flow.

Fire flow demands for existing commercial buildings in the Town are not readily available. It is difficult to estimate fire flows for commercial buildings since the required flow for each building is dependent on the floor area, type of construction, building use (hazard occupancy), sprinkler protection, and proximity to adjacent buildings.

Theoretical fire flows for various construction types were analyzed based on information in the *Water Supply for Public Fire Protection, Guide to Recommended Practice (1999)*, and are summarized in Table 4.1.

Table 4-1 - Fire Flow Requirements by Construction Type

Construction Type	Flow (L/min)	Duration (hr)
Residential – Wood Frame	4,000	1.5
Commercial	12,000	2.5

On May 19, 2016, SCM-Risk Management Services, with the assistance of Town staff, conducted fire flow tests at the following locations:

- Test #1: 12 Pollard Avenue
- Test #2: New Arena, 68 McNamara Drive
- Test #3: St. Thomas Line / Picco Drive intersection
- Test #4: Elizabeth Park Elementary, 80 Ellesmere Avenue
- Test #5: Paradise Elementary, 60 Karwood Drive

The results of the tests are included in Appendix B.

4.3.1 Existing Conditions

4.3.1.1 Supply

As mentioned in Section 2, BBBP WTP provides water to the Town. Since BBBP is a part of the RWS, and also supplies St. John's, Mount Pearl, CBS and Portugal Cove-St. Philip's, the analysis of BBBP reliable yield and WTP capacity have been addressed in the *"St. John's Regional Drinking Water Study"* report.

4.3.1.2 Storage

The capacities of the existing storage facilities (Camrose Drive and Southlands reservoirs) were analyzed for current conditions using the storage equation described in Section 4.2. Fire flow assumptions are based on the recommendations contained in the *Water Supply for Public Fire Protection (1999 Edition)* published by the Fire Underwriters Survey. Since the Southlands reservoirs also service portions of St. John's (RWS pressure zone BB-I) and Mount Pearl (RWS pressure zones MP-A and MP-B), and the Camrose Drive reservoir also services portions of Portugal Cove-St. Philip's (RWS pressure zone PS-E), the analysis includes maximum day demands for the respective pressure zones, as determined in the *St. John's Regional Drinking Water Study*. Table 4.2 presents a comparison of the existing required storage and available storage. As shown, both reservoirs have sufficient volumes.

Table 4-2 - Existing Conditions Storage Analysis

Reservoir	Pressure Zones	Storage Required (m^3)				Total Available (m^3)
		A	B	C	Total Required	
Southlands	PA-A, BB-I, MP-A & B	6,553	3,360	2,478	12,391	19,600
Camrose Drive	PA-B to PA-H, PS-E	2,312	1,800	1,028	5,140	10,540

Although the storage tanks have adequate volumes, the Town still experiences problems with maintaining an operating pressure of 40 psi at developments near the 190 m contour. The reservoirs are designed with top water levels (TWL) of 218 m. Discounting friction loss in the pipe, the head at the 190 m contour is approximately 28 m (40 psi) when the tanks are at TWL. Operationally, there are times when the tanks are not at TWL, therefore the head at the 190 m contour is less than 28 m (40 psi). Additionally, friction losses in the piping network further reduce the head available at the 190 m contour.

4.3.1.3 Distribution Pumps and Piping

According to Town officials, the water mains in Paradise are in good condition, with the exception of the mains in the Evergreen Village trailer park, which are reported to be in poor condition and breaks are frequent. The Town has noted that no thrust blocks or joint restraints were installed on these mains. Therefore, excavating for a repair often results in more breaks. In addition, these water mains do not have defined easements; they are located at the rear of residential properties and under buildings. The current water main locations make repair activities difficult for the Town.

For the hydraulic analyses, two scenarios were examined under maximum day demands, as follows:

1. With Paradise PS pumps running, Camrose Drive reservoir water level set at 0.5 m below TWL (the set point to start the pumps), and Southlands reservoirs set to TWL.
2. With Paradise PS pumps off and Camrose Drive and Southlands reservoirs set at TWL.

The existing transmission and distribution mains were assessed for both pressure and velocity under scenarios 1 and 2. Water main velocities did not exceed 1.5 m/s under either of these scenarios. However, residual pressures less than 40 psi were observed for both scenarios 1 and 2, particularly at elevations above 180 m.

Town officials identified several areas where pressure is less than 40 psi, as identified in Table 4-3.

Table 4-3 - Existing Areas with Pressure Less Than 40psi

Location	Pressure Zone
Elizabeth Park Elementary School (above 180 m contour)	PA-A
Islington Place (above 210 m contour, serviced by Donna Road PS)	PA-H
Hillsdale Crescent (above 180 m contour)	PA-A
West side of Karwood Drive (above 180 m contour)	PA-A

Most of these problem areas are at higher elevations (approximately 180 m or above). Scenario 2, described above, also produced low pressures at most of these locations, indicating that the model is producing reasonable results.

Donna Road Pumping Station

To further assess the claim of low pressure in PA-H, CBCL conducted a detailed site evaluation of the Donna Road Pumping Station on January 21, 2016. This pumping station is comprised of three pumping systems. A small skid-mounted triplex booster system with end suction centrifugal pumps (one at 5hp and two at 7.5hp) and pressure switch control was utilized when development in the area was much smaller. The system is complete but no longer normally used. A diesel fire pump and controller are present and will activate on a low pressure condition and is nominally rated at 63 L/s (1000 USgpm) flow at 38.7m (55 psi) head. These systems were original to the pumping station when it was constructed in the early 1990s.

The pumping station was upgraded with the addition of a new triplex pump system around 2009. The system is comprised of three skid-mounted FloFab model 2000-F1030A end suction centrifugal pumps fitted with 50hp motors and a control panel. The pumping system is configured as a constant pressure system utilizing a discharge header pressure transducer and variable speed drives to modulate output of the pumps. The pump duty point, based on nameplate information, is 31.5 L/s (500 USgpm) flow at 62.8m (89 psi) head. Based on this duty point, water pressure available at the higher elevations should be approximately 90 psi.

During the site visit, the pumping station inlet pressure was observed to be 43.6m (62 psi) and the discharge pressure was 83.1m (118 psi) with one pump operating at near full speed (59Hz). Based on the manufacturer's performance curve for this pump and the operational pressures recorded during the site visit, the pump is providing approximately 64.3 L/s (1020 USGPM) flow at 39.6m (56 psi) head which is far to the right of its optimal duty point as given on the manufacturer's performance curve. Note that there is no water meter present in the pumping station.

Pressure zone PA-H is reportedly isolated from the adjacent gravity system by closed isolation valves at the interconnections of PA-H and the gravity system. Based on a per capita usage rate of 736 L/person/d and an approximate population in PA-H of 1,300 persons, the average flow from this pumping station should be approximately 11.1 L/s (176 USGPM) and expected peak flows should be around 33.2 L/s (527 USGPM). These calculations show that this pumping station is pumping much more water than it should, which results in the low pressures being experienced at the higher elevations of this pressure zone. It can be concluded that excess water is being back fed to the adjacent gravity system (i.e. one or more of the isolation valves that should be closed are open), or there is leakage, or a combination of these two.

Also noted during the site visit was noticeable cavitation noise coming from the one operating pump. It is believed that this cavitation results from a combination of the high flow rate and the design of the intake piping. The intake piping is comprised of a 150mm header with a 100mm tee, 100mm butterfly, and short length of 100mm piping into the pump. End suction centrifugal pumps should be designed with a length of straight piping at the inlet equal to 10 pipe diameters (in this case 1,000mm) to allow water a straight run into the pump.

Fire Flows

The theoretical residential fire flow of 4,000 L/min was modeled in pressure zone PA-H, along with maximum day demands. It was assumed the Paradise PS pumps were off, and the Camrose Drive reservoir was at TWL. Under these circumstances the Donna Road pumps are sufficient to provide the maximum day demand and theoretical fire flow of 4,000 L/min to PA-H. The distribution mains in PA-H do not exceed the maximum design velocity of 3.0 m/s.

In pressure zone PA-B, the theoretical fire flow of 12,000 L/min was modeled in the commercial area of Kenmount Road Extension, along with maximum day demands. For this scenario, it was also assumed that the Camrose Drive reservoir is at TWL and the Paradise PS pumps are off. Under these conditions a residual pressure of 22 psi cannot be maintained in PA-B, PA-C, PA-D, PA-E or PA-F. In addition, the velocity in the 300 mm transmission main on Paradise Road is approximately 4.0 m/s, exceeding the maximum design velocity of 3.0 m/s. This is due in part to Kenmount Road Extension water main being a dead end; however, as development on Kenmount Road Extension continues this water main will be looped. An approximate fire flow of 1,000 L/min can be supplied to Kenmount Road Extension while achieving the pressure and velocity requirements. A theoretical residential fire flow of 4,000 L/min was also simulated in PA-B; on Topsail Road near McNamara Drive. Although the velocity requirement is met for a fire flow of 4,000 L/min, the residual pressure of 22 psi cannot be maintained, especially at elevations of 160 m and greater. An approximate fire flow of 1,000 L/min can be supplied at this location while achieving the pressure and velocity requirements elsewhere in pressures zones PA-B to PA-G.

A fire flow of 12,000 L/min was modeled in PA-A, near the Paradise Elementary. For this scenario it was again assumed that Camrose Drive and Southlands reservoirs are at TWL and the Paradise PS pumps are off. Under these assumptions, the fire flow of 12,000 L/min results in pressure less than 22 psi in the Karwood Drive and surrounding area as well as velocities larger than 3.0 m/s. Similarly, a fire flow of 12,000 L/min modeled near Elizabeth Park Elementary school results in pressures less than 22 psi in PA-A, particularly the Elizabeth Park area, and velocities greater than 3.0 m/s along Topsail Road. A residential fire flow of 4,000 L/min can be supplied throughout PA-A while meeting the design criteria.

As mentioned previously, there are no known fire demands; therefore, theoretical fire flows were analyzed in the InfoWater model. Fire flow tests #1, 2, 4 and 5 indicate that there will likely be issues attaining the sprinkler demands, without having a fire pump in the commercial building near the test location. This is due to the relatively low residual pressure observed during those tests. Although design flows are unknown, it is expected that the design flow for schools would be in the order of 2,800 L/min (750 USgpm) for sprinkler and hydrant loads. CBCL spoke with the Department of Education and learned that Elizabeth Park Elementary is equipped with a fire pump; however, Paradise Elementary and Holy Family Elementary are not. A fire demand of 2,800 L/min was entered in the InfoWater model at the location of each school (assuming only one fire demands at a time). The fire flows were modeled assuming the Paradise PS pumps were off and the Camrose Drive and Southlands reservoirs were at TWL. The model indicates that this demand can be achieved at Paradise Elementary and Elizabeth Park Elementary (PA-A), without resulting in pressures less than 22 psi or velocities greater than 3.0 m/s. However, a fire demand of 2,800 L/min at Holy Family resulted in pressures less than 22 psi in PA-B.

Table 4-4 summarizes the deficiencies noted for the analysis of the existing conditions. Figure 4.1 shows the service limits under existing conditions.

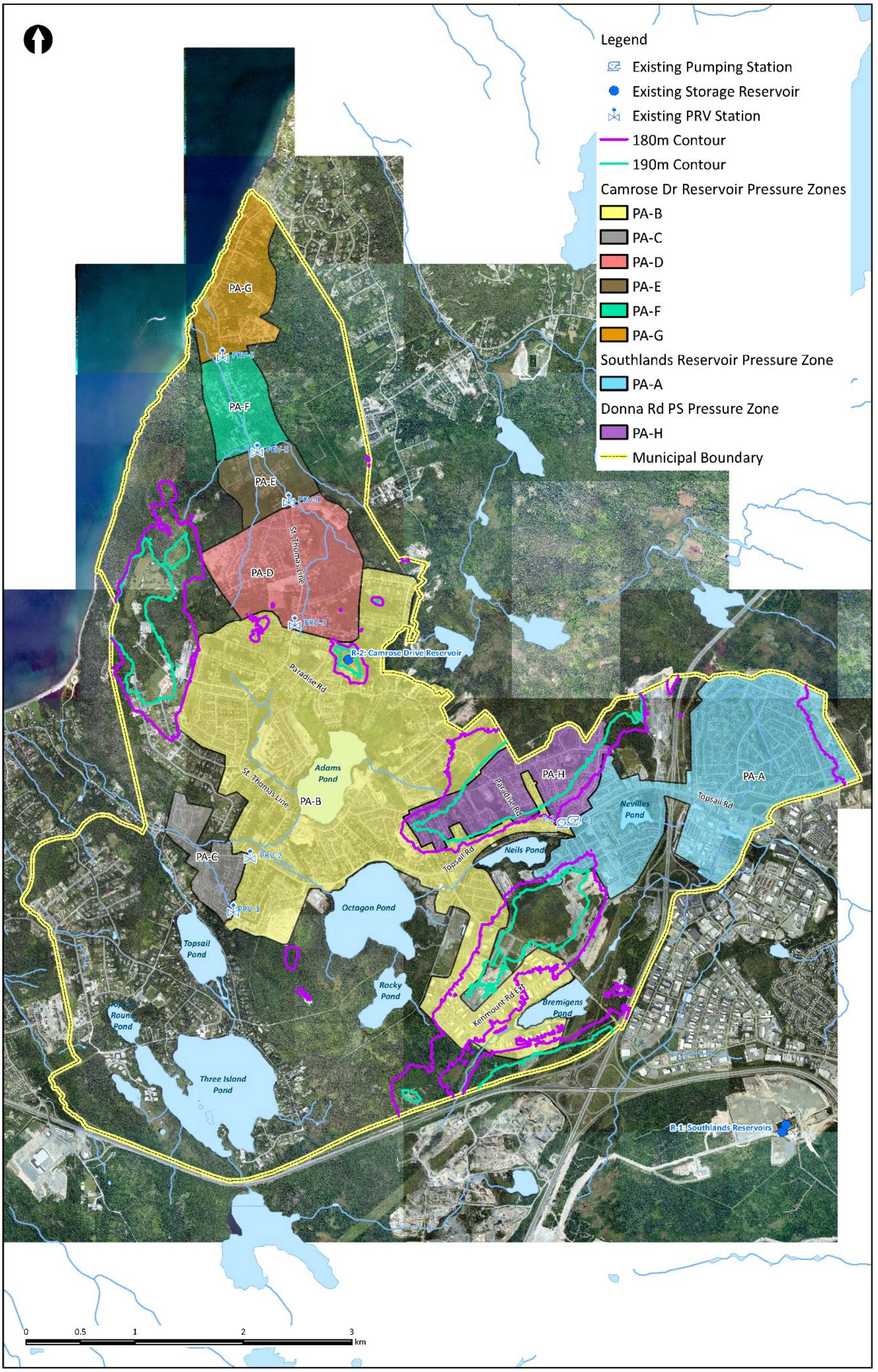


Figure 4-1 - Service Limits under Existing Conditions

Table 4-4 - Summary of Infrastructure Deficiencies for Existing Demands

Demand	Issue	Deficiency
Existing Maximum Day Demand (MDD)	Frequent water main breaks in the Evergreen Village trailer park (PA-A).	Lack of thrust blocks and restraining joints. No defined water main easement/inaccessibility.
Existing MDD	Pressure < 40 psi at or above the 180 m contour.	Water system cannot maintain pressure at elevations greater than 180 m.
Existing MDD + Commercial Fire Flow	Pressure < 22 psi & velocity > 3.0 m/s	
Existing MDD + Residential Fire Flow	Pressure < 22 psi	
Existing MDD + School Fire Flow	Pressure < 22 psi	

4.3.2 Future Conditions

4.3.2.1 Supply

As mentioned in Section 4.3.1.1 the capacity of the BBBP WTP for future demands is addressed in the *"St. John's Regional Drinking Water Study"* report.

