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APPENDIX 2-A - WATER SYSTEM GAP ANALYSIS AND STATUS QUO REPORTS

2 VOLUME 2: EXISTING WATER SYSTEMS

The CGS owns and operates six (6) municipal drinking water supply systems that service the various communities in the City, as listed below. Each respective Drinking Water Works Permit (DWWP) identification is included in the associated brackets.

1 Dowling Drinking Water System (DWWP 016-203)
2 Falconbridge Drinking Water System (DWWP 016-201)
3 Onaping/Levack Drinking Water System (DWWP 016-202)
4 Sudbury Drinking Water System (DWWP 016-206)
5 Valley Drinking Water System (DWWP 016-205)
6 Vermilion Drinking Water System (DWWP 016-204)

The existing CGS water systems and their components, including supply and distribution infrastructure, have been documented in the Water Baseline Review Report (WSP, 2015) that is found in Appendix 1-B. The report compiles and documents available information on the City's existing water infrastructure and establishes the baseline, or starting point, in the Master Plan's assessment of the water systems. The report also includes an overview of the regulatory requirements relevant to the planning and design of water systems in Ontario.

Additionally, a capacity review of each water system was conducted, through gap analysis, in order to determine future system requirements. The following sections of this report will summarize the information in the Water Baseline Review Report (WSP, 2015) as well as the Water System Gap Analysis and Status Quo Reports (WSP, 2015-2016) for each of the individual systems. The Water System Gap Analysis and Status Quo Reports can be found in Appendix 2-A. The following sections document the existing infrastructure within each of the six (6) water systems, and the infrastructure gaps within those systems. Volume 4 documents the water system alternative solutions developed to address the gaps identified in this Report.

2.1 DOWLING WATER SYSTEM

The Dowling Water System is located at the northwest end of the CGS along Route 144, between the communities of Onaping and Chelmsford. Figure 2-1 illustrates the existing water infrastructure in the Dowling Water System.

Figure 2-1 Dowling Water System: Existing Infrastructure

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2.1.1 EXISTING SYSTEM

The Dowling Water System is supplied by two (2) wells; the Riverside Well and the Lionel Well, both owned and operated by the CGS. The wells draw from an unconfined aquifer of sand and gravel deposit, located within the Onaping River watershed. Due to the unconfined nature of the soils and proximity to the river, the water source is classified as potential groundwater under the direct influence of surface water (GUDI).

The same treatment process exists at both wells, which consists of a UV primary disinfection system, a gas chlorination secondary disinfection system, and a fluoride injection system. The total rated capacity of the wells per the Permit to Take Water (PTTW) is 3,640 m^3/d. Table 2-1 summarizes the wells' process information.

Table 2-1 Dowling Wells' Process Information

WELL	SYSTEM RATED CAPACITY (M^3/D)[2]	PUMP TYPE[1]	OPERATING POINT[1]	STANDBY POWER[1]
Riverside Well	3,640	Vertical turbine well pump	42.1 L/s (3,640 m^3/d) at 71.6 m TDH	100 kW diesel generator set (located at Lionel, but services both wells)
Lionel Well		Vertical turbine well pump	42.1 L/s (3,640 m^3/d) at 68.6 m TDH	

[1] *Data obtained from the Dowling Drinking Water Works Permit, Number 016-203 Issue 1.*

[2] *Best practices assume largest well out of service to determine the rated capacity.*

The Dowling Water System consists of one (1) pressure zone, and storage is provided by one (1) elevated storage tank; the Dowling Elevated Tank. The tank's usable volume is 907 m^3, as calculated based on operating water levels.

2.1.2 EXISTING AND FUTURE REQUIREMENTS

Using the population projections and water demand rate development process described in Section 1.4 of Volume 1 of this report, Dowling's future water demand projections were calculated. Table 2-2 summarizes the Dowling demand criteria and the reference used to determine the criteria, and Table 2-3 summarizes the calculated demand projections.

Table 2-2 Dowling Water System Demand Criteria

CRITERIA	VALUE	REFERENCE
Average Day Residential Demand	200 L/cap/d	Rounded up average of historical values
Average Day Institutional & Commercial Demand	28 m^3/ha/d	MOECC Guidelines
Average Industrial Demand	35 m^3/ha/d	MOECC Guidelines
Domestic Demand Maximum Day Factor	2.71	Average of historical values
Domestic Demand Peak Hour Factor	3.75	MOECC Guidelines

Table 2-3 Dowling Water Demand Projections

YEAR	POPULATION	AVERAGE DAY DEMAND (M3/D)	MAXIMUM DAY DEMAND (M3/D)	PEAK HOUR DEMAND (M3/D)
Base	1,773	388	1,048	1,455
2016	1,837	401	1,085	1,503
2021	1,903	414	1,121	1,553

YEAR	POPULATION	AVERAGE DAY DEMAND (M3/D)	MAXIMUM DAY DEMAND (M3/D)	PEAK HOUR DEMAND (M3/D)
2026	1,965	458	1,239	1,716
2031	1,997	464	1,257	1,740
2036	2,017	468	1,267	1,755
2041	2,016	468	1,267	1,754

2.1.3 GAP ANALYSIS CONCLUSIONS

As previously mentioned, a gap analysis was conducted in order to determine existing and future water system deficiencies for each system. The following information is a summary of the Dowling Water System Gap Analysis and Status Quo Report (WSP, 2016), contained in Appendix 2-A. The report can be referenced for more details regarding the analysis of the Dowling Water System.

SUPPLY

Analysis of the Dowling Wells concluded that there would be sufficient capacity to service the population growth to the year 2041, though it is important to assess the true capacity of the wells to determine whether they can reliably produce flows equal to the rated capacity. A summary of the wells' capacity analysis can be seen in Figure 2-2 where the projected maximum day demands are plotted against the capacity of the wells.

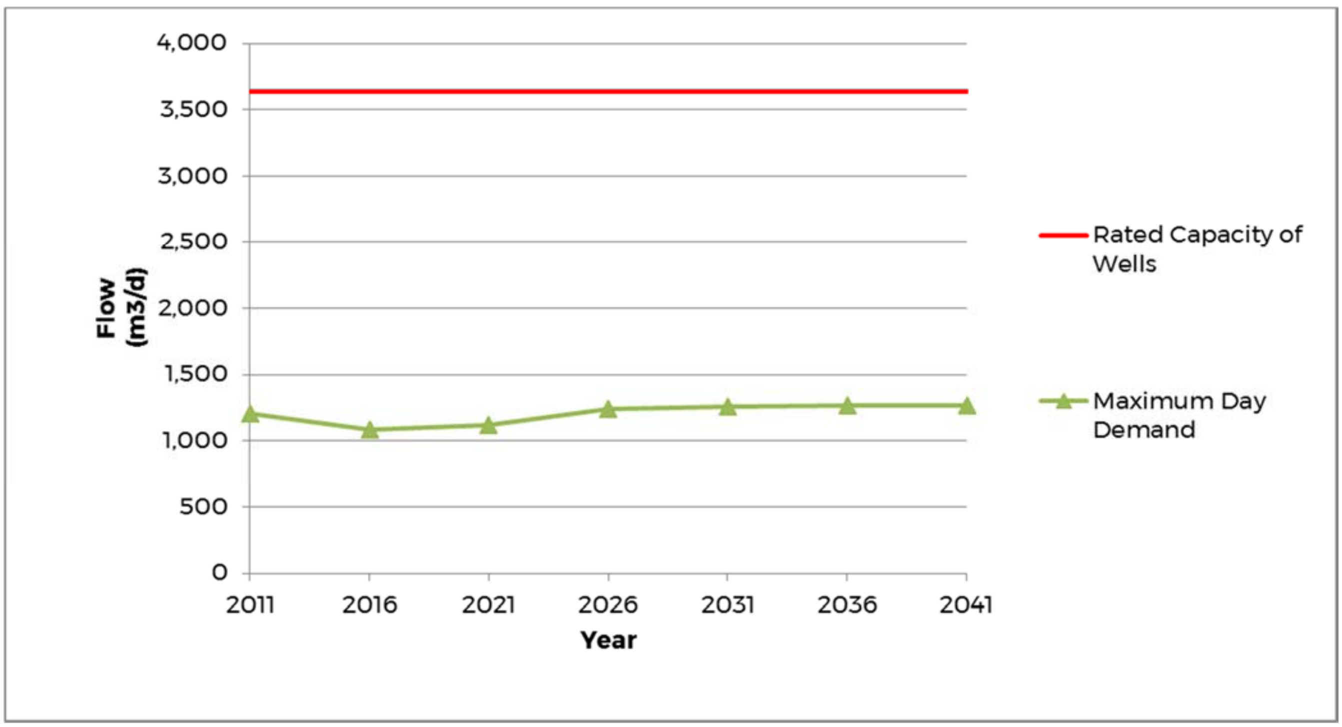

Figure 2-2 Dowling Water Demand Projections vs. Rated Capacity of the Wells

STORAGE

Analysis of the Dowling Elevated Tank concluded that no additional storage would be required for the Dowling Water System to service the population growth to the year 2041. The analysis undertaken abided to the process described in

Section 1.4.4.3 of Volume 1 of this report, for systems where supply exceeds the maximum day demand, and in this case, also exceeds peak hour demands. As mentioned, the Dowling Wells can supply 3,640 m^3/d which is greater than the maximum day demand in 2041, which was calculated to be 1,267 m^3/d, and also greater than the projected peak hour demand, which was calculated to be 1,754 m^3/d. Therefore, the fire flow requirements and peak hour demands can be met from a combination of the available storage volume and direct pumping from the wells, and no additional storage is required.

WATERMAINS

During assessment of the watermains within the Dowling Water System, hydraulic computer modeling identified that, like many water systems in the CGS, certain areas may not be able to deliver fire flows per current standards as outlined in Section 1.4.2 of Volume 1. Refer to the Dowling Water System Gap Analysis and Status Quo Report (WSP, 2016), contained in Appendix 2-A of this report, which outlines the areas identified to have pipe capacity deficiencies within the Dowling Water System.

2.2 FALCONBRIDGE WATER SYSTEM

Falconbridge is a small community located in the east end of the City of Greater Sudbury. A notable feature within the community is the Glencore Smelter Complex, located near Edison Road and Longyear Drive. Figure 2-3 illustrates the existing water infrastructure in the Falconbridge Water System.

Figure 2-3 Falconbridge Water System: Existing Infrastructure

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2.2.1 EXISTING WATER SYSTEM

The Falconbridge Water System is supplied by by three (3) non-GUDI wells; Wells No. 5, No. 6, and No. 7, all owned and operated by the CGS. Water from the wells is treated at the well house for Well No. 7. Chorine gas is used for disinfection, and a corrosion inhibitor is added to the treated water. The water is discharged to the Hardy Fluoridation Facility for fluoride addition, and non-fluoridated water is sent to the Nickel Rim and Airport reservoirs. Potable, fluoridated water enters the Falconbridge distribution system from the Hardy Fluoridation Facility. It should be noted that the maximum day capacity of the fluoridation facility is 727 m^3/d, and typically operates at 173 m^3/d.

The total rated capacity of the wells as prescribed by the PTTW is 4,251 m^3/d; however, the firm production capacity is 2,833 m^3/d. Table 2-4 summarizes the wells' process information.

Table 2-4 Falconbridge Wells' Process Information

WELL	RATED CAPACITY (M^3/D)[1,3]	PUMP TYPE[1]	OPERATING POINT[2]	STANDBY POWER[1]
Well No. 5	2,833	Submersible well pump	16.4 L/s at 130 m TDH	200 kW diesel generator
Well No. 6		Submersible well pump	16.4 L/s at 130 m TDH	
Well No. 7		Submersible well pump	16.4 L/s at 130 m TDH	

[1] *Data obtained from the Falconbridge Drinking Water Works Permit, Number 016-201 Issue 1.*
[2] *Obtained from the Falconbridge Wells Permit to Take Water.*
[3] *Best practices assume largest well out of service to determine the rated capacity.*

The Falconbridge Water System consists of a single pressure zone, and storage is provided by one (1) elevated storage tank; the Falconbridge Storage Tank. The tank provides a total of 1,136 m^3 of floating storage to the system.

The Falconbridge Water System also contains the Mott BPS, which boosts pressures in the area along the north-south portion of Edison Road, at the west end of Falconbridge. Table 2-5 summarizes the Mott BPS information and capacity.

Table 2-5 Falconbridge Water System Booster Pumping Station Summary

FACILITY	PUMP INFORMATION[1]	TOTAL CAPACITY (L/S)	FIRM CAPACITY (L/S)[2]
Mott BPS	Two, each rated at 2.5 L/s at 22.0 m TDH	5.0	2.5

[1] *Obtained from the Falconbridge water model.*
[2] *The Firm Capacity is calculated assuming the largest pump out of service.*

2.2.2 EXISTING AND FUTURE REQUIREMENTS

Using the population projections and water demand rate development process described in Section 1.4 of Volume 1 of this report, Falconbridge's future water demand projections were calculated. Table 2-6 summarizes the Falconbridge demand criteria and the reference used to determine the criteria, and Table 2-7 summarizes the calculated demand projections.

Table 2-6 Falconbridge Water System Demand Criteria

CRITERIA	VALUE	REFERENCE
Average Day Residential Demand	300 L/cap/d	Average of historical values, rounded up to nearest 50 L/cap/d

CRITERIA	VALUE	REFERENCE
Average Day Institutional & Commercial Demand	28 m^3/ha/d	MOECC Guidelines
Average Industrial Demand	35 m^3/ha/d	MOECC Guidelines
Domestic Demand Maximum Day Factor	2.12	Average of historical values
Domestic Demand Peak Hour Factor	3.47	Maximum of historical values

Table 2-7 Falconbridge Water Demand Projections

YEAR	POPULATION	AVERAGE DAY DEMAND (M^3/D)	MAXIMUM DAY DEMAND (M^3/D)	PEAK HOUR DEMAND (M^3/D)
Base	707	1,111	2,350	2,939
2016	724	1,116	2,365	3,869
2021	743	1,121	2,377	3,888
2026	759	1,191	2,526	4,132
2031	769	1,194	2,532	4,142
2036	775	1,205	2,556	4,180
2041	776	1,206	2,556	4,181

2.2.3 GAP ANALYSIS CONCLUSIONS

As previously mentioned, a gap analysis was conducted in order to determine existing and future water system deficiencies for each system. The following information is a summary of the Falconbridge Water System Gap Analysis and Status Quo Report (WSP, 2015), contained in Appendix 2-A. The report can be referenced for more details regarding the analysis of the Falconbridge Water System.

SUPPLY

Analysis of the Falconbridge Wells concluded that there would be sufficient capacity to service the population growth to the year 2041. This can be seen in Figure 2-4 where the projected maximum day demands are plotted against the capacity of the wells.

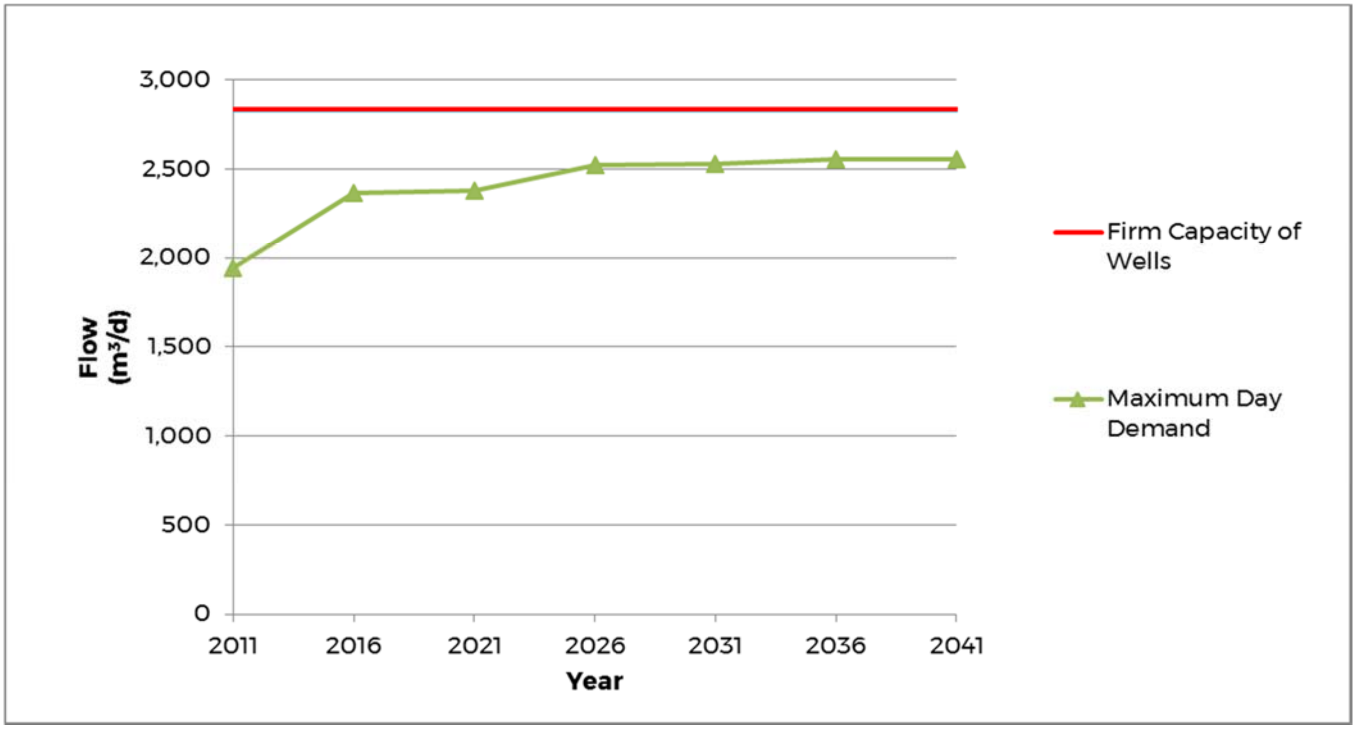

Figure 2-4 Falconbridge Water Demand Projections vs. Firm Capacity of the Wells

STORAGE

During the review of existing infrastructure, it was noted that the Falconbridge Storage Tank is aging and is in need of repairs, according to City staff. Additionally, analysis of the Falconbridge Storage Tank concluded that an additional 605 m^3 of storage would be required for the Falconbridge Water System to service the population growth to the year 2041. This can be seen in Figure 2-5 where the required water storage is plotted against the existing water storage capacity.

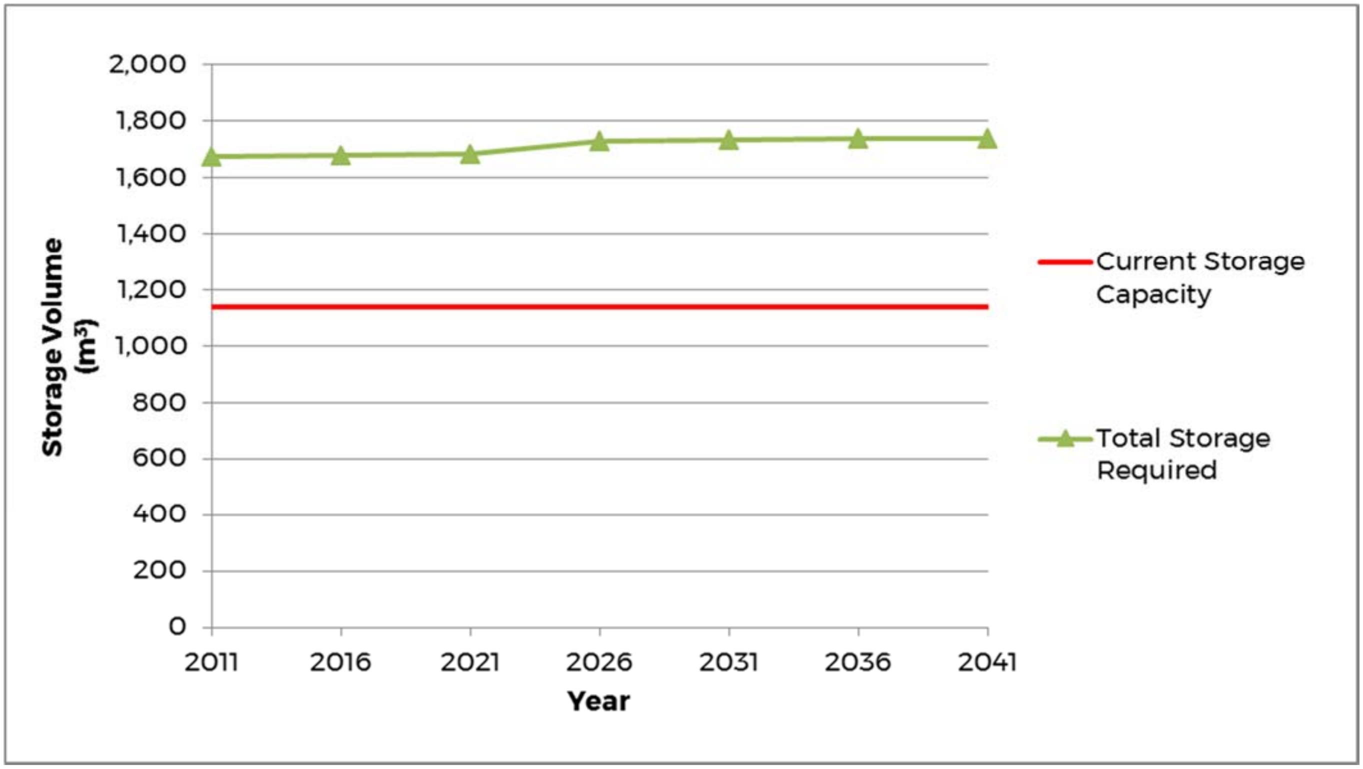

Figure 2-5 Falconbridge Water Required Storage vs. Existing Storage Capacity

WATERMAINS

During assessment of the watermains within the Falconbridge Water System, hydraulic computer modeling identified that in many areas of the system, watermains are 150 mm diameter or smaller and therefore may not have capacity to deliver fire flows that meet current standards. Similarly, areas with dead end watermains are not able to deliver fire flows that meet current standards. The Falconbridge Water System Gap Analysis and Status Quo Report (WSP, 2015), contained in Appendix 2-A of this report, which outlines areas identified to have pipe capacity deficiencies within the Falconbridge Water System.

2.3 ONAPING-LEVACK WATER SYSTEM

Levack and Onaping are small communities located in the north-west end of the City of Greater Sudbury. They are serviced by one (1) water system, and therefore have been included under the same section of the water infrastructure discussions throughout the Master Plan Report. Figure 2-6 illustrates the existing water infrastructure in the Onaping-Levack Water System.

Figure 2-6 Onaping-Levack Water System: Existing Infrastructure

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2.3.1 EXISTING SYSTEM

The Onaping-Levack Water System is supplied by three (3) non–GUDI wells; Wells No. 3, No. 4 and No. 5, all owned and operated by the CGS. Wells No. 3 and No. 4 are housed in a single pump house while Well No. 5 is housed in a separate building, which includes the common treatment facility for the entire system. The treatment processes include a chlorine gas system, fluoridation system, polyphosphate addition system and standby power. Sodium hydroxide is also added to control pH. The rated capacity of the wells as prescribed by the PTTW is 5,237 m^3/d. Table 2-8 summarizes the wells' process information.

Table 2-8 Onaping Wells Process Information

WELL	RATED CAPACITY (M^3/D)[2]	PUMP TYPE[1]	OPERATING POINT[1]	STANDBY POWER[1]
Well No. 3	5,237	Vertical turbine pump	30.3 L/s at 83 m TDH	250 kW diesel generator with ATS
Well No. 4		Vertical turbine pump	30.3 L/s at 83 m TDH	
Well No. 5		Vertical turbine pump with VFD	60.0 L/s at 83 m TDH	

[1] Data obtained from the Onaping/Levack DWWP.

[2] Best practices assume largest well out of service to determine the rated capacity.

The Onaping-Levack Water System also consists of three (3) pressure zones, and storage is provided by two (2) storage tanks; the Onaping Storage Tank and the Craig Mine Tank (which is not City owned and therefore not included in the Onaping-Levack System analysis). The Onaping Storage Tank has a capacity of 2,400 m^3.

The system also comprises a pressure control building (PCB) and the Frasier Pressure Reducing Valve (PRV). The PCB reduces the pressure in the Levack system and increases the pressure to the Craig Mine, and the PRV maintains higher pressures at the top of Frasier Avenue and Frasier Crescent, and reduces pressure at the bottom of Frasier Avenue.

2.3.2 EXISTING AND FUTURE REQUIREMENTS

Using the population projections and water demand rate development process described in Section 1.4 of Volume 1 of this report, Onaping-Levack's future water demand projections were calculated. Table 2-9 summarizes the Onaping-Levack demand criteria and the reference used to determine the criteria, and Table 2-10 summarizes the calculated demand projections.

Table 2-9 Onaping-Levack Water System Demand Criteria

CRITERIA	VALUE	REFERENCE
Average Day Residential Flow	350 L/cap/d	City's Engineering Design Manual, rounded down from 410 L/cap/d
Average Day Commercial and Institutional Flow	28 m^3/ha/d	MOECC guidelines
Average Day Industrial Flow	35 m^3/ha/d	MOECC guidelines
Domestic Demand Maximum Day Factor	1.70	Average of historical values
Domestic Demand Peak Hour Factor	3.27	Maximum of historical values

Table 2-10 Onaping-Levack Water Demand Projections

YEAR	POPULATION	AVERAGE DAY DEMAND (M3/D)	MAXIMUM DAY DEMAND (M3/D)	PEAK HOUR DEMAND (M3/D)
Base	2,112	1,708	2,853	5,259
2016	2,123	1,712	2,910	5,596
2021	2,135	1,716	2,917	5,609
2026	2,146	1,739	2,957	5,687
2031	2,154	1,742	2,962	5,696
2036	2,159	1,887	3,208	6,169
2041	2,159	1,887	3,208	6,169

2.3.3 GAP ANALYSIS CONCLUSIONS

As previously mentioned, a gap analysis was conducted in order to determine existing and future water system deficiencies for each system. The following information is a summary of the Onaping-Levack Water System Gap Analysis and Status Quo Report (WSP, 2015), contained in Appendix 2-A. The report can be referenced for more details regarding the analysis of the Onaping-Levack Water System.

An initial key noted issue within the Onaping-Levack Water System was that the Craig Mine can use booster pumps for approximately one (1) hour to fill their tank. When the mine's demands are high, this can occur as frequently as every four (4) hours, putting strain on the City's supply, and drawing from the Onaping Tank.

SUPPLY

Analysis of the Onaping Wells concluded that there would be sufficient capacity to service the population growth to the year 2041. This can be seen in Figure 2-7 where the projected maximum day demands are plotted against the capacity of the wells.

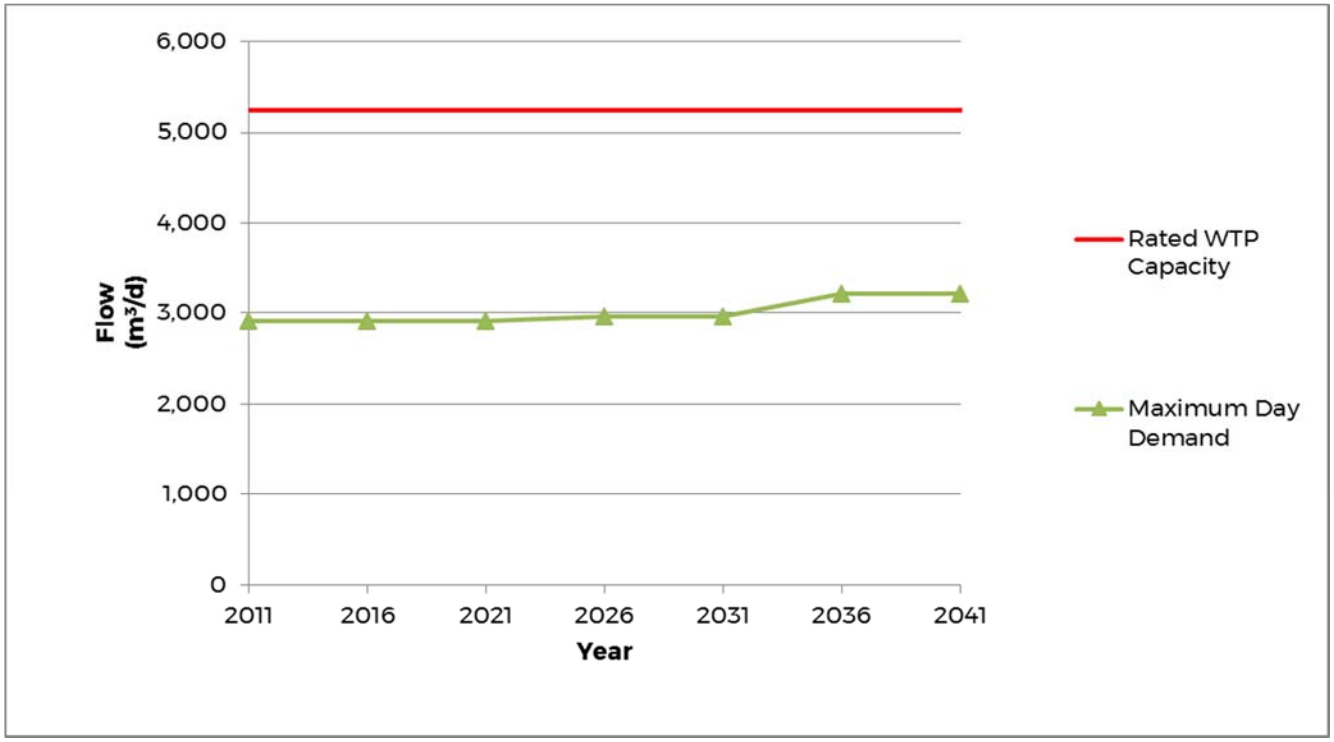

Figure 2-7 Onaping-Levack Water Demand Projections vs. Rated Capacity of the Wells

STORAGE

Analysis of the Onaping Storage Tank concluded that no additional storage would be required for the Onaping-Levack Water System to service the population growth to the year 2041. This can be seen in Figure 2-8 where the required water storage is plotted against the existing water storage capacity.

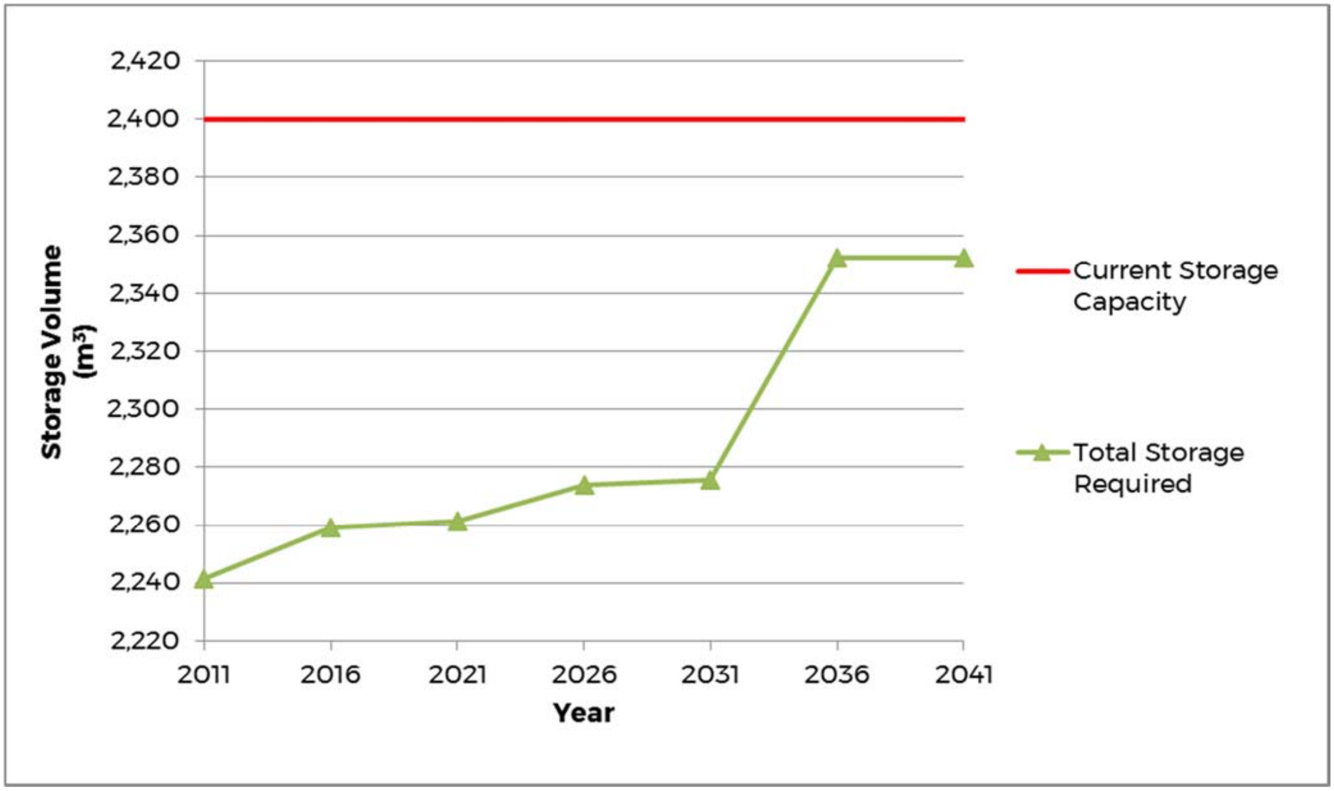

Figure 2-8 Onaping-Levack Water Required Storage vs. Existing Storage Capacity

WATERMAINS

During assessment of the watermains within the Onaping-Levack Water System, hydraulic computer modeling identified that fire flows are not met at the majority of the dead ends in the system. Water pressures were within an acceptable range, with the exception of the watermains nearby and entering the Craig Mine Tank, which were noted to be upwards of 100 psi. Additional data regarding the mine's water takings would be required to confirm the reason for the high pressure in this watermain.

Refer to the Onaping-Levack Water System Gap Analysis and Status Quo Report (WSP, 2015), contained in Appendix 2-A of this report, which outlines areas identified to have pipe capacity deficiencies within the Onaping-Levack Water System.

2.4 SUDBURY WATER SYSTEM

Sudbury is located centrally in the CGS and is the City's most populated area. The water system services the communities of Coniston, Garson, Sudbury, Wahnapitae, and Markstay-Warren. Figure 2-9 illustrates the existing water infrastructure in the Sudbury Water System.

Figure 2-9 Sudbury Water System: Existing Infrastructure

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2.4.1 EXISTING SYSTEM

The Sudbury Water System is supplied by two (2) surface water treatment plants; the David Street WTP and the Wanapitei WTP, and three (3) wells; Well No. 1, No. 2, and No. 3 (the Garson Wells). All of the facilities are owned and operated by the City of Greater Sudbury. Table 2-11 summarizes the Sudbury Water System supply capacities, and further information can be found in the proceeding sections.

Table 2-11 Sudbury Water System Supply Capacity Summary

WATER SUPPLY	RATED CAPACITY (M^3/D)	ESTIMATED ACTUAL CAPACITY (M^3/D)
Wanapitei WTP	54,000[1]	40,000[1]
David Street WTP	40,000[2]	37,260[2]
Garson Orell Well No. 1	1,572[3]	1,572[3]
Garson Well No. 2	2,981[4]	2,981[4]
Garson Orell Well No. 3	3,274[3]	0[5]
Sudbury System	**101,827**	**81,813**

[1] The rated capacity for the Wanapitei WTP is 54,000 m^3/d. It has been assumed that, as an outcome of a master plan project, the hydraulic limitations can be fixed, allowing the plant to deliver its rate capacity.

[2] Although the rated plant capacity is 40,000 m^3/d, the PTTW for this facility limits the monthly average day production to 27,760 m^3/d, corresponding to a maximum day amount of 37,260 m^3/d.

[3] Rated capacity obtained from Garson Orell Wells PTTW #5376-84BMP7.

[4] Rated capacity obtained from Garson Well 2 PTTW #5307-8YHNAM.

[5] Best practices assume largest well out of service to determine firm capacity.

WELLS

There are three (3) wells located in Garson and primarily supply the east end of the community of Garson, although the Garson and Sudbury communities are interconnected. Typically, the west end is fed from Sudbury surface water, while the east side is fed from the Garson Wells. The O'Neil Pressure Sustaining Valve (PSV) isolates the east and west sides of the Garson water distribution network. If pressure drops beyond a specific setpoint on either side of the valve, the PSV opens to feed water into the area of lower pressure.

The Garson Wells property has two (2) well houses, one (1) chemical building, and one (1) buried chlorine contact tank. Well Houses 1 and 2 contain the vertical turbine well pumps, pumping to a common 200 mm header to the chemical building. The raw water is then treated with sodium hypochlorite and fluoride prior to entering the contact tank. The buried process piping allows for isolation of the contact tank. Table 2-12 summarizes the wells' capacity and process information.

Table 2-12 Garson Wells' Process Information

WELL	RATED CAPACITY (M^3/D)	PUMP TYPE[1]	OPERATING POINT[1]	STANDBY POWER[1]
Well 1	See Table 2-11	Vertical turbine pump	22.7 L/s at 63.7 m TDH	125 kW diesel generator with automatic transfer switch (ATS)

WELL	RATED CAPACITY (M^3/D)	PUMP TYPE[1]	OPERATING POINT[1]	STANDBY POWER[1]
Well 2		Vertical turbine pump equipped with variable frequency drive (VFD)	34.5 L/s at 93.8 m TDH	None
Well 3		Vertical turbine pump	34 L/s at 64.0 m TDH	125 kW diesel generator with automatic transfer switch (ATS)

[1] *Data obtained from the Sudbury Drinking Water Works Permit, Number 016-206 Issue 2.*

WATER TREATMENT PLANTS

WANAPITEI WTP

The Wanapitei WTP is supplied by the Wanapitei River, and services Sudbury, Wahnapitae, Coniston, and Markstay-Warren. The plant is a conventional surface WTP, with a treatment process as follows:

1. Chlorine Gas or Chlorine Dioxide for Taste and Odour Control
2. Alum, Lime and Polymer Addition – Flash Mixing Chamber
3. Sedimentation Process
4. Filtration
5. UV Disinfection
6. Addition of Hydrated Lime, Fluoride, Chlorine (secondary disinfection), and Polyphosphate

According to the Wanapitei WTP Hydraulic Capacity Report (AECOM, 2009), the Wanapitei WTP is limited to a maximum flow of 44,000 m^3/d due to insufficient high lift pumping capacity and hydraulic pressure limitations of the existing transmission main between the plant and Sudbury. City operations staff has indicated that, in practice, the plant can operate between 40,000 to 42,000 m^3/d. For purposes of this study, a conservative plant production capacity of 40,000 m^3/d was used.

DAVID STREET WTP

The David Street WTP is supplied by Ramsey Lake, and services south, west, and downtown areas of Sudbury. The plant services Garson if there is low pressure in the Garson network..

The treatment process at the David Street WTP includes:

1. Sodium Hypochlorite/Sodium Permanganate Addition
2. Pre-Treatment Straining
3. First and Second Stage Membrane Tanks
4. UV Disinfection
5. Addition of Fluoride, Chlorine (secondary disinfection), Sodium Hydroxide Addition and Polyphosphate

According to the plant PTTW, the maximum permitted water taking is 40,000 m^3/d; however, the monthly average rate may not exceed 27,760 m^3/d. For the purpose of the Master Plan, under existing conditions, the plant's production capacity was estimated at 37,260 m^3/d.

BOOSTER PUMPING STATIONS

The Sudbury distribution system consists of thirteen (13) pressure zones, and eight (8) booster pumping stations. Information regarding the BPS capacities can be seen in Table 2-13.

Table 2-13 Sudbury Water System Booster Pumping Station Summary

FACILITY	PUMP INFORMATION	TOTAL CAPACITY (L/S)	FIRM CAPACITY (L/S)[2]
Algonquin	Two (2) centrifugal pumps with variable speed drives, each pump rated at 17.7 L/s at 16 m TDH	35.4	17.7
Copper Park[1]	Three (3) centrifugal pumps with variable speed drives; two (2) pumps rated at 10 L/s at 32 m TDH each and one (1) pump rated at 80 L/s at 38.5 m TDH	100	20.0
Jogues	Two (2) centrifugal pumps with variable speed drives, each pump rated at 11.4 L/s at 19.5 m TDH	22.8	11.4
Maley[1]	Two (2) vertical turbine pumps with variable speed drives, each pump rated at 45 L/s at 49 m TDH and one (1) centrifugal pump rated at 120 L/s at 56 m TDH.	210	90.0
Montrose	Two (2) centrifugal pumps, one (1) rated at 18.9 L/s at 22.9 m TDH and one (1) rated at 63.1 L/s at 22.9 m TDH	82.0	18.9
Moss	One (1) pump rated at 3.8 L/s	3.8	0
Snowdon	Two (2) centrifugal pumps, one (1) rated at 19.7 L/s at 29 m TDH and one (1) rated at 70 L/s (TDH not known)	89.7	19.7
Sunrise Ridge[1]	Three (3) centrifugal pumps with variable speed drives; two (2) pumps rated at 9.8 L/s at 164.9 m TDH each and one (1) pump rated at 81.5 L/s at 48.1 m TDH.	101	19.6

[1] Standby power available.

[2] Based on the largest pump out of service.

In addition to the above booster pumping stations, Laurentian University obtains water from the municipal supply and pressurizes the university campus through the Laurentian BPS. This BPS is owned and operated by Laurentian University and is therefore not included in this study.

STORAGE

Storage in the Sudbury Water System is provided by one (1) reservoir, the Ellis Reservoir. The reservoir is an in-ground dual cell reservoir and rechlorination facility that receives water directly from the Wanapitei and David Street WTPs. According to the DWWP, the reservoir has a capacity of 36,400 m^3. City staff has observed that when the reservoir is filled to its top water level, the frequency of watermain breaks in the surrounding area increases. As a result, the Ellis Reservoir is not filled to capacity, thereby reducing its useful volume. The reservoir is typically filled to a maximum useful volume of approximately 26,700 m^3.

2.4.2 EXISTING AND FUTURE REQUIREMENTS

Using the population projections and water demand rate development process described in Section 1.4 of Volume 1 of this report, Sudbury's future water demand projections were calculated. Table 2-14 summarizes the Sudbury demand criteria and the reference used to determine the criteria, and Table 2-15 summarizes the calculated demand projections.

Table 2-14 Sudbury Water System Demand Criteria

CRITERIA	VALUE	REFERENCE
Average Day Residential Demand	350 L/cap/d	Average of historical values, rounded up to nearest 50 L/cap/d
Average Day Institutional & Commercial Demand	28 m^3/ha/d	MOECC Guidelines
Average Industrial Demand	35 m^3/ha/d	MOECC Guidelines
Domestic Demand Maximum Day Factor	1.39	Highest historical value
Domestic Demand Peak Hour Factor	1.58	Highest historical value

Table 2-15 Sudbury Water Demand Projections

YEAR	POPULATION	AVERAGE DAY DEMAND (M^3/D)	MAXIMUM DAY DEMAND (M^3/D)	PEAK HOUR DEMAND (M^3/D)
Base	94,868	44,150	59,601	66,705
2016	95,826	50,486	70,259	79,823
2021	97,059	50,918	70,860	80,506
2026	98,330	54,720	76,151	86,517
2031	99,056	54,974	76,505	86,919
2036	99,506	64,566	89,853	102,085
2041	99,450	64,546	89,826	102,054

2.4.3 GAP ANALYSIS CONCLUSIONS

As previously mentioned, a gap analysis was conducted in order to determine existing and future water system deficiencies for each system. The following information is a summary of the Sudbury Water System Gap Analysis and Status Quo Report (WSP, 2016), contained in Appendix 2-A. The report can be referenced for more details regarding the analysis of the Sudbury Water System.

SUPPLY

Analysis of the Sudbury supply system concluded that there would be sufficient capacity to service the population growth to the year 2031; however, an additional supply of 8,013 m^3/d would be required to service growth to 2041. Generally, capacity upgrades are triggered when a system reaches 80% of current production capacity. In this case, this is at a maximum day flow of 65,450 m^3/d. This is summarized in Figure 2-10 where the projected maximum day demands are plotted against the capacity of the system.

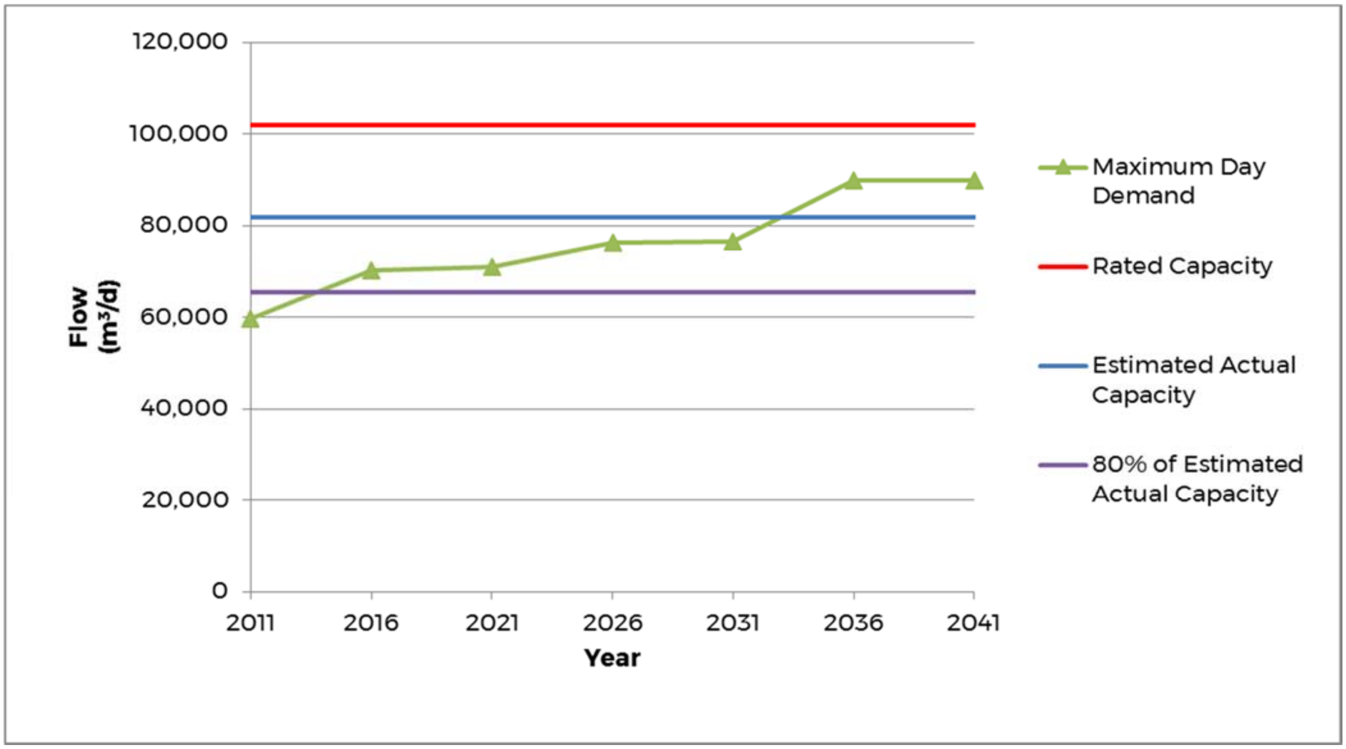

Figure 2-10 Sudbury Water Demand Projections vs. Rated Capacity and Estimated Actual Capacity of the Supply System

It should also be noted that Ramsey Lake is a vulnerable water supply and may not be sustainable in the future due to water quality threats, as documented in Source Water Protection documentation described in the Water Baseline Review Report (WSP, 2015). Similarly, the Garson Wells have detectable levels of tetrachloroethylene (PCE) and must continue to be monitored. The wells may require treatment in the future to meet water quality requirements, if PCE levels continue to increase.

Additionally, the David Street WTP has had operational and maintenance challenges in addition to issues with moisture and corrosion. It was also noted that there are ongoing issues with valves and analyzers.

STORAGE

Analysis of the Ellis Reservoir concluded that no additional storage would be required for the Sudbury Water System to service the population growth to the year 2041, pending improvements to the system that will allow for the use of the full tank volume. Without such improvements, the system has enough useable storage to service demands to 2031. By 2041, the system would have a deficit of 2,721 m^3. This is illustrated in Figure 2-11 where the required water storage is plotted against the existing water storage capacity.

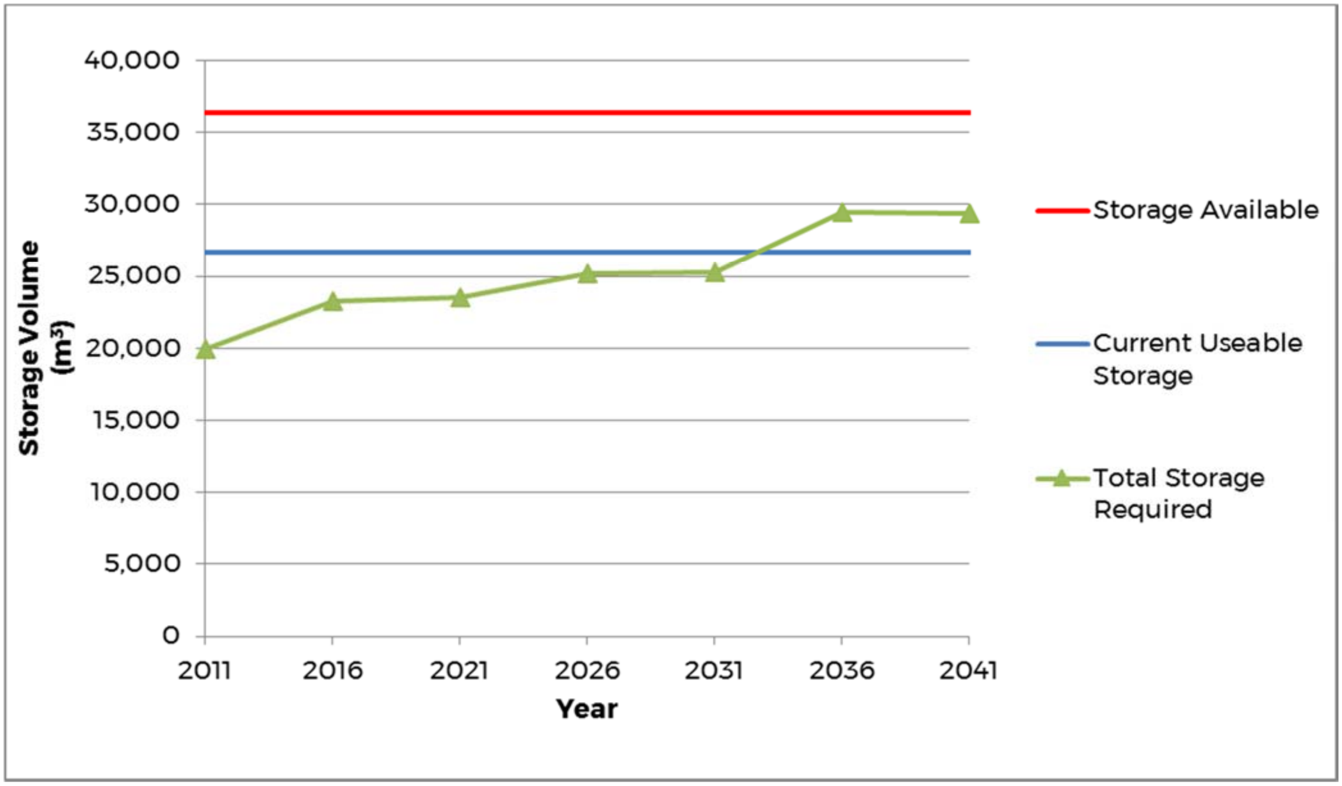

Figure 2-11 Sudbury Water Required Storage vs. Existing Usable Storage and Available Storage

WATERMAINS

During assessment of the watermains within the Sudbury Water System, hydraulic computer modeling identified that in many areas of the system, watermains are 150 mm diameter or smaller and therefore may not have capacity to deliver fire flows that meet current standards.

2.5 VALLEY WATER SYSTEM

Valley is located in the north end of the City of Greater Sudbury and is the second most populated area, following the community of Sudbury. The Valley Water System services the communities of Azilda, Blezard Valley, Capreol, Chelmsford, Hanmer, McCrea Heights, Val Therese, Val Caron, and portions of the rural community that have water servicing only. Figure 2-12 illustrates the existing water infrastructure in the Valley Water System.

Figure 2-12 Valley Water System: Existing Infrastructure

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2.5.1 EXISTING SYSTEM

The Valley Water System is supplied by thirteen (13) wells; eleven (11) in Valley East, and two (2) in Capreol, all owned and operated by the CGS. It should be noted that Well I has been turned off since 2013, which reduces the number of wells currently operational to twelve (12). The Valley East wells' aquifer is characterized as a non-GUDI, shallow, sand and gravel aquifer. Wells J and M (Capreol Wells) draw water from a common unconfined aquifer comprised mostly of sands and gravels, and classified as a GUDI water source with effective filtration, per the DWWP. The water is treated with UV irradiation for primary disinfection, chlorine gas and UV for secondary disinfection, and polyphosphate for iron and manganese sequestration. Fluoride is also added.

The total rated capacity for the system is 34,285 m^3/d; however, it is not possible to operate the system at its rated capacity due to well capacity constraints. A more realistic assessment of capacity, taking into account well pumping and drawdown limitations, identifies the available production capacity as 28,453 m^3/d, or a firm production capacity of 24,578 m^3/d Table 2-16 summarizes the wells' process information.

Table 2-16 Valley Wells' Process Information

WELL	RATED CAPACITY (M^3/D)[1]	ESTIMATED ACTUAL CAPACITY (M^3/D)[2]	PUMP TYPE[1]	WELL CAPACITY OPERATING POINT[1]	STANDBY POWER[4]
VALLEY WELLS					
Chenier	2,333	2,278	Vertical turbine well pump with variable speed control	26.5 L/s at 71.1 m TDH	150 kW diesel generator
Deschene	1,798	1,631	Vertical turbine well pump	20.8 L/s at 55.5 m TDH	50 kW diesel generator
Kenneth	2,288	1,521	Vertical turbine well pump	26.5 L/s at 56.4 m TDH	50 kW diesel generator
Frost	2,288	2,290	Vertical turbine well pump	26.5 L/s at 55.5 m TDH	70 kW diesel generator
Linden	3,269	2,506	Vertical turbine well pump	37.8 L/s at 61.6 m TDH	None
Well I	1,974	0	Vertical turbine well pump	29.9 L/s at 76.2 m TDH	150 kW diesel generator
Michelle	2,290	2,290	Vertical turbine well pump	26.5 L/s at 55.8 m TDH	None
Notre Dame	3,105	2,103	Vertical turbine well pump	35.9 L/s at 60.7 m TDH	70 kW diesel generator
Pharand	2,290	2,007	Vertical turbine well pump	26.5 L/s at 57.3 m TDH	None
Philippe	2,288	2,198	Vertical turbine well pump	26.5 L/s at 59.4 m TDH	50 kW diesel generator
Well R	3,162	3,014	Vertical turbine well pump with variable speed control	36.0 L/s at 72.8 m TDH	150 kW diesel generator

WELL	RATED CAPACITY (M^3/D)[1]	ESTIMATED ACTUAL CAPACITY (M^3/D)[2]	PUMP TYPE[1]	WELL CAPACITY OPERATING POINT[1]	STANDBY POWER[4]
CAPREOL WELLS					
Well J	3,273	2,740	Vertical turbine well pump with VFD	37.9 L/s at 91.4 m TDH	400 kW diesel generator located at Well M and servicing both wells
Well M	3,927	3,875[3]	Vertical turbine well pump with variable speed drive	45.4 L/s at 76.0 m TDH	
Total	**34,285**	**24,579**	-	-	-

[1] Data obtained from the Valley Municipal Drinking Water Licence, Number 016-105 Issue 4.
[2] Estimated based on discussions with City staff. Based on 2015 Max Day Capacities.
[3] Not included in total - based on the largest pump out of service (Best practice when determining Firm Capacity).
[4] Data obtained from the Valley Drinking Water Works Permit 016-205, Issue Number 3.

STORAGE

Table 2-17 summarizes the storage facilities in the Valley Water System and their usable volumes. It should be noted that although the entire Valley Water System is interconnected, each storage facility generally services its own specific community. It is also important to note that the Azilda and Chelmsford Tanks are located at the opposite end of the system from the supply wells, and are connected only by a single trunk watermain.

Table 2-17 Valley Water System Storage Summary

TANK	TYPE	USABLE VOLUME (M3)
Azilda	Standpipe	4,524
Chelmsford	Elevated	1,353
Val Caron	Ground Level	5,274

BOOSTER PUMPING STATIONS

Table 2-18 provides a summary of the BPSs in the Valley Water System and their capacities.

Table 2-18 Valley Water System Booster Pumping Station Summary

FACILITY	PUMP INFORMATION	TOTAL CAPACITY (L/S)	FIRM CAPACITY[1] (L/S)
Capreol BPS (supplied by Valley wells)	Three (3) constant speed centrifugal pumps, each rated at 34.3 L/s at 57.3 m TDH	102.9	68.6
Centennial BPS	Two (2) constant speed centrifugal pumps, one (1) rated at 4.4 L/s at 31 m TDH and one (1) rated 75 L/s at 18.3 m TDH	79.4	4.4

FACILITY	PUMP INFORMATION	TOTAL CAPACITY (L/S)	FIRM CAPACITY[1] (L/S)
Val Caron BPS (located on same site as Val Caron Storage Tank)	Two (2) constant speed centrifugal pumps, one (1) rated at 12 L/s at 32 m TDH and one (1) rated at 28 L/s at 32 m TDH One (1) 75 L/s fire pump	40	12

[1] *The firm capacity is calculated assuming the largest pump out of service.*

2.5.2 EXISTING AND FUTURE REQUIREMENTS

Using the population projections and water demand rate development process described in Section 1.4 of Volume 1 of this report, Valley's future water demand projections were calculated. Table 2-19 summarizes the Valley demand criteria and the reference used to determine the criteria, and Table 2-20 summarizes the calculated demand projections.

Table 2-19 Valley Water System Demand Criteria

CRITERIA	VALUE	REFERENCE
Average Day Residential Demand	250 L/cap/d	Average of historical values, rounded up to nearest 50 L/cap/d
Average Day Institutional & Commercial Demand	28 m^3/ha/d	MOECC Guidelines
Average Industrial Demand	35 m^3/ha/d	MOECC Guidelines
Domestic Demand Maximum Day Factor	1.46	Average of historical values
Domestic Demand Peak Hour Factor	2.18	Maximum of historical values

Table 2-20 Valley Water Demand Projections

YEAR	POPULATION	AVERAGE DAY DEMAND (M3/D)	MAXIMUM DAY DEMAND (M3/D)	PEAK HOUR DEMAND (M3/D)
Base	36,382	10,082	14,760	19,767
2016	37,235	10,295	15,031	22,456
2021	38,142	10,522	15,362	22,950
2026	38,965	12,100	17,665	26,391
2031	39,451	12,221	17,843	26,656
2036	39,737	17,124	25,001	37,350
2041	39,764	17,131	25,011	37,365

2.5.3 GAP ANALYSIS CONCLUSIONS

As previously mentioned, a gap analysis was conducted in order to determine existing and future water system deficiencies for each system. The following information is a summary of the Valley Water System Gap Analysis and Status Quo Report (WSP, 2015), contained in Appendix 2-A. The report can be referenced for more details regarding the analysis of the Valley Water System.

SUPPLY

Analysis of the Valley Wells concluded that there would be sufficient capacity to service the population growth to the year 2031; however, an additional 432 m^3/d would be required to service growth to 2041. This can be seen in Figure 2-14 where the projected maximum day demands are plotted against the capacity of the wells.

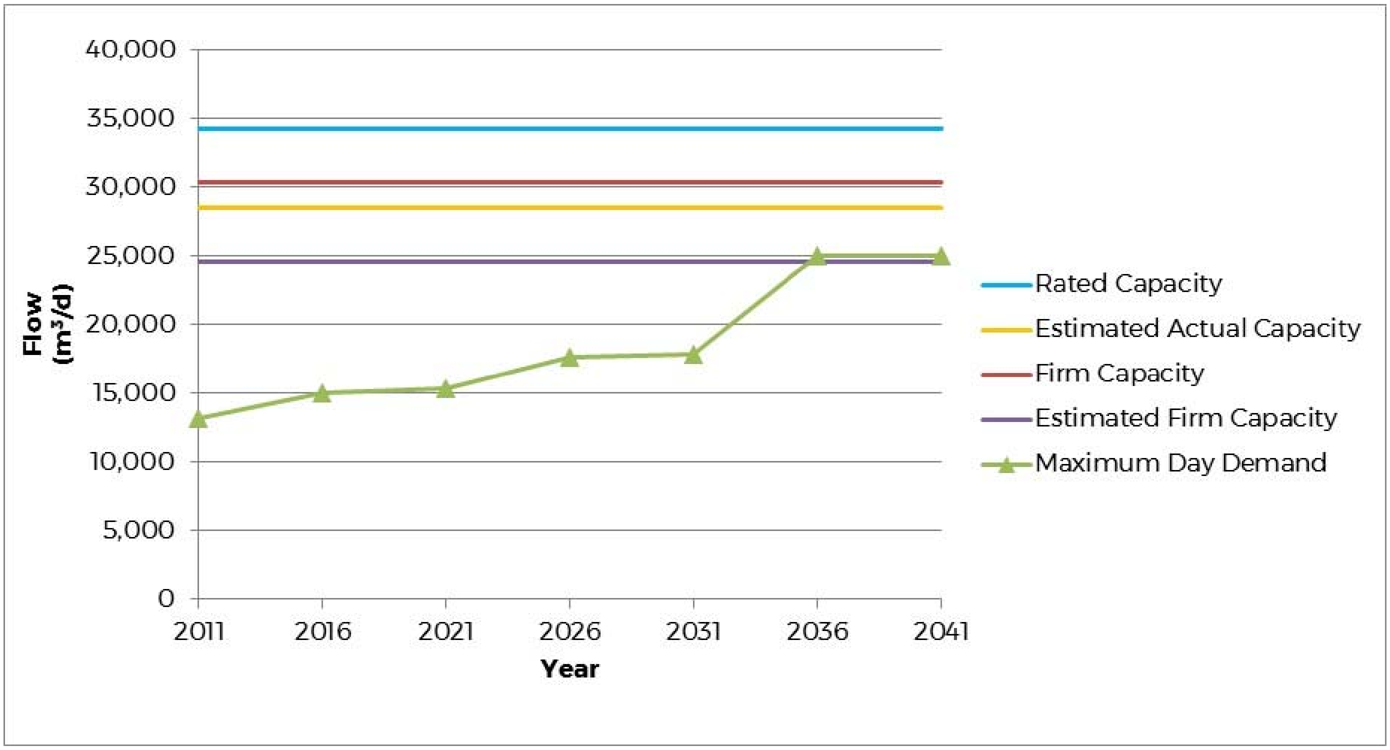

Figure 2-13 Valley Water Demand Projections vs. Rated and Firm Capacities

Data reported in the Annual Report for the Valley water supply (including Capreol) includes treated water chlorine residual, trihalomethanes (THMs), fluoride, and trace organic and inorganic chemicals. Data was reviewed from 2009 to 2013 to determine any historical issues at the wells. No exceedances were observed, except for elevated sodium levels at Philippe, Pharand, Michelle, and R Wells.

City operations staff have indicated several specific concerns with the Valley Wells, including:

- Operational issues with the UV system at Deschene and Kenneth Wells when using standby power
- Pharand Well has higher than average sodium levels
- Iron and manganese concentrations have increased at Michelle well, which has caused operational issues such as UV fouling, resulting in the need to use more chlorine
- Elevated concentrations of iron at Kenneth Well resulting in higher chlorine usage and higher maintenance costs for the UV system
- Elevated concentration of iron at Linden Well requiring more frequent maintenance of system analyzers

STORAGE

Analysis of the Valley Water System concluded that no additional storage would be required to service the population growth to the year 2041. This is demonstrated in Figure 2-14 where the required water storage is plotted against the existing water storage capacity.

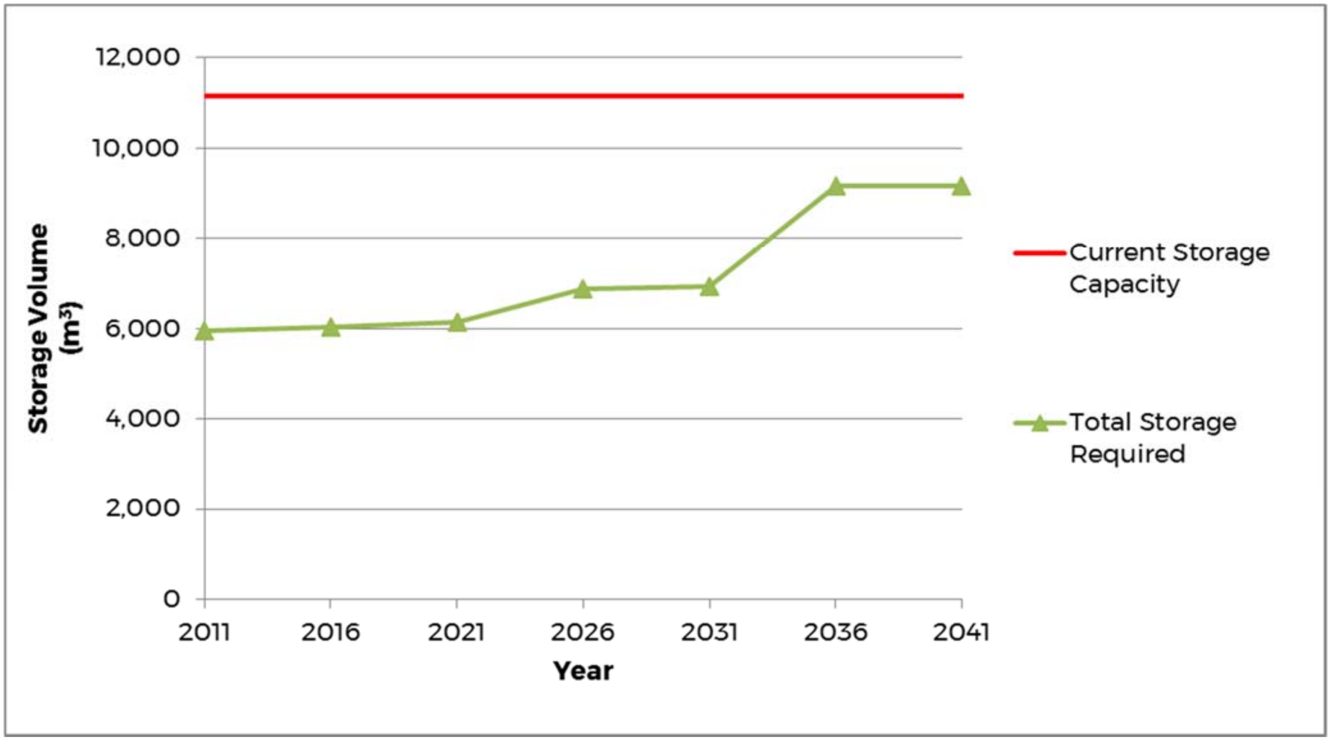

Figure 2-14 Valley Water Required Storage vs. Existing Storage Capacity

Although capacity analysis concluded that there would be sufficient storage to service population growth to 2041, the following concerns have been noted with the Azilda and Val Caron storage facilities in the Valley Water System:

AZILDA STANDPIPE

It was noted that the Azilda Standpipe has lower water elevations than the other two (2) storage facilities in the Valley water system. This, along with the three (3) facilities being located in the same pressure zone, results in water being distributed from the other two (2) storage facilities predominately, and water in the Azilda Standpipe tends to remain in the tank. This has historically caused stagnant water and freezing issues.

VAL CARON TANK

Tank may drain completely in the event of an emergency. Refilling may take days, impacting servicing to the McRea Heights neighbourhood. Valley Looping and Storage Class EA recommended the installation of automated, remotely controlled isolation valve.

WATERMAINS

During assessment of the watermains within the Valley Water System, hydraulic computer modeling identified that in many areas of the system, watermains are 150 mm diameter or smaller and therefore may not have capacity to deliver fire flows that meet current standards. The Valley Water System Gap Analysis and Status Quo Report (WSP, 2015), included in Appendix 2-A, outlines areas identified to have pipe capacity deficiencies within the Valley Water System.

2.6 VERMILION WATER SYSTEM

Vermilion is a water system located in the west end of the CGS. The system services Copper Cliff, Lively, Mikkola, Naughton, and Whitefish. Figure 2-15 illustrates the existing water infrastructure in the Vermilion Water System.

Figure 2-15 Vermilion Water System: Existing Infrastructure

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2.6.1 EXISTING SYSTEM

The Vermilion Water System receives water from the Vermilion WTP, which is owned and operated by a third party, Vale Limited (Vale) and has a rated capacity of 81,800 m^3/d. The City consumes approximately 20-30% of the Vermilion WTP capacity. The Vermilion WTP complies with all MOECC drinking water quality standards and requirements, and as such, possesses a drinking water works permit, a municipal drinking water licence, and an Operational Plan. The raw water comes from the nearby Vermilion River and the plant uses a conventional treatment process.

The Vermilion distribution system consists of a network of watermains mainly owned by the City however there are some watermains owned by Vale. The City also owns the Walden Standpipe. The standpipe has an effective storage of 2,662 m^3.

The following infrastructure is owned by Vale:

— 60,543 m^3 Copper Cliff Water Storage Tank
— Cobalt Booster Pumping Station (BPS)
— C.C. North Mine BPS
— Clarabelle North Mine BPS

It is important to note that Vale's Copper Cliff Water Storage Tank could provide some redundant supply in case of an emergency, but the volume dedicated for municipal use cannot be confirmed and has therefore not been included as useful volume in the Master Plan analysis.

2.6.2 EXISTING AND FUTURE REQUIREMENTS

Using the population projections and water demand rate development process described in Section 1.4 of Volume 1 of this report, Vermilion's future water demand projections were calculated. Table 2-21 summarizes the Vermilion demand criteria and the reference used to determine the criteria, and Table 2-22 summarizes the calculated demand projections.

Table 2-21 Vermilion Water System Demand Criteria

CRITERIA	VALUE	REFERENCE
Average Day Residential Demand	250 L/cap/d	MOECC Guidelines
Average Day Institutional & Commercial Demand	28 m^3/ha/d	MOECC Guidelines
Average Industrial Demand	35 m^3/ha/d	MOECC Guidelines
Average Industrial Demand (20% of Walden Industrial Park)	35 m^3/ha/d	Per Methodology in the Lively/Walden Environmental Summary Report (J.L. Richards & Associates Limited, 2013)
Average Industrial Demand (80% of Walden Industrial Park)	3 m^3/ha/d	Per Methodology in the Lively/Walden Environmental Summary Report (J.L. Richards & Associates Limited, 2013)
Average Industrial Demand (Existing Industrial Development in the Walden Industrial Park that is currently not serviced through the City's water supply)	3 m^3/ha/d	Per Methodology in the Lively/Walden Environmental Summary Report (J.L. Richards & Associates Limited, 2013)

CRITERIA	VALUE	REFERENCE
Domestic Demand Maximum Day Factor	1.90	MOECC Guidelines
Domestic Demand Peak Hour Factor	2.85	MOECC Guidelines

Table 2-22 Vermilion Water Demand Projections

YEAR	POPULATION	AVERAGE DAY DEMAND (M3/D)	MAXIMUM DAY DEMAND (M3/D)	PEAK HOUR DEMAND (M3/D)
Base	10,359	4,059	7,712	11,569
2016	10,845	4,212	8,003	12,004
2021	11,303	4,356	8,276	12,414
2026	11,686	5,315	10,098	15,148
2031	11,912	5,686	10,804	16,206
2036	12,050	6,646	12,627	18,941
2041	12,085	6,657	12,648	18,972

2.6.3 GAP ANALYSIS CONCLUSIONS

As previously mentioned, a gap analysis was conducted in order to determine existing and future water system deficiencies for each system. The following information is a summary of the Vermilion Water System Gap Analysis and Status Quo Report (WSP, 2016) contained in Appendix 2-A. The report can be referenced for more details regarding the analysis of the Vermilion Water System.

As a general note, through discussions with City staff it is understood that much of the City-owned infrastructure was grandfathered into the municipal system and information such as material and age of construction, as well as existing condition is not available.

SUPPLY

Analysis of the Vermilion Water System concluded that there would be sufficient capacity to service the population growth to the year 2041. This can be seen in Figure 2-16 where the projected maximum day demands are plotted against the capacity of the water treatment plant.

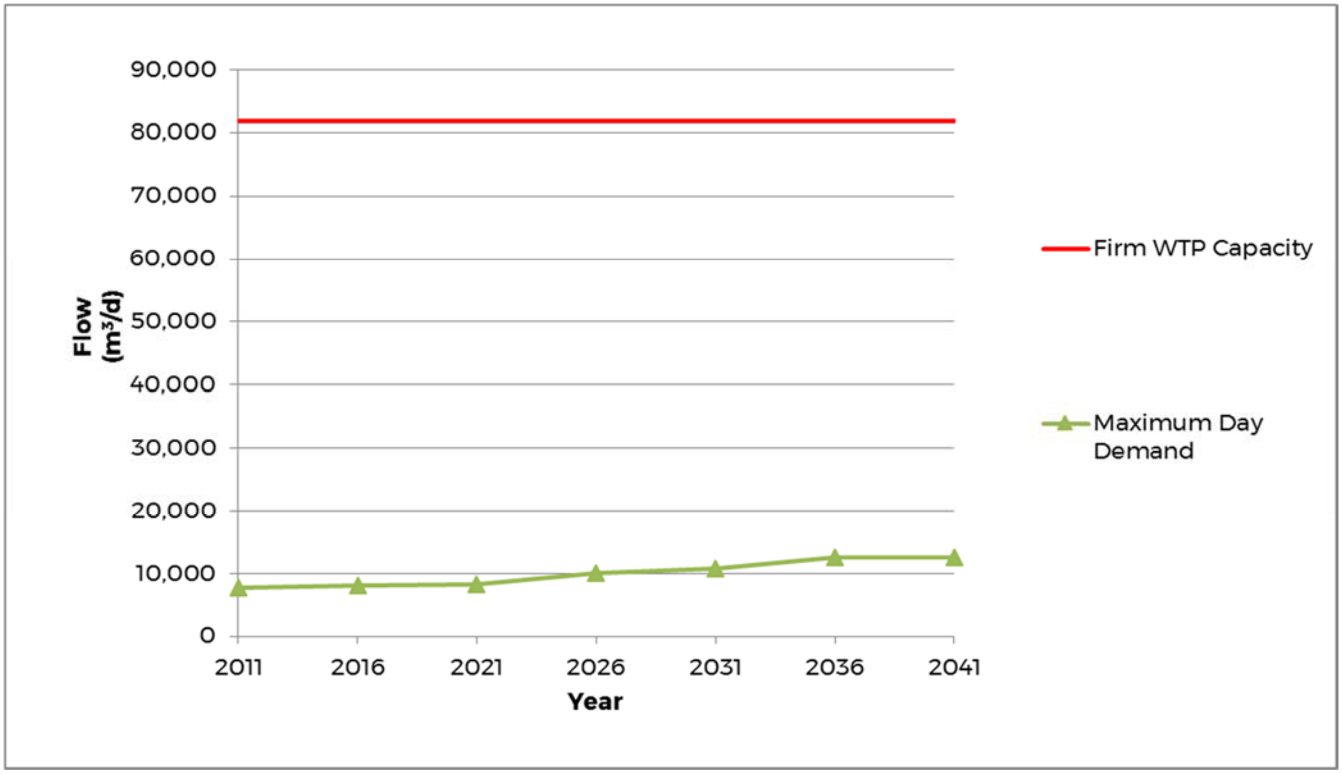

Figure 2-16 Vermilion Water Demand Projections vs. Firm Capacity

STORAGE

Analysis of the Vermilion Water System concluded that an additional 2,640 m^3 of storage would be required to service the population to 2041. This can be seen in Figure 2-17 where the required water storage is plotted against the existing water storage capacity.

