Servicing Capacity Study

Technical Memorandum #4
Water Distribution Capacity Assessment and Capital Plan

Final

Prepared for:
Municipality of East Hants

RVA 226421

March 23, 2023

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March 23, 2023

Municipality of East Hants
Municipal Office
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Attention: Derek Normanton, P.Eng.

Dear Mr. Normanton:

Re: Servicing Capacity Study - Technical Memorandum #4 - Water Distribution Capacity Assessment and Capital Plan

Please find enclosed Technical Memorandum #4 – Water Distribution Capacity Assessment and Capital Plan for the Regional and Shubenacadie water supply systems in East Hants. This document is one of six components to the Servicing Capacity Study. Kindly have this document reviewed and provide comments back to RVA for our consideration within the final document.

Should you have any questions, please don't hesitate to contract the undersigned.

Yours very truly,

R.V. ANDERSON ASSOCIATES LIMITED

Daniel Guan, CET
Civil Engineering Technologist

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Encls.

East Hants Servicing Capacity Study
Water Distribution Capacity Assessment and Capital Plan

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1.0 Introduction

The Municipality of East Hants (Municipality) has two separate water distribution systems: the Regional Water Distribution System and the Shubenacadie Water Distribution System. The Regional Water Distribution System services the community of Enfield, Elmsdale and Lantz, whereas the Shubenacadie Water Distribution System services the community of Shubenacadie.

As the Municipality continues to be one of the fastest growing communities in Nova Scotia, maintaining an accurate and reliable hydraulic model for its water distribution system is important to the Municipality. The completed water model can guide the Municipal with the provision of safe and reliable water to its residents and allow to prepare a capital investment program to meet the future servicing requirements.

1.1 Objective

R.V. Anderson Associates Limited (RVA) was retained by the Municipality to perform system assessments of the water distribution system through computer modeling to reflect current (2022) conditions and future (2047) conditions.

The objectives of this report are to:

- Develop water distribution model of the Municipality's water distribution system;
- Calibrate water distribution model based on the field test results;
- Identify existing and future system constraints and opportunities to improve the water network's system performance; and,
- Provide recommendations for any system upgrades required to meet the 25-year servicing requirements

The watermain hydraulic model together with its results documented in this report provides an opportunity for the Municipality to evaluate and improve system reliability and flexibility (i.e., pressures and fire flows) based on the current (2022) and the future (2047) population growth, and to use the model as a planning and analysis tool for future developments and system upgrades.

2.0 Water Distribution Model Development

Water modelling software InfoWater (Innovyze) was used to develop the water distribution system hydraulic model for the Municipality. It is a Global Information System (GIS)-based software that integrate GIS information from the Municipality so that the model build and model updates are completed in one GIS application. The hydraulic model provides a representation of the Municipality's water infrastructure where simulations can be conducted under various scenarios and planning horizons.

The spatial reference of the water network in the model was set match of the GIS data for consistency. The geographic and projected coordinate systems used to develop the model are listed below:

- Projected Coordinate System: NAD_1983_UTM_Zone_17N; and
- Geographic Coordinate System: GCS_North_American_1983

2.1 Modeling Theory

Hydraulic model represents pipes within a network using links connected at junctions or nodes. The GIS watermain shapefile was imported into the model. Nodes were inserted in the model where pipe material or diameter changes. The water usage or demands were assigned to each respective nodes within the network. Noted that not every pipe in the network is included, and small pipes delivering water to each individual user are omitted; relatively minor demands within the network, such as individual homes, are grouped and represented by a single node. Water is distributed through the network links from a storage facility (a reservoir, or tank) to the demand nodes by pumps. Various simulations are solved using a series of mass balance and energy conservation equations developed at each node and link in the distribution network.

2.2 Node Elevation Data

The junction/node Elevation data was obtained from the 2020 Lidar provided by Nova Scotia DataLocator.

Note that node elevations in the model are surface elevations and not watermain elevations. This is standard practice as the pressure and flow value produced by the model is simulating the available flow and pressure at grade.

2.3 Pipe Roughness Coefficients (C-Factors)

The Hazen Williams "C" factor is an empirical coefficient that related flow through a pipe of a certain size to the head loss across its length, according to the following relationship:

$$C = \frac{Q}{278.5 D^{2.63} \left(\frac{H_l}{L} \right)^{0.54}}$$

C = Hazen Williams C factor

Q = flow in pipe, L/s

D = pipe diameter, m

H_l = head loss, m

L = length, m

The C-factor will decline as the interior wall of an unlined pipe corrodes. The rate of decline for larger diameter pipes is typically slower than that of smaller diameter pipes. Watermains with pipe material like Iron mains are typically replaced or rehabilitated with cement mortar or epoxy lining when the C-factor declines to 50-60 (i.e., 50% of the capacity of a new watermain).

The initial C-Factors for various pipe sizes and materials used for calibrating the model were based on industry standards. These C-factors were then refined during the calibration process based on field tests.

The initial C-factors that were used in developing the hydraulic model are provided in the table 1-1 below:

TABLE 2.1 - INITIAL C-FACTOR

.l	)	
DI	All	120
HDPE	All	130

2.4 Regional Water Distribution System

The Municipality's Regional Water Distribution Network is a surface water-based system, consisting of the following:

- One (1) water intake structure and surface water treatment plant located in Enfield, Nova Scotia, known as Enfield Water Treatment Facility (WTP).
- Three (3) Standpipe
- Approximately 57 km of watermain
- One (1) Bulk Water Station

Figure 2.1 shows an overview of the Regional Water Distribution Network. The water network is operated as one single pressure zone. Based on the Lidar survey, low-ground elevations are located mostly on the east side of the system, while most high-ground elevations appear to be on the west and east side of the system.

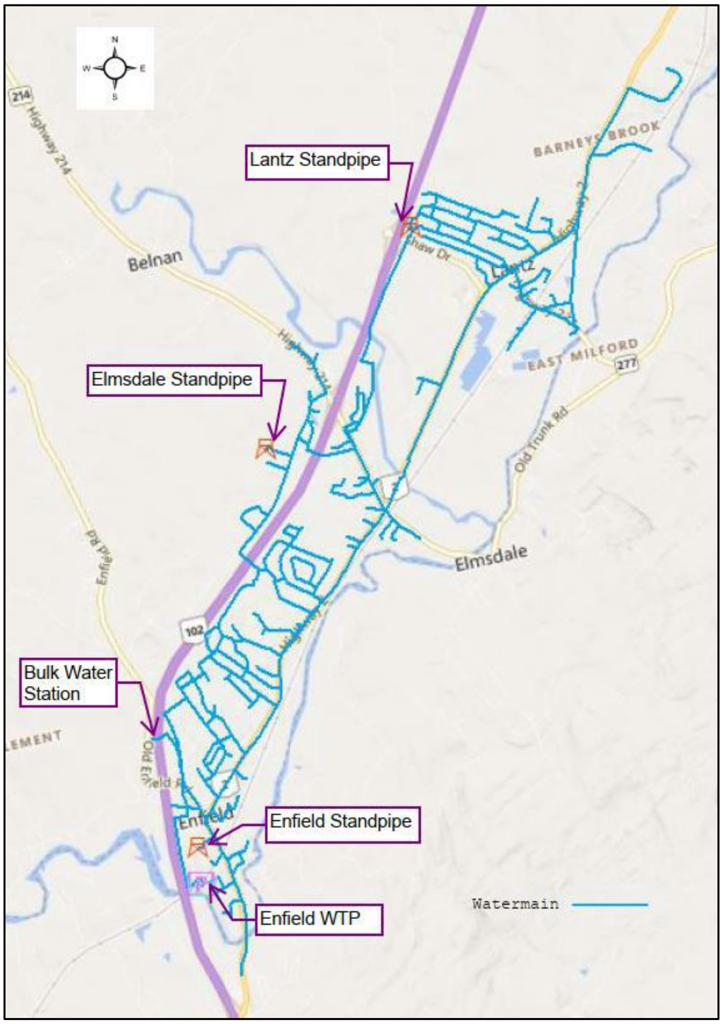


Figure 2.1 – Overview of the current Regional Water Distribution System

2.4.1 Water Sources

The Regional Water Distribution System is supplied by a single surface water treatment plan with a maximum allowable flow rate is 11,000 m³/d. Raw water from the Shubenacadie River is treated through filtration and disinfection prior to entering clear well. Water level in the clear well is assumed to be at 18.342 m (60% full) under all scenarios simulated using steady-state analysis in the hydraulic model.

2.4.2 High Lift Pumps

The three (3) high lift pumps (HLP) are identical. Their make and mode are Layne 11EM 4, 4 Stage Vertical Turbine Pump. Based on SCADA, they are normally operated in three (3) configurations to deliver treated water from clean well to the distribution system. Most of the time, only one (1) HLP is operated at 88% speed. When demand in the system surpass normal condition, the second HLP turns on at the same rate of 88% speed. When demand increase further, one of HLP ramp up to 90-95% speed, while the other maintain at 88% speed. In all configurations, the third HLP is offline for redundancy. In the hydraulic model, one (1) HLP is set to be in operation during peak hour demand; Two (2) HLP are set to be in operation in case of fire event in which is consistent with the hydraulic model setup and boundary conditions used.

With one HLP in operation at 88% speed, the typical outflow is 32.5 L/s. The typical discharge pressure at the WTP over the years from 2020 to 2022 ranged from 431 - 484 kPa (44 – 49.4 m) at grade with a Hydraulic Grade Line (HGL) of 61.9 – 67.4 m.

The pump curves created based on the manufacturer's pump data sheet is shown in Figure 2.2.

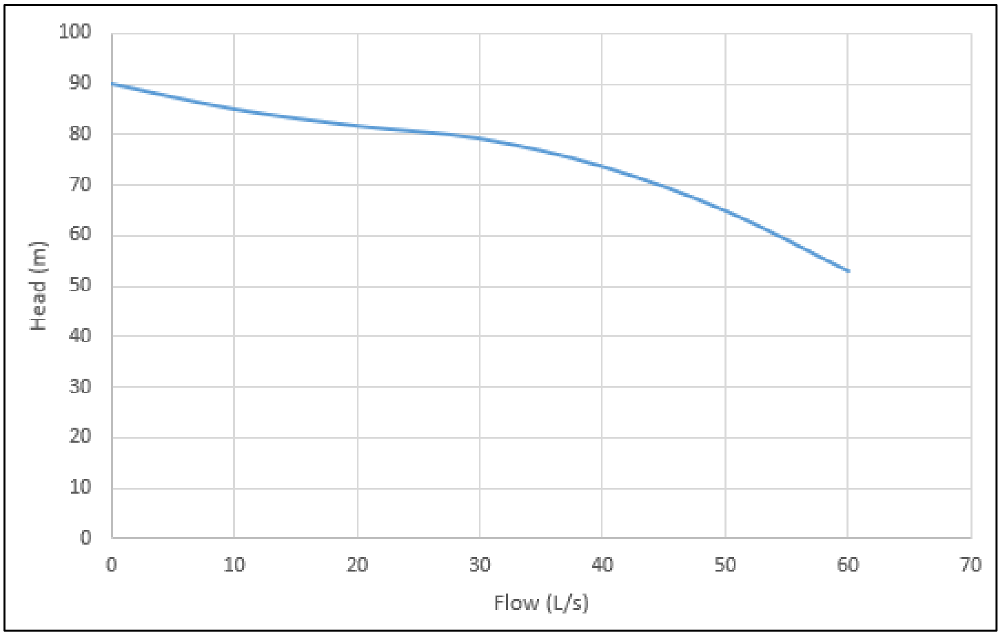


Figure 2.2 – Regional HLP Pump Curve

2.4.3 Water Storage

2.4.3.1 ENFIELD STANDPIPE

Enfield Standpipe is a cylindrical standpipe located at Convent Road, Enfield, with a diameter of 7.85 m and was built in 1977. Relevant elevation and water level in the standpipe, gathered from SCADA and as-built Drawings, are shown in Table 2.2. The average water level in the Enfield Standpipe over the years from 2020 to 2022 was 27.69m (HGL 63.36m) and was used during the hydraulic modeling. The total volume of water storage was calculated to be 1431 m³ at high operating level.

TABLE 2.2 – ENFIELD STANDPIPE

Location	Water Level (m)	HGL (m)
Bottom of Tank	-	35.67
Low Operating Level	25.30	60.97
High Operating Level	29.57	65.24
Top of Tank	31.11	66.78

2.4.3.2 ELMSDALE STANDPIPE

Elmsdale Standpipe is a cylindrical standpipe located at Venture Cresent, Elmsdale, with a diameter of 13.7 m and was built in 2005. Relevant elevation and water level in the standpipe, gathered from SCADA and as-built Drawings, are shown in Table 2.3. The average water level in the Elmsdale Standpipe over the years from 2020 to 2022 was 27.39 m (HGL 65.04m) and was used during the hydraulic modeling. The total volume of water storage was calculated to be 4067 m³ at high operating level.

TABLE 2.3 – ELMSDALE STANDPIPE

Location	Water Level	HGL (m)
Bottom of Tank	-	37.65
Low Operating Level	24.99	62.64
High Operating Level	29.26	66.91
Top of Tank	29.40	67.05

2.4.3.3 LANTZ STANDPIPE

Lantz Standpipe is a cylindrical standpipe located at Logan Drive, Lantz, with an estimated diameter of 9.14 m and the construction date is unknown. Relevant elevation and water level in the standpipe, gathered from SCADA and as-built Drawings, are shown in Table 2.4. The average water level in the Elmsdale Standpipe over the years from 2020 to 2022 was 26.57m (HGL 61.57m) and was used during the hydraulic modeling. The total volume of water storage was calculated to be 1940 m³ at high operating level.