4.3.2.2 Storage

The future developments in Paradise, discussed in Section 3.3, are planned for pressure zones PA-A (Karwood Drive), PA-B (Fairview Investments and local infill) and PA-D (Picco Ridge). The anticipated increase in maximum day demand for 2036 is 3,524 m^3/D, as indicated in Table 3-4. The anticipated increases in maximum day demands for the Southlands reservoirs were taken from the St. John's Regional Drinking Water Study. Table 4-5 illustrates the comparison of the future required storage for 2036 demands, and current available storage. As shown, both reservoirs have sufficient volumes for the predicted 2036 conditions.

Table 4-5 - Future (2036) Conditions Storage Analysis

Reservoir	Pressure Zones	Storage Required (m^3)				Total Available (m^3)
		A	B	C	Total Required	
Southlands	PA-A, BB-I, MP-A & B	7,105	3,360	2,616	13,081	19,600
Camrose Drive	PA-B to PA-H, PS-E	3,194	1,800	1,249	6,243	10,540

4.3.2.3 Distribution Pumps and Piping

The two scenarios described in 4.3.1.3 were simulated in the model using 2036 maximum day demands.

For scenario 1 (Camrose Drive reservoir below TWL, Paradise PS pumps running and Southlands reservoirs at TWL), pipe velocities do not exceed the design velocity of 1.5 m/s. However, pressures less than 40 psi are simulated in PA-A, above the 180 m contour.

For scenario 2 (Camrose Drive and Southlands reservoirs at TWL and Paradise PS pumps off), the velocity in the 300 mm transmission main on Paradise Road is 1.6 m/s, which marginally exceeds the maximum design value of 1.5 m/s. In addition, scenario 2 showed that there are operating pressures at less than 40 psi for elevations above the 160 m contour in pressure zones PA-B and PA-D.

Donna Road Pumping Station

No additional development is planned for PA-H, therefore the distribution mains in PA-H are adequately sized for 2036 conditions. However, based on observations during the January 2016 evaluation of the Donna Road PS pumps there are operational issues, as discussed in Section 4.3.1.3.

Fire Flows

Theoretical fire flows modeled for existing conditions did not meet the requirements for residual pressure or velocity, with the reservoirs are set at TWL. The additional maximum day demands for 2036 conditions will exacerbate the deficiencies. A fire flow of 2,800 L/min was modeled at the three schools assuming Paradise PS pumps were off and the Camrose Drive and Southlands reservoirs were at TWL. These fire flows can be achieved at Paradise Elementary and Elizabeth Park Elementary, without resulting in pressure or velocity deficiencies in PA-A, and without exacerbating the deficiencies in PA-B and PA-D, as described above. However, a fire flow of 2,800 L/min at Holy Family Elementary results in pressures less than 22 psi in PA-B.

A summary of the infrastructure deficiencies for the analysis of future conditions is presented in Table 4-6.

Table 4-6 - Summary of Infrastructure Deficiencies for Future Demands

Demand	Issue	Deficiency
Future MDD	Pressure < 40 psi at or above 180 m contour in PA-A	Water system cannot maintain pressure at elevations greater than 180 m.
Future MDD	Pressure < 40 psi at or above 160 m contour in PA-B & PA-D & velocity > 1.5 m/s	Water system cannot maintain pressure at elevations greater than 160 m.
Future MDD + Commercial Fire Flow	Pressure < 22 psi & velocity > 3.0 m/s	
Future MDD + Residential Fire Flow	Pressure < 22 psi	
Future MDD + School Fire Flow	Pressure < 22 psi in PA-B	

CHAPTER 5 PROPOSED IMPROVEMENTS

The proposed improvements outlined below address the system deficiencies identified in Chapter 4.

5.1 Evergreen Village Water Main Upgrades

A recurring problem for the Town is frequent water main breaks in the Evergreen Village trailer park. Town officials have indicated that the mains were installed without thrust blocks and joint restraints. Also, as discussed in Section 2.1, and illustrated on Figure 5.1, these mains are not easily accessible. It is proposed that the existing 150 mm water mains be abandoned and replaced such that they are realigned within road rights-of-way.

5.2 System Pressure Improvements

In general, there are two ways in which system pressures can be improved in the Town of Paradise: by providing an additional elevated storage tank or by providing constant pressure booster pumping stations to areas of low pressure.

Option 1 – Elevated Storage

The construction of a new storage tank at the high point of Neil's Pond Ridge would address the pressure deficiencies experienced above the 180 m contour in pressure zone PA-A, and allow for the proposed Karwood Town Centre development to be serviced up to the 195 m contour. Figure 5.2 shows the proposed pressure zone arrangements and locations of the new tank, water transmission main and pumping station. The transmission main and pumping station are required to move water to the new reservoir and to the north end of pressure zone PA-A. Figure 5.3 presents the proposed locations of the new tank, water transmission main and pumping station in greater detail.

Based on the hydraulic analysis of future maximum day demands which are concentrated in the south end of Paradise, the Camrose Drive reservoir cannot provide 40 psi above the 160 m contour. Therefore, the Fairview Investments development would be connected to the new tank under Option 1. This would reduce the future demand on the Camrose Drive reservoir and allow development up to the 195 m contour (although this elevation is not required for the Fairview Investments development).

Option 2 – Booster Stations

The construction of a booster pumping station at the higher elevations of Elizabeth Park would address the low pressure issues experienced in pressure zone PA-A. Also, a booster pumping station could be constructed to service the proposed Karwood Town Centre development. Figure 5.4 shows the proposed pressure zone arrangements under this scenario.

The Camrose Drive reservoir could service the Fairview Investments development up to the 180 m contour by installing a water transmission main on the east side of Adam's Pond.

Options Evaluation

The capital costs for each of the above options are:

- Option 1: $8,800,000 (includes new storage tank, transmission main and pumping station)
- Option 2: $6,000,000 (includes two new pumping stations and transmission main)

The annual operations and maintenance costs for each option are:

- Option 1: $0 (for the Town; the RWS would operate the pumping station)
- Option 2: $50,000

A life cycle cost evaluation was carried out using the present worth method and the following assumptions:

- Design life: 25 years
- Discount rate: 4%

Using the above assumptions, the present worth costs of each option are as follows:

- Option 1: $8,800,000
- Option 2: $6,800,000

Assuming a 70/30 (Provincial/Municipal) capital cost sharing arrangement (for illustrative purposes), the capital costs for the Town are reduced; however, the operations and maintenance costs remain the same. The present worth costs under a 70/30 cost sharing arrangement are:

- Option 1: $2,640,000
- Option 2: $2,600,000

This life cycle cost evaluation illustrates that the total costs for each option are similar. However, from a reliability perspective, it is in the Town's best interest to minimize the number of pumping stations that it must operate and maintain. Therefore, Option 1 is the recommended alternative and is carried forward in this report as the recommendation that will address the Town's pressure deficiencies and allow for future expansion.

5.3 Donna Road Pumping Station

As noted in section 4.3.1.3, the domestic pumps at the Donna Road pumping station are not operating properly as evidenced by the cavitation noise that was noted during the site visit. The reasons for the cavitation noise are likely a combination of the incorrect suction header design and the high flow rates.

Correcting the header configuration would be very difficult to do because there is inadequate space to provide the required 10 pipe lengths of pipe upstream from the suction ends of the pumps. Instead, it is recommended that the Town check the isolation valves that define PA-H to ensure that they are closed. If they are all closed, the Town should check for leaks in PA-H. Reducing the high flow rate at the domestic pumps should result in the pumps operating as recommended by the manufacturer.

5.4 Limits of Future Developments

Historically, the Town has limited development to the 190 m contour. This elevation was chosen as the highest point which could be serviced by the Camrose Drive and Southlands reservoirs while maintaining an operating pressure of 40 psi. However, taking the fluctuation of the reservoir water levels and friction losses in water transmission mains into account, it is not possible to consistently provide 40 psi at the 190 m contour.

It is recommended that the Town adopt a policy to limit future development to the 160 m contour for areas serviced by the Camrose Drive reservoir, and to the 180 m for areas serviced by the Southlands reservoir until the Neil's Pond Ridge reservoir is constructed.

After the Neil's Pond Ridge reservoir is constructed, the Camrose Drive reservoir would service lands up to the 180 m contour, and the Neil's Pond Ridge reservoir would service areas up to the 195 m contour.