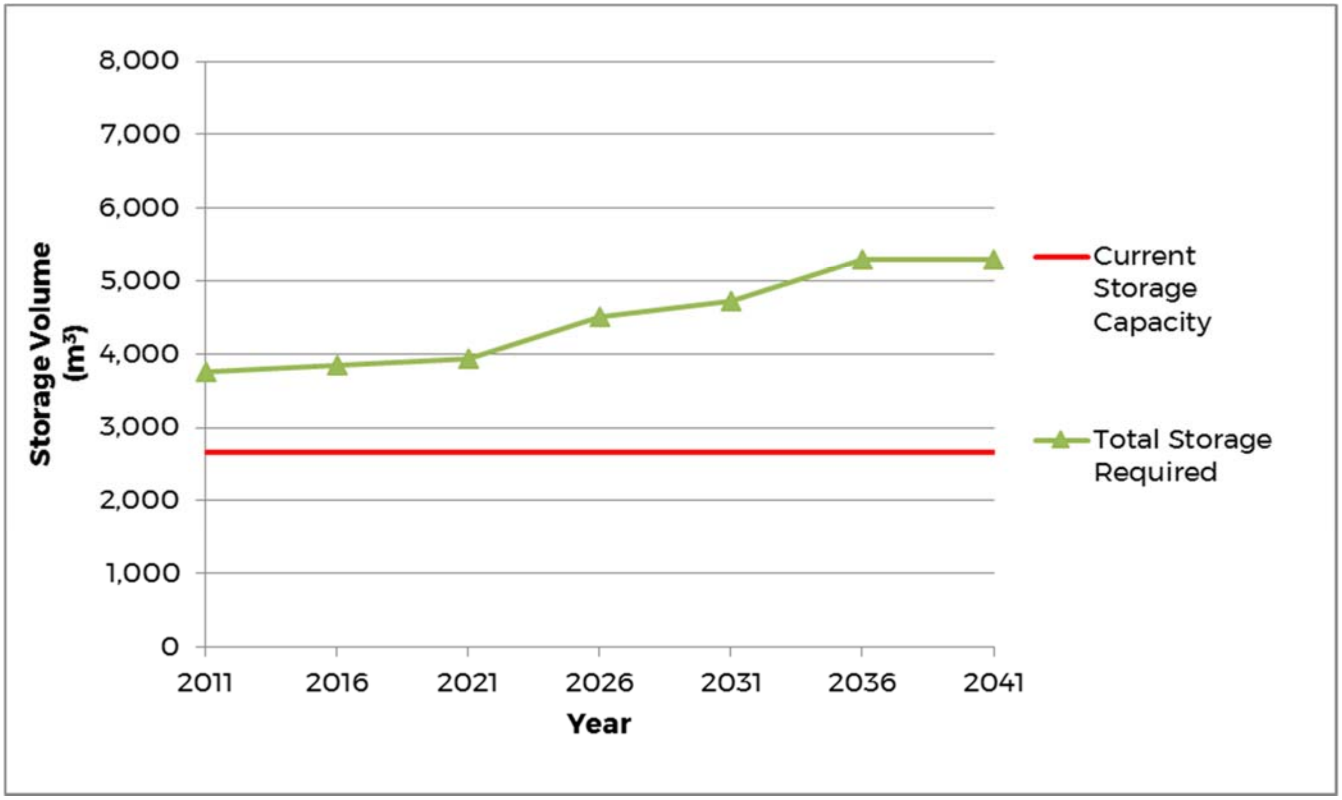

Figure 2-17 Vermilion Water Required Storage vs. Existing Storage Capacity

WATERMAINS

During assessment of the watermains within the Vermilion Water System, hydraulic computer modeling identified that in many areas of the system, watermains are 150 mm diameter or smaller and therefore may not have capacity to deliver fire flows that meet current standards. Similarly, areas with dead end watermains could not deliver flows that meet current fire flow standards. The Vermilion Water System Gap Analysis and Status Quo Report (WSP, 2016), included in Appendix 2-A, outlines areas identified to have pipe capacity deficiencies within the Vermilion Water System.

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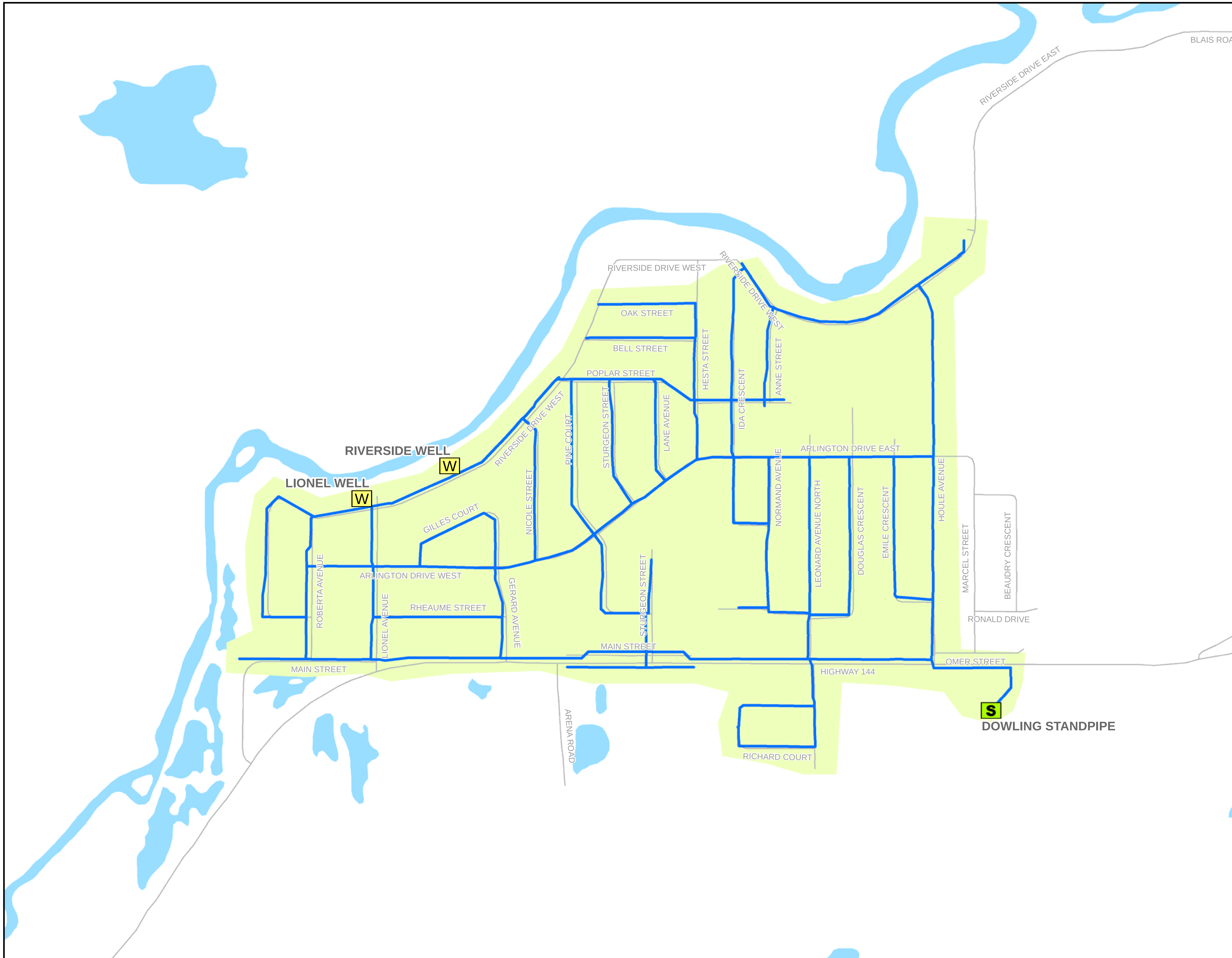


100 COMMERCE VALLEY DRIVE WEST
THORNHILL, ONTARIO,
CANADA, L3T 0A1
WWW.WSPGROUP.COM

200 BRADY STREET
SUDBURY, ONTARIO,
CANADA, P3A 5P3
WWW.GREATERSUDBURY.CA

LEGEND

W WELL

S STORAGE TANK

WATERMAIN

WATERMAIN

ROAD

WATERBODY

PRESSURE ZONE

DOWLING

SCALE:
0 55 110 220 Meters
1:9,000

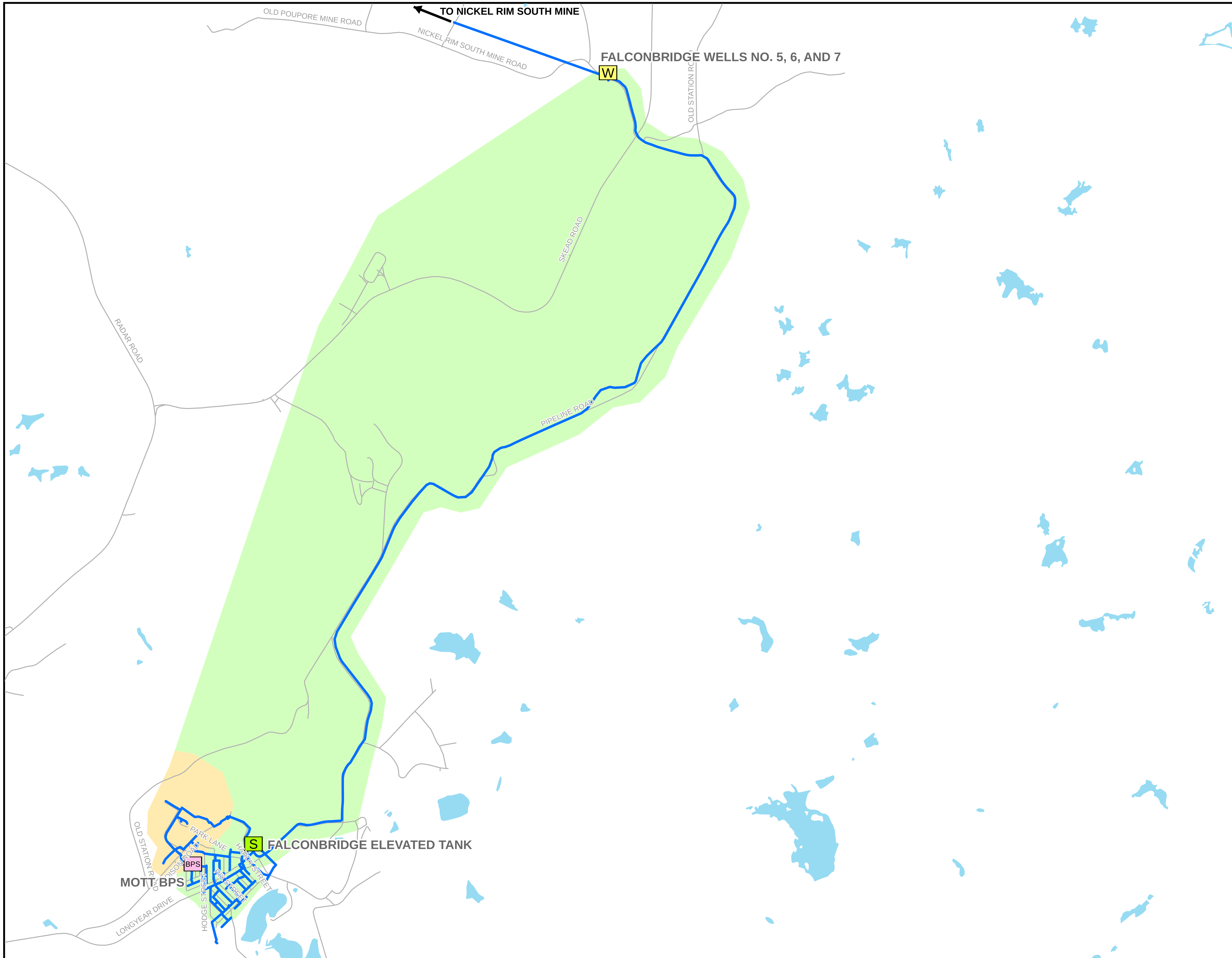
PROJECT:
Water and Wastewater Master Plan City of Greater Sudbury

MAP:
DOWLING WATER SYSTEM: EXISTING INFRASTRUCTURE

PROJECT NO.:	DATE:
121-23026-00	MARCH 2017

DRAWN BY:	CHECKED BY:	REPORT:	FIGURE NO.:
CM	KL	MP	2-1

File Location:S:\MA\12\121-23026-00 Sudbury W & WW Master Plan\11. Master Plan Document\Figures\Volume 2\Figure 2-1 Dowling Water System.mxd



100 COMMERCE VALLEY DRIVE WEST
THORNHILL, ONTARIO,
CANADA, L3T 0A1
WWW.WSPGROUP.COM

200 BRADY STREET
SUDBURY, ONTARIO,
CANADA, P3A 5P3
WWW.GREATERSUDBURY.CA

LEGEND

W WELL

S STORAGE TANK

BPS BOOSTER PUMPING STATION (BPS)

WATERMAIN

WATERMAIN

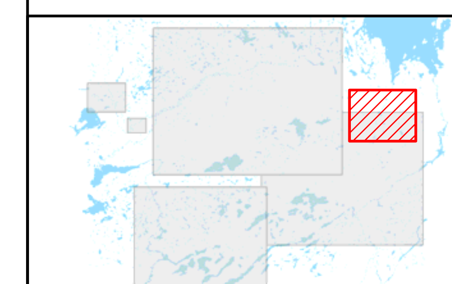
ROAD

WATERBODY

PRESSURE ZONE

FALCONBRIDGE VILLAGE (Not Boosted)

MOTT BOOSTER



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N W E S

PROJECT:
Water and Wastewater Master Plan City of Greater Sudbury

MAP:
FALCONBRIDGE WATER SYSTEM: EXISTING INFRASTRUCTURE

PROJECT NO.:		DATE:	
121-23026-00		MARCH 2017	
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CM	KL	MP	2-3

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200 BRADY STREET
SUDBURY, ONTARIO,
CANADA, P3A 5P3
WWW.GREATERSUDBURY.CA

LEGEND

W WELL

S STORAGE TANK

BPS BOOSTER PUMPING STATION (BPS)

WATERMAIN

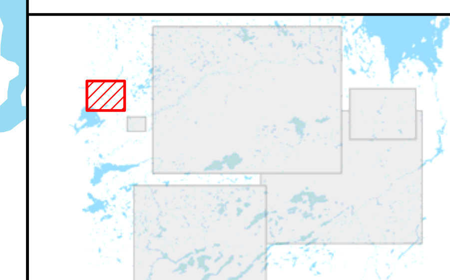
WATERMAIN

ROAD

WATERBODY

PRESSURE ZONE

ONAPING/LEVACK



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PROJECT:
Water and Wastewater
Master Plan
City of Greater Sudbury

MAP:
ONAPING-LEVACK
WATER SYSTEM:
EXISTING INFRASTRUCTURE

PROJECT NO.:		DATE:	
121-23026-00		MARCH 2017	
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CM	KL	MP	2-6

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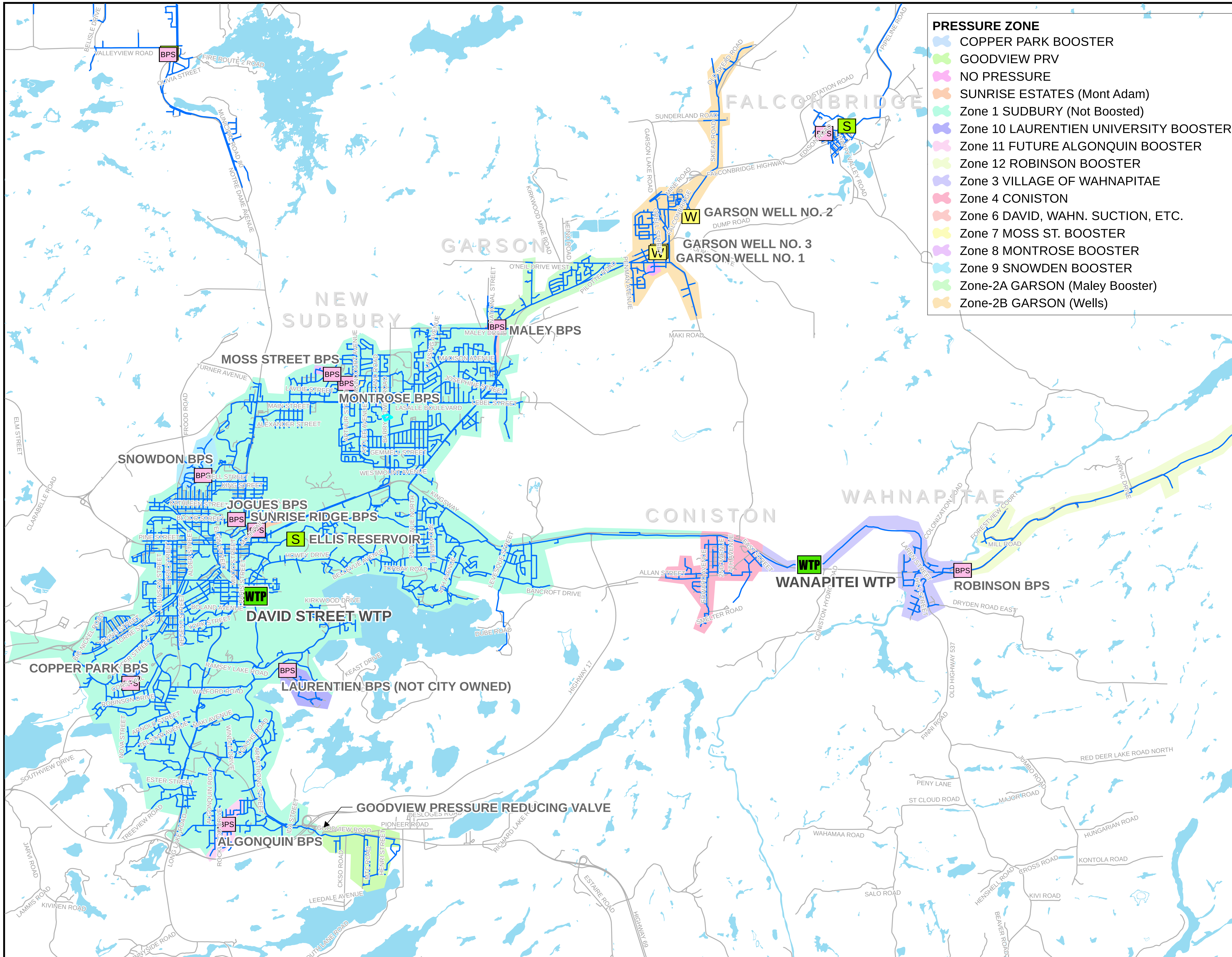

WSP
100 COMMERCE VALLEY DRIVE WEST
THORNHILL, ONTARIO,
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Greater Grand Sudbury
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SUDBURY, ONTARIO,
CANADA, P3A 5P3
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LEGEND

- WTP — WATER TREATMENT PLANT (WTP)
- W — WELL
- S — STORAGE TANK
- BPS — BOOSTER PUMPING STATION (BPS)
- WATERMAIN
- ROAD
- WATERBODY

SCALE:
0 500 1,000 2,000 Meters
1:80,000

PROJECT:
Water and Wastewater Master Plan
City of Greater Sudbury

MAP:
SUDBURY WATER SYSTEM: EXISTING INFRASTRUCTURE

PROJECT NO.:	DATE:
121-23026-00	MARCH 2017

DRAWN BY:	CHECKED BY:	REPORT:	FIGURE NO.:
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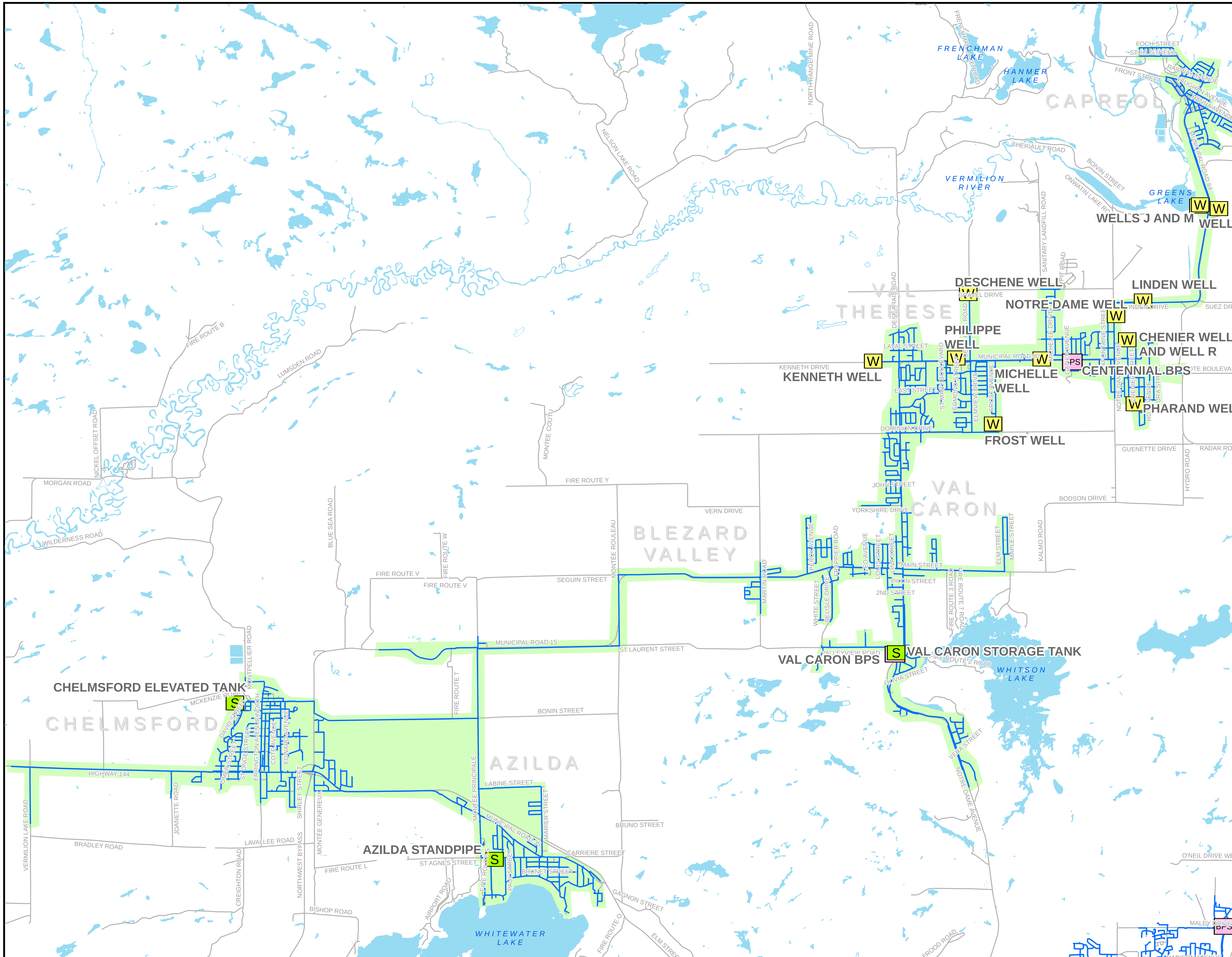

100 COMMERCE VALLEY DRIVE WEST
THORNHILL, ONTARIO,
CANADA, L3T 0A1
WWW.WSPGROUP.COM

Greater Grand Sudbury
200 BRADY STREET
SUDBURY, ONTARIO,
CANADA, P3A 5P3
WWW.GREATERSUDBURY.CA

LEGEND

- S STORAGE TANK
- W WELL
- BPS BOOSTER PUMPING STATION (BPS)
- WATERMAIN
- ROAD
- WATERBODY

PRESSURE ZONE

- VALLEY

SCALE:
0 500 1,000 2,000 Meters
1:80,000

PROJECT:
Water and Wastewater Master Plan
City of Greater Sudbury

MAP:
VALLEY WATER SYSTEM: EXISTING INFRASTRUCTURE

PROJECT NO.:	DATE:
121-23026-00	MARCH 2017

DRAWN BY:	CHECKED BY:	REPORT:	FIGURE NO.:
CM	KL	MP	2-12

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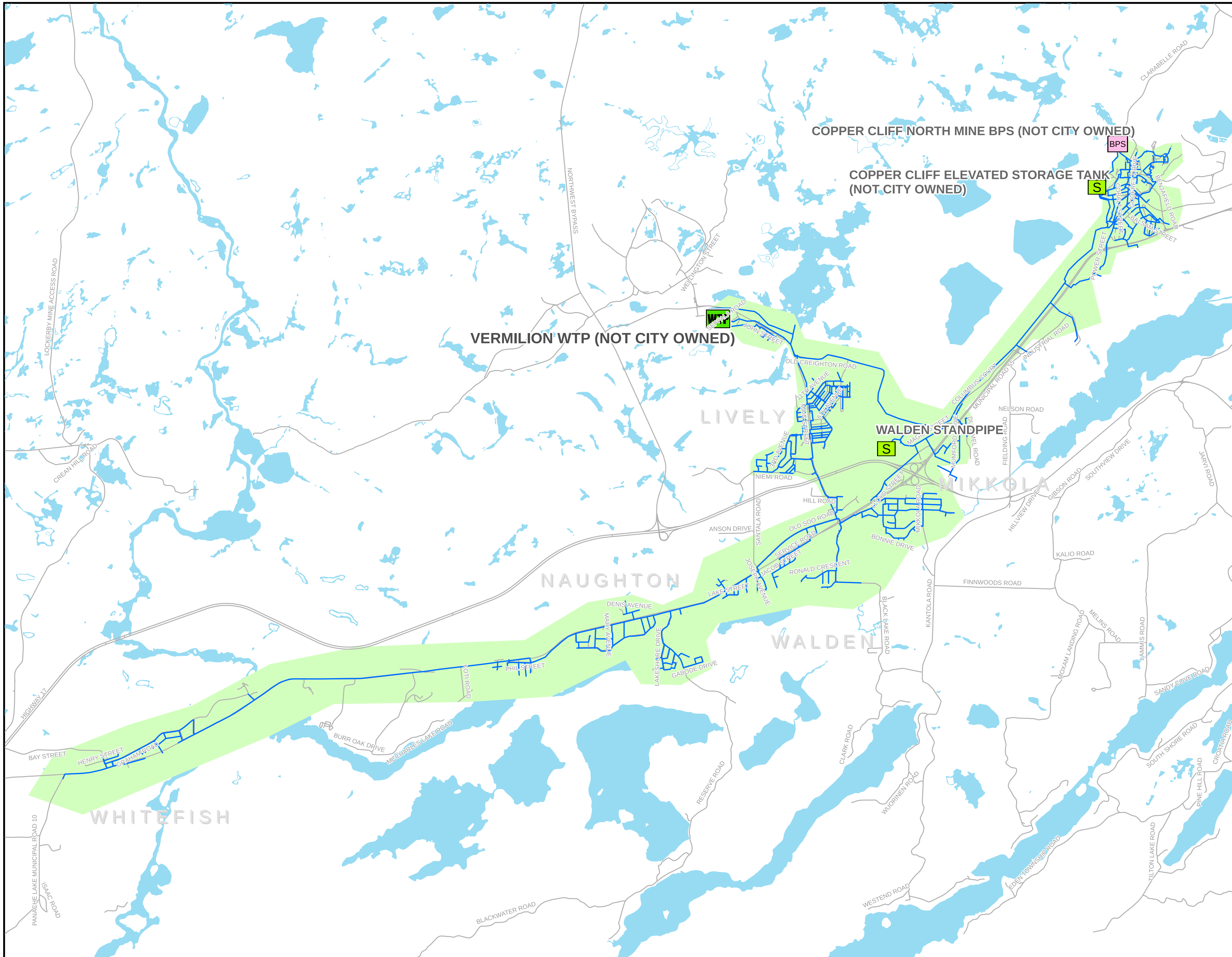

WSP
100 COMMERCE VALLEY DRIVE WEST
THORNHILL, ONTARIO,
CANADA, L3T 0A1
WWW.WSPGROUP.COM

Greater Grand Sudbury
200 BRADY STREET
SUDBURY, ONTARIO,
CANADA, P3A 5P3
WWW.GREATERSUDBURY.CA

LEGEND

- WTP — WATER TREATMENT PLANT (WTP)
- S — STORAGE TANK
- BPS — BOOSTER PUMPING STATION (BPS)
- WATERMAIN
- ROAD
- WATERBODY

PRESSURE ZONE

- VERMILION

SCALE:
0 412.5 825 1,650 Meters
1:65,617

PROJECT:
Water and Wastewater Master Plan City of Greater Sudbury

MAP:
VERMILION WATER SYSTEM: EXISTING INFRASTRUCTURE

PROJECT NO.:	DATE:
121-23026-00	MARCH 2017

DRAWN BY:	CHECKED BY:	REPORT:	FIGURE NO.:
CM	KL	MP	2-15

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APPENDIX 2-A

WATER SYSTEM GAP ANALYSIS AND STATUS QUO REPORTS

CITY OF GREATER SUDBURY WATER AND WASTEWATER MASTER PLAN

DOWLING WATER SYSTEM GAP ANALYSIS AND STATUS QUO REPORT

CITY OF GREATER SUDBURY

DRAFT

PROJECT NO.: 121-23026-00
DATE: NOVEMBER 2016

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APPENDICES

1 INTRODUCTION

The City of Greater Sudbury (CGS) retained WSP to undertake a Water and Wastewater Master Plan. The purpose of the Master Plan project is to establish servicing strategies for water and wastewater infrastructure for the core urban areas and surrounding communities in the City for the next 20 years, as part of the five-year review of the City's Official Plan. The Master Plan will identify potential projects to address the servicing needs for planned growth within the City. It is being conducted in accordance with the requirements set out in the Municipal Class Environmental Assessment (Class EA) document (June 2000 as amended in 2007, 2011, and 2015).

This report includes a capacity review of Dowling's existing water system. Based on population growth projections and design criteria discussed in the *Population and Unit Rates Technical Memorandum* (WSP, 2014), water demand projections were developed and used to determine future infrastructure needs to the 2041 and ultimate buildout planning horizons. This report assumes that the Dowling Water System would continue to be a stand-alone system. Any potential interconnections between Dowling and other systems are not considered as part of this report. Potential interconnections with other communities will be reviewed under separate cover, as part of the Master Plan.

The conclusions provided in this report will be the basis for the problem definition and evaluation of alternatives conducted as part of the Master Plan.

2 STUDY AREA

The Dowling Water System services the community of Dowling, located in the northwest end of Greater Sudbury along Route 144, between the communities of Onaping and Chelmsford.

Map 1 in **Appendix A** shows the Dowling study area and identifies current and future land use and development areas.

The majority of the existing development in Dowling is residential with small pockets of industrial, commercial and institutional areas in the south.

Based on the City's planning data, the majority of future growth within Dowling will be residential, as discussed further in Section 6.1.

3 OVERVIEW OF EXISTING SYSTEM

The Dowling water system is supplied by two wells, the Riverside and Lionel Wells, which are owned and operated by the City of Greater Sudbury. Both wells draw from an unconfined aquifer of sand and gravel deposit located within the Onaping River watershed. Due to the unconfined nature of the soils and proximity to the river, the water source is classified as potentially groundwater under the direct influence of surface water (GUDI).

The same treatment process is in place at both the Lionel and Riverside Wells, as illustrated in Figure 3-1.

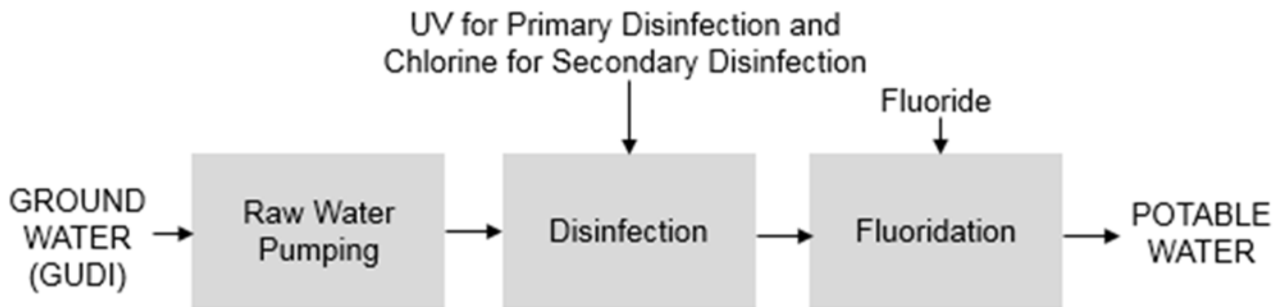

Figure 3-1 Dowling Wells' Process Flow Diagram (Typical for Both Riverside and Lionel Wells)

The rated capacity of the Riverside Well is 3,640 m^3/d (42.1 L/s) at 71.6 m total dynamic head (TDH), while the Lionel Well has a rated capacity of 3,640 m^3/d (42.1 L/s) at 68.6 m TDH (MOECC, 2010). The combined total rated (firm) capacity for Dowling is 3,640 m^3/d, as prescribed in the *Permit to Take Water* (MOECC, 2010) and discussed in the *Baseline Review Report – Water* (WSP, 2014). The Lionel Well also has an on-site standby diesel generator set which supplies emergency power to both well houses.

The distribution system consists of a single pressure zone with one elevated storage tank. Storage in the water distribution system in Dowling is provided by the Dowling Elevated Tank, which has a useable volume of 907 m^3 as calculated based on operating water levels.

Additional information on the existing systems is provided in the Baseline Review Report for Water Systems (WSP, 2014).

4 HISTORICAL FLOWS AND REVIEW OF OPERATIONAL DATA

Well data from 2009 to 2013 was reviewed and analyzed for this evaluation. Table 4-1 shows a summary of the data received, and indicates how it was used for the analysis.

Table 4-1 WTP Data Reviewed

DATA RECEIVED	PARAMETERS INCLUDED	DATA INTERVAL	USE IN ANALYSIS
Treated flow (2011-2013)	Flow in m^3/d	Hourly	To determine peak hourly flow
Annual Reports (2009-2013)	Total average daily flows, maximum daily flows Treated water characteristics	Daily	To determine average day, max day flow To assess performance of existing process and treated water characteristics
Annual Billing Data (2012)	Annual flow per customer in m^3	Annually	To determine the proportion of total water consumption corresponding to residential users

4.1 FLOW DATA

Water supply data from 2009 to 2013 was reviewed to determine historical water demands in Dowling. Average day, maximum day and peak hour demand data for the past five years is included in Table 4-2. It should be noted that peak hour data for the 2009-2010 period was not available. For reference, the combined rated capacity of the wells is 3,640 m^3/d.

Table 4-2 Historical Water Supply Data

YEAR	AVERAGE DAY DEMAND (M^3/D)*	MAXIMUM DAY DEMAND (M^3/D)[1]	PEAK HOUR DEMAND (M^3/D)[2]
2009	392	875	Not Available
2010	399	1,003	Not Available
2011	379	1,207	2,696
2012	382	1,108	3,616
2013	366	1,680	5,191

[1] *Dowling Drinking Water System Annual Reports (2009 – 2013).*

[2] *From hourly SCADA data.*

Data from 2013 indicates slightly lower than typical average day demands, but higher maximum day and peak hour demands. From discussion with City staff, 2013 was an atypically dry year. This may have resulted in more lawn watering, causing higher maximum day and peak hour demands. Based on the above, 2013 was considered an outlier.

A small decline in average day water consumption was noted from the demand values observed in 2009 and 2010 compared to the remaining years. The average consumption for the five year period was 384 m^3/d, or 388 m^3/d if the 2013 data is omitted.

The maximum day flow recorded in the past five years was 1,680 m^3/d, in 2013, which was significantly greater than the maximum day demands in previous years. The next highest maximum day was 1,207 m^3/d occurring in 2011.

Hourly flow data was only available from 2011 to 2012. The maximum peak hour value recorded during that period was 5,191 m^3/d in 2013, followed by 3,616 m^3/d (42 L/s), in 2011.

The historical average and maximum day flow requirements from Table 4-2 are plotted versus the rated combined well capacity in Figure 4-1.

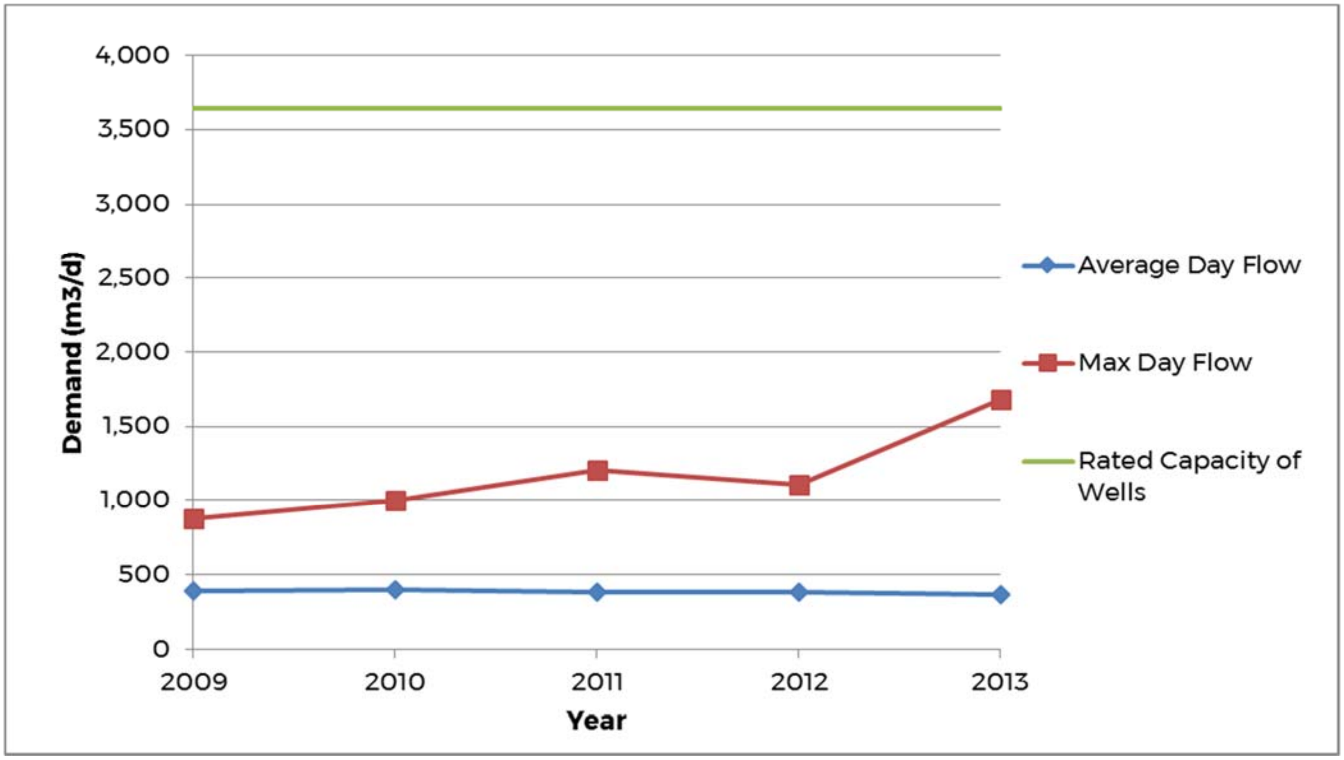

Figure 4-1 Historical Water Demands at the Wells

The peaking factors derived from historical data were compared to those documented in the *City's Engineering Design Manual* (City of Greater Sudbury, 2012) or those included in the *MOECC Guidelines* (MOECC, 2008). The analysis below excludes 2013 data since it was not in line with typical water consumption observed in the remaining years.

The maximum day to average day peaking factor corresponding to the maximum day flow recorded (1,207 m^3/d) was 3.18, while the average maximum day peaking factor was 2.71. The City's Engineering Design Manual specifies a maximum day factor of 2.50 for Dowling, which matches the value recommended in the *MOECC Guidelines* for communities with populations between 1,001 and 2,000, such as Dowling. On further review of the annual data, the maximum day factor is generally increasing. However, in the future, more stringent conservation measures will likely take place. As such, to avoid overestimating future demands, the average maximum day factor (2.71), excluding 2013, was adopted to evaluate future requirements.

The peak hour to average day factor corresponding to the highest peak hour flow recorded in 2012 (3,616 m^3/d) was 9.47, while the average peak hour factor was 8.29. On further review of the peak hour data, it was determined that these are not true peaks. The wells are operated at a high rate continuously for a few hours each day to fill the elevated tank, which then supplies the community. Therefore, the historical peak hour data for Dowling should not be used to calculate a peaking factor.

The *City's Engineering Design Manual* and the *MOECC Guidelines* specify a factor of 3.75. This value was used to estimate future peak flows.

4.2 RAW WATER CHARACTERISTICS

The Dowling wells are sourced from unconfined aquifers and classified as potentially GUDI. Limited data on the raw water characteristics was available for our review. In 2010, the Riverside Well highest sodium concentration was 35.90 mg/L. The raw water turbidity at each well ranged from 0.02 to 2.0 NTU (City of Greater Sudbury, 2009-2013).

4.3 OPERATIONAL DATA

Data reported in the *Annual Reports* for the Dowling Wells includes effluent chlorine residual, trihalomethanes (THMs), fluoride, and trace organic and inorganic chemicals such as arsenic.

Data was reviewed from 2009 to 2013 to determine any historical issues at the wells. No major issues were observed, except for elevated sodium levels at the Riverside Well in 2010, as noted above.

5 DESIGN CRITERIA

The following design criteria were used to assess the remaining capacity of the existing systems and to forecast future requirements for the water and wastewater systems. The unit rates used to estimate future water and wastewater flows correspond to the values included in the *Population Projections and Unit Rates Technical Memorandum* (WSP, 2014). Otherwise, design criteria recommended in the *MOECC Guidelines* and *City's Engineering Design Manual* were used.

5.1 UNIT WATER DEMAND CRITERIA

The water demand criteria shown in Table 5-1 are from the unit rates recommended in the *Populations and Unit Rates Technical Memorandum* (WSP, 2014). The rates were reviewed against historical data, MOECC *Guidelines* (MOECC, 2008), and current standards in the City's *Engineering Design Manual* (City of Greater Sudbury, 2012).

Both the *MOECC Guidelines* and *City Engineering Design Manual* recommend determining demands for institutional, commercial and industrial (ICI) users on a case by case basis. However, the following criteria for ICI demands were used for the purposes of this evaluation.

Table 5-1 Water System Design Criteria

CRITERIA	VALUE	REFERENCE
Average Day Residential Demand	200 L/cap/day	Rounded up average of historical values
Average Day Institutional & Commercial Demand	28 m^3/ha/d	MOECC Guidelines
Average Industrial Demand	35 m^3/ha/d	MOECC Guidelines
Domestic Demand Maximum Day Factor	2.71	Average of historical values
Domestic Demand Peak Hour Factor	3.75	MOECC Guidelines

Residential average day demands are obtained by multiplying the residential unit rate by the service population. Similarly, average ICI demands are obtained by multiplying the corresponding unit rates to the areas of development, assuming 100% of the area would be developed.

Maximum day and peak hour demands are obtained by multiplying the average day demand by the corresponding peaking factor.

Fire flow criteria are determined based on Fire Underwriters Survey (FUS) requirements. For the purposes of this evaluation a fire flow requirement of 4,500 L/min (75 L/s) is adopted for residential areas (City of Greater Sudbury, 2012) and 9,000 L/min (150 L/s) for ICI.

5.2 DESIGN CRITERIA FOR WATER SYSTEM COMPONENTS AND OPERATION

5.2.1 TREATMENT CAPACITY

Water supply facilities are designed to supply the maximum day demands of the system.

Treatment facilities must be designed in accordance with the *Procedure for Disinfection of Drinking Water in Ontario* (Ontario, 2006). Drinking water treatment systems that obtain water from a surface water or GUDI supply must achieve an overall performance providing as a minimum a 2-log (99%) removal or inactivation of *Cryptosporidium* oocysts, 3-log (99.9%) removal or inactivation of *Giardia* cysts, and 4-log (99.99%) removal or inactivation of viruses.

At least 0.5-log removal or inactivation of *Giardia* cysts and 2-log removal or inactivation of viruses must be provided through disinfection, while the remaining removal may be achieved through filtration or other equivalent treatment processes.

5.2.2 PUMPING CAPACITY

Pumping stations are rated based on their firm capacity. If sufficient floating storage is available in a particular pressure district, the MOECC defines firm capacity as the capacity of the station with the largest pump out of service. If there is insufficient or no floating storage, firm capacity is defined as the capacity with the two (2) largest pumps out of service (MOECC, 2008).

For each pressure district, the pumping stations have to be designed to provide peak hour or maximum day plus fire demands (whichever are greater), if no floating storage is available. If sufficient floating storage is available, then the pumping station only needs to be designed to provide maximum day demands.

The Dowling system consists of a single pressure district and is pressurized by the well pumps and elevated storage tank. Currently, the wells are not used as booster pumps and only operate for a few hours each day. Therefore, this system does not require pumping capacity for zone pressurization.

5.2.3 STORAGE CAPACITY

Storage requirements are based on the requirement to meet water demands that exceed the capacity of the treatment plant and to satisfy fire flow demands. When the capacity of the supply system is only capable of satisfying maximum day demands, storage requirements are determined using the following formula from the *MOECC Guidelines* (MOECC, 2008):

$$\text{Storage} = \text{A} + \text{B} + \text{C}$$

Where: A = Fire Storage, B = Equalization Storage = 25% of maximum day demand, and C = emergency storage = 25% of (A+B).

Fire storage is the product of the maximum fire flow required in the system and the corresponding fire duration based on Fire Underwriters requirements (Fire Underwriters Survey, 1999).

When the system can supply more than just the maximum day demand (but less than the peak demand), the fire storage requirements can be determined using the following formula:

$$\mathbf{A} = (\mathbf{Peak\ Demand} - \mathbf{Pumping\ Station\ Firm\ Capacity}) \times \mathbf{Fire\ Duration}$$

Where: peak demand is the greater of the peak hour demand and the maximum day plus fire demand.

Per *MOECC Guidelines*, floating storage should be designed such that the elevation of the equalization volume (B) is such that a minimum pressure of 275 kPa (40 psi) can be maintained in the system under peak hour flow conditions. The fire (A) and emergency (C) volumes should be at elevations that produce 275 kPa (40 psi) during peak hour demand conditions, and 140 kPa (20 psi) under the maximum day plus fire flow condition (MOECC, 2008).

5.2.4 DISTRIBUTION CAPACITY

Watermains have to be sized to carry the greater of the maximum day plus fire flow or peak hour demand. The range of acceptable pressures under normal conditions (average to peak hour flows) is 275 kPa (40 psi) to 690 kPa (100 psi), while during fire flow conditions pressures may drop to 140 kPa (20 psi) (MOECC, 2008). The maximum allowable water velocity in the distribution system is 3 m/s (MOECC, 2008).

6 FUTURE REQUIREMENTS

6.1 POPULATION PROJECTIONS

As part of the City of Greater Sudbury Master Plan, population forecasts were developed for the 2016, 2021, 2026, 2031, 2036, 2041 and Ultimate Buildout planning years. Ultimate Buildout is defined as an estimate of what the demand from the total population and total number of households in the City of Greater Sudbury would be based on lands that are currently designated for development in the Official Plan within the existing settlement boundaries.

The City supplied planning data sheets with properties and development potential and the vacant residential and ICI land inventory, and Hemson Consultants, on behalf of the City, provided supplementary population projections. Data was provided for each water system boundary. These data were used in conjunction to develop the targeted population growth for each horizon year, as well as development phasing (discussed in the next section and in detail in the *Populations and Unit Rates Technical Memorandum*, WSP 2014).

Based on the planning data, the population of Dowling is projected to increase by 244 people by 2041 and 1,948 by Ultimate Buildout. The population projections to be used in the Master Plan are summarized in Table 6-1.

Table 6-1 Dowling Water System Population Projections

YEAR	2011	2016	2021	2026	2031	2036	2041	BUILDOUT
Population	1,773	1,837	1,903	1,965	1,997	2,017	2,016	3,721

The City's planning data does not specify target years for employment growth. However, vacant lands designated as ICI properties have been assigned to different stages of the development process by the City. These stages are described below and apply to both ICI and residential areas.

— Draft Approved:
  — These are lands that have draft plan of subdivision approval under the Planning Act or have pending applications with the City. Typically, these lands are close to registration or few years away from development as the required conditions are satisfied
  — Development approvals are near complete, and development could take place at any time. Properties with this designation were set to take place in 2016.
— Legal Lots of Record:
  — These are existing lots, including lots in a registered plan of subdivision. Typically these lands are zoned, serviceable and only require building permit approval for development. In some cases a site plan approval/agreement may also be required.
  — Based on historical trends, development is approximately 15 years away from receiving draft approval. Properties with these designations were assigned to take place in 2026.
— Designated Developable:
  — These lands do not have any development approvals in place but are understood to be areas of future development as they are within the settlement boundary. Designated lands are typically a number of years away from being developed.
  — Based on historical trends, these properties are approximately 10 years away from receiving Legal Lot of Record designation. Designated Developable properties were assumed to take place in 2036.

These land supply categories stem from the land supply requirements that municipalities must maintain under Section 1.4 of the Provincial Policy Statement. In this context, Designated Development Lands would count towards Section 1.4.1 (a) and Legal Lots of record and Draft Approved Lands would count towards 1.4.1 (b). It is also important to note that the total supply is governed by PPS Section 1.1.2.

The targeted ICI development areas for each horizon year are summarized in Table 6-2.

Table 6-2 Dowling Water System Population Projections

	ICI DEVELOPMENT AREAS (HA)						
LAND USE	2016	2021	2026	2031	2036	2041	BUILDOUT
Institutional	0	0	0.47	0	0	0	0
Commercial	0	0	0.64	0	0	0	0
Industrial	0	0	0	0	0	0	0
Total	**0**	**0**	**1.11**	**0**	**0**	**0**	**0**

The above assumptions provide an estimate as to the ICI development time line. In reality, development may be more staggered. However, for purposes of infrastructure planning and to ensure that the appropriate infrastructure is in place by the appropriate planning horizon, the above assumptions are considered to be conservative.

6.2 PRIORITY EXTENSION LIST

The City has developed and maintained a Priority Extension List of existing residential and ICI streets that are not currently serviced by either or both municipal water or sewer, but at least one owner on the street has requested servicing. The City's policy on extension of services includes the following conditions:

- Before any project proceeds, the participation rate of benefitting property owners must be 100%, with those benefitting property owners funding 50% of the actual net cost of the project.
- The process must be initiated by property owners submitting a petition to the City of Greater Sudbury.
- At least 80% of the property owners in the project area must sign the petition.
- The project must be on the City's priority list for new servicing schemes, or, there must be demonstrated cause why the project should be included on the City's priority list for new servicing schemes.

In Dowling, one street has been placed on the priority list for sewer servicing. However, to date, the above conditions have not been met and City funding for extension requests is not available. Therefore, this street has not been included in the demand projections for infrastructure planning as part of the Master Plan.

6.3 PHASING OF FUTURE GROWTH

Growth areas were allocated based on population projections for individual developments and the overall target growth population projections for Dowling for the horizon years.

Hemson's supplementary tables were used to provide the target population, while the City's planning tables and vacant lot inventory were used to identify phasing of specific properties, and assignment of draft approved, legal lots of record, and designated development properties. In general, priority was given to draft approved properties, followed by legal lots of record and designated developable properties. In accordance with the Official Plan, the City has also assigned a target quantity of legal lots of record and designated developable properties to be developed in each horizon year. That is, legal lots of record should account for approximately 20% of all household growth, and designated developable lots are assigned 20% of the 20 year anticipated growth.

The future growth phasing plans are presented as Maps 4 to 10 in **Appendix A** at the end of this report.

6.4 FUTURE WATER DEMAND PROJECTIONS AND INFRASTRUCTURE NEEDS

The unit flow criteria indicated in Section 5.1 were used to estimate the future water demands in Dowling. In general, the projected flows were calculated by the following formula:

$$\textbf{Projected Average Day Demand} \\ = \textbf{Base Demand} + \textbf{Additional Residential Demand} + \textbf{Additional ICI Demand}$$

The demands corresponding to the population growth forecasts to ultimate buildout are listed in Table 6-3.

Table 6-3 Water Demand Projections

YEAR	POPULATION	AVERAGE DAY DEMAND (M^3/D)	MAXIMUM DAY DEMAND (M^3/D)	PEAK HOUR DEMAND (M^3/D)
Base	1,773	388	1,048	1,455[1]
2016	1,837	401	1,085	1,503
2021	1,903	414	1,121	1,553
2026	1,965	458	1,239	1,716
2031	1,997	464	1,257	1,740
2036	2,017	468	1,267	1,755
2041	2,016	468	1,267	1,754
Ultimate Buildout	3,721	809	2,190	3,033

[1] Historical peak hour demand for the base year was not available. Therefore, it was estimated by multiplying the base year average day demand by the peak hour factor (3.75).

The Base Demand was the average historical (2009 to 2013) average day demand for the community. The additional residential demand was calculated using the unit flow rate multiplied by the population growth, and similarly, the ICI demand was calculated using the unit flow rate for each type of development (industrial, commercial or institutional), multiplied by the growth in development area.

Maximum day and peak hour demands were calculated by applying the respective peaking factor to the average day demand.

A desktop analysis of historical water demands and future water demand projections is included in **Appendix B**.

6.4.1 SUPPLY CAPACITY

The rated combined capacity of the Dowling Wells is 3,640 m^3/d. Thus, Dowling has sufficient capacity to service the population growth to Ultimate Buildout. It is therefore very important to assess the true capacity of the wells to determine whether they can reliably produce flows equal to the rated capacity.

The projected maximum day demands are plotted versus the capacity of the well supply on Figure 6-1.

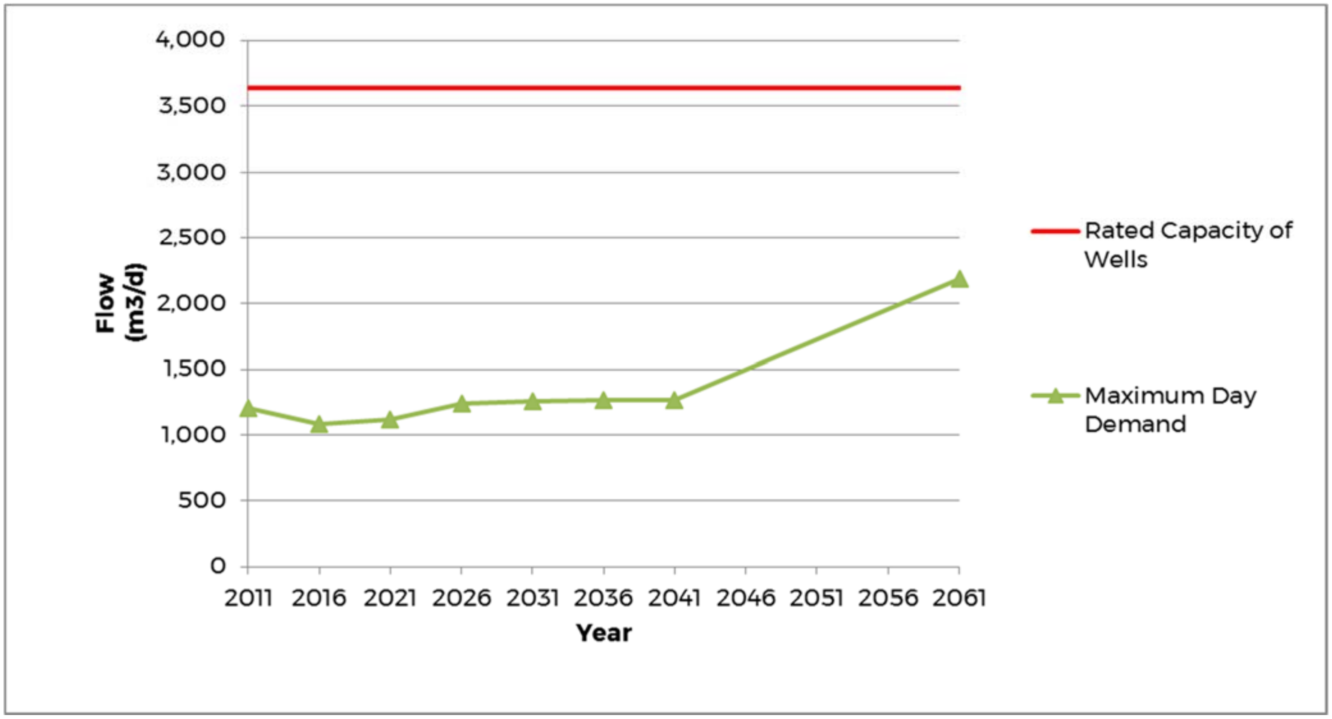

Figure 6-1 Water Demand Projections vs. Rated Capacity of the Wells

6.4.2 STORAGE CAPACITY

Storage in the distribution system is provided by an elevated tank with a useable volume of 907 m^3 as calculated based on operating levels. The storage assessment for the Dowling Water System follows the procedure described in Section 5.2 for systems where supply exceeds the maximum day demand, and in this case, also exceeds peak hour demands. The MOE A+B+C calculation does not apply.

The Dowling Water System can supply 3,640 m^3/d, which is more than maximum day demand in 2041 and at Ultimate Buildout (1,267 and 2,190 m^3/d, respectively), and more than the projected peak hour demand (1,754 and 3,033 m^3/d, respectively). Therefore, the fire flow requirements and peak hour demands can be met from a combination of the available storage volume and direct pumping from the wells.

Therefore, no additional storage is required in Dowling.

6.4.3 DISTRIBUTION NETWORK

The water model was used to identify system elements (i.e. watermains, pumps, storage tank) for which the capacity was exceeded by the projected water demands. The capacity of the system was assessed in terms of the available fire flows and system pressures.

For each planning scenario, watermains of the modelled network were reviewed to assess whether the required minimum fire flows (75 L/s in residential areas or 150 L/s in ICI areas) and pressures (over 20 psi under fire conditions and over 40 psi under normal conditions) were achieved. Furthermore, some new watermains were added to service greenfield areas in the south and east. A simplified, looped, watermain layout was assumed for these areas. In addition, watermains were added to existing areas without watermains in the model, but with meter data.

Future population and demands were loaded into the model based on the planning data and flow projections discussed in Section 7. Development in Dowling might deviate from this phasing scheme. Thus, it is recommended that the hydraulic water model be updated whenever a development application is submitted.

The findings from the water modeling are discussed in Section 7.

7 HYDRAULIC WATER SYSTEM MODELLING

An all-pipe model of the system including pipes, hydrants, storage tanks and system source was developed by the City using Bentley Systems' WaterGEMS hydraulic modeling software. This model was updated based on information provided by the City to reflect current system conditions.

The water model allows for simulations to be conducted that can be used to predict system responses to events under a wide range of conditions. Using simulations, problems can be anticipated in proposed or existing systems, and solutions can be evaluated before time, money, and materials are invested in a real-world project. Simulations can either be steady-state or extended-period. Steady-state simulations represent a snapshot in time and are used to determine the operating behaviour of a system under static conditions. This type of analysis can be useful in determining the short-term effect of fire flows or average demand conditions on the system. Extended period simulations (EPS) are used to evaluate system performance over time. This type of analysis allows modeling the filling and emptying of storage facilities, regulating valves opening and closing, and pressures and flow rates changing throughout the system in response to varying demand conditions and automatic control strategies.

Simulations including steady-state analysis of the Average Day, Maximum Day and Maximum Day + Fire conditions were carried out using the model. Fire flow simulations were carried out throughout the system to determine whether the system could deliver fire flows under the Maximum Day demands.

7.1 MODEL DEVELOPMENT

To model the current scenario, the following steps were taken:

- Total network demand on an average day basis was determined for the current scenario using 2012 water production data.
- The node demand allocations assigned in the model were based on 2012 meter records. Metered flows were assigned to the respective property. In cases where meter records showed zero flow, the value was manually adjusted to reflect a reasonable volume for a respective property, depending on land use.
- The maximum day peaking factor discussed in Table 4-1 were applied to the average day demand value to determine the maximum day demand.
- The maximum day demand plus fire flow was used to assess the system since it was greater than the peak hour demand.
- The model predictions were compared to real world hydrant flow test results at select locations, showing an overall agreement within 5.4%.

7.2 MODELING FINDINGS

7.2.1 FIREFIGHTING CAPACITY

An assessment of the available fire flows was conducted using the hydraulic model. As noted above, a fire flow requirement of 150 L/s was adopted for ICI while a value of 75 L/s was adopted for residential areas. Water model outputs, including maps showing fire flow analysis, are provided in **Appendix C**.

7.2.2 MODELED HYDRAULIC CAPACITY UNDER NORMAL CONDITIONS

Based on the system modeling, service pressures throughout the system under the maximum day demand scenario generally range between 60 and 80 psi (414 and 551 kPa) for all horizon years. Therefore, flows throughout the system are within the range prescribed in the MOECC Guidelines (40 to 100 psi) under normal conditions.

Maps showing pressures at nodes are presented in **Appendix C**.

8 CONCLUSIONS

An assessment of the Dowling water system was completed to identify infrastructure investment requirements to service forecasted growth in the community. The assessment involved a review of previous studies, an analysis of operations and flow data, and an evaluation of the capacity of the system.

The conclusions of the assessment are summarized below.

- Based on the rated capacity as well as historical and projected demands, the wells have sufficient capacity to service growth beyond 2041.
- No additional storage is required. Fire flow requirements and peak demands can be met through a combination of storage and direct pumping from the wells.
- Hydraulic modeling identified that some areas may not be able to deliver fire flows per current standards.

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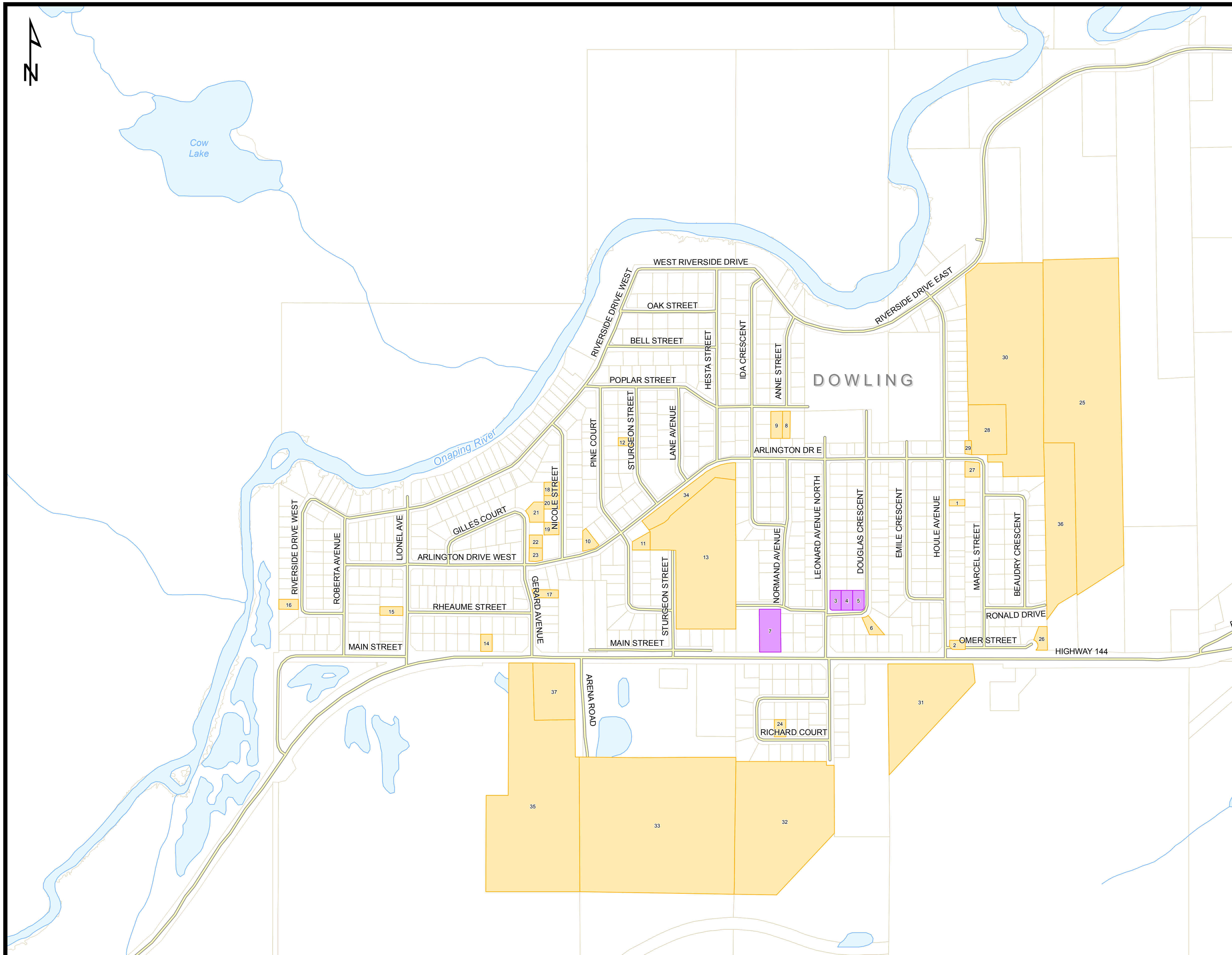
- WSP. (2014). Technical Memo No. 1: Population Projections and Unit Rate Development.

APPENDIX

A RESIDENTIAL AND ICI DEVELOPMENT AREAS



Legend

- Road
- Watercourse
- Waterbody
- Parcel
- Vacant ICI
- Vacant Residential

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Map 7

Dowling
2011 - Ultimate Buildout
Residential and ICI
Development Areas

APPENDIX

B WATER DEMAND AND CAPACITY ASSESSMENTS

DATA ANALYSIS

	Units	2009	2010	2011	2012	2013 (Omitted - Outlier)	Summary	Design Criterion	Comments
Average Day Flow	m^3/d	392	399	379	382	366	388	388	From Water Historical Production data. The daily production values for each facility was added together to determine the total daily production. 2013 was excluded from the set since the year was unusually dry and resulted in atypical demands.
Max Day Flow	m^3/d	875	1,003	1,207	1,108	1,680	1,048	1,048	The maximum value of the sum of water production for each facility was used. 2013 was excluded from the set since the year was unusually dry and resulted in atypical demands.
Max Day Factor		2.24	2.51	3.18	2.90	4.59	2.71	2.71	MOE Guidelines recommend a value of 2.50 for populations between 1,001 and 2,000. The maximum value over the past five years was 4.59, which was higher than previous years. Because there is no clear trend, it is suggested to adopt the maximum. 2013 was excluded from the set since the year was unusually dry and resulted in atypical demands.
Peak Hour (L/s)	L/s	Not Available		31	42	60	N/A		Peak values were available only for 2011-2013. 2013 was excluded from the set since the year was unusually dry and resulted in atypical demands.
Peak Hour (m^3/d)	m^3/d			2,696	3,616	5,191	N/A		
Peak Hour Factor				7.11	9.47	14.19	8.29	3.75	MOE Guidelines recommend a value of 3.75 for populations between 1,001 and 2,000. The historical peak hour data is artificially high since the City operates Dowling Wells to fill the storage tank, and not necessarily just to direct-pump into the system. The hourly data shows that the pumps are only run for a few hours each day. Therefore, this data should not be used to calculate a peaking factor
Population (Existing Areas)		1,773	1,773	1,773	1,773	1,773		1,773	
Ratio of Residential to Total Water Billed		0.82	0.82	0.82	0.82	0.82	0.818		Estimated amount of water consumption related to ICI based on metering data and obtained ratio of residential to total consumption.
Residential Flow (m^3/d)		320	326	310	312	299	314		Calculated based on ratio of residential consumption to total consumption.
Ratio of ICI to Total Water Billed		0.18	0.18	0.18	0.18	0.18	0.182		
		71	73	69	70	67	70		
Per Capita Residential Demand (m^3/cap/day)		0.181	0.184	0.175	0.176	0.169	0.177	0.200	Took average over 2009 to 2013 period, and rounded up.
Average Institutional Flow Unit Rate (m^3/ha/d)								28.0	*MOE Guidelines recommend a value of 28 m^3/ha/d.*
Average Commercial Flow Unit Rate (m^3/ha/d)								28.0	*MOE Guidelines recommend a value of 28 m^3/ha/d.*
Average Industrial Flow Unit Rate (m^3/ha/d)								35.0	*MOE Guidelines recommend a value of 35 m^3/ha/d for light industry and 55 m^3/ha/d for heavy industry.*

	2016	2021	2026	2031	2036	2041	Ultimate	Comments
Population (Existing Areas)	1,773	1,773	1,773	1,773	1,773	1,773	1,773	From data provided by Hemson grouped by water system.
Population Growth	64	130	193	225	244	244	1,949	
Total Population	1,837	1,903	1,965	1,997	2,017	2,016	3,721	From data provided by Hemson grouped by water system.
Residential Growth Area (ha)	12.96	9.01	0.00	3.44	0.00	0.00	53.10	From City's GIS database. 2036 and 2041 areas are included with 2031.
Residential Growth Area (ha) - Cumulative	12.96	21.97	21.97	25.41	25.41	25.41	78.51	
Institutional Growth Area (ha)	0.00	0.00	0.47	0.00	0.00	0.00	0.00	
Institutional Growth Area (ha) - Cumulative	0.00	0.00	0.47	0.47	0.47	0.47	0.47	From City's GIS database.
Commercial Growth Area (ha)	0.00	0.00	0.64	0.00	0.00	0.00	0.00	
Commercial Growth Area (ha) - Cumulative	0.00	0.00	0.64	0.64	0.64	0.64	0.64	From City's GIS database.
Industrial Growth Area (ha)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	
Industrial Growth Area (ha) - Cumulative	0.00	0.00	0.00	0.00	0.00	0.00	0.00	From City's GIS database.
ICI (ha) - Cumulative	0.00	0.00	1.11	1.11	1.11	1.11	1.11	Sum of Institutional, Commercial and Industrial areas
Total Growth Area (ha) - Cumulative	12.96	21.97	23.08	26.52	26.52	26.52	79.62	

	2016	2021	2026	2031	2036	2041	Ultimate
Average Residential and ICI Flows (m^3/d) - Existing	388	388	388	388	388	388	388
Average Residential Flows (m^3/d) - Growth	13	26	39	45	49	49	390
Average Residential Flows (m^3/d) - Total	401	414	427	433	437	437	778
Average Institutional Flow (m^3/d)	0	0	13	13	13	13	13
Average Commercial Flow (m^3/d)	0	0	18	18	18	18	18
Average Industrial Flow (m^3/d)	0	0	0	0	0	0	0
Average ICI Flow (m^3/d)	0	0	31	31	31	31	31
Average Day Flow (m^3/d)	401	414	458	464	468	468	809
Max Day Flow (m^3/d)	1,085	1,121	1,239	1,257	1,267	1,267	2,190
Peak Hour Flow (m^3/d)	1,503	1,553	1,716	1,740	1,755	1,754	3,033

ALTERNATIVE CALCULATION METHOD

Per Capita Demand (m^3/cap/day)	0.221	0.225	0.214	0.215	0.206	0.216
Max Day Factor						4.59
Peak Hour Factor						3.75

If ICI is not considered explicitly and demand is divided by total population.
The historical per capita consumption is applied for future development.

	2016	2021	2026	2031	2036	2041	Ultimate
Average Day Flow (m^3/d)	397	412	425	432	436	436	805
Max Day Flow (m^3/d)	1,824	1,890	1,952	1,984	2,003	2,003	3,696
Peak Hour Flow (m^3/d)	1,490	1,544	1,595	1,621	1,636	1,636	3,020

SENSITIVITY ANALYSIS

Analyze sensitivity of forecasted average day flows to unit rate

	Unit Rate (m^3/cap/d)	Average Day Flow (m^3/d) 2016	2021	2026	2031	2036	2041	Ultimate
Using a consolidated per capita flow		397	412	425	432	436	436	805
Using estimated average	0.200	401	414	458	464	468	468	809
City Standards	0.41	414	441	498	511	519	519	1218

Analyze sensitivity of forecasted flows to max day peaking factor

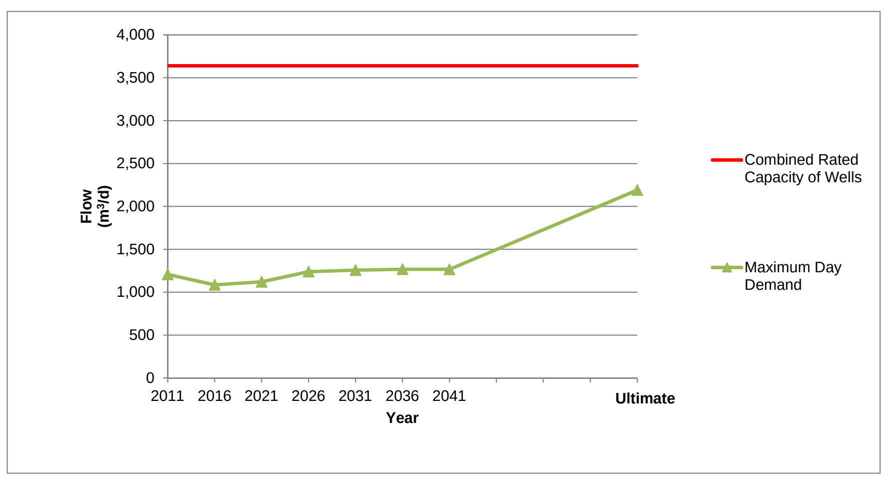
	Max Day Peaking Factor	Max Day Flow (m^3/d) 2016	2021	2026	2031	2036	2041	Ultimate
2009-2013 average of peaking factors	3.08	1,236	1,277	1,412	1,431	1,443	1,443	2,495
2012 peaking factor (maximum historical)	4.59	1,085	1,121	1,239	1,257	1,267	1,267	2,190
MOE Guidelines	2.50	1002	1035	1144	1160	1170	1170	2022

Analyze sensitivity of forecasted flows to peak hour factor

	Peak Hour Peaking Factor	Peak Hour (m^3/d) 2016	2021	2026	2031	2036	2041	Ultimate
MOE Guidelines	3.75	1,503	1,553	1,716	1,740	1,755	1,754	3,033

CAPACITY CHECK

	2011	2016	2021	2026	2031	2036	2041	Ultimate
Combined Rated Capacity of Wells	3,640	3,640	3,640	3,640	3,640	3,640	3,640	3,640
Actual Capacity of Wells	3,640	3,640	3,640	3,640	3,640	3,640	3,640	3,640
Maximum Day Demand	1,207	1,085	1,121	1,239	1,257	1,267	1,267	2,190
Peak Hour Demand		1,503	1,553	1,716	1,740	1,755	1,754	3,033



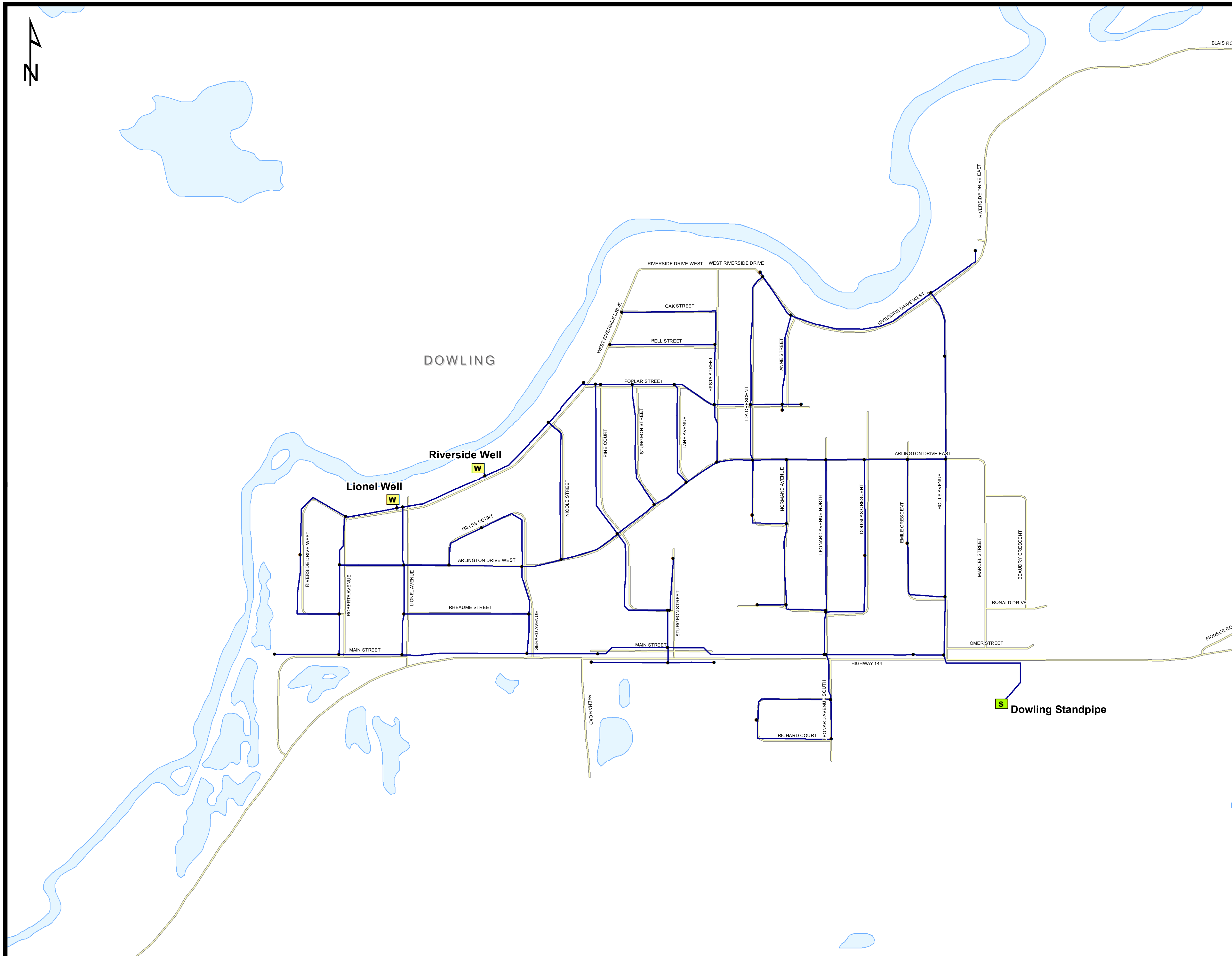
STORAGE REQUIREMENTS

Storage Available		
Elevated Tank (m^3)	907	Calculated from operating levels.
Total Storage (m^3)	907	
Maximum Fire flow Requirements (L/s)	150	
Fire Duration (hrs)	2	
Minimum Fire Flow Requirement for Residential Areas (L/s)	75	From CGS Engineering Design Manual
Fire Duration (hrs)	1.75	From Fire Underwriters Survey Requirements corresponding to 75 L/s

MOE A + B + C Calculation is not applicable because there is available pumping capacity.