TABLE 2.4 – LANTZ STANDPIPE

Location	Water Level (m)	HGL (m)
Bottom of Tank	-	35.00 *
Low Operating Level	24.99	59.99
High Operating Level	29.57	64.57
Top of Tank	-	n/a

Note: * Elevation of Bottom of Tank is obtained from Google Earth. As such, number in Elevation column are estimated.

2.4.4 Population Estimate

The Municipality provided RVA with low, mid and high-range population estimates from 2022 to 2045 for demand allocation and analysis purpose of this study. Projected population from 2045 to 2047 are currently not available. RVA was instructed to use the mid-range population projection. Based on this mid-range projection, the estimated population in 2045 is 24,392. See Table 2.5 below.

Additionally, for the purposes of calculating the per capita water usage, RVA used Census data and linear interpolation to determine the approximate population between 2016 – 2021 using the Municipality's data from 2022 – 2045.

The Municipality of East Hants provided the following supporting information with their population projections:

> "*Development in the South Corridor and Commercial Growth Management Area is the most predictable as most of the bulk lands are either already approved for development, have active applications for development, or there is speculation about future development in those areas. Based on this information, planning staff put together the table above of estimated build out times for each approved, proposed, and speculative development. From there, staff projected how many new units they anticipate could be developed per development each year. "

> "*Disclaimer: development and population values are estimates generated by Municipal staff based on a combination factor including, but not limited to known/proposed future development, development trends, zoning, land vacancy, and the 2021 Census data for average number of people per dwelling in the area. Low, mid, and high estimates for development and population have been calculated to cover a range of possible scenarios. These numbers are to be used as a guide only; the Municipality does not guarantee the accuracy of any data provided herein."

TABLE 2.5 – POPULATION ESTIMATES

Year	Population
2016 *	6,807
2020 **	7,831
2021	8,087
2022	9,040
2028	16,085
2034	20,365
2040	23,185
2045	24,392

* Statistics Canada, 2016 Census

** Interpolated between Census and projected 2022 population

2.4.5 Historical Water Demand and Factors

The Municipality monitors treated water flow from the WTP to the distribution system and water level fluctuations in each standpipe in the system. Table 2.6 below summary the average daily water usage within Regional Water System between 2020 and 2022.

TABLE 2.6 – AVERAGE DAILY DEMAND IN REGIONAL SYSTEM

Year	Average Daily Demand (m^3/d)
2020	2,614
2021	2,414
2022	2,770

Accordingly, the average daily demands varied between 2,414 and 2,770 m^3/d, with a total average daily demand of 2,599 m^3/d (30.1 L/s) from 2020 to 2022.

Maximum daily demand and peak hourly demand were determined by comparing the outflow volume from the WTP and water level fluctuations in each standpipe to attain the actual volume consumed by residents. The maximum daily factor was calculated by dividing the maximum weekly average flow by the daily average flow. The maximum daily factor turns out to be 1.48.

Peak hourly factor was calculated by dividing the maximum hourly flow, attained in the date with the maximum daily demand, by the daily average flow. The peak hourly factor turns out to be 4.07.

The Per Capita Water Usage of the Regional Water Distribution System was calculated using historical water use data, as well as Census data and linear interpolation to determine the approximate population between 2016 and 2021 using the Municipality's data from 2022 – 2045. The Per Capita Water Usage was calculated to be 313 L/Cap/Day.

2.4.5.1 LARGE WATER USERS

A review of the largest water users was undertaken based on 2022 Q3 billing data. Water consumption of each of these users in 2022 Q3 was compared as a percentage of the total daily flow of the systems. The recorded top water demand consumptions and respective user locations are listed in Table 2.7. The remainder of the water users were found to have minor impact and thus not reviewed further.

TABLE 2.7 – TOP WATER USER UNDER AVERAGE DAILY DEMAND IN THE REGIONAL SYSTEM

Location	Large Users -% of Total Daily Flow
1101 HIGHWAYS 2	2.38%
6767 HIGHWAYS 2	2.16%
707 HIGHWAYS 2	1.16%
269 HIGHWAYS 214	1.10%
604 HIGHWAYS 2	1.03%
20 CONCORDE WAY	0.84%
14 COMMERCE COURT	0.74%
192 HIGHWAYS 2	0.47%
161 HIGHWAYS 277	0.38%
6 HORNE ROAD	0.38%
Total	10.65%

2.4.5.2 BULK WATER STATION

The Municipality has one (1) bulk water station located at 129 Old Enfield Road, Enfield. RVA compared the usage log provided by the Municipality from 2018 to 2021. Table 2.8 below presents a summary of the average daily flow, maximum daily flow and peak hourly flow from 2018 to 2021.

TABLE 2.8 – BULK WATER STATION CONSUMPTION

Year	Average Daily Demand (L/s	Max Daily Demand (L/s)	Peak Hourly Demand (L/s)
2018	0.94	3.92	22.48
2019	0.97	3.64	16.53
2020	1.34	5.62	23.01
2021	1.17	4.67	21.36

Based on the above table, the overall average daily demand is 1.11 L/s, while maximum daily demand and peak hourly demand over the years are 5.62 and 23.01 L/s, respectively, over a period of four years.

2.4.6 Demand Allocation

Water demand scenarios were developed in the hydraulic model to simulate Average Daily Demand (ADD), Maximum Daily Demand (MDD) and Peak Hourly Demand (PHD). In the absence of geo-coded customer water billing, in each scenario, demand from Top 10 large water users and bulk water station were assigned to the relevant junctions, and the remaining water demand were evenly distributed to the rest of junctions in the system.

2.4.7 Operational Control Narrative

The operational control points in the hydraulic model for HLPs and standpipes were set based on SCADA information provided by the Municipality. It is understood that under normal operating conditions, the discharge pressure at the WTP is 461 kPa (67 psi) at grade. However, control narrative regarding the ON/OFF setpoint of HLP in terms of water level in each standpipe is not available. It is assumed that under normal condition, one (1) HLP turn on when all three standpipes reach the low operating level and remain in operation until all three standpipes reach each respective high operating water level.

2.5 Shubenacadie Water Distribution System

The Shubenacadie Water Distribution Network is a well-based system, consisting of the following:

- Two (2) underground wells associated with water treatment plant in Shubenacadie
- One (1) standpipe
- Approximately 9.1 km of watermain

The water network is operated as one single pressure zone. Overview of the current water distribution system is shown in Figure 2.3. High ground elevations appear to be on the southeast side of the system. The rest of the area are in low ground elevations.

Figure 2.3 – Overview of the current Shubenacadie Water Distribution System

2.5.1 Water Sources

The Shubenacadie Water Treatment Plant is located at 2882 Highway 2 Shubenacadie, Nova Scotia, and currently runs off two groundwater wells, PW-2 (installed in 2007) and PW-2019 (installed in 2019). When the plant was first commissioned, it was supplied by wells PW-1 and PW-2; however, PW-1, has since been decommissioned because the internal components of the well failed. The WTP has a design capacity of 1,090.2 m³/d, an approved average withdrawal limit of 740 m³/d, and an approved maximum withdrawal limit of 1,000 m³/d. Raw water from the production wells is treated by filtration and disinfection prior to entering the distribution system.

2.5.2 Well Pumps

There are currently two (2) ITT Goulds Model 160L 15 Well pumps, as well as a shelf spare. Each has a rated capacity of 9.5 L/s @ 76 m head. One (1) Well pump is set to be in operation in the current steady state during peak hour demand in the model; Two (2) HLP are set to be in operation in case of fire event in the model. It should be noted that there is no information available regarding the discharge pressure from the WTP.

2.5.3 Water Storage

Shubenacadie Standpipe is a cylindrical standpipe located at Second Street, Shubenacadie, with a diameter of 10.23 m. Relevant elevation and water level in the standpipe, gathered from record drawings, are shown in Table 2.9. The average water level in the Shubenacadie Standpipe over the years from 2020 to 2022 was 30.28 m (HGL76.93m). The total volume of water storage was calculated to be 2,741.1 m³ at high operating level.

TABLE 2.9 – SHUBENACADIE STANDPIPE

Location	Water Level	HGL (m)
Bottom of Tank	-	46.65
Low Operating Level *	-	n/a
High Operating Level	33.35	80.00
Top of Tank	34.07	80.72

* Low operating level not available in record drawings

2.5.4 Population Estimate

The Municipality of East Hants provided RVA with low, mid, and high-range population estimates from 2022 – 2047 for the purpose of this assessment. RVA was instructed to use the mid-range population projection for this study. Based on this mid-range projection, the estimated population in 2047 is 1033. See Table 2.10 below. RVA used Census data and linear interpolation to determine the approximate population between 2016 – 2021 using the Municipality's data from 2022 – 2047.

The Municipality of East Hants provided the following supporting information with their population projections:

> "Population estimates for Shubenacadie are based on 2.5 people/dwelling and the potential development as described in the development table. Very little development has been approved in Shubenacadie over the last number of years due to a lack of service capacity. The current construction of a new sewer treatment plant is expected to trigger some increase in development in the area. Low, medium

and high development scenarios have been broken out below based on development estimates."

"*Disclaimer: development and population values are estimates generated by Municipal staff based on a combination factor including, but not limited to: known/proposed future development, development trends, zoning, land vacancy, and the 2021 Census data for average number of people per dwelling in the area. Low, mid, and high estimates for development and population have been calculated to cover a range of possible scenarios. These numbers are to be used as a guide only; the Municipality does not guarantee the accuracy of any data provided herein."

TABLE 2.10 – POPULATION ESTIMATES

Year	Population
2016 *	735
2021 **	775
2022	785
2027	850
2032	920
2037	958
2042	995
2047	1033

* Statistics Canada, 2016 Census

** Interpolated between Census and projected 2022 population

2.5.5 Historical Water Demand and Factors

The Municipality monitors treated water flow from the WTP to the distribution system. Table 2.11 below summary the average daily water usage in Regional Water System between 2017 and 2021.

TABLE 2.11 – AVERAGE DAILY DEMAND IN SHUBENACADIE SYSTEM

Year	Average Daily Demand (m^3/d)
2017	282
2018	273
2019	327
2020	277
2021	244

The average daily demands varied between 244 and 327 m³/d, with an overall average daily demand of 281 m³/d (3.25 L/s).

Maximum daily demand and peak hourly demand were determined by comparing the outflow volume from the WTP and water level fluctuations in each standpipe to attain the actual volume consumed by residents. Maximum daily factor was calculated by dividing the maximum weekly average flow by the daily average flow. The maximum daily factor turns out to be 2.01. Peak hourly factor was calculated by dividing the maximum hourly flow, attained in the date with the maximum daily demand, by the daily average flow. The peak hourly factor turns out to be 3.86.

The Per Capita Water Usage of the Regional Water Distribution System was calculated using historical water use data, as well as Census data and linear interpolation to determine the approximate population between 2016 and 2021 using the Municipality's data from 2022 – 2047. The Per Capita Water Usage was calculated to be 366 L/Cap/Day.

2.5.6 Demand Allocation

Water demand scenarios were developed in the hydraulic model to simulate Average Daily Demand (ADD), Maximum Daily Demand (MDD) and Peak Hourly Demand (PHD). In the absence of geo-coded customer water billing, in each scenario, demand was evenly distributed to each junction in the system.

2.5.7 Operational Control Narrative

Operational control points in the hydraulic model for well pumps and standpipe were set based on the information provided by the Municipality. It is understood that under normal operating conditions, the discharge pressure at the WTP is 565 kPa (82 psi) at grade. However, control narrative regarding the ON/OFF setpoint of well pumps is not available. It is assumed that under normal condition, one (1) well pump turn on when the standpipe reach the low operating level and remain in operation until the standpipe reach the high operating water level.

3.0 Water Distribution Model Calibration

Model calibration is the process of comparing the model results with actual field results, then adjusting the model parameters to improve the overall accuracy of the model results. The intent is to bring the modeling results as close as possible to the field results. All calibrations were completed after the model is set up with the parameters based on the system information and demands as noted in the sections above.

3.1 Methodology

Steady-state and fire flow method was used to calibrate the hydraulic model. This method generally involves estimating the C-factor, then comparing the simulated results with measured results. The model is then updated manually at the field test locations by adjusting the C-factors. If the simulated and measured results do not agree within the acceptable range, then the C-factors demands are revised, and the process is repeated until the results are within the acceptable range. The following procedures were used for the calibration work:

- Compare the measured static Hydraulic Grade Line (HGL) and the simulated HGL for each test. The static HGL is defined as the hydraulic grade at the test location during regular ADD conditions.
- Check the ground elevation at the location where the residual pressure was measured for each test.
- Compare the measured residual HGL and the simulated HGL for each test. The residual HGL is defined as the hydraulic grade at the test hydrant location when the hydrant is flowing at a measured rate.
- Check the pipe connectivity at each test location.
- Check water demands in the vicinity of each test location.
- If necessary, revise the ground elevation, pipe connectivity and/or water demands and re-run the simulations and repeat steps 1-5.
- Compare the measured head loss and the simulated head loss for each test. The head loss is the difference between the static HGL and the residual HGL at the test location.
- If necessary, revise the roughness coefficients of pipe segments (C-factors) and re-run the simulations and then repeat the steps.