Figure 5-1 - Evergreen Village

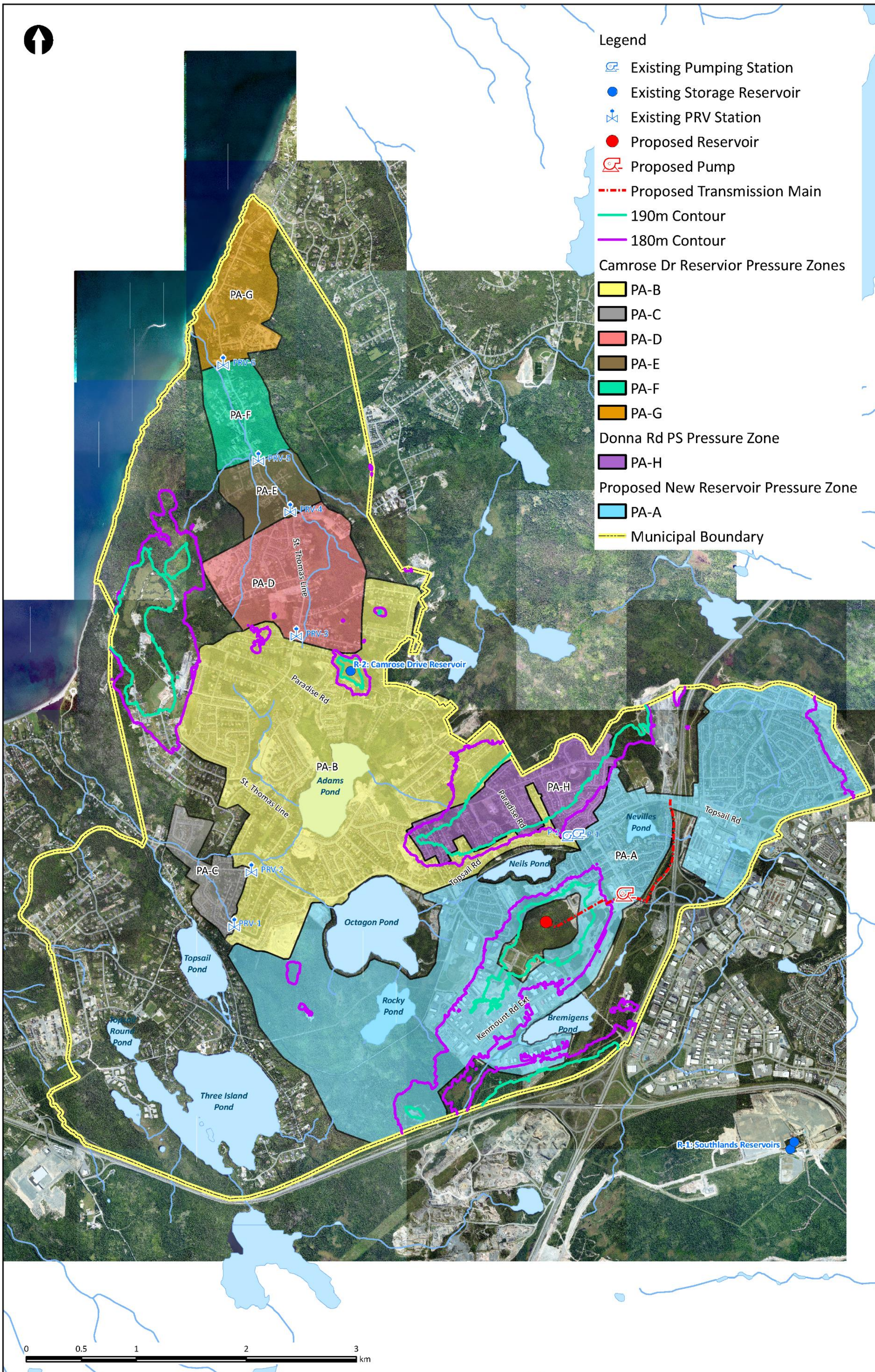


Figure 5-2 - Option 1

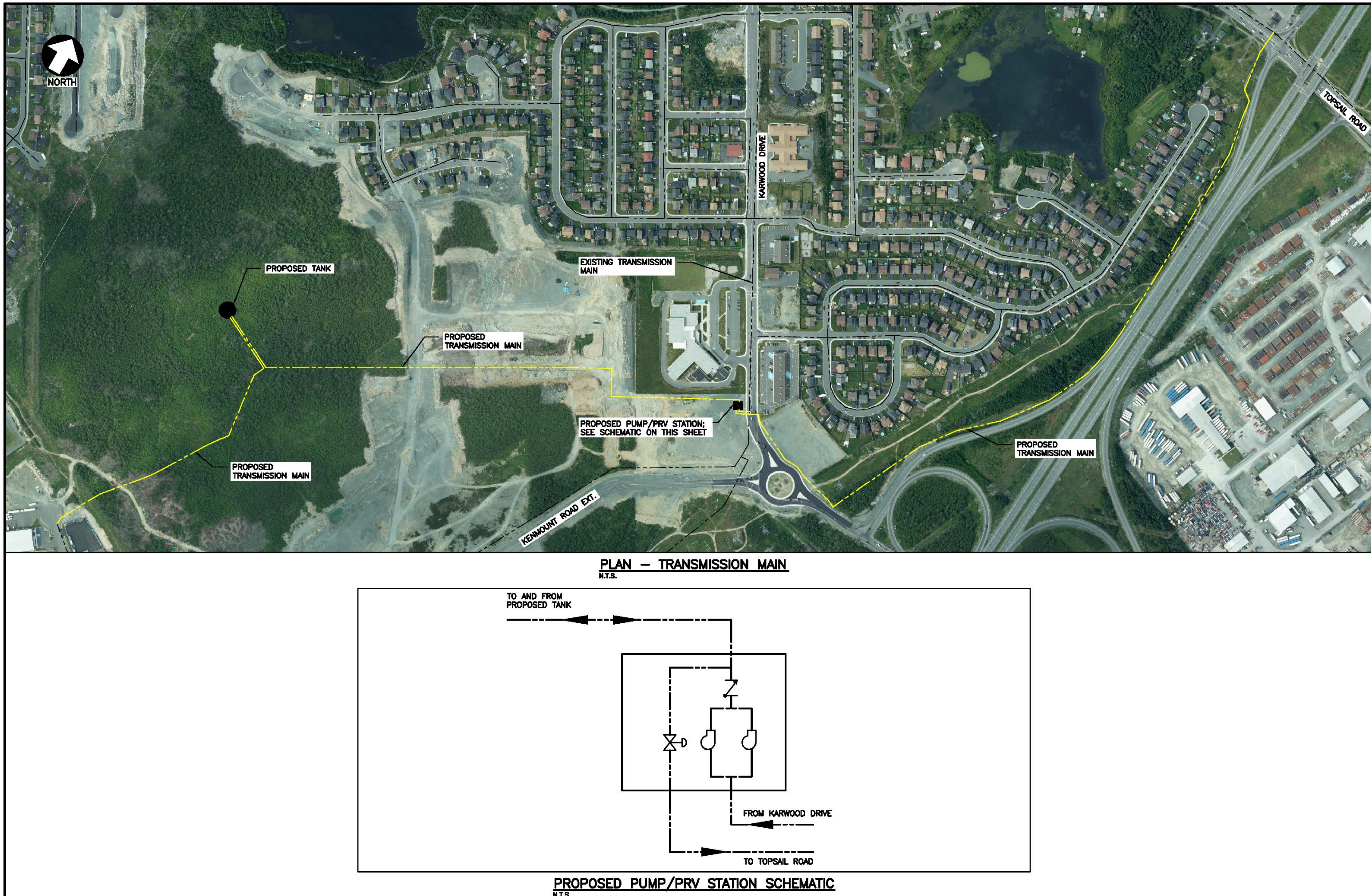


Figure 5-3 - Option 1 (in detail)

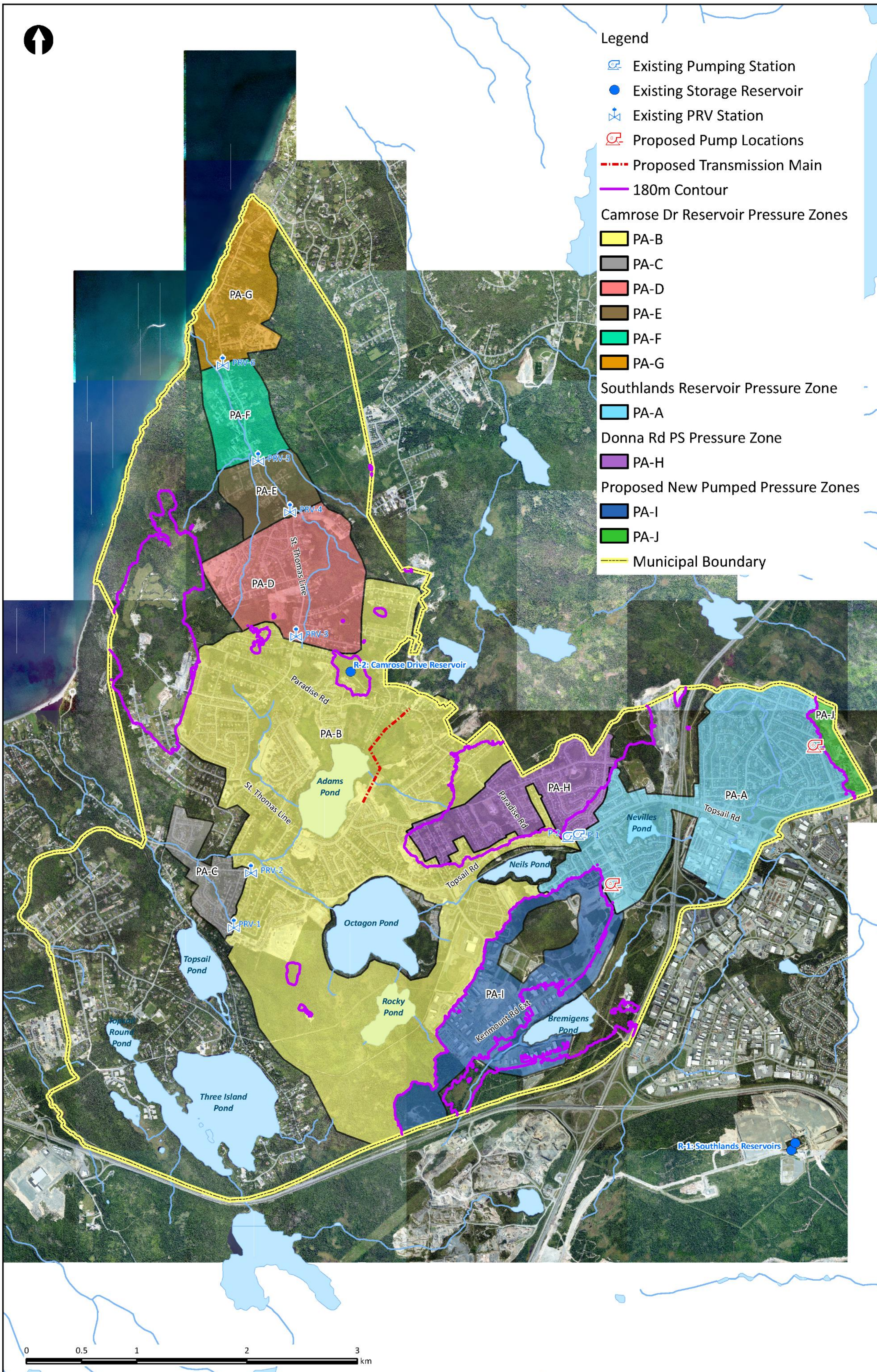
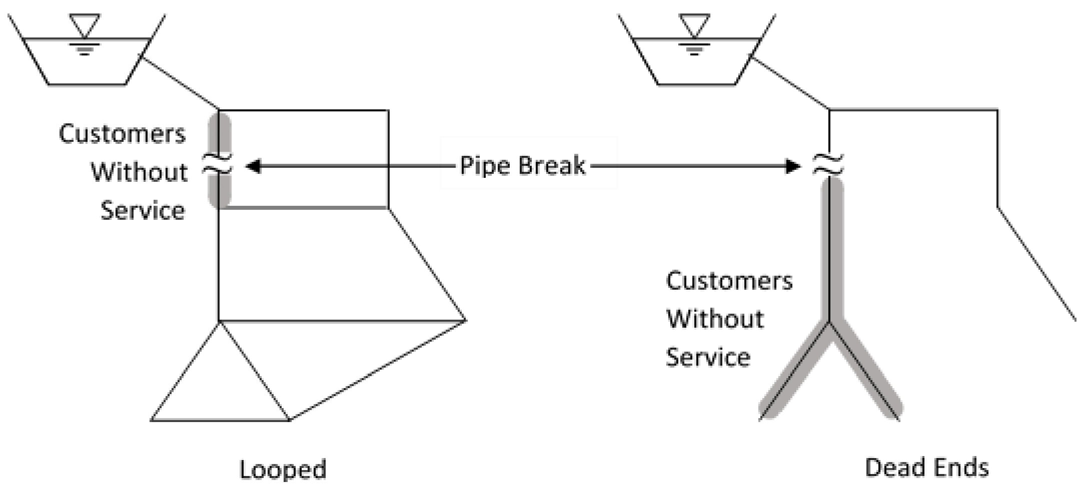


Figure 5-4 - Option 2

5.5 Looping of Existing Water Mains

There are many dead end water mains throughout the Town; cul-de-sacs, unconnected streets, and closed valves (isolating pressure zones) all create dead end branches in the Town's water system. Some disadvantages of dead end mains in water systems include:

- Limited water available for firefighting;
- Service disruption during pipe repair (as illustrated in Figure 5.4);
- Possible sediment accumulation due to stagnate water at the dead end (also possible bacterial growth), and
- Difficulty maintaining desired chlorine residual.



Source: Advanced Water Distribution Modeling and Management, Haestad Methods Water Solutions

Figure 5-5 - Looped and Dead End Networks After Network Failure

In some instances the existing dead ends are unavoidable. For example, the water main along St. Thomas Line, north of the intersection with Paradise Road, can be thought of as a dead end as water is only supplied from one direction.

It is recommended that the Town, where possible, install water main connections that result in looping. This effort should focus on cul-de-sacs and unconnected streets.

CHAPTER 6 COST OPINIONS AND IMPLEMENTATION SCHEDULE

6.1 Cost Opinions

Cost opinions for the proposed improvements outlined in Chapter 5 are presented in Table 6-1. Detailed break-downs of these opinions are provided in Appendix C.

Table 6-1 - Cost Opinions

Description	Cost
Evergreen Village Water Main Upgrades	$4,152,000
Neil's Pond Ridge Reservoir	$8,800,000

These cost opinions are Class 'D' estimates (±20%). Design development contingencies of 10%, construction contingencies of 20%, engineering and HST have been included in these cost opinions. Allowances for land acquisition have not been included.

Further, the above opinions of probable costs are presented on the basis of experience, qualifications and best judgement. They have been prepared in accordance with acceptable principles and practices. Sudden market trend changes, non-competitive bidding situations, unforeseen labour and material adjustments and the like are beyond the control of CBCL Limited. We cannot warrant or guarantee that actual costs will not vary significantly from the opinions provided.

6.2 Implementation Schedule

The implementation schedule for the recommendations noted above is presented in Table 6-2. The construction of an additional storage tank at Neil's Pond Ridge addresses operational issues and future development concerns. Due to the nature of the operational issue, that is, low distribution system pressure, it is recommended that the Town consider implementing this recommendation over the short-term (within five years). Similarly, the Evergreen Village Water Main Upgrades will address a significant maintenance concern and should be done over the short-term.

Table 6-2 - Implementation Schedule

Term	Description
Short-term	Evergreen Village Water Main Upgrades
Short-term	Additional Storage: Neil's Pond Ridge

CHAPTER 7 OPERATIONS REVIEW

7.1 Water Loss Control

As water treatment and transportation costs for the Regional Water System continue to increase, the Town of Paradise will have to pay more for potable water and will ultimately be challenged with recouping these costs from its residents in a fair and equitable manner. To minimize the overall cost that must be borne by Paradise ratepayers, it makes sense for Paradise to identify how its water is being used and what portion is being lost through leaks and other sources.

The American Water Works Association (AWWA) Manual M36 entitled “Water Audits and Loss Control Programs” provides clear justification for the responsible management of potable water resources and guidance on how to account for water that enters and leaves the Town’s distribution system. At the heart of the M36 approach to water management is the w*ater audit* and the w*ater balance*. The water audit traces the flow of water from the source or entry point into the community’s water distribution system, through its distribution network and to its customers; whereas, the water balance is a summary of all of the components that comprise the water audit. In theory, the total amount of water entering the distribution system should equal the total amount leaving the system.

The importance of understanding how water enters and leaves the distribution system and how to subsequently reduce water loss cannot be overemphasized. As outlined in AWWA M36, the control of water losses in a distribution system results in the following primary benefits:

- Enhanced water resources management, by limiting unnecessary or wasteful source water withdrawals.
- Financially, by optimizing revenue recovery and promoting equity among ratepayers.
- Operationally, by minimizing distribution system disruptions, optimizing supply efficiency, and generating reliable performance data.
- System integrity, by reducing the risk of contamination (by cross-connection to sanitary sewers).

The first step in establishing a water loss control program is to carry out a water audit. The auditing process occurs at three levels: top-down approach, component analysis and bottom-up approach. Each level adds further refinement to the water audit and is described in detail in AWWA M36. It is recommended that the Town of Paradise start with the top-down approach, which is outlined in Table 7-1. CBCL, during our draft report review meeting, will discuss this table in detail and make suggestions regarding how the Town can best get started on their water audit.

Table 7-1 - Water Balance (from AWWA M36)

<table>
<tr><td rowspan="9">Water from own Sources (corrected for known errors)
0.0</td><td rowspan="11">System Input Volume
2.77M m^3</td><td>Water Exported
0.0</td><td rowspan="5">Authorized Consumption</td><td rowspan="3">Billed Authorized Consumption</td><td>Billed Water Exported</td><td rowspan="3">Revenue Water</td></tr>
<tr><td rowspan="10">Water Supplied
2.77M m^3</td><td>Billed Metered Consumption</td></tr>
<tr><td>Billed Unmetered Consumption</td></tr>
<tr><td rowspan="2">Unbilled Authorized Consumption</td><td>Unbilled Metered Consumption</td><td rowspan="8">Non-Revenue Water</td></tr>
<tr><td>Unbilled Unmetered Consumption</td></tr>
<tr><td rowspan="6">Water Losses</td><td rowspan="3">Apparent Losses</td><td>Unauthorized Consumption</td></tr>
<tr><td>Customer Metering Inaccuracies</td></tr>
<tr><td>Systematic Data Handling Errors</td></tr>
<tr><td rowspan="3">Real Losses</td><td>Leakage on Transmission and Distribution Mains</td></tr>
<tr><td rowspan="2">Water Imported
2.77M m^3*</td><td>Leakage and Overflows at Utility's Storage Tanks</td></tr>
<tr><td>Leakage on Service Connections Up to Point of Customer Metering</td></tr>
</table>

*Approximate annual volume for 2014

Table 7-2 - Water Balance Terms and Definitions (from AWWA M36)

Water Balance Component	Definition
System Input Volume	The annual volume input to the water supply system.
Authorized Consumption	The annual volume of metered and/or unmetered water taken by registered customers, the water supplier, and others who are authorized to do so.
Water Losses	The difference between System Input Volume and Authorized Consumption, consisting of Apparent Losses plus Real Losses.
Apparent Losses	Unauthorized Consumption, all types of customer metering inaccuracies and systematic data handling errors.
Real Losses	The Annual Volumes lost through all types of leaks, breaks, and overflows on mains, service reservoirs, and service connections, up to the point of customer metering.
Revenue Water	Those components of System Input that are billed and produce revenue.
Nonrevenue Water	The sum of Unbilled Authorized Consumption, Apparent Losses, and Real Losses. Also, this Value can be determined as the difference between System Input Volume and Billed Authorized Consumption.