APPENDIX

C WATER MODEL RESULTS



Legend

- W — Well
- S — Storage Tank
- BPS — Booster Pumping Station (BPS)
- WTP — Water Treatment Plant (WTP)
- Pipe
- Road
- Waterbody

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Sudbury Water and Wastewater Master Plan

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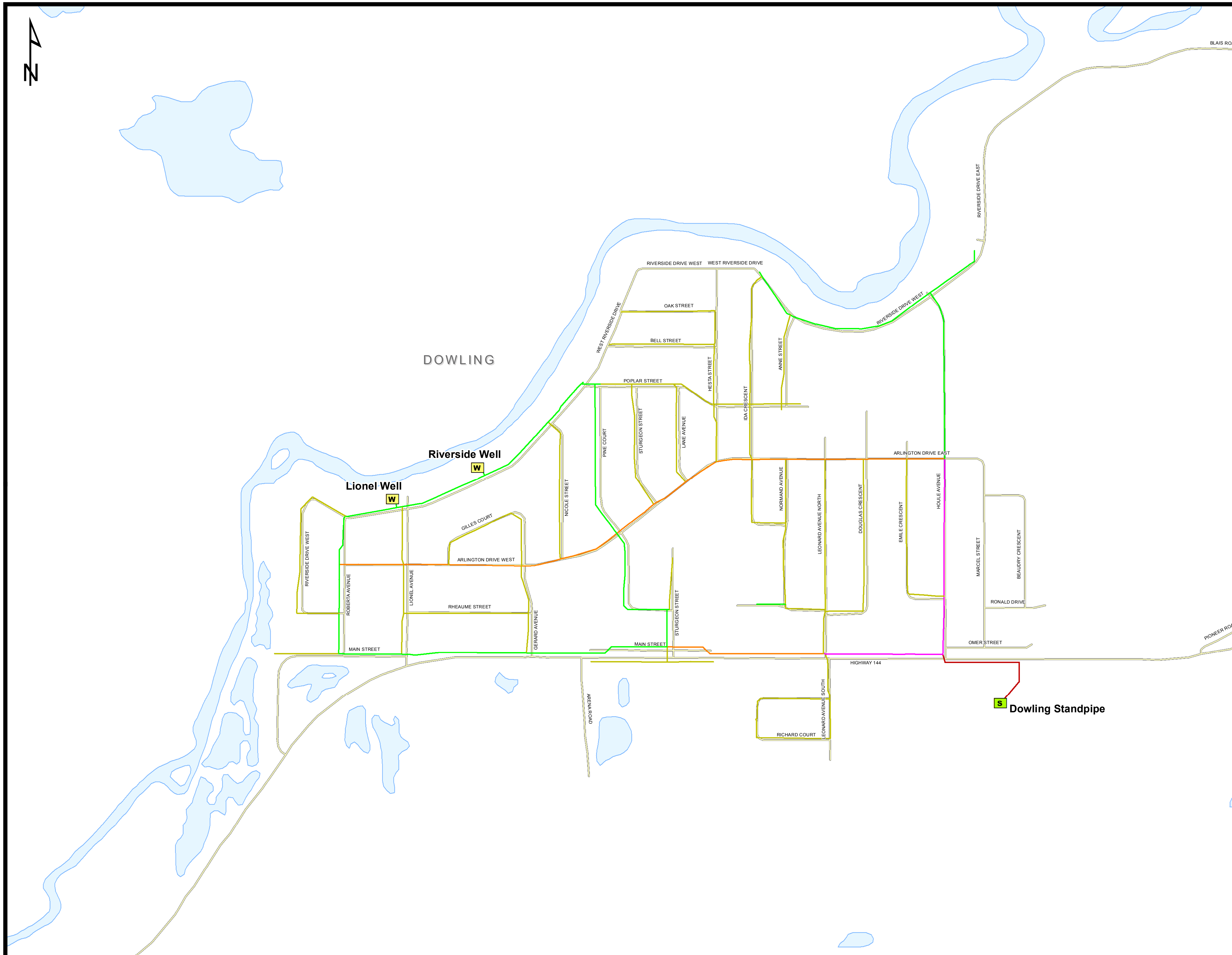
Map 1

Dowling Water System - Key Plan



Legend

- W Well
- S Storage Tank
- BPS Booster Pumping Station (BPS)
- WTP Water Treatment Plant (WTP)
- Road
- Waterbody

Pipe: Diamater (mm)

- <= 150
- <= 200
- <= 250
- <= 300
- <= 400
- <= 600
- > 600

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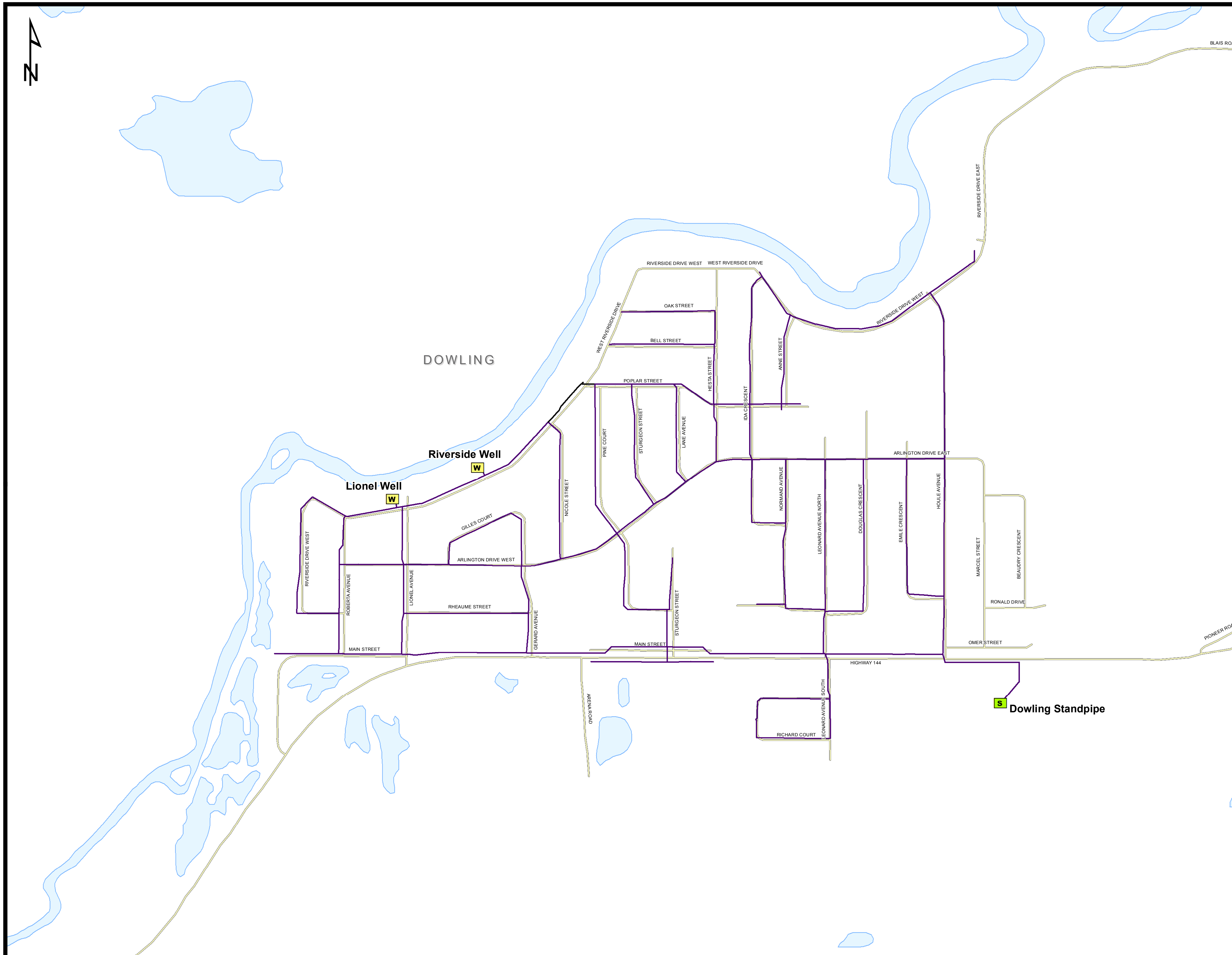
Map 2

Dowling Water System - Pipe Diameter



Legend

W Well

S Storage Tank

BPS Booster Pumping Station (BPS)

WTP Water Treatment Plant (WTP)

Road

Waterbody

Material

Asbestos Cement

Cast iron

Concrete

Copper

Ductile Iron

Galvanized iron

PVC

Polyethylene

Steel

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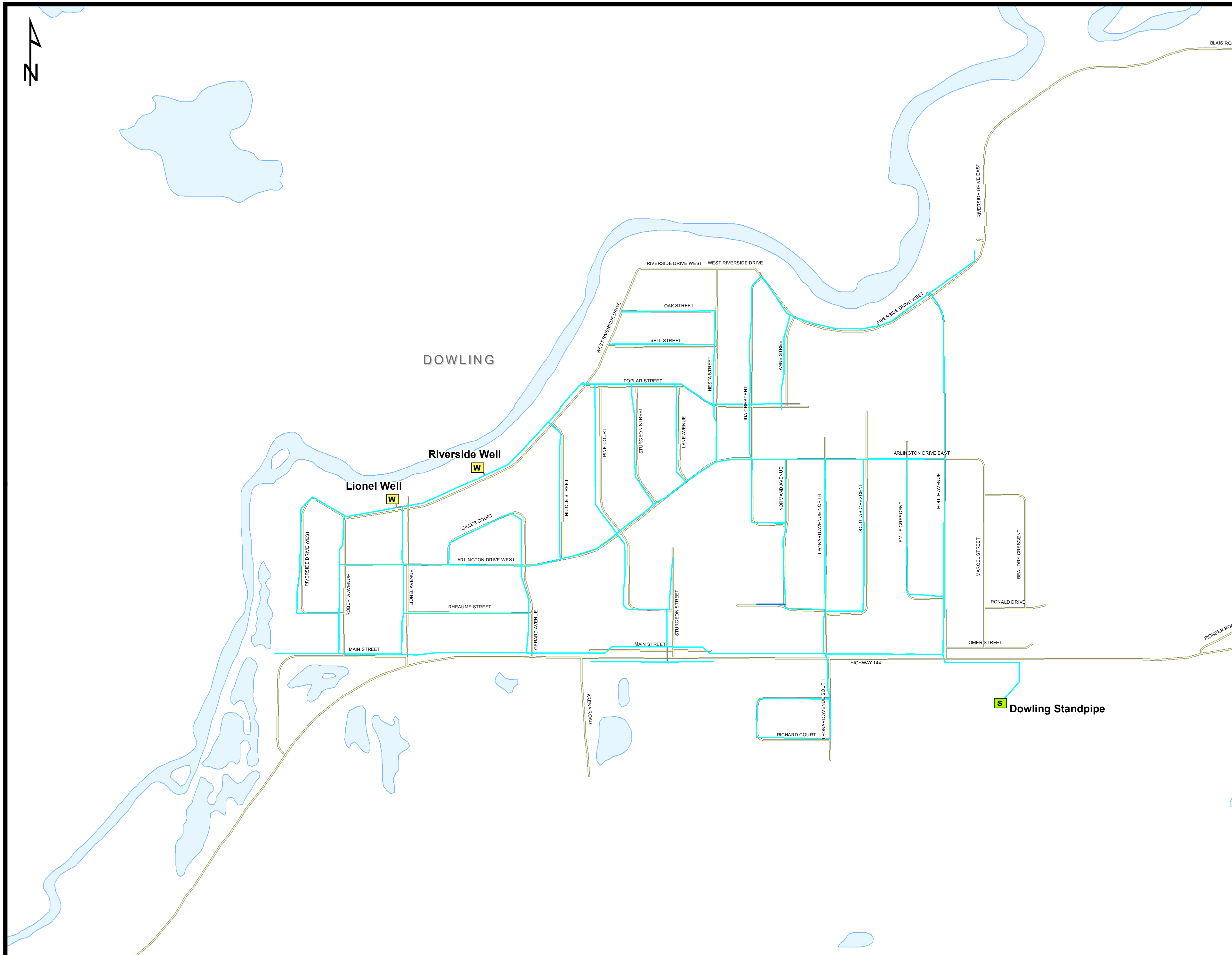
Map 3

Dowling Water System - Pipe Material



Legend

- Well
- Storage Tank
- Booster Pumping Station (BPS)
- Water Treatment Plant (WTP)
- Road
- Waterbody

Installed Year

- N/A
- <= 1930
- <= 1940
- <= 1950
- <= 1960
- <= 1970
- <= 1980
- <= 1990
- <= 2000
- <= 2010
- > 2011

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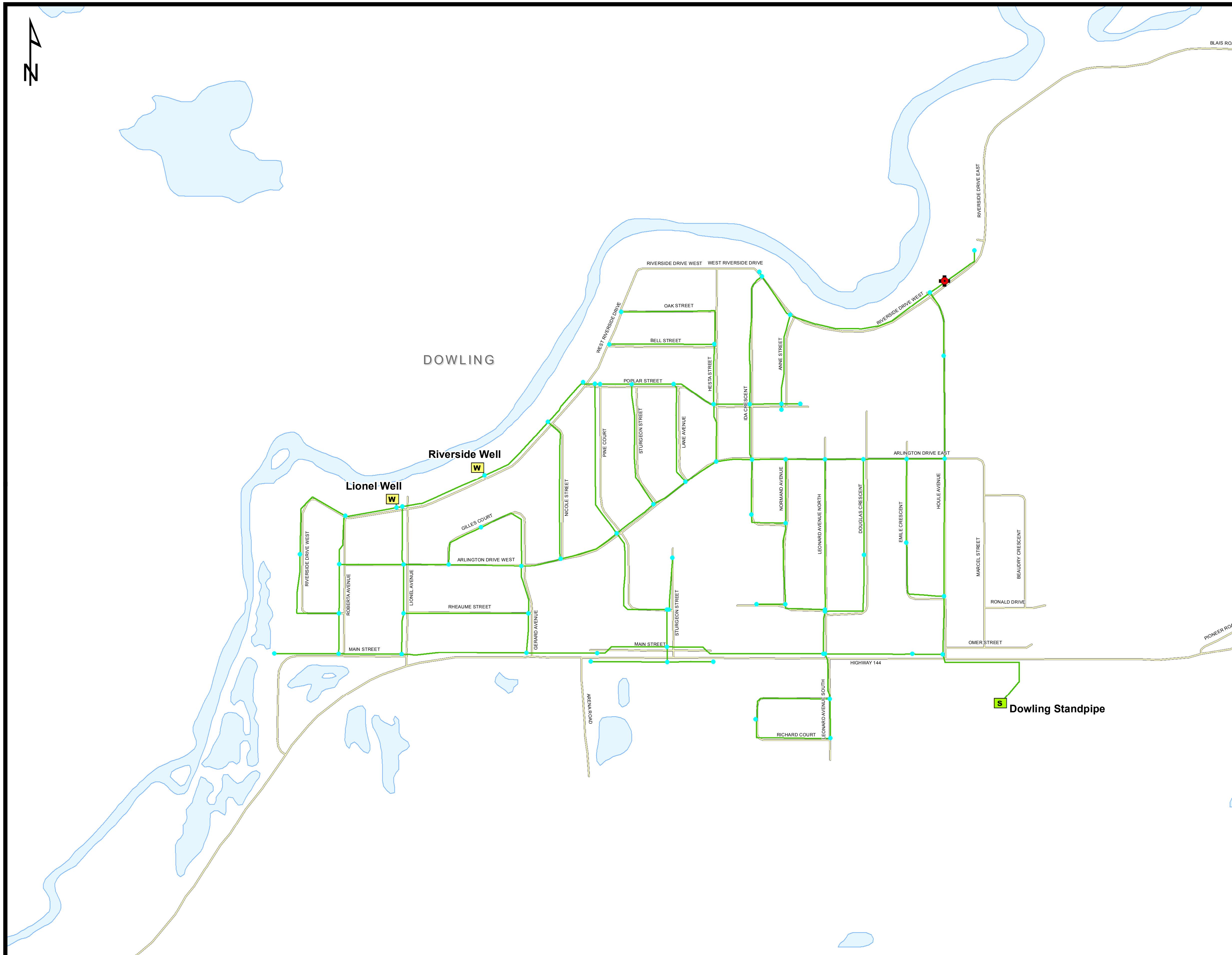
Map 4

Dowling Water System - Pipe Age



Legend

Well

Storage Tank

Booster Pumping Station (BPS)

Water Treatment Plant (WTP)

Hydrants Test

Road

Waterbody

Junction: Pressure (psi)

<= 20

<= 40

<= 60

<= 80

<= 100

> 100

Pipe: Headloss Gradient (m/km)

<= 1.5

<= 2.0

<= 3.0

<= 5.0

> 5.0

MDD/ADD = 2.71

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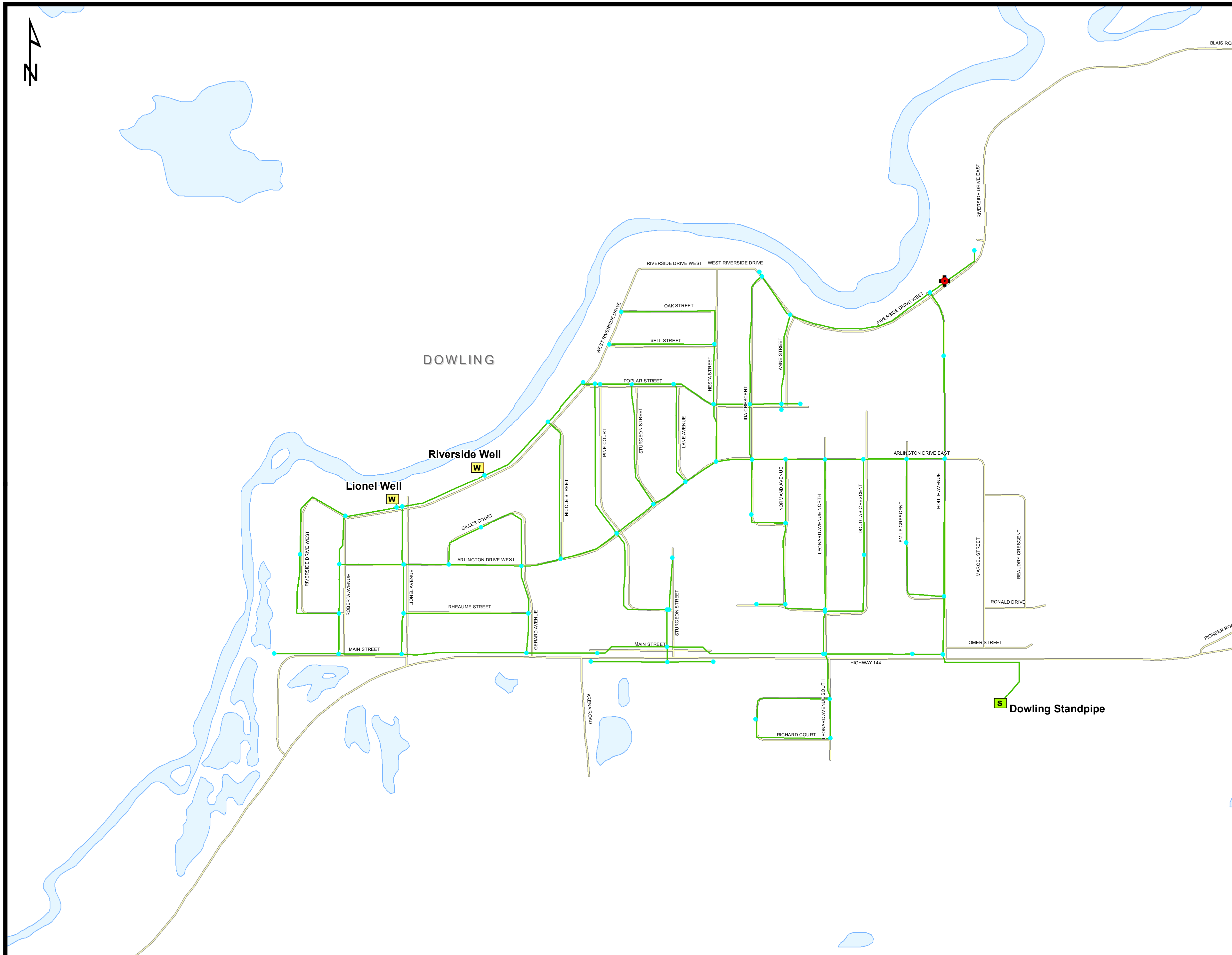
Map 5

Dowling Water System - Pipe Headloss and Junction Pressure

Scenario: 2011ADD



Legend

- Well
- Storage Tank
- Booster Pumping Station (BPS)
- Water Treatment Plant (WTP)
- Hydrants Test
- Road
- Waterbody

Junction: Pressure (psi)

- <= 20
- <= 40
- <= 60
- <= 80
- <= 100
- > 100

Pipe: Headloss Gradient (m/km)

- <= 1.5
- <= 2.0
- <= 3.0
- <= 5.0
- > 5.0

MDD/ADD = 2.71

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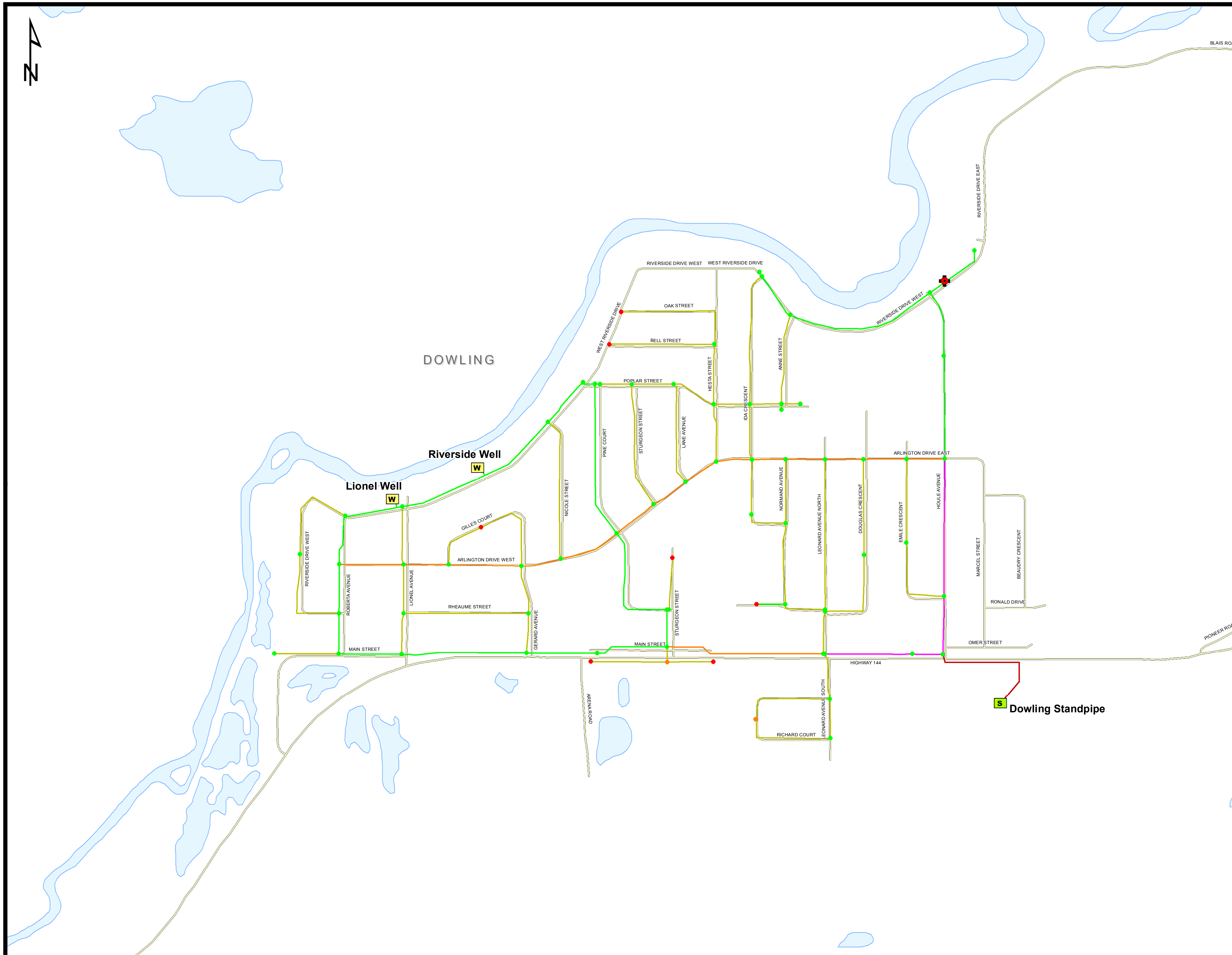
Map 6

Dowling Water System - Pipe Headloss and Junction Pressure

Scenario: 2011MDD



Legend

- W Well
- S Storage Tank
- BPS Booster Pumping Station (BPS)
- WTP Water Treatment Plant (WTP)
- Hydrants Test
- Road
- Waterbody

Junction: Fire Flow (Available) (L/s)

- <= 65 (Res) or 130 (ICI)
- < 75 (Res) or 150 (ICI)
- >= 75 (Res) or 150 (ICI)

Pipe: Diamater (mm)

- <= 150
- <= 200
- <= 250
- <= 300
- <= 400
- <= 600
- \> 600

MDD/ADD = 2.71

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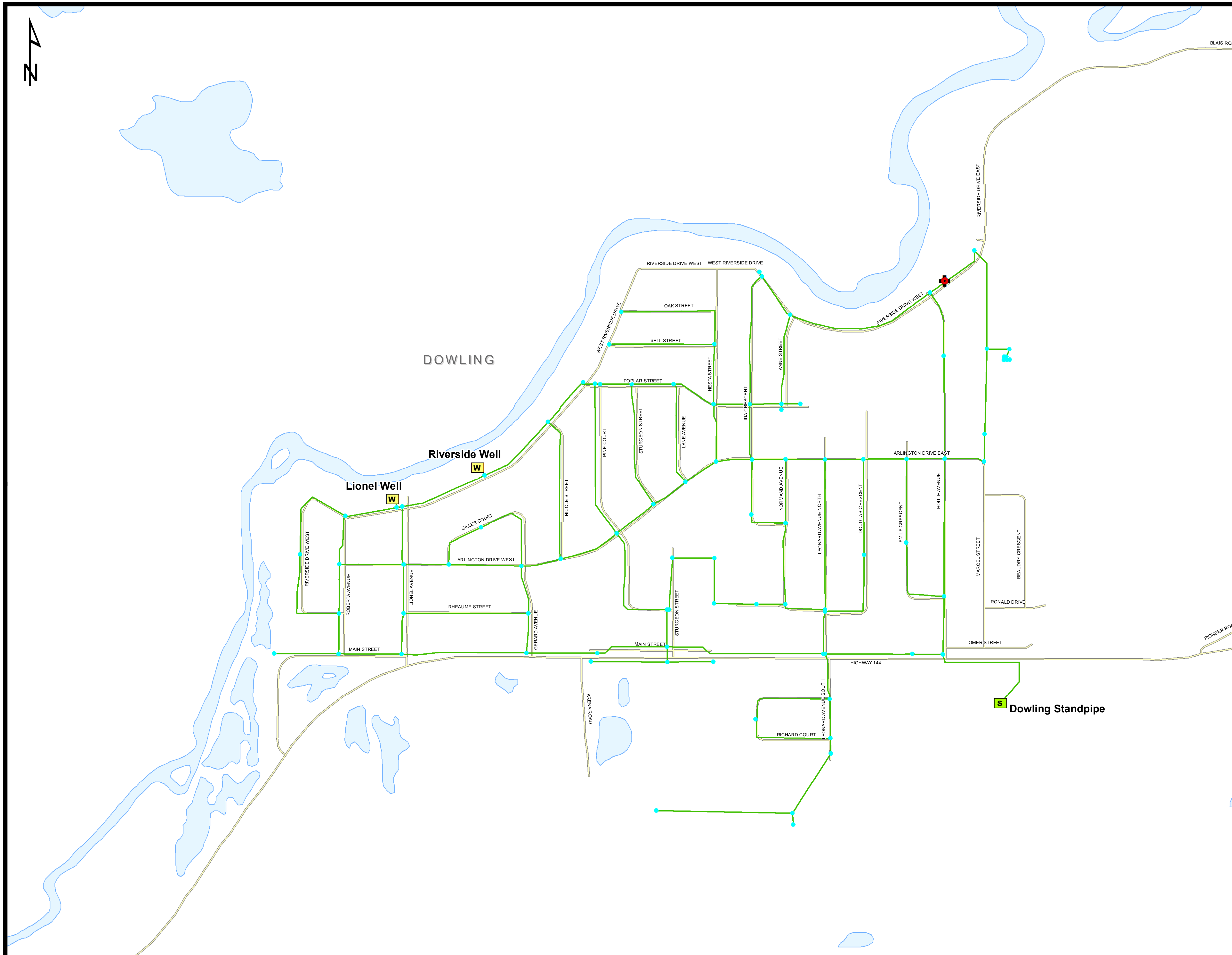
Map 7

Dowling Water System - Fire Flow Availability

Scenario: 2011MDD+FF



Legend

- W Well
- S Storage Tank
- BPS Booster Pumping Station (BPS)
- WTP Water Treatment Plant (WTP)
- Hydrants Test
- Road
- Waterbody

Junction: Pressure (psi)

- <= 20
- <= 40
- <= 60
- <= 80
- <= 100
- > 100

Pipe: Headloss Gradient (m/km)

- <= 1.5
- <= 2.0
- <= 3.0
- <= 5.0
- > 5.0

MDD/ADD = 2.71

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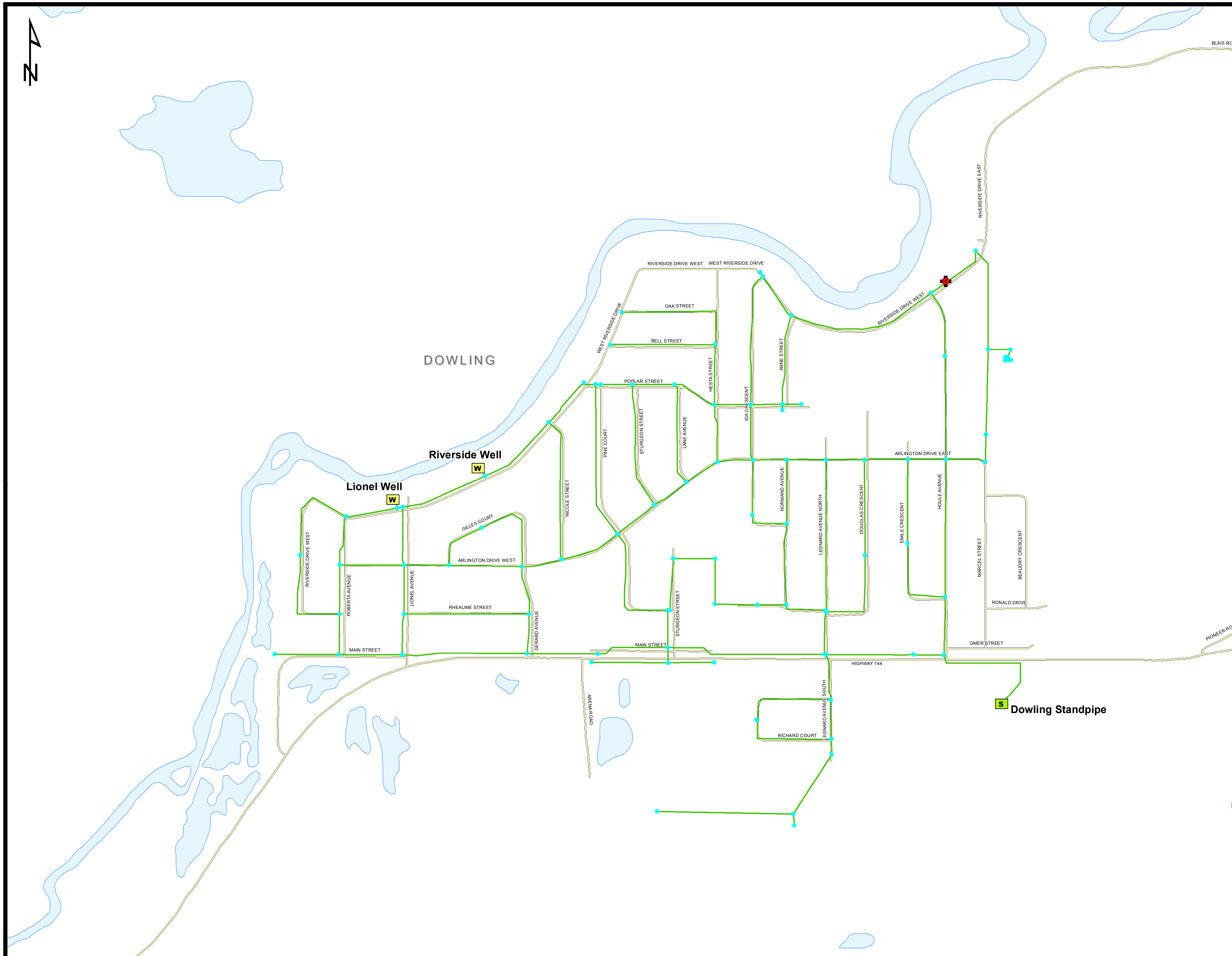
Map 8

Dowling Water System - Pipe Headloss and Junction Pressure

Scenario: 2014ADD



Legend

- W Well
- S Storage Tank
- BPS Booster Pumping Station (BPS)
- WTP Water Treatment Plant (WTP)
- Hydrants Test
- Road
- Waterbody

Junction: Pressure (psi)

- <= 20
- <= 40
- <= 60
- <= 80
- <= 100
- > 100

Pipe: Headloss Gradient (m/km)

- <= 1.5
- <= 2.0
- <= 3.0
- <= 5.0
- > 5.0

MDD/ADD = 2.71

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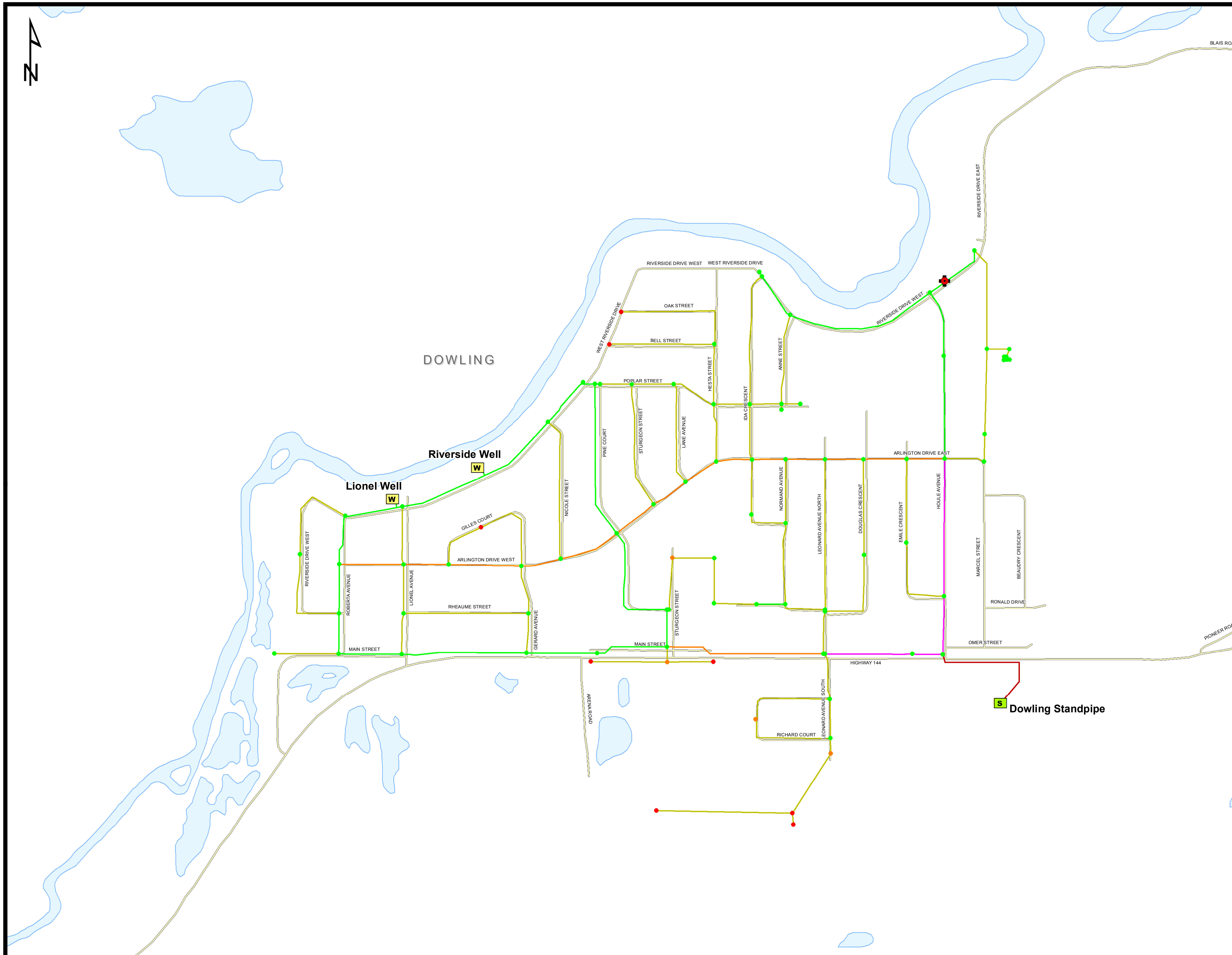
Map 9

Dowling Water System - Pipe Headloss and Junction Pressure

Scenario: 2014MDD



Legend

- W Well
- S Storage Tank
- BPS Booster Pumping Station (BPS)
- WTP Water Treatment Plant (WTP)
- Hydrants Test
- Road
- Waterbody

Junction: Fire Flow (Available) (L/s)

- <= 65 (Res) or 130 (ICI)
- < 75 (Res) or 150 (ICI)
- >= 75 (Res) or 150 (ICI)

Pipe: Diamater (mm)

- <= 150
- <= 200
- <= 250
- <= 300
- <= 400
- <= 600
- > 600

MDD/ADD = 2.71

Sudbury Greater | Grand

WSP

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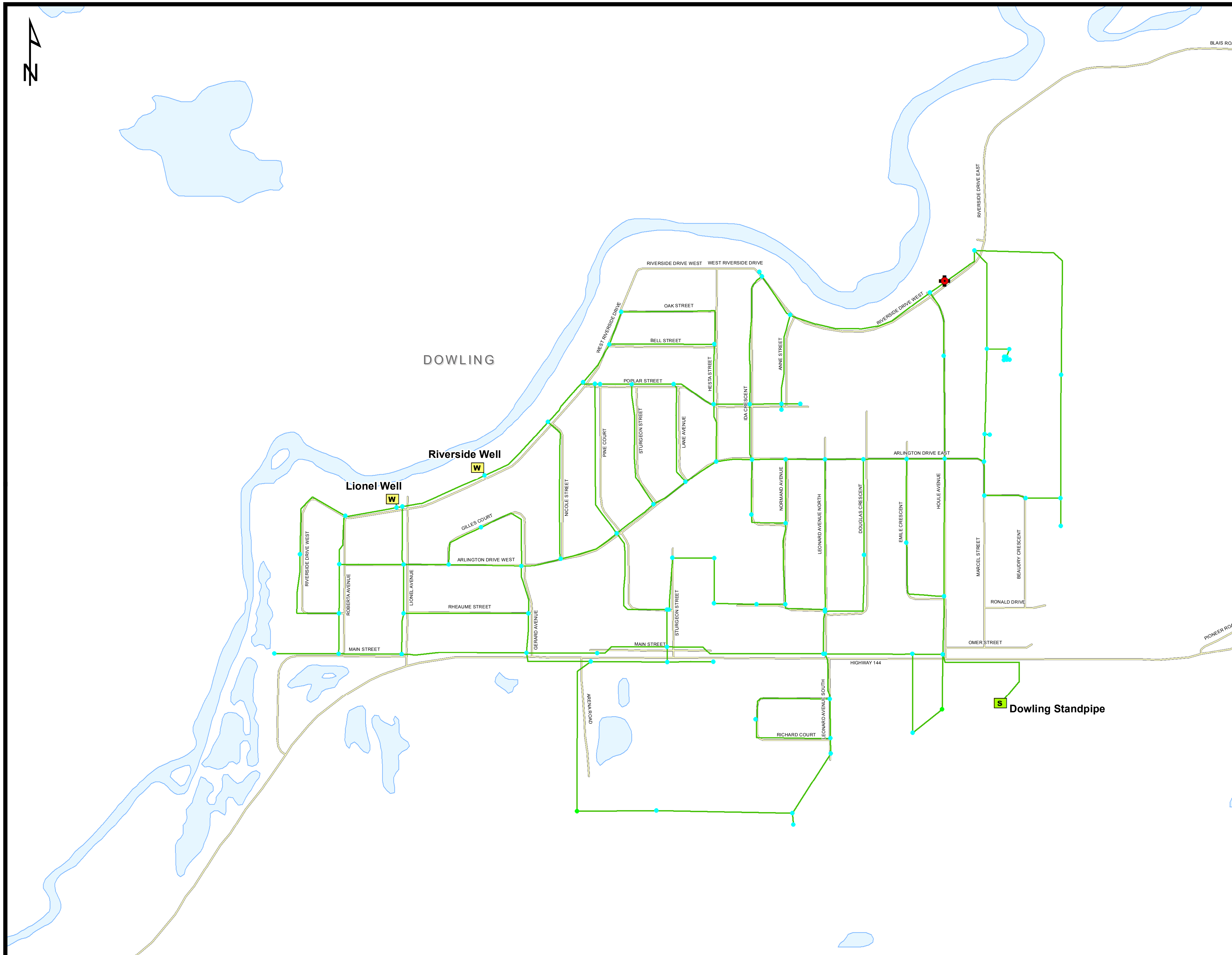
Map 10

Dowling Water System - Fire Flow Availability

Scenario: 2014MDD+FF



Legend

- W Well
- S Storage Tank
- BPS Booster Pumping Station (BPS)
- WTP Water Treatment Plant (WTP)
- Hydrants Test
- Road
- Waterbody

Junction: Pressure (psi)

- <= 20
- <= 40
- <= 60
- <= 80
- <= 100
- > 100

Pipe: Headloss Gradient (m/km)

- <= 1.5
- <= 2.0
- <= 3.0
- <= 5.0
- > 5.0

MDD/ADD = 2.71

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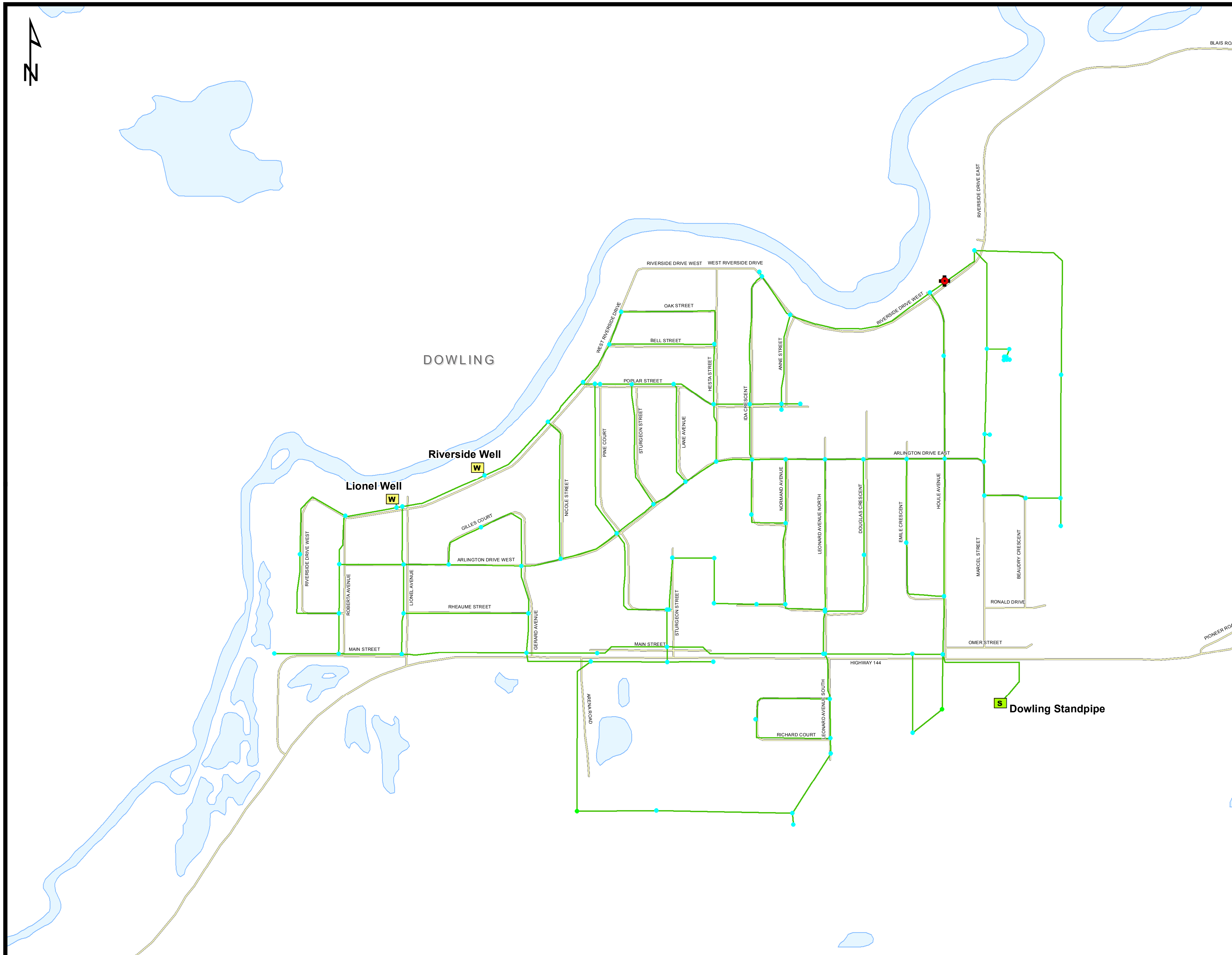
Map 11

Dowling Water System - Pipe Headloss and Junction Pressure

Scenario: ULTADD



Legend

- Well
- Storage Tank
- Booster Pumping Station (BPS)
- Water Treatment Plant (WTP)
- Hydrants Test
- Road
- Waterbody

Junction: Pressure (psi)

- <= 20
- <= 40
- <= 60
- <= 80
- <= 100
- > 100

Pipe: Headloss Gradient (m/km)

- <= 1.5
- <= 2.0
- <= 3.0
- <= 5.0
- > 5.0

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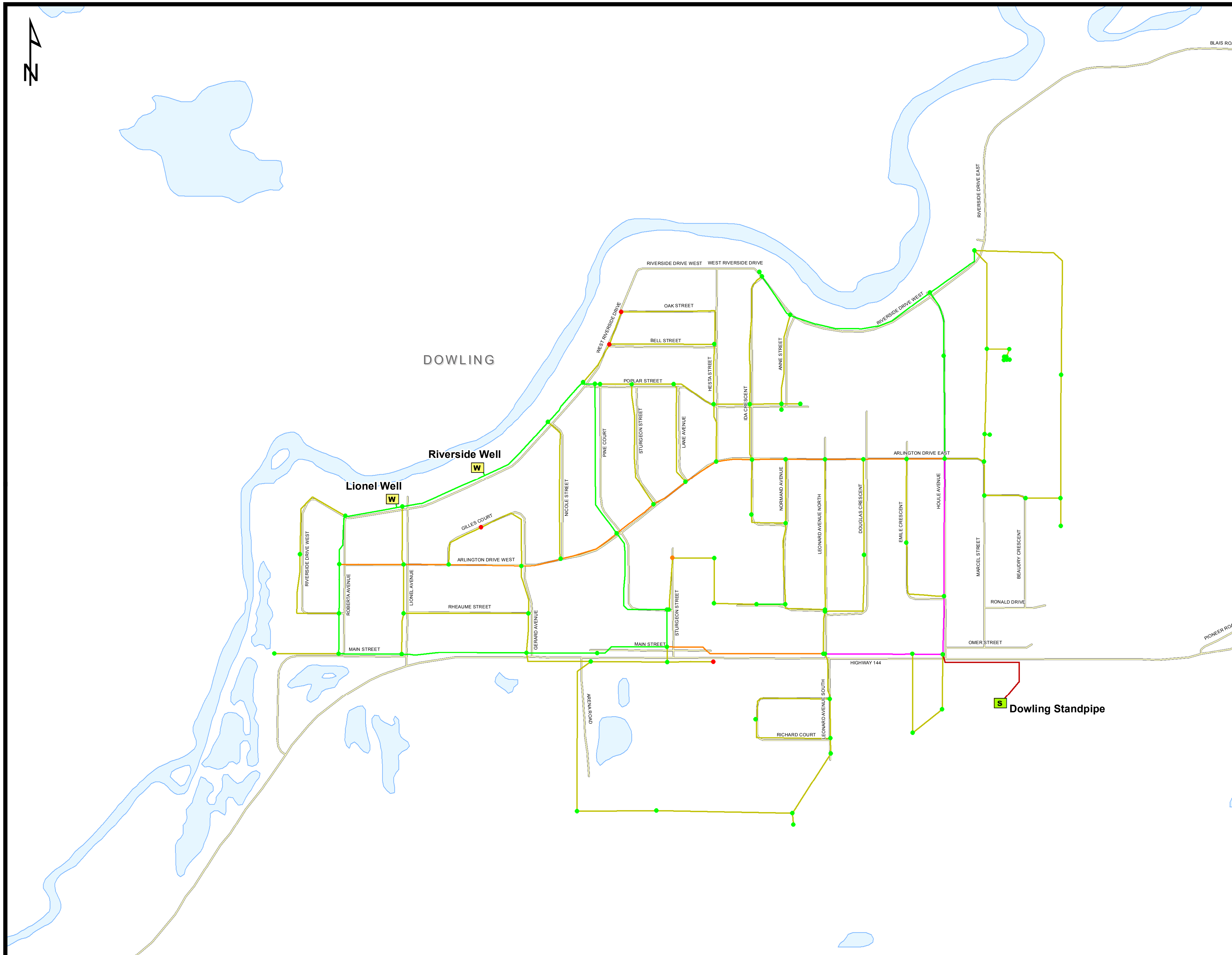
Map 12

Dowling Water System - Pipe Headloss and Junction Pressure

Scenario: ULTMDD



Legend

- W Well
- S Storage Tank
- BPS Booster Pumping Station (BPS)
- WTP Water Treatment Plant (WTP)
- Road
- Waterbody

Junction: Fire Flow (Available) (L/s)

- <= 65 (Res) or 130 (ICI)
- < 75 (Res) or 150 (ICI)
- >= 75 (Res) or 150 (ICI)

Pipe: Diamater (mm)

- <= 150
- <= 200
- <= 250
- <= 300
- <= 400
- <= 600
- > 600

MDD/ADD = 2.71

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Map 13

Dowling Water System - Fire Flow Availability

Scenario: ULTMDD+FF

CITY OF GREATER SUDBURY WATER AND WASTEWATER MASTER PLAN

FALCONBRIDGE WATER SYSTEM GAP ANALYSIS AND STATUS QUO REPORT

CITY OF GREATER SUDBURY

DRAFT

PROJECT NO.: 121-23026-00
DATE: APRIL 2015

WSP
100 COMMERCE VALLEY DRIVE WEST
THORNHILL, ON, CANADA L3T 0A1

TEL.: +1 905 882-1100
FAX: +1 905 882-0055
WSP.COM

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APPENDICES

1 INTRODUCTION

The City of Greater Sudbury (CGS) retained WSP (previously GENIVAR) to undertake a Water and Wastewater Master Plan. The purpose of the Master Plan project is to establish servicing strategies for water and wastewater infrastructure for the core urban areas and surrounding communities in the City for the next 20 years, as part of the five-year review of the City's Official Plan. The Master Plan will identify potential projects to address the servicing needs for planned growth within the City. It is being conducted in accordance with the requirements set out in the Municipal Class Environmental Assessment (Class EA) document (June 2000 as amended in 2007 and in 2011).

This report includes a capacity review of the Falconbridge existing water and wastewater systems. Based on population growth projections and design criteria discussed in the *Population and Unit Rates Technical Memorandum* (WSP, 2014) water demands and wastewater generation projections were developed and used to determine future infrastructure needs to the 2041 and ultimate buildout planning horizons. This report assumes that the Falconbridge Water and Wastewater Systems would continue to be stand-alone systems. Any potential interconnections between Falconbridge and other systems are not considered as part of this report. Potential interconnections with other communities will be reviewed under separate cover, as part of the Master Plan.

The conclusions provided in this report will be the basis for the problem definition and evaluation of alternatives conducted as part of the Master Plan.

Additional information on the existing water and wastewater systems is provided in the Baseline Review Reports for Water and Wastewater Systems (WSP, 2014).

2 STUDY AREA

Falconbridge is a small community located in the east end of the City of Greater Sudbury. The system is supplied by a single well-based drinking water system and a single wastewater system.

Map 1 in **Appendix A** show the Falconbridge study area and identifies future land use and development areas, including vacant residential and industrial, commercial, and institutional (ICI) areas. Additional information on population growth and development phasing is provided in the Unit Rates and Population Projections Technical Memorandum (WSP, 2014).

Existing development in the study area is mixed, and includes residential as well as industrial land uses. Notably, the Glencore Smelter Complex is located in Falconbridge, near Edison Road and Longyear Drive.

Based on the City's planning data, little growth is expected for Falconbridge. The area population is expected to increase from 707 in 2011 to 855 by Ultimate Buildout.

ICI growth is expected to be primarily institutional with small amounts of commercial and industrial. Growth is discussed further in Section 5.1.

3 OVERVIEW OF EXISTING WATER SYSTEM

The Falconbridge Water System services the community of Falconbridge and supplies water to three heavy industrial users: Glencore, Nickel Rim Mine, and the airport. The system is supplied by three non-GUDI wells (Wells 5, 6, and 7) located at the north end of the community. Prior to entering the distribution system, water is disinfected, and then fluoridated at the Hardy Fluoridation Facility. However, non-fluoridated water is supplied to the Nickel Rim and airport reservoirs.

The total rated well supply capacity for the system is 2,713 m^3/d, in accordance with the Permit to Take Water. However, the maximum day capacity of the fluoridation facility is 727 m^3/d, and typically operates at 173 m^3/d, as described in the *Baseline Review Report – Water* (WSP, 2014).

Access to the wells is via a private road through the Glencore Smelter Complex. City operations staff requires special training to enter the complex. All of the wells are owned and operated by the City of Greater Sudbury. Additional information on the existing systems is provided in the Baseline Review Report for Water Systems (WSP, 2014).

3.1 FALCONBRIDGE WELLS

Each of the three wells is equipped with a well pump and located in individual well houses. Water from all three wells is treated at the well house for Well 7. Standby power for all three wells is also located at Well 7 (200 kW diesel generator). Chorine gas is used for disinfection and a corrosion inhibitor is added to the treated water. The water is discharged to the Hardy Fluoridation Facility for fluoride addition, and a side-stream is sent to the Nickel Rim and Airport reservoirs. Potable, fluoridated water enters the Falconbridge distribution system from the Hardy Fluoridation Facility. The process is illustrated in the diagram below and a summary of the process equipment at each facility is provided in Figure 3-1.

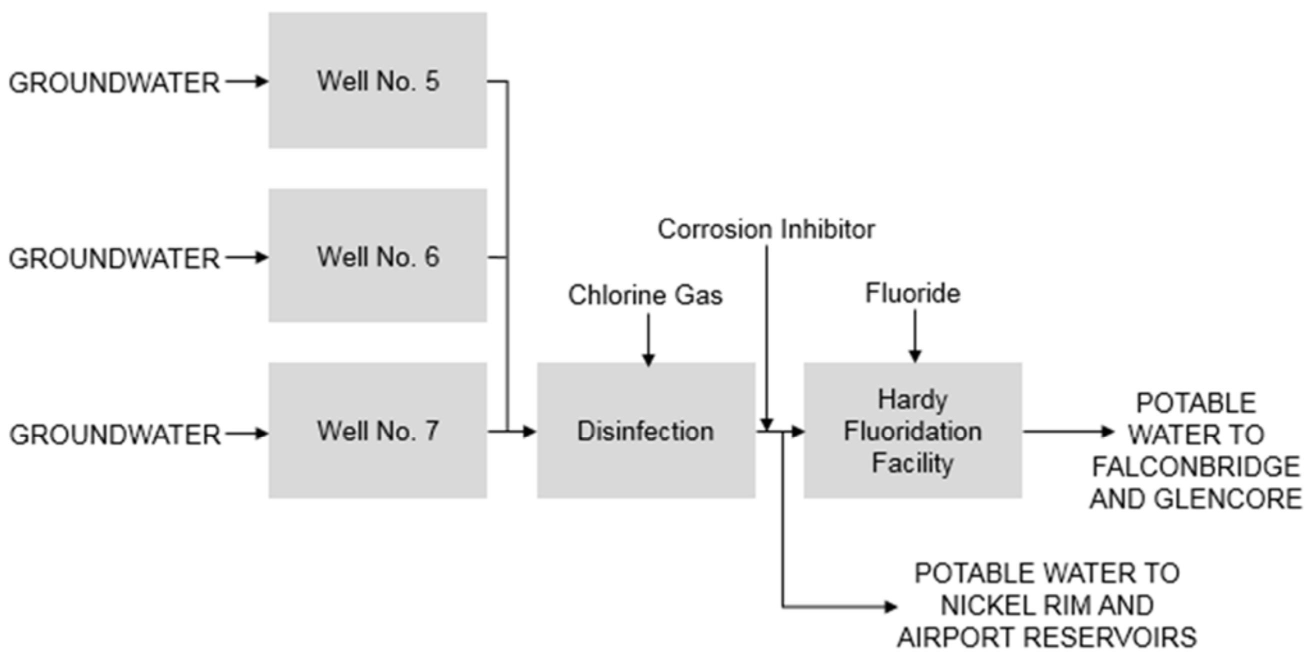

Figure 3-1 Falconbridge Wells' Process Flow Diagram

Table 3-1 Falconbridge Wells' Process Information

WELL	PUMP TYPE[1]	CAPACITY[2]	STANDBY POWER[1]
Well 5	Submersible well pump	16.4 L/s (1417 m^3/day) at 130 m TDH	200 kW diesel generator
Well 6	Submersible well pump	16.4 L/s (1417 m^3/day) at 130 m TDH	
Well 7	Submersible well pump	16.4 L/s (1417 m^3/day) at 130 m TDH	

[1] Obtained from the Falconbridge Drinking Water Works Permit, Number 016-201 Issue 1.

[2] Obtained from the Falconbridge Wells Permit to Take Water.

3.2 DISTRIBUTION SYSTEM

The Falconbridge distribution system consists of the following infrastructure:

— Falconbridge Storage Tank
— Mott Booster Pumping Station
— Hardy Fluoridation Facility

The Falconbridge Storage Tank is described in Table 3-2.

Table 3-2 Falconbridge Water System Storage Summary

TANK	STYLE	DIA. (M)[1]	BASE EL. (M)[1]	LOW WATER LEVEL (M)[1]	HIGH WATER LEVEL (M)[1]	DWWP TOTAL VOLUME (M3)
Falconbridge	Elevated	13.1	330	357	367	1,136

[1] Obtained from the as-built drawings for the elevated tank.

The Falconbridge Elevated Tank was constructed in 1962 and is reaching the end of its useful service life, typically estimated to be 60 years. However, there have not been any reported operational or structural concerns. As a preventative measure, it is recommended that the City continue to periodically inspect the tank.

The Falconbridge Booster Pumping Station is described in Table 3-3.

Table 3-3 Falconbridge Water System Booster Pumping Station Summary

FACILITY	BOOSTED AREA	PUMP INFORMATION*	TOTAL CAPACITY (L/S)	FIRM CAPACITY** (L/S)
Mott BPS	Southwest part of Falconbridge	Two, each rated at 2.5 L/s at 22.0 m TDH	5.0	2.5

[1] Obtained from the Falconbridge water model.

[2] The Firm Capacity is calculated assuming the largest pump out of service.

The Mott BPS was constructed in 1983 and boosts pressures in the area along the north-south portion of Edison Road, at the west end of Falconbridge, as pictured in Figure 3-2.

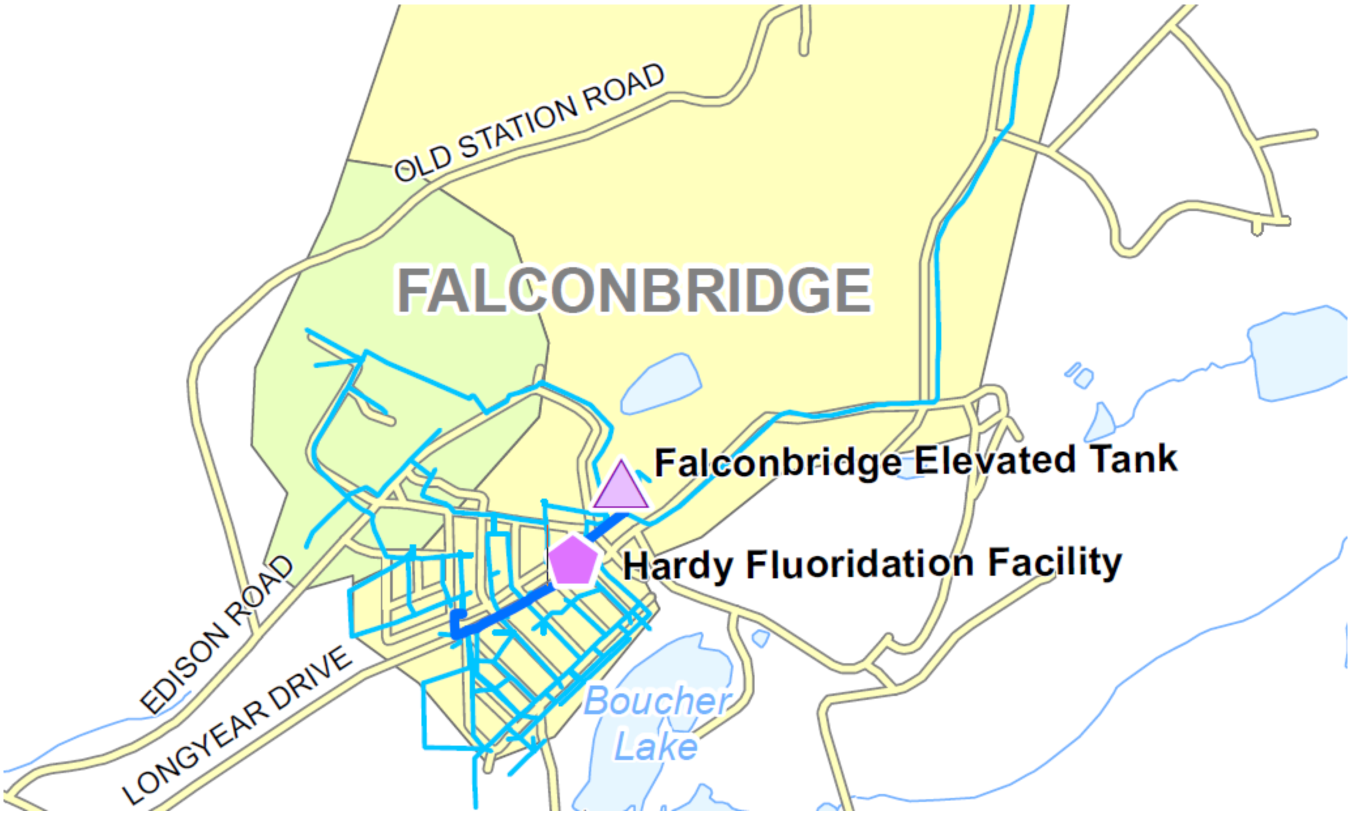

Figure 3-2 Area Boosted by the Mott BPS

3.3 KNOWN CHALLENGES

The Falconbridge Water System was originally built by a local mining company to service their employees. Over time, the population grew and the City obtained ownership of the system. Due to the system's history and age, infrastructure was not built in accordance with current industry or City standards. For example, parts of the system consist of backyard watermains which are now inaccessible due to fences, pools, and garages. Similarly, the wells are located away from the populated part of town, and access is only possible by a private road through the Glencore Smelter Complex. Although access is granted to City staff as required, special training is needed. This limits the number of City operators who can access the site. Finally, little as-built information is available for this system and the age of the watermains is not known.

In addition, the City runs a program instructing about five customers (exact number varies annually) in the Falconbridge Water System to run a small amount of water through their taps in the winter months to prevent water services from freezing on the municipal side. The specific number of customers included in the program varies annually depending on the expected winter temperatures.

Some service connections in Greater Sudbury freeze due to the shallow depth of bury; older homes were constructed prior to the current standards for depth of bury and are more vulnerable to freezing.

Customers who are requested to run their water are asked to run a small flow, equivalent to about the thickness of a pencil or approximately 0.06 L/s, between December 1 and April 1. In Falconbridge, this results in a total of about 3,000 m^3 per season, or 26 m^3/d. In the winter, this accounts for approximately 1% of the firm well production capacity (2,834 m^3/d).

4 HISTORICAL FLOWS AND REVIEW OF OPERATIONAL DATA

Water supply data from the Falconbridge Wells from 2009 to 2013 was reviewed and analyzed for this evaluation. Table 4-1 shows a summary of the data received, and indicates how it was used for the analysis.

Table 4-1 Water Supply Data Reviewed

DATA RECEIVED	PARAMETERS INCLUDED	DATA INTERVAL	USE IN ANALYSIS
Treated flow (2011-2013)	Flow in m^3/d	Hourly	To determine peak hourly flow
Annual Reports (2009-2013)	Total average daily flows, maximum daily flows Treated water characteristics	Daily	To determine average day, max day flow To assess performance of existing process and treated water characteristics
Annual Billing Data (2012)	Annual flow per customer in m^3	Annually	To determine the proportion of total water consumption corresponding to residential users

4.1 FLOW DATA

Water supply data from 2009 to 2013 was reviewed to determine historical water demands in the Falconbridge Water System. Average day and maximum day demand data for the past five years, and peak hour data for the past three years (2011-2013) is included in Table 4-2.

Table 4-2 Historical Water Supply Data

YEAR	AVERAGE DAY DEMAND (M3/D)[1]	MAXIMUM DAY DEMAND (M3/D)[1]	PEAK HOUR DEMAND (M3/D)[2]
2009	1,161	2,995	Not Available
2010	1,157	2,107	Not Available
2011	1,156	1,945	2,584
2012	1,021	1,701	2,562
2013	1,058	3,005	3,670

[1] *Falconbridge Drinking Water System Annual Reports (2009 – 2013).*
[2] *From hourly SCADA data.*

Average day water consumption was consistent between 2009 and 2013. The average consumption for the five year period was 1,111 m^3/d.

The highest maximum day flow recorded in the past four years was 3,005 m^3/d, occurring in 2013. The average historical maximum day demand is 2,350 m^3/d.

Hourly flow data was only available from 2011 to 2013. The maximum peak hour value recorded during that period was 3,670 m^3/d in 2013, and the average was 2,939 m^3/d.

The peaking factors derived from historical data were compared to those documented in the *City's Engineering Design Manual* (City of Greater Sudbury, 2012) and those included in the *MOECC Guidelines* (MOE, 2008).

The maximum day to average day peaking factor corresponding to the maximum day flow recorded (3,005 m^3/d in 2013) was 2.84, while the average maximum day peaking factor was 2.12. The City's Engineering Design Manual specifies a maximum day factor of 2.75 for Falconbridge, which matches the corresponding value recommended in the *MOECC Guidelines*. The highest maximum day factor (2.84) was adopted to evaluate future requirements.

The peak hour to average day factor corresponding to the highest peak hour flow recorded in 2013 (3,670 m^3/d) was 3.47, while the average peak hour factor was 2.74.

The *City's Engineering Design Manual* and the *MOE Guidelines* specify a peak hour factor of 4.13. For purposes of estimating future demands, the historical maximum value (3.47) was adopted.

4.2 RAW WATER CHARACTERISTICS AND SECURITY OF SUPPLY

The Falconbridge Wells aquifer is classified as non-GUDI and has good water quality. There have been slightly high levels of sodium in the treated water (21.5 mg/L recorded in 2010). City operations staff has noted that the elevated sodium is at one of the three wells, and the remaining two do not have sodium concerns.

4.3 OPERATIONAL DATA

Data reported in the *Annual Reports* for the Falconbridge supply facilities includes effluent chlorine residual, trihalomethanes (THMs), fluoride, and trace organic and inorganic chemicals.

Data was reviewed from 2009 to 2013 to determine any historical issues at the wells. No exceedances were observed, except for elevated sodium levels in the blended water, as noted above.

4.4 HEAVY INDUSTRIAL WATER USAGE

There are some municipal water customers that consume substantial amounts of water annually, and include the Greater Sudbury Airport Reservoir, Glencore's Nickel Rim Mine Reservoir, and Glencore Operations (excluding the reservoir). The former two receive water that has not been fluoridated, while Glencore operations receives fluoridated water. The total approximate annual water billed to these customers was 218,245 m^3 in 2012, for an average day demand of 597 m^3/d. This is a substantial proportion, 22%, of the firm well capacity of 2,713 m^3/d.

Although this water usage should be included in the current and future demand requirements, it may be prudent to exclude the amount that is not fluoridated from the storage assessment since these flows supply third-party storage facilities, for their own uses. Similarly, if the remaining Glencore operations have on-site private water storage, this would further reduce the municipal storage needs.

This approach would avoid overestimating current and future needs. However, this study assumes the more conservative approach and includes all demands in the storage assessment.

5 DESIGN CRITERIA

The following design criteria were used to assess the remaining capacity of the existing systems and to forecast future requirements for the water and wastewater systems. The unit rates used to estimate future water and wastewater flows correspond to the values included in the *Population Projections and Unit Rates Technical Memorandum* (WSP, 2014). Otherwise, design criteria recommended in the *MOECC Guidelines* and *City's Engineering Design Manual* were used.

5.1 UNIT WATER DEMAND CRITERIA

The water demand criteria shown in are from the unit rates recommended in the *Populations and Unit Rates Technical Memorandum* (WSP, 2014). The rates were reviewed against historical data, MOECC *Guidelines* (MOE, 2008), and current standards in the City's *Engineering Design Manual* (City of Greater Sudbury, 2012).

Both the *MOECC Guidelines* and *City Engineering Design Manual* recommend determining demands for institutional, commercial and industrial (ICI) users on a case by case basis. However, the following criteria for ICI demands were used for the purposes of this evaluation.

Table 5-1 Falconbridge Water System Design Criteria

CRITERIA	VALUE	REFERENCE
Average Day Residential Demand	300 L/cap/day	Average of historical values, rounded up to nearest 50 L/cap/day
Average Day Institutional & Commercial Demand	28 m^3/ha/d	MOE Guidelines
Average Industrial Demand	35 m^3/ha/d	MOE Guidelines
Domestic Demand Maximum Day Factor	2.12	Average of historical values
Domestic Demand Peak Hour Factor	3.47	Maximum of historical values

Residential average day demands are obtained by multiplying the residential unit rate by the service population. Similarly, average ICI demands are obtained by multiplying the corresponding unit rates to the areas of development, assuming 100% of the area would be developed.

Maximum day and peak hour demands are obtained by multiplying the average day demand by the corresponding peaking factor.

For purposes of this study, and in line with city standards and practices, a residential fire flow of 75 L/s over 1.75 hours and ICI fire flow of 150L/s over 2 hours were used.

5.2 DESIGN CRITERIA FOR WATER SYSTEM COMPONENTS AND OPERATION

5.2.1 TREATMENT CAPACITY

Water supply facilities are designed to supply the maximum day demands of the system.

Treatment facilities must be designed in accordance with the *Procedure for Disinfection of Drinking Water in Ontario* (Ontario, 2006). Drinking water treatment systems that obtain water from a surface water or GUDI well supply must achieve an overall performance providing as a minimum a 2-log (99%) removal or inactivation of *Cryptosporidium* oocysts, 3-log (99.9%) removal or inactivation of *Giardia* cysts, and 4-log (99.99%) removal or inactivation of viruses.

At least 0.5-log removal or inactivation of *Giardia* cysts and 2-log removal or inactivation of viruses must be provided through disinfection, while the remaining removal may be achieved through filtration or other equivalent treatment processes.

5.2.2 PUMPING CAPACITY

Pumping stations are rated based on their firm capacity. If sufficient floating storage is available in a particular pressure district, the MOE defines firm capacity as the capacity of the station with the largest pump out of service. If there is insufficient or no floating storage, firm capacity is defined as the capacity with the two (2) largest pumps out of service (MOE, 2008).

For each pressure district, the pumping stations have to be designed to provide peak hour or maximum day plus fire demands (whichever are greater), if no floating storage is available. If sufficient floating storage is available, then the pumping station only needs to be designed to provide maximum day demands.

The Falconbridge system consists of two main pressure districts: the area boosted by the Mott BPS and the remaining area. Most of Falconbridge is supplied from the high lift pumps at Well 7 and the Falconbridge Elevated Tank. The area serviced by the Mott BPS receives water from the BPS.

5.2.3 STORAGE CAPACITY

Storage requirements are based on the requirement to meet water demands that exceed the capacity of the treatment plant and to satisfy fire flow demands. When the capacity of the supply system is only capable of satisfying maximum day demands, storage requirements are determined using the following formula from the *MOE Guidelines* (MOE, 2008):

$$\mathbf{Storage = A + B + C}$$

Where: A = Fire Storage, B = Equalization Storage = 25% of maximum day demand, and C = emergency storage = 25% of (A+B).

Fire storage is the product of the maximum fire flow required in the system and the corresponding fire duration based on Fire Underwriters requirements (Fire Underwriters Survey, 1999).

When the system can supply more than just the maximum day demand (but less than the peak demand), the fire storage requirements can be determined using the following formula:

$$\mathbf{A = (Peak\ Demand - Pumping\ Station\ Firm\ Capacity) \times Fire\ Duration}$$

Where: peak demand is the greater of the peak hour demand and the maximum day plus fire demand.

Per *MOE Guidelines*, floating storage should be designed such that the elevation of the equalization volume (B) is such that a minimum pressure of 275 kPa (40 psi) can be maintained in the system under peak hour flow conditions. The fire (A) and emergency (C) volumes should be at elevations that produce 275 kPa (40 psi) during peak hour demand conditions, and 140 kPa (20 psi) under the maximum day plus fire flow condition (MOE, 2008).

5.2.4 DISTRIBUTION CAPACITY

Watermains have to be sized to carry the greater of the maximum day plus fire flow or peak hour demand. The range of acceptable pressures under normal conditions (average to peak hour flows) is 275 kPa (40 psi) to 690 kPa (100 psi), while during fire flow conditions pressures may drop to 140 kPa (20 psi) (MOECC, 2008). The maximum allowable water velocity in the distribution system is 3 m/s (MOECC, 2008).

6 FUTURE REQUIREMENTS

6.1 POPULATION PROJECTIONS

As part of the City of Greater Sudbury Master Plan, population forecasts were developed for the 2016, 2021, 2026, 2031, 2036, 2041 and Ultimate Buildout planning years. Ultimate Buildout is defined as an estimate of what the demand from the total population and total number of households in the City of Greater Sudbury would be based on lands that are currently designated for development in the Official Plan within the existing settlement boundaries.

The City supplied planning data sheets with properties and development potential and the vacant residential and ICI land inventory, and Hemson Consultants, on behalf of the City, provided supplementary population projections. Data was provided for each water and wastewater system boundary. These data were used in conjunction to develop the targeted population growth for each horizon year, as well as development phasing (discussed in the next section and in detail in the *Populations and Unit Rates Technical Memorandum*, WSP 2014).

In cases where the City's planning data sheets and Hemson's population projections forecasted fewer development units than the vacant land inventory for an area, then specific parcels (up to the City's and Hemson's unit projections) of developable units were selected. These parcels were selected based on the rationale provided in the City's Official Plan. That is, the Official Plan prioritizes that development take place in areas that are currently serviced, or where servicing can easily be extended. This focuses growth in existing urban areas until supply is no longer available in these areas.

Based on the planning data, the population of Falconbridge is projected to increase by 69 people in 2041 and 148 by Ultimate Buildout. The population projections to be used in the Master Plan are summarized in Table 6-1.