Standard practice defines a calibrated model as one where the existing distribution system is calibrated to within ±5% static pressure, ±5% residual pressure, and ±10% fire flow at 138 kPa (20 psi) at all recorded points for the calibration conditions. However, at any step in the procedure, inconsistent data should be challenged and discarded if there are doubts about field data quality.

3.2 Field Testing

Nine (9) strategic location for fire hydrant tests were selected by RVA. A map of locations is available in Appendix A.

Field testing was originally conducted on October 13, 2022 and retested on November 16, 2022 by Aqua Data Inc. Due to a technical error and desirable field condition, Aqua Data

Inc. only opened one (1) 2.5” outlet for testing instead of two (2) 2.5” outlet, which may cause inadequate pressure drop and underestimate field fire flow results.

To interpret the test data to flow rates from the outlet of the hydrant, the following formula was used,

$$Q_F = 1.883\ (c)\ (d^2)\ \sqrt{p}$$

where:

Q_F = Total residual flow during the test, Litre per second (L/s);

c = Coefficient of discharge;

d = Diameter of the outlet, inches; and

p = Pitot pressure, pounds per square inch (psi).

NFPA 291 (2010) recommends that a residual pressure of 138 kPa (20 psi) be maintained in fire hydrants for them to be effective for firefighting and preventing the contamination of public water supplies by infiltration.

To obtain the fire hydrant flow at 138 kPa (20 psi), the following formula was used,

$$Q_R = Q_F\ (h_r \div h_f)^{0.54}$$

where:

Q_R = Flow predicted at 20 psi, L/s;

Q_F = Total flow measured during the test, L/s;

h_r = Pressure drop to the desired residual pressure, psi; and

h_f = Pressure drop measured during the test, psi.

Table 3.1 below presents a summary of the result of the field test.

TABLE 3.1 – HYDRANT TEST

Test #	Test Hydrant Static Pressure (kPa)	Flow Hydrant Static Pressure (kPa)	Test Hydrant Residual Pressure (kPa)	Pressure Drop in Test Hydrant (kPa)	Pitot Reading (psi)	Fire Flow @ 20 psi (L/s)
1	469	303	372	96	9.9	68
2	358	345	317	41	19.9	117
3	413	413	372	41	21.8	107
4	469	420	441	28	23.8	198
5	345	345	317	28	21.8	147
6	372	351	372	0	22.8	340
7	413	455	372	41	26.8	153
8	606	586	579	28	33.7	284
9	634	634	593	41	27.8	214

It was noted that in Test #1, the difference between the test hydrant's static pressure and flow hydrant's static pressure was significant. As static pressure is a function of ground elevation, it means the elevation of two hydrants are not at the same range and would cause inaccuracy of results when estimating the fire flow at the test hydrant. In addition, in Test #6, zero pressure drop in test hydrant was observed. Theoretically and practically, it is impossible to have zero pressure drop in hydrant testing given that there is no interference of pressure sustaining valve or pumping station nearby. The doubtful result in Test #6 could be caused by human error.

Considering these factors, the results in Test #1 and #6 are not considered.

3.3 Model Calibration

3.3.1 Pre-Calibration

The boundary condition of the calibration scenario was setup based on the water consumption and water level in the storage facility on the date of hydrant testing to match the field condition.

Table 3.2 and Table 3.3 below summarize the comparison of the hydrant tests and the existing model prior to any calibration adjustments.

TABLE 3.2 – STATIC PRESSURE COMPARISON

Test #	Test Hydrant Static Pressure In Field (kPa)	Test Hydrant Static Pressure In Model (kPa)	Variance between Model and Field
2	358	355	1%
3	413	422	-2%
4	469	481	-3%
5	345	353	-2%
7	413	427	-3%
8	606	625	-3%
9	634	663	-4%

It can be observed that all static pressure variance between field condition and model condition are within the tolerable range of 5%. An acceptable static pressure variance is indicative of boundary condition regularity. This means that the settings in the model simulations, including assumed elevated tank levels, pump curve, and node elevations, are close to the field conditions. As such, no further adjustment is required for these parameters.

TABLE 3.3 – RESIDUAL PRESSURE & FIRE FLOW COMPARISON – UNCALIBRATED

Test #	Test Hydrant Residual Pressure In Field (kPa)	Test Hydrant Residual Pressure In Model (kPa)	Variance between Model and Field	Field Flow @ 138 kPa (L/s)	Model Flow @ 138 kPa (L/s)	Variance between Model and Field
2	317	305	4%	117	106	10%
3	372	388	-4%	107	126	-15%
4	441	454	-3%	198	239	-17%
5	317	329	-4%	147	175	-16%
7	372	376	-1%	153	144	6%
8	579	607	-5%	284	353	-20%
9	593	638	-7%	214	288	-26%

In terms of the flow situation, the residual pressure variance between field condition and model condition are within the tolerable range of 5%, except Test #7. As to Fire flow @ 138 kPa, variance between field condition and model condition are significant with most of them exceeding the tolerance of 10%. That could indicate inconsistencies of Watermain C-factor and/or network connectivity between model simulations and field condition.

3.3.2 Model Adjustment

During flow conditions, discrepancies became obvious and were mainly caused by incorrect pipe roughness and/or incorrect pipe connectivity. In the major deviations, the model overestimated the flows compared to the field conditions. This could indicate the initial C-factor used were more radical than the actual condition indicated.

The water network model was disaggregated into separate logical calibration groups based on the known physical characteristics of the associated pipes (material, location), and watermain C-factor in the critical area were revised in a trial-and-error manner until the variances within the tolerance are reached between observed and simulated conditions. Table 3.4 below indicate the revised watermain C-factor that satisfy the purpose of calibration.

TABLE 3.4 - ADJUSTED C-FACTOR

Pipe Material	Diameter (mm)	C-Factor
DI	All	108 - 120
PVC	All	110 - 125
HDPE	All	130
AC	All	90

3.3.3 Post-Calibration

Table 3.5 below presents a summary of the comparison of the hydrant tests and the hydraulic model after the calibration adjustments.

It can be seen that all residual pressure variance between field condition and model condition are inside the tolerable range of 5%. And all flow variance between field condition and model condition are inside the tolerable range of 10%.

Based on these results, it can be concluded that the calibrated model matches the field condition and can be used to analyze the performance of the water distribution system. However, the resulting hydraulic conditions analyzed during the course of this study are only based on the current conditions of the system. It is recommended to re-calibrate the model periodically to reflect the physical changes in both operations and facilities in the system to assure an acceptable confidence level of the model.

TABLE 3.5 – RESIDUAL PRESSURE & FIRE FLOW COMPARISON - CALIBRATED

Test #	Test Hydrant Residual Pressure In Field (kPa)	Test Hydrant Residual Pressure In Model (kPa)	Variance between Model and Field	Field Flow @ 138 kPa (L/s)	Model Flow @ 138 kPa (L/s)	Variance between Model and Field
2	317	309	3%	117	110	6%
3	372	385	-3%	107	119	-10%
4	441	450	-2%	198	220	-10%
5	317	326	-3%	147	162	-9%
7	372	374	-1%	153	141	8%
8	579	600	-4%	284	301	-6%
9	593	621	-5%	214	220	-3%

4.0 System Analysis

According to the Atlantic Canada Guidelines, the distribution system should be designed to maintain a minimum pressure of 275 kPa (40 psi) at grade at Peak Hour Demand (PHD). During a fire flow event, the system should maintain a minimum pressure of 140 kPa (20 psi) under Maximum Daily Demand (MDD). The recommended working pressure in the distribution system should be between 410 and 550 kPa (60 – 80 psi). Pressures higher than 620 kPa (90 psi) should be avoided if possible because it can cause excessive leakage in the system and damage to the household plumping system.

The requirement for fire protection for the Municipality's water system is not available. It was assumed that a minimum fire protection of 60 L/s is required in the Municipality's water system.

4.1 Existing Conditions

Steady-State scenarios were simulated in both the Regional and Shubenacadie water distribution systems for the present day conditions. Figure 4.1 to 4.4 presents the performance of the distribution under PHD and MDD scenarios.

Figure 4.1 shows the resulting system pressures within the Regional Water Distribution System under existing PHD condition. The results of the hydraulic modelling indicate that under existing PHD condition, pressures within the regional water distribution system are mostly above the minimum maintaining pressure of 275 kPa, except for a few locations with noticeably high elevations. These low-pressure areas are located at the north end of the system with a steep topography where the elevation difference between the high and low elevation areas is approximately 20m causing a significant pressure drops. It should be noted that the HGL within the system were set between 62 m and 65 m under normal operating conditions. In addition, residential units with a ground elevation of 35 m located west and north sides of the town barely achieve a minimum required pressure of 275 kPa.

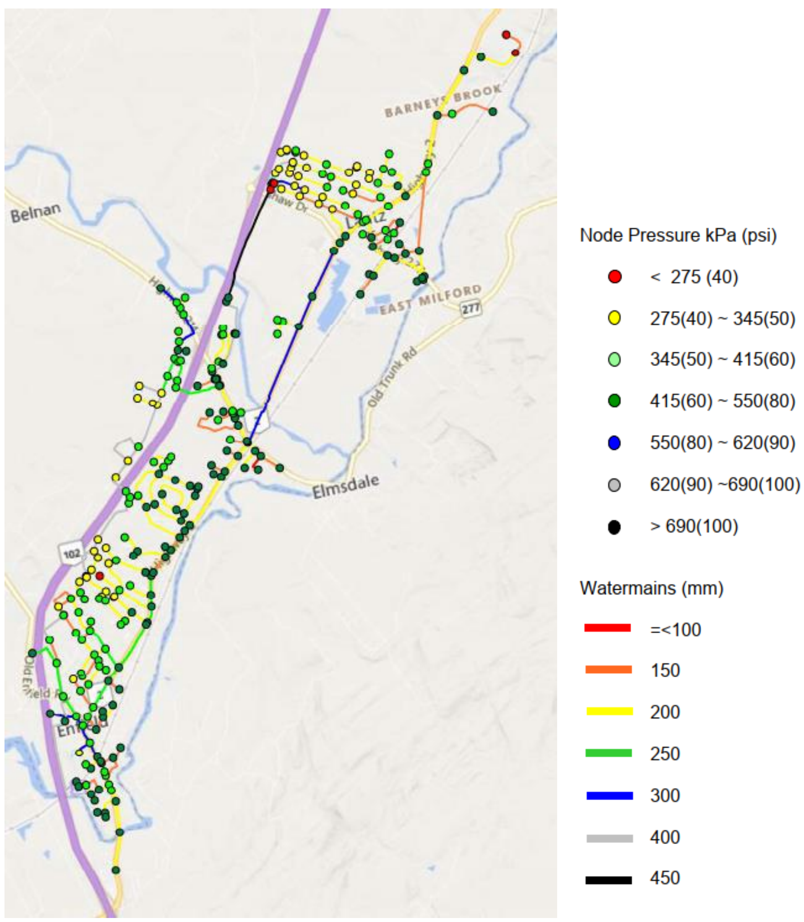


Figure 4.1 – Regional System Pressure under Existing PHD Condition

Figure 4.2 shows the resulting system pressures within the Shubenacadie Water Distribution System under existing PHD condition. The hydraulic modelling results show that under the existing PHD condition, pressures within the system are above the minimum maintaining pressure of 275 kPa; however, the resulting pressures at the north side of the system exceed the pressure upper limit of 620 kPa. Additional field investigation is recommended to confirm the high pressures experienced on this area. Once confirmed, a pressure reducing valve can be considered to reduce the pressures and maintain them to within acceptable range.

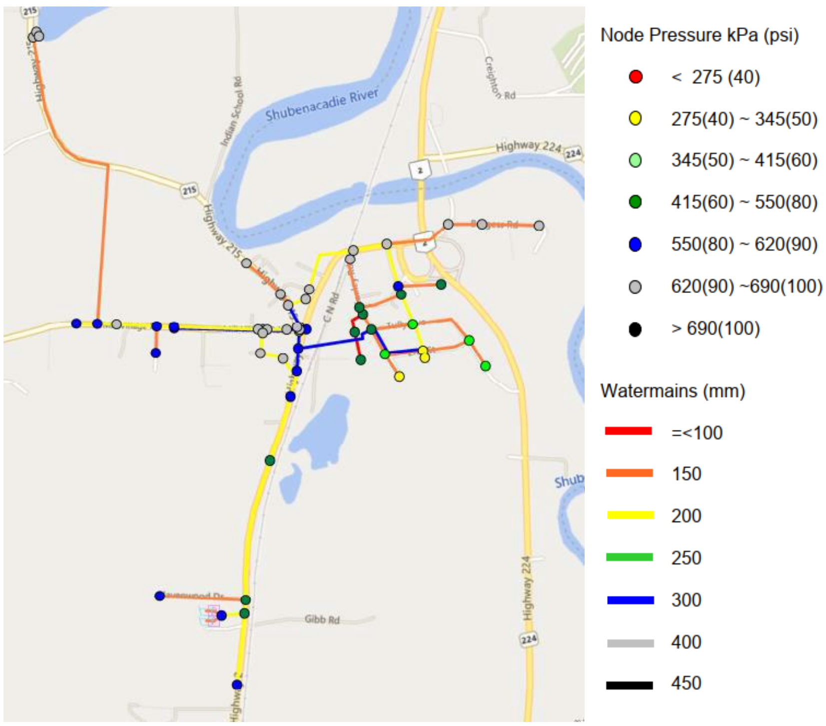


Figure 4.2 – Shubenacadie System Pressure under Existing PHD Condition

Figure 4.3 shows the resulting available fire flows within the Regional Water Distribution System under existing MDD plus fire condition. The results of the hydraulic modelling show that under existing MDD plus fire flow scenario, the available fire flows on most areas within the system can achieve the minimum fire flow requirement of 60 L/s, except on areas with dead-end watermains which is to be expected since the fire flow availability on these cases are normally lower than properly looped watermain connections. These low fire flows can be improved by looping or practically connecting the dead-end watermain to an existing watermain loop for better performance of the system and improved water quality. It can also be noticed from Figure 4.3 that the north side of Lantz are experiencing low fire flow availability due to combination of relatively high elevations and dead-end watermains.