After completing the top-down water audit, the Town may decide to implement a water loss control program. Water loss control or conservation measures are varied, and can include more accurate accounting of water usage, rate structure changes, policy development and physical incentives to reduce water usage, such as low flow plumbing fixtures. Descriptions of the more salient loss control measures are presented below.

7.1.1 Customer Metering

Universal metering is common across Canada; however, it has not yet been implemented anywhere in Newfoundland and Labrador. Implementation costs are high; however, metering allows for cost recovery (i.e. charging for water usage) to be done on a more equitable basis – the customer pays for the amount of water that they use. Further, the effectiveness of water efficiency programs, such as the installation of low flow fixtures, can be measured accurately, and the payback calculated with confidence.

Universal metering is also the foundation of developing a full-cost recovery system for the Town's water distribution infrastructure. Currently, the Town knows how much water that they purchase from the Regional Water System and the cost of the water. However, without metering, there is a high level of uncertainty associated with where the water goes after it enters the distribution system. In evaluating the feasibility of implementing a universal metering program, many aspects of the water system must be considered, including the following:

- Total water currently supplied;
- Water demand projections;
- Unit cost of water purchased from the Regional Water System;
- Anticipated increases in the cost of water purchased;
- Water distribution system operations and maintenance costs;
- Water rates in the Town of Paradise; and
- Possible water rate structures that would allow for full cost recovery.

With the information currently available, the capital cost and associated payback for the implementation of a universal metering program can be assessed as follows.

- Total amount of water supplied to the Town (2014): 2,770,000 m^3
- Total cost of water supplied: 2,770,000 m^3 x \$0.589/m^3 = \$1,631,530
- Cost to supply and install meters:
    - Residential: 7,500 units x \$700/unit =\$5,250,000
    - Commercial/Industrial: 500 units x \$2,000/unit = \$1,000,000
    - Total: \$6,250,000

Assuming that water usage would drop by 20% with the implementation of universal metering alone, it would take decades to pay back the capital investment using only the monies saved from purchasing less water. It would be more realistic to assume that the homeowners or business owners would pay for some portion of the supply and installation costs. Assuming that the costs of purchasing the meters are covered by the property owners, the Town's cost to supply and install meters can be approximated as follows:

- Residential: 7,500 units x \$350/unit =\$2,625,500
- Commercial/Industrial: 500 units x \$800/unit = \$400,000
- Total: \$3,025,000

Again assuming that water usage would drop by 20%, the payback period under the above scenario is 10 years under 2014 conditions. Operations and maintenance costs should be factored into a more detailed analysis.

7.1.2 Leak Detection

Financial losses associated with leaks include the operation and maintenance costs associated with pumping water, damage to the existing pipe network, erosion of pipe bedding, labour to find, fix and repair leaks, and potential damage to roads and buildings. Major drivers for a water loss control program are economic constraints, public health risks, water conservation and limited treatment capacity.

Leaks are commonly pinpointed using acoustic devices which detect the sound or vibration induced by water leaking from pressurized pipes. Leak sounds are transmitted through the pipe itself, depending on the pipe size and material, and through the surrounding soil in the immediate area of the leak. Leak detection crews work to locate leaks by listening at accessible points such as fire hydrants and valves. Leak noise correlators provide further pinpointing of leaks. Other methods for leak detection include tracer gas, infrared imaging and ground penetrating radar. The use of these alternate techniques is limited, potentially costly and their effectiveness is not as well established as other acoustic methods.

Several factors such as pipe size, pipe material, depth, soil type, water table, system pressure, interfering noise and sensitivity of equipment will determine the effectiveness of acoustic leak detection. The pipe material and diameter have a significant effect on the attenuation of leak signals in the pipe. Leaks in metal pipes are typically easier to locate than leaks in plastic pipes. Larger diameter pipes attenuate sound, making leaks more difficult to detect. Leaks tend to be more audible in sandy soils than clay, and more audible on hard surfaces (asphalt or concrete) than grassed. High pipe pressure may increase the leak sound. Weaker signals are expected at joints or valves when compared to splits or corrosion pits.

Upon completing the water audit, the Town will understand, with some confidence, the amount of real losses in their distribution system. Even by being conservative in estimating the authorized consumption, there is likely a case for a leak detection program. Estimates of how much water can be saved by finding leaks can be translated into a cost by multiplying the volume of water saved by the unit cost. The cost of a leak detection program can then be justified to Council.

7.1.3 District Metering

Effectively measuring the variation in leakage in a water distribution system requires the development of district metered areas (DMAs). DMAs are isolated areas within a distribution system that are separately metered. Meter data is used to quantify the collective leakage rate within the DMA. Analysis of this data will provide an early warning of increased leakage and guidance to assist a Town’s leak detection crew in locating and repairing major leaks.

A DMA should be kept to a reasonable size to establish daily diurnal flow variation and infer leakage rates from night time flow rates, based on the number and type of properties within the zone. A DMA may range from less than 1,000 properties for less densely populated areas to up to 5,000 properties in more densely populated areas. The water distribution computer model that was developed for this study will be a useful tool for establishing DMA boundaries. Specifically, the model can be used to check minimum flow and pressure requirements, check for looping and redundancy, and check that water quality is not adversely affected.

To assess leakage, the ratio of minimum night time flow to the average daily rate (night flow factor) in a DMA is compared to published rates to determine if there is excessive leakage. Areas which show signs of excessive leakage should be further assessed by step testing. Step testing involves systematically subdividing the area and measuring flow rates while turning off valves to cut of different sections. A significantly lower rate of flow indicates excessive leakage in the last area that was shut off.

District metering and flow measurement can be labour-intensive and costly because they are often performed at night. Permanent flow meters connected to a SCADA are recommended. The collected and transmitted data is then available to be automatically analyzed to detect unusual water use. With gained experience in analyzing typical water use patterns, it may be determined that increased flow rates are due to new leaks.

Due to its size and the presence of known leaks, Elizabeth Park could be used as DMA. The Town could consider establishing a pilot DMA for Elizabeth Park to evaluate its effectiveness.

7.2 Isolation Valve Maintenance

The AWWA Manual M44 entitled "Distribution Valves: Selection, Installation, Field Testing and Maintenance" provides guidance on maintenance procedures, scheduling and record keeping. In general, the following key points should be considered in the development of a valve maintenance program:

- The critical valves in the distribution system should be identified. For Paradise, these would include valves on primary distribution lines, including the water mains in Topsail Road, Paradise Road and St. Thomas Line.
- Inspections should be conducted on a regular basis; annually if possible. Inspections should include examining the condition of the valve box or chamber, operating the valve several times, and lubricating the valve as required.
- Preventative maintenance should be carried out as recommended by the manufacturer.
- All valves should be cycled from full open to full close and back to their regular position all at once over a five-year period.
- Repairs should be made promptly and correctly.
- Recording keeping is essential.

For record keeping, the Town could start with developing a valve record using the template presented in Appendix D (from the AWWA M44). A GIS would be very useful in maintaining and updating the valve records.

7.3 Geographical Information System (GIS)

Paradise should develop a Geographic Information System (GIS) which would be used as a database for the Town's existing water infrastructure information (including water main locations, sizes, age, valve locations, etc.). The GIS database is also a useful tool for record keeping; information regarding breaks, repairs, maintenance, etc., can be stored in the geodatabase and referenced to the specific location. Similarly, forms completed during water audits (M36 forms) and isolation valve maintenance exercises (M44 forms) can be incorporated in the GIS.

7.4 Residual Chlorine Monitoring

7.4.1 Water Quality

Water quality data from June 2014 to September 2015 has been provided by the Town of Paradise for review. While a large volume of data has been provided, the graphs below have been generated to summarize some of the water quality parameters which may be relevant for distribution system water quality monitoring and control. Figures 7.1 and 7.2 below show the results of total chlorine measurements in the distribution system from June 2014 to November 2015.

Total chlorine levels in the distribution system are fairly constant, with no significant system-wide seasonal variation discernable from the raw sample results shown in Figure 7.1. Total chlorine averages at individual sampling points ranged between 1.0 and 1.5 mg/L for the sampling period, as shown in Figure 7.2. The maximum total chlorine levels were below 2.2 mg/L in all samples. Monochloramine has a Health Canada maximum acceptable concentration (MAC) of 3.0 mg/L, and the minimum combined chlorine residual for chloraminated systems recommended by the AWWA is 1.0 mg/L. Fifteen of the 22 sample locations had minimum total chlorine measurements of less than 1.0 mg/L; therefore, monochloramine concentrations at these locations were less than 1.0 mg/L. Although the provincial regulatory limit does not follow this guideline, it can be used to evaluate which areas of the distribution system may be problematic in terms of water age or disinfectant residual demand. Locations which have total chlorine residuals less than 1.0 mg/L in greater than 10% of samples taken include:

- 83 Trail's End Drive
    - 48% of measurements less than 1.0 mg/L;
    - Lowest average total chlorine of all sample locations (1.04 mg/L);
    - Lowest total chlorine residual recorded (0.49 mg/L).
- 753 St. Thomas Line
    - 34% of measurements less than 1.0 mg/L;
    - Second lowest average total chlorine residual recorded (1.06 mg/L).
- 77 Picco Drive
    - 24% of measurements less than 1.0 mg/L.
- 16 Dina Place
    - 18% of measurements less than 1.0 mg/L.
- RWPS004 (Depot)
    - 14% of measurements less than 1.0 mg/L;
    - Second lowest total chlorine residual recorded (0.5 mg/L).
- 33 Whalen's Crescent
    - 10% of measurements less than 1.0 mg/L.

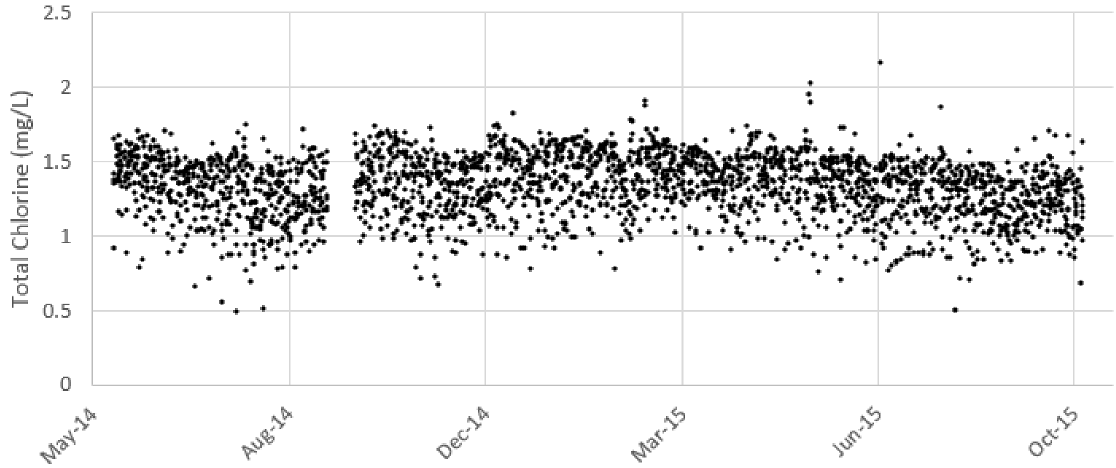


Figure 7-2 - Total Chlorine Concentrations

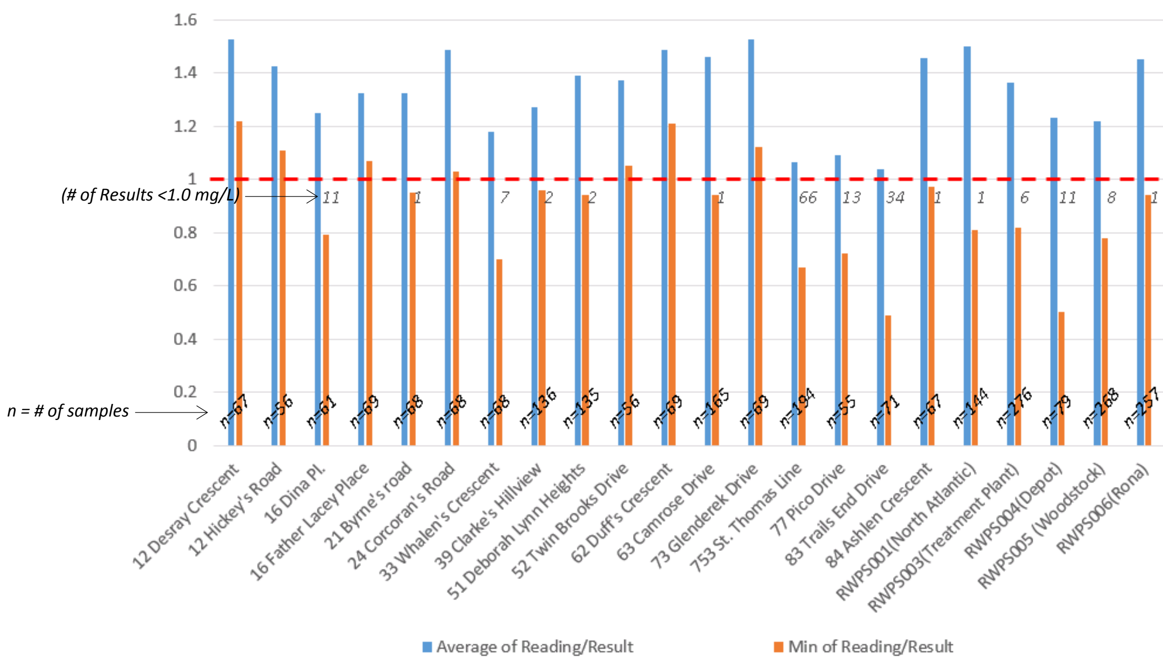


Figure 7-1 - Total Chlorine Concentrations – Minimum and Average by Location

The regular occurrence of lower than average residuals at the furthest ends of the distribution system does not necessarily indicate a deficiency with the existing system; it is impractical to maintain a constant residual throughout a distribution system where water age and interaction with pipes and storage tanks varies. However, the results should be taken into account when designing upgrades or additions to the distribution system, as well as when planning sampling and flushing locations and frequency.