Table 6-1 Falconbridge Water System Population Projections

	2011	2016	2021	2026	2031	2036	2041	ULTIMATE BUILDOUT
Falconbridge	707	724	743	759	769	775	776	855

The City's planning data does not specify target years for employment growth. However, vacant lands designated as institutional, commercial, and industrial (ICI) properties have been assigned to different stages of the development process by the City. These stages are described below.

- Draft Approved:
  - These are lands that have draft plan of subdivision approval under the Planning Act or have pending applications with the City. Typically, these lands are close to registration or few years away from development as the required conditions are satisfied
  - Development approvals are near complete, and development could take place at any time. Properties with this designation were set to take place in 2016.
- Legal Lots of Record:
  - These are existing lots, including lots in a registered plan of subdivision. Typically these lands are zoned, serviceable and only require building permit approval for development. In some cases a site plan approval/agreement may also be required.
  - Based on historical trends, development is approximately 15 years away from receiving draft approval. Properties with these designations were assigned to take place in 2026.
- Designated Developable:
  - These lands do not have any development approvals in place but are understood to be areas of future development as they are within the settlement boundary. Designated lands are typically a number of years away from being developed.

— Based on historical trends, these properties are approximately 10 years away from receiving Legal Lot of Record designation. Designated Developable properties were assumed to take place in 2036.

These land supply categories stem from the land supply requirements that municipalities must maintain under Section 1.4 of the Provincial Policy Statement. In this context, Designated Development Lands would count towards Section 1.4.1 (a) and Legal Lots of record and Draft Approved Lands would count towards 1.4.1 (b). It is also important to note that the total supply is governed by PPS Section 1.1.2.

The targeted ICI development areas for each horizon year are summarized in the table below.

Table 6-2 Falconbridge Water System ICI Projections

	ICI DEVELOPMENT AREAS (HA)						
LAND USE	2016	2021	2026	2031	2036	2041	BUILDOUT
Institutional	0.00	0.00	1.70	0.00	0.33	0.00	0.00
Commercial	0.00	0.00	0.17	0.00	0.00	0.00	0.00
Industrial	0.00	0.00	0.36	0.00	0.00	0.00	0.00
Total	**0.00**	**0.00**	**2.23**	**0.00**	**0.33**	**0.00**	**0.00**

The above assumptions provide an estimate as to the ICI development time line. In reality, development may be more staggered. However, for purposes of infrastructure planning and to ensure that the appropriate infrastructure is in place by the appropriate planning horizon, the above assumptions are considered to be conservative.

6.2 PHASING OF FUTURE GROWTH

Growth areas were allocated based on population projections for individual developments and the overall target growth population projections for the Falconbridge area for the horizon years.

Hemson's supplementary tables were used to provide the target population, while the City's planning tables and vacant lot inventory were used to identify phasing of specific properties, and assignment of draft approved, legal lots of record, and designated development properties. In general, priority was given to draft approved properties, followed by legal lots of record and designated developable properties. In accordance with the Official Plan, the City has also assigned a target quantity of legal lots of record and designated developable properties to be developed in each horizon year. That is, legal lots of record should account for approximately 20% of all household growth, and designated developable lots are assigned 20% of the 20 year anticipated growth.

The future growth phasing plans were presented in the *Unit Rates and Population Projections Technical* Memorandum (WSP, 2014).

6.3 FUTURE WATER DEMAND PROJECTIONS AND INFRASTRUCTURE NEEDS

The unit flow criteria listed in Section 5.1 were used to estimate the future water demands in the Falconbridge Water System. In general, the projected flows were calculated by the following formula:

$$\textbf{Projected Average Day Demand} \\ = \textbf{Base Demand} + \textbf{Additional Residential Demand} + \textbf{Additional ICI Demand}$$

The demands corresponding to the population growth forecasts to ultimate buildout are listed in Table 6-3.

Table 6-3 Water Demand Projections for the Falconbridge Water System

YEAR	POPULATION	AVERAGE DAY DEMAND (M3/D)	MAXIMUM DAY DEMAND (M3/D)	PEAK HOUR DEMAND (M3/D)
Base	707	1,111	2,350	2,939
2016	724	1,116	2,365	3,869
2021	743	1,121	2,377	3,888
2026	759	1,191	2,526	4,132
2031	769	1,194	2,532	4,142
2036	775	1,205	2,556	4,180
2041	776	1,206	2,556	4,181
Ultimate Buildout	855	1,229	2,606	4,263

The Base Demand was the average historical (2009 to 2013) average day, maximum day, and peak hour demand for the community. The additional residential demand was calculated using the unit flow rate multiplied by the population growth, and similarly, the ICI demand was calculated using the unit flow rate for each type of development (industrial, commercial or institutional), multiplied by the growth in development area.

Maximum day and peak hour demands were calculated by applying the respective peaking factor to the average day demand.

A desktop analysis of historical water demands and future water demand projections is included in **Appendix B**.

6.3.1 SUPPLY CAPACITY

The rated total and firm capacities for the Falconbridge Water System are summarized in the table below. The rated capacity is that which is listed in the facility's PTTW. The firm capacity is defined as the total rated or estimated operating capacity, less the one largest pump.

Table 6-4 Supply Capacity of the Falconbridge Water System

SOURCE	RATED CAPACITY(M3/D)	FIRM CAPACITY (M3/D)
Falconbridge Wells	4,251	2,713

The value corresponding to the firm capacity (2,713 m^3/d) was used for comparison against future needs of the Falconbridge Water System.

The projected maximum day demands are plotted versus the total system rated and firm capacities on Figure 6-1.

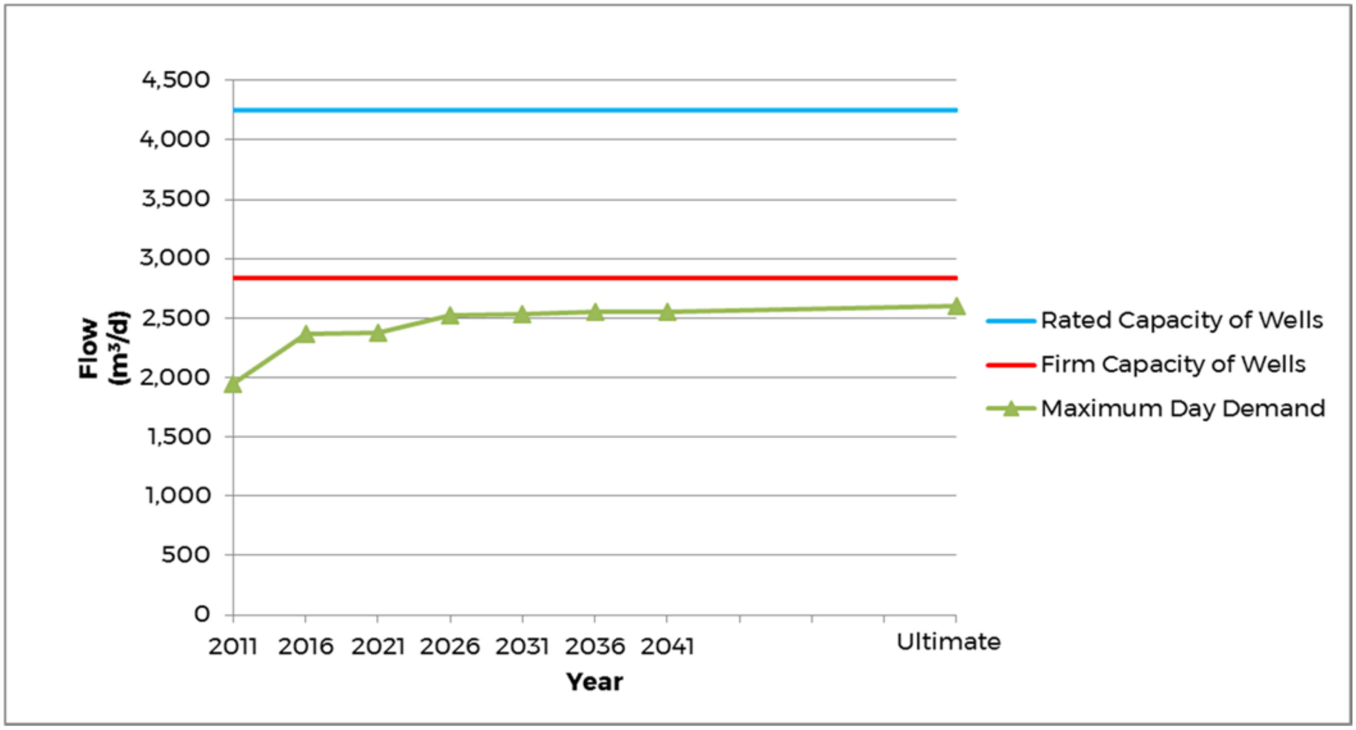

Figure 6-1 Water Demand Projections vs. Firm Capacity of the Wells

Based on the current and future demands, and the rated firm capacity of the wells, there is enough water supply capacity in Falconbridge to service growth to Ultimate Buildout.

6.3.2 STORAGE CAPACITY

Storage in the distribution system is provided by the Falconbridge Elevated Tank (1,136 m^3).

Applying the formula to determine storage requirements indicated previously, the corresponding fire storage requirement would be 1,080 m^3. Using the maximum day demand required to service growth to 2041, the corresponding equalization storage requirement would be 261 m^3 and the emergency storage would be 335 m^3. The total required storage to service growth to 2041 would be 1,741 m^3 and the total required storage to service the Ultimate Buildout growth scenario would be 1,756 m^3.

Therefore, additional storage is required in Falconbridge. The amount of storage required for each horizon year is shown in the figure below. As can be seen, there is currently a water storage deficiency in the Falconbridge Water System.

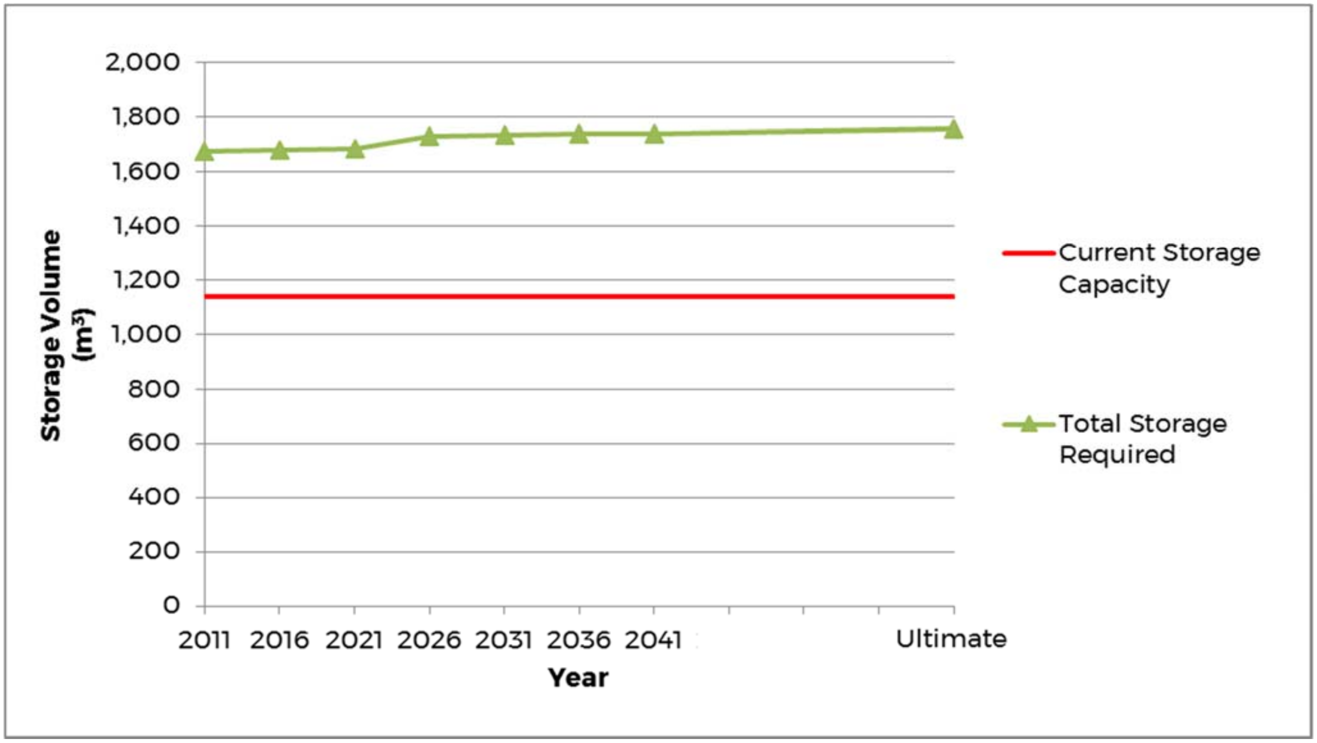

Figure 6-2 Available Storage Capacity Compared to Future Needs

6.3.3 DISTRIBUTION NETWORK

The water model was used to identify system elements (i.e. watermains, pumps, storage tank) for which the capacity was exceeded by the projected water demands. The capacity of the system was assessed in terms of the available fire flows and system pressures.

For each planning scenario, watermains of the modelled network were reviewed to assess whether the required minimum fire flows (75 L/s in residential areas or 150 L/s in ICI areas) and pressures (over 20 psi under fire conditions and over 40 psi under normal conditions) were achieved. Furthermore, some new watermains were added to service greenfield areas where development was planned. A simplified watermain layout was assumed for these areas.

Future populations and demands were loaded into the model based on the planning data and flow projections discussed in Section 6.3. Development that would take place as part of the Urban Expansion Area has been excluded from the Ultimate Buildout modeling scenario to avoid overestimating demands. In general, development in Falconbridge might deviate from the proposed phasing scheme. Thus, it is recommended that the hydraulic water model be updated whenever a development application is submitted.

The findings from the water modeling are discussed in Section 7.2 and presented in **Appendix C**.

7 HYDRAULIC WATER SYSTEM MODELLING

An all-pipe model of the system including pipes, hydrants, storage tanks and system source was developed by the City using Bentley Systems' WaterGEMS hydraulic modeling software. This model was updated based on information provided by the City to reflect current system conditions.

The water model allows for simulations to be conducted that can be used to predict system responses to events under a wide range of conditions. Using simulations, problems can be anticipated in proposed or existing systems, and solutions can be evaluated before time, money, and materials are invested in a real-world project. Simulations can either be steady-state or extended-period. Steady-state simulations represent a snapshot in time and are used to determine the operating behaviour of a system under static conditions. This type of analysis can be useful in determining the short-term effect of fire flows or average demand conditions on the system. Extended period simulations (EPS) are used to evaluate system performance over time. This type of analysis allows modeling the filling and emptying of storage facilities, regulating valves opening and closing, and pressures and flow rates changing throughout the system in response to varying demand conditions and automatic control strategies.

Simulations including steady-state analysis of the Average Day, Maximum Day and Maximum Day + Fire conditions were carried out using the model. Fire flow simulations were carried out throughout the system to determine whether the system could deliver fire flows under the Maximum Day demands.

7.1 WATER MODEL DEVELOPMENT

To model the current scenario, the following steps were taken:

- Total network demand on an average day basis was determined for the current scenario using 2012 water production data.
- The node demand allocations assigned in the model were based on 2012 meter records. Metered flows were assigned to the respective property. In cases where meter records showed zero flow, the value was manually adjusted to reflect a reasonable volume for a respective property, depending on land use.
- The maximum day peaking factor discussed in Section 4.1, above, were applied to the average day demand value to determine the maximum day demand.
- The maximum day demand plus fire flow was used to assess the system since it was greater than the peak hour demand.

7.2 MODELING FINDINGS

7.2.1 FIREFIGHTING CAPACITY

Firefighting capacity was assessed for the distribution system, with exception of areas not designed to convey fire flows. These include areas that were constructed under different design standards; these areas have small diameter (150 mm or less) watermains and no fire hydrants.

There are several portions of the water network that cannot supply the required fire flow demands.

Water model outputs, including maps showing fire flow analysis, are provided in **Appendix C.**

7.2.2 MODELED HYDRAULIC CAPACITY UNDER NORMAL CONDITIONS

Based on the system modeling, service pressures throughout the system under the maximum day demand scenario generally range between 40 and 80 psi (276 and 552 kPa) for all scenarios. Therefore, flows throughout the system are within the range prescribed in the MOECC Guidelines (40 to 100 psi) under normal conditions.

Maps showing pressures at nodes are presented in **Appendix C**.

8 CONCLUSIONS AND RECOMMENDATIONS

An assessment of the Falconbridge Water and Wastewater Systems was completed to identify infrastructure investment requirements to service forecasted growth in the community. The assessment involved a review of previous studies, an analysis of operations and flow data from the water and wastewater facilities, and an evaluation of the capacity of the system.

The conclusions of the assessment are summarized below.

- Based on the estimated firm well capacity of the Falconbridge Water System as well as historical and projected maximum day demands, no additional supply will be needed to service growth to Ultimate Buildout.
- There is not enough storage capacity in the system to service the current or future system. The current (2011) storage deficit is approximately 0.5 ML, growing to 0.6 ML to service growth to Ultimate Buildout.
- There are several portions of the water network that cannot supply the required fire flow demands.

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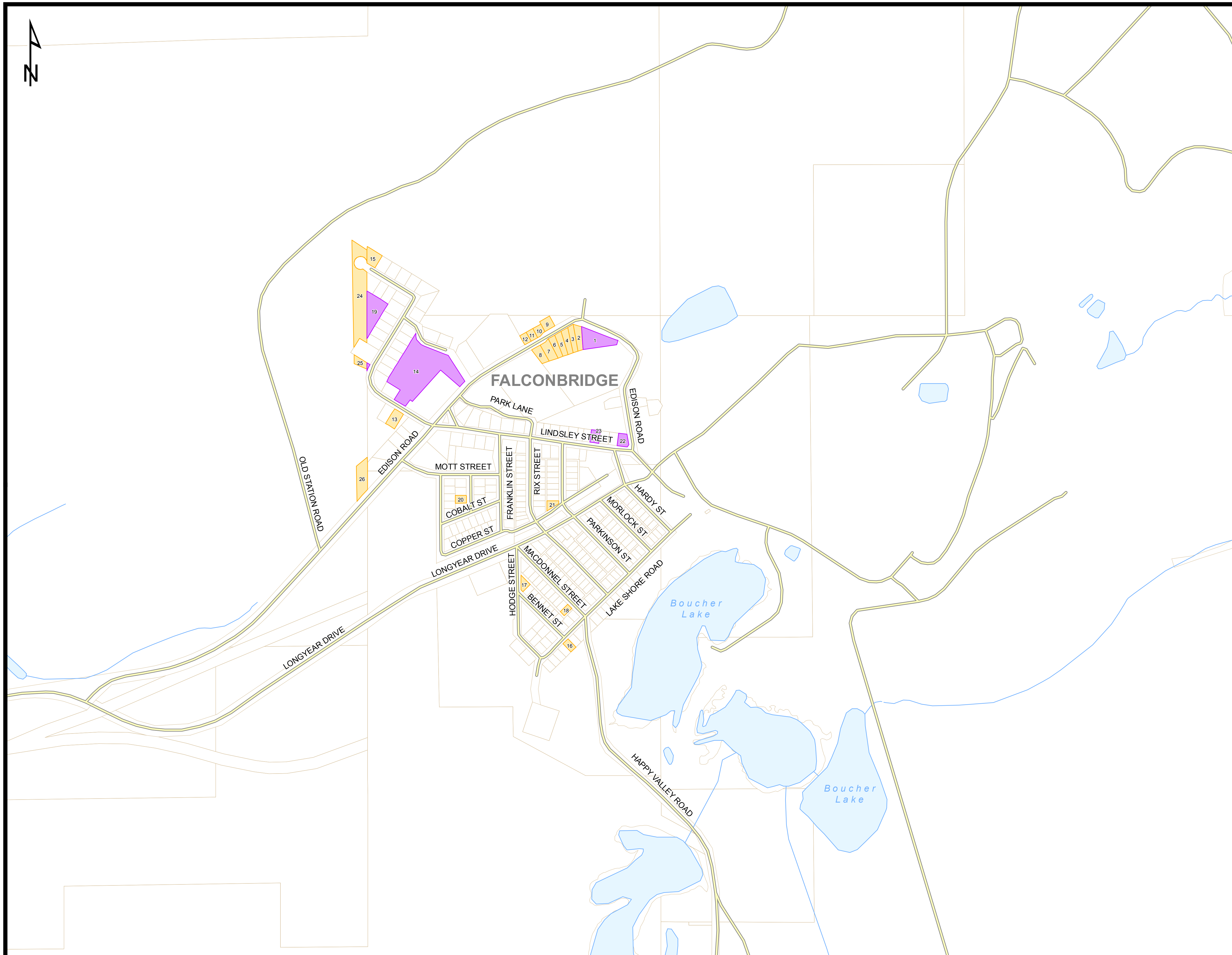
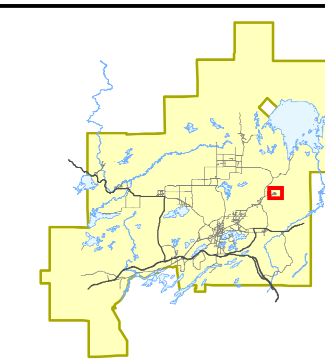
APPENDIX

A RESIDENTIAL AND ICI DEVELOPMENT AREAS



Legend
Road
Waterbody
Watercourse
Parcel
Vacant ICI
Vacant Residential



Greater Sudbury | WSP

Project No. 121-23026-00

Sudbury Water and Wastewater Master Plan

0 100 200 300 400 500 m

1:10,000

Map 1

Falconbridge
2011 - Ultimate Buildout
Residential and ICI
Development Areas

APPENDIX

B WATER DEMAND AND CAPACITY ASSESSMENTS

DATA ANALYSIS

	2009	2010	2011	2012	2013	Summary	Design Criterion	Comments
Average Day Flow (m^3/d)	1,161	1,157	1,156	1,021	1,058	1,111	1,111	From Water Historical Production data. The daily production values for each facility were added together to determine the total daily production.
Max Day Flow (m^3/d)	2,995	2,107	1,945	1,701	3,005	2,350	2,350	
Max Day Factor	2.58	1.82	1.68	1.66	2.84	2.12	2.12	MOE Guidelines recommend a value of 2.75 for populations between 500 and 1,000. This is in line with the historical maximum.
Peak Hour (L/s)	Not Available		29.9	29.7	42.5			Peak values were available only for 2011-2013.
Peak Hour (m^3/d)			2,584	2,562	3,670	2,939		
Peak Hour Factor			2.24	2.51	3.47	2.74	3.47	MOE Guidelines recommend a value of 4.13 for populations between 500 and 1,000. However, the historical maximum was higher than other historical values, and so this value was adopted.
Population (Existing Areas)	707	707	707	707	707		707	

	2016	2021	2026	2031	2036	2041	Ultimate	Comments
Population (Existing Areas)	707	707	707	707	707	707	707	From data provided by Hemson grouped by water system.
Population Growth	17	35	52	62	68	69	148	
Total Population	724	743	759	769	775	776	855	From data provided by Hemson grouped by water system.
Residential Growth Area (ha)	0.48	0.67	0.37	0.33	0.86	0.00	0.00	From City's GIS database. 2041 and Ultimate areas are included with 2036.
Residential Growth Area (ha) - Cumulative	0.5	1.1	1.5	1.8	2.7	2.7	2.7	
Institutional Growth Area (ha)	0.0	0.0	1.70	0.0	0.33	0.0	0.0	
Institutional Growth Area (ha) - Cumulative	0.0	0.0	1.70	1.70	2.03	2.03	2.03	From City's GIS database.
Commercial Growth Area (ha)	0.0	0.0	0.17	0.0	0.0	0.0	0.0	
Commercial Growth Area (ha) - Cumulative	0.0	0.0	0.17	0.17	0.17	0.17	0.17	From City's GIS database.
Industrial Growth Area (ha)	0.0	0.0	0.36	0.0	0.0	0.0	0.0	
Industrial Growth Area (ha) - Cumulative	0.0	0.0	0.36	0.36	0.36	0.36	0.36	From City's GIS database.
ICI (ha) - Cumulative	0.00	0.00	2.24	2.24	2.57	2.57	2.57	Sum of Institutional, Commercial and Industrial areas
Total Growth Area (ha) - Cumulative	0.5	1.1	3.7	4.1	5.3	5.3	5.3	

	2009	2010	2011	2012	2013	Summary	Design Criterion	Comments
Ratio of Residential to Total Water Billed	0.19	0.19	0.19	0.19	0.19	0.19		Estimated amount of water consumption related to ICI based on metering data and obtained ratio of residential to total consumption.
Residential Flow (m^3/d)	218	217	217	192	199	209		Calculated based on ratio of residential consumption to total consumption.
Ratio of ICI to Total Water Billed	0.81	0.81	0.81	0.81	0.81	0.812		
	942	939	939	829	859	902		
Per Capita Residential Demand (m^3/cap/day)	0.308	0.307	0.307	0.271	0.281	0.295	0.300	Took average over 2009 to 2013 period. The trend is generally consistent.
Average Institutional Flow Unit Rate (m^3/ha/d)							28.0	*MOE Guidelines recommend a value of 28 m^3/ha/d.*
Average Commercial Flow Unit Rate (m^3/ha/d)							28.0	*MOE Guidelines recommend a value of 28 m^3/ha/d.*
Average Industrial Flow Unit Rate (m^3/ha/d)							35.0	*MOE Guidelines recommend a value of 35 m^3/ha/d for light industry and 55 m^3/ha/d for heavy industry.*

	2016	2021	2026	2031	2036	2041	Ultimate
Average Residential and ICI Flows (m^3/d) - Existing	1,111	1,111	1,111	1,111	1,111	1,111	1,111
Average Residential Flows (m^3/d) - Growth	5	11	16	19	20	21	44
Average Residential Flows (m^3/d) - Total	1,116	1,121	1,126	1,129	1,131	1,131	1,155
Average Institutional Flow (m^3/d)	0	0	48	48	57	57	57
Average Commercial Flow (m^3/d)	0	0	5	5	5	5	5
Average Industrial Flow (m^3/d)	0	0	13	13	13	13	13
Average ICI Flow (m^3/d)	0	0	65	65	74	74	74
Average Day Flow (m^3/d)	1,116	1,121	1,191	1,194	1,205	1,206	1,229
Max Day Flow (m^3/d)	2,365	2,377	2,526	2,532	2,556	2,556	2,606
Peak Hour Flow (m^3/d)	3,869	3,888	4,132	4,142	4,180	4,181	4,263

ALTERNATIVE CALCULATION METHOD

This method does not distinguish between Residential and ICI water consumption.

	2009	2010	2011	2012	2013	Summary	Design Criterion	Comments
Per Capita Demand (m^3/cap/day)	1.641	1.636	1.635	1.444	1.497		1.570	*If ICI is not considered explicitly and demand is divided by total population.*
Max Day Factor							2.84	*The historical per capita consumption is applied for future development.*
Peak Hour Factor							3.47	

	2016	2021	2026	2031	2036	2041	Ultimate
Average Day Flow (m^3/d)	1,137	1,166	1,193	1,208	1,218	1,219	1,343
Max Day Flow (m^3/d)	3,229	3,311	3,386	3,431	3,457	3,460	3,812
Peak Hour Flow (m^3/d)	3,943	4,044	4,136	4,190	4,222	4,226	4,656

SENSITIVITY ANALYSIS

Analyze sensitivity of forecasted average day flows to unit rate

	Unit Rate (m^3/cap/d)	Average Day Flow (m^3/d) 2016	2021	2026	2031	2036	2041	Ultimate
Using a consolidated per capita flow	**1.570**	1,137	1,166	1,193	1,208	1,218	1,219	1,343
Using estimated average	**0.300**	1,116	1,121	1,191	1,194	1,205	1,206	1,229
City Standards	**0.41**	1,118	1,125	1,197	1,201	1,213	1,213	1,246

Analyze sensitivity of forecasted flows to max day peaking factor

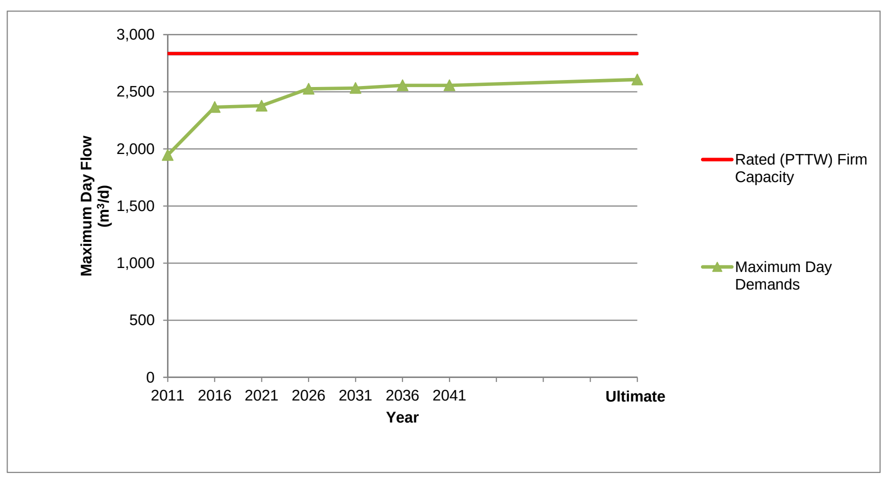
	Max Day Peaking Factor	Max Day Flow (m^3/d) 2016	2021	2026	2031	2036	2041	Ultimate
2009-2013 average of peaking factors	**2.12**	2,363	2,374	2,523	2,529	2,553	2,553	2,603
Maximum historical max day factor	**2.84**	2,365	2,377	2,526	2,532	2,556	2,556	2,606
MOE Guidelines	**1.65**	1841	1850	1966	1971	1989	1989	2028

Analyze sensitivity of forecasted flows to peak hour factor

	Peak Hour Peaking Factor	Peak Hour (m^3/d) 2016	2021	2026	2031	2036	2041	Ultimate
Using historical highest peak factor	3.47	3,869	3,888	4,132	4,142	4,180	4,181	4,263
Using average of historical peaking factors	2.74	3,054	3,069	3,261	3,269	3,300	3,300	3,365
MOE Guidelines	2.48	2,767	2,781	2,955	2,962	2,990	2,990	3,049

CAPACITY CHECK

	2011	2016	2021	2026	2031	2036	2041	Ultimate
Rated (PTTW) Firm Capacity	2,834 4,251	2,834	2,834	2,834	2,834	2,834	2,834	2,834
Maximum Day Demands	1,945	2,365	2,377	2,526	2,532	2,556	2,556	2,606
Peak Hour Demands	2,584	3,869	3,888	4,132	4,142	4,180	4,181	4,263

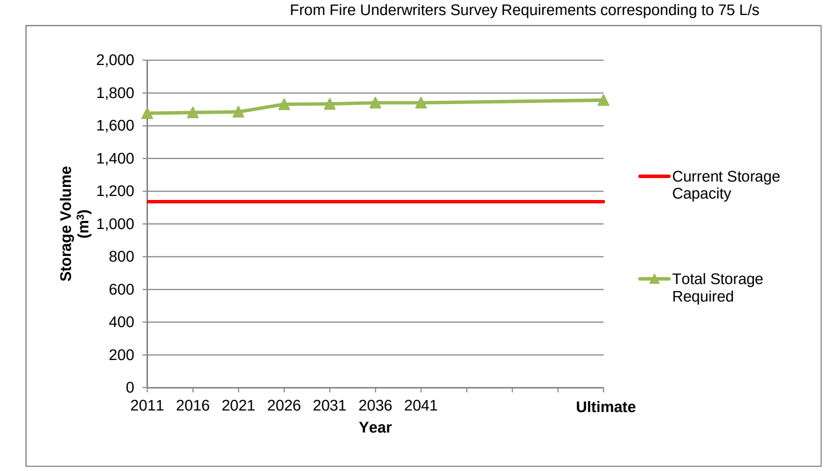


STORAGE REQUIREMENTS

Storage Available			
Elevated Tank (m^3)	1,136	(Useable Volume)	From DWWP 016-201 (September 13, 2011)
Total Storage (m^3)	1,136		
Maximum Fire flow Requirements (L/s) - Existing Network	150		
Maximum Fire flow Requirements (L/s) - Future (starting in 2021)	150		
Fire Duration (hrs)	2		
Minimum Fire Flow Requirement for Residential Areas (L/s)	75		From CGS Engineering Design Manual
Fire Duration (hrs)	1.75		From Fire Underwriters Survey Requirements corresponding to 75 L/s

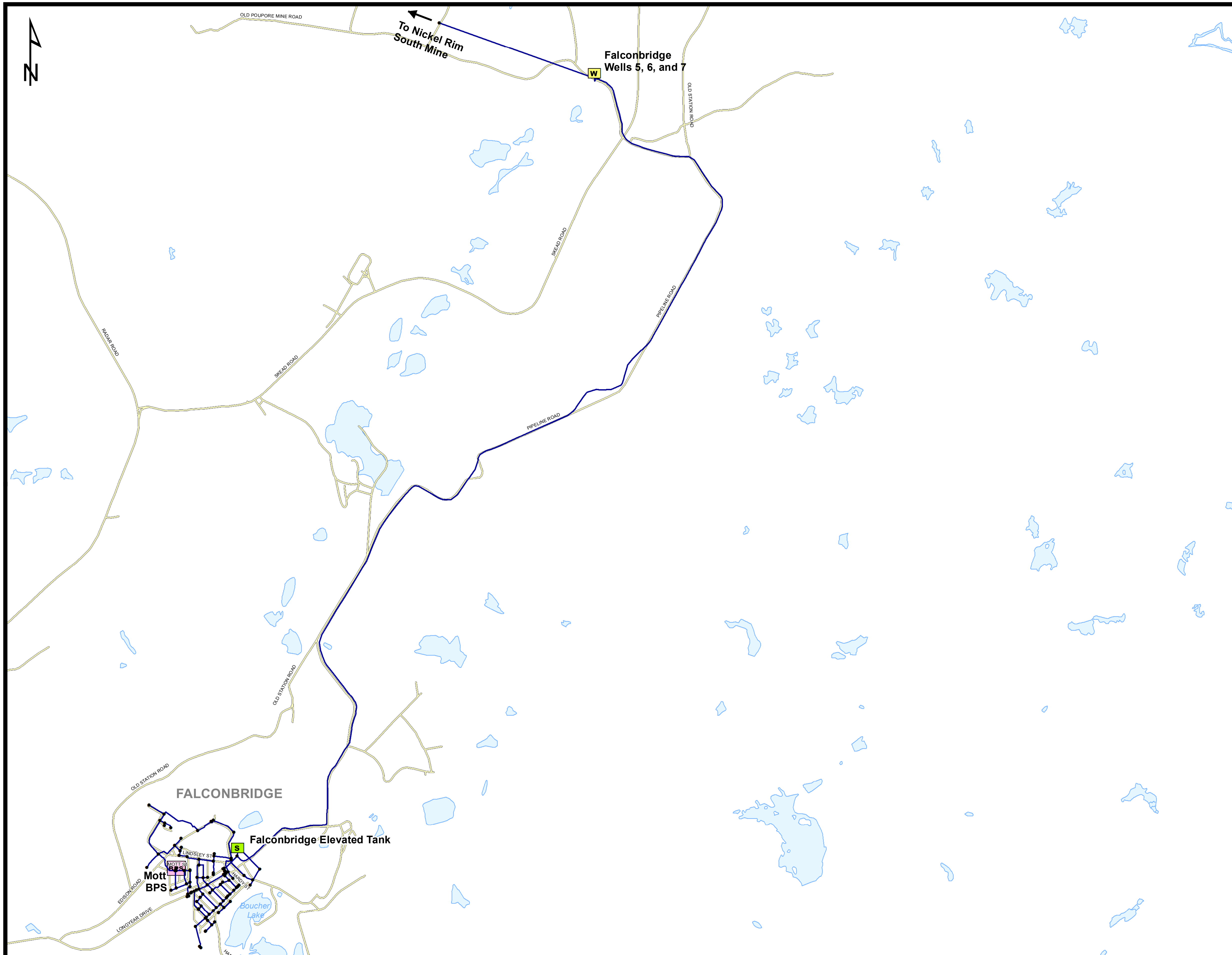
	Max Day Demand (m^3/d)	Required Fire Flow (m^3/d)	Max Day + Fire (m^3/d)	Peak Hour (m^3/d)	A - Fire Storage (m^3)	B - Equalization Storage (m^3)	C - Emergency Storage (m^3)	A + B + C = Storage Required (m^3)	Storage Available (m^3)	Deficit (m^3)
2011	1,044	12,960	14,004	2,939	1,080	261	335.3	1,676	1,136	540
2016	1,059	12,960	14,019	3,869	1,080	265	336.2	1,681	1,136	545
2021	1,071	12,960	14,031	3,888	1,080	268	336.9	1,685	1,136	549
2026	1,220	12,960	14,180	4,132	1,080	305	346.2	1,731	1,136	595
2031	1,226	12,960	14,186	4,142	1,080	307	346.6	1,733	1,136	597
2036	1,250	12,960	14,210	4,180	1,080	312	348.1	1,741	1,136	605
2041	1,250	12,960	14,210	4,181	1,080	313	348.1	1,741	1,136	605
Ultimate	1,300	12,960	14,260	4,263	1,080	325	351.3	1,756	1,136	620

Residential Max Day (Base Year) = 1044 m3/day
ICI Max Day (Base Year) = 1306 m3/day
*future ICI development is all within network, not assumed to be extension of existing ICI users



APPENDIX

C WATER MODEL RESULTS



Legend

- W Well
- S Storage Tank
- BPS Booster Pumping Station (BPS)
- WTP Water Treatment Plant (WTP)
- Pipe
- Road
- Waterbody

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April 04 2016

Sudbury Water and Wastewater Master Plan

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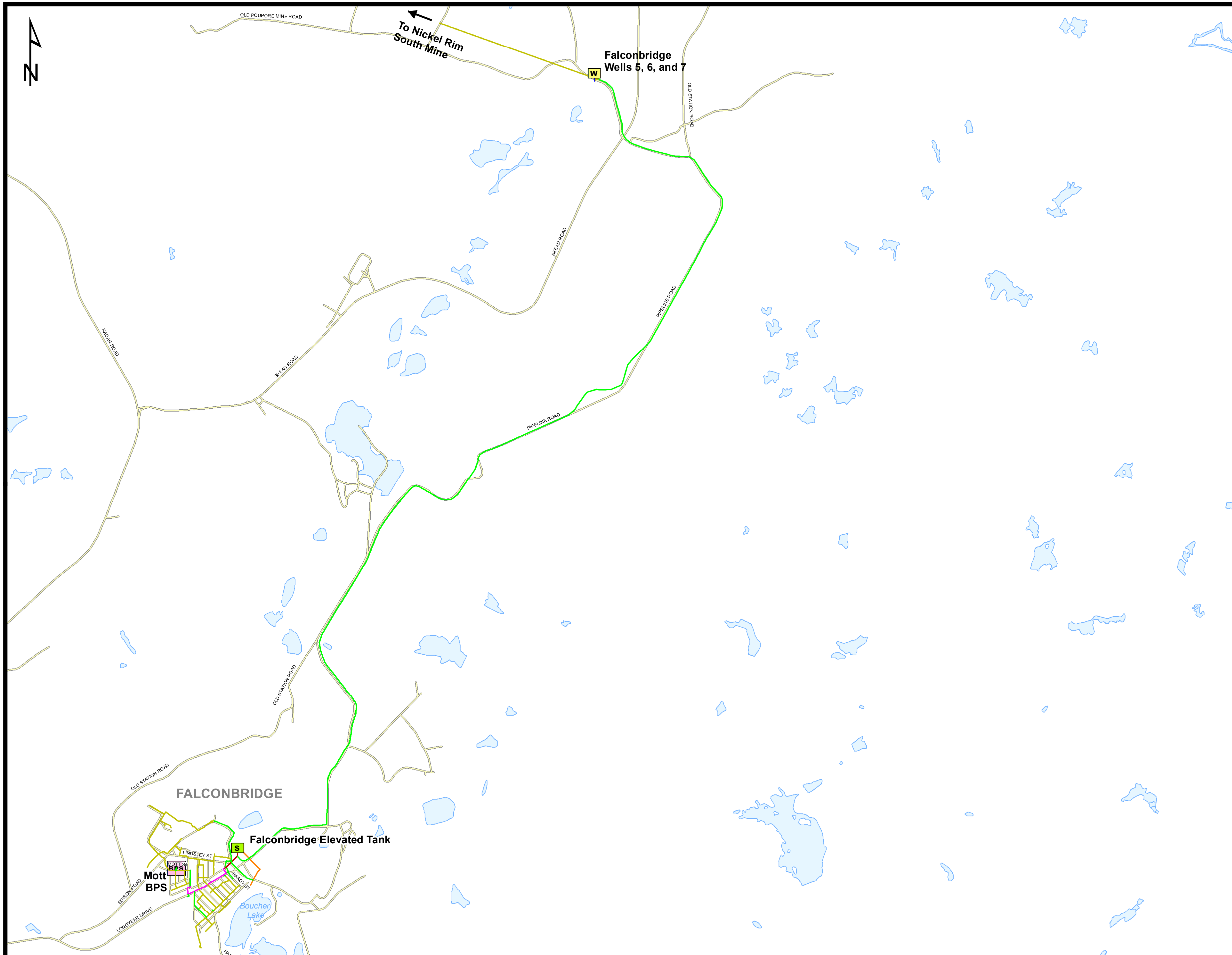
Map 1

Falconbridge Water System - Key Plan



Legend

- W Well
- S Storage Tank
- BPS Booster Pumping Station (BPS)
- WTP Water Treatment Plant (WTP)
- Road
- Waterbody

Pipe: Diamater (mm)

- <= 150
- <= 200
- <= 250
- <= 300
- <= 400
- <= 600
- > 600

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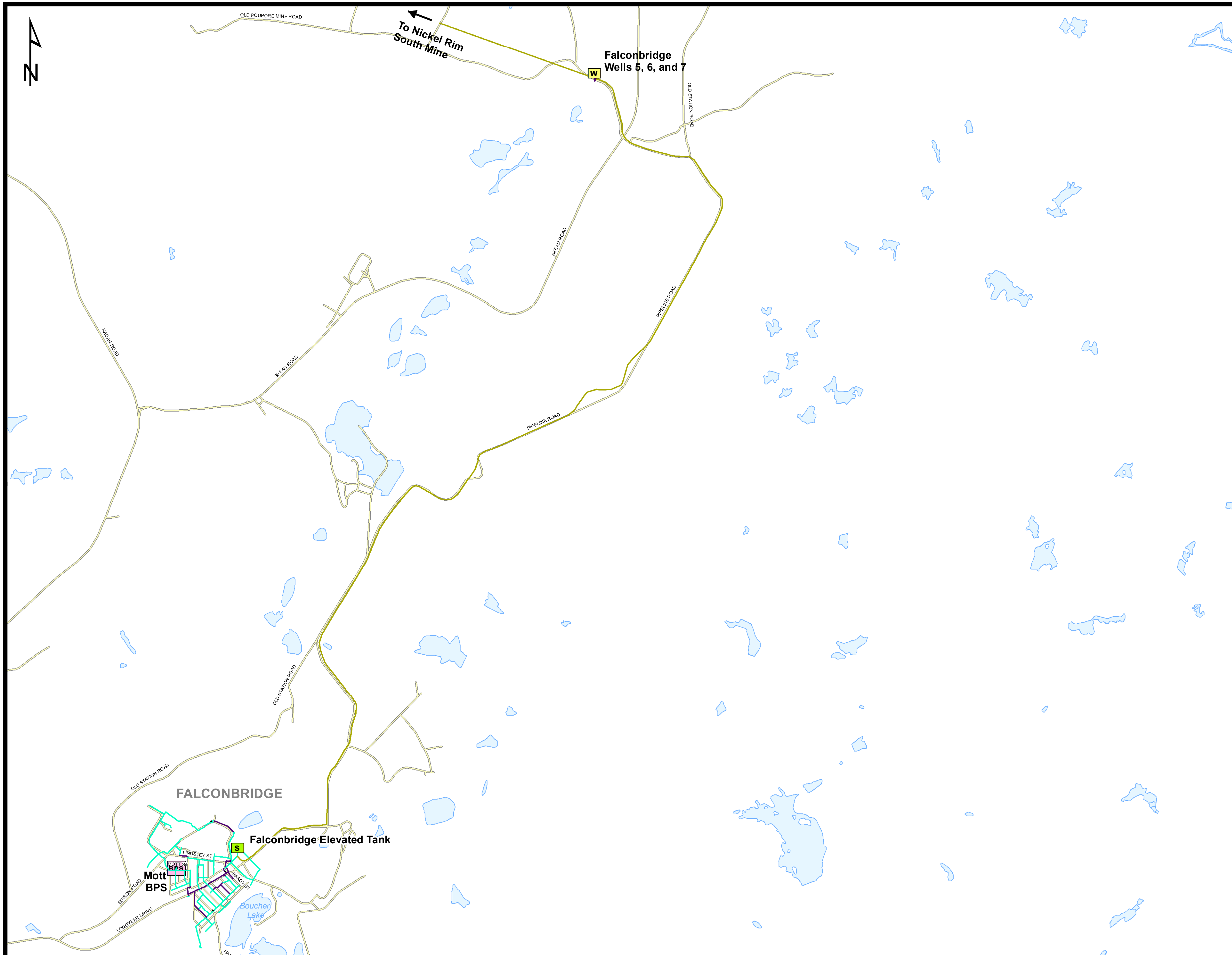
Map 2

Falconbridge Water System - Pipe Diameter



Legend

- W — Well
- S — Storage Tank
- BPS — Booster Pumping Station (BPS)
- WTP — Water Treatment Plant (WTP)
- Road
- Waterbody

Material

- Asbestos Cement
- Cast iron
- Concrete
- Copper
- Ductile Iron
- Galvanized iron
- PVC
- Polyethylene
- Steel

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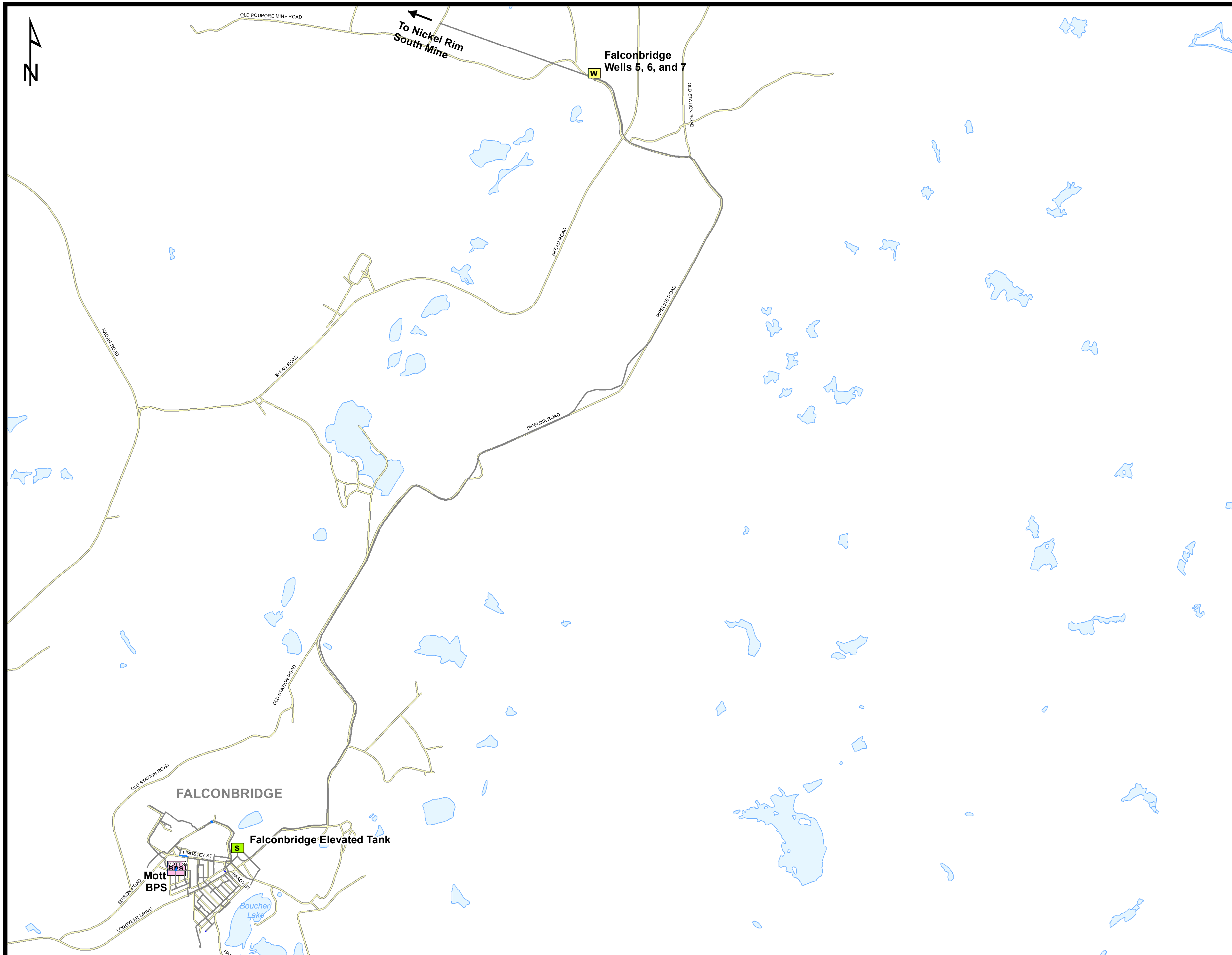
Map 3

Falconbridge Water System - Pipe Material



Legend

- W Well
- S Storage Tank
- BPS Booster Pumping Station (BPS)
- WTP Water Treatment Plant (WTP)
- Road
- Waterbody

Installed Year

- N/A
- <= 1930
- <= 1940
- <= 1950
- <= 1960
- <= 1970
- <= 1980
- <= 1990
- <= 2000
- <= 2010
- > 2011

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Sudbury Water and Wastewater Master Plan

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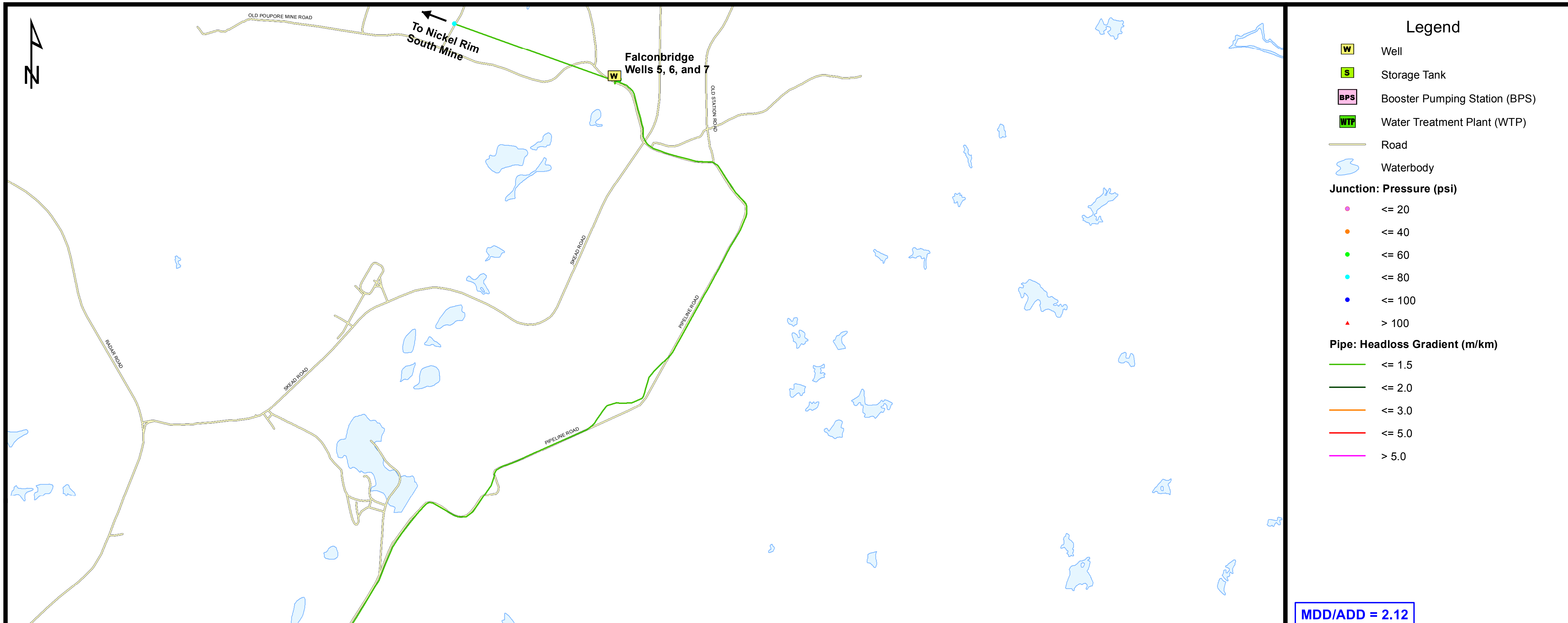
Map 4

Falconbridge Water System - Pipe Age



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0 500 1,000 1,500 2,000 m

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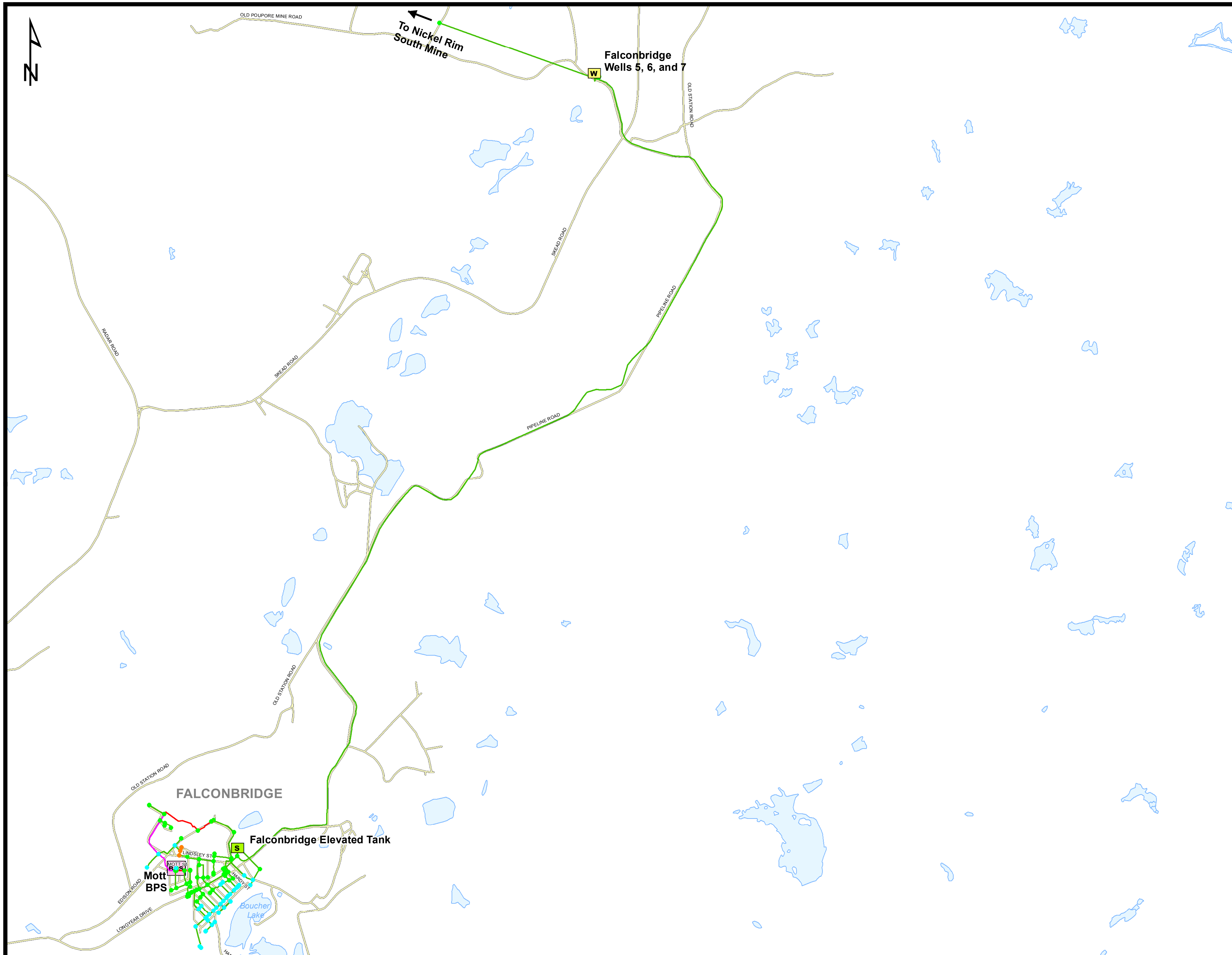
Map 5

Falconbridge Water System - Pipe Headloss and Junction Pressure

Scenario: 2011ADD



Legend

- Well
- Storage Tank
- Booster Pumping Station (BPS)
- Water Treatment Plant (WTP)
- Road
- Waterbody

Junction: Pressure (psi)

- <= 20
- <= 40
- <= 60
- <= 80
- <= 100
- > 100

Pipe: Headloss Gradient (m/km)

- <= 1.5
- <= 2.0
- <= 3.0
- <= 5.0
- > 5.0

MDD/ADD = 2.12

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Sudbury Water and Wastewater Master Plan

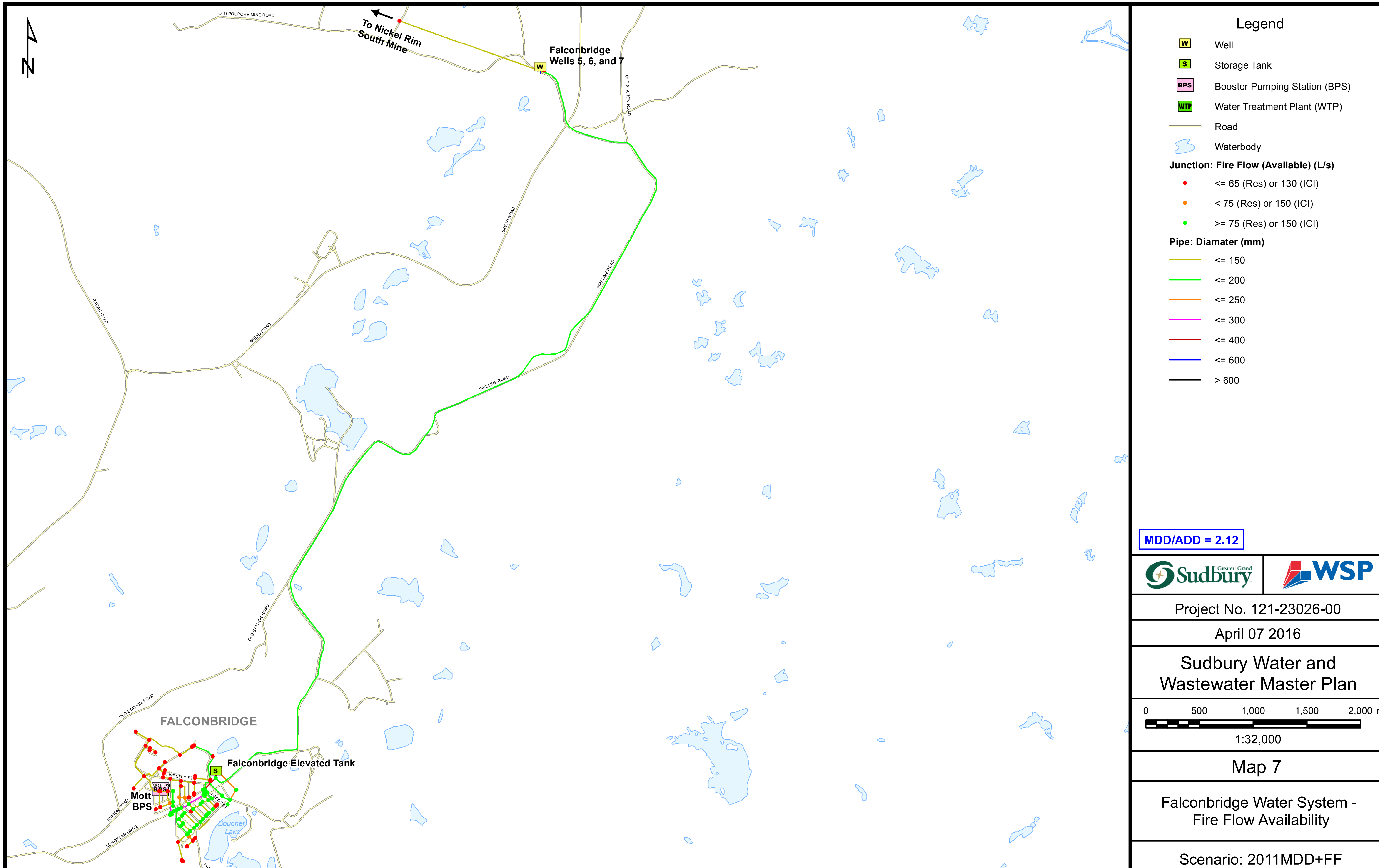
0 500 1,000 1,500 2,000 m

1:32,000

Map 6

Falconbridge Water System - Pipe Headloss and Junction Pressure

Scenario: 2011MDD



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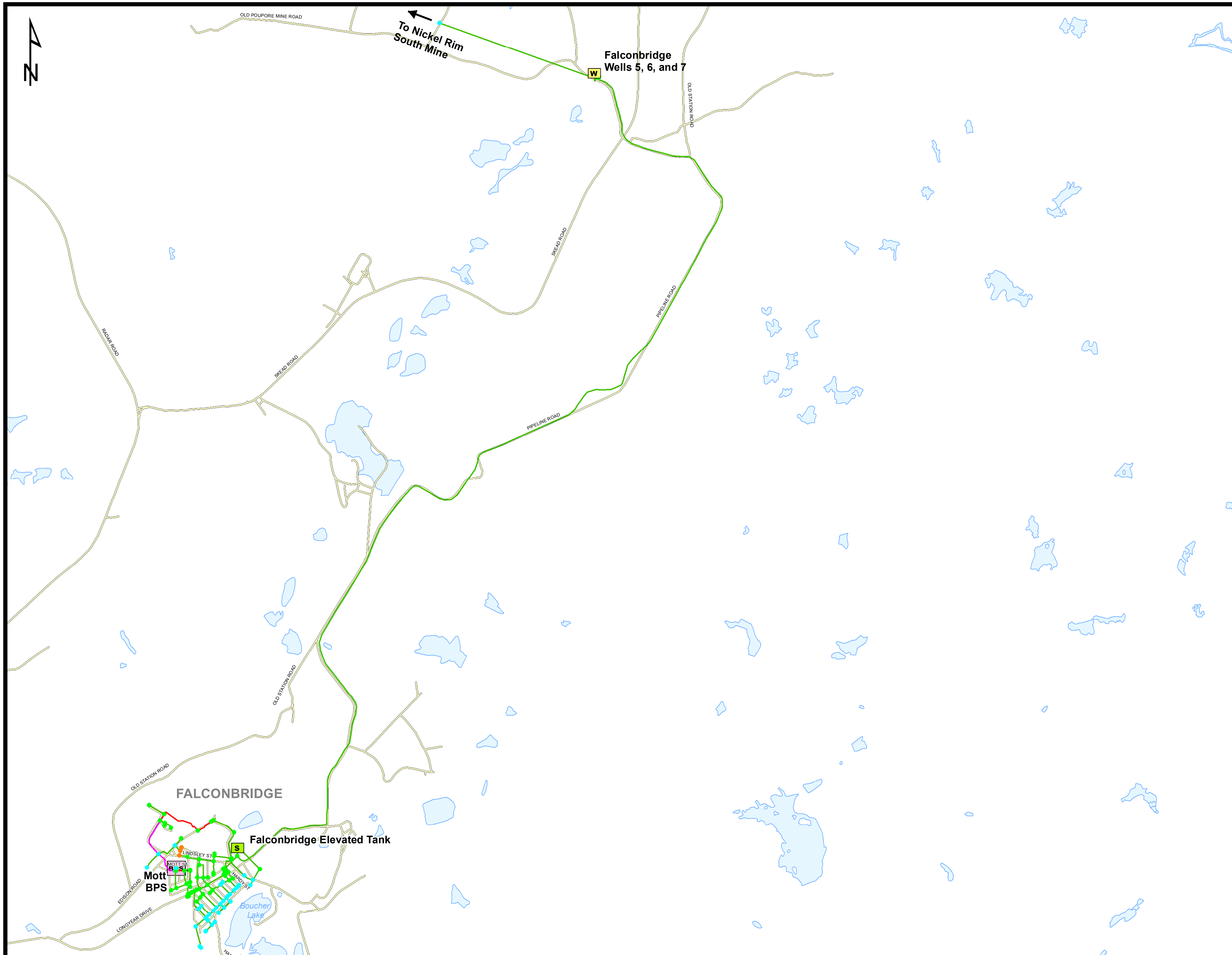
Sudbury Water and Wastewater Master Plan

Map 7

Falconbridge Water System - Fire Flow Availability

Scenario: 2011MDD+FF



Legend

- W Well
- S Storage Tank
- BPS Booster Pumping Station (BPS)
- WTP Water Treatment Plant (WTP)
- Road
- Waterbody

Junction: Pressure (psi)

- <= 20
- <= 40
- <= 60
- <= 80
- <= 100
- > 100

Pipe: Headloss Gradient (m/km)

- <= 1.5
- <= 2.0
- <= 3.0
- <= 5.0
- > 5.0

MDD/ADD = 2.12

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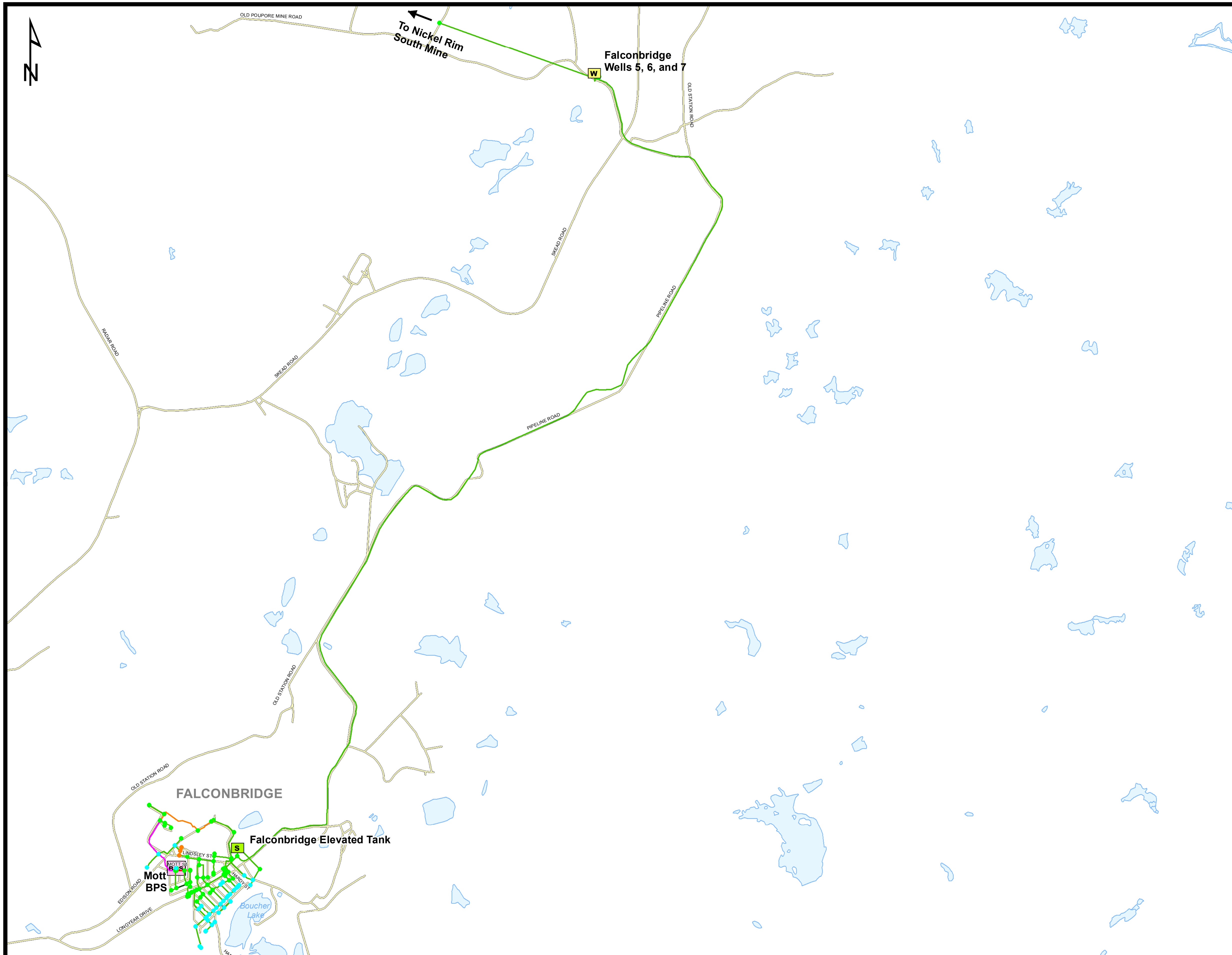
Map 8

Falconbridge Water System - Pipe Headloss and Junction Pressure

Scenario: 2041ADD



Legend

W	Well
S	Storage Tank
BPS	Booster Pumping Station (BPS)
WTP	Water Treatment Plant (WTP)
	Road
	Waterbody

Junction: Pressure (psi)

- <= 20
- <= 40
- <= 60
- <= 80
- <= 100
- > 100

Pipe: Headloss Gradient (m/km)

- <= 1.5
- <= 2.0
- <= 3.0
- <= 5.0
- > 5.0

MDD/ADD = 2.12

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Sudbury Water and Wastewater Master Plan

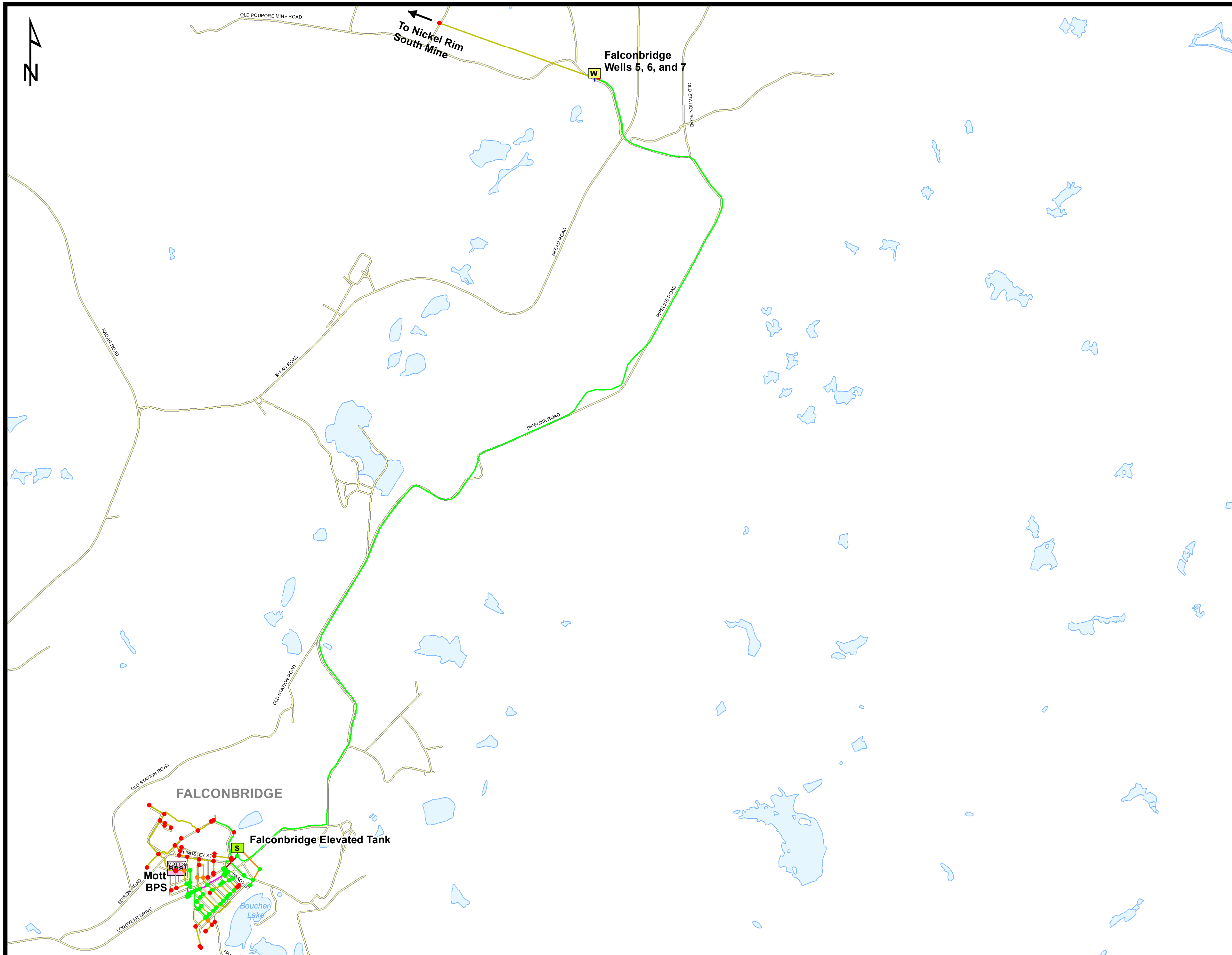
0 500 1,000 1,500 2,000 m

1:32,000

Map 9

Falconbridge Water System - Pipe Headloss and Junction Pressure

Scenario: 2041MDD



Legend

- W Well
- S Storage Tank
- BPS Booster Pumping Station (BPS)
- WTP Water Treatment Plant (WTP)
- Road
- Waterbody

Junction: Fire Flow (Available) (L/s)

- <= 65 (Res) or 130 (ICI)
- < 75 (Res) or 150 (ICI)
- >= 75 (Res) or 150 (ICI)

Pipe: Diamater (mm)

- <= 150
- <= 200
- <= 250
- <= 300
- <= 400
- <= 600
- > 600

MDD/ADD = 2.12

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Sudbury Water and Wastewater Master Plan

0 500 1,000 1,500 2,000 m

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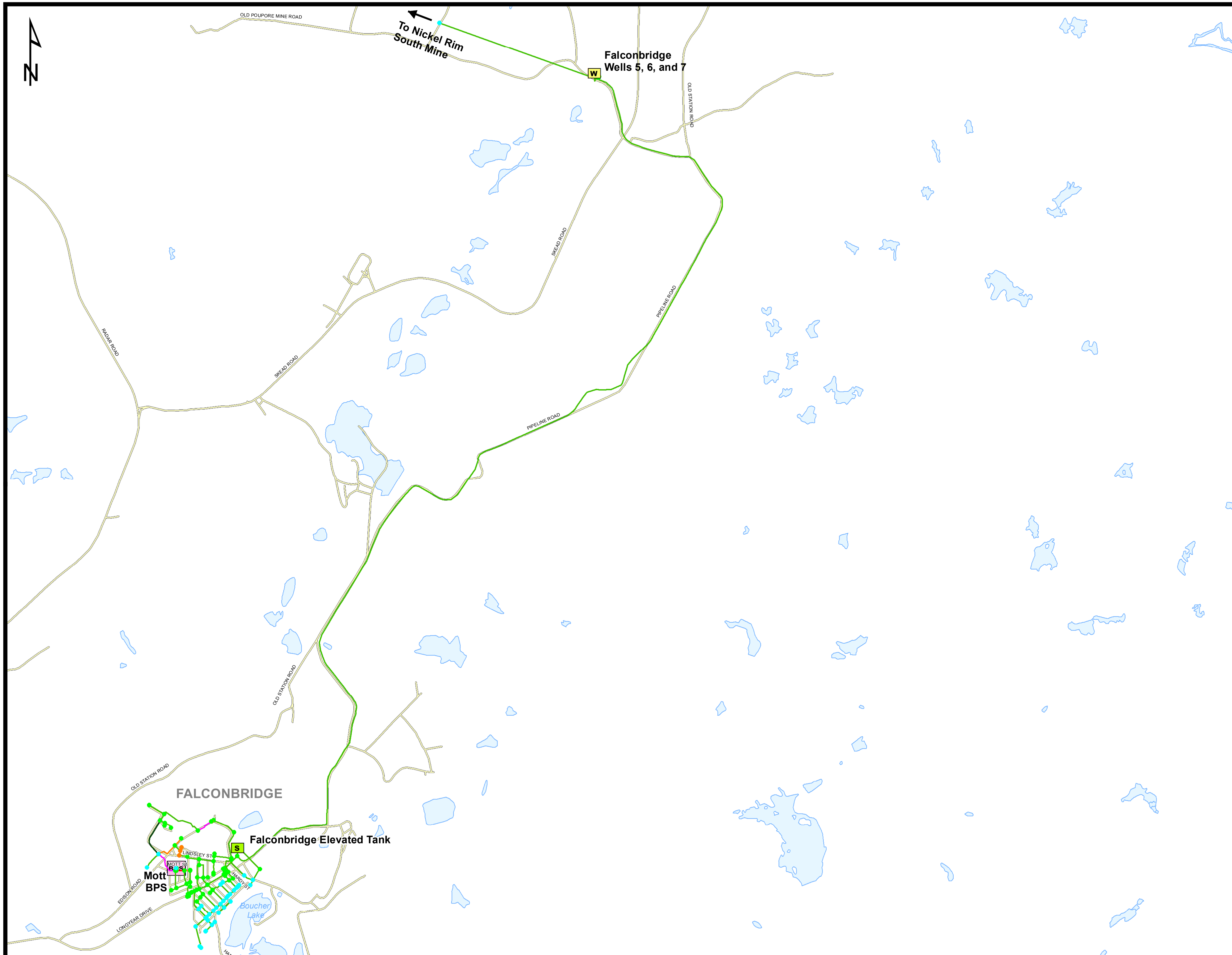
Map 10

Falconbridge Water System - Fire Flow Availability

Scenario: 2041MDD+FF



Legend

- W Well
- S Storage Tank
- BPS Booster Pumping Station (BPS)
- WTP Water Treatment Plant (WTP)
- Road
- Waterbody

Junction: Pressure (psi)

- <= 20
- <= 40
- <= 60
- <= 80
- <= 100
- > 100

Pipe: Headloss Gradient (m/km)

- <= 1.5
- <= 2.0
- <= 3.0
- <= 5.0
- > 5.0

MDD/ADD = 2.12

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Sudbury Water and Wastewater Master Plan

0 500 1,000 1,500 2,000 m

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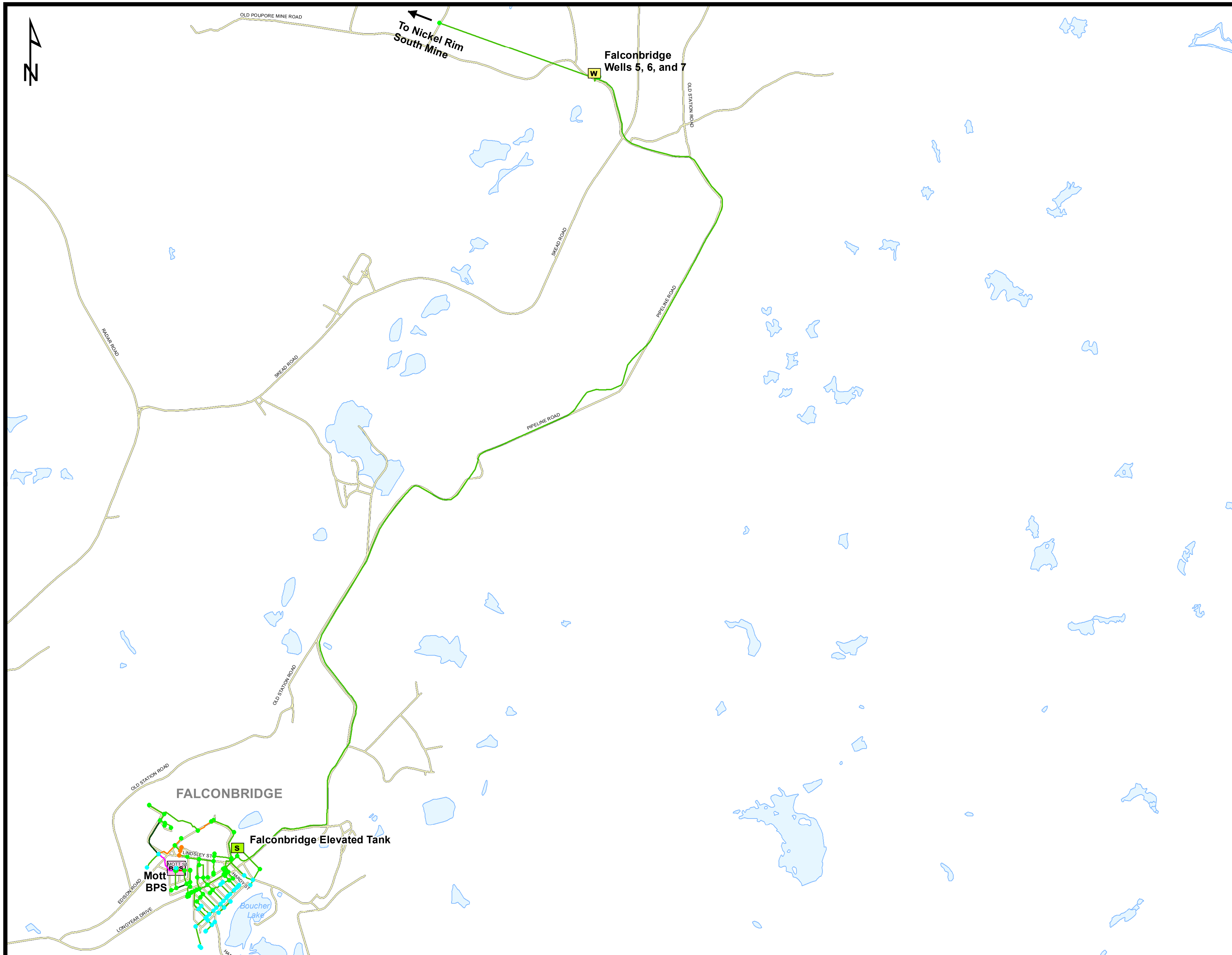
Map 11

Falconbridge Water System - Pipe Headloss and Junction Pressure

Scenario: ULTADD



Legend

- W Well
- S Storage Tank
- BPS Booster Pumping Station (BPS)
- WTP Water Treatment Plant (WTP)
- Road
- Waterbody

Junction: Pressure (psi)

- <= 20
- <= 40
- <= 60
- <= 80
- <= 100
- > 100

Pipe: Headloss Gradient (m/km)

- <= 1.5
- <= 2.0
- <= 3.0
- <= 5.0
- > 5.0

MDD/ADD = 2.12

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0 500 1,000 1,500 2,000 m

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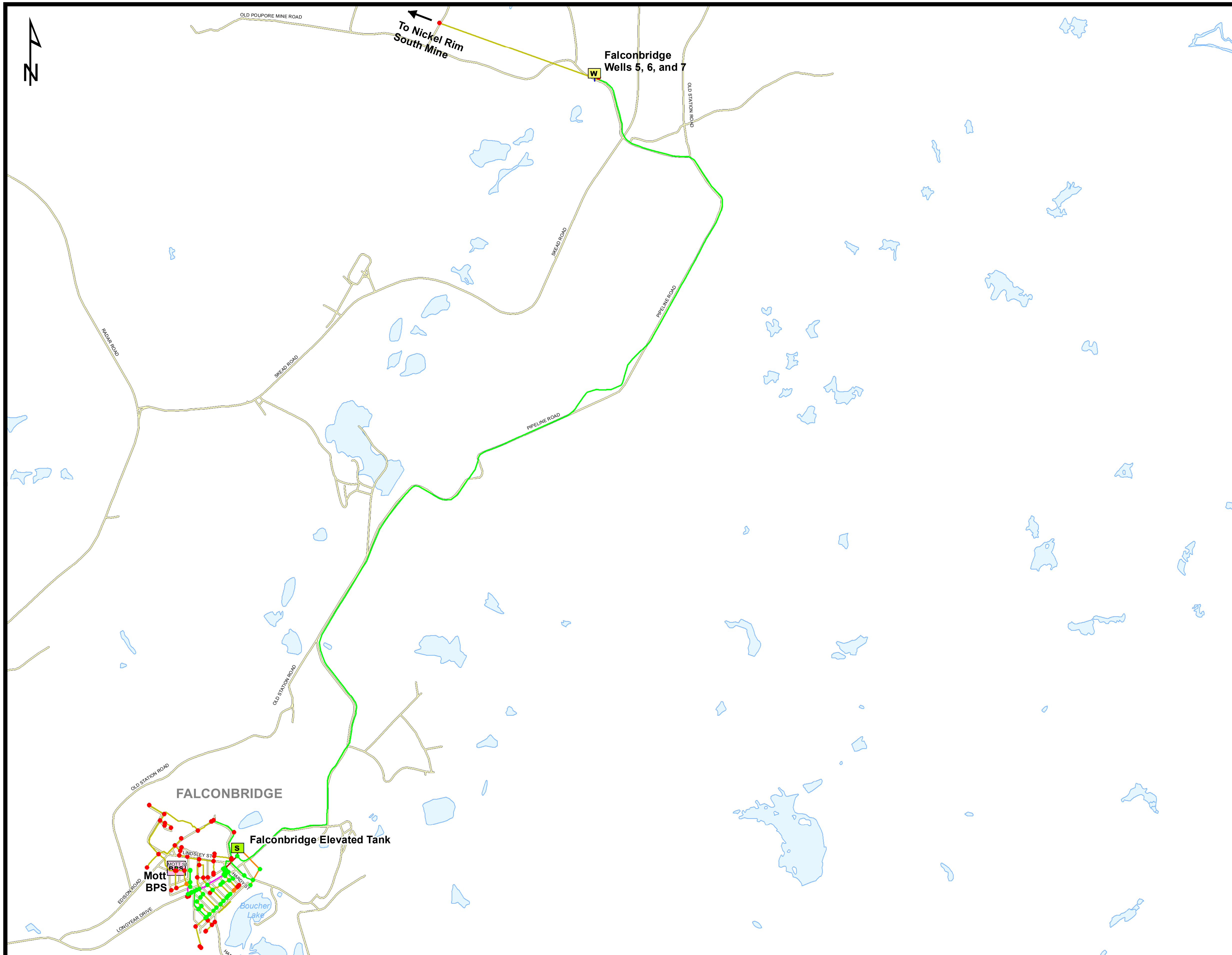
Map 12

Falconbridge Water System - Pipe Headloss and Junction Pressure

Scenario: ULTMDD



Legend

- W Well
- S Storage Tank
- BPS Booster Pumping Station (BPS)
- WTP Water Treatment Plant (WTP)
- Road
- Waterbody

Junction: Fire Flow (Available) (L/s)

- <= 65 (Res) or 130 (ICI)
- < 75 (Res) or 150 (ICI)
- >= 75 (Res) or 150 (ICI)

Pipe: Diamater (mm)

- <= 150
- <= 200
- <= 250
- <= 300
- <= 400
- <= 600
- > 600

MDD/ADD = 2.12

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0 500 1,000 1,500 2,000 m

1:32,000

Map 13

Falconbridge Water System - Fire Flow Availability

Scenario: ULTMDD+FF

CITY OF GREATER SUDBURY WATER AND WASTEWATER MASTER PLAN

ONAPING-LEVACK WATER SYSTEM GAP ANALYSIS AND STATUS QUO REPORT

CITY OF GREATER SUDBURY

DRAFT

PROJECT NO.: 121-26026-00
DATE: OCTOBER 2015

WSP
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TEL.: +1 905 882-1100
FAX: +1 905 882-0055
WSP.COM

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APPENDICES

1 INTRODUCTION

The City of Greater Sudbury (CGS) retained WSP to undertake a Water and Wastewater Master Plan. The purpose of the Master Plan project is to establish servicing strategies for water and wastewater infrastructure for the core urban areas and surrounding communities in the City for the next 20 years, as part of the five-year review of the City's Official Plan. The Master Plan will identify potential projects to address the servicing needs for planned growth within the City. It is being conducted in accordance with the requirements set out in the Municipal Class Environmental Assessment (Class EA) document (June 2000 as amended in 2007 and in 2011).