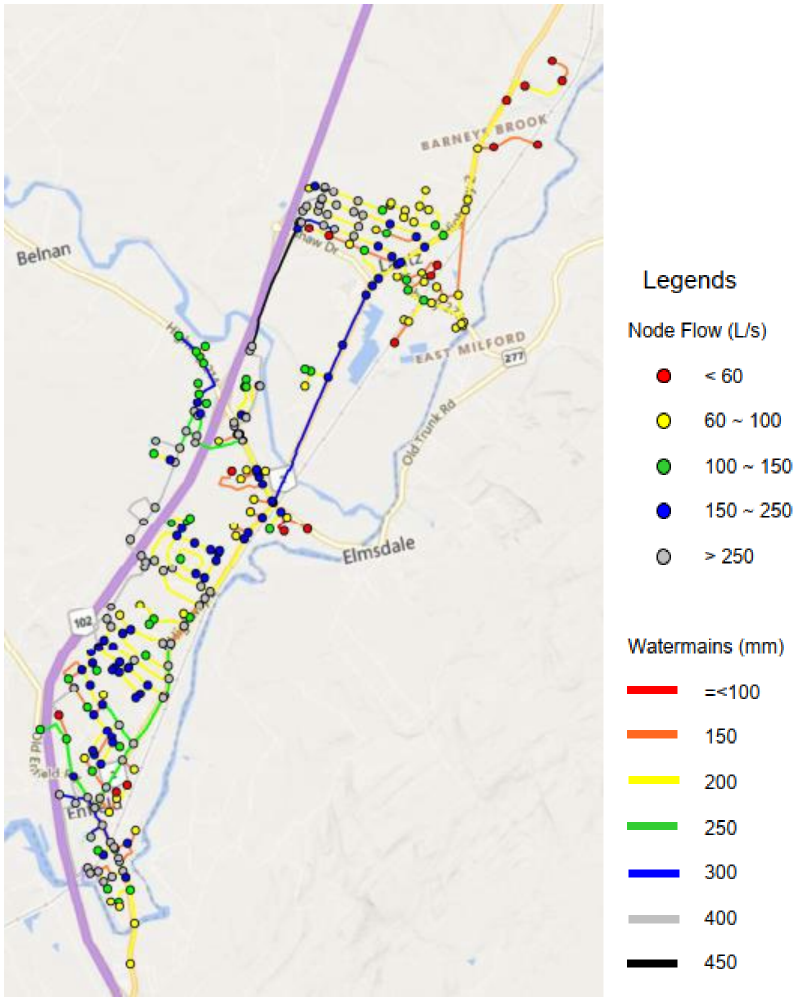


Figure 4.3 – Regional System Available Fire Flow under Existing MDD Plus Fire Condition

Figure 4.4 shows the resulting available fire flows within the Shubenacadie Water Distribution System under existing MDD plus fire condition. It can be observed from figure that the Shubenacadie system is capable of providing 60 L/s of fire flow or higher. Available fire flows lower than 60 L/s can be expected mostly on 150mm diameter connections and/or dead-end watermains.

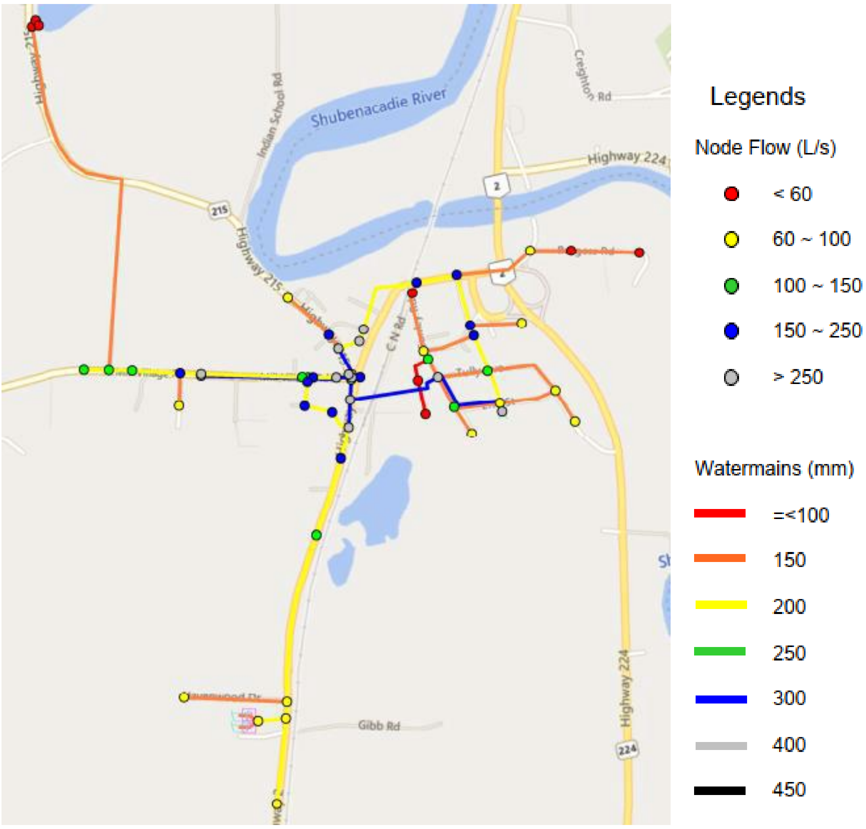


Figure 4.4 – Shubenacadie System Available Fire Flow under Existing MDD Plus Fire Condition

4.2 Future Conditions

The Municipality provided RVA with low, mid, and high-range population estimates from 2022 – 2047 for the purpose of this assessment. RVA was instructed to use the mid-range population projection for this study. Mid-range projection includes the future developments that are already approved and will be approved. RVA used the same boundary condition in the existing model to simulate the future condition for a lateral comparison.

Table 4.1 lists the mid-range future development with each total number of units, projected population and estimated buildout years in the Regional Water Distribution System.

TABLE 4.1 – MID-RANGE PROPOSED DEVELOPMENT

Location	# of Total Unit	Buildout Projected Population*	Estimated Buildout Years
428 Highway 2, Enfield	62	155	2
553 Highway 2, Elmsdale	58	145	2
Lantz	1500	3750	10
Lantz	2205	5513	20
Bakery Lane	72	180	3
159 Highway 2, Enfield	72	180	3
Tyler Street Extension, Elmsdale	88	220	2
Highway 214, Elmsdale	94	235	3
John Murray Drive, Enfield	10	25	2
Kali Lane, Elmsdale	16	40	2
Pinehill/Elmwood, Elmsdale	660	1650	10
163 Highway 214, Elmsdale	11	28	-
Melody Lane, Lantz	40	100	-
Corner of Old Enfield Rd and Old Horne Settlement Rd	10	25	-
Mariah Drive, Lantz	104	260	5
Highway 2, Elmsdale	16	40	2
532 Highway 2, Elmsdale	42	105	1
450 Highway 2, Enfield	126	315	6
432 Highway 2, Enfield	40	100	2
161 Highway 277, Lantz	16	40	2
Milford	1495	3738	20

* Density: 2.5 person per dwelling, based on average people per dwelling in the combined Corridor/Indian Brook ADA region from the 2021 census

Figure 4.5 below illustrates the locations and sizes of the development (bigger size of purple dot representing larger projected population in the area).

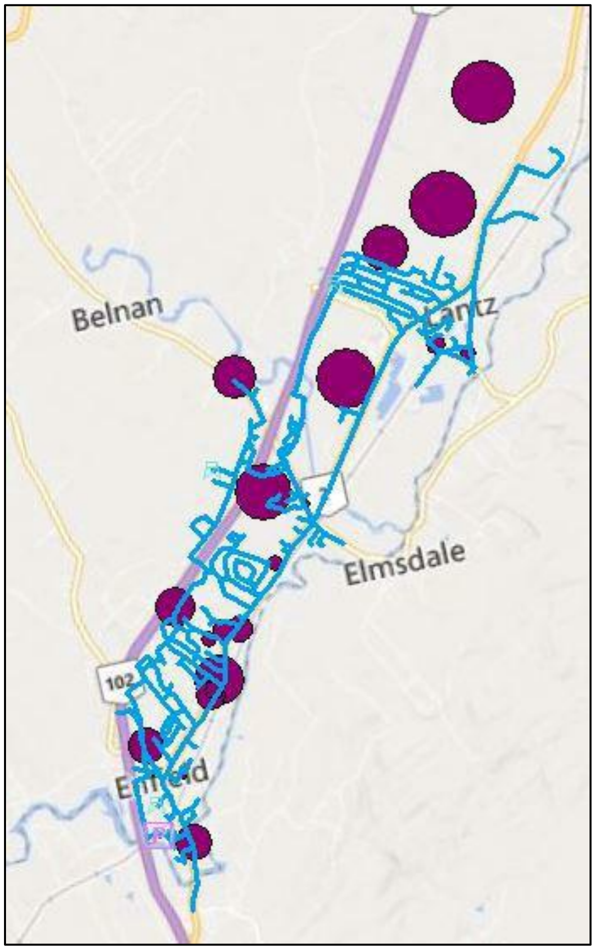


Figure 4.5 – Locations & Sizes of Mid-range Future Developments

It can be seen that the proposed developments are expected to spread out in the Regional System with large population growth on the northern side of the system which is currently experiencing service issues (i.e., low pressures and available fire flows). Estimated demands from the proposed developments were allocated to the closet existing water system junctions in the hydraulic model.

As for Shubenacadie, since the locations of the proposed developments is currently unknown, estimated demands based on the project growth were evenly distributed and allocated to each junction in the hydraulic model.

Steady state scenarios were simulated for both Regional and Shubenacadie System for the future (2047) conditions. Figures 4.6 – 4.9 present the resulting system performance of the distribution under PHD and MDD conditions.

Figure 4.6 shows the resulting pressures within the Regional System under future PHD condition. The results of the hydraulic model show that under the future PHD condition, there is a significant impact to pressures due to the tremendous population growth, especially on the northern side of the system are expected to experience pressures below 275 kPa. Water supply and system upgrades must be considered to address this issue.

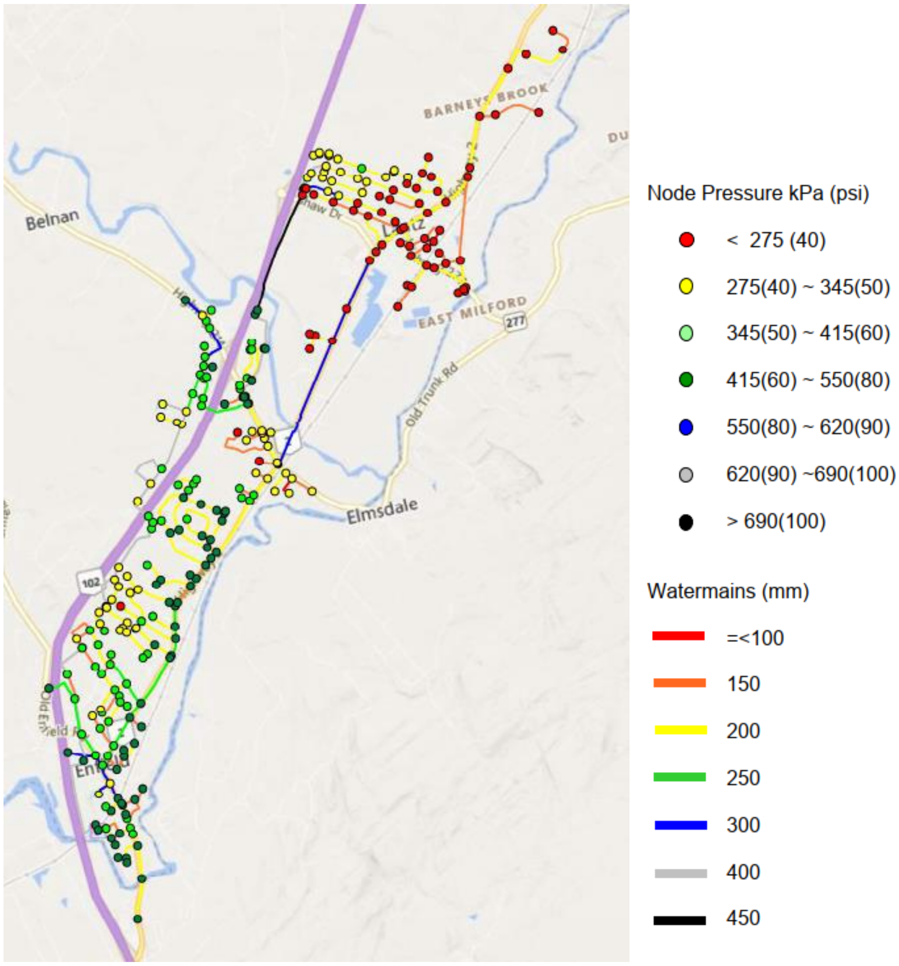


Figure 4.6 – Regional System Pressure under Future PHD Condition

Low pressure issue is caused by head loss increase throughout the system. More specifically, head loss can be calculated using the continuity equation and Darcy's equation in the following:

Darcy's equation

$$H_f = f \frac{L}{D} \frac{v^2}{2\,g}$$

Where: Hf = head loss

F = friction factor

D = diameter of pipe

V = fluid velocity

g = gravitational acceleration

Continuity equation

$$Q = A_1V_1 = A_2V_2$$

Where: Q = volumetric flow rate (demand)

A = the cross-sectional area of flow

V = flow velocity

Based on these equations, when demand increases, flow velocity in the pipe increase, which causing the head loss surge and pressure decrease. According to Municipal Standard, elevations above 35 m are subject to water pressure below 275 kPa. However, as head loss increase along with demand increase, it is expected that more area would experience water pressure below 275 kPa in the future.