Heterotrophic Plate Count (HPC) is a measure of microbiological activity in the distribution system, and is generally considered a test parameter which can be used to provide operational guidance to distribution system operators. The results of the HPCs in various parts of the distribution system over time are most useful to provide an indication of the relative water quality, as opposed to absolute water quality. While there is no maximum acceptable concentration or operational guidance value for HPC according to Health Canada, the United States Environmental Protection Agency (USEPA) mandates a maximum concentration of 500 CFU/100 mL. For a chloraminated system, an increase in HPC over the normal levels may indicate a proliferation of nitrifying bacteria in the distribution system piping and storage reservoirs. This may be in response to an elevated level of ammonia in the water, which would indicate an imbalance in the chlorine-to-ammonia disinfectant residual, or a breakdown, of the chloramine disinfectants due to more rapid chemical reactions or excessive water age. Six locations in the distribution system were regularly tested for HPC and the results, by date and location, are shown below in Figure 7.3 and 7.4, respectively. The results originally listed as “TNTC” (Too Numerous To Count) have been changed to 2,420, which is typically the maximum numerical value assigned in HPC analysis. This change was made to provide a more accurate representation of the maximum and average test results for locations where the TNTC results occurs, which can only be regarded as approximations.

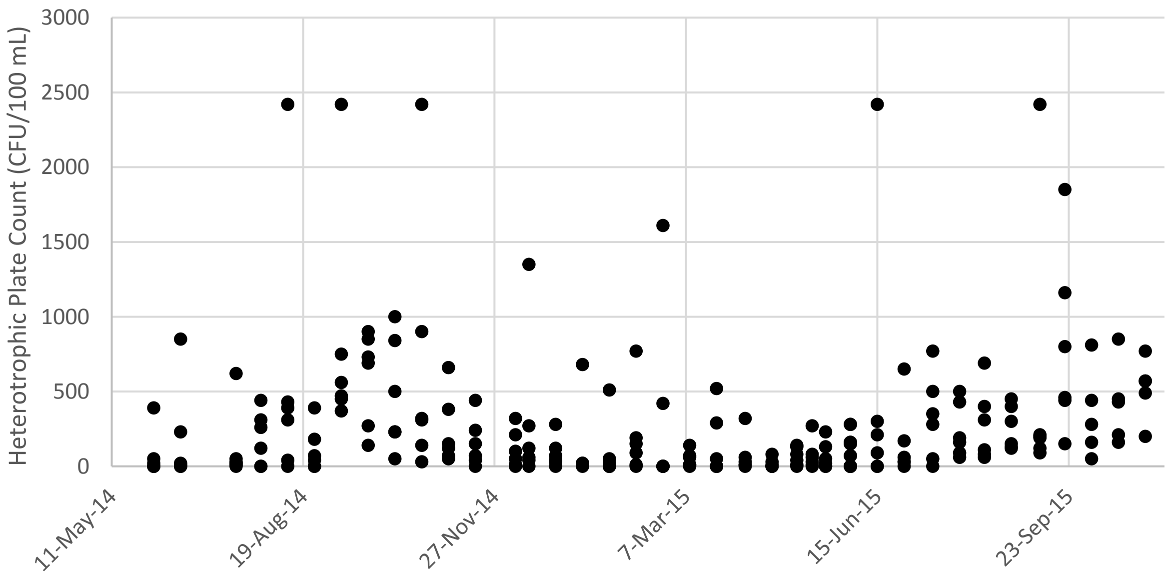


Figure 7-3 - Heterotrophic Plate Counts – All Measurements by Date

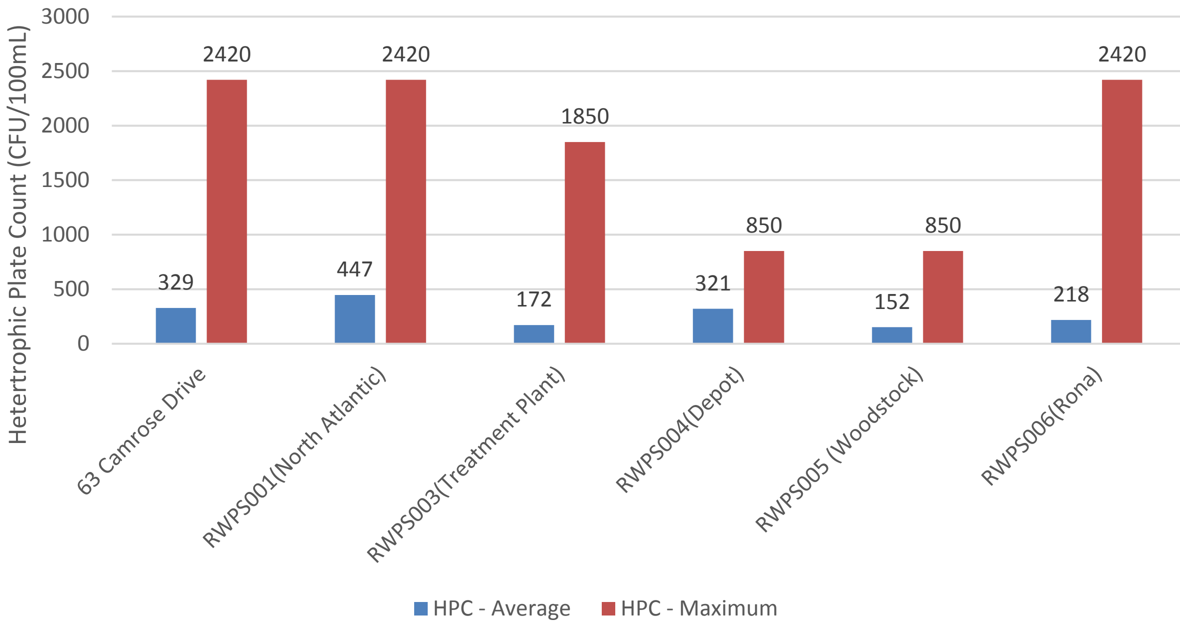


Figure 7-4 - Heterotrophic Plate Counts – Average and Maximum by Location (n=39 at all locations)

The HPC results show average measurements less than 500 CFU/100 mL, and maximum results greater than 500 CFU/100 mL, at all sample locations. The highest average result was recorded at RWPS001 (North Atlantic) as 447 CFU/100 mL. Based on the data, there may be a seasonal impact to water quality whereby HPC counts tend to be higher in the warmer months, though a larger data set would be required to draw statistically significant conclusions.

None of the other water quality parameters available showed significant variation in water quality by location. The parameters reviewed include:

- Total Iron;
- pH;
- Colour; and
- Turbidity.

7.4.2 Disinfection By-Products

The Town of Paradise is responsible for maintaining an adequate disinfection residual throughout the distribution system to ensure that secondary disinfection, which is the prevention of microbial re-growth in the distribution system, is preserved at all times. Secondary disinfection in Paradise's water distribution system is provided by chloramination at the BBBP water treatment plant. Chloramination for secondary disinfection has been implemented, rather than chlorination, in order to mitigate the formation of trihalomethane (THM) and haloacetic acid (HAA) disinfection by-products (DBPs).

The oxidation potential of chloramine is significantly lower than that of free chlorine, but the biocidal effectiveness of chloramine on pathogenic microorganisms in protective biofilms (in distribution systems) has been found to be proportionately higher than free chlorine, with respect to their oxidation potentials. The lower oxidation potential of chloramine results in lower reaction rates between the disinfectant and metallic pipe walls, dissolved organics and metals, and other reactants as compared to those expected when free chlorine is present in chlorinated systems. This results in a lower overall production of DBPs while preventing the depletion of the disinfectant in the system.

While chloraminated distribution systems are less likely to produce elevated THMs and HAAs, they should not be considered as a cure-all for water systems dealing with this issue. Chloramines do react with dissolved organics to form THMs and HAAs, although the rate of reaction is lower than that of free chlorine. Monochloramine is the target chemical species used for chloramination disinfection. Monochloramine itself is considered a health concern by Health Canada, which has listed a maximum acceptable concentration of 3 mg/L in the Guidelines for Canadian Drinking Water Quality (GCDWQ). There are also other DBPs and undesirable chemical compounds which can form when using chloramination which are not normally of concern in chlorinated supplies. These compounds are introduced and briefly discussed below.

Ammonia, Nitrate and Nitrite

If the concentration of ammonia is significantly higher than the optimum ratio of ammonia to chlorine, excess ammonia will remain in stable solution in the distribution system. Free ammonia in drinking water imparts a distinct, undesirable taste and odour to the water. While ammonia at typical levels found in drinking water systems is not considered hazardous from a health perspective, persistent free ammonia in the distribution system may support growth of nitrifying bacteria which convert ammonia into nitrate and nitrite, which are both health concerns. Health Canada lists the maximum acceptable concentration of nitrate and nitrite as 45 and 3 mg/L, respectively, in the GCDWQ. Nitrate and nitrite in drinking water supplies are considered a possible carcinogen and can cause Methaemoglobinaemia (blue baby syndrome) along with other adverse health effects at elevated levels.

Dichloramines and Trichloramines

While the intention of chloramination is to produce monochloramines, some portion of the chloramines produced will exist as di- and trichloramines. These form more prevalently when the ratio of chlorine to ammonia is significantly higher than the optimum ratio during production, and when the pH is lower. However, even under realistically ideal conditions, the practical process of chemical injection in water treatment processes inevitably leads to the production of some amount of these undesirable compounds. Both the di- and tri- type compounds carry significantly lower oxidation potential, and may impart poor taste and odour to drinking water. Dichloramines are also a precursor to the formation of N-Nitrosodimethylamine (NMDA).

N-Nitrosodimethylamine (NDMA)

NDMA is produced in a reaction between chloramines and dissolved organic molecules which contain nitrogen. NDMA is of particular concern, as it is considered to be a probable carcinogen even at extremely low levels (GCDWQ MAC of 0.00004 mg/L - 40 parts per trillion), which can be difficult to detect and cannot be measured without specialized laboratory analysis. NDMA has been listed by the Department of Environment and Conservation as a special parameter for monitoring in chloraminated drinking water supplies, following the same maximum limit listed above.

For chloraminated systems, disinfectant residual is typically recorded as combined chlorine, or monochloramine. Combined chlorine may be calculated as the difference between total and free chlorine. In a chloraminated system, ideally there is a negligible concentration of free chlorine as it suggests an imbalance between chlorine and ammonia used in the disinfection process, or a water age which is too high, contributing to the breakdown of monochloramine.

7.4.3 Distribution System Flushing

The primary determinants of water quality for Paradise are the water source and water treatment process, neither of which can be controlled by the utility staff. However, while pumps, PRVs and reservoirs are primarily designed to ensure adequate supply and pressure for the drinking water distribution system, these are tools at the disposal of the distribution system operators to influence water quality as well. The most important of these tools is distribution system flushing.

The formation of DBPs associated with chloramination is influenced by many of the same factors associated with DBPs formed in chlorination. Generally, factors which increase reaction rate or allow reactions to occur for longer should be mitigated to reduce the overall formation of chloraminated DBPs. These factors include temperature, water age, DBP pre-cursor concentrations, chloramine concentrations, etc. From the perspective of managing the water distribution system, the factor which can be best controlled is the water age. Water age is increased in systems with excess reservoir storage capacity, large diameter transmission piping, dead ends, poor storage tank turnover, etc. Hydrant flushing is a tool which can be used to minimize water age in the distribution system, along with providing other potential benefits to water quality and overall distribution system performance.

Flushing methods can be categorized into 3 types, each which best fit a specific objective and are accompanied by their own unique advantages and drawbacks:

- Conventional/Spot Flushing;
- Continuous Blowoff/Bleed Flushing; and
- Unidirectional Flushing (UDF).

Conventional/Spot Flushing

Spot flushing is typically done in response to a water quality event (i.e. if a low disinfectant residual is detected or in response to customer complaints) or as a mechanism to test fire flow at a given location in the distribution system. The intended goal of spot flushing is often to restore the disinfectant residual in a specific location, or to “clean out” a pipe which has accumulated sediment, resulting in poor water quality (lowered disinfectant residual due to elevated demand, taste and odour issues due to nitrification, elevated turbidity, biofilm formation, etc.). Periodic flushing at a dead end, done to restore disinfectant residual, would be considered spot flushing. Spot flushing is typically done with minimal planning or design to account for impacts on adjacent areas of the distribution system.

Continuous Blowoff/Bleed Flushing

Bleed flushing is where a continuous flow (from either a hydrant, designated service location, or sampling tap) is kept up in order to promote flow to one area in the distribution system. Bleed flushing is normally done in an area of the distribution system where demand is proportionally low relative to the transmission main pipe size; this commonly occurs at dead ends in the system. Bleed flushing does not typically provide scouring ability to rehabilitate pipe interior, but is used to ensure the required disinfectant residual is maintained to users in the low-demand area.

Unidirectional Flushing

The UDF method is considered the most advanced type, which typically is executed as part of a designed flushing program. The UDF method utilizes valves in the distribution system to create flushing loops, which allow a controlled flow from the middle of the distribution system (i.e. the storage reservoir) to the periphery. By isolating the flushing loops, a specific section of the distribution system can be flushed with minimal impact on the rest of the system (such as discoloured water in areas not targeted at the time of flushing). This is the common issue which occurs when spot flushing is utilized to target specific transmission or arterial mains.

Table 7-3 - Required Flow and Hydrants to Produce Adequate Velocity

Nominal Pipe Diameter (mm)	Flow Required for 1.8 m/s velocity, m^3/h (USgpm)
100	50 (235)
150	120 (525)
200	210 (940)
250	330 (1470)
300	460 (2,010)
400	850 (3,750)
450	1,080 (4,750)
600	1,920 (8,450)

The flow that can be produced from a single hydrant is dependent on the outlet pressure; at typical distribution pressure a single hydrant could be used to flush a pipe with a nominal diameter up to 300 mm. However, multiple hydrants may be used to generate the flows and velocities desired according to the UDF plan for larger diameter piping, or if hydrant discharge pressure is low. A calibrated water model can be used to determine the number of hydrants that will be required, and pressure and flow measurements taken during the flushing can be used to further refine the model itself.

A proper UDF should be designed with the following objectives in mind:

- Public notification of the flushing program dates, locations and objectives, and emphasizing the expected improvement to water quality;
- A step-by-step procedure should be developed and communicated to field operations staff prior to initiating any flushing, including:
  - Closing of valves before flushing to isolate flushing loop;
  - Sequence of hydrants to open;
  - Prior training and instructions for pressure and flow gauging;
  - Flushing duration at each hydrant;
  - Prior training and instructions on gauging water quality; and
  - Opening of valves to reconnect loop to the rest of the system.
- Isolate flushing loops using valves in distribution system;
- Flush from the center of the distribution system to the periphery;
- Flush from larger pipes to smaller;

- Achieve flows of 1.8 m/s - 3.0 m/s in targeted pipes 600 mm in diameter or less;
- Flush during low flow demand conditions (ideally overnight); and
- Design work plan to complete flushing for planned loop within single shift;
- 1,000 - 2,000 m of pipe per shift may be a realistic target to begin until feedback from operations staff can be included in planning.