This report includes a capacity review of the existing Onaping-Levack water system. Based on population growth projections and design criteria discussed in the Population and Unit Rates Technical Memorandum (WSP, 2014) water generation projections were developed and used to determine future infrastructure needs to the 2041 and ultimate buildout planning horizons.

This report assumes that the Onaping-Levack Water System would continue to be a stand-alone system. Any potential interconnections between the Onaping-Levack system and other systems are not considered as part of this report. Potential interconnections with other communities will be reviewed under separate cover, as part of the Master Plan.

The conclusions provided in this report will be the basis for the problem definition and evaluation of alternatives conducted as part of the Master Plan.

Additional information on the existing wastewater system is provided in the Baseline Review Report - Water (WSP, 2014).

2 STUDY AREA

Levack and Onaping are small communities located in the north-west end of the City of Greater Sudbury. The system is supplied by a single water system comprised of three wells.

Map 1 in **Appendix A** shows the Levack and Onaping study area and identifies future land use and development areas, including vacant residential and industrial, commercial, and institutional (ICI) areas. Additional information on population growth and development phasing is provided in the Unit Rates and Population Projections Technical Memorandum (WSP, 2014).

Existing development in the study area is mixed, and includes residential as well as industrial land uses. Notably, the Glencore Nickel mine being the most significant user in the system is located on Regional Road 8.

Based on the City's planning data, growth is not significant in Levack and Onaping. The area population is expected to increase from 2,112 in 2011 to 2,159 in 2041 and 2,477 by Ultimate Buildout – a total growth of 365 residents.

ICI growth is expected to be mixed. Generally; however, there is low ICI growth projected in Onaping-Levack. Growth is discussed further in Section 6.1.

3 OVERVIEW OF EXISTING SYSTEM

The Onaping-Levack Water System services the communities of Onaping and Levack. In 2010, the CGS connected the Onaping and Levack Water Systems and formed the Onaping-Levack Supply and Distribution System. This system includes three wells (Onaping Wells No. 3, 4, and 5), the Onaping Elevated Tank, the Craig Mine Tank (not City owned), and a Pressure Control Building (PCB).

Additional information on the existing system is provided in the Baseline Review Report - Water (WSP, 2014).

3.1 ONAPING WELLS

The Onaping wells draw water from a non–GUDI water source. Onaping Wells 3 and 4 are housed in a single pump house while Onaping Well 5 is housed in a separate building that includes the common treatment facility for the entire system. The treatment processes include a chlorine gas system, fluoridation system, polyphosphate addition system and standby power. Sodium hydroxide (caustic soda) is also added to control pH.

The process is illustrated in Figure 3-1 and a summary of the process equipment at each facility is provided in Table 3-1.

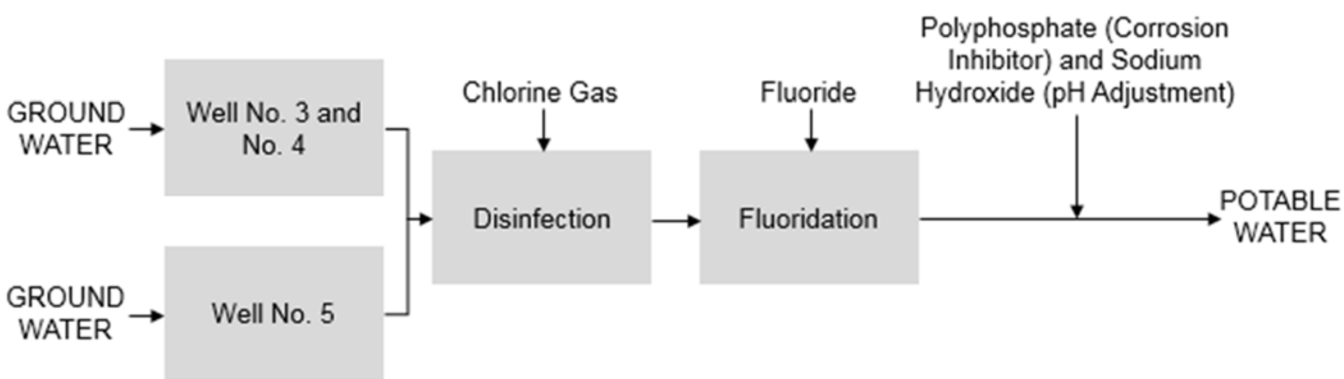

Figure 3-1 Onaping Wells' Process Flow Diagram

Table 3-1 Onaping Wells' Process Information[1]

WELL	PUMP TYPE	OPERATING POINT	STANDBY POWER
Well 3	Vertical turbine pump	30.3 L/s at 83 m TDH	250 kW diesel generator with ATS
Well 4	Vertical turbine pump	30.3 L/s at 83 m TDH	
Well 5	Vertical turbine pump with VFD	60.0 L/s at 83 m TDH	

[1] *Data obtained from the Onaping-Levack DWWP.*

3.2 DISTRIBUTION SYSTEM

The Onaping-Levack distribution system consists of a single pressure district and of the following infrastructure:

— 2,400 m^3 Onaping Elevated Tank

— Pressure Control Building (PCB)

— Fraser Pressure Reducing Valve (PRV)

— A number of watermains

The PCB reduces the pressure of the water entering the Levack network and increases the pressure of the water sent to Glencore's Craig Mine while also limiting the flow (using a valve).

The Craig Mine can use booster pumps for approximately one hour to fill their water tank. When the mine's demands are high, this can occur as frequently as every four hours, putting strain on the City's supply and drawing from the Onaping Elevated Tank.

The system also includes the Fraser PRV. This valve maintains higher pressures at the top of Fraser Avenue and Fraser Crescent and reduces pressure at the bottom of Fraser Avenue.

Table 3-2 Onaping-Levack Water System Storage

TANK	STYLE	DIA. (M)[1]	BASE EL. (M)[1]	LOW WATER LEVEL (M)[1]	HIGH WATER LEVEL (M)[1]	CALCULATED USEABLE VOLUME (M3)[2]	DWWP TOTAL VOLUME (M3)
Onaping Elevated Tank	Elevated	11.6	284	402	414	2,400	2,400

[1] *Obtained from the as-built drawings for the elevated tank.*

[2] *Based on the Onaping-Levack DWWP.*

3.3 KNOWN CHALLENGES

In addition to concerns discussed in previous sections, the Onaping-Levack Water System has the following known challenges:

— City operations staff has indicated that, during harsh winters, watermain services along 1st Avenue and Levack Drive may freeze due to insufficient ground cover. Additionally, some watermains and services in Levack are located in backyards, rather than roads, which poses a challenge for maintenance since staff have limited accessibility to the infrastructure.

— City staff have also reported that the watermain from the Onaping Elevated Tank has high operational pressures. This is primarily due to the difference in elevation between the PCB and the tank.

— Water consumption in Onaping-Levack has been increasing since 2009, despite the fact that the communities' population has not been increasing.

— The increase in water consumption was thought to be the result of the City's program to flow a small stream of additional water in the winter months to prevent freezing in the municipal water network; however, it was determined that water losses through this program are limited (about 1% of the total water losses). The increased water consumption and high unbilled water rate could potentially be partially attributed to the fact the City runs water at the Fraser Lift Station in the winter to prevent freezing.

— The distribution system includes small diameter galvanized watermains located in backyards.

— City staff have indicated that leaks and breaks in this system do not surface and therefore are difficult to locate without tracking spikes in usage.

— City operations has noted that the caustic system was installed as a trial, but became a requirement to prevent lead leaching into private plumbing and therefore potable water. As such, it is undersized, requires frequent maintenance, filling, and upgrades.

4 HISTORICAL FLOWS AND REVIEW OF OPERATIONAL DATA

Water supply data from the Onaping Wells from 2009 to 2013 was reviewed and analyzed for this evaluation. Table 4-1 shows a summary of the data received, and indicates how it was used for the analysis.

Table 4-1 Water Supply Data Reviewed

DATA RECEIVED	PARAMETERS INCLUDED	DATA INTERVAL	USE IN ANALYSIS
Treated flow (2011-2013)	Flow in m^3/d	Hourly	To determine peak hourly flow
Annual Reports (2009-2013)	Total average daily flows, maximum daily flows Treated water characteristics	Daily	To determine average day, max day flow To assess performance of existing process and treated water characteristics
Annual Billing Data (2012)	Annual flow per customer in m^3	Annually	To determine the proportion of total water consumption corresponding to residential users

4.1 FLOW DATA

Water supply data from 2009 to 2013 was reviewed to determine historical water demands in the Onaping-Levack Water System. Average day and maximum day demand data for the past five years, and peak hour data for the past three years (2011-2013) is included in Table 4-2.

Table 4-2 Historical Water Supply Data

YEAR	AVERAGE DAY DEMAND (M3/D)[1]	MAXIMUM DAY DEMAND (M3/D)[1]	PEAK HOUR DEMAND (M3/D)[2]
2009	1,287	2,501	Not Available
2010	1,521	2,459	Not Available
2011	2,010	2,906	4,952
2012	1,687	3,511	5,515
2013	2,033	2,886	5,308

[1] Onaping-Levack Drinking Water System Annual Reports (2009 – 2013).

[2] From hourly SCADA data.

Water demands in Onaping-Levack have increased since 2009, albeit the population has not increased. The average consumption from 2009 to 2013 was 1,708 m^3/d.

The highest maximum day flow recorded in the past five years was 3,511 m^3/d, occurring in 2012. The average historical maximum day demand is 2,853 m^3/d. As such, the max day factor calculated using the 2012 maximum day flown was used to determine the unit rate for future growth.

Hourly flow data was only available from 2011 to 2013, the data used to determine the peak hour flow. The maximum peak hour value recorded during that period was 5,515 m^3/d in 2012, and the average was 5,259 m^3/d.

The peaking factors derived from historical data were compared to those documented in the *City's Engineering Design Manual* (City of Greater Sudbury, 2012) and those included in the *MOECC Guidelines* (MOE, 2008).

The maximum day to average day peaking factor corresponding to the maximum day flow recorded (3,511 m^3/d in 2012) was 2.08, while the average maximum day peaking factor was 1.70. The highest maximum day factor (2.08) was adopted to evaluate future requirements.

The peak hour to average day factor corresponding to the highest peak hour flow recorded in 2012 (5,515 m^3/d) was 3.27, while the average peak hour factor was 2.78.

The *City's Engineering Design Manual* and the *MOECC Guidelines* specify a peak hour factor of 4.13. For purposes of estimating future demands, the historical maximum value (3.27) was adopted, since the historic data simply couldn't support the use of a higher factor.

4.2 RAW WATER CHARACTERISTICS AND SECURITY OF SUPPLY

The Onaping Wells are classified as non-GUDI and have good water quality. However, there have been elevated levels of sodium in the treated water. This is discussed further in the section below.

4.3 OPERATIONAL DATA

Data reported in the Annual Reports for the wells includes effluent chlorine residual, trihalomethanes (THMs), fluoride, and trace organic and inorganic chemicals.

Data was reviewed from 2009 to 2013 to determine any historical issues.

The Onaping-Levack Water System has historically had elevated levels of sodium, ranging generally from 50 to 90 mg/L. Sodium levels greater than 10 mg/L trigger public notification, in accordance with Public Health department requirements.

In addition, there have been historical exceedances of lead in private plumbing in the Onaping-Levack Water System. However, no exceedances of lead in the distribution system were noted.

5 DESIGN CRITERIA

The following design criteria were used to assess the remaining capacity of the existing systems and to forecast future requirements for the water and wastewater systems. The unit rates used to estimate future water and wastewater flows correspond to the values included in the *Population Projections and Unit Rates Technical Memorandum* (WSP, 2014). Otherwise, design criteria recommended in the *MOECC Guidelines* and *City's Engineering Design Manual* were used.

5.1 UNIT WATER DEMAND CRITERIA

The water demand criteria shown in Table 5-1 are from the unit rates recommended in the *Populations and Unit Rates Technical Memorandum* (WSP, 2014). The rates were reviewed against historical data, MOECC *Guidelines* (MOE, 2008), and current standards in the City's *Engineering Design Manual* (City of Greater Sudbury, 2012).

Both the *MOECC Guidelines* and *City Engineering Design Manual* recommend determining demands for institutional, commercial and industrial (ICI) users on a case by case basis. However, the following criteria for ICI demands were used for the purposes of this evaluation.

Table 5-1 Water System Design Criteria

CRITERIA	VALUE	REFERENCE
Average Day Residential Flow	350 L/cap/day	City's Engineering Design Manual, rounded down from 410 L/ca/d
Average Day Commercial and Institutional Flow	28 m^3/ha/d	MOECC guidelines
Average Day Industrial Flow	35 m^3/ha/d	MOECC guidelines
Domestic Demand Maximum Day Factor	1.70	Average of historical values
Domestic Demand Peak Hour Factor	3.27	Maximum of historical values

Residential average day demands are obtained by multiplying the residential unit rate by the service population. Similarly, average ICI demands are obtained by multiplying the corresponding unit rates to the areas of development, assuming 100% of the area would be developed and assuming 100% lot coverage on these properties.

Maximum day and peak hour demands are obtained by multiplying the average day demand by the corresponding peaking factor.

For purposes of this study, and in line with city standards and practices, a residential fire flow of 75 L/s over 1.75 hours and ICI fire flow of 150L/s over 2 hours were used.

5.2 DESIGN CRITERIA FOR WATER SYSTEM COMPONENTS AND OPERATION

5.2.1 TREATMENT CAPACITY

Water supply facilities are designed to supply the maximum day demands of the system.

Treatment facilities must be designed in accordance with the *Procedure for Disinfection of Drinking Water in Ontario* (Ontario, 2006).

5.2.2 PUMING CAPACITY

Pumping stations are rated based on their firm capacity. If sufficient floating storage is available in a particular pressure district, the MOECC defines firm capacity as the capacity of the station with the largest pump out of service. If there is insufficient or no floating storage, firm capacity is defined as the capacity with the two (2) largest pumps out of service (MOE, 2008).

For each pressure district, the pumping stations have to be designed to provide peak hour or maximum day plus fire demands (whichever are greater), if no floating storage is available. If sufficient floating storage is available, then the pumping station only needs to be designed to provide maximum day demands.

5.2.3 STORAGE CAPACITY

Storage requirements are based on the requirement to meet water demands that exceed the capacity of the treatment plant and to satisfy fire flow demands. When the capacity of the supply system is only capable of satisfying maximum day demands, storage requirements are determined using the following formula from the *MOE Guidelines* (MOE, 2008):

$$\mathbf{Storage = A + B + C}$$

Where: A = Fire Storage, B = Equalization Storage = 25% of maximum day demand, and C = emergency storage = 25% of (A+B).

Fire storage is the product of the maximum fire flow required in the system and the corresponding fire duration based on Fire Underwriters requirements (Fire Underwriters Survey, 1999).

When the system can supply more than just the maximum day demand (but less than the peak demand), the fire storage requirements can be determined using the following formula:

$$\mathbf{A = (Peak\ Demand - Pumping\ Station\ Firm\ Capacity) \times Fire\ Duration}$$

Where: peak demand is the greater of the peak hour demand and the maximum day plus fire demand.

Per *MOECC Guidelines*, floating storage should be designed such that the elevation of the equalization volume (B) is such that a minimum pressure of 275 kPa (40 psi) can be maintained in the system under peak hour flow conditions. The fire (A) and emergency (C) volumes should be at elevations that produce 275 kPa (40 psi) during peak hour demand conditions, and 140 kPa (20 psi) under the maximum day plus fire flow condition (MOE, 2008).

5.2.4 DISTRIBUTION CAPACITY

Watermains have to be sized to carry the greater of the maximum day plus fire flow or peak hour demand. The MOECC Guidelines recommend that the range of acceptable pressures under normal conditions (average to peak hour flows) is 275 kPa (40 psi) to 690 kPa (100 psi), while during fire flow conditions pressures may drop to 140 kPa (20 psi) (MOE, 2008). The maximum allowable water velocity in the distribution system is 3 m/s (MOE, 2008).

6 FUTURE REQUIREMENTS

6.1 POPULATION PROJECTIONS

As part of the City of Greater Sudbury Master Plan, population forecasts were developed for the 2016, 2021, 2026, 2031, 2036, 2041 and Ultimate Buildout planning years. Ultimate Buildout is defined as an estimate of what the demand from the total population and total number of households in the City of Greater Sudbury would be based on lands that are currently designated for development in the Official Plan within the existing settlement boundaries.

The City supplied planning data sheets with properties and development potential and the vacant residential and ICI land inventory, and Hemson Consultants, on behalf of the City, provided supplementary population projections. Data was provided for each water and wastewater system boundary. These data were used in conjunction to develop the targeted population growth for each horizon year, as well as development phasing (discussed in the next section and in detail in the *Populations and Unit Rates Technical Memorandum*, WSP 2014).

In cases where the City's planning data sheets and Hemson's population projections forecasted fewer development units than the vacant land inventory for an area, then specific parcels (up to the City's and Hemson's unit projections) of developable units were selected. These parcels were selected based on the rationale provided in the City's Official Plan. That is, the Official Plan prioritizes that development take place in areas that are currently serviced, or where servicing can easily be extended. This focuses growth in existing urban areas until supply is no longer available in these areas.

Based on the planning data, the population Onaping-Levack is projected to increase by 47 people in 2041 and 365 by Ultimate Buildout. The population projections to be used in the Master Plan are summarized in Table 6-1.

Table 6-1 Onaping-Levack Water System Population Projections

	2011	2016	2021	2026	2031	2036	2041	ULTIMATE BUILDOUT
Onaping-Levack	2,112	2,123	2,135	2,146	2,154	2,159	2,159	2,477

The City's planning data does not specify target years for employment growth. However, vacant lands designated as institutional, commercial, and industrial (ICI) properties have been assigned to different stages of the development process by the City. These stages are described below.

- Draft Approved:
  - These are lands that have draft plan of subdivision approval under the Planning Act or have pending applications with the City. Typically, these lands are close to registration or few years away from development as the required conditions are satisfied
  - Development approvals are near complete, and development could take place at any time. Properties with this designation were set to take place in 2016.
- Legal Lots of Record:
  - These are existing lots, including lots in a registered plan of subdivision. Typically these lands are zoned, serviceable and only require building permit approval for development. In some cases a site plan approval/agreement may also be required.
  - Based on historical trends, development is approximately 15 years away from receiving draft approval. Properties with these designations were assigned to take place in 2026.
- Designated Developable:

- These lands do not have any development approvals in place but are understood to be areas of future development as they are within the settlement boundary. Designated lands are typically a number of years away from being developed.
- Based on historical trends, these properties are approximately 10 years away from receiving Legal Lot of Record designation. Designated Developable properties were assumed to take place in 2036.

These land supply categories stem from the land supply requirements that municipalities must maintain under Section 1.4 of the Provincial Policy Statement. In this context, Designated Development Lands would count towards Section 1.4.1 (a) and Legal Lots of record and Draft Approved Lands would count towards 1.4.1 (b). It is also important to note that the total supply is governed by PPS Section 1.1.2.

The targeted ICI development areas for each horizon year are summarized in Table 6-2.

Table 6-2 Onaping-Levack ICI Population Projections

	ICI DEVELOPMENT AREAS (HA)					
LAND USE	2016	2021	2026	2031	2036	2041
Institutional	0.00	0.00	0.00	0.00	2.78	0.00
Commercial	0.00	0.00	0.71	0.00	0.00	0.00
Industrial	0.00	0.00	0.00	0.00	1.86	0.00
Total	**0.00**	**0.00**	**0.71**	**0.00**	**4.64**	**0.00**

The above assumptions provide an estimate as to the ICI development time line. In reality, development may be more staggered. However, for purposes of infrastructure planning and to ensure that the appropriate infrastructure is in place by the appropriate planning horizon, the above assumptions are considered to be conservative.

6.2 PHASING OF FUTURE GROWTH

Growth areas were allocated based on population projections for individual developments and the overall target growth population projections for the Levack and Onaping area for the horizon years.

Hemson's supplementary tables were used to provide the target population, while the City's planning tables and vacant lot inventory were used to identify phasing of specific properties, and assignment of draft approved, legal lots of record, and designated development properties. In general, priority was given to draft approved properties, followed by legal lots of record and designated developable properties. In accordance with the Official Plan, the City has also assigned a target quantity of legal lots of record and designated developable properties to be developed in each horizon year. That is, legal lots of record should account for approximately 20% of all household growth, and designated developable lots are assigned 20% of the 20 year anticipated growth.

The future growth phasing plans were presented in the *Unit Rates and Population Projections Technical* Memorandum (WSP, 2014).

6.3 FUTURE WATER DEMAND PROJECTIONS AND INFRASTRUCTURE NEEDS

The unit flow criteria listed in Section 5.1 were used to estimate the future water demands in the Onaping-Levack Water System. In general, the projected flows were calculated by the following formula:

$$\begin{aligned}&\textbf{Projected Average Day Demand}\\&\qquad = \textbf{Base Demand} + \textbf{Additional Residential Demand} + \textbf{Additional ICI Demand}\end{aligned}$$

The demands corresponding to the population growth forecasts to ultimate buildout are listed in Table 6-3.

Table 6-3 Water Demand Projections for the Onaping-Levack Water System

YEAR	POPULATION	AVERAGE DAY DEMAND (M3/D)	MAXIMUM DAY DEMAND (M3/D)	PEAK HOUR DEMAND (M3/D)
Base	2,112	1,708	2,853	5,259
2016	2,123	1,712	2,910	5,596
2021	2,135	1,716	2,917	5,609
2026	2,146	1,739	2,957	5,687
2031	2,154	1,742	2,962	5,696
2036	2,159	1,887	3,208	6,169
2041	2,159	1,887	3,208	6,169
Ultimate Buildout	2,477	1,998	3,397	6,533

The Base Demands were the highest historical (2009 to 2013) average day and maximum day demand as well as peak hour for the community. The additional residential demand was calculated using the unit flow rate multiplied by the population growth, and similarly, the ICI demand was calculated using the unit flow rate for each type of development (industrial, commercial or institutional), multiplied by the growth in development area.

Maximum day and peak hour demands were calculated by applying the respective peaking factor to the average day demand.

A desktop analysis of historical water demands and future water demand projections is included in **Appendix B**.

6.3.1 SUPPLY CAPACITY

The Onaping-Levack Water System is supplied by 3 wells, all located in Onaping. The rated combined capacity of the Onaping Wells, as listed in the facility's PTTW is 5,237 m^3/d. This value was used for comparison against future needs of the Onaping-Levack System.

In accordance with the PTTW, the total water permitted to be taken from the well field may not exceed 5,237 m^3/d. That is, the PTTW allows pumping from a single well or a combination of wells, provided that the total volume taken is no more than 5,237 m^3/d.

The projected maximum day demands are plotted versus the total rated and firm production system capacities on Figure 6-1.

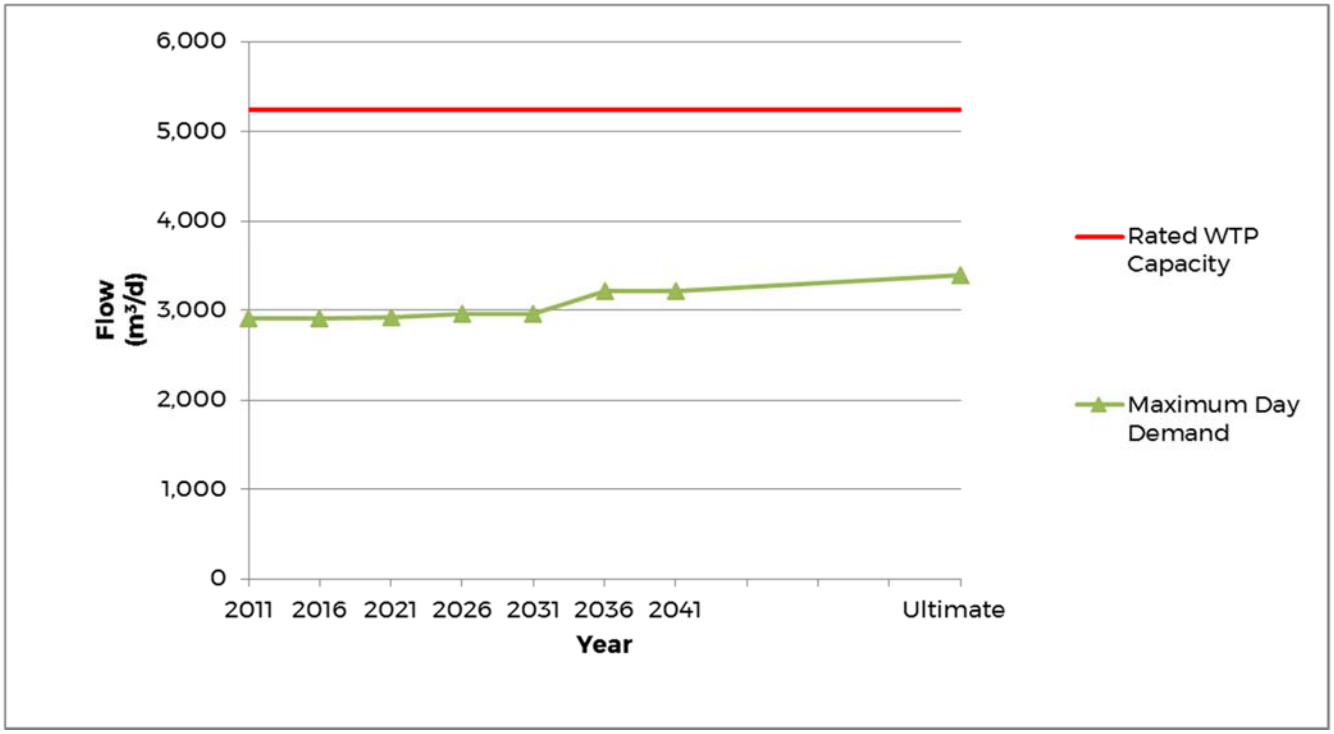

Figure 6-1 Water Demand Projections Compared to Rated Total and Estimated Firm Capacities

Therefore, the Onaping-Levack Water System has sufficient maximum day capacity to service planned population growth to Ultimate Buildout.

6.3.2 STORAGE CAPACITY

Storage in the distribution system is provided by the Onaping Elevated Tank. The tank has a usable volume of 2,400 m^3 and its low and high water elevations are 402 m and 414 m, respectfully.

Applying the MOECC A+B+C formula to determine storage requirements, the corresponding fire storage requirement would be 1,080 m^3. Using the maximum day demand required to service growth to 2041 (3,208 m^3/d), the corresponding equalization storage requirement would be 802 m^3 and the emergency storage would be 471 m^3. The total required storage to service growth to 2041 would be 2,353 m^3 and the total required storage to service the Ultimate Buildout growth scenario would be 2,411 m^3. Therefore, the existing total storage volume of 2,400 m^3 provides sufficient storage for the Onaping-Levack Water System to service growth to 2041 and Ultimate Buildout as the difference of 11 m^3 at Ultimate Buildout is negligible.

The amount of storage required for each horizon year is shown in the figure below.

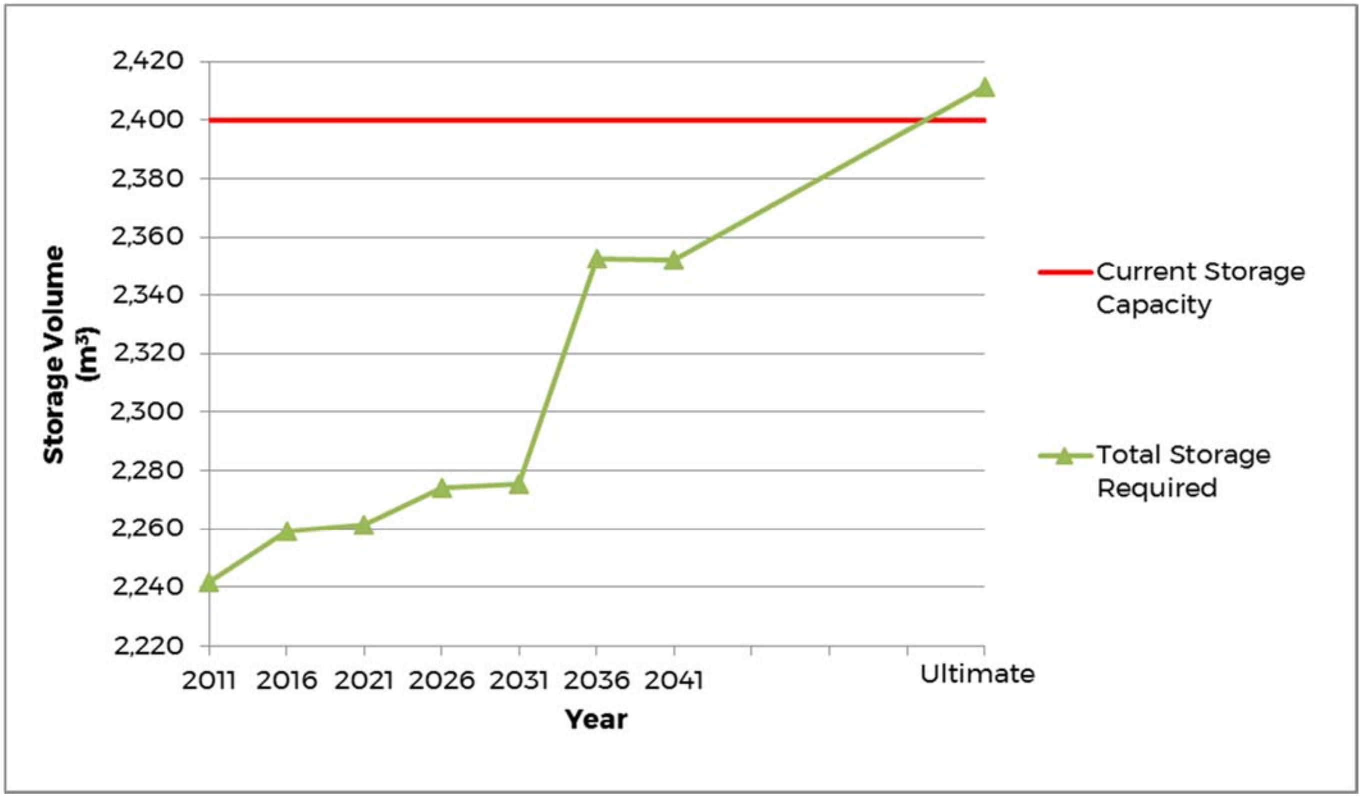

Figure 6-2 Available Storage Capacity Compared to Future Needs

6.3.3 DISTRIBUTION NETWORK

The water model was used to identify system elements (i.e. watermains, pumps, storage tank) for which the capacity was exceeded by the projected water demands. The capacity of the system was assessed in terms of the available fire flows and system pressures.

For each planning scenario, watermains of the modelled network were reviewed to assess whether the required minimum fire flows (75 L/s in residential areas or 150 L/s in ICI areas) and pressures (over 20 psi under fire conditions and over 40 psi under normal conditions) were achieved. Furthermore, some new watermains were added to service greenfield areas where development was planned. A simplified watermain layout was assumed for these areas.

Future populations and demands were loaded into the model based on the planning data and flow projections discussed in earlier in Section 6.3. In general, development might deviate from the proposed phasing scheme. Thus, it is recommended that the hydraulic water model be updated whenever a development application is submitted.

The findings from the water modeling are discussed in Section 7 and presented in **Appendix C**.

7 HYDRAULIC MODELING

An all-pipe model of the system including pipes, hydrants, storage tanks and system source was developed by the City using Bentley Systems' WaterGEMS hydraulic modeling software. This model was updated based on information provided by the City to reflect current system conditions.

The water model allows for simulations to be conducted that can be used to predict system responses to events under a wide range of conditions. Using simulations, problems can be anticipated in proposed or existing systems, and solutions can be evaluated before time, money, and materials are invested in a real-world project. Simulations can either be steady-state or extended-period. Steady-state simulations represent a snapshot in time and are used to determine the operating behaviour of a system under static conditions. This type of analysis can be useful in determining the short-term effect of fire flows or average demand conditions on the system. Extended period simulations (EPS) are used to evaluate system performance over time. This type of analysis allows modeling the filling and emptying of storage facilities, regulating valves opening and closing, and pressures and flow rates changing throughout the system in response to varying demand conditions and automatic control strategies.

Simulations including steady-state analysis of the Average Day, Maximum Day and Maximum Day + Fire conditions were carried out using the model. Fire flow simulations were carried out throughout the system to determine whether the system could deliver fire flows under the Maximum Day demands.

7.1 WATER MODEL DEVELOPMENT

To model the current scenario, the following steps were taken:

- Total network demand on an average day basis was determined for the current scenario using 2012 water production data.
- The node demand allocations assigned in the model were based on 2012 meter records. Metered flows were assigned to the respective property. In cases where meter records showed zero flow, the value was manually adjusted to reflect a reasonable volume for a respective property, depending on land use.
- The maximum day peaking factor was applied to the average day demand value to determine the maximum day demand.
- The maximum day demand plus fire flow was used to assess the system since it was greater than the peak hour demand.

7.2 MODELING FINDINGS

7.2.1 FIREFIGHTING CAPACITY

An assessment of the available fire flows was conducted using the hydraulic model. As noted above, a fire flow requirement of 150 L/s was estimated for ICI areas, while a value of 75 L/s was adopted for residential areas. The model revealed that, under 2011, 2041 and Ultimate Buildout conditions, fire flows are not met at some of the dead ends in the system. Water model outputs, including maps showing fire flow analysis, are provided in **Appendix C.**

7.2.2 MODELLED HYDRAULIC CAPACITY UNDER NORMAL CONDITIONS

Based on the system modeling, service pressures throughout the system under the maximum day demand scenario generally range between 40 and 100 psi for all scenarios, apart for a few exceptions noted below. Therefore, flows throughout the system are within the range prescribed in the MOECC Guidelines (40 to 100 psi) under normal conditions.

The two exceptions are the pressures within the watermains near the Craig Mine Standpipe as well as the watermain that feeds the Craig Mine Standpipe, which were indicated to have pressure upwards of 100 psi.

Though pressures greater than 100 psi are usually a concern that is not the case for the watermain for the Craig Mine supply. The high pressures in the system are in an area that is low-lying and that does not include service connections. The pressure in the watermain feeding the Craig Mine is also higher than 100 psi, due to the elevation difference between the PCB and the receiving tank.

Maps showing pressures at nodes are presented in **Appendix C**.

8 CONCLUSIONS AND RECOMMENDATIONS

An assessment of the Onaping-Levack Water System was completed to identify infrastructure investment requirements to service forecasted growth in the community. The assessment involved a review of previous studies, an analysis of operations and flow data from the water facilities, and an evaluation of the capacity of the system.

The conclusions of the assessment are summarized below.

— The Onaping-Levack Water System has sufficient water supply and storage capacity to service existing and future populations up to Ultimate Buildout.

— The model revealed that flows meet current fire flow standards in most locations in the Onaping-Levack Water System. Some dead-end watermains deliver less than the current standard fire flows.

— Water pressures in the system recorded per the modeling exercise were within an acceptable range, with the exception of the pressure in the watermains nearby to and feeding the Craig Mine Standpipe which were noted to be upwards of 100 psi. Additional data regarding the mine's water takings would be required to confirm the reason for the high pressure in this watermain.

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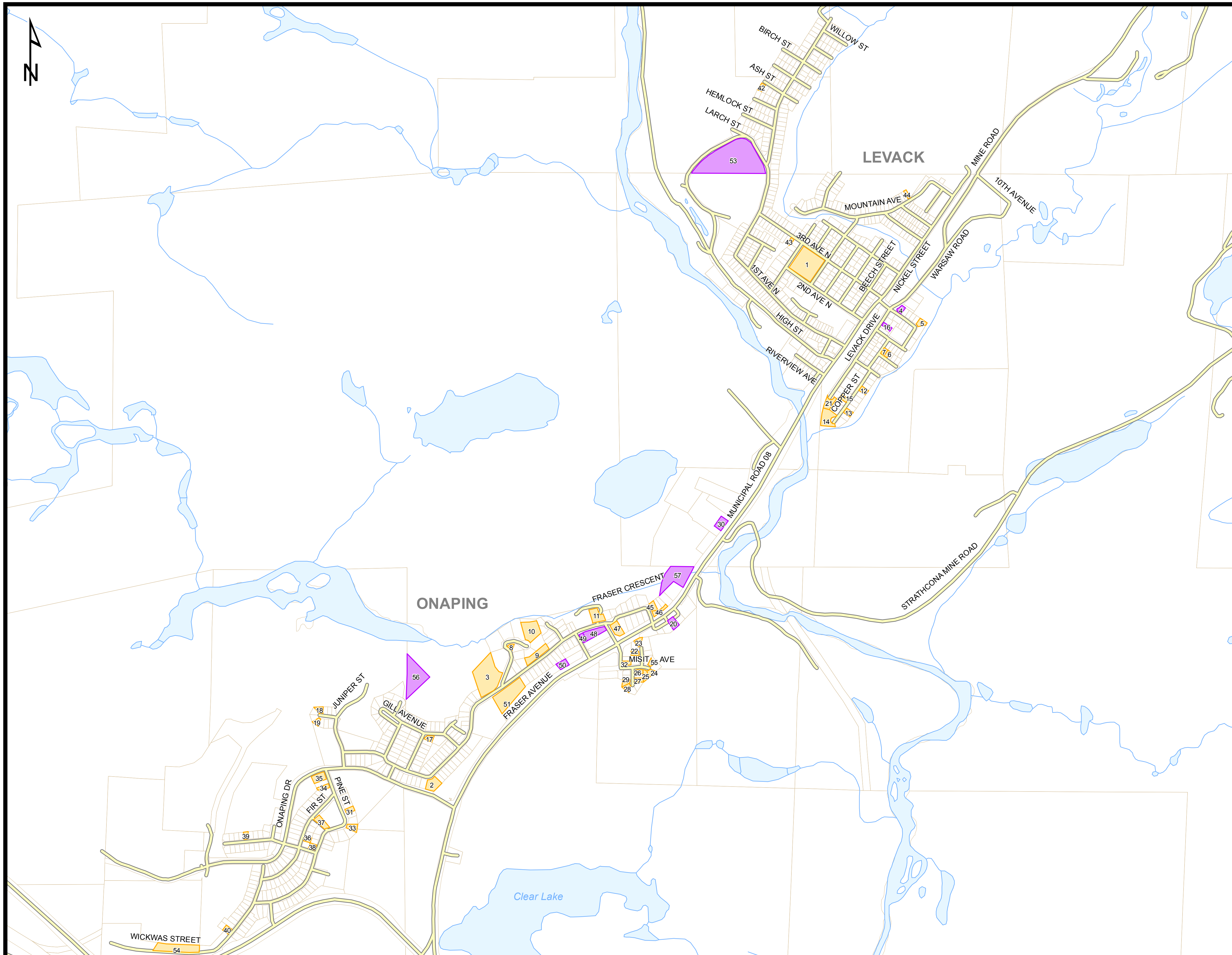
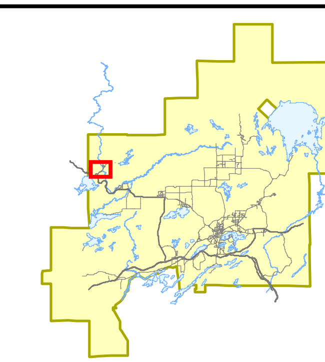
APPENDIX

A

RESIDENTIAL AND ICI DEVELOPMENT AREAS



Legend
- Road
- Watercourse
- Waterbody
- Parcel
- Vacant ICI
- Vacant Residential



Greater Grand Sudbury | WSP

Project No. 121-23026-00

Sudbury Water and Wastewater Master Plan

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Map 1

Levack
2011 - Ultimate Buildout Residential and ICI Development Areas

APPENDIX

B WATER DEMAND AND CAPACITY ASSESSMENTS

DATA ANALYSIS

	2009	2010	2011	2012	2013	Summary	Design Criterion	Comments
Average Day Flow (m^3/d)	1,287	1,521	2,010	1,687	2,033	1,708	1,708	From Water Historical Production data. The daily production values for Wells 3, 4 and 5 were added together to determine the total daily production.
Max Day Flow (m^3/d)	2,501	2,459	2,906	3,511	2,886	2,853	2,853	The maximum value of the sum of water production for Wells 3, 4 and 5 was used.
Max Day Factor	1.94	1.62	1.45	2.08	1.42	1.70	1.70	MOE Guidelines recommend a value of 2.25 for populations between 2,001 and 3,000. The maximum value over the past five years was 2.08. This value was adopted.
Peak Hour (L/s)	Not Available		57	64	61			Peak values were available only for 2011-2013.
Peak Hour (m^3/d)			4,952	5,515	5,308	5,259		
Peak Hour Factor			2.46	3.27	2.61	2.78	3.27	MOE Guidelines recommend a value of 3.38 for populations between 2,001 and 3,000. The maximum value over the past five years was 3.27. This value was adopted.
Population (Existing Areas)	2,112	2,112	2,112	2,112	2,112		2,112	
Ratio of Residential to Total Water Billed	0.43	0.43	0.43	0.43	0.43	0.431		Estimated amount of water consumption related to ICI based on metering data and obtained ratio of residential to total consumption.
Residential Flow (m^3/d)	555	656	866	727	876	736		Calculated based on ratio of residential consumption to total consumption.
Ratio of ICI to Total Water Billed	0.57	0.57	0.57	0.57	0.57	0.569		
	732	865	1143	960	1157	971		
Per Capita Residential Demand (m^3/cap/day)	0.263	0.310	0.410	0.344	0.415	0.348	0.350	Took average over 2009 to 2013 period. CGS Engineering Design Manual does not include a value for average per capita water consumption. However, it includes a value of 410 L/cap/day for per capita wastewater generation.
Average Institutional Flow Unit Rate (m^3/ha/d)							28.0	*MOE Guidelines recommend a value of 28 m3/ha/d.*
Average Commercial Flow Unit Rate (m^3/ha/d)							28.0	*MOE Guidelines recommend a value of 28 m3/ha/d.*
Average Industrial Flow Unit Rate (m^3/ha/d)							35.0	*MOE Guidelines recommend a value of 35 m3/ha/d for light industry and 55 m3/ha/d for heavy industry.*

	2016	2021	2026	2031	2036	2041	Ultimate	Comments
Population (Existing Areas)	2,112	2,112	2,112	2,112	2,112	2,112	2,112	From data provided by Hemson grouped by water system.
Population Growth	11	23	34	42	47	47	365	
Total Population	2,123	2,135	2,146	2,154	2,159	2,159	2,477	From data provided by Hemson grouped by water system.
Residential Growth Area (ha)	1.25	0.05	0.07	0.08	4.64	0.00	6.76	From City's GIS database.
Residential Growth Area (ha) - Cumulative	1.25	1.30	1.37	1.45	6.09	6.09	12.85	
Institutional Growth Area (ha)	0.00	0.00	0.00	0.00	2.78	0.00	0.00	
Institutional Growth Area (ha) - Cumulative	0.00	0.00	0.00	0.00	2.78	2.78	2.78	From City's GIS database.
Commercial Growth Area (ha)	0.00	0.00	0.71	0.00	0.00	0.00	0.00	
Commercial Growth Area (ha) - Cumulative	0.00	0.00	0.71	0.71	0.71	0.71	0.71	From City's GIS database.
Industrial Growth Area (ha)	0.00	0.00	0.00	0.00	1.86	0.00	0.00	
Industrial Growth Area (ha) - Cumulative	0.00	0.00	0.00	0.00	1.86	1.86	1.86	From City's GIS database.
ICI (ha) - Cumulative	0.00	0.00	0.71	0.71	5.35	5.35	5.35	Sum of Institutional, Commercial and Industrial areas
Total Growth Area (ha) - Cumulative	1.25	1.30	2.08	2.16	11.44	11.44	18.20	

	2016	2021	2026	2031	2036	2041	Ultimate
Average Residential and ICI Flows (m^3/d) - Existing	1,708	1,708	1,708	1,708	1,708	1,708	1,708
Average Residential Flows (m^3/d) - Growth	4	8	12	15	16	16	128
Average Residential Flows (m^3/d) - Total	1,712	1,716	1,719	1,722	1,724	1,724	1,835
Average Institutional Flow (m^3/d)	0	0	0	0	78	78	78
Average Commercial Flow (m^3/d)	0	0	20	20	20	20	20
Average Industrial Flow (m^3/d)	0	0	0	0	65	65	65
Average ICI Flow (m^3/d)	0	0	20	20	163	163	163
Average Day Flow (m^3/d)	1,712	1,716	1,739	1,742	1,887	1,887	1,998
Max Day Flow (m^3/d)	2,910	2,917	2,957	2,962	3,208	3,208	3,397
Peak Hour Flow (m^3/d)	5,596	5,609	5,687	5,696	6,169	6,169	6,533

ALTERNATIVE CALCULATION METHOD

	2009	2010	2011	2012	2013	Design Criterion	Comments
Per Capita Demand (m^3/cap/day)	0.609	0.720	0.952	0.799	0.963	0.808	*If ICI is not considered explicitly and demand is divided by total population.*
Max Day Factor						2.08	*The historical per capita consumption is applied for future development.*
Peak Hour Factor						3.27	

	2016	2021	2026	2031	2036	2041	Ultimate
Average Day Flow (m^3/d)	1,717	1,726	1,735	1,742	1,745	1,745	2,002
Max Day Flow (m^3/d)	3,573	3,593	3,612	3,625	3,633	3,633	4,168
Peak Hour Flow (m^3/d)	5,613	5,644	5,673	5,694	5,707	5,707	6,547

SENSITIVITY ANALYSIS

Analyze sensitivity of forecasted average day flows to unit rate

Average Day Flow (m^3/d)

	Unit Rate (m^3/cap/d)	2016	2021	2026	2031	2036	2041	Ultimate
Using a consolidated per capita flow		1,717	1,726	1,735	1,742	1,745	1,745	2,002
Using estimated average	0.350	1,712	1,716	1,739	1,742	1,887	1,887	1,998
City Standards	0.41	1712	1717	1741	1745	1890	1890	2020

Analyze sensitivity of forecasted flows to max day peaking factor

Max Day Flow (m^3/d)

	Max Day Peaking Factor	2016	2021	2026	2031	2036	2041	Ultimate
2009-2013 average of peaking factors	1.70	2,912	2,919	2,959	2,964	3,210	3,210	3,399
2012 peaking factor (maximum historical)	2.08	2,910	2,917	2,957	2,962	3,208	3,208	3,397
MOE Guidelines	2.25	3851	3860	3914	3920	4245	4245	4496

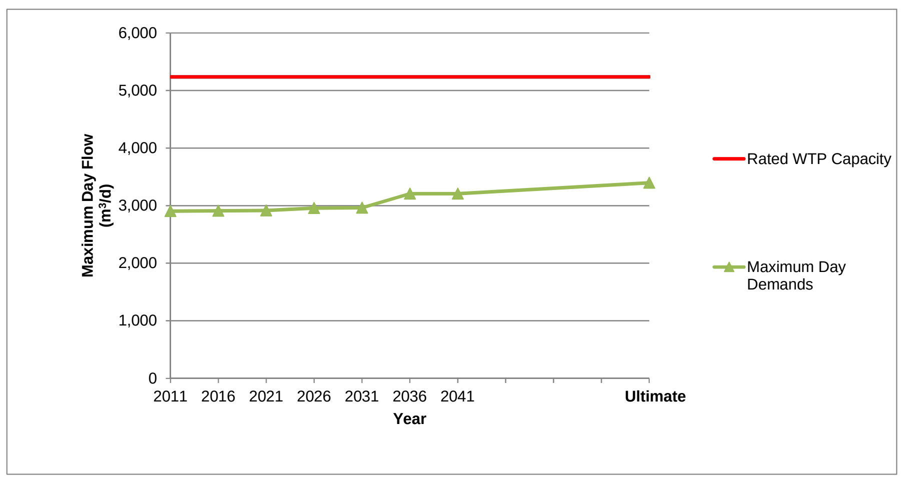
Analyze sensitivity of forecasted flows to peak hour factor

Peak Hour (m^3/d)

	Peak Hour Peaking Factor	2016	2021	2026	2031	2036	2041	Ultimate
Using historical peak factor	3.27	5,596	5,609	5,687	5,696	6,169	6,169	6,533
MOE Guidelines	3.38	5785	5799	5879	5889	6377	6377	6753

CAPACITY CHECK

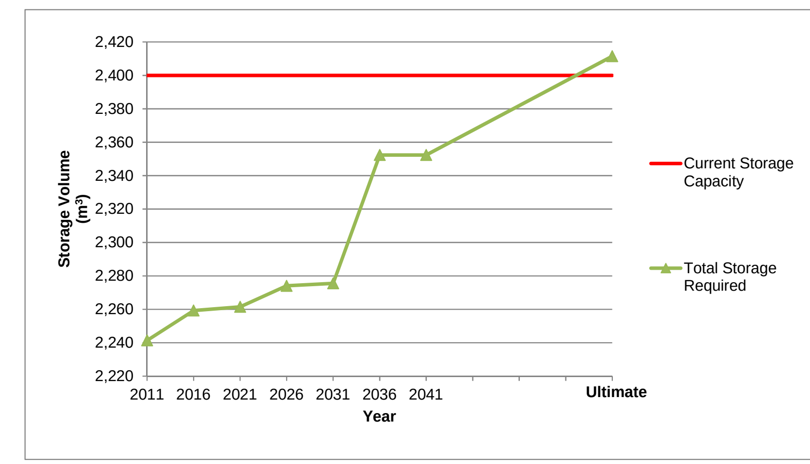
	2011	2016	2021	2026	2031	2036	2041	Ultimate
Rated WTP Capacity	5,237	5,237	5,237	5,237	5,237	5,237	5,237	5,237
Actual WTP Capacity	5,237	5,237	5,237	5,237	5,237	5,237	5,237	5,237
Maximum Day Demands	2,906	2,910	2,917	2,957	2,962	3,208	3,208	3,397
Peak Hour Demands	4,952	5,596	5,609	5,687	5,696	6,169	6,169	6,533



STORAGE REQUIREMENTS

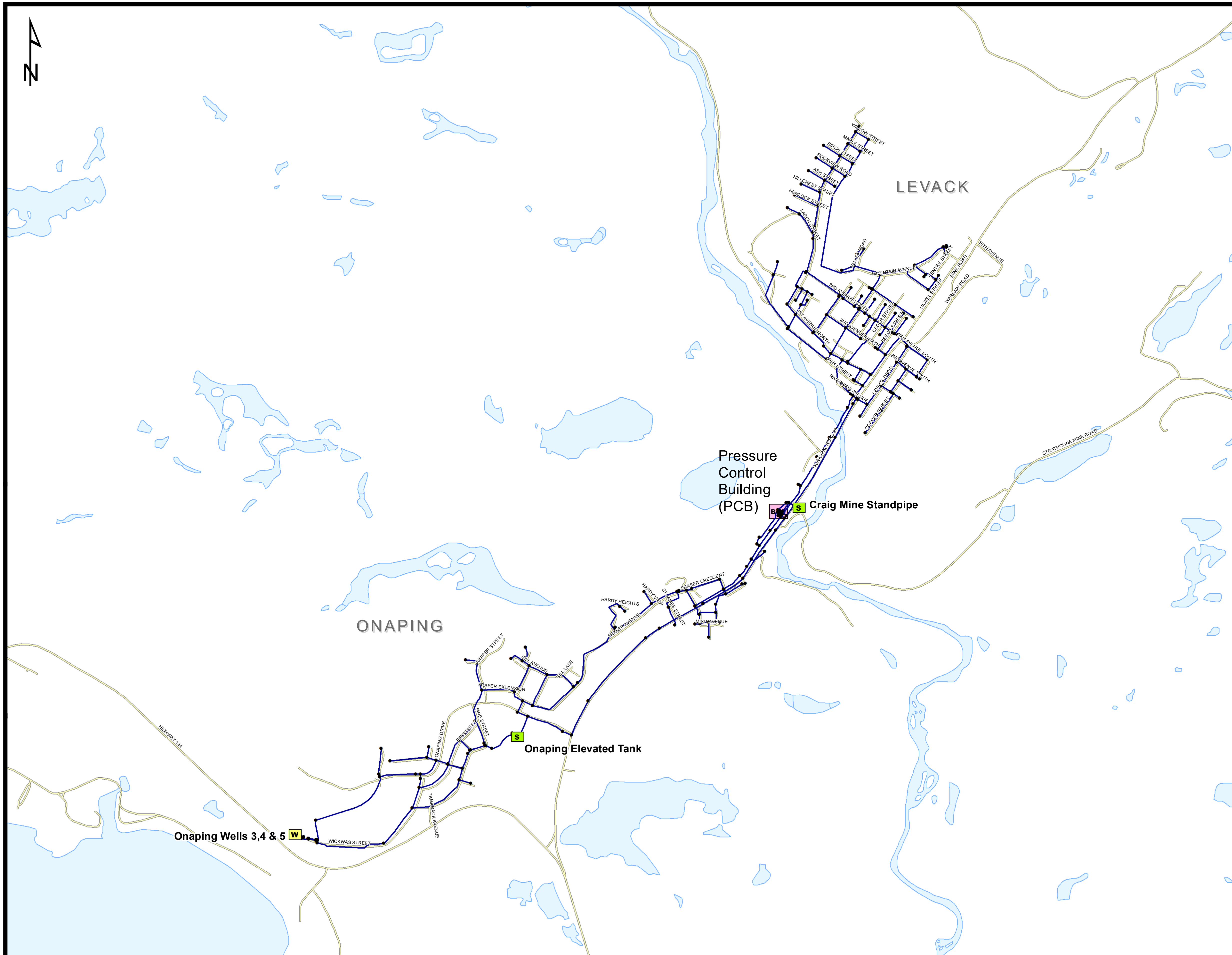
Storage Available		
Elevated Tank (m^3)	2,400	From DWWP 016-202 (September 13, 2011)
Total Storage (m^3)	2,400	
Maximum Fire flow requirements (L/s)	150	
Fire Duration (hrs)	2	
Minimum Fire Flow Requirement for Residential Areas (L/s)	75	From CGS Engineering Design Manual
Fire Duration (hrs)	1.75	From Fire Underwriters Survey Requirements corresponding to 75 L/s

	Max Day Demand (m^3/d)	Required Fire Flow (m^3/d)	Max Day + Fire (m^3/d)	Peak Hour (m^3/d)	A - Fire Storage (m^3)	B - Equalization Storage (m^3)	C - Emergency Storage (m^3)	A + B + C = Storage Required (m^3)	Storage Available (m^3)	Deficit (m^3)
2011	2,853	12,960	15,813	5,259	1,080	713	448	2,241	2,400	0
2016	2,910	12,960	15,870	5,596	1,080	727	452	2,259	2,400	0
2021	2,917	12,960	15,877	5,609	1,080	729	452	2,261	2,400	0
2026	2,957	12,960	15,917	5,687	1,080	739	455	2,274	2,400	0
2031	2,962	12,960	15,922	5,696	1,080	740	455	2,276	2,400	0
2036	3,208	12,960	16,168	6,169	1,080	802	470	2,352	2,400	0
2041	3,208	12,960	16,168	6,169	1,080	802	470	2,352	2,400	0
Ultimate	3,397	12,960	16,357	6,533	1,080	849	482	2,411	2,400	11



APPENDIX

C WATER MODEL RESULTS



Legend

- Well
- Storage Tank
- Booster Pumping Station (BPS)
- Water Treatment Plant (WTP)
- Pipe
- Road
- Waterbody

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Sudbury Water and Wastewater Master Plan

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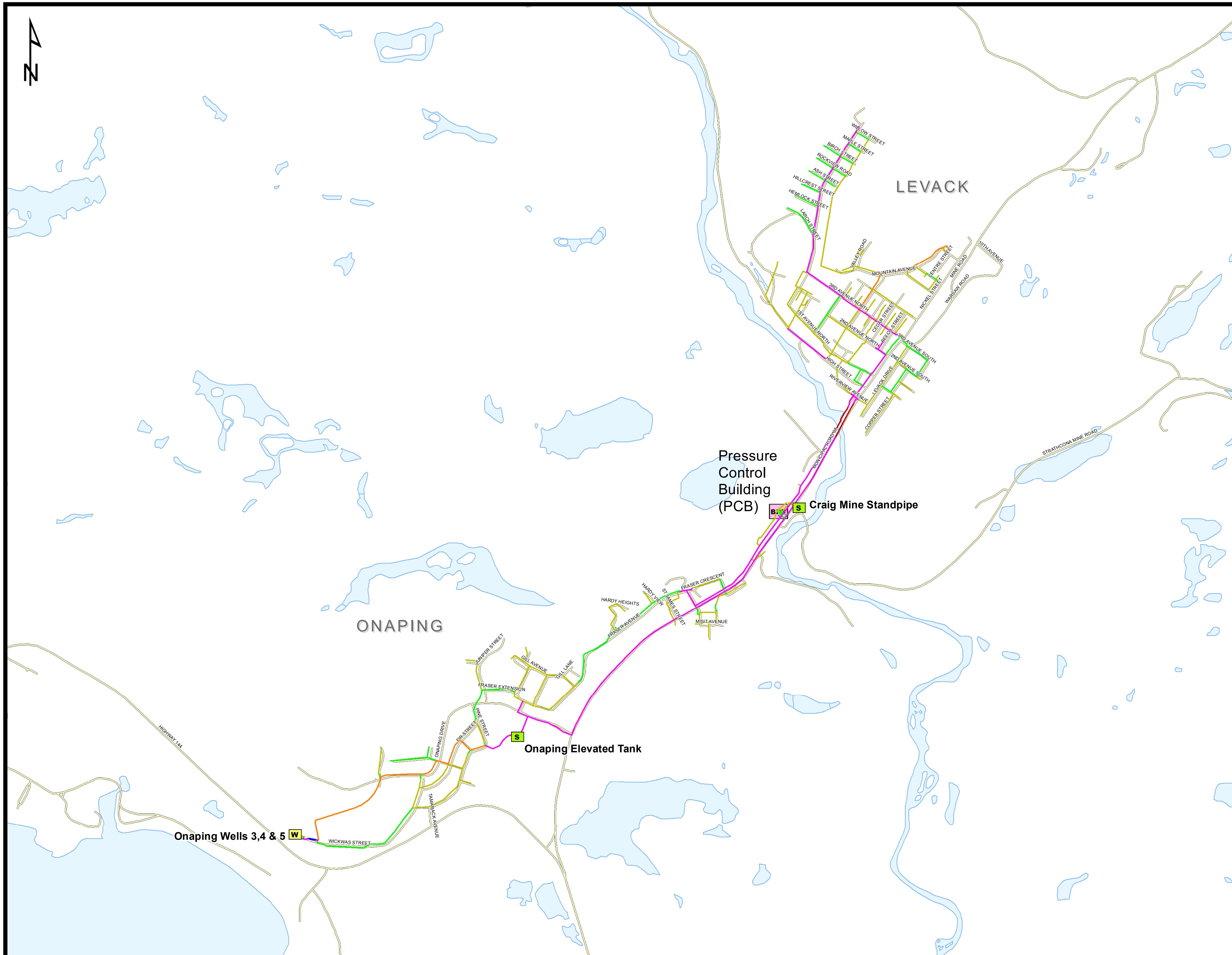
Map 1

Onaping & Levack Water System - Key Plan



Legend

- Well
- Storage Tank
- Booster Pumping Station (BPS)
- Water Treatment Plant (WTP)
- Road
- Waterbody

Pipe: Diamater (mm)

- <= 150
- <= 200
- <= 250
- <= 300
- <= 400
- <= 600
- > 600

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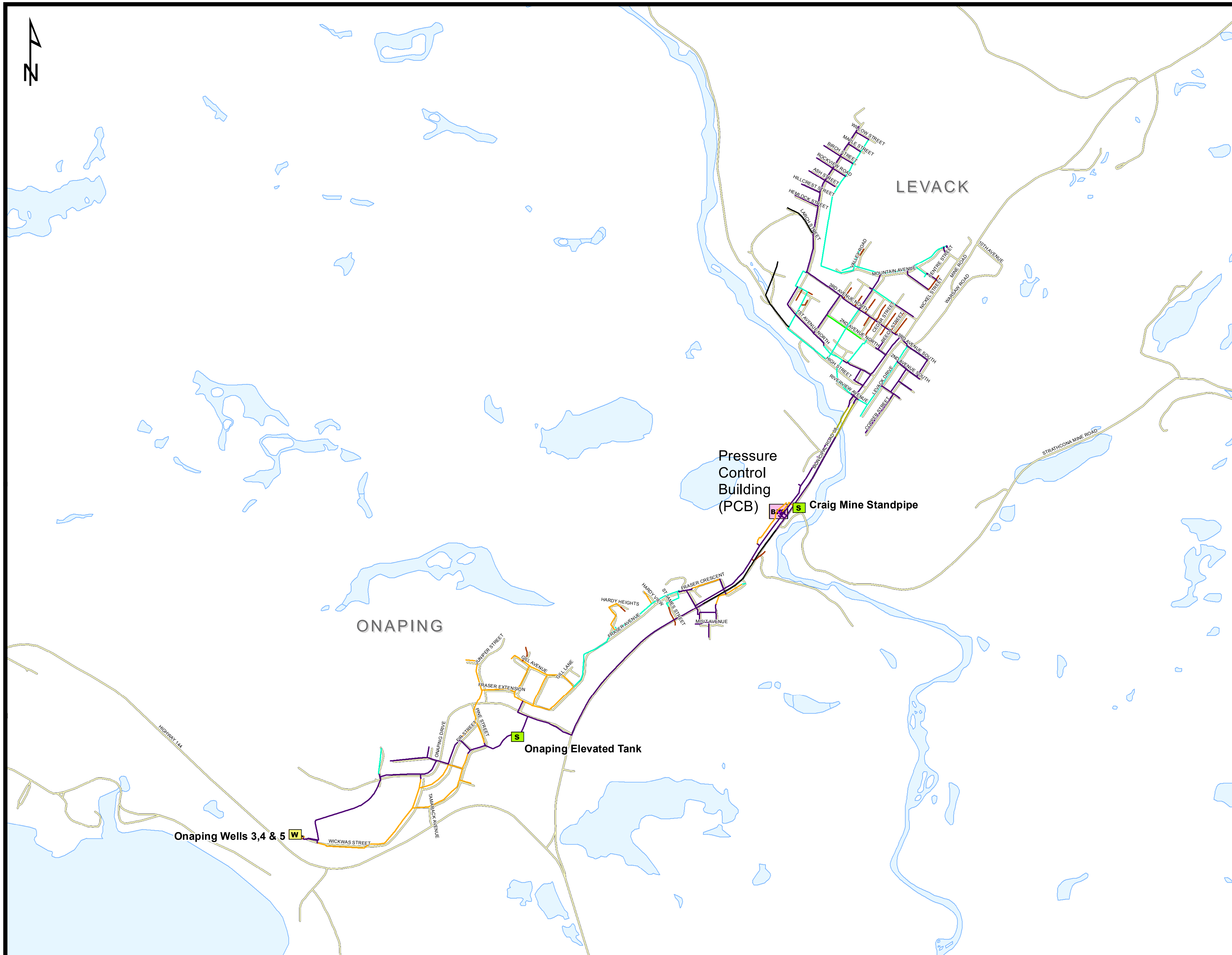
Map 2

Onaping & Levack Water System - Pipe Diameter



Legend

- W Well
- S Storage Tank
- BPS Booster Pumping Station (BPS)
- WTP Water Treatment Plant (WTP)
- Road
- Waterbody

Material

- Asbestos Cement
- Cast iron
- Concrete
- Copper
- Ductile Iron
- Galvanized iron
- PVC
- Polyethylene
- Steel

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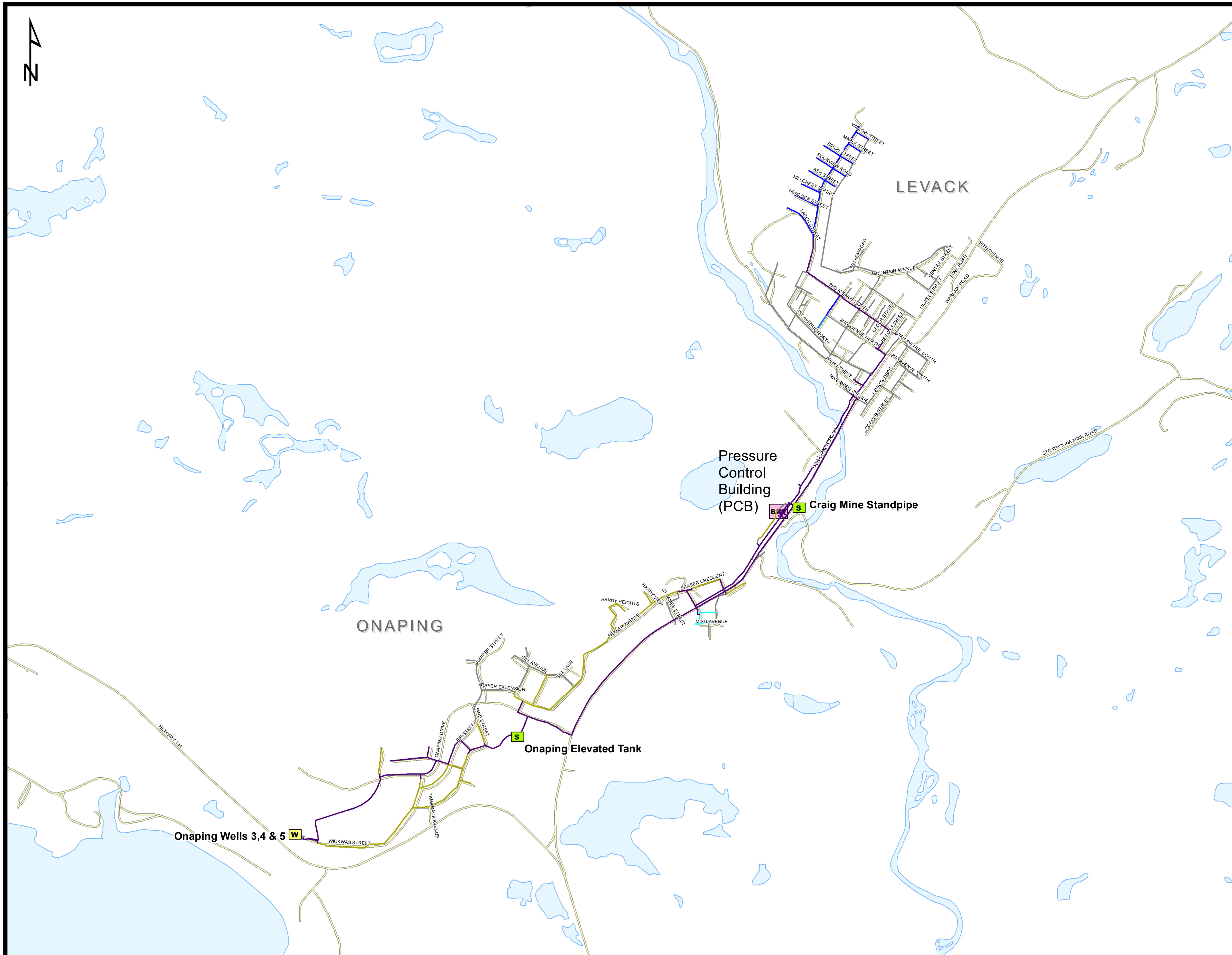
Map 3