Figure 4.7 shows the resulting system pressures within the Shubenacadie Water Distribution System. The results of the hydraulic modelling show that under future PHD conditions, pressures within the Shubenacadie water system are expected to be above the minimum maintaining pressure of 275 kPa; therefore, the existing water distribution system is capable of meeting the required pressures for the estimated future population growth.

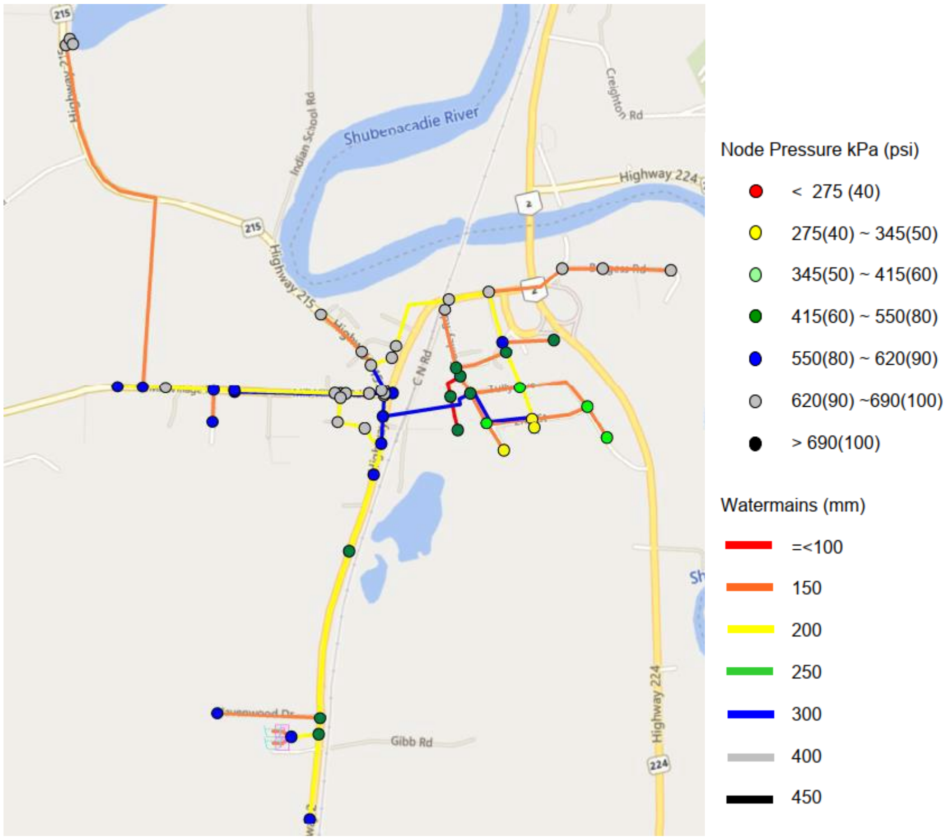


Figure 4.7 – Shubenacadie System Pressure under Future PHD Condition

Figure 4.8 shows the resulting available fire flows within the Regional Water Distribution System under future MDD plus fire condition. The results of the hydraulic modelling show that under future MDD plus fire flow scenario, the available fire flows on the southeast end within the system can achieve the minimum fire flow requirement of 60 L/s, except on areas with dead-end watermains. It can also be noticed from Figure 4.8 that fire flows on the north side of Lantz are getting worse and drop below 60 L/s due to combination of relatively high elevations, dead-end watermains and huge population growth.

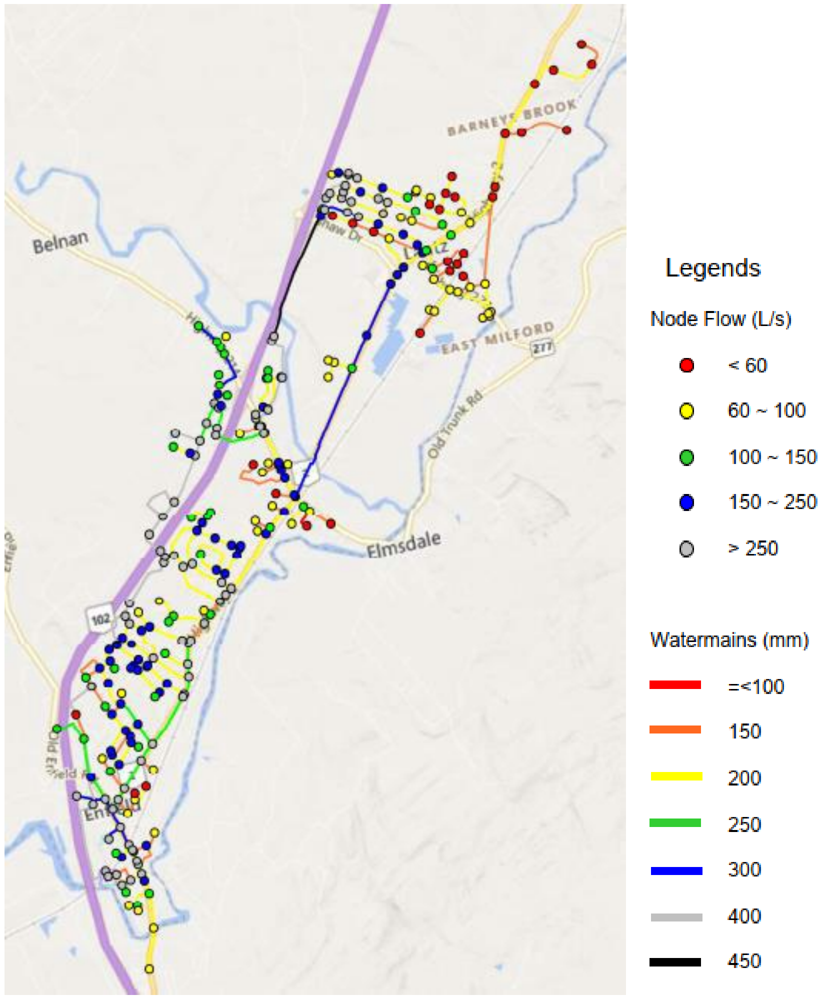


Figure 4.8 – Regional System Available Fire Flow under Future MDD Plus Fire Condition

Figure 4.9 shows the resulting available fire flows within the Shubenacadie Water Distribution System under the future MDD plus fire condition. Based on the results of the hydraulic modelling, it can be observed that available fire flow range under future MDD plus fire flow scenario is similar to that of the existing MDD plus fire flow condition.

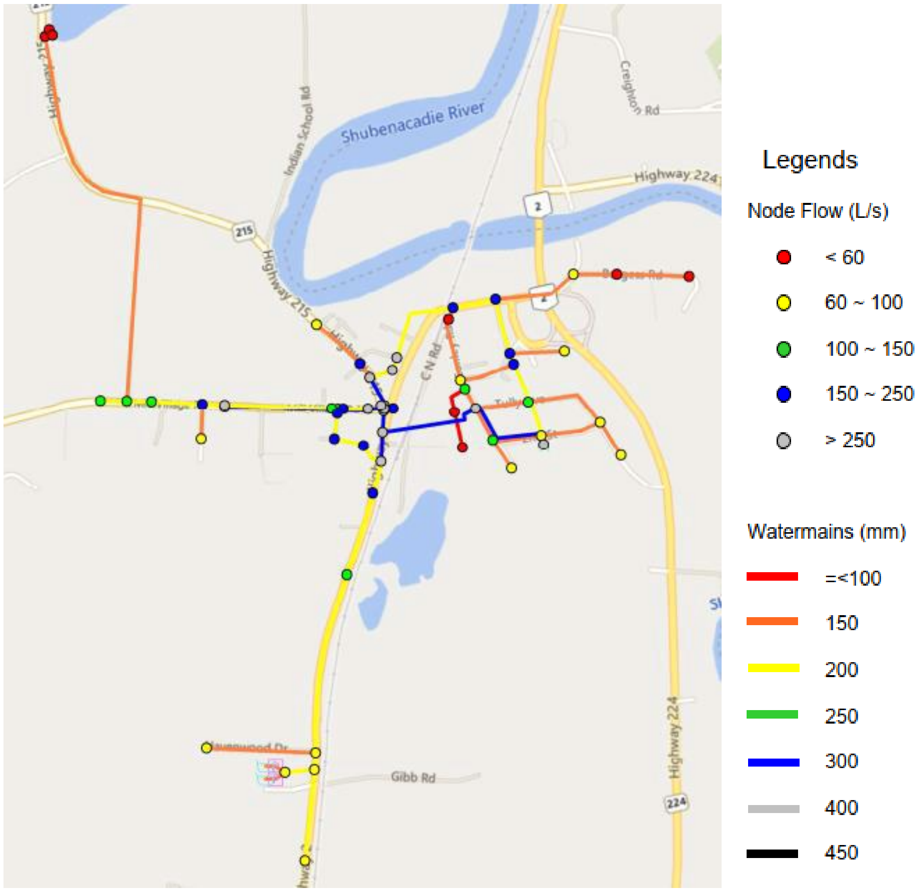


Figure 4.9 – Shubenacadie System Available Fire Flow in MDD under Future Condition

Based on the overall results, the Regional Water Distribution System is expected to experience the most impact with significant system performance reduction (low pressures and fire flow availability) due to the high projected population growth under future conditions. The results of the hydraulic modeling also shows that the Shubenacadie system is capable of accommodating the future population growth and water demands.

4.3 Treated Water Storage

Water storage facilities provide water reserves to the distribution system to meet the water demand that exceeds the water supply capacity of the WTP or in case of power outage, breaks in watermains, unexpected shutdowns of water-supply facilities, etc. It also provides efficient operational redundancy of the water system along with the provision of fire protection.

According to the Atlantic Canada Guidelines, the total water storage requirements should be calculated as follows:

Total Treated Water Storage Requirement = A+B+C

Where:

A = Fire Storage (Design Fire Flow x Design Fire Duration)

B = Equalization Storage (25% of Maximum Daily Demand)

C = Emergency Storage (25% of A + B)

Fire Storage (A) is calculated based on the theoretical fire flows over an estimated duration. The Municipality was unable to provide fire flow requirements as such fire protection requirement in this study was based on Ministry of the Environment, Conservation and Parks (MECP) Design Guidelines for Drinking Water Systems as shown in the following Table 4.2.

Equalization Storage (B) is calculated as 25% of the maximum day demand. This storage allows for the fluctuation in water demand throughout the day.

Emergency Storge (C) is calculated as 25% of the sum of the Fire Storage (A) and Equalization Storage (B). Thie storage accounts for unexpected issues that may arise within the water system, such as watermain break or equipment failure.

TABLE 4.2 – MECP FIRE FLOW GUIDELINES

Equivalent Population	Suggested Fire Flow (L/s)	Duration (hours)
500 – 1,000	38	2
1,000	64	2
1,500	79	2
2,000	95	2
3,000	110	2
4,000	125	2
5,000	144	2
6,000	159	3
10,000	189	3
13,000	220	3
17,000	250	4
27,000	318	5
33,000	348	5
40,000	378	6

Using the formula above, along with the maximum daily demand and fire flow requirement, the available and required storage volumes for each distribution system including storage capacities were calculated and shown in Table 4.3 and Table 4.5.

TABLE 4.3 – CUREENT & FUTURE STORAGE REQUIREMENTS

System	Population	Fire Flow (L/s)	Fire Duration (h)	A: Fire Storage (m³)	B: Equalization Storage (m³)	C: Emergency Storage (m³)	Total Required Storage (m³)
Current							
Regional	8,087	189	3	2,041	962	751	3,753
Shubenacadie	785	64	2	461	141	150	752
2047							
Regional	24,392	318	5	5,724	2,850	2,143	10,717
Shubenacadie	1,033	79	2	569	190	190	948

* Fire flows and durations based on MECP recommendations for small communities

TABLE 4.4 – EXISTING SYSTEM AVAILABLE STORAGE

System	System	Existing Total Storage * (m³)	Dead Storage ** (m³)	Available Storage*** (m³)

Enfield WTP****	1,166	-	1,166
Enfield Standpipe	1,431	560	871

Elmsdale Standpipe	Regional System		4,312	1,369	2,943
		Lantz Standpipe	1,940	824	1,116
		Total	8,849	2,753	6,096
	Shubenacadie System	Shubenacadie Standpipe	2,741	1,093	1,654
		Total	2,741	1,093	1,654

* Calculated based on high operating water level in each standpipe multiplied by its area of base, see section 2.4.3 and 2.5.3

** only water in the upper portion of the standpipes furnish usable system pressure. Water in lower portion is considered as dead/inactive storage. The inactive level of water in a standpipe can be determined by computing the minimum acceptable pressure (140 kPa) during fire event at the highest house in the service area of the standpipe and then adding to that figure an estimate of the head loss between the service location and the location of the standpipe. In the regional and Shubenacadie System, the highest house is located beside the standpipe, therefore, head loss is negligible.