CHAPTER 8 CONCLUSIONS AND RECOMMENDATIONS

8.1 Conclusions

The key conclusions of the water master plan include the following for the **existing** system:

1. Both the Camrose Drive and Southlands Reservoirs have sufficient volumes.
2. When the water level in the reservoirs is less than the top water level, a residual pressure of 40 psi above the 180 m contour cannot be achieved.
3. An assessment of the Donna Road Pumping Station indicates that the pumping station is pumping much more water than it should, thus resulting in the low pressures reported at the higher elevations in PA-H. This may be due to excess water being back fed to PA-A or PA-B, leakage issues, or a combination of both.
4. The water mains in Paradise are in relatively good condition, with the exception of Evergreen Village trailer park, where the mains are reported to be in poor condition and frequent breaks occur. The water mains were installed without thrust blocks or joint restraints. Therefore, excavating for repair often results in more breaks. In addition, the water mains do not have defined easements; they are located under residential properties to the rear of the properties. The current locations make repair activities difficult for the Town.
5. Low residual pressures observed during fire flow tests indicate that there will likely be issues in obtaining sprinkler demands for commercial buildings without installing fire pumps.
6. Pressures less than 22 psi and velocities greater than 3.0 m/s are experienced when simulating theoretical commercial fire flows. Pressures less than 22 psi are experienced when simulating theoretical residential fire flows.
7. Total chlorine levels in the system are fairly constant, with no significant system-wide seasonal variations discernable from the samples provided.
8. Six chlorine residual sampling locations have total chlorine residuals less than 1.0 mg/L in more than 10% of the samples provided for analysis. These locations, listed in Section 7.4.1, should be considered when planning sampling and flushing locations and frequency.
9. Heterotrophic Plate Counts indicated higher levels of microbial activity in the distribution system during the warmer months of the year. There are many possible contributing factors to this, but HPCs is often used as an indicator of excess ammonia in the distribution system, which leads to the growth of nitrifying bacteria.
10. Total Iron, Colour, Turbidity and pH, were found to be within the Health Canada Guidelines for Canadian Drinking Water Quality at all sample locations.

The key conclusions for the **future** conditions include the following:

1. Both the Camrose Drive and Southlands Reservoirs have sufficient volumes for the estimated future demands.
2. When Camrose Drive reservoir is less than TWL, Paradise PS pumps are running, and the Southlands reservoir is at TWL, pressures of 40 psi cannot be maintained in PA-A above the 180 m contour for future maximum day demands.
3. When the water levels in the reservoirs are at TWL and Paradise PS pumps are off, velocities in the water main on Paradise Road exceed the design limit, and a residual pressure of 40 psi above the 160 m contour cannot be achieved for future maximum day demands.
4. No additional development should be approved for PA-H.
5. Pressures less than 22 psi and velocities greater than 3.0 m/s are experienced when simulating theoretical commercial fire flows. Pressures less than 22 psi are experienced when simulating theoretical residential fire flows.

8.2 Recommendations

The following recommendation should be implemented by the Town:

1. The existing 150 mm water main in the Evergreen Village Trailer Park should be abandoned and replaced, such that it is realigned within the road right-of-way.
2. A new storage tank should be constructed at the high point of Neils Pond Ridge to address the pressure deficiencies experienced at the higher elevations. Pressure zone PA-A should be removed from the Southlands Reservoir and serviced by the new tank. The proposed Karwood Drive development and Fairview Investments development should also be serviced by the new tank.
3. In addition to the new storage tank and proposed service areas, the Town should adopt a policy to limit the future development to the 180 m contour. This policy is only effective if the new tank and proposed service areas scheme are implemented.
4. The Town should check the isolation valves that define PA-H to ensure that they are closed. If they are all closed, the Town should check for leaks in PA-H.
5. The Town should, where possible, install water main connections that result in looping. The effort should focus on cul-de-sacs and unconnected streets.
6. The Town should consider completing a water audit.
7. The Town should consider implementing universal metering.
8. The Town should start a valve maintenance program and develop a valve record in GIS.
9. The Town should develop a GIS database to record information on the water infrastructure.
10. The Town should monitor ammonia, nitrate, nitrite, di- and tri-chloramines, and NDMA, which are undesirable by-products of the chloramination process.
11. The Town should continue with their residual chlorine monitoring program and conduct scheduled flushing activities to improve water age and hence water quality.
12. Flushing should be done in order to reduce water age at problematic locations in the distribution system. The most comprehensive system flushing is done as part of a UDF program. A UDF program should be developed with considerable forethought, allowing operations staff to follow step-by-step procedures and provide the best possible outcome.

Prepared by:
Greg Sheppard, P.Eng.
Project Manager

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APPENDIX A

Fire Flow Tests

2016-06-01

CBCL Limited
187 Kenmount Road
St. John's, NL
A1B 3P9

Attn: Mr. G. Sheppard P. Eng.

Re: Water Flow Test
Paradise

Please Find attached five (5) completed "Water Flow Test" data sheets illustrating the results of the test carried out on your behalf on 2016-06-01 at various locations throughout the Town of Paradise

In summary the following results were recorded:

Test #1 Pollard Avenue

Static pressure 50 psi

920 usgpm flowing at a residual pressure of 40 psi
237 usgpm flowing at a residual pressure of 48 psi
546 usgpm flowing at a residual pressure of 45 psi

Test #2 McNamara Drive

Static pressure 60 psi

1060 usgpm flowing at a residual pressure of 40 psi
254 usgpm flowing at a residual pressure of 55 psi
610 usgpm flowing at a residual pressure of 50 psi

Test #3 St. Thomas Line

Static pressure 103psi

1320 usgpm flowing at a residual pressure of 65 psi
365 usgpm flowing at a residual pressure of 94 psi
484 usgpm flowing at a residual pressure of 92 psi

Test #4 Elizabeth Park Elementary

Static pressure 35 psi

750 usgpm flowing at a residual pressure of 26 psi
187 usgpm flowing at a residual pressure of 30 psi
446 usgpm flowing at a residual pressure of 28 psi

Test #5 Paradise Elementary

Static pressure 50 psi

840 usgpm flowing at a residual pressure of 30 psi
237 usgpm flowing at a residual pressure of 45 psi
538 usgpm flowing at a residual pressure of 41 psi

Trusting this information is complete and sufficient for your records. If we can be of further assistance please do not hesitate to contact this office.

R. W. Blundon
Risk Management Services.

rms
RISK MANAGEMENT SERVICES

#300, 238 Brownlow Avenue
Dartmouth, Nova Scotia
B3B 1Y2

T: 902.423.9287
Toll Free: 1.800.639.4528
F: 902.423.7376

An SCM company
www.scm-rms.ca

TEST # **1**

WATER SUPPLY TEST

Name of Risk:	Not applicable	File No.:	
Address:	Pollard Avenue	Test By:	**Mr. R. Blundon**
Municipality:	Paradise	Date:	**2016-06-01**

SYSTEM DATA:

Size of Main: **6** Dead End: ☒ Two Ways: ☐ Loop: ☐

Source Reliable: ☒ Yes ☐ No If not explain:

Comments:

TEST DATA:

Location of test hydrants: Residual: **First hydrant Pollare Ave. (gauge 023)**

Flow: **Last hydrant Polard Ave. (gauge 003)**

Static Pressure: **50** Time: A.M. **1:00** P.M.

Test No.	No. of Outlets	Orifice Size (in.)	Pitot Reading (psig)	Equivalent Flow gpm (U.S.)	Total Flow gpm (U.S.)	Residual Pressure (psig)	Comments
1	**1**	**2 1/2**	**30**	**920**	**920**	**40**	
1	**1**	**1 1/8**	**40**	**237**	**237**	**48**	
1	**1**	**1 3/4**	**36**	**546**	**546**	**45**	

Additional information/sketch/etc.:

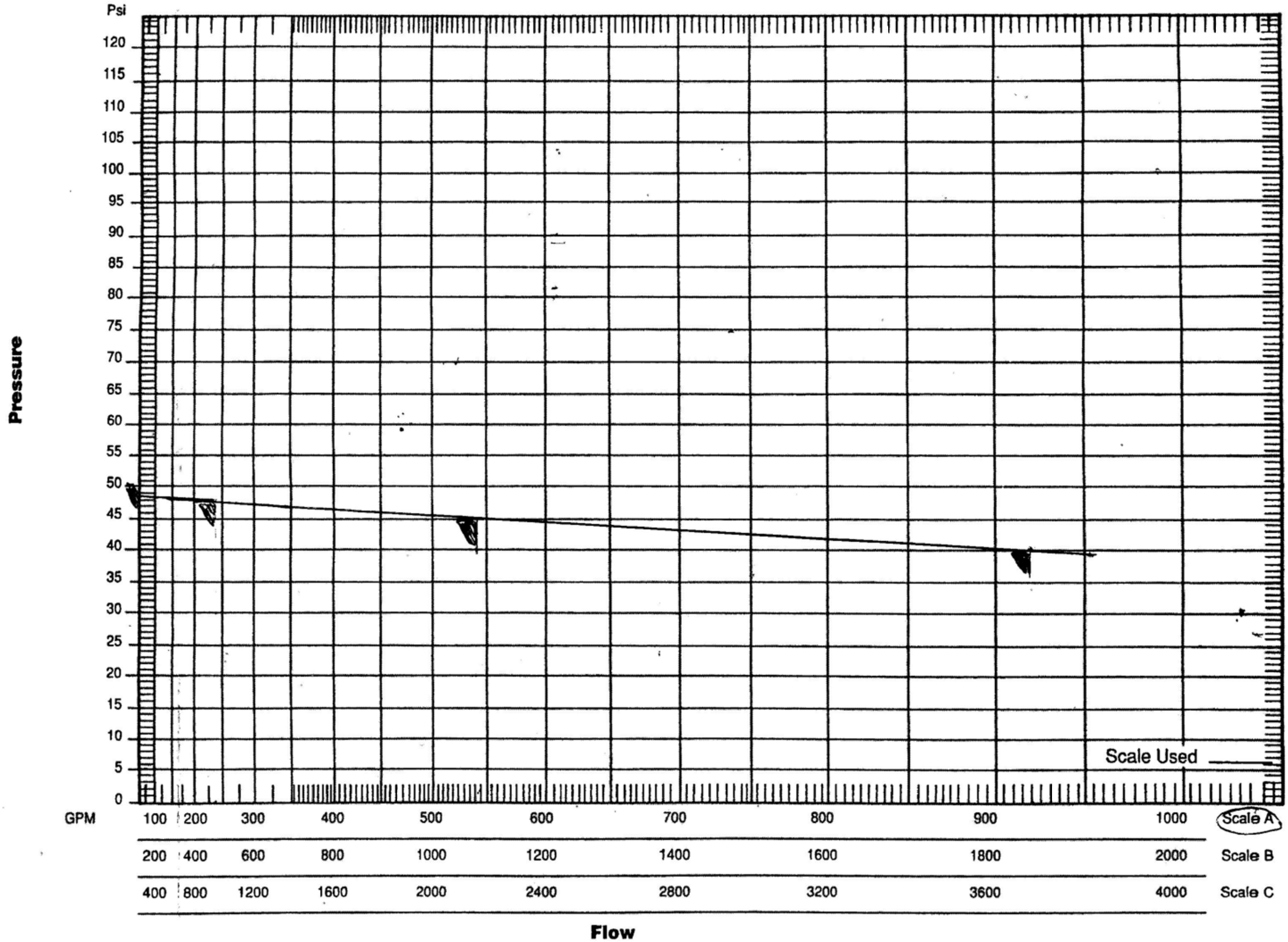
Name and address of municipal authority that should receive a copy:

* RMS reports, prepared in compliance with commonly accepted risk control standards existing at the time services are rendered, are developed from an inspection of the premises and/or from data supplied by or on behalf of the purchaser. RMS does not purport to list all hazards. While changes and modifications, referred to in the reports are designed to upgrade protection and loss prevention of premises, RMS assumes no responsibility for management and control of these activities. RMS will not be responsible to the Purchaser for any losses or damages, whether consequential or other, however caused, incurred or suffered as a result of the services being provided.

TEST #1

Water Flow Test Summary

Location POLLARD AVE Municipality PARADISE, NC Date 2016-06-01



TEST # **2**

WATER SUPPLY TEST

Name of Risk: Not applicable
Address: McNamara Drive
Municipality: Paradise

File No.:
Test By: **Mr. R. Blundon**
Date: **2016-06-01**

SYSTEM DATA:

Size of Main: **12**
Dead End: ☐ Two Ways: ☒ Loop: ☐
Source Reliable: ☒ Yes ☐ No
If not explain:
Comments:

TEST DATA:

Location of test hydrants: Residual: **Opposite 107 McNamara Drive (gauge 023)**
Flow: **Opposite 117 McNamara Drive (gauge 003)**
Static Pressure: **60** Time: A.M. **1:45** P.M.