Onaping & Levack Water System - Pipe Material



Legend

- W — Well
- S — Storage Tank
- BPS — Booster Pumping Station (BPS)
- WTP — Water Treatment Plant (WTP)
- Road
- Waterbody

Installed Year

- N/A
- <= 1930
- <= 1940
- <= 1950
- <= 1960
- <= 1970
- <= 1980
- <= 1990
- <= 2000
- <= 2010
- > 2011

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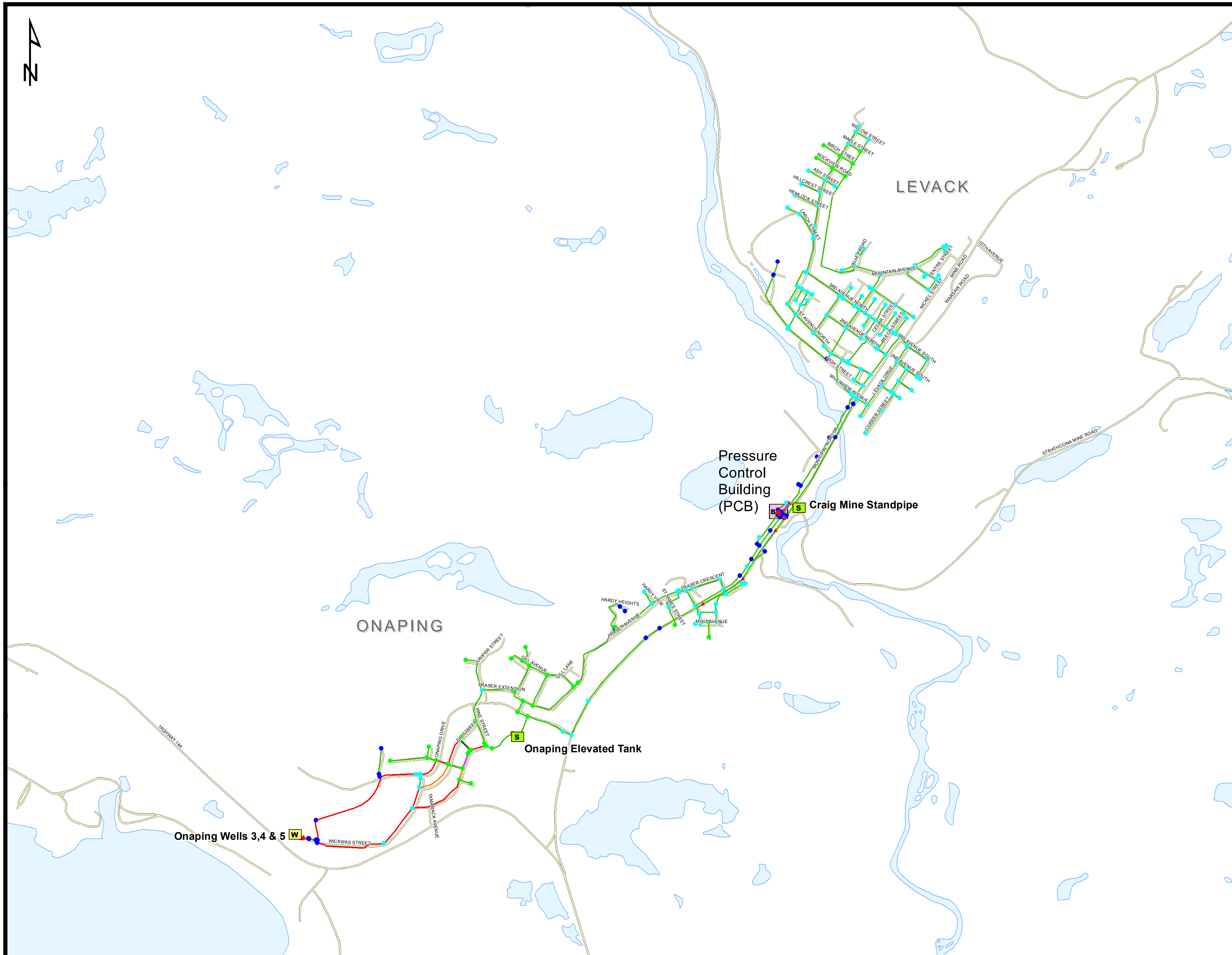
Map 4

Onaping & Levack Water System - Pipe Age



Legend

- Well
- Storage Tank
- Booster Pumping Station (BPS)
- Water Treatment Plant (WTP)
- Road
- Waterbody

Junction: Pressure (psi)

- <= 20
- <= 40
- <= 60
- <= 80
- <= 100
- > 100

Pipe: Headloss Gradient (m/km)

- <= 1.5
- <= 2.0
- <= 3.0
- <= 5.0
- > 5.0

MDD/ADD = 1.70

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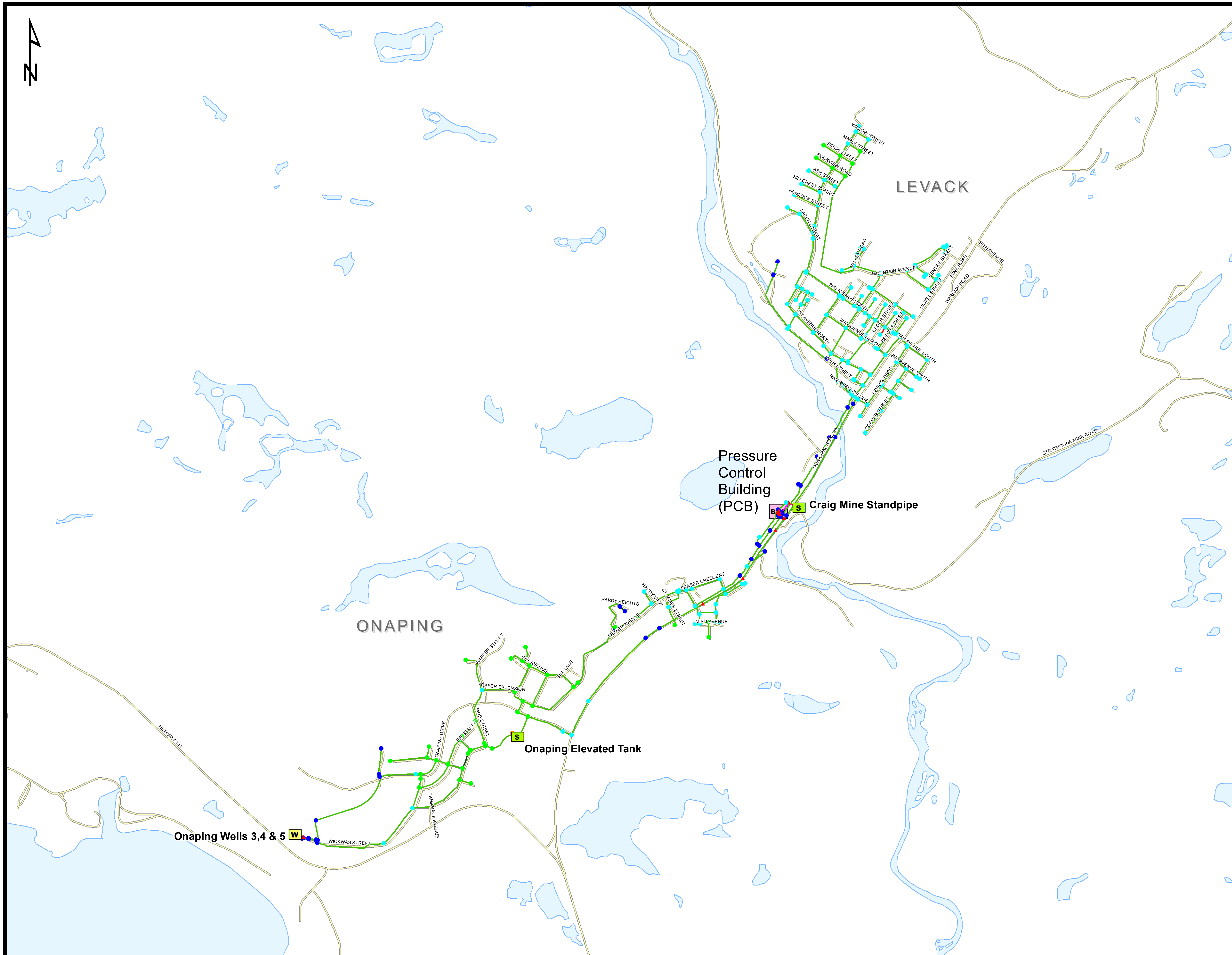
Map 5

Onaping & Levack Water System - Pipe Headloss and Junction Pressure

Scenario: 2011ADD



Legend

W Well

S Storage Tank

BPS Booster Pumping Station (BPS)

WTP Water Treatment Plant (WTP)

Road

Waterbody

Junction: Pressure (psi)

- <= 20
- <= 40
- <= 60
- <= 80
- <= 100
- > 100

Pipe: Headloss Gradient (m/km)

- <= 1.5
- <= 2.0
- <= 3.0
- <= 5.0
- > 5.0

MDD/ADD = 1.70

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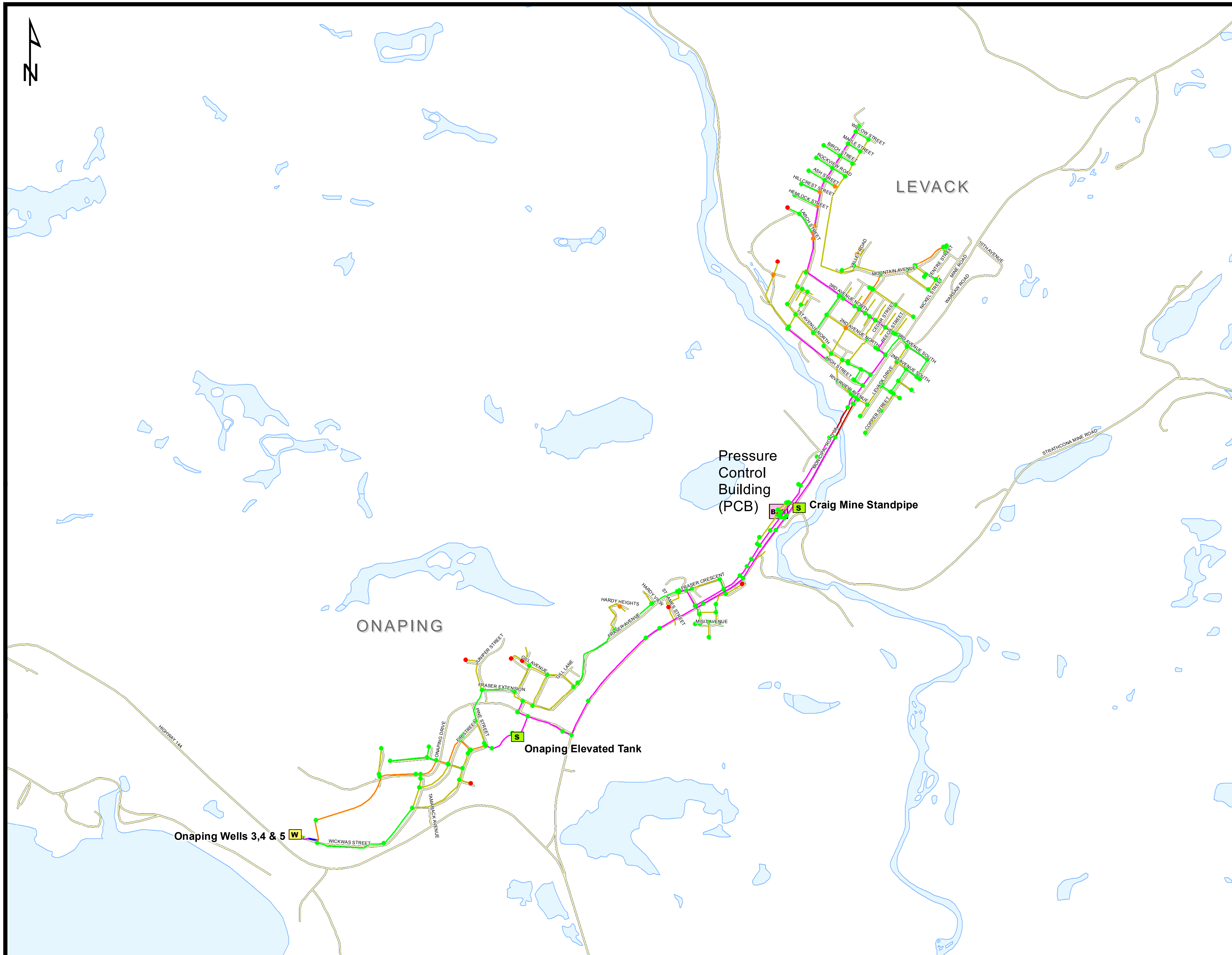
Map 6

Onaping & Levack Water System - Pipe Headloss and Junction Pressure

Scenario: 2011MDD



Legend

- Well
- Storage Tank
- Booster Pumping Station (BPS)
- Water Treatment Plant (WTP)
- Road
- Waterbody

Junction: Fire Flow (Available) (L/s)

- <= 65 (Res) or 130 (ICI)
- < 75 (Res) or 150 (ICI)
- >= 75 (Res) or 150 (ICI)

Pipe: Diamater (mm)

- <= 150
- <= 200
- <= 250
- <= 300
- <= 400
- <= 600
- > 600

MDD/ADD = 1.70

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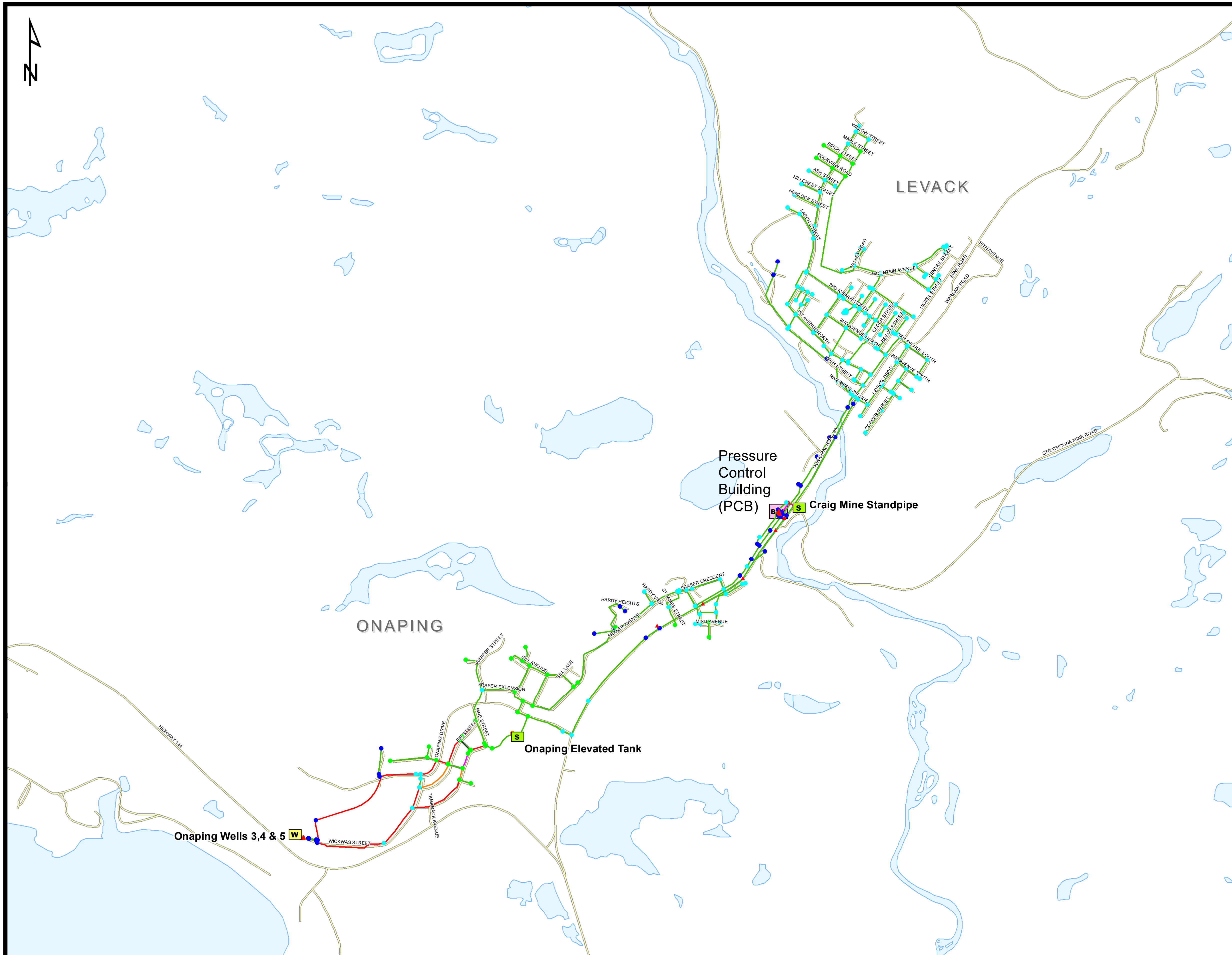
Map 7

Onaping & Levack Water System - Fire Flow Availability

Scenario: 2011MDD+FF



Legend

- Well
- Storage Tank
- Booster Pumping Station (BPS)
- Water Treatment Plant (WTP)
- Road
- Waterbody

Junction: Pressure (psi)

- <= 20
- <= 40
- <= 60
- <= 80
- <= 100
- > 100

Pipe: Headloss Gradient (m/km)

- <= 1.5
- <= 2.0
- <= 3.0
- <= 5.0
- > 5.0

MDD/ADD = 1.70

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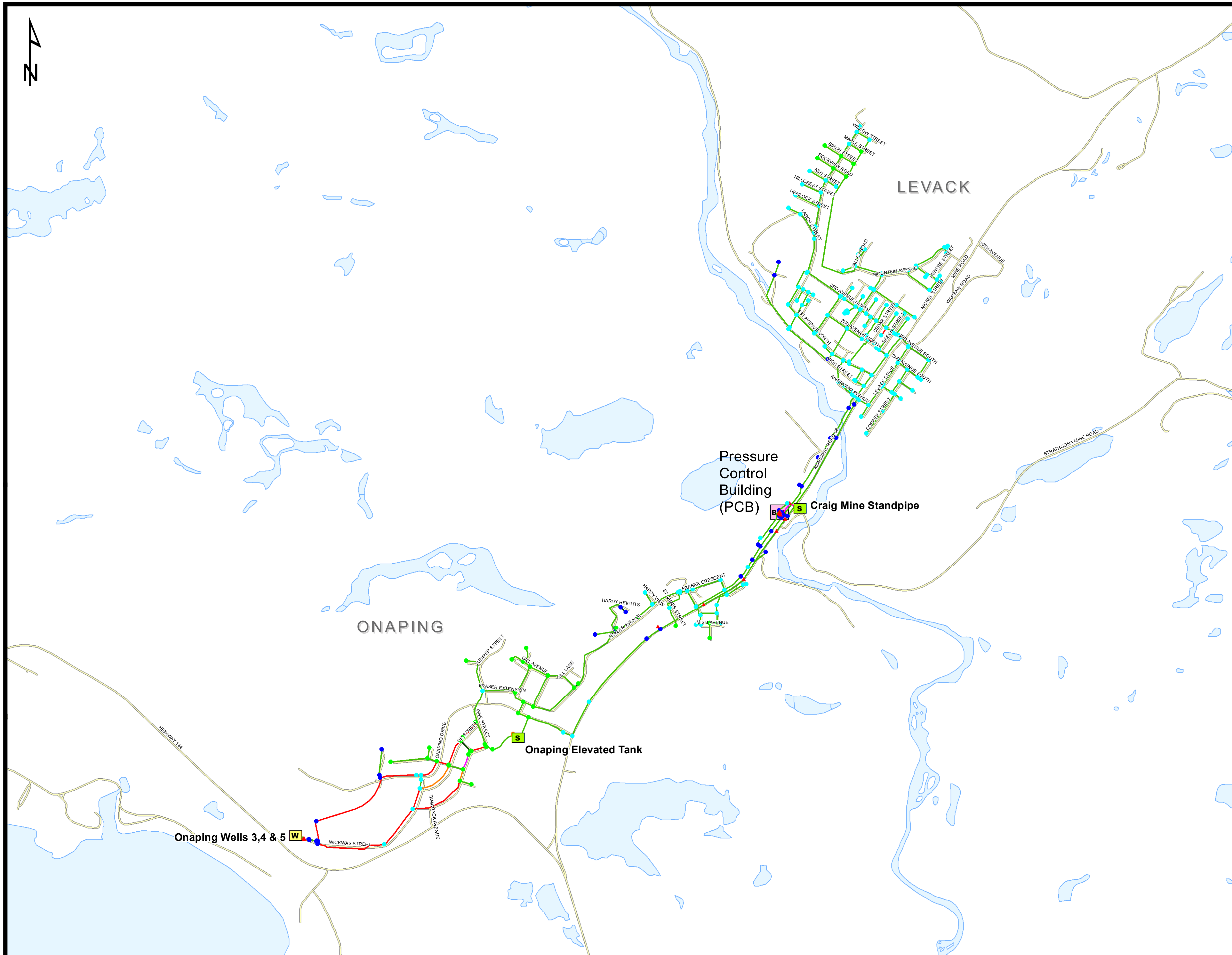
Map 8

Onaping & Levack Water System - Pipe Headloss and Junction Pressure

Scenario: 2041ADD



Legend

- W Well
- S Storage Tank
- BPS Booster Pumping Station (BPS)
- WTP Water Treatment Plant (WTP)
- Road
- Waterbody

Junction: Pressure (psi)

- <= 20
- <= 40
- <= 60
- <= 80
- <= 100
- > 100

Pipe: Headloss Gradient (m/km)

- <= 1.5
- <= 2.0
- <= 3.0
- <= 5.0
- > 5.0

MDD/ADD = 1.70

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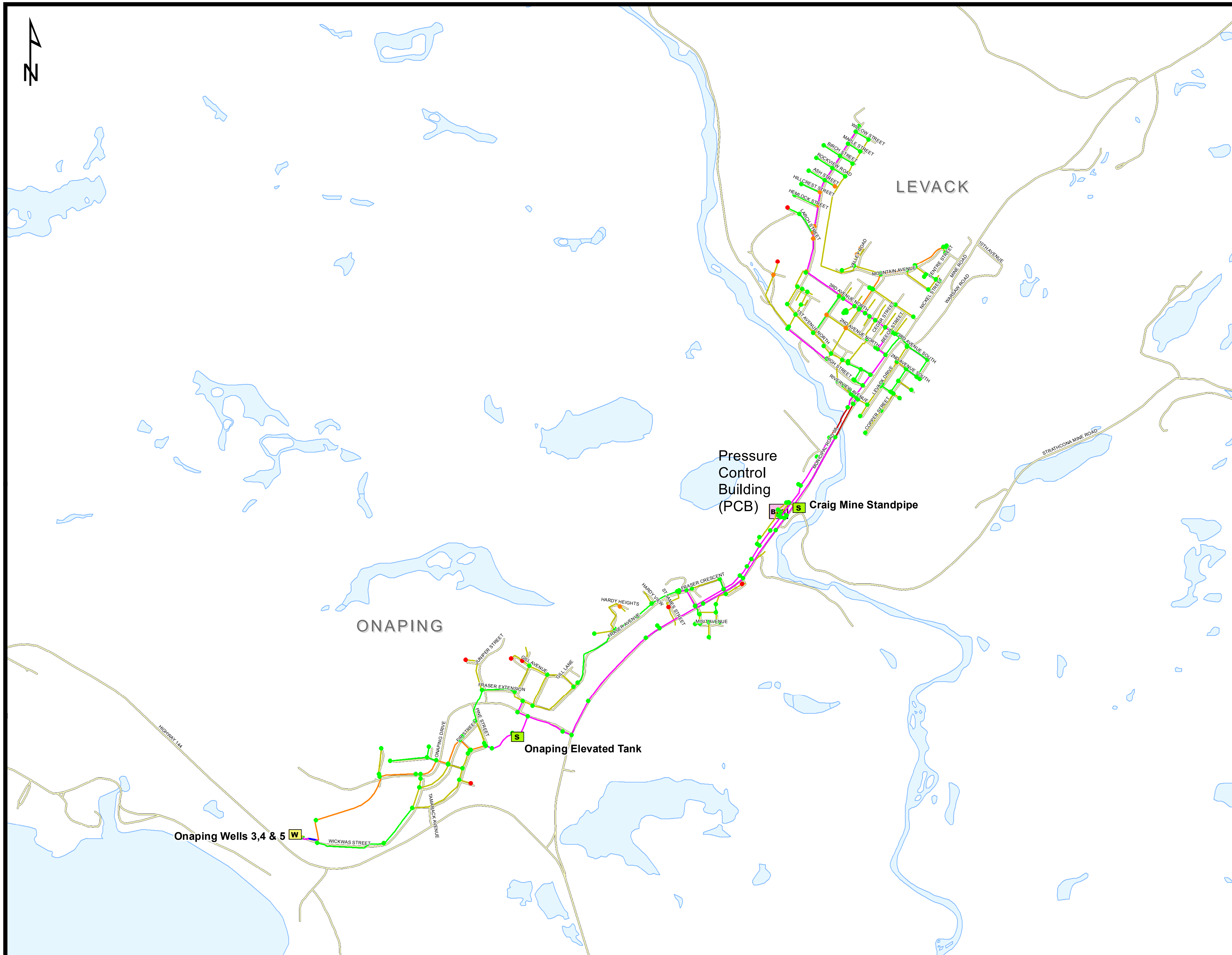
Map 9

Onaping & Levack Water System - Pipe Headloss and Junction Pressure

Scenario: 2041MDD



Legend

- W Well
- S Storage Tank
- BPS Booster Pumping Station (BPS)
- WTP Water Treatment Plant (WTP)
- Road
- Waterbody

Junction: Fire Flow (Available) (L/s)

- <= 65 (Res) or 130 (ICI)
- < 75 (Res) or 150 (ICI)
- >= 75 (Res) or 150 (ICI)

Pipe: Diamater (mm)

- <= 150
- <= 200
- <= 250
- <= 300
- <= 400
- <= 600
- > 600

MDD/ADD = 1.70

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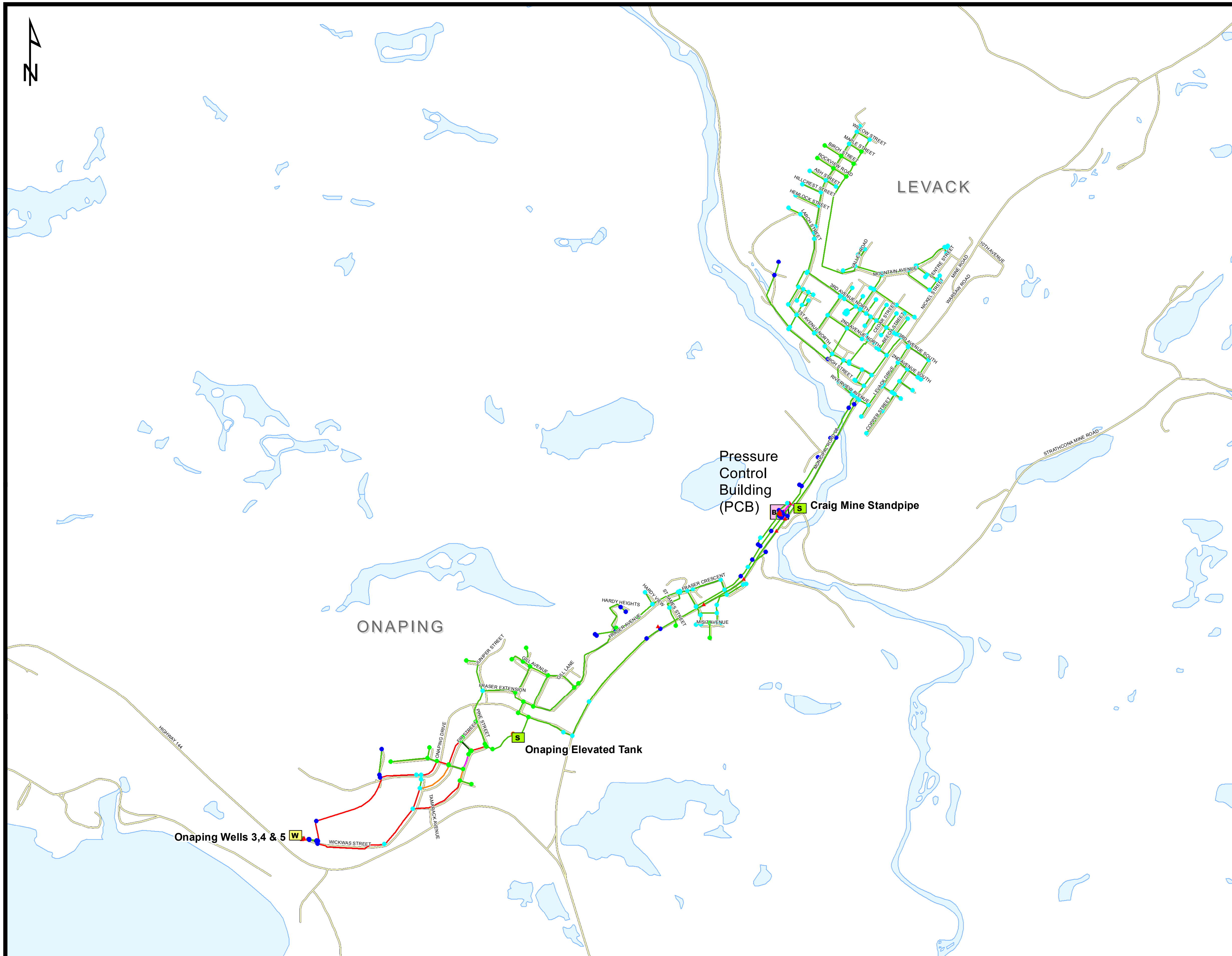
Map 10

Onaping & Levack Water System - Fire Flow Availability

Scenario: 2041MDD+FF



Legend

- Well
- Storage Tank
- Booster Pumping Station (BPS)
- Water Treatment Plant (WTP)
- Road
- Waterbody

Junction: Pressure (psi)

- <= 20
- <= 40
- <= 60
- <= 80
- <= 100
- > 100

Pipe: Headloss Gradient (m/km)

- <= 1.5
- <= 2.0
- <= 3.0
- <= 5.0
- > 5.0

MDD/ADD = 1.70

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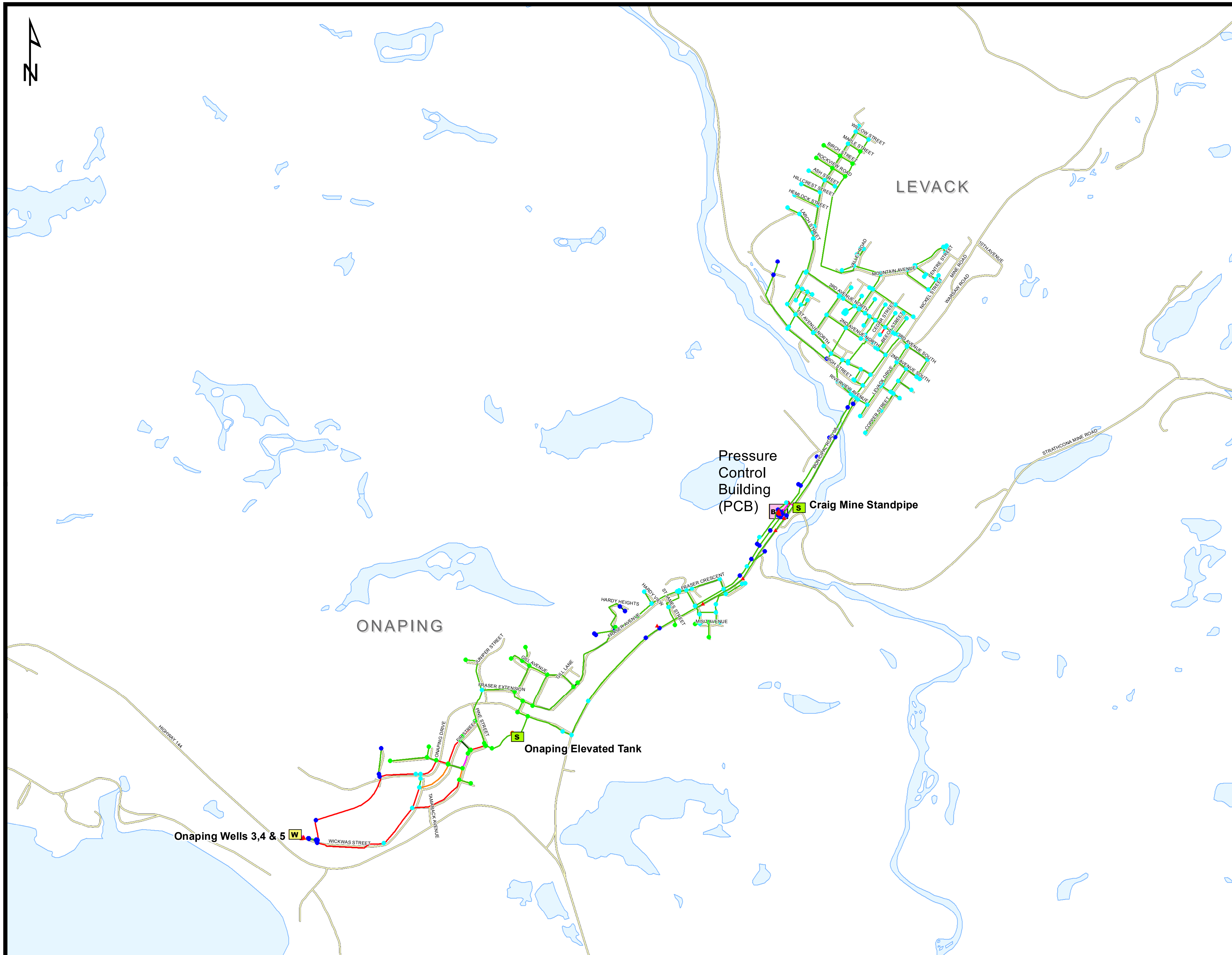
Map 11

Onaping & Levack Water System - Pipe Headloss and Junction Pressure

Scenario: ULTADD



Legend

- W Well
- S Storage Tank
- BPS Booster Pumping Station (BPS)
- WTP Water Treatment Plant (WTP)
- Road
- Waterbody

Junction: Pressure (psi)

- <= 20
- <= 40
- <= 60
- <= 80
- <= 100
- > 100

Pipe: Headloss Gradient (m/km)

- <= 1.5
- <= 2.0
- <= 3.0
- <= 5.0
- > 5.0

MDD/ADD = 1.70

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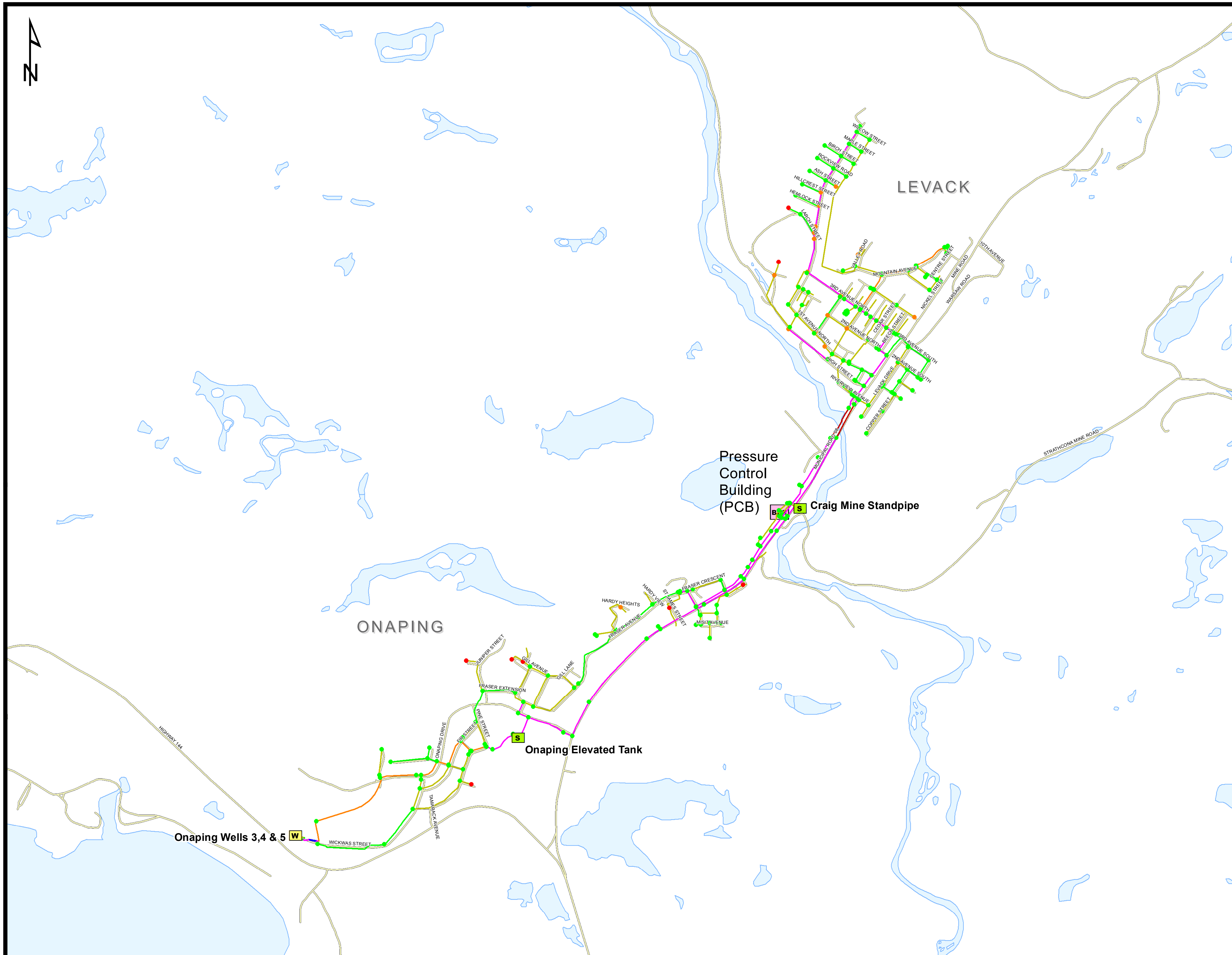
Map 12

Onaping & Levack Water System - Pipe Headloss and Junction Pressure

Scenario: ULTMDD



Legend

- Well
- Storage Tank
- Booster Pumping Station (BPS)
- Water Treatment Plant (WTP)
- Road
- Waterbody

Junction: Fire Flow (Available) (L/s)

- <= 65 (Res) or 130 (ICI)
- < 75 (Res) or 150 (ICI)
- >= 75 (Res) or 150 (ICI)

Pipe: Diamater (mm)

- <= 150
- <= 200
- <= 250
- <= 300
- <= 400
- <= 600
- > 600

MDD/ADD = 1.70

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Sudbury Water and Wastewater Master Plan

0 200 400 600 800 1,000 m

1:20,000

Map 13

Onaping & Levack Water System - Fire Flow Availability

Scenario: ULTMDD+FF

CITY OF GREATER SUDBURY WATER AND WASTEWATER MASTER PLAN

SUDBURY WATER SYSTEM GAP ANALYSIS AND STATUS QUO REPORT

CITY OF GREATER SUDBURY

DRAFT

PROJECT NO.: 121-23026-00
DATE: NOVEMBER 2016

WSP
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THORNHILL, ON, CANADA L3T 0A1

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APPENDICES

1 INTRODUCTION

The City of Greater Sudbury (CGS) retained WSP to undertake a Water and Wastewater Master Plan. The purpose of the Master Plan project is to establish servicing strategies for water and wastewater infrastructure for the core urban areas and surrounding communities in the City for the next 20 years, as part of the five-year review of the City's Official Plan. The Master Plan will identify potential projects to address the servicing needs for planned growth within the City. It is being conducted in accordance with the requirements set out in the Municipal Class Environmental Assessment (Class EA) document (June 2000 as amended in 2007 and in 2011).

This report includes a capacity review of Sudbury's existing water system. Based on population growth projections and design criteria discussed in the *Population and Unit Rates Technical Memorandum* (WSP, 2014), water demand projections were developed and used to determine future infrastructure needs to the 2041 and Ultimate Buildout planning horizons.

This report assumes that the Sudbury Water System would continue to be a stand-alone system. Any potential interconnections between Sudbury and other systems are not considered as part of this report. Potential interconnections with other communities will be reviewed under separate cover, as part of the Master Plan.

The conclusions provided in this report will be the basis for the problem definition and evaluation of alternatives conducted as part of the Master Plan.

Additional information on the existing water system is provided in the Baseline Review Report for Water Systems (WSP, 2014).

2 STUDY AREA

The Sudbury Water System is located centrally in the City of Greater Sudbury and is the City's most populated area. It services the communities of Sudbury (including New Sudbury and Downtown), Coniston, Wanapitei, and Garson. The neighbouring community of Markstay in the Municipality of Markstay-Warren is also supplied by the Sudbury Water System. The system is supplied by two surface water treatment plants (WTPs) as well as three wells.

Maps 1 and 2 in Appendix A shows the Sudbury study area and identifies future land use and development areas, including vacant residential and industrial, commercial, and institutional (ICI) areas.

Additional information on population growth and development phasing is provided in the Unit Rates and Population Projections Technical Memorandum (WSP, 2014).

Existing development in the study area is mixed, and includes residential as well as ICI land uses.

Based on the City's planning data, the Sudbury area population with municipal water servicing is expected to increase from 94,868 in 2011 to 99,450 by 2041 and 126,663 by Ultimate Buildout.

ICI growth is expected to be primarily industrial with some commercial and a small amount of institutional. Growth is discussed further in Section 6.1.

3 OVERVIEW OF EXISTING SYSTEM

The Sudbury Water System services the communities of Coniston, Garson, Sudbury, and Wahnapitae. The system is supplied by two surface water treatment plants and three wells. The David Street WTP is supplied by Ramsey Lake and the Wanapitei WTP is supplied by the Wanapitei River. Three wells are located in Garson and primarily supply the east end of the community of Garson, although the Garson and Sudbury communities are interconnected.

The total rated capacity for the system (David Street WTP, Wanapitei WTP, and Garson Wells) is 101,827 m^3/d, as described in the *Baseline Review Report – Water* (WSP, 2014). However, it is not possible to operate the system at its rated capacity due to constraints at the David Street and Wanapitei WTPs and the distribution system piping. Therefore, the estimated production capacity for the system is 81,813 m^3/d, as detailed in Section 3.4.

All of the facilities are owned and operated by the City of Greater Sudbury.

Additional information on the existing systems is provided in the Baseline Review Report for Water Systems (WSP, 2014).

3.1 DAVID STREET WATER TREATMENT PLANT

The David St. WTP is a surface water plant drawing water from Ramsey Lake. According to the plant PTTW, the maximum permitted water taking is 40,000 m^3/d, on any given day; however, the monthly average rate may not exceed 27,760 m^3/d. This corresponds to a maximum day production capacity of approximately 37,260 m^3/d when the maximum day design peaking factor of 1.39 is applied to the monthly average.

The plant was constructed in the early 1900s and has undergone numerous upgrades. In 2004, major upgrades were made to install a membrane ultrafiltration system and a UV disinfection system. The existing process flow diagram is presented in Figure 3-1.

The David Street WTP has operational and maintenance challenges. The plant has problems with moisture and corrosion and has consistent issues with valves and analyzers.

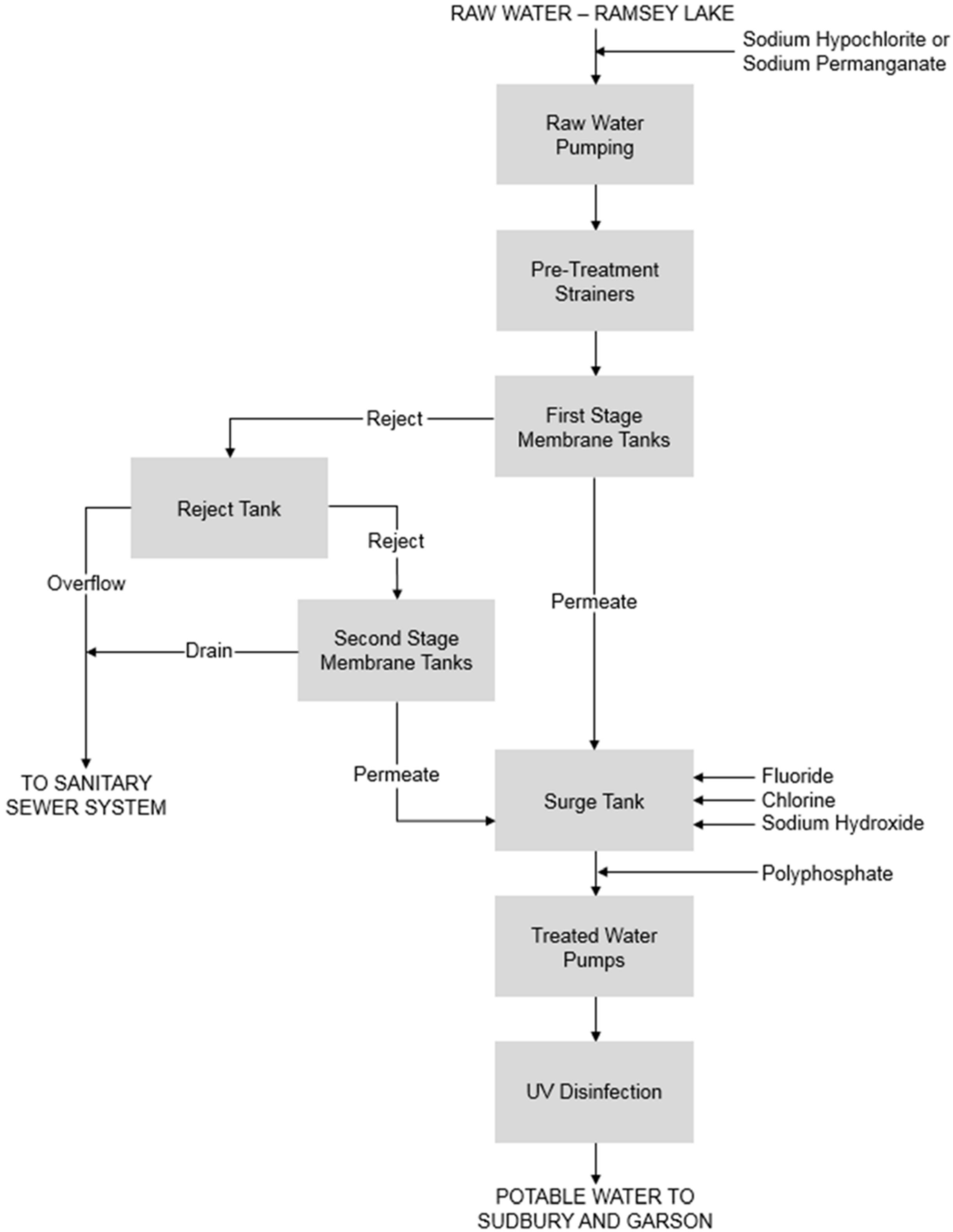

Figure 3-1 David Street WTP Process Flow Diagram

3.2 WANAPITEI WATER TREATMENT PLANT

The Wanapitei WTP services the City of Sudbury, the communities of Wahnapitae and Coniston, and the Municipality of Markstay-Warren. The plant is located at 49 Hwy 17 East in Coniston. It is a conventional surface water treatment plant, which draws water from the Wanapitei River. The plant was constructed in the 1970s and has since undergone several upgrades to enhance treatment efficiency, increase production, and to reduce energy costs. According to the *Wanapitei WTP Hydraulic Capacity Report* (AECOM, 2009), the plant is limited to a maximum flow of 44,000 m^3/d due to insufficient high lift pumping capacity and hydraulic pressure limitations of the existing transmission main between Coniston and Sudbury. However, City operations staff has indicated that, in practice, the plant operates between 40,000 to 42,000 m^3/d. A process flow block diagram is shown in Figure 3-2.

Raw water is drawn by five raw water pumps. It is then pretreated with chlorine gas or chlorine dioxide for taste and odor control. When high levels of organics are present in the raw water, chlorine dioxide is dosed to reduce the formation of trihalomethanes (THMs) and other disinfection by products (DBPs). The raw water is mixed with alum in the flash mixing chamber. After sedimentation, the water flows through four dual media (silica sand/anthracite coal) gravity filters. The filtered water is then treated with hydrated lime (for pH /alkalinity adjustment), fluoride, chlorine, and polyphosphate to reduce corrosion in the distribution system. The treated water is then disinfected using an ultraviolet (UV) system.

The Wanapitei WTP includes five high lift pumps that discharge treated water to a single 750 mm diameter watermain to the Sudbury Distribution System and a 250 mm diameter watermain to the communities of Wahnapitae and Markstay-Warren.

The plant is equipped with a hydropneumatic tank fed off the 750 mm discharge to protect the Sudbury Distribution System from hydraulic transients.

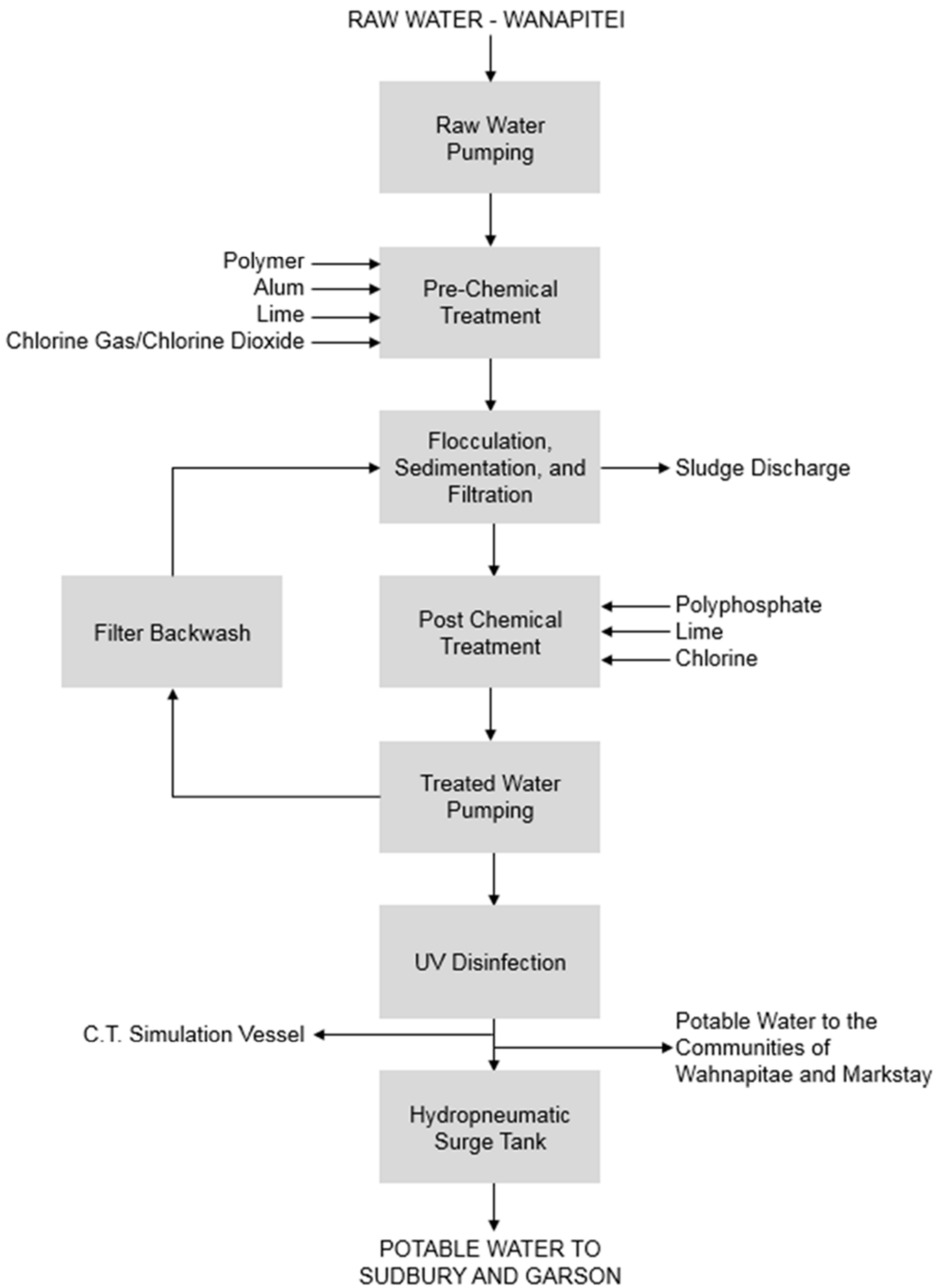

Figure 3-2 Wanapitei WTP Process Flow Diagram

3.3 GARSON WELLS

The Garson groundwater system consists of three wells, Garson Wells No. 1, 2 and 3, normally servicing the eastern area of Garson. The wells also service the west side of town if the pressure in the west drops below the pressure in the east side through the O'Neil Pressure Sustaining Valve (PSV).

Garson Well 2 is located on the east side of Falconbridge Highway at Spruce Street. This well house is not equipped with standby power supply. A vertical turbine well pump equipped with a variable frequency drive (VFD) draws water which is then chlorinated and fluoridated. The raw water contains iron.

Garson Wells 1 and 3 are located on the south side of Falconbridge Road at Orell Street. The property has two well houses, one chemical building, and one buried chlorine contact tank. Well Houses 1 and 2 contain the vertical turbine well pumps, pumping to a common 200 mm header to the chemical building. The raw water is then treated with sodium hypochlorite and fluoride prior to entering the contact tank. The buried process piping allows for isolation of the contact tank. An exterior standby generator with a nominal rating of 125 kW, automatic transfer switch, and 100 L capacity double-walled fuel tank can be used for Wells 1 and 3.

Wells 1 and 3 have elevated levels of tetrachloroethylene, but the levels do not exceed the regulated Maximum Acceptable Concentration (MAC).

When the duty well switches over from Well 2 to Well 1 or 3 (or the reverse), flow in part of the distribution system reverses. The flow reversal results in movement of water that was previously stagnant. Therefore, during a switch, chlorine residuals tend to decrease, but continue to meet regulatory requirements.

Table 3-1 summarizes additional process information for the Garson Wells, and a process flow chart is shown in Figure 3-3.

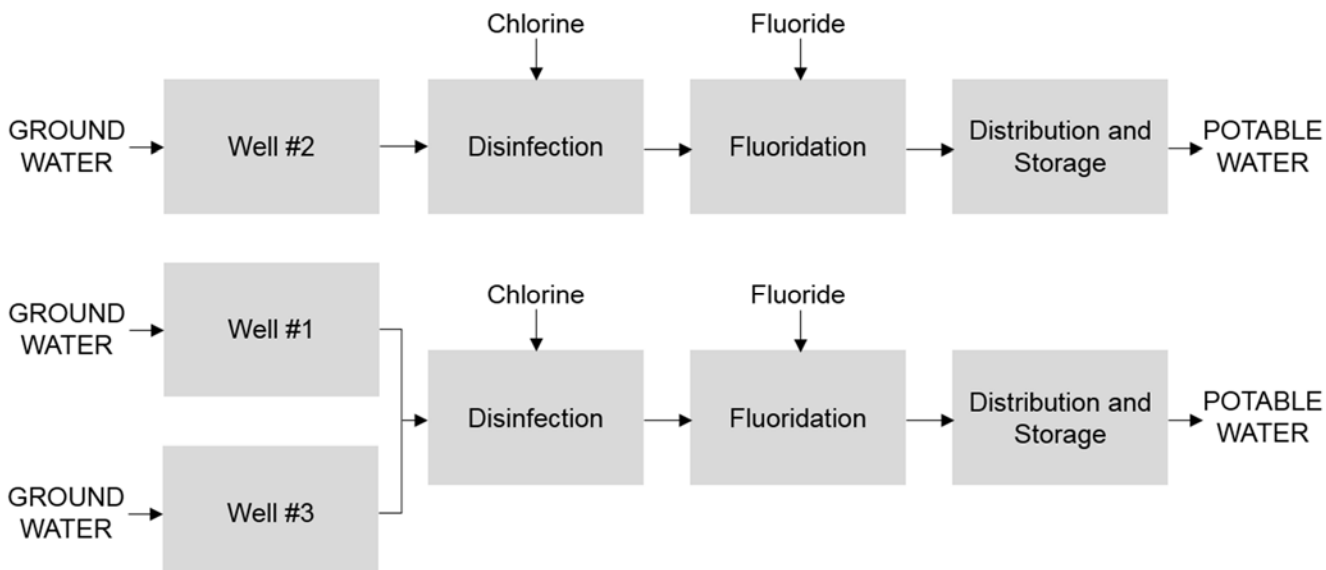

Figure 3-3 Garson Wells' Process Flow Diagram

Table 3-1 Garson Wells' Process Information[1]

SOURCE	PUMP TYPE	OPERATING POINT	STANDBY POWER
Well 2	Vertical turbine pump equipped with variable frequency drive (VFD)	34.5 L/s at 93.8 m TDH	None

SOURCE	PUMP TYPE	OPERATING POINT	STANDBY POWER
Well 1	Vertical turbine pump	22.7 L/s at 63.7 m TDH	125 kW diesel generator with automatic transfer switch (ATS)
Well 3	Vertical turbine pump	34 L/s at 64.0 m TDH	

[1] Data obtained from the Sudbury Drinking Water Works Permit, Number 016-206 Issue 2.

3.4 WATER SUPPLY CAPACITY AND LIMITATIONS

As noted in the previous sections, the Wanapitei and David Street WTPs do not operate at their maximum rated capacities.

The David Street WTP is rated for a maximum day production capacity of 40,000 m^3/d. However, the Permit to Take Water (PTTW) for this facility limits the monthly average production capacity to 27,760 m^3/d. This corresponds to a maximum day production capacity of about 37,260 m^3, calculated by applying the maximum day design peaking factor of 1.38 (discussed in Section 5) to the monthly average amount. Historically (2008-2013), this plant has operated at 34,367 m^3/d once and only rarely operated between 25,000 and 28,000 m^3/d. The David Street WTP is typically operated below 25,000 m^3/d. The historical data is illustrated in the figure below.

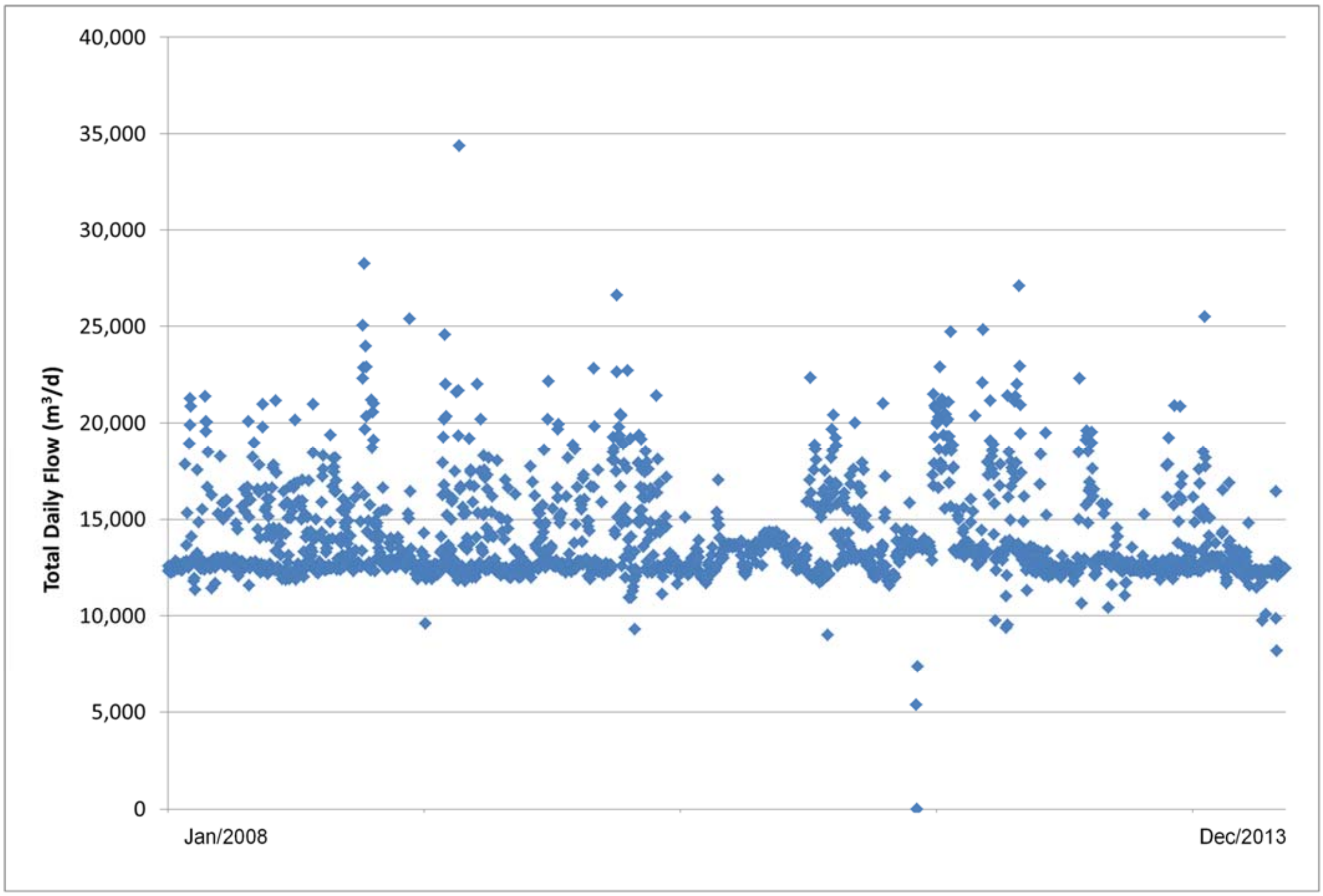

Figure 3-4 David Street WTP Historical Daily Production

The Wanapitei WTP is rated to produce 54,000 m^3/d. However, there are hydraulic restrictions in the distribution system near the plant, limiting output. As a result, the plant is normally operated at no more than 40,000 m^3/d, as illustrated in the figure below. However, for master planning purposes, it has been assumed that the output can be increased to capacity (54,000 m^3/d) by removing the hydraulic restrictions as an outcome of the master plan.

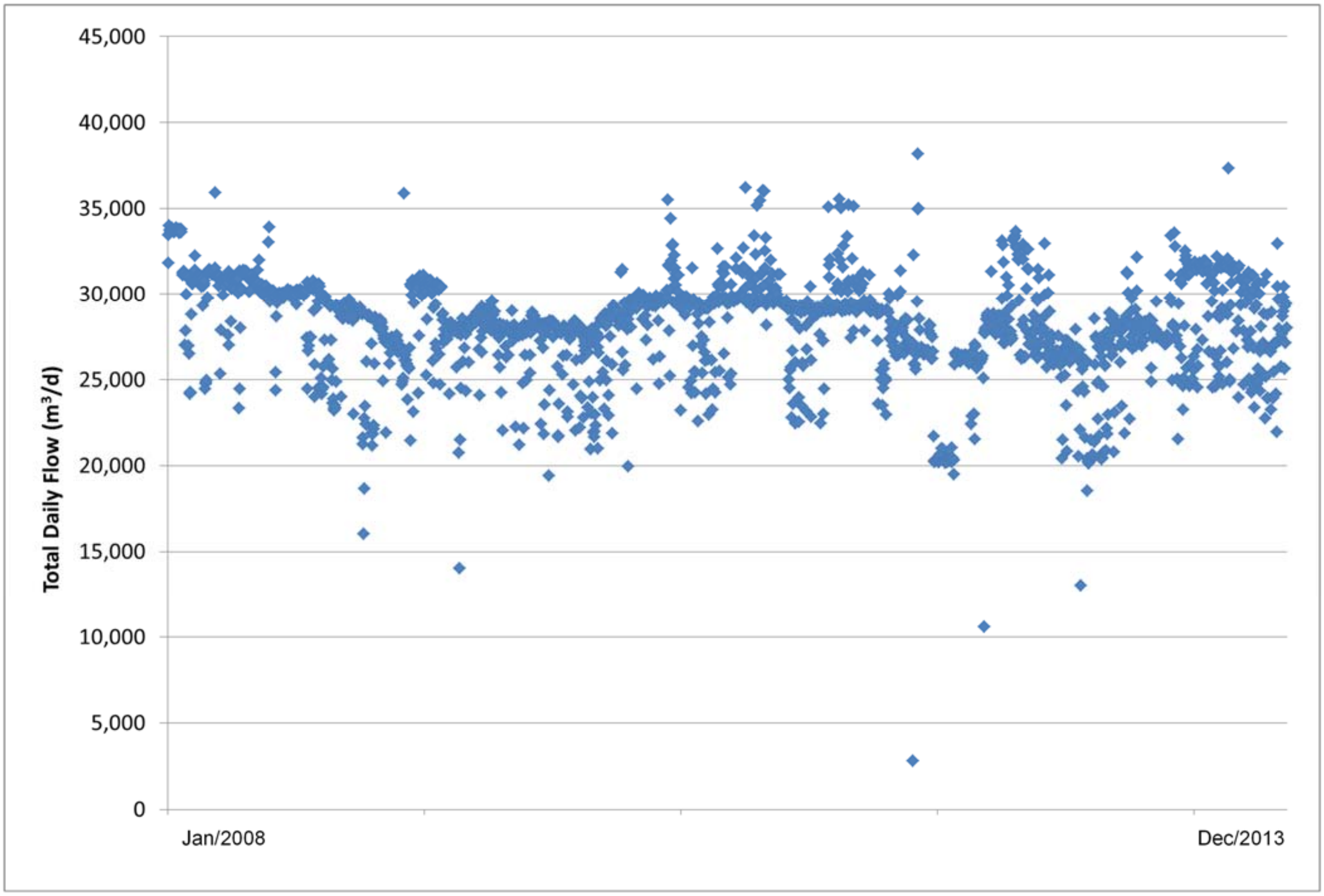

Figure 3-5 Wanapitei WTP Historical Daily Production

The Garson Wells do not have any reported capacity constraints; however, elevated levels of tetrachloroethylene (PCE) have been reported at these wells. Although this does not directly impact the production capacity, future reliability of these wells may be limited if PCE levels rise and if the water is not treated for PCE removal.

Taking the above limitations into consideration, the estimated production capacity of the Sudbury water facilities is 81,813 m^3/d. The rated and estimated production capacities for each plant and well are listed in Table 3-2.

Table 3-2 Sudbury System Rated and Estimated Actual Capacity

WATER SUPPLY	RATED CAPACITY (M3/D)	ESTIMATED ACTUAL CAPACITY (M3/D)
Wanapitei WTP	54,000[1]	54,000[1]
David Street WTP	40,000[2]	37,260[2]
Garson Orell Well No. 1	1,572[3]	1,572[3]
Garson Well No. 2	2,981[4]	2,981[4]
Garson Orell Well No. 3	3,274[3]	0[5]
Sudbury System	**101,827**	**81,813**

[1] *The rated capacity for the Wanapitei WTP is 54,000 m^3/d. It has been assumed that, as an outcome of a master plan project, the hydraulic limitations can be fixed, allowing the plant to deliver its rate capacity.*

[2] Although the rated plant capacity is 40,000 m^3/d, the PTTW for this facility limits the monthly average day production to 27,760 m^3/d, corresponding to a maximum day amount of 37,260 m^3/d.
[3] Rated capacity obtained from Garson Orell Wells PTTW #5376-84BMP7.
[4] Rated capacity obtained from Garson Well 2 PTTW #5307-8YHNAM.
[5] Best practices assume largest well out of service to determine firm capacity.

3.5 DISTRIBUTION SYSTEM

The Sudbury distribution system consists of the following infrastructure:

— One Storage Tank
— Eight Booster Pumping Stations
— A number of watermains

The Ellis Reservoir, constructed in 1997, is an in-ground dual cell reservoir and rechlorination facility that receives water directly from the Wanapitei and David Street WTPs. The reservoir is in good condition based on observations from City staff. According to the Drinking Water Works Permit, the reservoir has a capacity of 36,400 m^3. Its top water level is 324.6 m and its low water level is 317.1 m; however, City staff has observed that when the reservoir is filled to its top water level, the frequency of watermain breaks in the surrounding area increases. As a result, the Ellis Reservoir is not filled to capacity, thereby reducing its useful volume. The reservoir is typically filled to a water level of 321.1 m to 322.6 m, for a maximum useful volume of approximately 26,700 m^3.

There is no additional storage available at the David Street or Wanapitei WTPs. All storage at these plants is fully utilized for the required chlorine contact time.

Table 3-3 Sudbury Water System Booster Pumping Stations

FACILITY	PUMP INFORMATION	TOTAL CAPACITY (L/S)	FIRM CAPACITY[2] (L/S)
Algonquin	Two centrifugal pumps with variable speed drives, each pump rated at 17.7 L/s at 16 m TDH	35.4	17.7
Copper Park[1]	Three centrifugal pumps with variable speed drives; two pumps rated at 10 L/s at 32 m TDH each and one pump rated at 80 L/s at 38.5 m TDH	100	20.0
Jogues	Two centrifugal pumps with variable speed drives, each pump rated at 11.4 L/s at 19.5 m TDH	22.8	11.4
Maley[1]	Two vertical turbine pumps with variable speed drives, each pump rated at 45 L/s at 49 m TDH and one centrifugal pump rated at 120 L/s at 56 m TDH.	210	90.0

FACILITY	PUMP INFORMATION	TOTAL CAPACITY (L/S)	FIRM CAPACITY[2] (L/S)
Montrose	Two centrifugal pumps, one rated at 18.9 L/s at 22.9 m TDH and one rated at 63.1 L/s at 22.9 m TDH	82.0	18.9
Moss	One pump rated at 3.8 L/s	3.8	0
Snowdon	Two centrifugal pumps, one rated at 19.7 L/s at 29 m TDH and one rated at 70 L/s (TDH not known)	89.7	19.7
Sunrise Ridge[1]	Three centrifugal pumps with variable speed drives; two pumps rated at 9.8 L/s at 164.9 m TDH each and one pump rated at 81.5 L/s at 48.1 m TDH.	101	19.6

[1] *Standby power available.*

[2] *Based on the largest pump out of service.*

Only the Copper Park, Maley, and Sunrise Ridge BPS's have standby power.

In addition to the above booster pumping stations, Laurentian University obtains water from the municipal supply and pressurizes the water system on the university campus through the Laurentian BPS. This BPS is owned and operated by Laurentian University and is therefore not included in this study.

There are many 150 mm diameter watermains in the system, particularly in the west end. Such small diameter mains were constructed in accordance with design standards in place at the time of construction and are not meant to deliver fire flows. Accordingly, these mains also do not have hydrants installed on them.

3.6 KNOWN CHALLENGES

In addition to concerns discussed in previous sections, the Sudbury Water System has the following known challenges:

- The watermain along Maley Drive breaks frequently and has been out of service since 2013 to prevent additional breaks. WSP has completed a review of the proposed 600 mm watermain currently being designed by others, for construction in the near future.
- The watermain connecting Marcus Drive and Bancroft Avenue watermains reduces from 750 to 400 mm diameter for valving.
- There are many 35 and 50 mm galvanized or copper watermains in the west end and Downtown Sudbury.
- There is a high frequency of watermain breaks as well as many dead end watermains in the area north east of the Snowdon BPS.
- Watermains on and surrounding Moonlight Avenue are mainly cast iron and have a high breakage frequency.
- Watermains in the Gatchell area have a high breakage frequency due to pipe age.
- Kingsway trunk watermain has high breakage frequency.
- Hydrants are flushed frequently in the west end to maintain chlorine residual.
- There is a single watermain from the Wanapitei WTP into Sudbury. However, there are plans to twin this watermain in the near future.

In addition, the intake for the Wanapitei WTP is located on the Wanapitei River, which is used for hydro power generation. Ontario Power Generation (OPG) controls flow and water levels in the river at the Wanapitei Lake Dam. There are several generating stations located up and downstream of the plant, controlling water levels. Most notably, the nearby Stinson Generating Station and dam is located just upstream of the plant, while the Coniston Generating Station and dam is just downstream. Therefore, the supply is subject to limitations governed by OPG, as indicated in the Tier One Water Budget and Water Quantity Stress Assessment (Golder Associates Ltd., 2008).

The City runs a program instructing about 121 customers (exact number varies annually) in the Sudbury Water System to run a small amount of water through their taps in the winter months to prevent water services from freezing on the municipal side. The specific number of customers included in the program varies annually depending on the expected winter temperatures.

Some service connections in Greater Sudbury freeze due to the shallow depth of bury; older homes were constructed prior to the current standards for depth of bury and are more vulnerable to freezing.

In the year reviewed, four addresses were located in Coniston, 18 in Garson, and 99 in Sudbury.

Customers who are requested to run their water are asked to run a small flow, equivalent to about the thickness of a pencil or approximately 0.06 L/s, between December 1 and April 1. In Sudbury, this results in a total of about 75,000 m^3 per season, or 627 m^3/d. In the winter, this accounts for less than 1% of the estimated production capacity of 81,813 m^3/d.

The Sudbury Water System includes several areas where trunk watermains are not looped. This increases the risk of increased water age, reduced chlorine residual, and lower available flows. It also increases the risk of water supply concerns in case of a break on a major trunk watermain, since there would not be a second feed. Opportunities for looping will be reviewed in the Alternative Solutions report. In addition to potential concerns with looping, all storage and supply (excluding the Garson wells, which typically only service East Garson) in Sudbury is located in the same general corridor, between Ramsey Lake and Kingsway/Highway 17.

4 HISTORICAL FLOWS AND REVIEW OF OPERATIONAL DATA

Water supply data from 2009 to 2013 from the David Street WTP, Wanapitei WTP, and Garson Wells was reviewed and analyzed for this evaluation. Table 4-1 shows a summary of the data received, and indicates how it was used for the analysis.

Table 4-1 Water Supply Data Reviewed

DATA RECEIVED	PARAMETERS INCLUDED	DATA INTERVAL	USE IN ANALYSIS
Treated flow (2011-2013)	Flow in m^3/d	Hourly	To determine peak hourly flow
Annual Reports (2009-2013)	Total average daily flows, maximum daily flows Treated water characteristics	Daily	To determine average day, max day flow To assess performance of existing process and treated water characteristics
Annual Billing Data (2012)	Annual flow per customer in m^3	Annually	To determine the proportion of total water consumption corresponding to residential users

4.1 FLOW DATA

Water supply data from 2009 to 2013 was reviewed to determine historical water demands in the Sudbury Water System. Average day and maximum day demand data for the past five years, and peak hour data for the past three years (2011-2013) is included in Table 4-2. For reference, the estimated production capacity is 81,813 m^3/d, as discussed in Section 3.4.

Table 4-2 Historical Water Supply Data

YEAR	AVERAGE DAY DEMAND (M3/D)[1]	MAXIMUM DAY DEMAND (M3/D)[1]	PEAK HOUR DEMAND (M3/D)[2]
2009	43,153	54,554	Not Available
2010	43,411	57,592	Not Available
2011	44,150	51,558	65,443
2012	42,189	56,391	66,705
2013	42,827	59,601	65,135

[1] Sudbury Drinking Water System Annual Reports (2009 – 2013).

[2] From hourly SCADA data.

Average day water consumption was consistent between 2009 and 2013. The average consumption for the five year period was 43,146 m^3/d, while the highest was 44,150 m^3/d.

The highest maximum day flow recorded in the past four years was 59,601 m^3/d, occurring in 2013. This amounts to 81% of the estimated production capacity. The average historical maximum day demand is 55,939 m^3/d, or 76% of the estimated production capacity.

Hourly flow data was only available from 2011 to 2013. The maximum peak hour value recorded during that period was 66,705 m^3/d in 2012, and the average was 65,761 m^3/d.

The peaking factors derived from historical data were compared to those documented in the *City's Engineering Design Manual* (City of Greater Sudbury, 2012) and those included in the *MOECC Guidelines* (MOE, 2008).