*** Available (usable) storage is equal to total storage minus dead storage

**** Storage in WTP is based on the volume of clear well. Based on the Enfield WTP Upgrade As-built Drawings dated August 2008, the exact dimension is not available. The estimated base area of 218.7 m^2 is scale measured in Drawing S-4. The depth of 5.33 m is calculated by floor elevation minus the elevation of the bottom. The actual volume of clear well should be verified prior to the detail design of the NWSF.

TABLE 4.5 – SYSTEM STORAGE CAPACITIES

System	Condition	Total Available/Required Storage (m^3)
Regional	Current Required	3,753
	2047 Required	10,717
	Existing Capacity	6,096
	2047 Surplus/Deficit	-4,621
Shubenacadie	Current Required	752
	2047 Required	948
	Existing Capacity	1,648
	2047 Surplus/Deficit	700

5.0 Recommendations

5.1 Region Water Distribution System

The results of the hydraulic model show that the existing water distribution system is not capable of accommodating the projected population growth resulting in storage deficits, pressure and flow issues. Therefore, system upgrades to the existing water distribution were proposed to improve the hydraulic conditions, as presented below.

It was found out during the analysis that the Regional system will have a storage deficiency of 4,621 m^3 in 2047. Therefore, it is necessary to construct a new water storage facility

(NWSF) to increase the storage capacity by 5,000 m³ to satisfy the future needs. The additional size would provide more operational flexibility in the system. The NWSF is aimed at consolidating the regional water storage systems, optimizing current pumping operations, reducing long-term lifecycle costs and providing capacity of the regional system required to support projected growth to 2047.

Since the main function of the NWSF is to pressurize water for the distribution system, adequate water level can ensure the hydrostatic pressure, driven by gravity, forces the water down and provide enough pressure through the service area. The location of the NWSF therefore is recommended to be located at the highest ground elevation at the northern side of the system where the most anticipated growth is located. The Municipality is planning to locate the NWSF near Highway 102, about 1,400m north of Ryan Avenue.

5.1.1 Cost of Upgrades

The NWSF could be either an elevated tank (ET) or a standpipe. Assuming that a standpipe is selected as the NWSF and built with the bottom of structure at an elevation of 39 m, to provide storage capacity of 5,000 m³ the standpipe requires a diameter of 19 m and 26 m tall. The cost of this standpipe including standpipe itself, process, structural, architectural, site work, electrical & C&I, HVAC, etc. is estimated to be approximately eight million dollars. However, standpipes taller than 15 m are usually uneconomical, and the dead storage may require draining and cleaning to maintain water quality. Cost of a new ET with the same scope of work would range approximately from eight to nine million dollars. It is also necessary to test and ensure that the HLPs in the WTP could provide sufficient head to fill the NWSF to its maximum operating level.

The low operating level of the NWSF is determined by calculating the minimum acceptable pressure (275 kPa) at the highest residential elevation in the service area, plus the head loss between the critical service and the NWSF. In this case, head loss is negligible as the NWSF is designed to be located close to the service area. Assuming that the highest residential elevation in the northern area to be 36 m based on Google Earth, and the resulting low operating level of the NWSF (bottom of equalization storage) is estimated to be a HGL of 65 m from grade. On the other hand, the high operating level of the NWSF is determined by calculating the maximum acceptable pressure (620 kPa) at the lowest residential elevation in the service area. According to the 2020 Lidar survey provided by Nova Scotia DataLocator, the lowest elevation of the residential unit is approximately 11.8 m. Based on this value, the high operating level of water should then be equal to or less than a HGL of 75 m from grade.

However, considering the normal maximum HGL of three existing standpipes are between 63 and 65 m, the NWSF should have the same maximum HGL to balance the entire distribution system. To attain sufficient equalization and fire/emergency storage in the NWS, the low and high operating level would need to set at 59 m and 65 m from grade, respectively. That means to maintain minimum pressure of 275 kPa, houses with an elevation more than 31 m at the northern side of the system should be equipped with a private booster pump with backflow prevention which are subject to approval by Municipal Engineer.

Figure 5.1 below illustrates the areas with the elevation above 31 m on the north side of the distribution system.

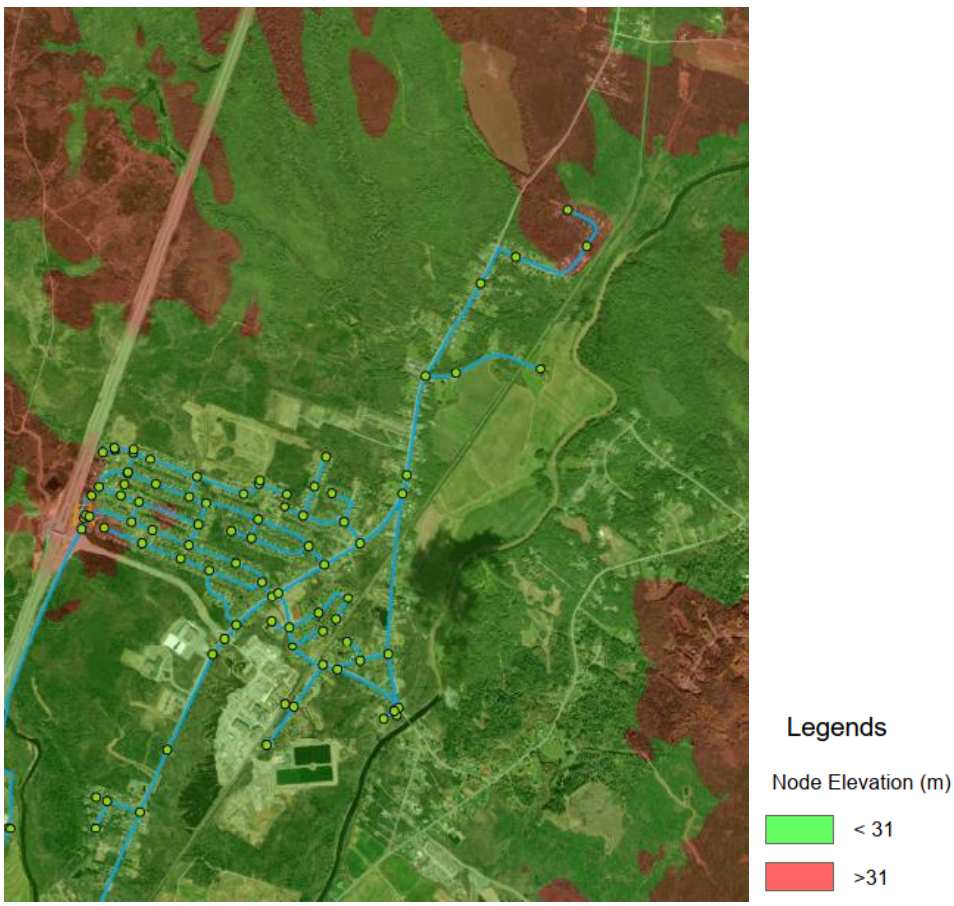


Figure 5.1 – Areas with Elevation Above 31 m in Northern Regional System

Two new 450mm transmission watermain providing bidirectional flow between the existing water system and the NWSF can reduce head loss inside the pipeline. The total approximate length would be 4 km depending on the location of the NWSF and the route of transmission watermain. The estimated cost of this transmission main would be six million dollars.

In addition, existing watermain on Highway 2 from Earls Court to Mader Street need to upsize to 400mm diameter to improve supply redundancy between the NWSF and the rest of the water system. The estimated cost of this upgrade would be one and a half million dollars.

Figure 5.2 and 5.3 show the locations of the NWSF and the new watermain together with the resulting pressures and available FF under future conditions.

With the addition of these new upgrades to the system, improvements to the resulting pressures can be observed under future PHD within the northern part of the system. However, marginal pressure drop below 275 kPa around Lantz standpipe and several high-elevation areas are still expected which can be solved by installing private booster pumps with backflow prevention.

In terms of available FF, most of the locations in the system are expected to improve with FF above 60 L/s with the exception of some dead-end which can be avoided by looping (tie-in) of watermain wherever practical.

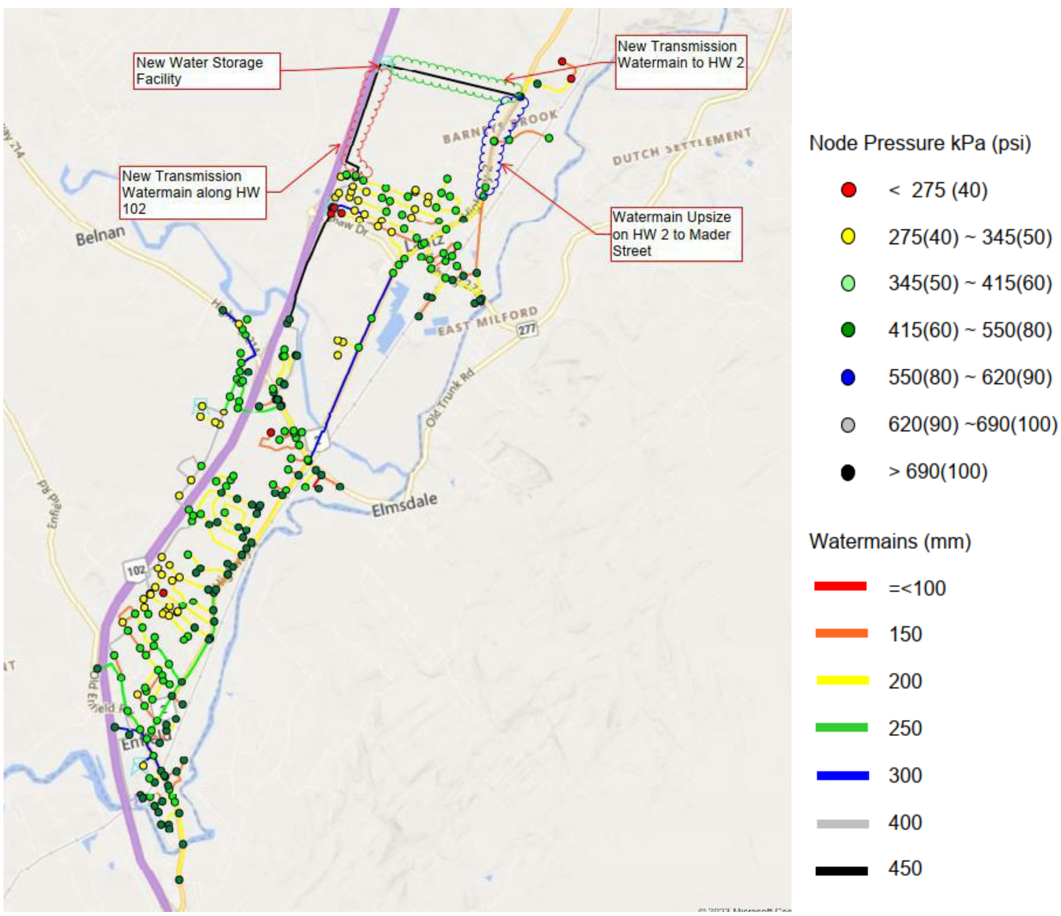


Figure 5.2 – Regional System Pressure under Future PHD Condition with NWSF & New Transmission Watermain

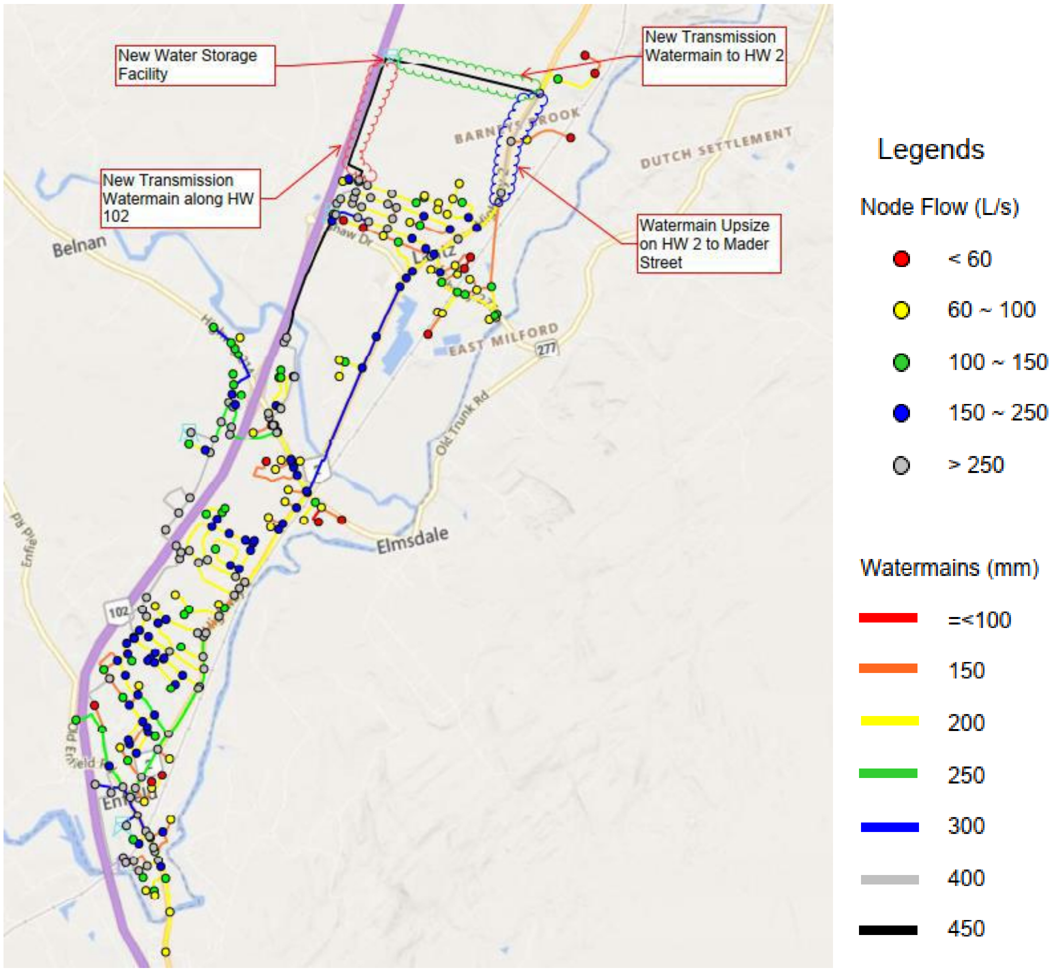


Figure 5.3 – Regional System Available Fire Flow under Future MDD + Fire Condition with NWSF & New Transmission Watermain

5.1.2 Storage Evaluation

According to Table 4.4, the system's total storage volume is 8849 m^3 out of which 2,753 m^3 is dead storage. Therefore, the system's usable volume for operational purposes is 6,096 m^3.