Test No.	No. of Outlets	Orifice Size (in.)	Pitot Reading (psig)	Equivalent Flow gpm (U.S.)	Total Flow gpm (U.S.)	Residual Pressure (psig)	Comments
1	**1**	**2 1/2**	**40**	**1060**	**1060**	**40**	
1	**1**	**1 1/8**	**46**	**254**	**254**	**55**	
1	**1**	**1 3/4**	**45**	**610**	**610**	**50**	

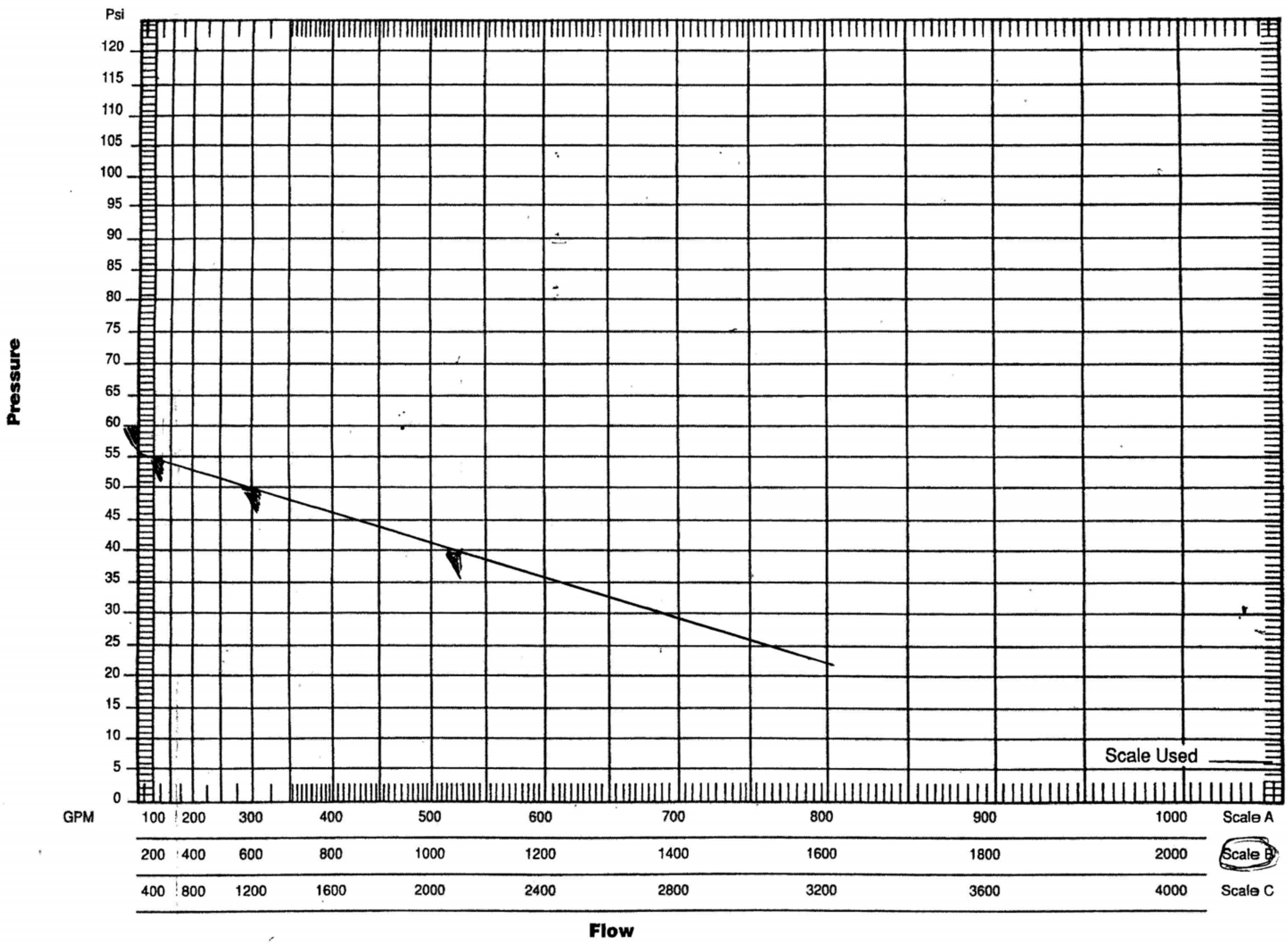
Additional information/sketch/etc.:

Name and address of municipal authority that should receive a copy:

TEST # 2

Water Flow Test Summary

Location McNAMARA DRIVE Municipality PARADISE, NL Date 2016-06-01



TEST # **3**

WATER SUPPLY TEST

Name of Risk:	Not applicable	File No.:	
Address:	St. Thomas Line	Test By:	**Mr. R. Blundon**
Municipality:	Paradise	Date:	**2016-06-01**

SYSTEM DATA:

Size of Main: **12** Dead End: ☒ Two Ways: ☐ Loop: ☐

Source Reliable: ☒ Yes ☐ No If not explain:

Comments:

TEST DATA:

Location of test hydrants: Residual: **Opposite 259 St. Thomas Line (gauge 023)**

Flow: **Opposite 273 St. Thomas Line (gauge 003)**

Static Pressure: **103** Time: A.M. **2:30** P.M.

Test No.	No. of Outlets	Orifice Size (in.)	Pitot Reading (psig)	Equivalent Flow gpm (U.S.)	Total Flow gpm (U.S.)	Residual Pressure (psig)	Comments
1	**1**	**2 1/2**	**62**	**1320**	**1320**	**65**	
1	**1**	**1 1/8**	**95**	**365**	**365**	**94**	
1	**1**	**1 3/4**	**87**	**484**	**484**	**92**	

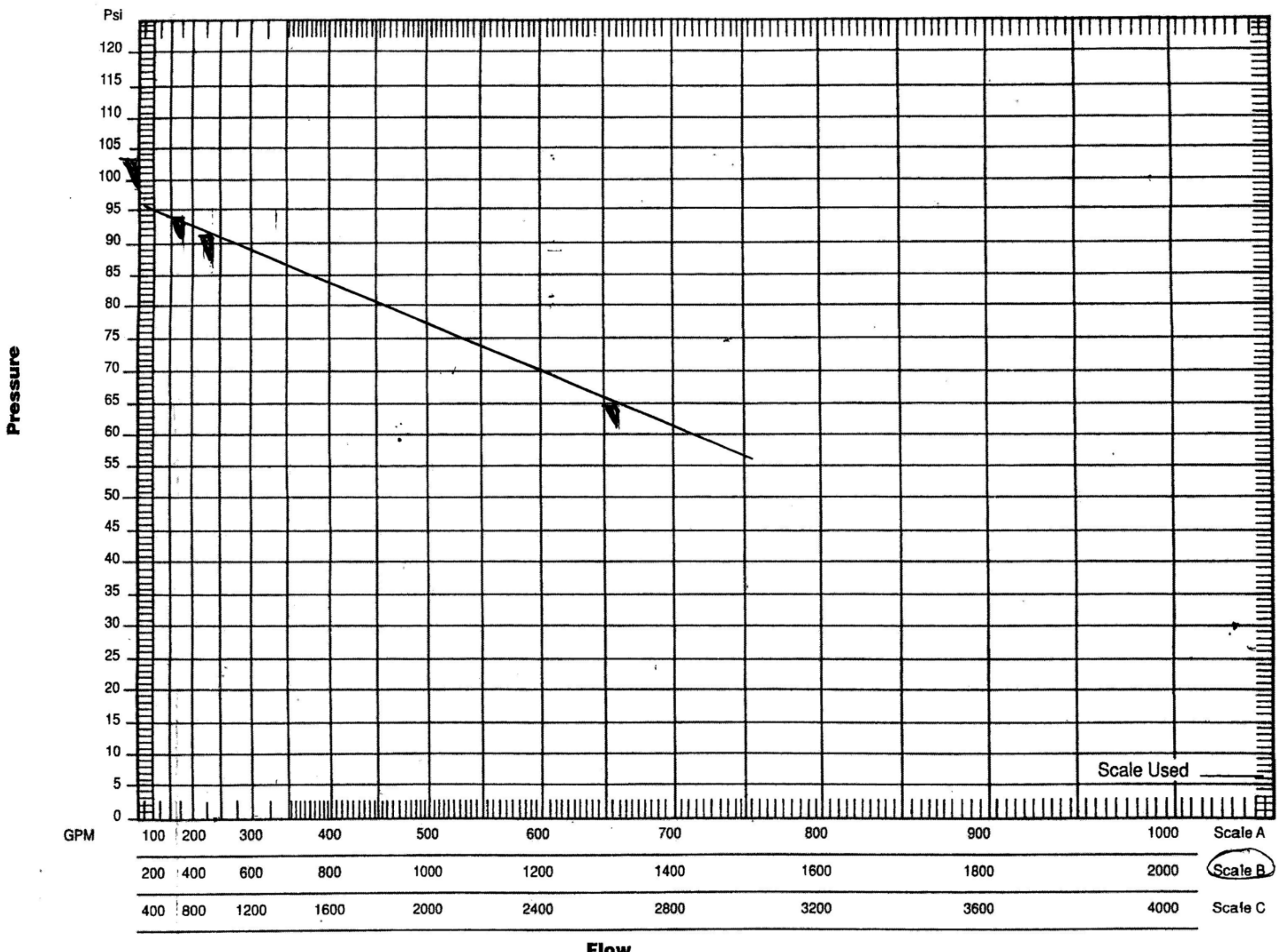
Additional information/sketch/etc.:

Name and address of municipal authority that should receive a copy:

TEST #3

Water Flow Test Summary

Location ST. THOMAS LINE Municipality PARADISE, NL Date 2016-06-01



TEST # **4**

WATER SUPPLY TEST

Name of Risk: Elizabeth Park Elementary
Address: 80 Ellesmere Avenue
Municipality: Paradise

File No.:
Test By: **Mr. R. Blundon**
Date: **2016-06-01**

SYSTEM DATA:

Size of Main: **8** Dead End: ☐ Two Ways: ☒ Loop: ☐
Source Reliable: ☒ Yes ☐ No If not explain:
Comments:

TEST DATA:

Location of test hydrants: Residual: **Opposite 234 Elizabeth Drive (gauge 023)**
Flow: **Last hydrant Ellsmere Avenue (gauge 003)**
Static Pressure: **35** Time: A.M. **3:00** P.M.

Test No.	No. of Outlets	Orifice Size (in.)	Pitot Reading (psig)	Equivalent Flow gpm (U.S.)	Total Flow gpm (U.S.)	Residual Pressure (psig)	Comments
1	**1**	**2 1/2**	**20**	**750**	**750**	**26**	
1	**1**	**1 1/8**	**25**	**187**	**187**	**30**	
1	**1**	**1 3/4**	**24**	**446**	**446**	**28**	

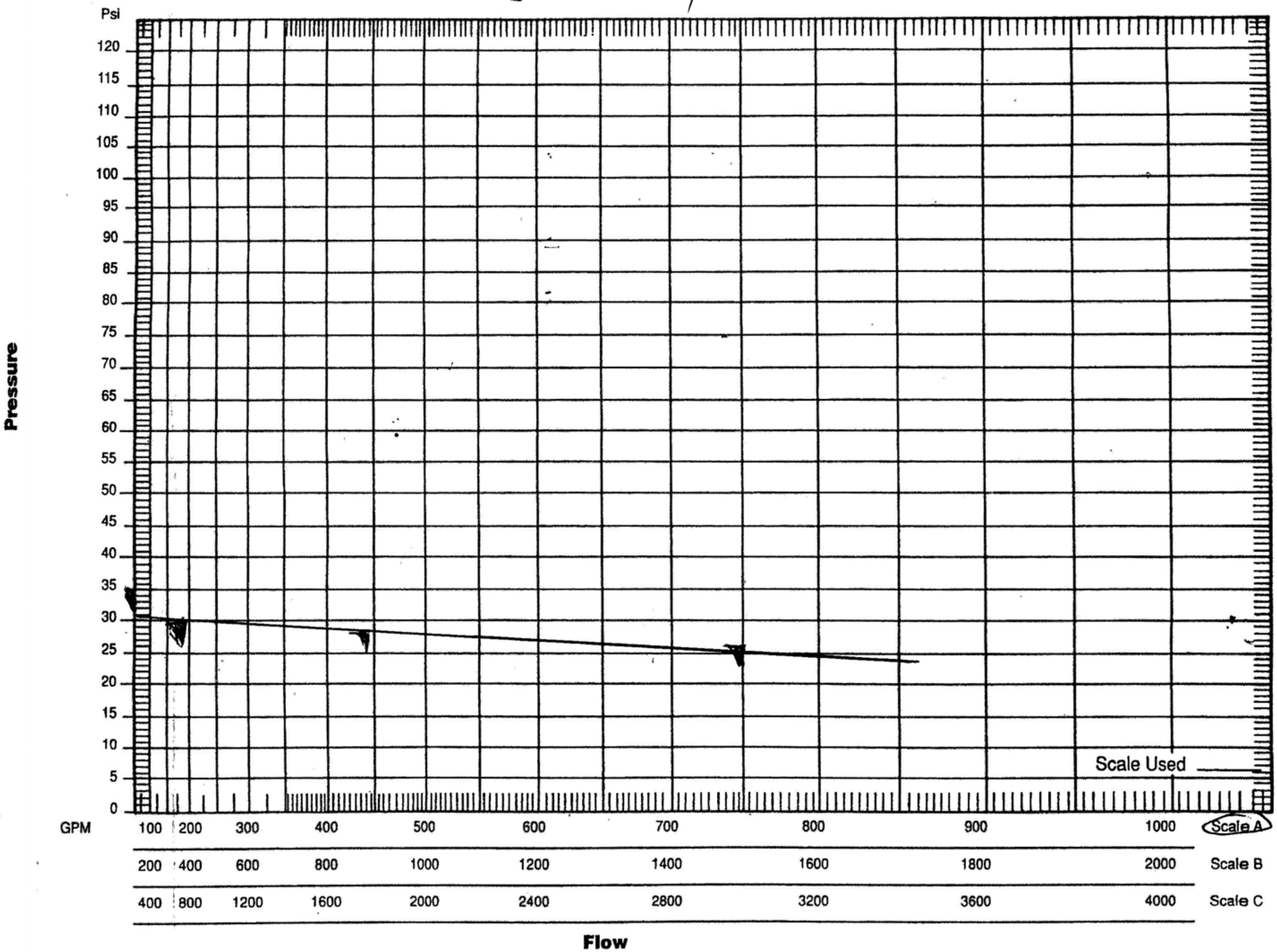
Additional information/sketch/etc.:

Name and address of municipal authority that should receive a copy:

TEST # 4

Water Flow Test Summary

Location ELIZABETH PARK ELEMENTARY Municipality PARADISE, NL Date 2016-06-01



TEST # **5**

WATER SUPPLY TEST

Name of Risk:	Paradise Elementary	File No.:	
Address:	60 Karwood Drive	Test By:	**Mr. R. Blundon**
Municipality:	Paradise	Date:	**2016-06-01**

SYSTEM DATA:

Size of Main: **8** Dead End: ☐ Two Ways: ☒ Loop: ☐

Source Reliable: ☒ Yes ☐ No If not explain:

Comments:

TEST DATA:

Location of test hydrants: Residual: **Parking lot in front of school (gauge 023)**

Flow: **Parking lot north of school (gauge 003)**

Static Pressure: **50** Time: A.M. **4:00** P.M.