The maximum day to average day peaking factor corresponding to the maximum day flow recorded (59,601 m^3/d in 2013) was 1.39, while the average maximum day peaking factor was 1.30. The City's Engineering Design Manual specifies a maximum day factor of 1.65 for Sudbury, which matches the corresponding value recommended in the *MOECC Guidelines.* The highest maximum day factor (1.39) was adopted to evaluate future requirements.

The peak hour to average day factor corresponding to the highest peak hour flow recorded in 2012 was 1.58, while the average peak hour factor was 1.53.

The *City's Engineering Design Manual* and the *MOECC Guidelines* specify a peak hour factor of 2.48. For purposes of estimating future demands, the historical maximum value (1.58) was adopted.

4.2 RAW WATER CHARACTERISTICS AND SECURITY OF SUPPLY

Source water protection studies and water budgets have been completed for the watersheds for the Sudbury water facilities, and most recently updated in September 2014. A water budget is a tool to identify the sources of water input to and output from a watershed or water system. They are used to characterize the pathways of water movement through a watershed and help understand water quantity issues, as well as water quality issues. Additional information for each system is provided in the Baseline Review Report for Water Systems (WSP, 2014), and highlighted in the sections below.

4.2.1 DAVID STREET WTP: RAMSEY LAKE

Tier 1, 2, and 3 stress assessments were completed for the David Street WTP and Ramsey Lake subwatershed. The findings of the Tier 1 and 2 assessments triggered a Tier 3 study to assess water quantity and quality threats. In summary, the Tier 3 assessment found that water quantity was not threatened (designation of 'low), although threats to water quality was assessed a 'high' risk.

Briefly, water quality threats included contamination from:

— Sodium from road salt application, snow storage, septic systems, as well as general handling and storage of road salt.
— Microcystin LR due to elevated phosphorus from waste disposal, septic systems, sewage lift stations, agriculture (e.g. commercial fertilizer, livestock, farm animal yards, etc.), and non-agricultural sources (e.g. untreated stormwater from stormwater retention ponds).
— Operation of waste disposal sites
— Stormwater runoff into the Ramsey Lake Intake
— Transportation of hazardous substances along transportation corridors (roadways, railways)

Sodium levels have been steadily increasing since 1991 from 32 mg/L to approximately 58 mg/L in 2013. Although 200 mg/L is the Ontario Drinking Water Quality Standard for sodium, values above 20 mg/L must be reported to a local medical officer of health.

Microcystin LR is a toxin sometimes produced by cyanobacteria (also known as blue-green algae) and is listed as a parameter of concern in the Ontario Drinking Water Quality Standards. High levels of phosphorous tend to promote cyanobacteria, some of which produce Microcystin. Therefore, the presence of phosphorous is associated with this issue. Several blooms have occurred in the last five years.

Several policies have been developed to address potential threats to Ramsey Lake within the Source Protection Plan for the Greater Sudbury Watersheds published in September 2014. The type of policy tools used to address these threats include education & outreach, land use planning, monitoring, prescribed instruments (such as legal instruments required by the Province of Ontario), risk management plans, transition provisions and specified actions. The implementation of these policies will help mitigate potential threats and reduce water quality issues to Ramsey Lake. However, to determine whether these actions actually reduce water quality threats will require extensive monitoring and reporting. Further details on each policy and monitoring policies are provided in the Source Protection Plan report.

Overall, as stated in the Source Water Protection Plan Report, David Street WTP and Ramsey Lake have a low risk of quantity concerns, but the risk of having issues related to water quality is high.

4.2.2 WANAPITEI WTP: WANAPITEI RIVER

Through the Tier 1 assessment, the Wanapitei River Subwatershed was determined to have a low risk of threats to water quantity. As such, the study for this subwatershed was completed at Tier 1, so water quality threats were not reviewed (water quality is reviewed under Tier 3).

4.2.3 GARSON WELLS AQUIFER

The Garson wells have detectable levels of tetrachloroethylene (PCE) historically ranging from 0 to 3.74 µg/L, and exhibiting an increase over time, as shown in Figure 4-1. The current maximum acceptable concentration (MAC) of PCE in drinking water is 30 µg/l, or about one order of magnitude greater than the highest value detected in the water. However, in September 2014, Health Canada proposed reducing the MAC to 10 µg/L (Health Canada, 2014), meaning that the detected amounts are approaching half of the limit. In addition, the MOECC recently requested the City to install groundwater monitoring wells in the area surrounding the Garson production wells. The purpose is to monitor PCE levels in the aquifer and provide an indication of potential future PCE levels. The City has installed four monitoring wells: MH12-01, MH12-02, MH12-03, and MH12-04. Measurements have been taken since November 2012 and levels at MH12-01 and MH12-04 have been below 0.25 µg/L since then. However, levels at MH12-02 have ranged between 0.87 µg/L and 2.98 µg/L while those at MH12-03 have range between 2.75 µg/L and 6.92 µg/L. A summary of the data is presented in the graph below.

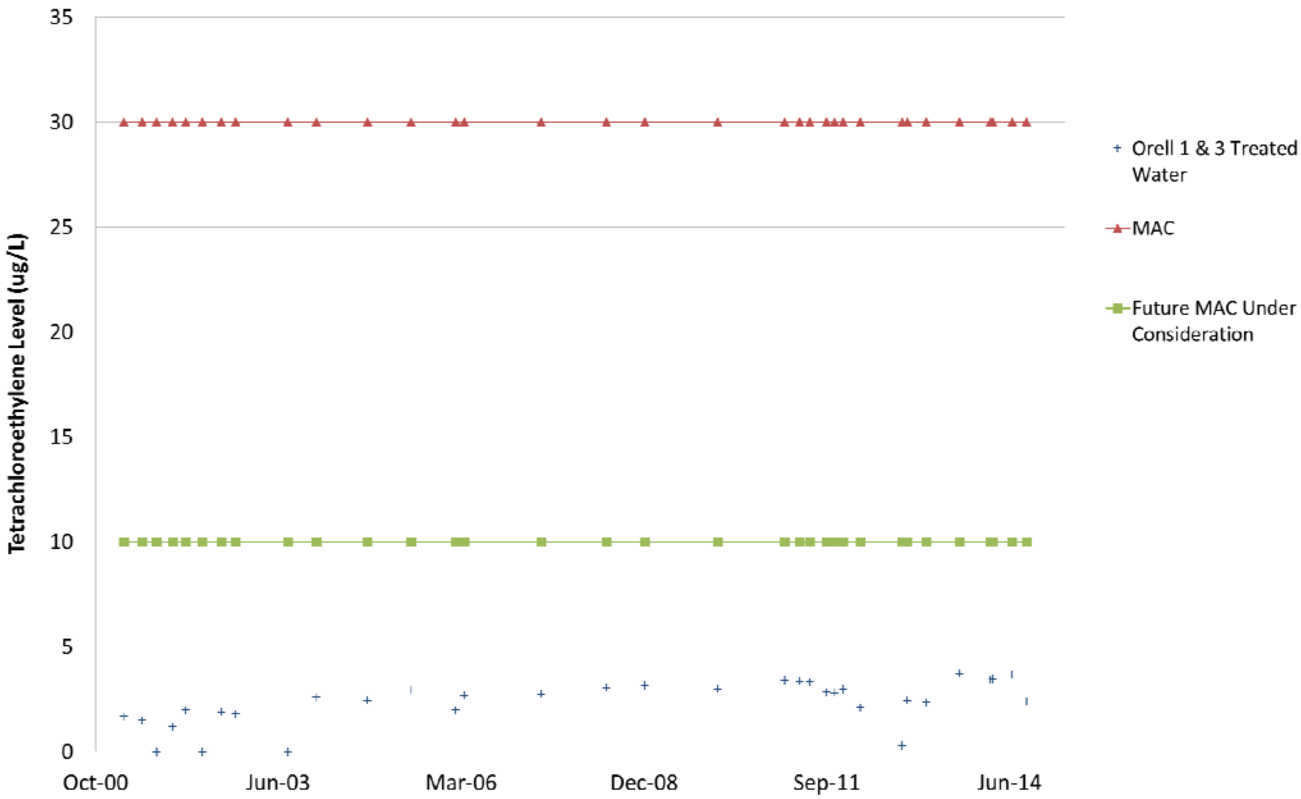

Figure 4-1 Historical PCE Levels in Treated Water at Wells 1 and 3

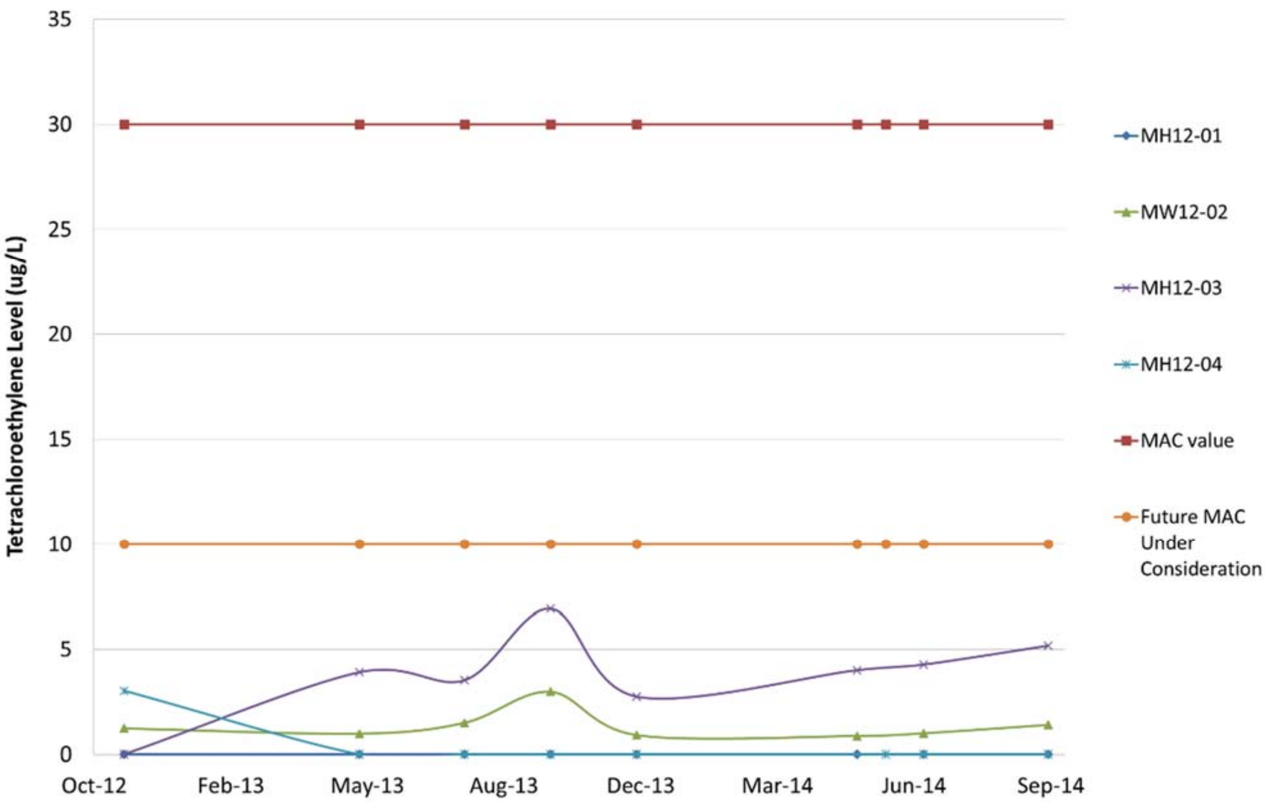

Figure 4-2 Monitoring Well PCE Levels

Currently, water from the Garson wells is disinfected prior to entering the distribution system, with no further treatment. PCE, however, requires advanced water treatment such as application of one or a combination of the following technologies:

— Adsorption by granular activated carbon (GAC)
— Air stripping by packed tower aeration (PTA)
— Ozonation or advanced oxidation
— Reverse osmosis (RO)

Assuming that advanced treatment is not provided and that raw water PCE levels continue to rise, there is a risk that the Garson wells may not be able to continue supplying to the distribution system. A sensitivity analysis was completed to quantify the potential risk of losing the Garson Wells due to PCE contamination, and the impact on supply capacity. This is discussed in Section 6.4.2.

4.2.4 OPERATIONAL DATA

Data reported in the *Annual Reports* for the Sudbury supply facilities includes effluent chlorine residual, trihalomethanes (THMs), fluoride, and trace organic and inorganic chemicals.

Data was reviewed from 2009 to 2013 to determine any historical issues. No exceedances were observed, except for elevated sodium levels David Street WTP (55.1 mg/L) and Garson Well 2 (60.3 mg/L).

5 DESIGN CRITERIA

The following design criteria were used to assess the remaining capacity of the existing systems and to forecast future requirements for the water and wastewater systems. The unit rates used to estimate future water and wastewater flows correspond to the values included in the *Population Projections and Unit Rates Technical Memorandum* (WSP, 2014). Otherwise, design criteria recommended in the *MOECC Guidelines* and *City's Engineering Design Manual* were used.

5.1 UNIT WATER DEMAND CRITERIA

The water demand criteria shown in Table 5-1 are from the unit rates recommended in the *Populations and Unit Rates Technical Memorandum* (WSP, 2014). The rates were reviewed against historical data, MOECC *Guidelines* (MOE, 2008), and current standards in the City's *Engineering Design Manual* (City of Greater Sudbury, 2012).

Both the *MOECC Guidelines* and *City Engineering Design Manual* recommend determining demands for institutional, commercial and industrial (ICI) users on a case by case basis. However, the following criteria for ICI demands were used for the purposes of this evaluation.

Table 5-1 Water System Design Criteria

CRITERIA	VALUE	REFERENCE
Average Day Residential Demand	350 L/cap/day	Average of historical values, rounded up to nearest 50 L/cap/day
Average Day Institutional & Commercial Demand	28 m^3/ha/d	MOECC Guidelines
Average Industrial Demand	35 m^3/ha/d	MOECC Guidelines
Domestic Demand Maximum Day Factor	1.38	Highest historical value
Domestic Demand Peak Hour Factor	1.58	Highest historical value

Residential average day demands are obtained by multiplying the residential unit rate by the service population. Similarly, average ICI demands are obtained by multiplying the corresponding unit rates to the areas of development, assuming 100% of the area would be developed and assuming 100% lot coverage on these properties.

Maximum day and peak hour demands are obtained by multiplying the average day demand by the corresponding peaking factor.

For purposes of this study, and in line with City standards and practices, a residential fire flow of 75 L/s over 1.75 hours and ICI fire flow of 150L/s over 2 hours were used.

5.2 DESIGN CRITERIA FOR WATER SYSTEM COMPONENTS AND OPERATION

5.2.1 TREATMENT CAPACITY

Water supply facilities are designed to supply the maximum day demands of the system.

Treatment facilities must be designed in accordance with the *Procedure for Disinfection of Drinking Water in Ontario* (Ontario, 2006). Drinking water treatment systems that obtain water from a surface water or GUDI well supply must achieve an overall performance providing as a minimum a 2-log (99%) removal or inactivation of *Cryptosporidium* oocysts, 3-log (99.9%) removal or inactivation of *Giardia* cysts, and 4-log (99.99%) removal or inactivation of viruses.

At least 0.5-log removal or inactivation of *Giardia* cysts and 2-log removal or inactivation of viruses must be provided through disinfection, while the remaining removal may be achieved through filtration or other equivalent treatment processes.

5.2.2 PUMPING CAPACITY

Pumping stations are rated based on their firm capacity. If sufficient floating storage is available in a particular pressure district, the MOECC defines firm capacity as the capacity of the station with the largest pump out of service. If there is insufficient or no floating storage, firm capacity is defined as the capacity with the two (2) largest pumps out of service (MOE, 2008).

For each pressure district, the pumping stations have to be designed to provide peak hour or maximum day plus fire demands (whichever are greater), if no floating storage is available. If sufficient floating storage is available, then the pumping station only needs to be designed to provide maximum day demands.

Most pressure districts in Sudbury service only small areas that are generally at a higher ground elevation compared to the surrounding area. In these cases, floating storage is not available and the districts are pressurized by the respective booster pumping station.

5.2.3 STORAGE CAPACITY

Storage requirements are based on the requirement to meet water demands that exceed the capacity of the treatment plant and to satisfy fire flow demands. When the capacity of the supply system is only capable of satisfying maximum day demands, storage requirements are determined using the following formula from the *MOE Guidelines* (MOE, 2008):

$$\mathbf{Storage = A + B + C}$$

Where: A = Fire Storage, B = Equalization Storage = 25% of maximum day demand, and C = emergency storage = 25% of (A+B).

Fire storage is the product of the maximum fire flow required in the system and the corresponding fire duration based on Fire Underwriters requirements (Fire Underwriters Survey, 1999).

When the system can supply more than just the maximum day demand (but less than the peak demand), the fire storage requirements can be determined using the following formula:

$$\mathbf{A = (Peak\ Demand - Pumping\ Station\ Firm\ Capacity) \times Fire\ Duration}$$

Where: peak demand is the greater of the peak hour demand and the maximum day plus fire demand.

Per *MOECC Guidelines*, floating storage should be designed such that the elevation of the equalization volume (B) is such that a minimum pressure of 275 kPa (40 psi) can be maintained in the system under peak hour flow conditions. The fire (A) and emergency (C) volumes should be at elevations that produce 275 kPa (40 psi) during peak hour demand conditions, and 140 kPa (20 psi) under the maximum day plus fire flow condition (MOE, 2008).

5.2.4 DISTRIBUTION CAPACITY

Watermains have to be sized to carry the greater of the maximum day plus fire flow or peak hour demand. The MOECC Guidelines recommend that the range of acceptable pressures under normal conditions (average to peak hour flows) is 275 kPa (40 psi) to 690 kPa (100 psi), while during fire flow conditions pressures may drop to 140 kPa (20 psi) (MOE, 2008). The maximum allowable water velocity in the distribution system is 3 m/s (MOE, 2008).

6 FUTURE REQUIREMENTS

6.1 POPULATION PROJECTIONS

As part of the City of Greater Sudbury Master Plan, population forecasts were developed for the 2016, 2021, 2026, 2031, 2036, 2041 and Ultimate Buildout planning years. Ultimate Buildout is defined as an estimate of what the demand from the total population and total number of households in the City of Greater Sudbury would be based on lands that are currently designated for development in the Official Plan within the existing settlement boundaries.

The City supplied planning data sheets with properties and development potential and the vacant residential and ICI land inventory, and Hemson Consultants, on behalf of the City, provided supplementary population projections. Data was provided for each water system boundary. These data were used in conjunction to develop the targeted population growth for each horizon year, as well as development phasing (discussed in the next section and in detail in the *Populations and Unit Rates Technical Memorandum*, WSP 2014).

In cases where the City's planning data sheets and Hemson's population projections forecasted fewer development units than the vacant land inventory for an area, then specific parcels (up to the City's and Hemson's unit projections) of developable units were selected. These parcels were selected based on the rationale provided in the City's Official Plan. That is, the Official Plan prioritizes that development take place in areas that are currently serviced, or where servicing can easily be extended. This focuses growth in existing urban areas until supply is no longer available in these areas.

Based on the planning data, the population of Sudbury is projected to increase by 4,583 people in 2041 and 31,796 by Ultimate Buildout.

The population projections to be used in the Master Plan are summarized in Table 6-1.

Table 6-1 Sudbury Water System Population Projections

YEAR	POPULATION
2011	94,868
2016	95,826
2021	97,059
2026	98,330
2031	99,056
2036	99,506
2041	99,450
Ultimate Buildout	126,663

The City's planning data does not specify target years for employment growth. However, vacant lands designated as ICI properties have been assigned to different stages of the development process by the City. These stages are described below and apply to both ICI and residential areas.

- Draft Approved:
  - These are lands that have draft plan of subdivision approval under the Planning Act or have pending applications with the City. Typically, these lands are close to registration or few years away from development as the required conditions are satisfied
  - Development approvals are near complete, and development could take place at any time. Properties with this designation were set to take place in 2016.
- Legal Lots of Record:

  - These are existing lots, including lots in a registered plan of subdivision. Typically these lands are zoned, serviceable and only require building permit approval for development. In some cases a site plan approval/agreement may also be required.
  - Based on historical trends, development is approximately 15 years away from receiving draft approval. Properties with these designations were assigned to take place in 2026.
- Designated Developable:
  - These lands do not have any development approvals in place but are understood to be areas of future development as they are within the settlement boundary. Designated lands are typically a number of years away from being developed.
  - Based on historical trends, these properties are approximately 10 years away from receiving Legal Lot of Record designation. Designated Developable properties were assumed to take place in 2036.

These land supply categories stem from the land supply requirements that municipalities must maintain under Section 1.4 of the Provincial Policy Statement. In this context, Designated Development Lands would count towards Section 1.4.1 (a) and Legal Lots of record and Draft Approved Lands would count towards 1.4.1 (b). It is also important to note that the total supply is governed by PPS Section 1.1.2.

The targeted ICI development areas for each horizon year are summarized in Table 6-2

Table 6-2 Sudbury Water System ICI Projections

	ICI DEVELOPMENT AREAS (HA)					
LAND USE	2016	2021	2026	2031	2036	2041
Institutional	0.00	0.00	8.11	0.00	0.00	0.00
Commercial	0.00	0.00	23.1	0.00	76.8	0.00
Industrial	171.5	0.00	71.0	0.00	208.1	0.00
Total	**171.5**	**0.00**	**102.2**	**0.00**	**284.9**	**0.00**

The above assumptions provide an estimate as to the ICI development time line. In reality, development may be more staggered. However, for purposes of infrastructure planning and to ensure that the appropriate infrastructure is in place by the appropriate planning horizon, the above assumptions are considered to be conservative.

6.2 PRIORITY EXTENSION LIST

The City has developed and maintained a Priority Extension List of existing residential and ICI streets that are not currently serviced by either or both municipal water or sewer, but at least one owner on the street has requested servicing. The City's policy on extension of services includes the following conditions:

- Before any project proceeds, the participation rate of benefitting property owners must be 100%, with those benefitting property owners funding 50% of the actual net cost of the project.
- The process must be initiated by property owners submitting a petition to the City of Greater Sudbury.
- At least 80% of the property owners in the project area must sign the petition.
- The project must be on the City's priority list for new servicing schemes, or, there must be demonstrated cause why the project should be included on the City's priority list for new servicing schemes.

In Sudbury, three streets have been placed on the priority list for sewer servicing. However, to date, the above conditions have not been met and City funding for extension requests is not available. Therefore, these streets have not been included in the demand projections for infrastructure planning as part of the Master Plan.

6.3 PHASING OF FUTURE GROWTH

Growth areas were allocated based on population projections for individual developments and the overall target growth population projections for Sudbury for the horizon years.

Hemson's supplementary tables were used to provide the target population, while the City's planning tables and vacant lot inventory were used to identify phasing of specific properties, and assignment of draft approved, legal lots of record, and designated development properties. In general, priority was given to draft approved properties, followed by legal lots of record and designated developable properties. In accordance with the Official Plan, the City has also assigned a target quantity of legal lots of record and designated developable properties to be developed in each horizon year. That is, legal lots of record should account for approximately 20% of all household growth, and designated developable lots are assigned 20% of the 20 year anticipated growth.

The future growth phasing plans were presented in the *Unit Rates and Population Projections Technical* Memorandum (WSP, 2014).

6.4 FUTURE WATER DEMAND PROJECTIONS AND INFRASTRUCTURE NEEDS

The unit flow criteria listed in Section 5.1 were used to estimate the future water demands in the Sudbury Water System. In general, the projected flows were calculated by the following formula:

$$\textbf{Projected Average Day Demand} = \textbf{Base Demand} + \textbf{Additional Residential Demand} + \textbf{Additional ICI Demand}$$

The demands corresponding to the population growth forecasts to ultimate buildout are listed in Table 6-3.

Table 6-3 Sudbury Water System Water Demand Projections

YEAR	POPULATION	AVERAGE DAY DEMAND (M3/D)	MAXIMUM DAY DEMAND (M3/D)	PEAK HOUR DEMAND (M3/D)
Base	94,868	44,150	59,601	66,705
2016	95,826	50,486	70,259	79,823
2021	97,059	50,918	70,860	80,506
2026	98,330	54,720	76,151	86,517
2031	99,056	54,974	76,505	86,919
2036	99,506	64,566	89,853	102,085
2041	99,450	64,546	89,826	102,054
Ultimate Buildout	126,663	74,071	103,081	117,113

The Base Demands were the highest historical (2009 to 2013) average day and maximum day demand as well as peak hour for the community. The additional residential demand was calculated using the unit flow rate multiplied by the population growth, and similarly, the ICI demand was calculated using the unit flow rate for each type of development (industrial, commercial or institutional), multiplied by the growth in development area.

Maximum day and peak hour demands were calculated by applying the respective peaking factor to the average day demand.

A desktop analysis of historical water demands and future water demand projections is included in **Appendix B.**

6.4.1 SUPPLY CAPACITY

The Sudbury Water System is supplied by two surface water treatment plants located in Sudbury and three wells, located in Garson. The Garson Wells generally only service the east end of Garson, but are interconnected with the rest of the Sudbury Water System through an isolation valve. In an emergency, the valve can be opened to integrate both systems together and permit flow to or from Garson.

The rated and estimated operating capacities for the Sudbury Water System were discussed previously in Section 3.4. The production capacity of the system is estimated to be 81,813 m^3/d.

The projected maximum day demands are plotted versus the total rated and firm production system capacities on Figure 6-1.

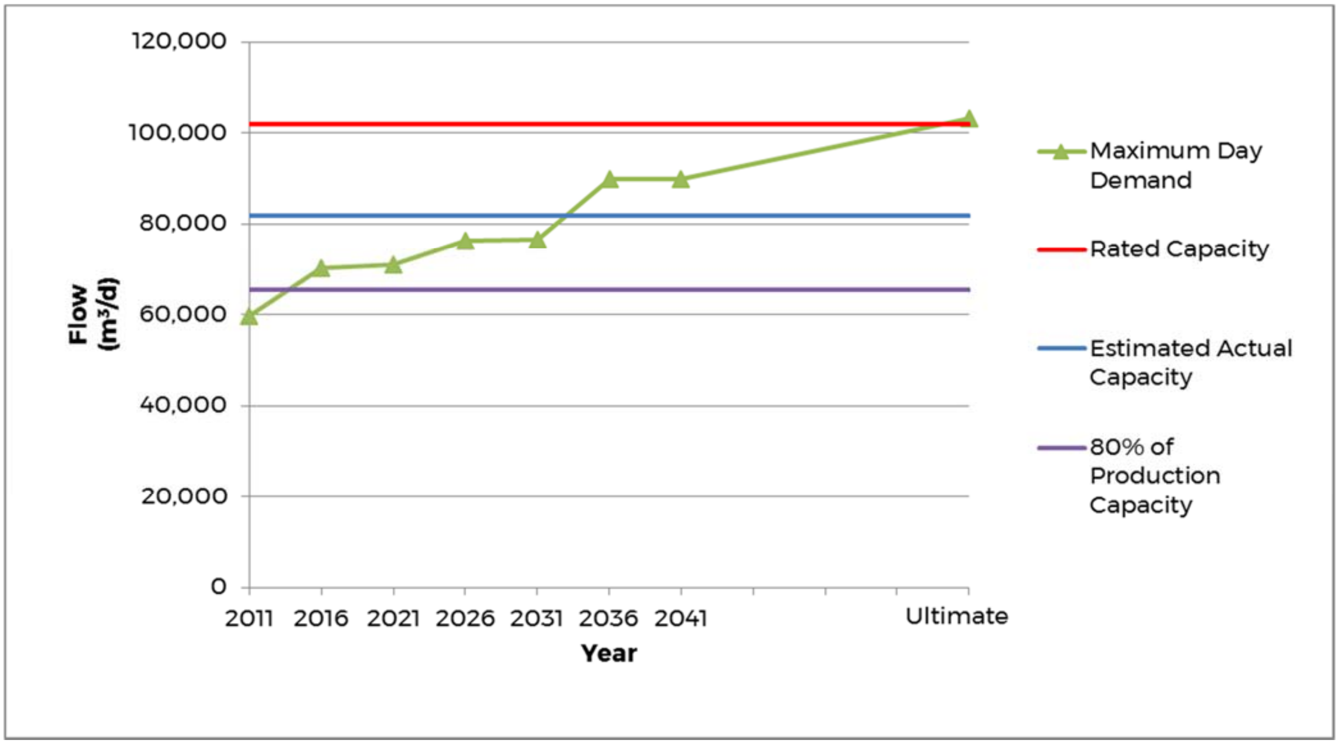

Figure 6-1 Water Demand Projections Compared to Rated Total and Estimated Firm Capacities

Therefore, the Sudbury Water System has sufficient maximum day capacity to service planned population growth to 2031. Additional supply is required to service growth beyond 2031.

However, generally capacity upgrades are triggered when a system reaches 80% of current production capacity. In this case, this is a maximum day flow of 65,450 m^3/d and means that planning for additional capacity should begin immediately.

6.4.2 SUPPLY CAPACITY SENSITIVITY ANALYSIS

A sensitivity analysis was completed on the supply capacity of the Sudbury Water System and the impacts on future system needs. Each scenario is described briefly in the table below.

The table also includes the production capacity, as well as 80% of the production capacity. Planning for additional supply should begin when demand reaches 80% of the production capacity.

Table 6-4 Sensitivity Analysis Scenarios

SCENARIO	DESCRIPTION	PRODUCTION CAPACITY (M3/D)	80% OF PRODUCTION CAPACITY (M3/D)
1 (Best Case)	Assumes all facilities operate to their rated capacity, and all facilities are in service	101,827	81,462
2 (Base)	Assumes the largest well (Well 3) is out of service	73,553	58,842
3	Scenario 2, but also assumes loss of all wells	69,000	55,200
4	Scenario 2, but also assumes loss of David Street WTP	49,827	39,862
5	Scenario 2, but also assumes the loss of Wanapitei WTP	34,827	27,862

The sensitivity analysis is presented in Figure 6-2.

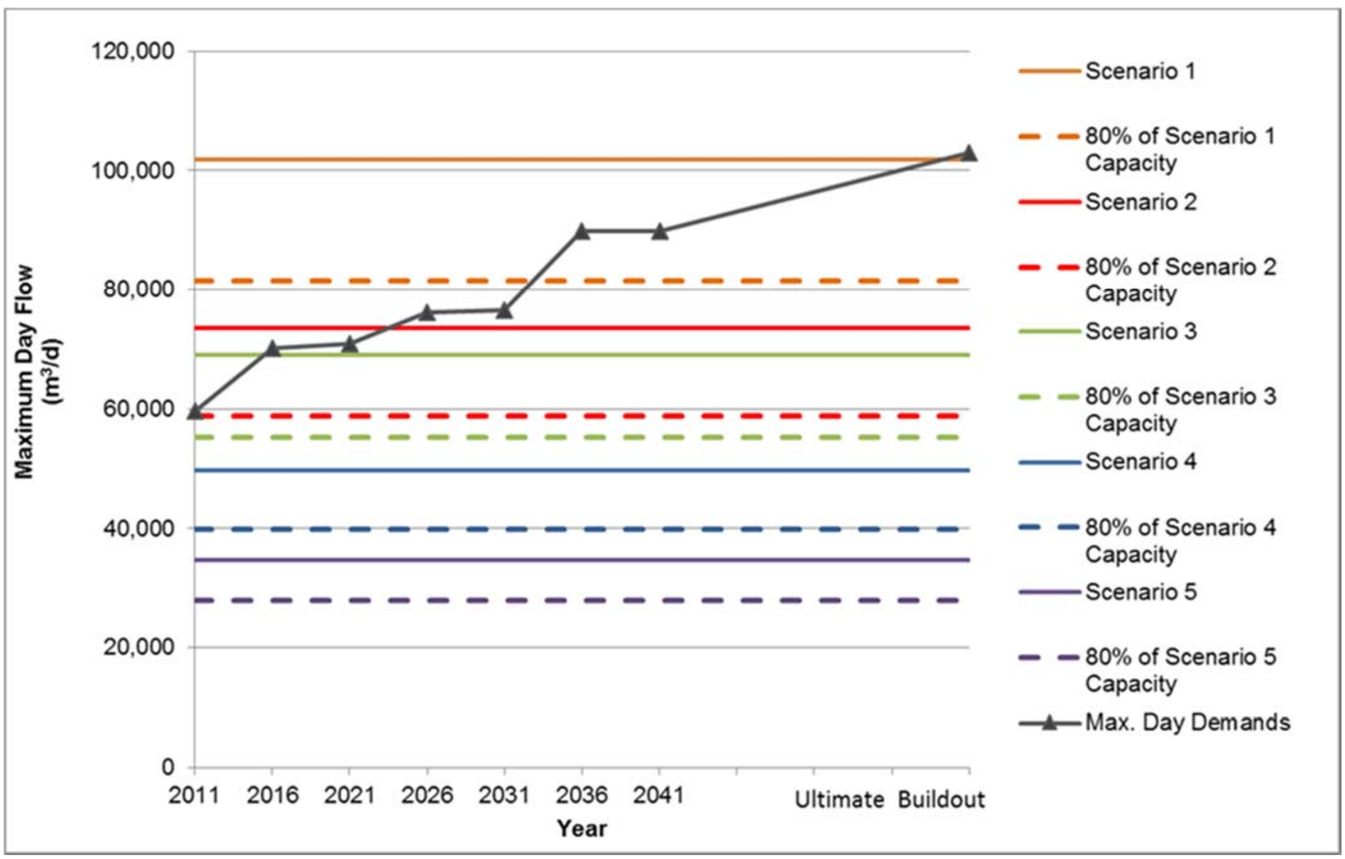

Figure 6-2 Sudbury Water Supply Sensitivity Analysis

Scenario 1 represents the “Best Case” scenario where all WTPs and wells operate to their full rated capacity. In practice, this is not feasible due to hydraulic and other limitations discussed earlier in this report.

Under the Base Case (Scenario 2), the system has sufficient capacity to service growth to 2021. However, loss of all of the Garson Wells results in an urgent need for additional capacity before 2016.

Loss of either the David Street WTP or Wanapitei WTP signifies a substantial drop in capacity, and inability of the system to meet current demands.

Inability to use the Garson Wells is a risk since the wells currently have detectable levels of PCE, and no treatment for PCE removal. If PCE levels continue to increase, and the Maximum Acceptable Concentration (MAC) drops, the wells would no longer be a reliable water supply without addition of treatment. This makes the system currently vulnerable.

6.4.3 STORAGE CAPACITY

Storage in the distribution system is provided by one storage tank, the Ellis Reservoir, located in Sudbury. The tank has a usable volume of 36.4 ML, but is only filled to 26.7 ML due to an increase in watermain breaks when the tank is filled beyond this volume, as observed by City staff. There is negligible distribution storage available at the David Street and Wanapitei WTPs. All storage at the plants is utilized for required chlorine contact time.

Applying the formula to determine storage requirements indicated previously (Section 5.2.3), the corresponding fire storage requirement would be 1.1 ML. Using the maximum day demand required to service current populations (59,601 m^3/d), the corresponding equalization storage requirement would be 14.9 ML and the emergency storage would be 4.0 ML. The total required storage to service current populations would be 20.0 ML, less than the current usable storage volume of 26.7 ML.

The total required storage to service growth to 2041 would be 29.4 ML and to Ultimate Buildout would be 33.6 ML (deficit of 19.5 ML). These volumes are less than the available storage volume of 36.4 ML. If modifications or upgrades can be made to the distribution system to prevent watermain breaks from occurring when the Ellis Reservoir is filled to its higher useable volume (36.4 ML), the system would have enough storage to service growth to Ultimate Buildout.

Therefore, the existing available storage provides sufficient capacity for the Sudbury Water System through to Ultimate Buildout, pending improvements to the system that allow use of the full volume. Without such improvements, the system has enough useable storage to service demands to at least 2031; by Ultimate Buildout, the system would have a potential deficit of 6.9 ML in this scenario.

The amount of storage required for each horizon year is shown in the figure below and compared to the currently useable storage volume (26.7 ML) and the total useable storage volume (36.4 ML).

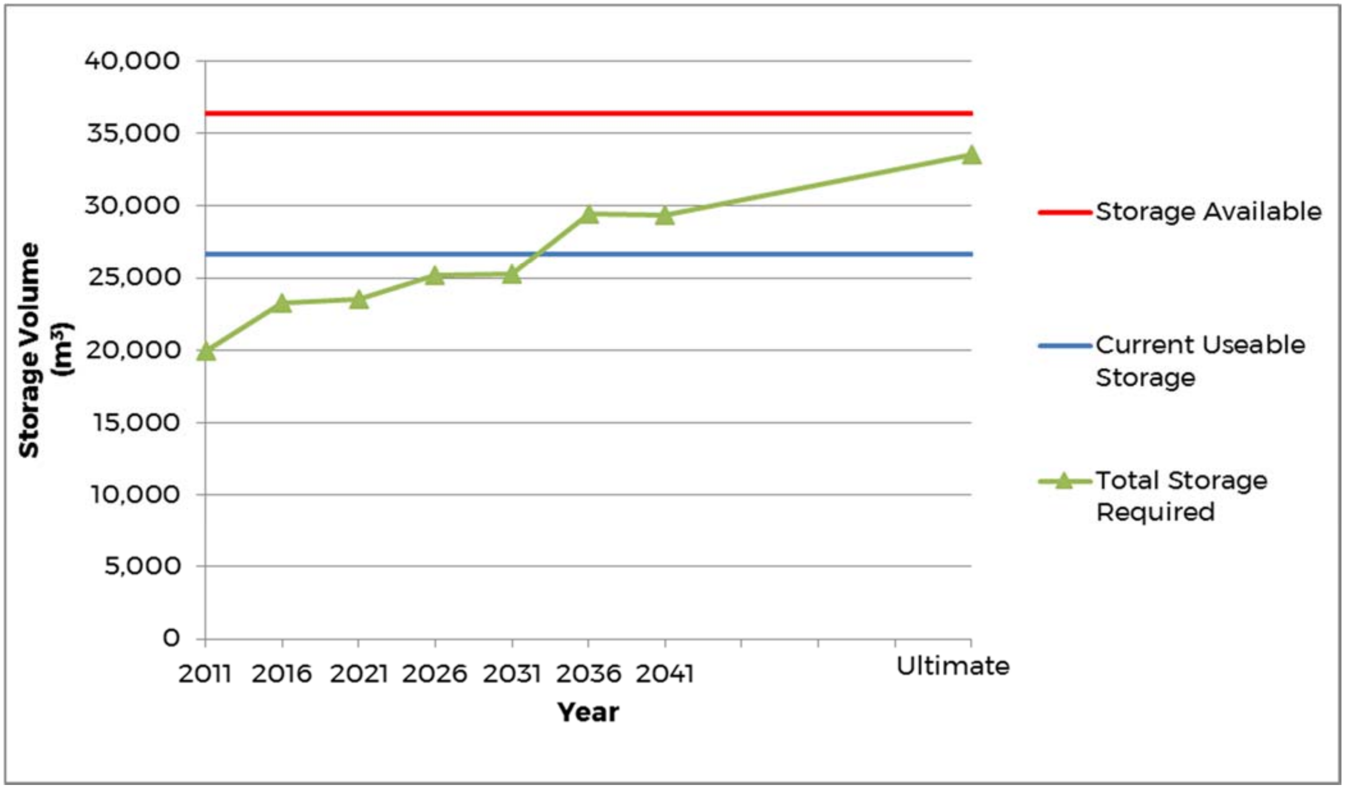

Figure 6-3 Available Storage Capacity Compared to Future Needs

6.4.4 DISTRIBUTION NETWORK

The water model was used to identify system elements (i.e. watermains, pumps, storage tank) for which the capacity was exceeded by the projected water demands. The capacity of the system was assessed in terms of the available fire flows and system pressures.

For each planning scenario, watermains of the modelled network were reviewed to assess whether the required minimum fire flows (75 L/s in residential areas or 150 L/s in ICI areas) and pressures (over 20 psi under fire conditions and over 40 psi under normal conditions) were achieved. Furthermore, some new watermains were added to service greenfield areas where development was planned. A simplified watermain layout was assumed for these areas.

Future populations and demands were loaded into the model based on the planning data and flow projections discussed in earlier in Section 6.4. In general, development might deviate from the proposed phasing scheme. Thus, it is recommended that the hydraulic water model be updated whenever a development application is submitted.

The findings from the water modeling are discussed in Section 7.1.2 and presented in **Appendix C**.

7 HYDRAULIC MODELING

An all-pipe model of the system including pipes, key hydrants, storage tanks and system water sources was developed by the City using Bentley Systems' WaterGEMS hydraulic modeling software. This model was updated based on information provided by the City to reflect current system conditions. WSP has also reviewed its earlier modeling memos for this area to ensure consistency with earlier studies, including but not limited to:

— Feb. 28, 2011 "Wanapitei Trunk Model Evaluation and Interconnect Diameter", dealing with the optimal inter-connections between the existing trunk main and a proposed, parallel main to the south. Inter-connection locations included Coniston (south end) and along Moonlight, where a 400mm replacement main was recommended.
— Nov. 11, 2011 "Proposed Surge Control Tanks at the Wanapitei WTP", dealing with the impact of new surge tanks on hydraulic transients along the existing trunk main and proposed 400mm inter-connect along Moonlight.
— Feb. 27, 2015 "Maley Drive Watermain Review", dealing with the proposed 600 mm replacement main's residence time, air handling and general impact on transmission.

The water model allows for simulations that can predict system responses to events under a wide range of conditions. Using simulations, problems can be anticipated in proposed or existing systems, and solutions can be evaluated before time, money, and materials are invested in a real-world project. Simulations can either be steady-state or extended-period.

Steady-state simulations represent a snapshot in time and are used to determine the operating behaviour of a system under static conditions. This type of analysis can be useful in determining the short-term effect of fire flows or average demand conditions on the system. Extended period simulations (EPS) are used to evaluate system performance over time. This type of analysis allows modeling the filling and emptying of storage facilities, regulating valves opening and closing, and pressures and flow rates changing throughout the system in response to varying demand conditions and automatic control strategies.

Simulations including steady-state analysis of the Average Day, Maximum Day and Maximum Day + Fire conditions were carried out using the model. Fire flow simulations were carried out throughout the system to determine whether the system could deliver fire flows under the Maximum Day demands.

7.1 WATER MODEL DEVELOPMENT

To model the current scenario, the following steps were taken:

— Total network demand on an average day basis was determined for the current scenario using 2012 water production data.
— The node demand allocations assigned in the model were based on 2012 meter records, assigned to the respective property and converted from an annual volume to the Average Day Demand (ADD) in L/s. In other cases where meter records showed zero flow, the value was manually adjusted to reflect a reasonable volume (e.g.: ADD) for a respective property, depending on land use.
— The maximum day peaking factor used for modeling purposes was 1.39. The peaking factor was applied to the average day demand value to determine the maximum day demand.
— The maximum day demand plus fire flow was used to assess the system since it was greater than the peak hour demand. The fire flow that was simulated depended on land use (that is, residential area fire flows were limited to 75 L/s while ICI area fire flows were higher at 150 L/s).

7.1.1 FIREFIGHTING CAPACITY

Firefighting capacity was assessed for the distribution system, with exception of areas not designed to convey fire flows. These include areas that were constructed under different design standards; these areas have small diameter (150 mm or less) watermains and no fire hydrants. As such, these were not included in the below assessment.

As noted above, fire flow requirements of 75 L/s for residential areas and 150 L/s for ICI areas were used. Based on these criteria, the model revealed that flows meet current fire flow standards in most areas of Sudbury. There are small areas throughout the distribution system that do not meet current fire flow standards, as illustrated in **Appendix C**. Similar trends are observed for 2041 and Ultimate Buildout scenarios, as shown in **Appendix C**.

Water model outputs, including maps showing fire flow analysis, are provided in **Appendix C.**

7.1.2 MODELED HYDRAULIC CAPACITY UNDER NORMAL CONDITIONS

Based on the system modeling, service pressures throughout the system under the maximum day demand scenario generally range between 40 and 105 psi (276 and 724 kPa) for 2011 ADD. There are five (acceptable) exceptions in Zone 1 where pressure is between 38 and 40 psi, as listed below. Note also that the MOECC recommends, but does not require, pressures of at least 40 psi during average day demands, as indicated in the guidelines.

- Antwerp Avenue in Zone1, serviced by a 150 mm Cast Iron pipe installed in 1945, with an effective roughness parameter C=35 – as for many pipes in the area. These pipes need to be rehabilitated or replaced to improve hydraulic performance and reduce the risk of poor water quality.
- Pressures between 105 and 108 psi in the vicinity of the discharge header of the David St. WTP.
- Pressures between 105 and 119 psi along the trunk watermain from the Wanapitei WTP to just past Coniston.
- Two proposed developments on high ground result in pressures below 40 psi during ULTADD, including parcels with ID 3444 (south-west end of Zone 1) and 3383 west of the intersection of Montrose Ave. and Woodbine Ave.

For the Ultimate Buildout ADD, the corresponding extreme pressures are 27 and 105 psi (186 and 724 kpa), with the same local exceptions as were noted for 2011ADD.

Therefore, flows throughout the system are generally within the range prescribed in the MOECC Guidelines (40 to 100 psi) under normal conditions, but are slightly lower beyond 2011 in some areas.

During 2011 ADD or MDD, minimum pressures exceed 40 psi throughout Zones 2 to 12; as well as the Copper Park boosted zones and the Mount Adam and Goodview PRV Zones. Maximum pressures are generally below 100 psi with a few local exceptions, near locations discussed above. In future years, different booster or PRV settings may be required to limit minimum and maximum pressures, possibly combined with pipe relining or local pressure control, as documented separately in the Alternatives report.

Maps showing pressures at nodes are presented in **Appendix C**.

8 CONCLUSIONS AND RECOMMENDATIONS

An assessment of the Sudbury Water System was completed to identify infrastructure investment requirements to service forecasted growth in the community. The assessment involved a review of previous studies, an analysis of operations and flow data from the water facilities, and an evaluation of the capacity of the system.

The conclusions and recommendations of the assessment are summarized below.

— Based on the estimated production capacity of the Sudbury Water System as well as historical and projected demands, the system has sufficient capacity to service growth to 2031. Additional supply will be needed to service growth beyond 2031 and planning should begin immediately since the system is currently at 80% capacity.

— Ramsey Lake is a vulnerable water supply and may not be sustainable in the future due to water quality threats, as documented in Source Water Protection documentation. Similarly, the Garson Wells have detectable levels of tetrachloroethylene and must continue to be monitored. The wells may require treatment in the future to meet water quality requirements, if PCE levels continue to increase.

— The system has enough storage at the Ellis Reservoir for servicing to 2031. Beyond 2031, the system should be modified to remove the hydraulic restriction and therefore allow full use of the Ellis Reservoir total capacity (36.4ML)

— The model revealed that flows meet current fire flow standards in most areas of the Sudbury Water System, except in certain areas with high ground throughout the system. Note that small diameter watermains (150 mm or smaller) were constructed to meet design standards in place at the time of construction and may not meet current standards. Such small diameter water mains were not designed to supply fire flows.

— In many cases, very old and/or rough cast iron distribution mains dissipate excessive amounts of energy, limiting the available fire flow as well as peak hour pressures. Similarly, excessive breakage near the Ellis Reservoir and along Maley Drive have limited the City's ability to operate the system in terms of maximum levels and pressures, respectively. The works required to bring areas of the City up to operating pressure limits and/or fire flow targets will be modelled and packaged on an incremental cost-benefit basis in the Alternatives report.

— The transmission main from the Wanapitei WTP to the Ellis Reservoir has a pressure limitation that should be addressed to remove operational constraint and reduce the risk of a break. The location that would require repair is difficult to access and there may be alternatives, such as a locally-twinned line and/or a booster pumping station, both limiting pressures in the affected area. Another alternative is to construct a parallel trunk. These options will be modelled and discussed in the Alternatives Report.

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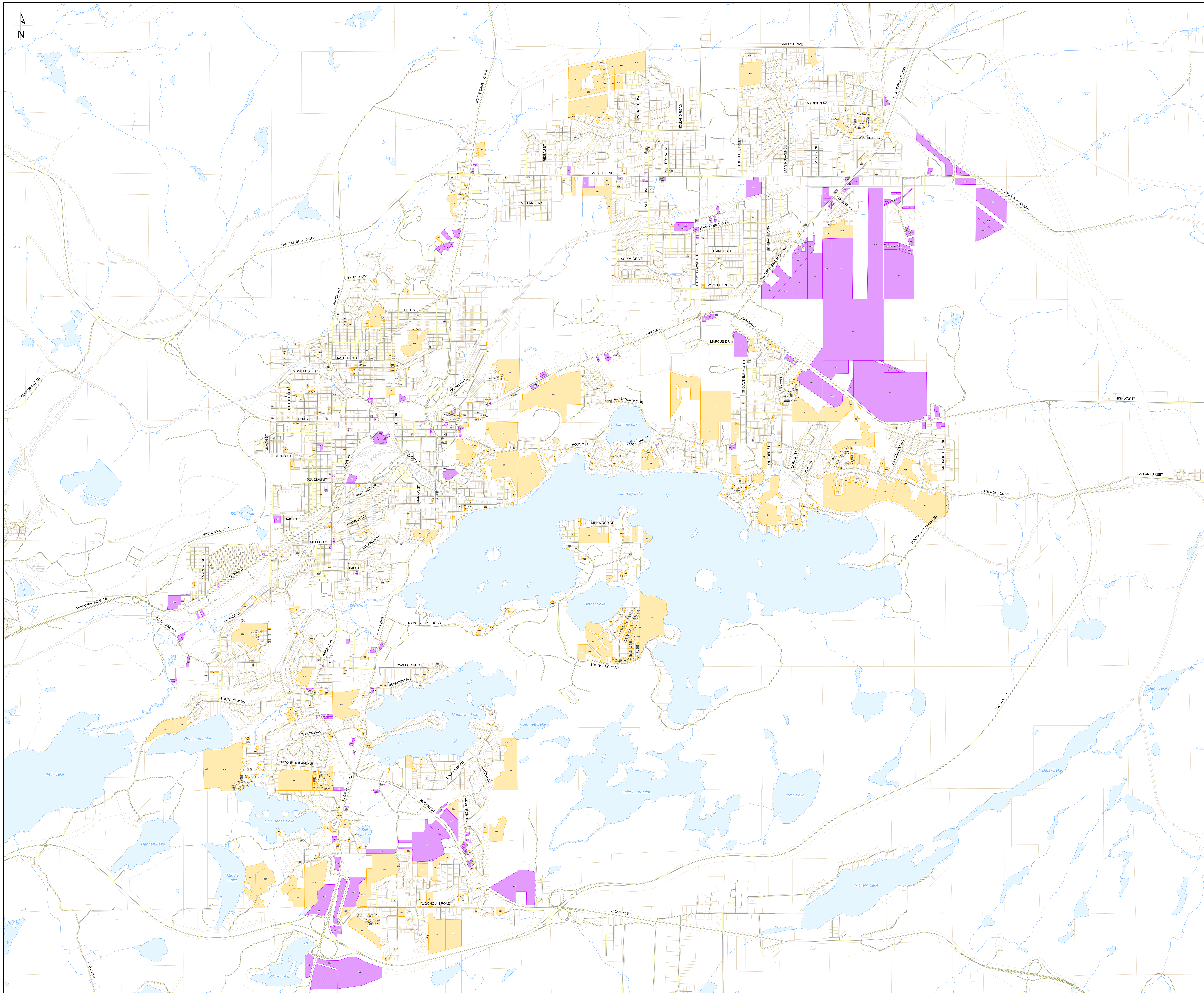
APPENDIX

A RESIDENTIAL AND ICI DEVELOPMENT AREAS



Legend

- Road
- Railway
- Watercourse
- Waterbody
- Parcel
- Vacant ICI
- Vacant Residential

Sudbury | WSP

Project No. 121-23026-00

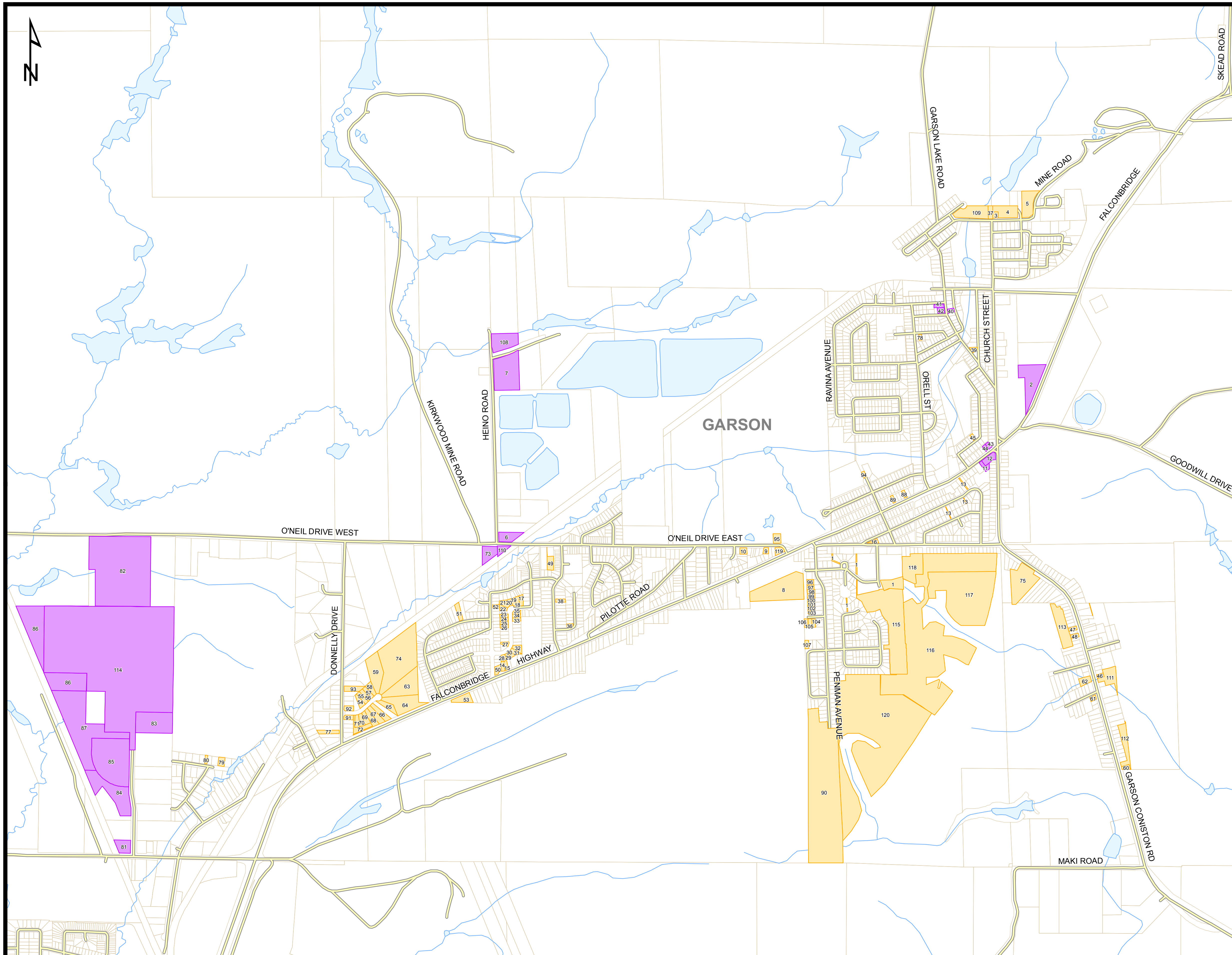
Sudbury Water and Wastewater Master Plan

0 500 1,000 Meters

1:15,000

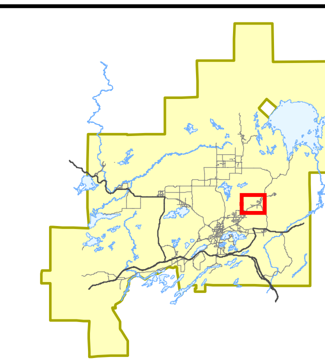
Map 1

Sudbury
2011 - Ultimate Buildout
Residential and ICI
Development Areas



Legend

- Road
- Watercourse
- Waterbody
- Parcel
- Vacant ICI
- Vacant Residential



Greater Grand Sudbury | WSP

Project No. 121-23026-00

Sudbury Water and Wastewater Master Plan

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Map 2

Garson
2011 - Ultimate Buildout
Residential and ICI
Development Areas

APPENDIX

B WATER DEMAND CAPACITY ASSESSMENTS

Sudbury - Water Demand Forecasts 5/11/2017

(Includes: Sudbury, Coniston, and Wahnapitae)

DATA ANALYSIS

	2009	2010	2011	2012	2013	Average	Design Criterion	Comments
Average Day Flow (m^3/d)	43,153	43,411	44,150	42,189	42,827	43,146	44,150	From Water Historical Production data. The daily production values for each facility were added together to determine the total daily production.
Max Day Flow (m^3/d)	54,554	57,592	51,558	56,391	59,601	55,939	59,601	
Max Day Factor	1.26	1.33	1.17	1.34	1.39	1.30	1.39	MOE Guidelines recommend a value of 1.65 for populations between 75,000 and 150,000. The maximum value over the past five years was 1.39, which is generally consistent with previous years.
Peak Hour (L/s)	Not Available		757	772	754			Peak values were available only for 2011-2013.
Peak Hour (m^3/d)			65,443	66,705	65,135	65,761	66,705	
Peak Hour Factor			1.48	1.58	1.52	1.53	1.58	MOE Guidelines recommend a value of 2.48 for populations between 75,000 and 150,000. The maximum value over the past five years was 1.58, which is generally consistent with previous years.

	2009	2010	2011	2012	2013	Design Criterion	2016	2021	2026	2031	2036	2041	Ultimate	Comments
Population (Existing Areas)	94,868	94,868	94,868	94,868	94,868	94,868	94,868	94,868	94,868	94,868	94,868	94,868	94,868	From data provided by Hemson grouped by water system.
Population Growth							958	2,191	3,462	4,188	4,639	4,583	31,796	
Total Population							95,826	97,059	98,330	99,056	99,506	99,450	126,663	From data provided by Hemson grouped by water system.
Residential Growth Area (ha)							10.2	8.4	16.3	28.3	11.1	10.1	345.7	From City's GIS database. 2036 and 2041 areas are included with 2031.
Residential Growth Area (ha) - Cumulative							10.2	18.6	34.9	63.2	74.3	84.4	430.1	
Institutional Growth Area (ha)							0.0	0.0	8.11	0.0	0.0	0.0	0.0	
Institutional Growth Area (ha) - Cumulative							0.0	0.0	8.11	8.11	8.11	8.11	8.11	From City's GIS database.
Commercial Growth Area (ha)							0.0	0.0	23.1	0.0	76.8	0.0	0.0	
Commercial Growth Area (ha) - Cumulative							0.0	0.0	23.09	23.09	99.93	99.93	99.93	From City's GIS database.
Industrial Growth Area (ha)							171.5	0.0	70.96	0.0	208.1	0.0	0.0	
Industrial Growth Area (ha) - Cumulative							171.5	171.5	242.4	242.4	450.5	450.5	450.5	From City's GIS database.
ICI (ha) - Cumulative							171.5	171.5	273.6	273.6	558.5	558.5	558.5	Sum of Institutional, Commercial and Industrial areas
Total Growth Area (ha) - Cumulative							181.7	190.1	308.6	336.9	632.9	643.0	988.7	

	2009	2010	2011	2012	2013	Average	Design Criterion	Comments
Ratio of Residential to Total Water Billed	0.74	0.74	0.74	0.74	0.74	0.74		Estimated amount of water consumption related to ICI based on metering data and obtained ratio of residential to total consumption.
Residential Flow (m^3/d)	31933	32124	32671	31220	31692	31,928		Calculated based on ratio of residential consumption to total consumption.
Ratio of ICI to Total Water Billed	0.26	0.26	0.26	0.26	0.26	0.260		
	11220	11287	11479	10969	11135	11,218		
Per Capita Residential Demand (m^3/cap/day)	0.337	0.339	0.344	0.329	0.334	0.337	0.350	Took average over 2009 to 2013 period. The trend is generally consistent.
Average Institutional Flow Unit Rate (m^3/ha/d)							28.0	*MOE Guidelines recommend a value of 28 m^3/ha/d.*
Average Commercial Flow Unit Rate (m^3/ha/d)							28.0	*MOE Guidelines recommend a value of 28 m^3/ha/d.*
Average Industrial Flow Unit Rate (m^3/ha/d)							35.0	*MOE Guidelines recommend a value of 35 m^3/ha/d for light industry and 55 m^3/ha/d for heavy industry.*

	2016	2021	2026	2031	2036	2041	Ultimate
Average Residential and ICI Flows (m^3/d) - Existing	44,150	44,150	44,150	44,150	44,150	44,150	44,150
Average Residential Flows (m^3/d) - Growth	335	767	1,212	1,466	1,624	1,604	11,128
Average Residential Flows (m^3/d) - Total	44,485	44,916	45,361	45,615	45,773	45,753	55,278
Average Institutional Flow (m^3/d)	0	0	227	227	227	227	227
Average Commercial Flow (m^3/d)	0	0	647	647	2798	2798	2798
Average Industrial Flow (m^3/d)	6,001	6,001	8,485	8,485	15,768	15,768	15,768
Average ICI Flow (m^3/d)	6,001	6,001	9,358	9,358	18,793	18,793	18,793
Average Day Flow (m^3/d)	50,486	50,918	54,720	54,974	64,566	64,546	74,071
Max Day Flow (m^3/d)	70,259	70,860	76,151	76,505	89,853	89,826	103,081
Peak Hour Flow (m^3/d)	79,823	80,506	86,517	86,919	102,085	102,054	117,113

ALTERNATIVE CALCULATION METHOD This method does not distinguish between Residential and ICI water consumption.

	2009	2010	2011	2012	2013	Average	Design Criterion	Comments
Per Capita Demand (m^3/cap/day)	0.455	0.458	0.465	0.445	0.451	0.455	0.455	*If ICI is not considered explicitly and demand is divided by total population.*
Max Day Factor							1.39	*The historical per capita consumption is applied for future development.*
Peak Hour Factor							1.58	

	2016	2021	2026	2031	2036	2041	Ultimate
Average Day Flow (m^3/d)	43,582	44,143	44,721	45,051	45,256	45,230	57,607
Max Day Flow (m^3/d)	60,651	61,431	62,236	62,695	62,980	62,945	80,169
Peak Hour Flow (m^3/d)	68,907	69,794	70,708	71,230	71,554	71,514	91,082

SENSITIVITY ANALYSIS

Analyze sensitivity of forecasted average day flows to unit rate

Average Day Flow (m^3/d)

	Unit Rate (m^3/cap/d)	2016	2021	2026	2031	2036	2041	Ultimate
Using a consolidated per capita flow	0.455	43,582	44,143	44,721	45,051	45,256	45,230	57,607
Using estimated average	0.350	50,486	50,918	54,720	54,974	64,566	64,546	74,071
City Standards	0.41	50,543	51,049	54,927	55,225	64,844	64,821	75,978

Analyze sensitivity of forecasted flows to max day peaking factor

Max Day Flow (m^3/d)

	Max Day Peaking Factor	2016	2021	2026	2031	2036	2041	Ultimate
2009-2013 average of peaking factors	1.30	65,500	66,060	70,992	71,322	83,766	83,741	96,098
Maximum historical max day factor	1.39	70,259	70,860	76,151	76,505	89,853	89,826	103,081
MOE Guidelines	1.65	83,302	84,014	90,287	90,706	106,533	106,501	122,217

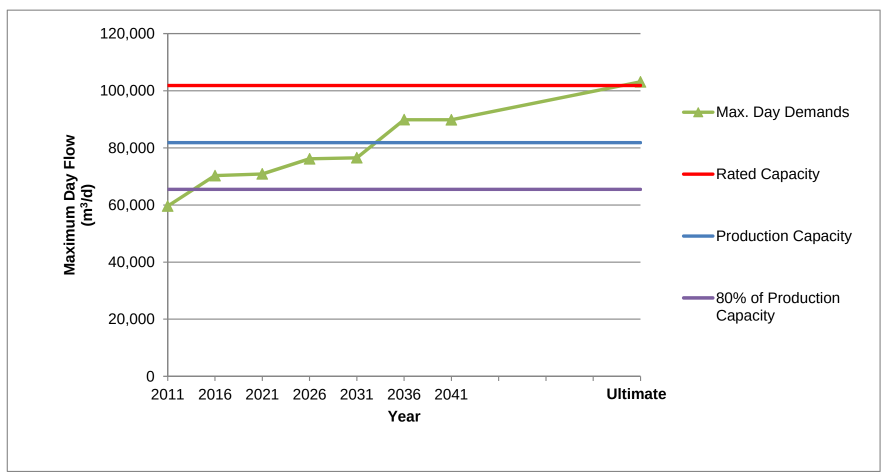
Analyze sensitivity of forecasted flows to peak hour factor

Peak Hour (m^3/d)

	Peak Hour Peaking Factor	2016	2021	2026	2031	2036	2041	Ultimate
Using historical highest peak factor	1.58	79,823	80,506	86,517	86,919	102,085	102,054	117,113
Using average of historical peaking factors	1.53	77,147	77,807	83,617	84,005	98,662	98,633	113,187
MOE Guidelines	2.48	125,205	126,276	135,704	136,335	160,123	160,074	183,695

CAPACITY CHECK

	2011	2016	2021	2026	2031	2036	2041	Ultimate
Rated Capacity	101,827	101,827	101,827	101,827	101,827	101,827	101,827	101,827
Production Capacity	81,813	81,813	81,813	81,813	81,813	81,813	81,813	81,813
Max. Day Demands	59,601	70,259	70,860	76,151	76,505	89,853	89,826	103,081
Peak Hour Demands	65,443	79,823	80,506	86,517	86,919	102,085	102,054	117,113
80% of Production Capacity	65,450	65,450	65,450	65,450	65,450	65,450	65,450	65,450

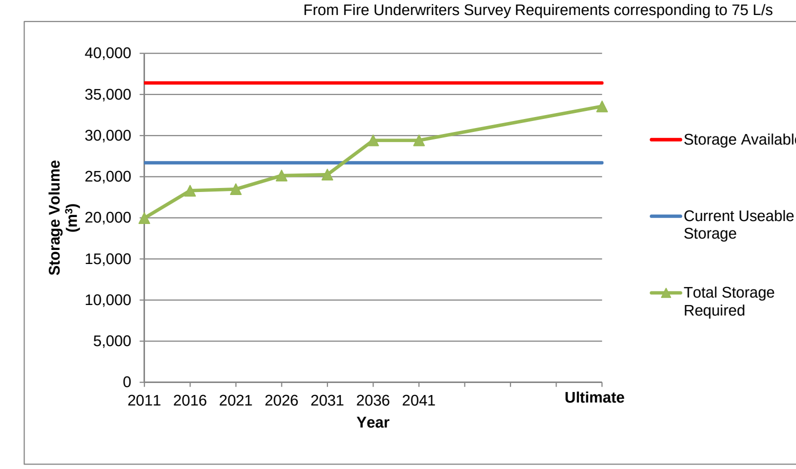


STORAGE REQUIREMENTS

Storage Available			
Total Available Storage (m^3)	36,400		From DWWP 016-206, Issue 2 (November 3, 2011)
Total Storage (m^3)	26,700	Usable volume	Estimated from City's typical highest operating level
Maximum Fire flow Requirements (L/s)	150		
Fire Duration (hrs)	2		
Minimum Fire Flow Requirement for Residential Areas (L/s)	75		From CGS Engineering Design Manual
Fire Duration (hrs)	1.75		From Fire Underwriters Survey Requirements corresponding to 75 L/s

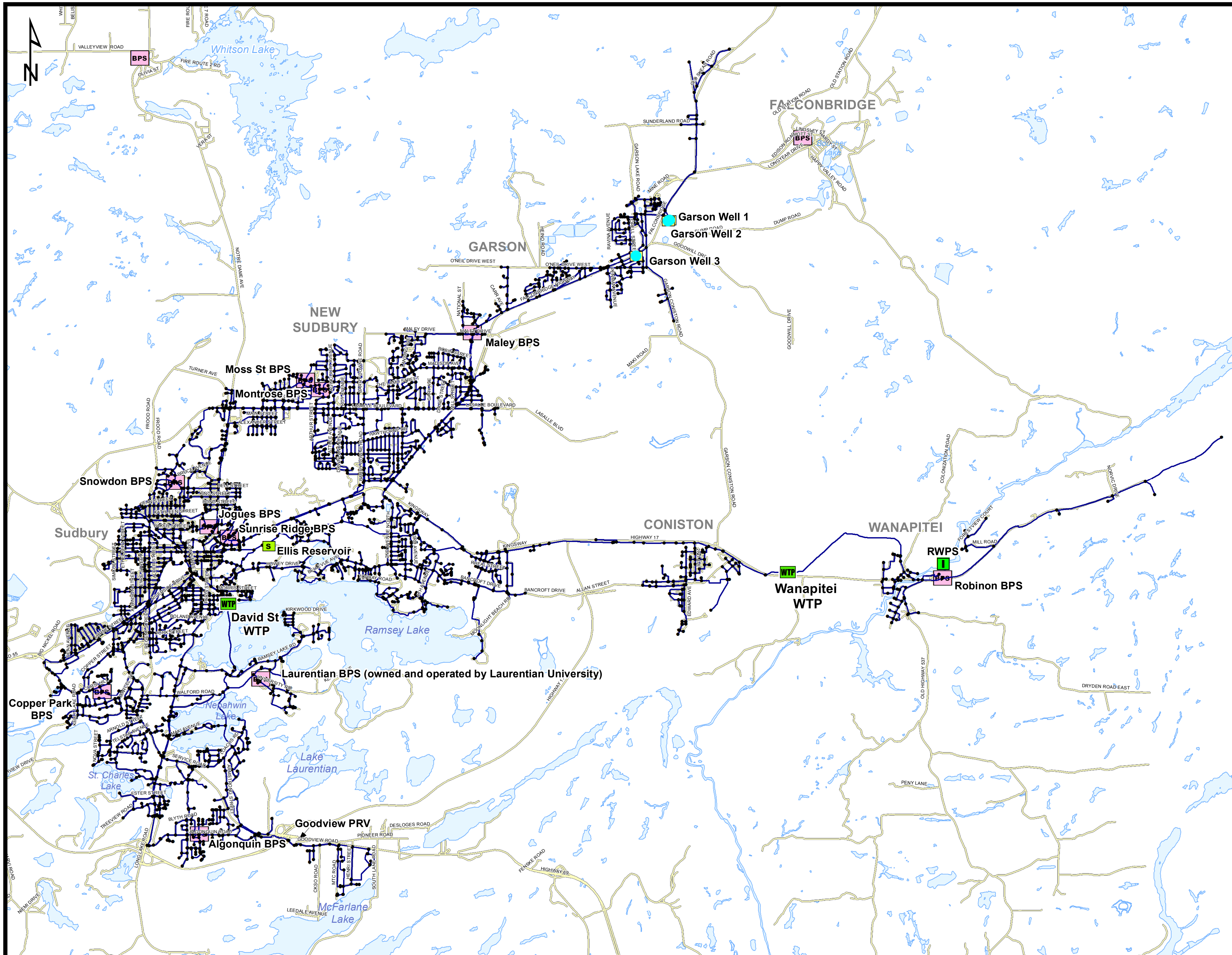
	Max Day Demand (m^3/d)	Required Fire Flow (m^3/d)	Max Day + Fire (m^3/d)	Peak Hour (m^3/d)	A - Fire Storage (m^3)	B - Equalization Storage (m^3)	C - Emergency Storage (m^3)	A + B + C = Storage Required (m^3)	Current Useable Storage (m^3)	Deficit (m^3)*
2011	59,601	12,960	72,561	65,761	1,080	14,900	3995.1	19,975	26,700	0
2016	70,259	12,960	83,219	79,823	1,080	17,565	4661.2	23,306	26,700	0
2021	70,860	12,960	83,820	80,506	1,080	17,715	4698.7	23,494	26,700	0
2026	76,151	12,960	89,111	86,517	1,080	19,038	5029.4	25,147	26,700	0
2031	76,505	12,960	89,465	86,919	1,080	19,126	5051.5	25,258	26,700	0
2036	89,853	12,960	102,813	102,085	1,080	22,463	5885.8	29,429	26,700	2,729
2041	89,826	12,960	102,786	102,054	1,080	22,457	5884.1	29,421	26,700	2,721
Ultimate	103,081	12,960	116,041	117,113	1,080	25,770	6712.6	33,563	26,700	6,863

* Deficit is calculated against the current useable storage, not the total available storage in Sudbury.