Table 5.1 summarizes the total water storage requirements calculated based on the method in Section 4.3 for each planning year, which includes the storage needs for fire, equalization, and emergency situations. Additionally, the table compares the total required storage with the available usable storage in the system, identifying any storage deficits under each planning year.

Based on the comparison results, the current system's storage capacity is sufficient to meet the total storage requirements until 2025. However, a storage deficit is anticipated to occur in 2026, highlighting the need for a new water storage facility by that year.

TABLE 5.1 – WATER STORAGE DEFICIT

Year	Population	Max Day Demand (m3/s)	Max Fire Flow (L/s)	Duration (hour)	Fire Storage (m3) [A]	Equalization Storage (m3) [B]	Emergency Storage (m3) [C]	Total Required Storage (m3) [D]	Available Storage (m3)	Storage Deficit (Yes/No)
2021	8087	3746	189	3	2041	937	744	3722	6096	No
2022	9040	4188	189	3	2041	1047	772	3860	6096	No
2023	10615	4917	220	3	2376	1229	901	4507	6096	No
2024	11767	5451	220	3	2376	1363	935	4673	6096	No
2025	12720	5892	220	3	2376	1473	962	4811	6096	No
2026	13860	6421	250	4	3600	1605	1301	6506	6096	Yes
2027	14997	6947	250	4	3600	1737	1334	6671	6096	Yes
2028	16085	7451	250	4	3600	1863	1366	6829	6096	Yes
2029	17120	7931	318	5	5724	1983	1927	9633	6096	Yes

5.2 Shubenacadie Distribution System

Shubenacadie has little growth over the years due to a lack of service capacity. The Municipality expects the current construction of a new sewer treatment plant would trigger certain population increase in Shubenacadie. Based on the analysis in Section 4.2, the existing water system can accommodate the estimated future growth. However, the issue of high pressures on the north and west side of the system should be addressed with consideration of Pressure Reducing Valve (PRV) in the area.

Since the ground elevation of the residential units in Shubenacadie vary from 8 m to 46 m according to the 2020 Lidar provided by Nova Scotia DataLocator, the elevation difference of 38 m make the Municipality struggle to operate the distribution system as a single pressure zone as the pressure at the lowest elevation of the residential unit would exceed the maximum acceptable pressure of 620 kPa when the highest elevation of the residential unit maintains the minimum acceptable pressure of 275 kPa. According to Municipal Standard, elevations below 17 m are subject to water pressure over 620 kPa.

Figure 5.4 below shows the locations with elevation less than 17 m.

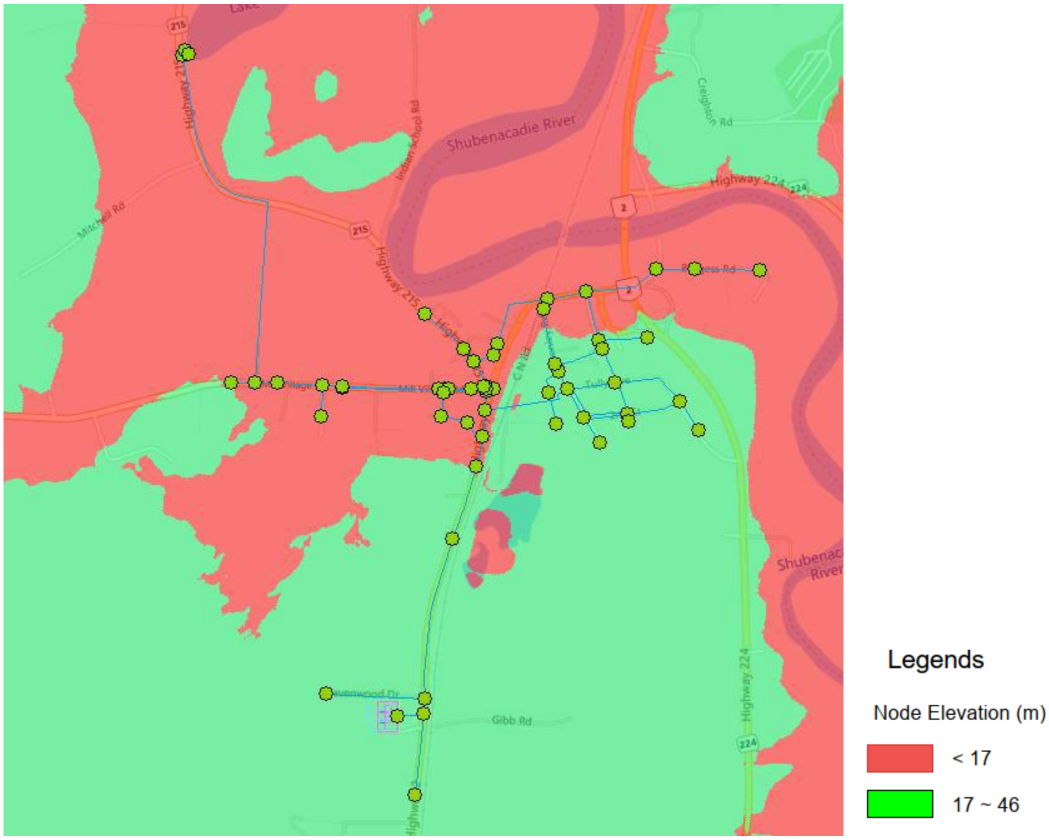


Figure 5.4 – Gound Elevation in Shubenacadie

5.2.1 Cost of Upgrades

Dividing the distribution system to two different pressure zones is a feasible solution to maintain the system within the acceptable range and would require the following upgrades:

- Installation of a 300mm diameter dedicated transmission watermain from the WTP to the standpipe along the existing piping route with an estimated length of 1.7 km. The estimated cost is two million dollars.
- Installation of two PRV on the connection of high zone and low zone (set pressure zone boundary at the elevation of 17 m) to restrict the pressure to below 620 kPa at the northern and western parts of the system. Two connections instead of one connection would provide redundancy of the system in case of maintenance or watermain break. The cost of one PRV chamber is approximately one hundred thousand dollars. Total estimated cost would be two hundred thousand dollars.

Another consideration is that the payback period of these upgrades can be long for a projected population of less than 1,200. The alternative approach could be lower the operating level in the standpipe and discharge pressure at the WTP to ensure the lowest elevation of the residential units maintain a pressure less than 620 kPa. At the same time, provide a dedicated (private) booster pump with backflow prevention for each residential units with a pressure less than 275 kPa. The estimated number of houses requiring a dedicated booster pump is 14 which are all located near the standpipe. However, it would have to be arranged between the Municipality and the homeowner regarding the installation, payment, and maintenance of the private booster pump.

Figures 5.5 and 5.6 below show the system performance of the Shubenacadie system after the proposed upgrades with two pressure zones.

The result of the hydraulic model shows that the pressures in the low zone are below 550 kPa while the pressures in high zone maintain above the minimum acceptable pressure of 275 kPa.

In terms of FF availability, most locations in the system can achieve FF above 60 L/s, except some dead-end which can be avoided by looping (tie-in) of watermain wherever practical. The Shubenacadie water distribution system has adequate storage for the future condition, and thus no additional storage is recommended.

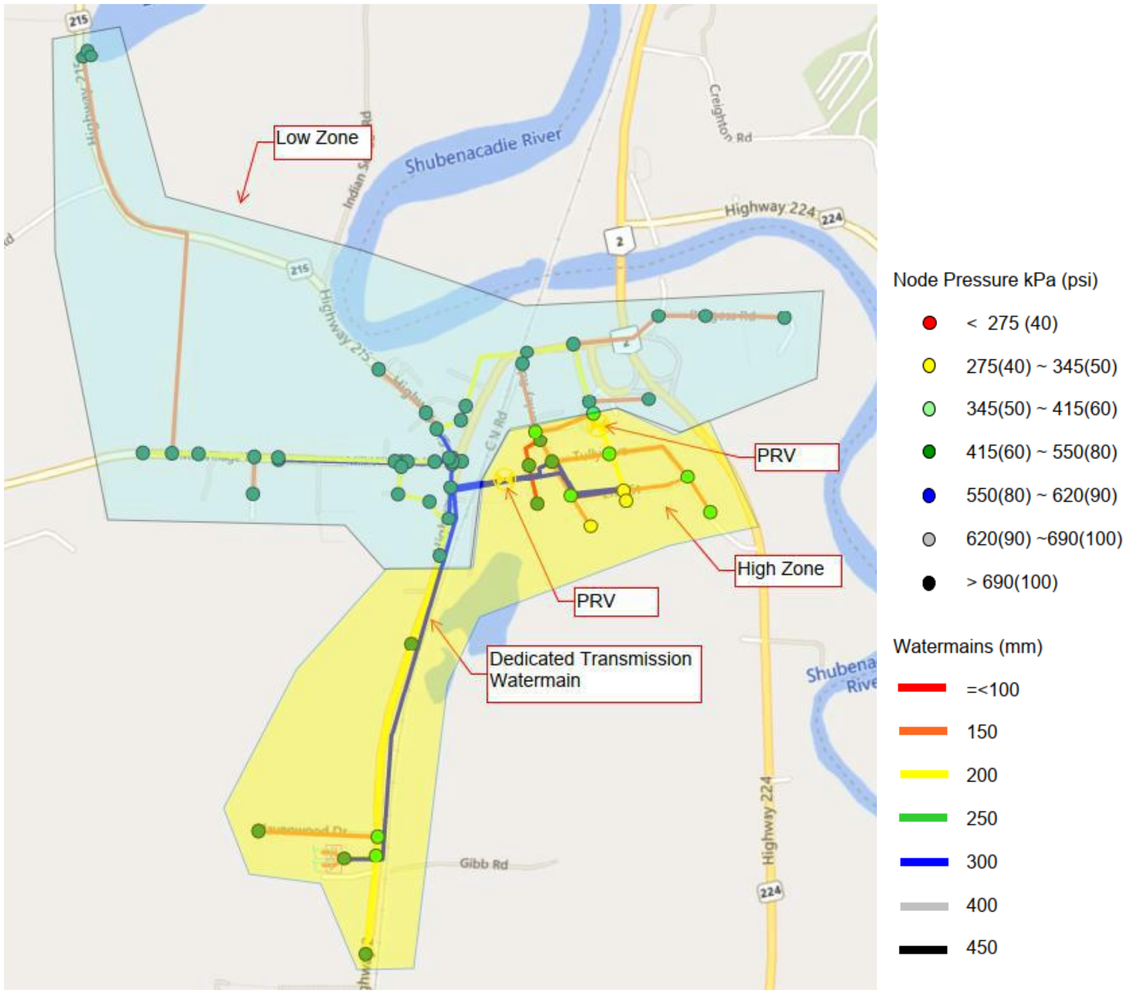


Figure 5.5 – Shubenacadie System Pressure in PHD under Future Condition with Two Pressure Zones