Test No.	No. of Outlets	Orifice Size (in.)	Pitot Reading (psig)	Equivalent Flow gpm (U.S.)	Total Flow gpm (U.S.)	Residual Pressure (psig)	Comments
1	**1**	**2 1/2**	**25**	**840**	**840**	**30**	
1	**1**	**1 1/8**	**40**	**237**	**237**	**45**	
1	**1**	**1 3/4**	**35**	**538**	**538**	**41**	

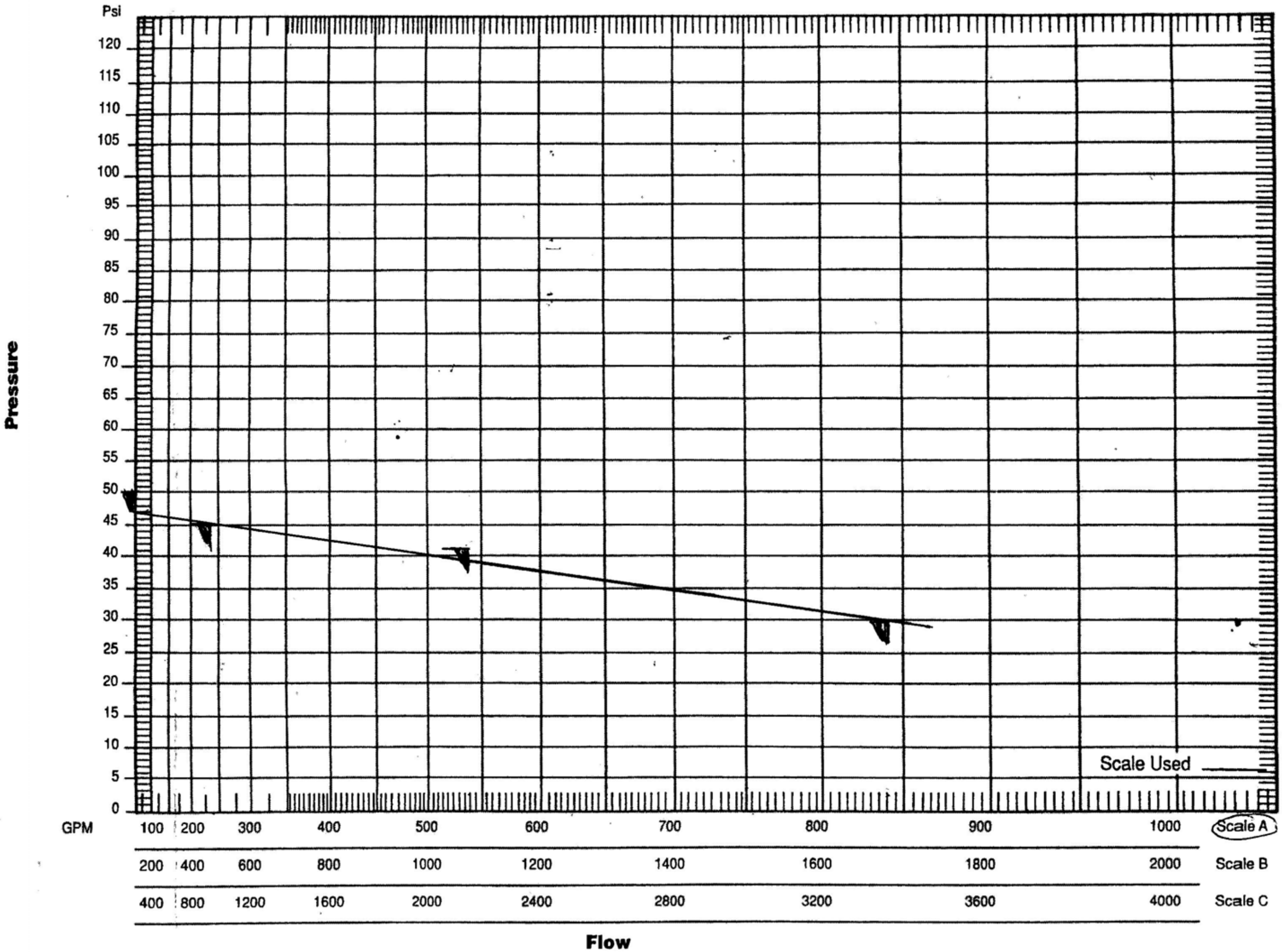
Additional information/sketch/etc.:

Name and address of municipal authority that should receive a copy:

Test #5

Water Flow Test Summary

Location PARADISE ELEMENTARY Municipality PARADISE Date 2016-06-01



APPENDIX B

Cost Opinions

Project: Town of Paradise - Water System Master Plan
Client: Town of Paradise
Date: December 15th, 2015
Estimate: Evergreen Village Water Main Upgrades Class 'D'

OPINION OF PROBABLE COST

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

SECTION	DESCRIPTION	UNIT	QUANTITY	UNIT PRICE	TOTAL
DIVISION # 1					
01010	**Mobilization and Demobilization**				
	(not greater than 5% if on the Island, or 10% if in Labrador, or 15% north of Cartwright, of item a. "sub-total" on last page)	L.S.	Unit	$ 100,000.00	$ 100,000.00
01560	**Environmental Requirements**				
	Silt Fence	m	100	$ 5.00	$ 500.00
01570	**Traffic Regulations**				
	Flagpersons Wages	Hour	1200	$ 30.00	$ 36,000.00
01580	**Projects Sign**				
	Project Sign	L.S.	1	$ 1,500.00	$ 1,500.00
01710	**Reinstatement and Cleaning**				
	1. Supply & Placing Topsoil	m^2	1000	$ 5.00	$ 5,000.00
	2. Supply & Placement of Sods	m^2	1000	$ 5.00	$ 5,000.00
DIVISION #2					
02223	**Excavation, Trenching & Backfilling**				
	Main Trench Excavation				
	1. Rock	m^3	2370	$ 65.00	$ 154,050.00
	2. Common	m^3	7100	$ 30.00	$ 213,000.00
	Service Trench Excavation				
	1. Rock	m^3	1610	$ 65.00	$ 104,650.00
	2. Common	m^3	4840	$ 30.00	$ 145,200.00
	Imported Common Backfill	m^3	2000	$ 15.00	$ 30,000.00
	Granular Pipe Bedding				
	1. Type 1	m^3	1770	$ 45.00	$ 79,650.00
	Supply & Placement of Marking Tape				
	1. Plastic Tape	m	2445	$ 3.00	$ 7,335.00

Project: Town of Paradise - Water System Master Plan
Client: Town of Paradise
Date: December 15th, 2015
Estimate: Evergreen Village Water Main Upgrades Class 'D'

OPINION OF PROBABLE COST

02224	**Roadway Excavation , Embankment & Compaction**				
	Mass Excavation & Backfill				
	1. Common	m^3	3025	$ 25.00	$ 75,625.00
02233	**Selected Granular Base & Sub Base Materials**				
	1. Class "A" Granular Base	tonne	2950	$ 22.00	$ 64,900.00
	2. Class "B" Granular Sub-Base	tonne	5900	$ 20.00	$ 118,000.00
02552	**Hot Mix Asphalt Concrete Mix**				
	Asphaltic Concrete				
	1. Surface Course	tonnes	1670	$ 160.00	$ 267,200.00
	2. Base Course	tonnes	1670	$ 160.00	$ 267,200.00
02574	**Reshaping & Patching Asphalt Pavement**				
	Removal of Asphalt Pavement	m^2	17115	$ 5.00	$ 85,575.00
	Cutting of Asphalt Pavement	m	500	$ 5.00	$ 2,500.00
	Full Depth Asphalt Patch (75mm)	m^2	1500	$ 40.00	$ 60,000.00
02713	**Water Mains**				
	Supply & Installation of Water Main				
	1. PVC, DR-18, 150mm	m	2645	$ 125.00	$ 330,625.00
	Supply & Installation of Service Pipe to R.O.W.				
	1. Municipex, 25mm	m	2150	$ 25.00	$ 53,750.00
	Supply & Installation of Fitting				
	1. End Caps/Plugs (150 mm)	Each	3	$ 150.00	$ 450.00
	2. Bends (150 mm)	Each	22	$ 335.00	$ 7,370.00
	3. Tees (150x150)	Each	27	$ 500.00	$ 13,500.00
	4. Tapped Couplings (25mm off 150mm)	Each	215	$ 280.00	$ 60,200.00
	5. Corp. Stops (25mm)	Each	215	$ 205.00	$ 44,075.00
	6. Curb Stops & Boxes (25mm)	Each	215	$ 360.00	$ 77,400.00
	Supply & Install of Fire Hydrants (6.5')	Each	20	$ 7,500.00	$ 150,000.00
	Supply & Placement of Conc. Thrust Blocks	m^3	35	$ 500.00	$ 17,500.00
	Supply & Placement of Joint Restraints				
	1. 150 mm	Each	237	$ 125.00	$ 29,625.00
	Supply and Install Valves including Valve Boxes				
	1. 150mm	Each	46	$ 3,500.00	$ 161,000.00
	Swabbing of water lines				
	1. 150 mm	m	2445	$ 5.00	$ 12,225.00
	Locating & connecting to existing system	Each	1	$ 2,500.00	$ 2,500.00

Project: Town of Paradise - Water System Master Plan
Client: Town of Paradise
Date: December 15th, 2015
Estimate: Evergreen Village Water Main Upgrades Class 'D'

OPINION OF PROBABLE COST

(A) Subtotal	**$**	**2,783,105.00**
(B) Contingency - (20%)	**$**	**556,621.00**
(C) Engineering - (10%)	**$**	**333,972.60**
(D) Subtotal	**$**	**3,673,698.60**
(E) HST 13%	**$**	**477,580.82**
(F) GRAND TOTAL	**$**	**4,151,279.42**

Project: Water System Master Plan
Client: Town of Paradise
Date: December 9, 2016
Estimate: Option 1: Neils Pond Ridge Storage Tank, Pumping Station and Transmission Main; Class 'D'

OPINION OF PROBABLE COST

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

SECTION	DESCRIPTION	UNIT	QUANTITY	UNIT PRICE	TOTAL
DIVISION # 1					
01010	**Mobilization and Demobilization**				
	(not greater than 5% if on the Island, or 10% if in Labrador, or 15% north of Cartwright, of item a. "sub-total" on last page)	L.S.	Unit	$ 250,000.00	$ 250,000.00
01560	**Environmental Requirements**				
	Silt Fence	m	100	$ 5.00	$ 500.00
01570	**Traffic Regulations**				
	Flagpersons Wages	Hour	500	$ 30.00	$ 15,000.00
01580	**Projects Sign**				
	Project Sign	L.S.	1	$ 1,500.00	$ 1,500.00
01710	**Reinstatement and Cleaning**				
	1. Supply & Placing Topsoil	m^2	13000	$ 5.00	$ 65,000.00
	2. Supply & Placement of Hydroseeding	m^2	13000	$ 5.00	$ 65,000.00
DIVISION #2					
02223	**Excavation, Trenching & Backfilling**				
	Main Trench Excavation				
	1. Rock	m^3	2325	$ 65.00	$ 151,125.00
	2. Common	m^3	5425	$ 30.00	$ 162,750.00
	Imported Common Backfill	m^3	2000	$ 15.00	$ 30,000.00
	Granular Pipe Bedding				
	1. Type 1	m^3	2700	$ 45.00	$ 121,500.00
	Supply & Placement of Marking Tape				
	1. Metallic Tape	m	2220	$ 3.00	$ 6,660.00

Project: Water System Master Plan
Client: Town of Paradise
Date: December 9, 2016
Estimate: Option 1: Neils Pond Ridge Storage Tank, Pumping Station and Transmission Main; Class 'D'

OPINION OF PROBABLE COST

02233	**Selected Granular Base & Sub Base Materials**				
	1. Class "A" Granular Base	tonne	240	$ 22.00	$ 5,280.00
	2. Class "B" Granular Sub-Base	tonne	480	$ 20.00	$ 9,600.00
02552	**Hot Mix Asphalt Concrete Mix**				
	Asphaltic Concrete				
	1. Surface Course	tonnes	70	$ 160.00	$ 11,200.00
	2. Base Course	tonnes	77	$ 160.00	$ 12,320.00
02574	**Reshaping & Patching Asphalt Pavement**				
	Removal of Asphalt Pavement	m^2	800	$ 10.00	$ 8,000.00
	Cutting of Asphalt Pavement	m	300	$ 5.00	$ 1,500.00
	Patching of Asphalt Pavement	m^2	200	$ 75.00	$ 15,000.00
02713	**Water Mains**				
	Supply & Installation of Water Main				
	1. PVC, DR-18, 400mm	m	2220	$ 450.00	$ 999,000.00
	Supply & Installation of Fitting				
	1. Bends (400 mm)	Each	20	$ 2,500.00	$ 50,000.00
	3. Tees (400x400)	Each	2	$ 3,500.00	$ 7,000.00
	Supply & Placement of Conc. Thrust Blocks	m^3	5	$ 1,000.00	$ 5,000.00
	Supply & Placement of Joint Restraints				
	1. 400 mm	Each	60	$ 250.00	$ 15,000.00
	Supply and Install Valves including Valve Boxes				
	1. 400mm	Each	10	$ 6,500.00	$ 65,000.00
	Swabbing of water lines				
	1. 400 mm	m	2220	$ 5.00	$ 11,100.00
	Locating & connecting to existing system	Each	2	$ 2,500.00	$ 5,000.00
02601	**Manhole, Catchbasins, Ditch Inlets & Valve Chambers**				
	Isolation Valve Chamber Complete	Each	1	$ 50,000.00	$ 50,000.00
	Other				
	4ML Bolted Steel Tank Complete	L.S.	1	$ 1,600,000.00	$ 1,600,000.00
	New Pumphouse - Site, Bldg, M&E Complete	L.S.	1	$ 2,200,000.00	$ 2,200,000.00
	Land Acquisition	L.S.	1	$ 100,000.00	$ 100,000.00

Project: Water System Master Plan
Client: Town of Paradise
Date: December 9, 2016
Estimate: Option 1: Neils Pond Ridge Storage Tank, Pumping Station and Transmission Main; Class 'D'

OPINION OF PROBABLE COST

(A) Subtotal	**$**	**6,039,035.00**
(B) Contingency - (15%)	**$**	**905,855.25**
(C) Engineering - (10%)	**$**	**694,489.03**
(D) Subtotal	**$**	**7,639,379.28**
(E) HST 15%	**$**	**1,145,906.89**
(F) GRAND TOTAL	**$**	**8,785,286.17**

APPENDIX C

Sample Valve Report

VALVE REPORT	VALVE NUMBER:
	CADASTRAL:

VALVE LOCATION

STREET: ____________ AVENUE: ____________

VALVE ALIGNMENT: ____________

GPS COORDINATES: ____________

WATERMAIN ☐ **HYDRANT LEAD** ☐ **SERVICE LEAD** ☐ **INFORMATION**

ALIGNMENT: ____________

SIZE: ________ mm TYPE: ________ MAKE: ________

VALVE INFORMATION

VALVE SIZE: ________ mm TYPE: ________ MAKE: ________

MODEL: ________ CLASS: ________

VALVE PURPOSE: MAIN CONTROL ☐ HYDRANT CONTROL ☐ SERVICE CONTROL ☐

GEARED: YES ☐ NO ☐ OTHER ☐ ________

TO OPEN TURN: LEFT ☐ RIGHT ☐ OTHER ☐ ________

VALVE STATUS: OPEN ☐ CLOSE-STOP ☐ OTHER ☐ ________

KEYWAY: CAST IRON ☐ ENCASED IN P.V.C ☐

VALVE INSTALLATION DATE: ________ 19 ________

PROJECT NUMBER ORW.O. &ACCT.: ____________

FOREMAN OR CONTRACTOR: ____________ YARD: ________

IN SERVICE DATE: ________ APPLICATION NO: ________

REMARKS:

FIELD SKETCH

PLEASE INCLUDE COUPLINGS, FITTINGS CHAMBER, PROP. LINES CURBLINE, SIDEWALK ETC.