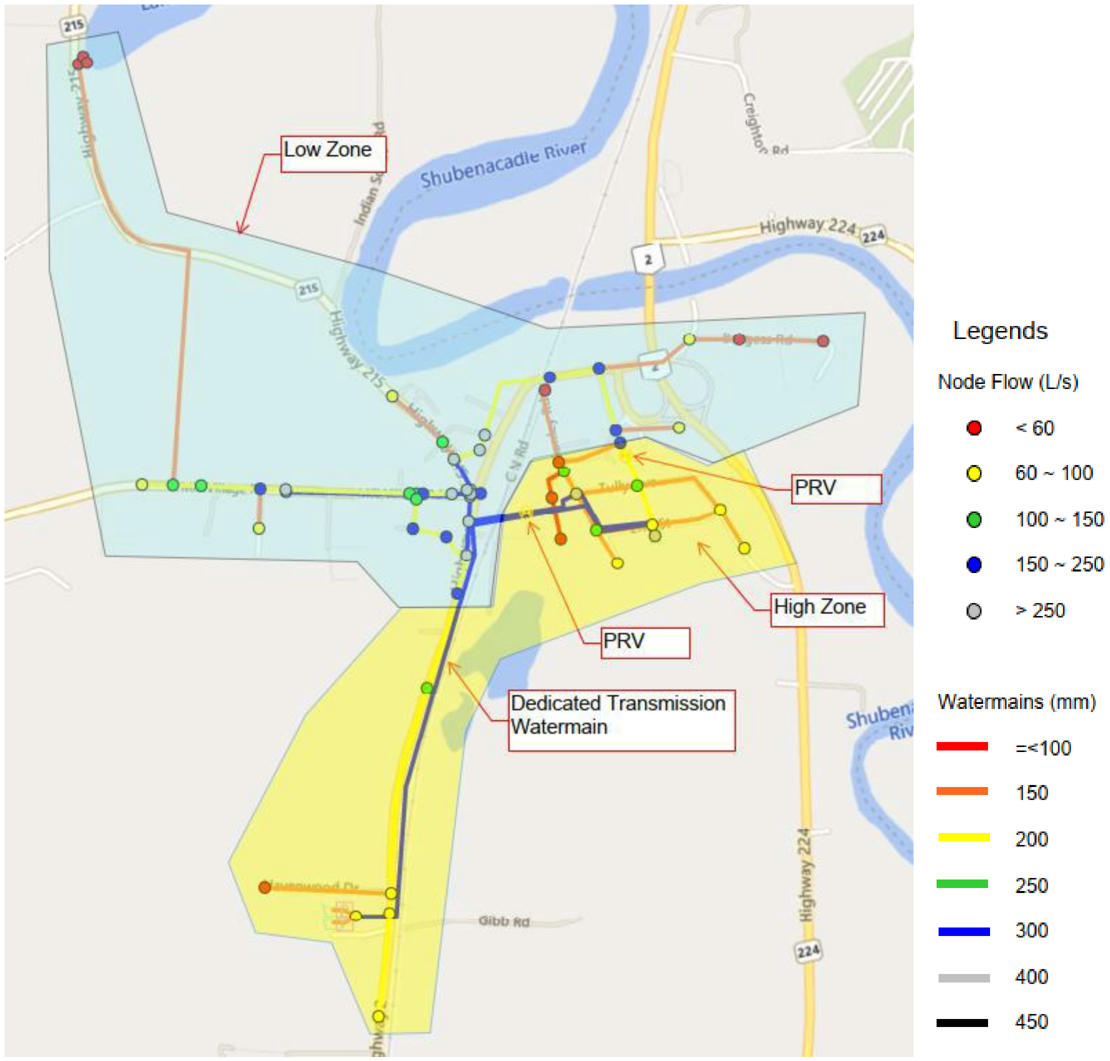


Figure 5.6 – Shubenacadie System Available Fire Flow in MDD under Future Condition with Two Pressure Zones

6.0 Conclusion

RVA developed and calibrated the Regional and Shubenacadie hydraulic water model for the Municipality of East Hants. All future opportunities to make changes to the distribution system should be modeled to take advantage of possible improvements. Re-calibration should be undertaken following major system changes, or when minor changes become numerous.

Based on the result of the calibrated hydraulic model, the existing Regional System can not accommodate the projected population growth. The recommended upgrades to the existing system to meet the future needs and their estimated high level cost are listed in Table 6.1 below:

TABLE 6.1 – RECOMMENDED UPGRADES IN REGIONAL SYSTEM

Upgrades	Items	Estimated Cost* 2023 $
New Water Storage Facility	Option 1: Standpipe	8 m
	Option 2: Elevated Tank	8 ~ 9 m
New Transmission Watermain	450mm diameter, 4 km in length	6 m
Watermain Upsize	400mm diameter, 1 km in length	1.5 m
Total Cost		15.5 ~ 16.5 m

* k presents hundred thousand dollars; m presents million dollars.

* Costs are based on house knowledge for similar sized tanks, not considering site constraints and local market changes.

In addition, based on the results of calibrated hydraulic model, the existing Shubenacadie System can accommodate the projected population growth. However, it is recommended to address the high-pressure issue on the northern and western side of the system. The approaches and their estimated cost are listed in Table 6.2 below:

TABLE 6.2 – RECOMMENDED UPGRADES IN REGIONAL SYSTEM

Upgrades	Items	Estimated Cost* 2023 $
Separate the system into two pressure zones	300mm Diameter Dedicated Watermain from WTP to Standpipe, estimated length of 1.7 km.	2 m
	Two PRV chamber	0.2 m
Total Cost		2.2 m

* k presents hundred thousand dollars; m presents million dollars.

7.0 References

S. Devereaux, “WTP Upgrade Technical Memo”, Dillon Consulting, August 25, 2021

L. A. Nunn, “Municipality of East Hants Shubenacadie Water System Annual Report 2021”, East Hants. NS, March 29, 2022

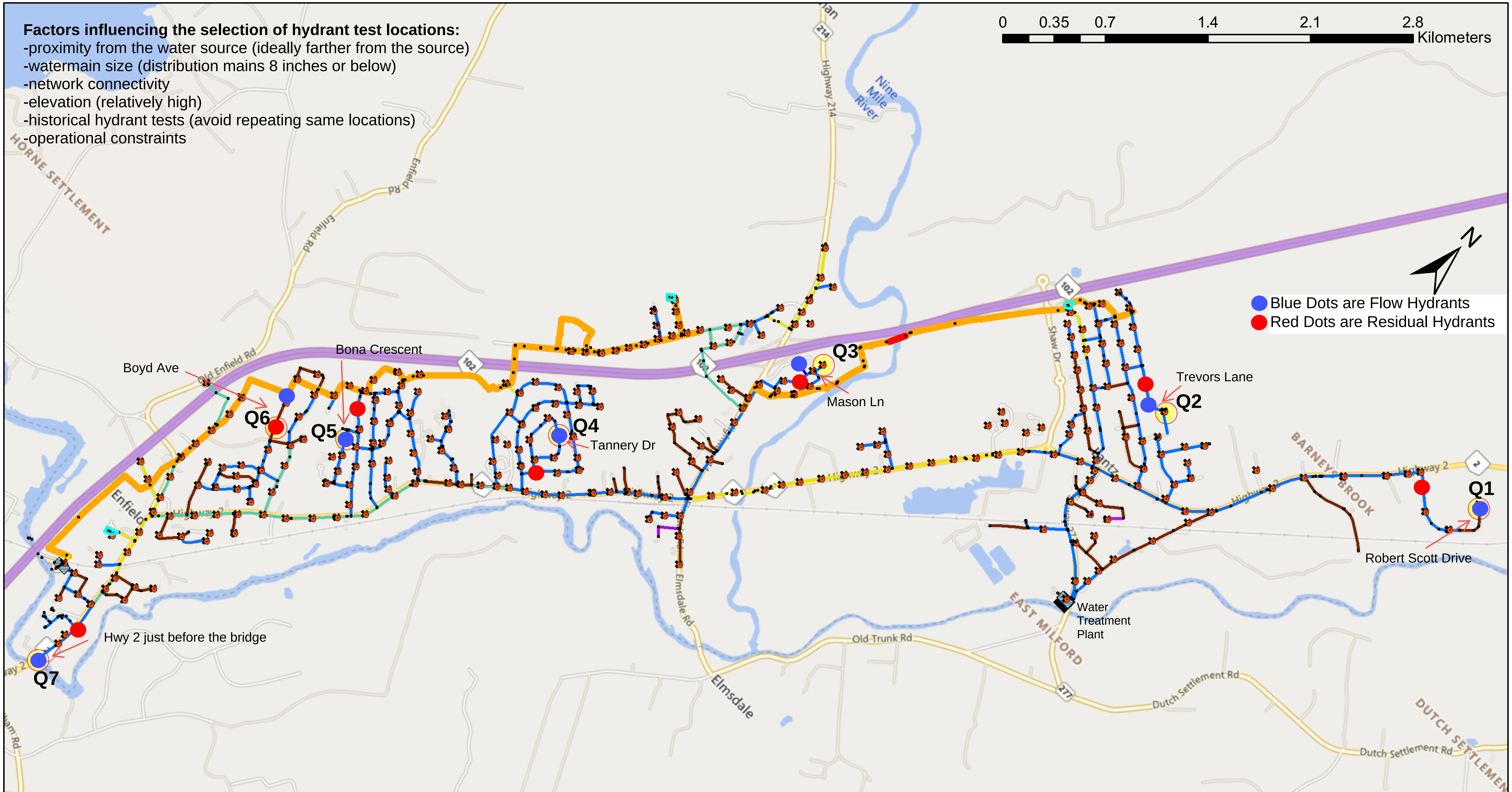
Atlantic Canada Guidelines for the Supply, Treatment, Storage, Distribution, and Operation of Drinking Water Supply Systems, September 2004

Guidelines for the Design of Water Storage Facilities, Ministry of the Environment Conservation and Parks, 2019

Municipal Standards, East Hants, July 2022

APPENDIX A

FIRE HYDRANT TEST LOCATIONS



EAST HANTS

East Hants Water Modelling

FIGURE A-1
Proposed Hydrant Test Locations in South Corridor and Commercial (SCC)

LEGEND

Diameter, mm (in)
≤ 100 (4)
150 (6)
200 (8)
250 (10)
300 (12)
400 (16)
450 (18)

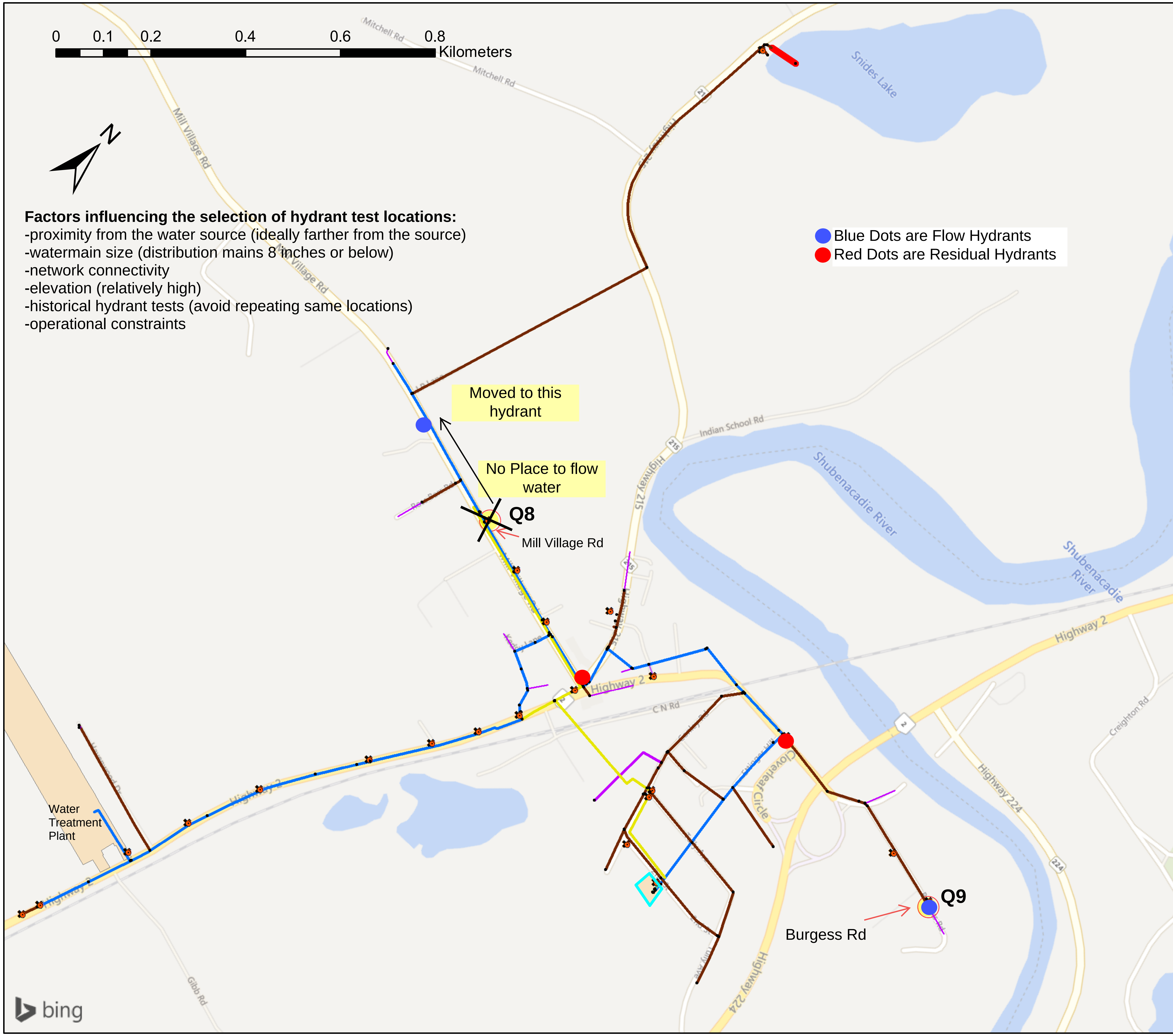
Nodes

- hydrant
- proposed hydrant test location
- standpipe

arva

RVA Project Number

226421



East Hants Water Modelling

FIGURE A-2
Proposed Hydrant Test Locations in Shubenacadie

LEGEND

Diameter, mm (in)

- ≤ 100 (4)
- 150 (6)
- 200 (8)
- 250 (10)
- 300 (12)
- 400 (16)
- 450 (18)

Nodes

- hydrant
- proposed hydrant test location
- standpipe

RVA Project Number

226421