APPENDIX

C WATER MODEL RESULTS



Legend

- W Well
- S Storage Tank
- Raw Water Pumping Station (RWPS)
- BPS Booster Pumping Station (BPS)
- WTP Water Treatment Plant (WTP)
- Pipe
- Road
- Waterbody

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Project No. 121-23026-00

April 04 2016

Sudbury Water and Wastewater Master Plan

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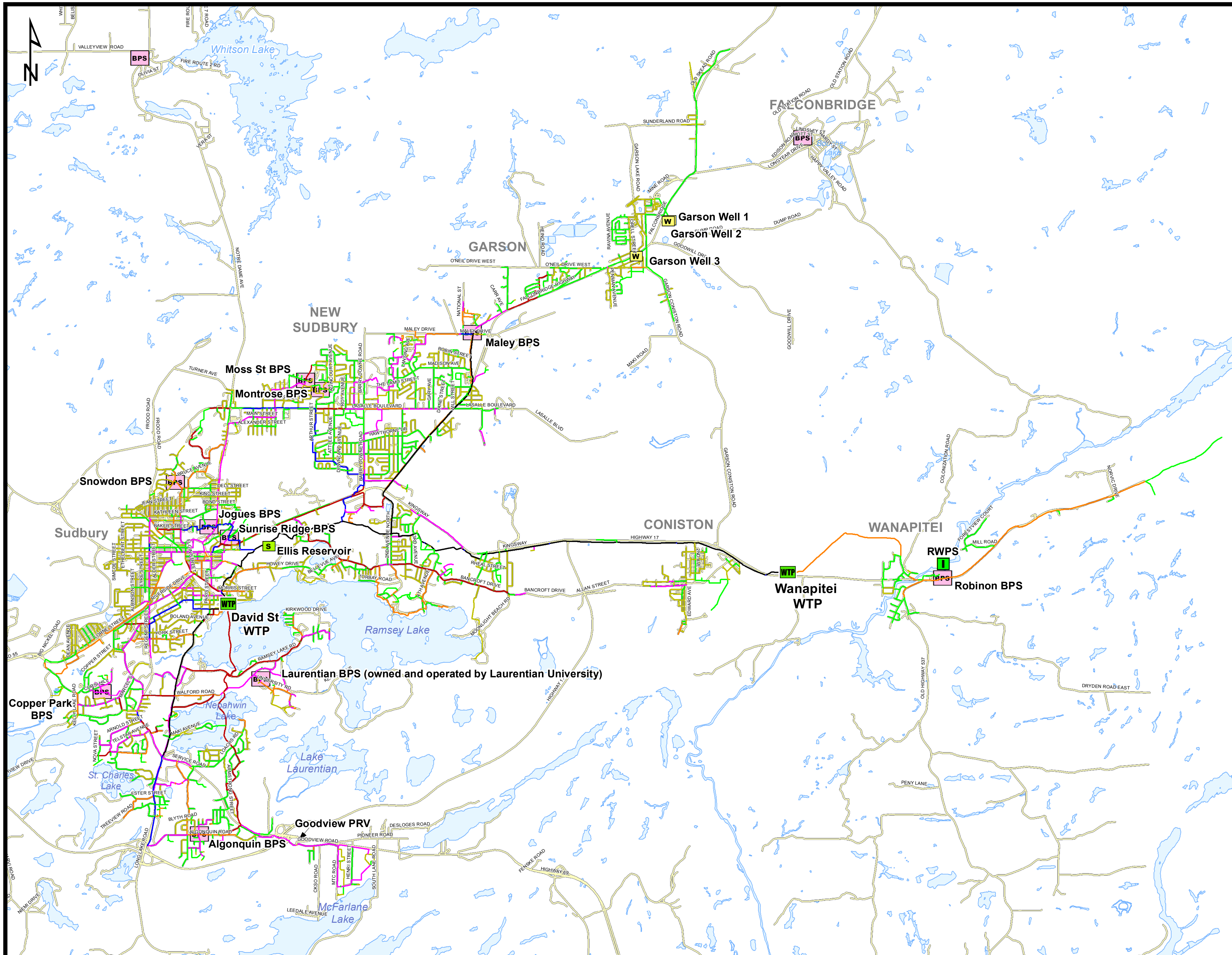
Map 1

Sudbury Water System - Key Plan



Legend

Symbol	Description
W	Well
S	Storage Tank
I	Raw Water Pumping Station (RWPS)
BPS	Booster Pumping Station (BPS)
WTP	Water Treatment Plant (WTP)
	Road
	Waterbody

Pipe: Diamater (mm)

- <= 150
- <= 200
- <= 250
- <= 300
- <= 400
- <= 600
- > 600

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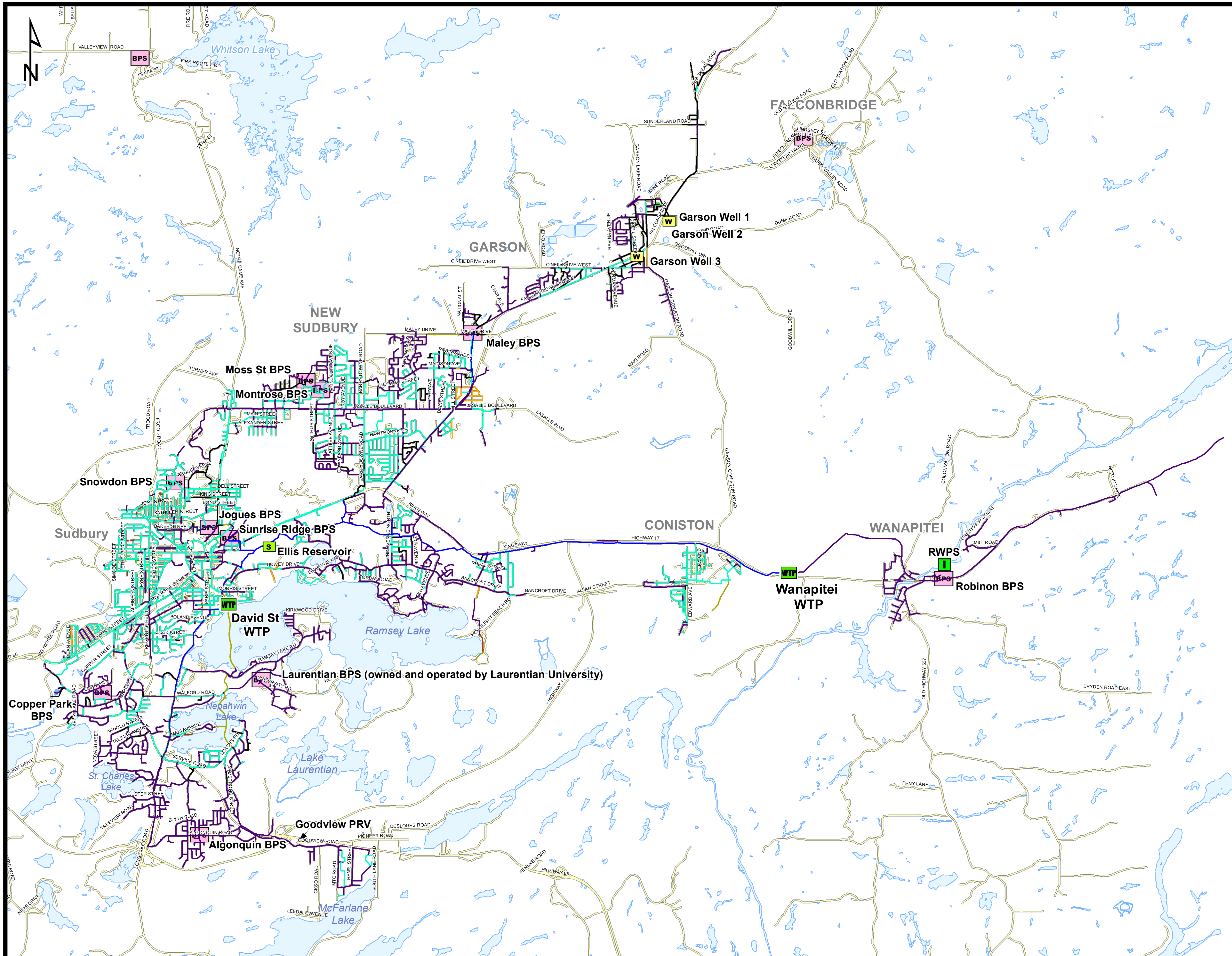
Map 2

Sudbury Water System - Pipe Diameter



Legend

- W Well
- S Storage Tank
- Raw Water Pumping Station (RWPS)
- BPS Booster Pumping Station (BPS)
- WTP Water Treatment Plant (WTP)
- Road
- Waterbody

Material

- Asbestos Cement
- Cast iron
- Concrete
- Copper
- Ductile Iron
- Galvanized iron
- PVC
- Polyethylene
- Steel

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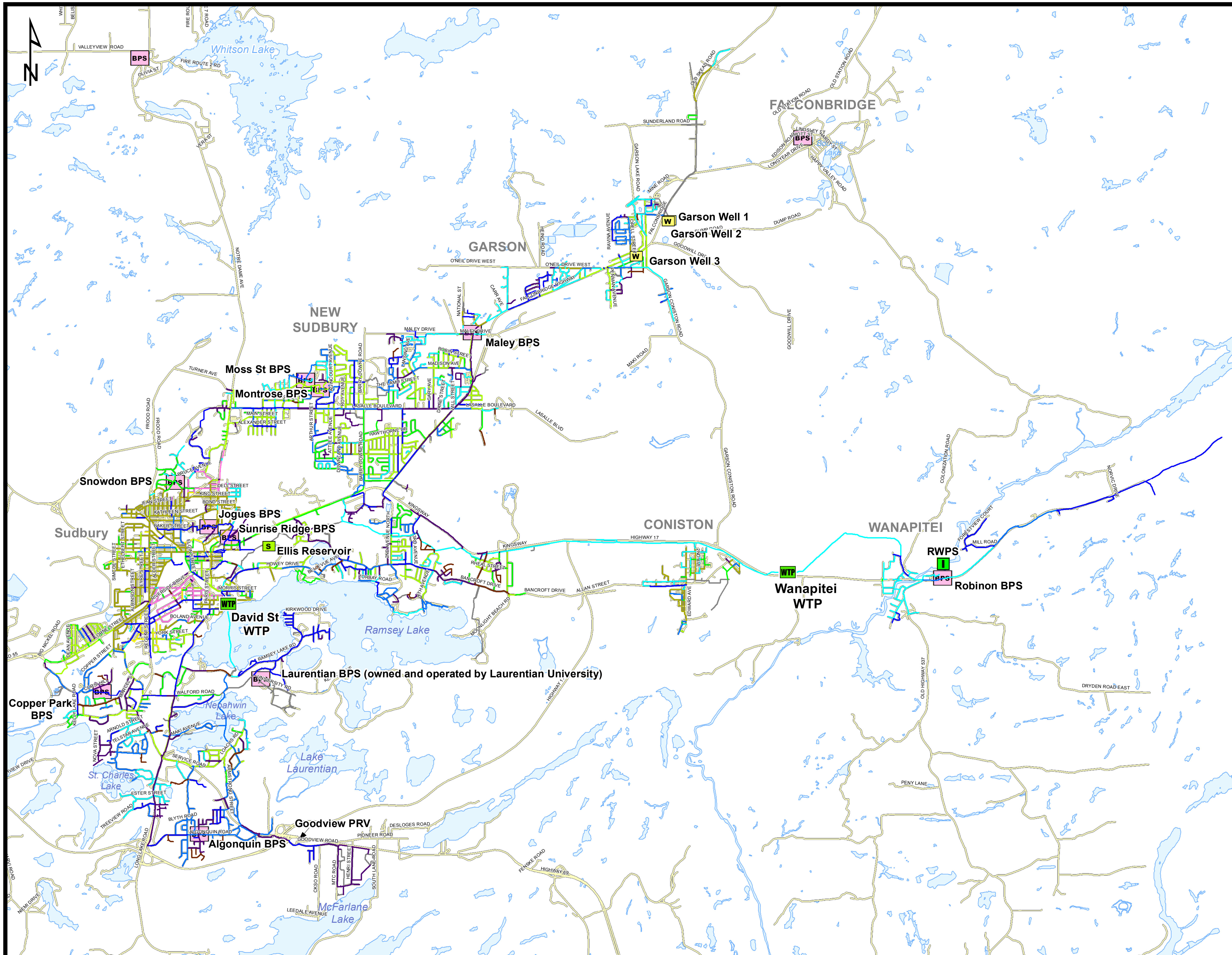
Map 3

Sudbury Water System - Pipe Material



Legend

- Well
- Storage Tank
- Raw Water Pumping Station (RWPS)
- Booster Pumping Station (BPS)
- Water Treatment Plant (WTP)
- Road
- Waterbody

Installed Year

- N/A
- <= 1930
- <= 1940
- <= 1950
- <= 1960
- <= 1970
- <= 1980
- <= 1990
- <= 2000
- <= 2010
- > 2011

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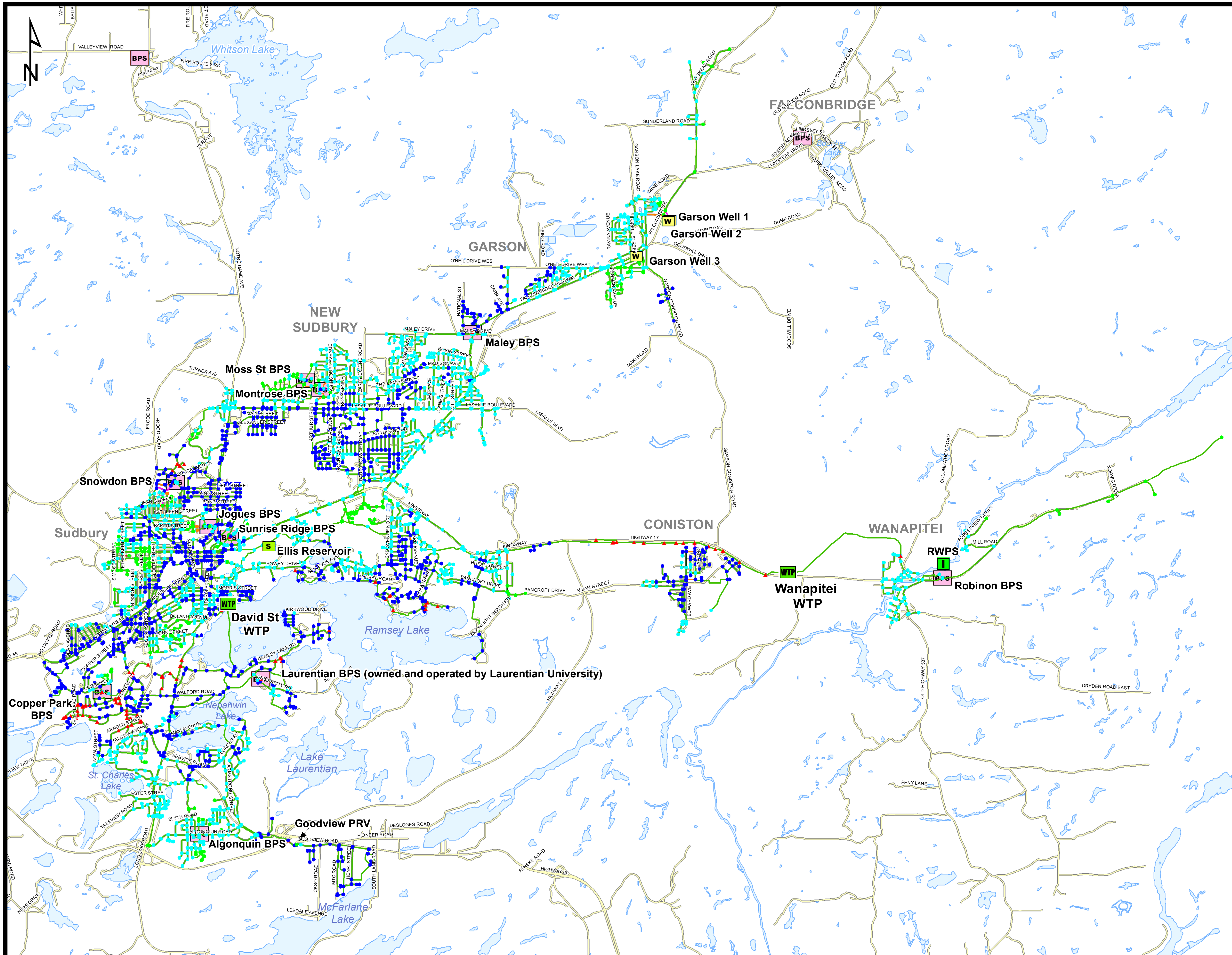
Map 4

Sudbury Water System - Pipe Age



Legend

- W Well
- S Storage Tank
- Raw Water Pumping Station (RWPS)
- BPS Booster Pumping Station (BPS)
- WTP Water Treatment Plant (WTP)
- Road
- Waterbody

Junction: Pressure (psi)

- <= 20
- <= 40
- <= 60
- <= 80
- <= 100
- > 100

Pipe: Headloss Gradient (m/km)

- <= 1.5
- <= 2.0
- <= 3.0
- <= 5.0
- > 5.0

MDD/ADD = 1.39

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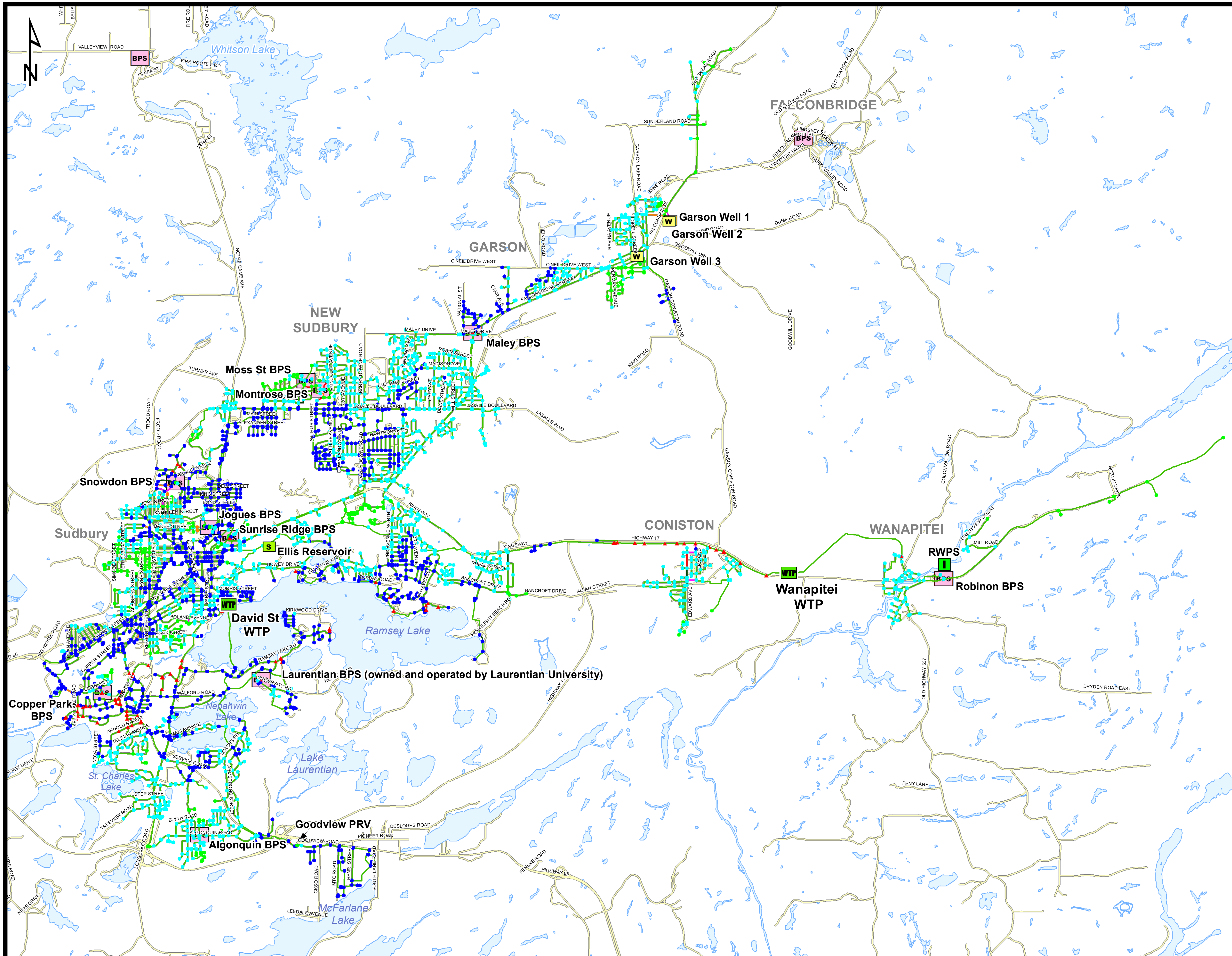
Map 5

Sudbury Water System - Pipe Headloss and Junction Pressure

Scenario: 2011ADD



Legend

- W Well
- S Storage Tank
- Raw Water Pumping Station (RWPS)
- BPS Booster Pumping Station (BPS)
- WTP Water Treatment Plant (WTP)
- Road
- Waterbody

Junction: Pressure (psi)

- <= 20
- <= 40
- <= 60
- <= 80
- <= 100
- > 100

Pipe: Headloss Gradient (m/km)

- <= 1.5
- <= 2.0
- <= 3.0
- <= 5.0
- > 5.0

MDD/ADD = 1.39

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Sudbury Water and Wastewater Master Plan

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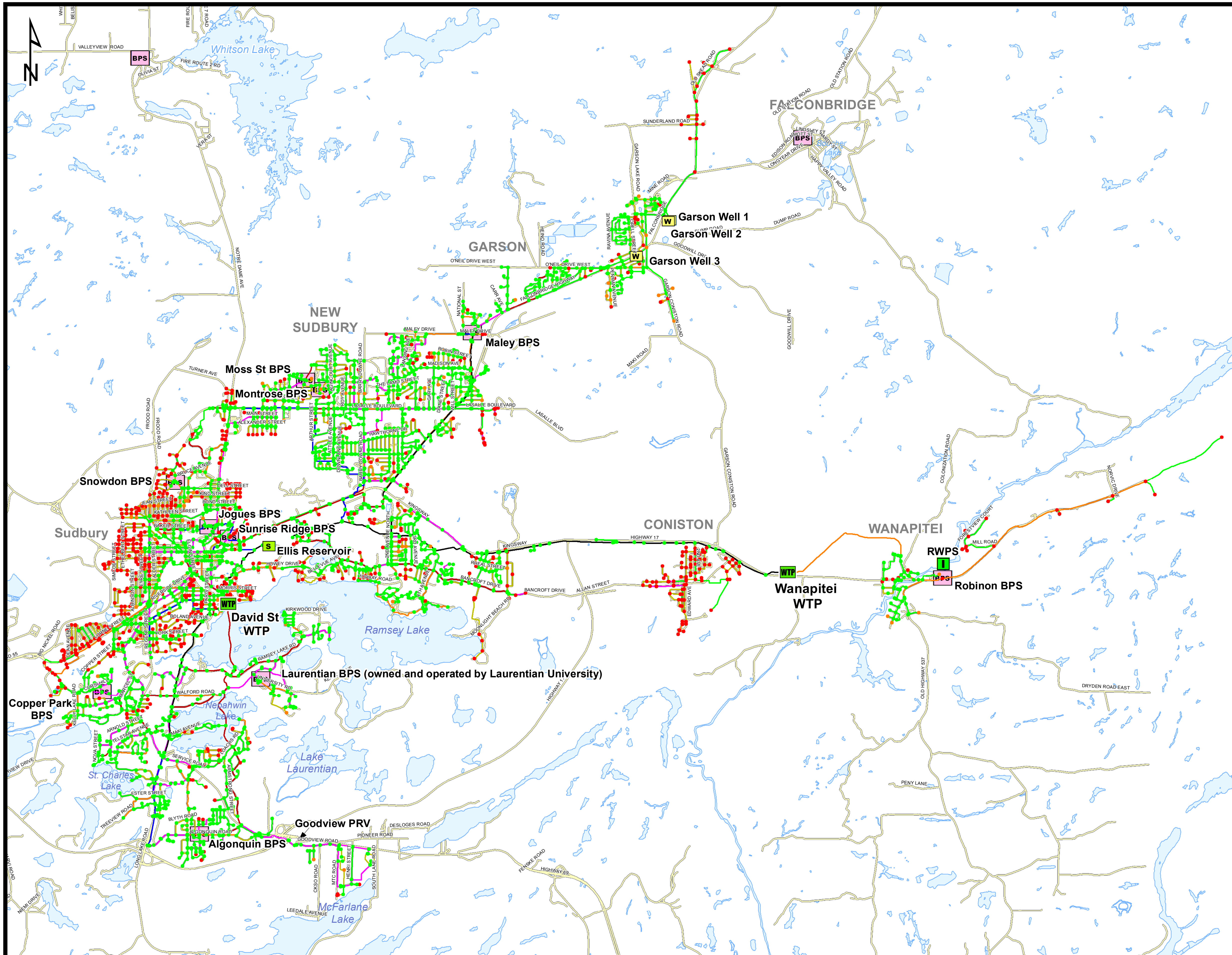
Map 6

Sudbury Water System -
Pipe Headloss and Junction Pressure

Scenario: 2011MDD



Legend

- W — Well
- S — Storage Tank
- Raw Water Pumping Station (RWPS)
- BPS — Booster Pumping Station (BPS)
- WTP — Water Treatment Plant (WTP)
- Road
- Waterbody

Junction: Fire Flow (Available) (L/s)

- <= 65 (Res) or 130 (ICI)
- < 75 (Res) or 150 (ICI)
- >= 75 (Res) or 150 (ICI)

Pipe: Diamater (mm)

- <= 150
- <= 200
- <= 250
- <= 300
- <= 400
- <= 600
- > 600

MDD/ADD = 1.39

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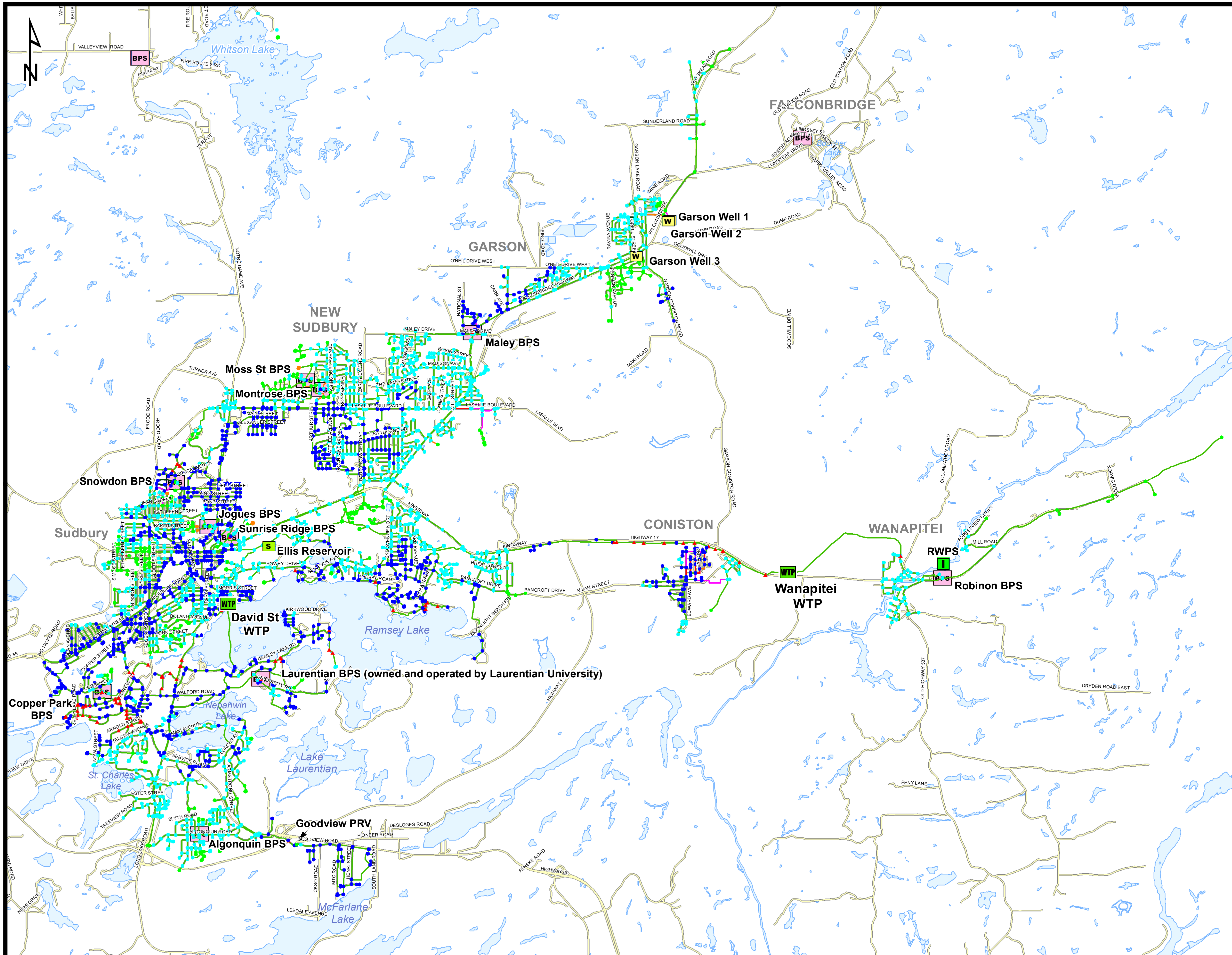
Map 7

Sudbury Water System - Fire Flow Availability

Scenario: 2011MDD+FF



Legend

- Well
- Storage Tank
- Raw Water Pumping Station (RWPS)
- Booster Pumping Station (BPS)
- Water Treatment Plant (WTP)
- Road
- Waterbody

Junction: Pressure (psi)

- <= 20
- <= 40
- <= 60
- <= 80
- <= 100
- > 100

Pipe: Headloss Gradient (m/km)

- <= 1.5
- <= 2.0
- <= 3.0
- <= 5.0
- > 5.0

MDD/ADD = 1.39

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Sudbury Water and Wastewater Master Plan

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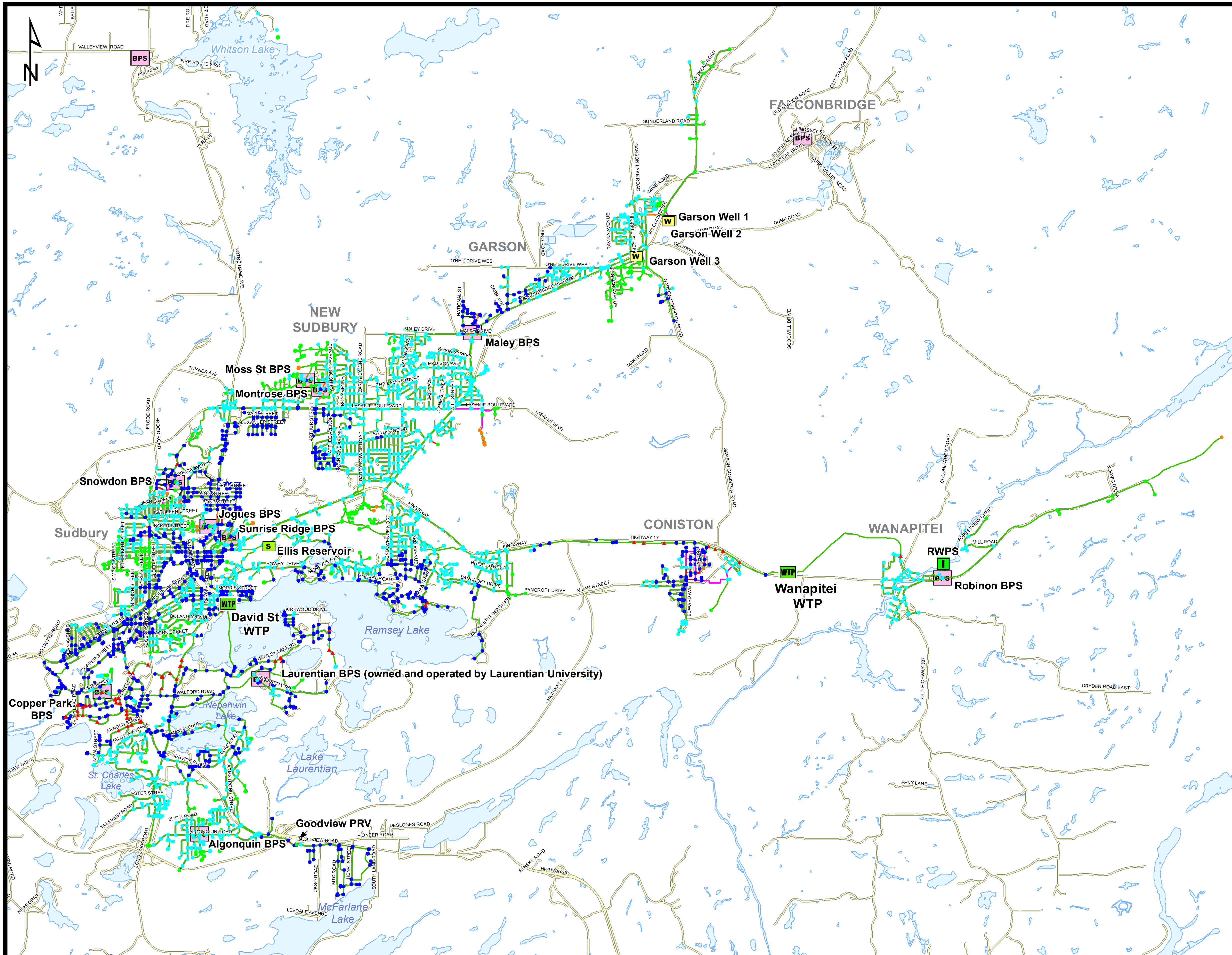
Map 8

Sudbury Water System - Pipe Headloss and Junction Pressure

Scenario: 2041ADD



Legend

- Well
- Storage Tank
- Raw Water Pumping Station (RWPS)
- Booster Pumping Station (BPS)
- Water Treatment Plant (WTP)
- Road
- Waterbody

Junction: Pressure (psi)

- <= 20
- <= 40
- <= 60
- <= 80
- <= 100
- > 100

Pipe: Headloss Gradient (m/km)

- <= 1.5
- <= 2.0
- <= 3.0
- <= 5.0
- > 5.0

MDD/ADD = 1.39

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Sudbury Water and Wastewater Master Plan

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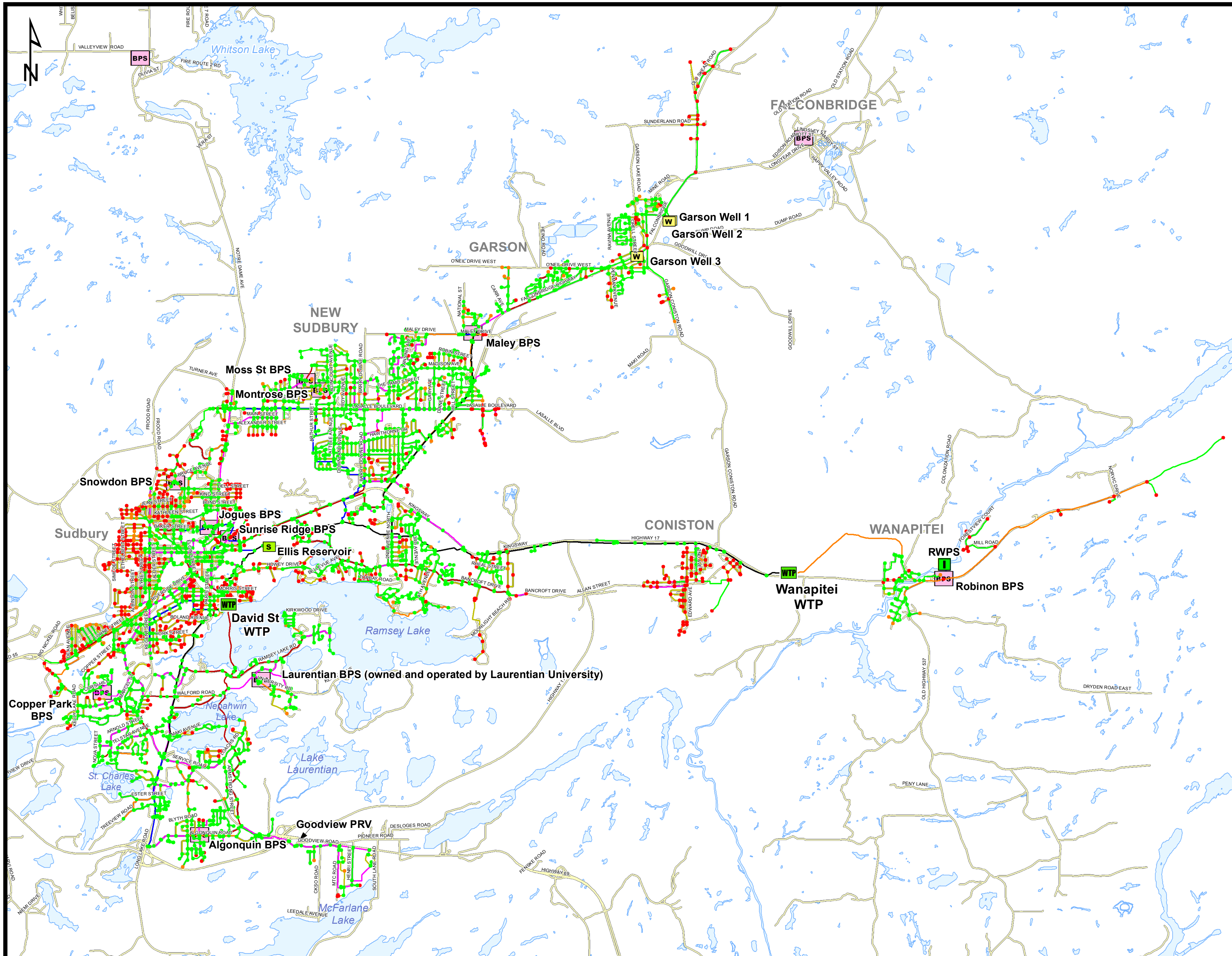
Map 9

Sudbury Water System - Pipe Headloss and Junction Pressure

Scenario: 2041MDD



Legend

W	Well
S	Storage Tank
I	Raw Water Pumping Station (RWPS)
BPS	Booster Pumping Station (BPS)
WTP	Water Treatment Plant (WTP)
	Road
	Waterbody

TOTFFAVAIL

- <= 65 (Res) or 130 (ICI)
- < 75 (Res) or 150 (ICI)
- >= 75 (Res) or 150 (ICI)

Pipe: Diamater (mm)

- <= 150
- <= 200
- <= 250
- <= 300
- <= 400
- <= 600
- > 600

MDD/ADD = 1.39

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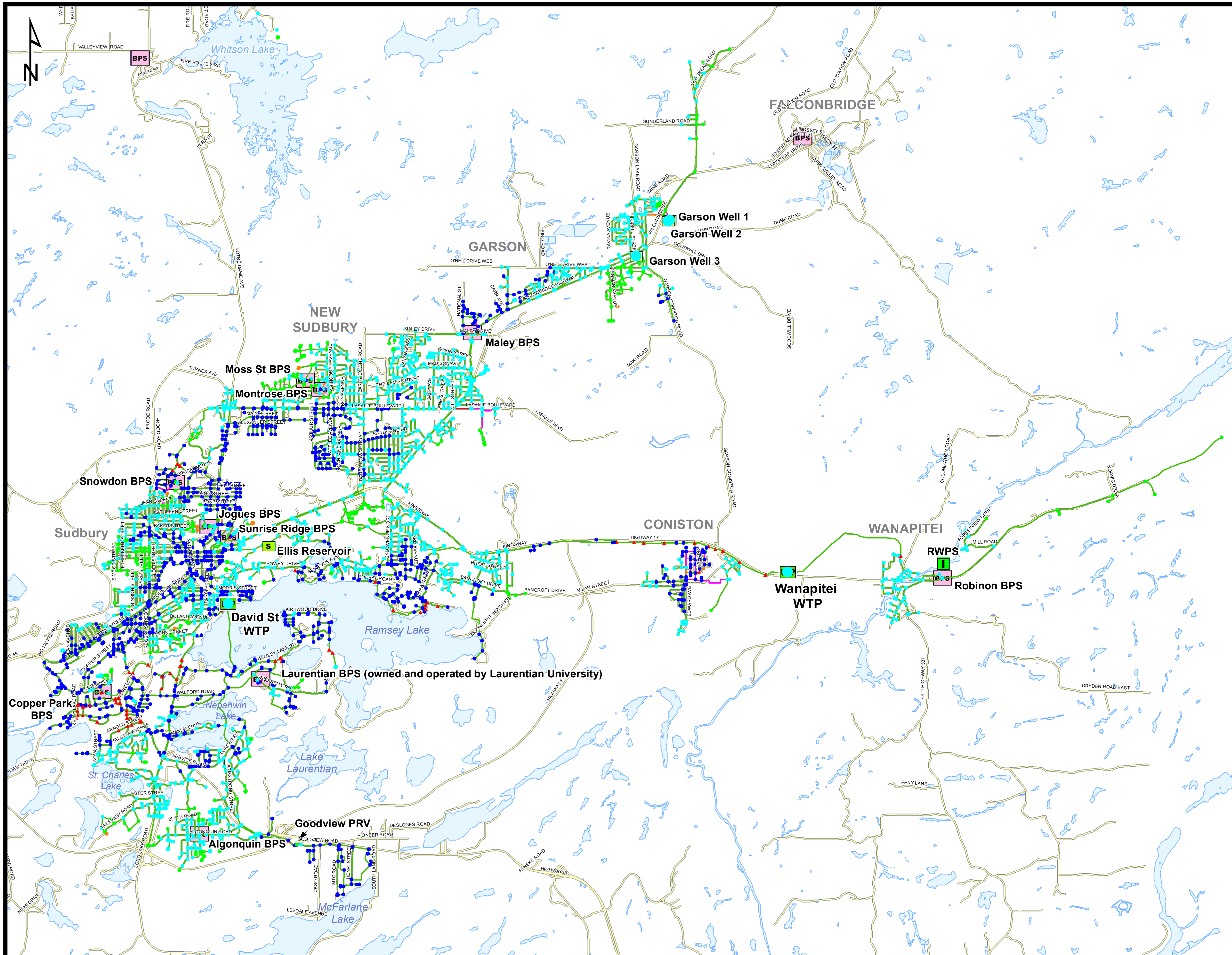
Map 10

Sudbury Water System - Fire Flow Availability

Scenario: 2041MDD+FF



Legend

- W — Well
- S — Storage Tank
- Raw Water Pumping Station (RWPS)
- BPS — Booster Pumping Station (BPS)
- WTP — Water Treatment Plant (WTP)
- Road
- Waterbody

Junction: Pressure (psi)

- <= 20
- <= 40
- <= 60
- <= 80
- <= 100
- > 100

Pipe: Headloss Gradient (m/km)

- <= 1.5
- <= 2.0
- <= 3.0
- <= 5.0
- > 5.0

MDD/ADD = 1.39

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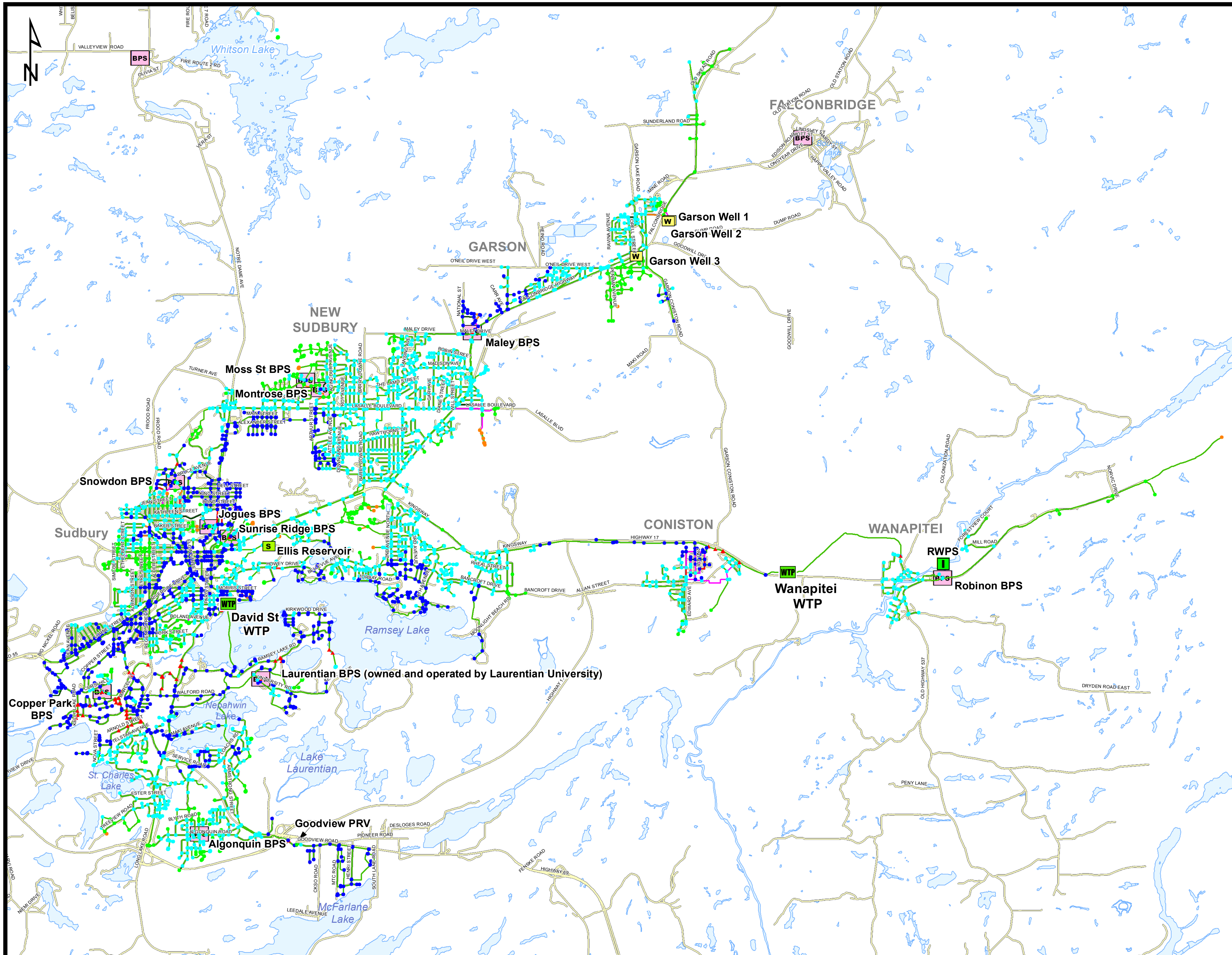
Map 11

Sudbury Water System - Pipe Headloss and Junction Pressure

Scenario: ULTADD



Legend

- W Well
- S Storage Tank
- Raw Water Pumping Station (RWPS)
- BPS Booster Pumping Station (BPS)
- WTP Water Treatment Plant (WTP)
- Road
- Waterbody

Junction: Pressure (psi)

- <= 20
- <= 40
- <= 60
- <= 80
- <= 100
- > 100

Pipe: Headloss Gradient (m/km)

- <= 1.5
- <= 2.0
- <= 3.0
- <= 5.0
- > 5.0

MDD/ADD = 1.39

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Sudbury Water and Wastewater Master Plan

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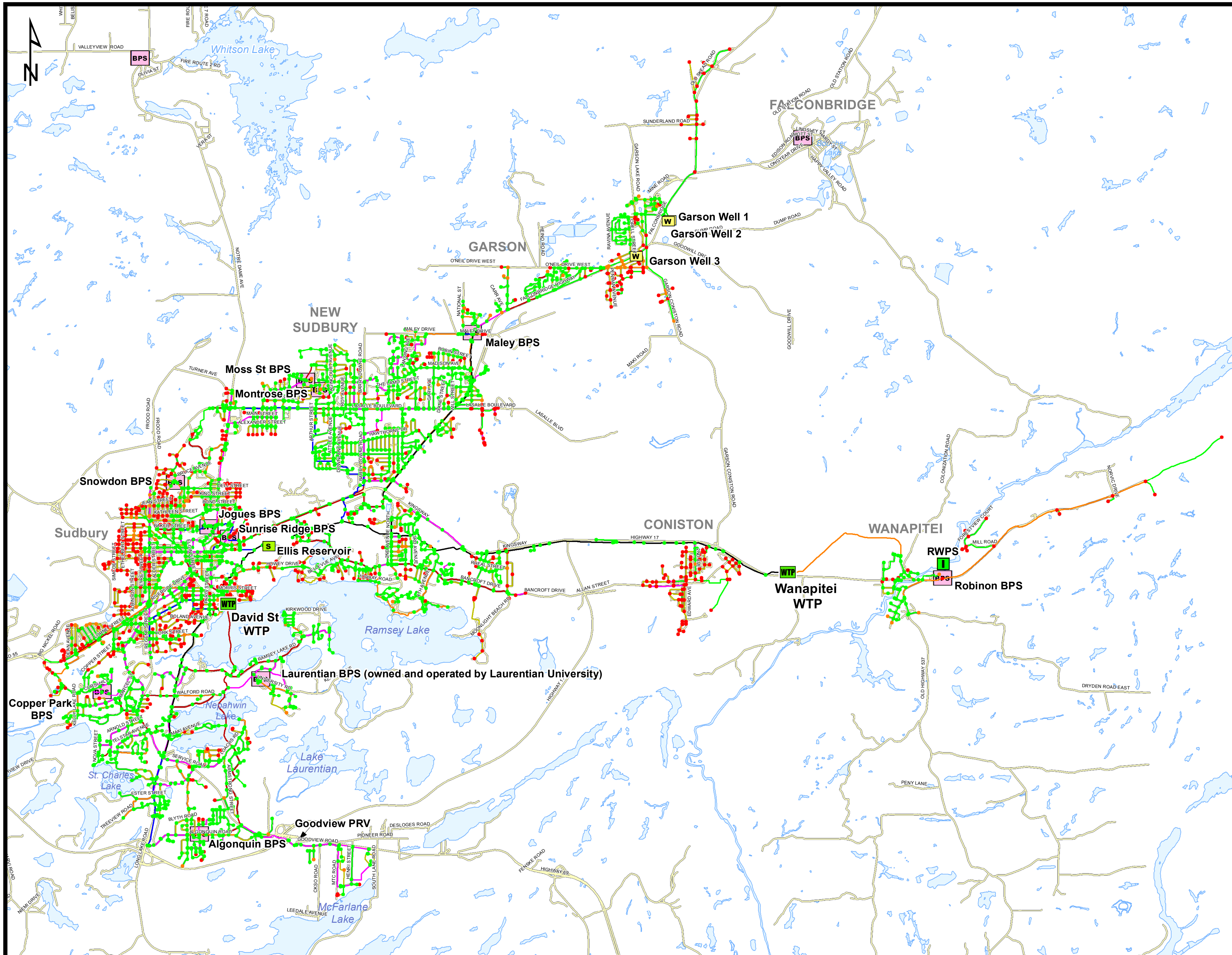
Map 12

Sudbury Water System - Pipe Headloss and Junction Pressure

Scenario: ULTMDD



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Sudbury Water and Wastewater Master Plan

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Map 13

Sudbury Water System - Fire Flow Availability

Scenario: ULTMDD+FF

CITY OF GREATER SUDBURY WATER AND WASTEWATER MASTER PLAN

VALLEY WATER SYSTEM GAP ANALYSIS AND STATUS QUO REPORT

CITY OF GREATER SUDBURY

DRAFT

PROJECT NO.: 121-23026-00
DATE: DECEMBER 2016

WSP
100 COMMERCE VALLEY DRIVE WEST
THORNHILL, ON, CANADA L3T 0A1

TEL.: +1 905 882-1100
FAX: +1 905 882-0055
WSP.COM

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1 INTRODUCTION

The City of Greater Sudbury (CGS) retained WSP to undertake a Water and Wastewater Master Plan. The purpose of the Master Plan project is to establish servicing strategies for water and wastewater infrastructure for the core urban areas and surrounding communities in the City for the next 20 years, as part of the five-year review of the City's Official Plan. The Master Plan will identify potential projects to address the servicing needs for planned growth within the City. It is being conducted in accordance with the requirements set out in the Municipal Class Environmental Assessment (Class EA) document (June 2000 as amended in 2007 and in 2011).

This report includes a capacity review of Valley's existing water system. Based on population growth projections and design criteria discussed in the *Population and Unit Rates Technical Memorandum* (WSP, 2014(a)), water demand projections were developed and used to determine future infrastructure needs to the 2041 and Ultimate Buildout planning horizons.

This report assumes that the Valley Water System would continue to be a stand-alone system. Any potential interconnections between Valley and other systems are not considered as part of this report. Potential interconnections with other communities will be reviewed under separate cover, as part of the Master Plan.

The conclusions provided in this report will be the basis for the problem definition and evaluation of alternatives conducted as part of the Master Plan.

Additional information on the existing water and wastewater systems is provided in the Baseline Review Reports for Water (WSP, 2014).

2 STUDY AREA

Valley is located in the north end of the City of Greater Sudbury and is the second most populated area, following the community of Sudbury. The Valley system is supplied by a well-based drinking water system.

Maps 1 to 4 in **Appendix A** show the communities of Azilda, Capreol, Chelmsford, and Valley East and identify future land use and development areas, including vacant residential and industrial, commercial, and institutional (ICI) areas.

The community of Valley East includes an area designated in the Official Plan as the "Urban Expansion Reserve". These lands are considered to be included in future urban growth and are necessary to complete the desired community structure of the Valley East Urban Area, but are not required during the Plan period. Accordingly, these lands have been placed in the Urban Expansion Reserve to restrict uses to those that would not prejudice the sound urban development of this area in the future. This Urban Expansion Reserve area generally comprises the area between communities of Val Caron, Val Therese and Hanmer and is approximately 1,197 ha in size. At a density of 12 units per hectare, the Urban Expansion Reserve has a residential unit potential of 14,364, or a population of 31,313.

Additional information on population growth and development phasing is provided in the Unit Rates and Population Projections Technical Memorandum (WSP, 2014).

Existing development in the study area is mixed, and includes residential as well as ICI land uses.

Based on the City's planning data, the Valley area population (including Azilda, Capreol, Chelmsford, Hanmer, Val Caron, Val Therese and rural areas of Valley that have only water servicing) is expected to increase from 36,382 in 2011 to 57,641 by Ultimate Buildout, excluding the Urban Expansion Reserve. When the Urban Expansion Reserve is included with the Ultimate Buildout scenario, the total population grows to 88,954. However, as indicated in the Official Plan, development of the Urban Expansion Reserve may only take place after all other Official Plan designated development has taken place. Therefore, the population growth attributed to the Urban Expansion Reserve has not been included in the Ultimate Buildout scenario.

ICI growth is expected to be primarily industrial with some commercial and a small amount of institutional. Growth is discussed further in Section 6.1.

3 OVERVIEW OF EXISTING WATER SYSTEM

The Valley Water System services the communities of Azilda, Capreol, Chelmsford, Hanmer, Val Therese, Val Caron, and portions of the rural community that have water servicing only. The system is supplied by 13 wells, two of which are located in Capreol, and the remaining 11 in Hanmer and Val Therese.

The total rated capacity for the system is 34,285 m^3/d. However, it is not possible to operate the system at its rated capacity due to well capacity constraints. A more realistic assessment of capacity, taking into account well pumping and drawdown limitations, identifies the available production capacity as 28,453 m^3/d, or a firm production capacity of 24,579 m^3/d with the largest well out of service, as detailed in Section 4.2.

All of the wells are owned and operated by the City of Greater Sudbury.

Additional information on the existing systems is provided in the Baseline Review Report for Water Systems (WSP, 2014).

3.1 VALLEY EAST WELLS

The Valley East Wells aquifer is characterized as a non-GUDI, shallow sand and gravel aquifer. There are 11 wells in Valley East. All the wells are located throughout the Hanmer and Val Therese communities.

Each well located in Valley is equipped with a vertical turbine well pump, a UV system for primary disinfection, a chlorine gas system for secondary disinfection, and fluoride injection equipment. Some of the wells also have standby diesel generators, as summarized in Table 3-1.

A typical process flow diagram is provided below, and a summary of the process equipment at each facility is provided in Table 3-1.

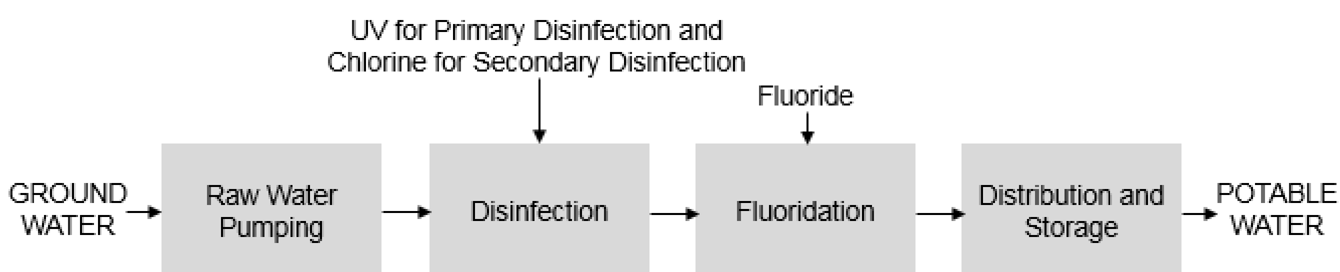

Figure 3-1 Valley Process Flow Diagram (typical for all Valley East wells)

Table 3-1 Valley Wells Process Information[1]

WELL	PUMP TYPE	RATED CAPACITY OPERATING POINT
Deschene	Vertical turbine well pump	20.8 L/s at 55.5 m TDH
Kenneth	Vertical turbine well pump	26.5 L/s at 56.4 m TDH
Philippe	Vertical turbine well pump	26.5 L/s at 59.4 m TDH
Frost	Vertical turbine well pump	26.5 L/s at 55.5 m TDH
Notre Dame	Vertical turbine well pump	35.9 L/s at 60.7 m TDH

Linden	Vertical turbine well pump	37.8 L/s at 61.6 m TDH
Pharand	Vertical turbine well pump	26.5 L/s at 57.3 m TDH
Michelle	Vertical turbine well pump	26.5 L/s at 55.8 m TDH
Well I	Vertical turbine well pump	29.9 L/s at 76.2 m TDH
Well R	Vertical turbine well pump with variable speed control	36.0 L/s at 72.8 m TDH
Chenier	Vertical turbine well pump with variable speed control	26.5 L/s at 71.1 m TDH

[1] *Data obtained from the Valley Drinking Water Works Permit, Number 016-205 Issue 2.*

3.2 CAPREOL WELLS

The Capreol portion of the system includes Wells J and M. The wells draw water from a common unconfined aquifer comprised mostly of sands and gravels, and classified as a GUDI water source

The two wells are located approximately 30 meters apart on the east side of Greens Lake and west of Municipal Road No.84. Each of the wells includes a vertical turbine well pump. The wells discharge into a common header. The water is treated with UV irradiation for primary disinfection, chlorine gas for secondary disinfection, and polyphosphate for iron and manganese sequestration. Fluoride is also added. Polyphosphate is also injected into the water at the treated water header leaving the pumphouse as a corrosion control method. The emergency diesel generator for both wells is located at Well "M". Table 3-2 summarizes key process information.

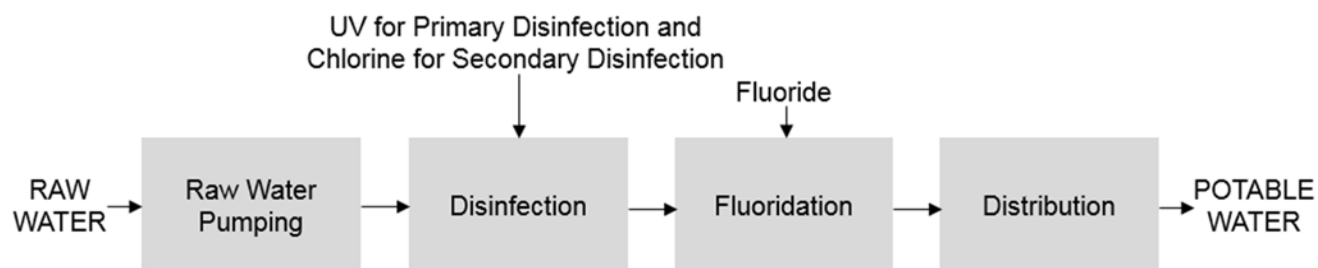

Figure 3-2 Capreol Wells Process Flow Diagram (typical for both Wells J and M)

Table 3-2 Capreol Wells Process Information[1]

WELL	PUMP TYPE	OPERATING POINT	STANDBY POWER
Well J	Vertical turbine well pump with VFD	37.9 L/s at 91.4 m TDH	400 kW diesel generator with Automatic Transfer Switch (generator supplies both well houses)
Well M	Vertical turbine well pump with VFD	45.4 L/s at 76.0 m TDH	

[1] *Data obtained from the Valley Drinking Water Works Permit, Number 016-205 Issue 2.*

3.3 DISTRIBUTION SYSTEM

The Valley distribution system consists of the following infrastructure:

— Three Storage Tanks
— Three Booster Pumping Stations
— A number of watermains

Storage facilities are described further in the table below.

Table 3-3 Valley Storage

TANK	STYLE	DIA. (M)	BASE EL. (M)	LOW WATER LEVEL (M)	HIGH WATER LEVEL (M)	USEABLE VOLUME (ML)[1]	DWWP USEABLE VOLUME (ML)
Azilda	Standpipe	12.2	285	288.3	327	4.5	4.5
Chelmsford	Elevated	12.8	285	317	327	1.3	1.4
Val Caron	Ground Level	26	317	317.1	327	5.2	5.6

[1] *These values were referenced in the Valley Water Supply Looping/Storage Project Class EA (R.V. Anderson, 2012).*

The Azilda Standpipe was constructed c. 1980 and has historically had problems with stagnant water and freezing in the winter. The City has plans to install a mixing system in the near future, which would improve circulation, prevent freezing, and improve water age in the tank.

The Chelmsford Elevated Tank was constructed c. 1970 and no operating concerns were identified by City operations staff.

The Val Caron Tank is a ground level tank that was constructed c.1973. The tank is located on the same site as the Val Caron Booster Pumping Station (BPS), which draws water from the tank. The Valley Looping and Storage Class EA (R.V. Anderson, 2012) indicated that the Val Caron Tank may drain completely in the event of an emergency. Refilling the tank may take days, impacting servicing to the McRea Heights neighbourhood. Therefore, the Class EA recommended installation of an automated, remotely controlled isolation valve at the tank and to measure water level rate of change at the tank.

The booster stations in the Valley Water System are described further in the table below.

Table 3-4 Booster Pumping Stations

FACILITY	BOOSTED AREA	PUMP INFORMATION	TOTAL CAPACITY (L/S)	FIRM CAPACITY (L/S)[1]
Capreol BPS (supplied by Valley wells)	Capreol	Three constant speed centrifugal pumps, each rated at 34.3 L/s at 57.3 m TDH	102.9	68.6
Centennial BPS	Lapointe Street & Centennial Drive area, south of Old Hwy 69	Two constant speed centrifugal pumps, one rated at 4.4 L/s at 31 m TDH and one rated 75 L/s at 18.3 m TDH	79.4	4.4
Val Caron BPS (located on same site as Val Caron Storage Tank)	McRea Heights	Two constant speed centrifugal pumps, one rated at 12 L/s at 32 m TDH and one rated at 28 L/s at 32 m TDH One 75 L/s fire pump	40	12

[1]*The Firm Capacity is calculated assuming the largest pump out of service.*

The Capreol BPS boosts pressure into Capreol, which is usually supplied by the Capreol wells and operated independently of the rest of the Valley system.

The Centennial BPS was constructed c. 1990 and boosts pressures in the area of Lapointe Street and Centennial Drive, south of Old Highway 69. A storage tank is planned for this site in the near future, as discussed in Section 6.4.2. The boosted area is at a higher ground elevation than the surrounding neighbourhood, and pressures are typically in the 38 – 42 psi (262 – 290 kPa) range. This range of pressures is generally near or at the minimum 40 psi (276 kPa) required by the MOECC, although on the low end of the required range.

The Valley distribution system includes a single watermain connecting the supply in Hamner and Val Therese to Chelmsford and Azilda. In the event of a watermain break on this line, repairs would be time-consuming since there are few isolation valves on the line, as indicated by City staff. This also poses a potential risk to providing a continuous water supply to Azilda and Chelmsford.

3.4 KNOWN CHALLENGES

In addition to concerns discussed in previous sections, the Valley Water System has the following known challenges:

- The production capacity of the Valley and Capreol wells is much less than the rated capacity. This is due to elevated turbidity at higher flow rates as well as iron and manganese clogging well screens. The wells are rehabilitated every three years, but City staff has noted decreasing well capacity over time, even after frequent rehabilitation. This concern is discussed further in Section 4.2.
- As indicated in the previous section, the Val Caron Tank may drain completely in the event of an emergency, delaying a restart in servicing to the McRea Heights neighbourhood (R.V. Anderson, 2012). The Valley Looping and Storage Class EA recommended installation of an automated, remotely controlled isolation valve at the tank as well as measuring the rate of change of the water level of the tank.
- As noted in the previous section, the watermain between the east end of Valley and Azilda/Chelmsford is a single feed without isolation valves. This makes the system vulnerable in case of a watermain break since the repair would be time-consuming.
- As noted in Section 3.3, City operations staff has noted that the Azilda Standpipe has a lower rate of turnover in winter months compared to summer. This leads to increased water age and increased potential for water freezing in the tank. The City has plans to install a mixing system in the tank in the near future to improve turnover.

In addition, the City runs a program instructing about 72 customers (exact number varies annually) in the Valley Water System to run a small amount of water through their taps in the winter months to prevent water services from freezing on the municipal side. The specific number of customers included in the program varies annually depending on the expected winter temperatures.

Some service connections in Greater Sudbury freeze due to the shallow depth of bury; older homes were constructed prior to the current standards for depth of bury and are more vulnerable to freezing.

Most of the vulnerable connections are located in Capreol (30), Hanmer (24), and Val Caron (12), with the remaining located in other areas of the system.

Customers who are requested to run their water are asked to run a small flow, equivalent to about the thickness of a pencil or approximately 0.06 L/s, between December 1 and April 1. In Valley, this results in a total of about 45,000 m^3 per season, or 373 m^3/d. In the winter, this accounts for approximately 2% of the firm production capacity of 19,267 m^3/d of the Valley wells.

4 HISTORICAL FLOWS AND REVIEW OF OPERATIONAL DATA

Water supply data from the Valley and Capreol Wells from 2009 to 2013 was reviewed and analyzed for this evaluation. Table 4-1 below shows a summary of the data received, and indicates how it was used for the analysis.

Table 4-1 Water Supply Data Reviewed

DATA RECEIVED	PARAMETERS INCLUDED	DATA INTERVAL	USE IN ANALYSIS
Treated flow (2011-2013)	Flow in m^3/d	Hourly	To determine peak hourly flow
Annual Reports (2009-2013)	Total average daily flows, maximum daily flows Treated water characteristics	Daily	To determine average day, max day flow To assess performance of existing process and treated water characteristics
Annual Billing Data (2012)	Annual flow per customer in m^3	Annually	To determine the proportion of total water consumption corresponding to residential users

4.1 FLOW DATA

Water supply data from 2010 to 2013 was reviewed to determine historical water demands in the Valley Water System. Average day and maximum day demand data for the past four years, and peak hour data for the past three years (2011-2013) is included in Table 4-2 below. For reference, the combined rated capacity of the Valley Water Supply System is 34,226 m^3/d (MOECC, 2011), assuming all wells perform to rated capacity. However, as detailed in Section 4.2, the firm production capacity is 24,579 m^3/d.

Table 4-2 Historical Water Supply Data

YEAR	AVERAGE DAY DEMAND (M3/D)[1]	MAXIMUM DAY DEMAND (M3/D)[1]	PEAK HOUR DEMAND (M3/D)[2]
2010	9,999	17,601	Not Available
2011	10,114	13,201	19,007
2012	10,080	14,055	18,189
2013	10,135	14,182	22,105

[1] Valley Drinking Water System Annual Reports (2009 – 2013).

[2] From hourly SCADA data.

Average day water consumption was consistent between 2010 and 2013. The average consumption for the five year period was 10,082 m^3/d.

The highest maximum day flow recorded in the past four years was 17,601 m^3/d, occurring in 2010. The average historical maximum day demand is 14,760 m^3/d.

Hourly flow data was only available from 2011 to 2013. The maximum peak hour value recorded during that period was 22,105 m^3/d in 2013, and the average was 19,767 m^3/d.

Note that although the rated supply system capacity far exceeds the historical average day and maximum day demands, the wells do not operate at their rated capacity. The actual production capacity is discussed in further detail in the next section, Section 4.2.

The peaking factors derived from historical data were compared to those documented in the *City's Engineering Design Manual* (City of Greater Sudbury, 2012) and those included in the *MOECC Guidelines* (MOECC, 2008).

The maximum day to average day peaking factor corresponding to the maximum day flow recorded (17,601 m^3/d in 2010) was 1.76, while the average maximum day peaking factor was 1.46. The City's Engineering Design Manual specifies a maximum day factor of 1.8 for Valley, which matches the value recommended in the *MOECC Guidelines* for communities with populations between 25,001 and 50,000, such as Valley. Historically, the maximum day demand in 2010 was much higher than later years. The highest maximum day factor (1.76) was adopted to evaluate future requirements.

The peak hour to average day factor corresponding to the highest peak hour flow recorded in 2013 (22,105 m^3/d) was 2.18, while the average peak hour factor was 1.95.

The *City's Engineering Design Manual* and the *MOECC Guidelines* specify a peak hour factor of 2.7. For purposes of estimating future demands, the historical maximum value (2.18) was adopted.

4.2 PRODUCTION CAPACITY OF VALLEY WELLS

The Valley wells are rated for a total capacity of 34,796 m^3/d, assuming all pumps and wells are in service. However, historically, these wells have been unable to produce water in quantities that meet the rated capacity, or, when stressed to their rated capacity, turbidity levels increase. In other cases, the wells are unable to produce their rated capacity due to frequent plugging or encrustation of the well casing with iron or manganese.

To better assess the remaining capacity of the Valley Water System, it is recommended that the actual production capacity of the wells be used in lieu of the rated capacity.

The actual production capacity of each well has been estimated through discussions with City staff and review of historical pumping data.

Table 4-3 Comparison of Rated and Estimated Actual Production Capacities

WELL	TOTAL RATED CAPACITY (M^3/D)[1]	ESTIMATED ACTUAL CAPACITY (M^3/D)[2]
Chenier	2,333	2,278
Deschene	1,798	1,631
Kenneth	2,288	1,521
Frost	2,288	2,290
Well I	1,974	0 (offline)
Well J (Capreol)	3,273	2,740
Linden	3,269	2,506
Well M (Capreol)	3,927	3,875
Michelle	2,290	2,290
Notre Dame	3,105	2,103

Pharand	2,290	2,007
Philippe	2,288	2,198
Well R	3,162	3,014
Total	**34,285**	**28,453**
Firm Capacity (largest one well out of service)	**30,358**	**24,579**

[1] *Data obtained from the Valley Municipal Drinking Water Licence, Number 016-105 Issue 4.*

[2] *Estimated based on discussions with City staff. Based on 2015 Max Day Capacities.*

As indicated in Table 4-3, the total production capacity of the Valley Wells is less than the total rated capacity. Moreover, the total production capacity assumes all wells are in service, except for Well I, which is out of service due to high turbidity, low production capacity, and frequent instrument issues due to iron and manganese. In order to prepare a conservative analysis (which is in line with MOECC standards and best management practices) of the remaining capacity of the Valley Water System, the total production capacity, less the largest one well, will be used in this report. This amounts to a firm production capacity of 24,579 m^3/d (excludes Well R).

Figure 4-1, below, compares this firm capacity to the rated capacity and historical demands.

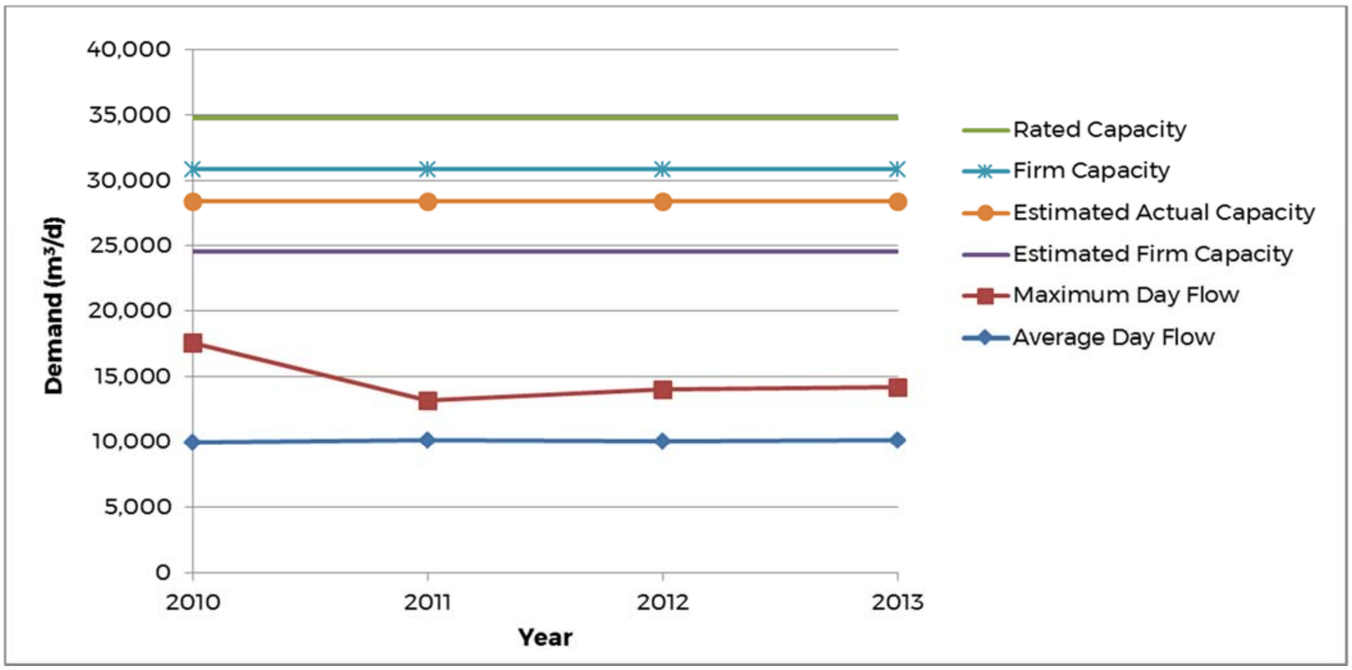

Figure 4-1 Comparison of Firm Production Capacity to Historical Demands

Comparing the firm production capacity against historical maximum day demands shows that the system is capable of supplying current demands.

4.3 RAW WATER CHARACTERISTICS AND SECURITY OF SUPPLY

4.3.1 VALLEY WELLS

The Valley Wells aquifer is classified as non-GUDI. Several wells, however, have higher than average levels of sodium or iron/manganese and other operational concerns. These include:

- Higher than average sodium levels at Pharand Well
- Turbidity level at Michelle Well has increased to 0.5 NTU
- Elevated levels of iron at Kenneth and Linden Wells

In addition, as discussed in Section 4.2, all of the wells operate below their rated capacities, and Well I is shut off due to high turbidity levels. If the raw water aesthetics continue to decline, the production capacity of the Valley Wells would also decrease. This represents a potential risk to the Valley Water System supply.

4.3.2 CAPREOL WELLS

The Capreol Wells draw water from an unconfined aquifer with effective in-situ filtration, comprised mostly of sands and gravels, and classified as a potentially GUDI water source (City of Greater Sudbury, 2011). As with the Valley Wells, the Capreol Wells operate below their respective rated capacities. This is due to elevated levels of iron and manganese that clog the well screen.

4.4 OPERATIONAL DATA

Data reported in the *Annual Reports* for the Valley supply facilities (including Capreol) includes effluent chlorine residual, trihalomethanes (THMs), fluoride, and trace organic and inorganic chemicals.

Data was reviewed from 2009 to 2013 to determine any historical issues at the wells. No exceedances were observed, except for elevated sodium levels at Philippe, Pharand, Michelle, and R Wells.

5 DESIGN CRITERIA

The following design criteria were used to assess the remaining capacity of the existing systems and to forecast future requirements for the water and wastewater systems. The unit rates used to estimate future water and wastewater flows correspond to the values included in the *Population Projections and Unit Rates Technical Memorandum* (WSP, 2014). Otherwise, design criteria recommended in the *MOECC Guidelines* and *City's Engineering Design Manual* were used.

5.1 UNIT WATER DEMAND CRITERIA

The water demand criteria shown in Table 5-1 are from the unit rates recommended in the *Populations and Unit Rates Technical Memorandum* (WSP, 2014). The rates were reviewed against historical data, MOECC *Guidelines* (MOECC, 2008), and current standards in the City's *Engineering Design Manual* (City of Greater Sudbury, 2012).

Both the *MOECC Guidelines* and *City Engineering Design Manual* recommend determining demands for institutional, commercial and industrial (ICI) users on a case by case basis. However, the following criteria for ICI demands were used for the purposes of this evaluation.

Table 5-1 Valley Water System Design Criteria

CRITERIA	VALUE	REFERENCE
Average Day Residential Demand	250 L/cap/day	Average of historical values, rounded up to nearest 50 L/cap/day
Average Day Institutional & Commercial Demand	28 m^3/ha/d	MOECC Guidelines
Average Industrial Demand	35 m^3/ha/d	MOECC Guidelines
Domestic Demand Maximum Day Factor	1.46	Average of historical values
Domestic Demand Peak Hour Factor	2.18	Maximum of historical values

Residential average day demands are obtained by multiplying the residential unit rate by the service population. Similarly, average ICI demands are obtained by multiplying the corresponding unit rates to the areas of development, assuming 100% of the area would be developed.

Maximum day and peak hour demands are obtained by multiplying the average day demand by the corresponding peaking factor.

For purposes of this study, and in line with city standards and practices, a residential fire flow of 75 L/s over 1.75 hours and ICI fire flow of 150L/s over 2 hours were used.

5.2 DESIGN CRITERIA FOR WATER SYSTEM COMPONENTS AND OPERATION

5.2.1 TREATMENT CAPACITY

Water supply facilities are designed to supply the maximum day demands of the system.

Treatment facilities must be designed in accordance with the *Procedure for Disinfection of Drinking Water in Ontario* (Ontario, 2006). Drinking water treatment systems that obtain water from a surface water or GUDI well supply must achieve an overall performance providing as a minimum a 2-log (99%) removal or inactivation of *Cryptosporidium* oocysts, 3-log (99.9%) removal or inactivation of *Giardia* cysts, and 4-log (99.99%) removal or inactivation of viruses.

At least 0.5-log removal or inactivation of *Giardia* cysts and 2-log removal or inactivation of viruses must be provided through disinfection, while the remaining removal may be achieved through filtration or other equivalent treatment processes.

5.2.2 PUMPING CAPACITY

Pumping stations are rated based on their firm capacity. If sufficient floating storage is available in a particular pressure district, the MOECC defines firm capacity as the capacity of the station with the largest pump out of service. If there is insufficient or no floating storage, firm capacity is defined as the capacity with the two (2) largest pumps out of service (MOECC, 2008).

For each pressure district, the pumping stations have to be designed to provide peak hour or maximum day plus fire demands (whichever are greater), if no floating storage is available. If sufficient floating storage is available, then the pumping station only needs to be designed to provide maximum day demands.

The Valley system consists of two main pressure districts: Capreol and Valley (servicing Hanmer, Val Therese, Val Caron, Azilda, and Chelmsford).

The Capreol pressure district is pressurized directly by the Capreol Wells and typically operates independently of the rest of the Valley Water System.

The Valley pressure district is supplied and pressurized directly by the Valley Wells as well as the Val Caron, Azilda, and Chelmsford storage tanks. The Valley pressure district includes two sub-districts, boosted respectively by the Centennial and Val Caron Booster Pumping Stations. These are small areas that are pressurized by the respective booster pumping stations. The rest of the Valley pressure district relies on overall system pressure, and is not boosted.

5.2.3 STORAGE CAPACITY

Storage requirements are based on the requirement to meet water demands that exceed the capacity of the treatment plant and to satisfy fire flow demands. When the capacity of the supply system is only capable of satisfying maximum day demands, storage requirements are determined using the following formula from the *MOE Guidelines* (MOECC, 2008):

$$\boldsymbol{Storage = A + B + C}$$

Where: A = Fire Storage, B = Equalization Storage = 25% of maximum day demand, and C = emergency storage = 25% of (A+B).

Fire storage is the product of the maximum fire flow required in the system and the corresponding fire duration based on Fire Underwriters requirements (Fire Underwriters Survey, 1999).

When the system can supply more than just the maximum day demand (but less than the peak demand), the fire storage requirements can be determined using the following formula:

$$\boldsymbol{A = (Peak\ Demand - Pumping\ Station\ Firm\ Capacity) \times Fire\ Duration}$$

Where: peak demand is the greater of the peak hour demand and the maximum day plus fire demand.

Per *MOECC Guidelines*, floating storage should be designed such that the elevation of the equalization volume (B) is such that a minimum pressure of 275 kPa (40 psi) can be maintained in the system under peak hour flow conditions. The fire (A) and emergency (C) volumes should be at elevations that produce 275 kPa (40 psi) during peak hour demand conditions, and 140 kPa (20 psi) under the maximum day plus fire flow condition (MOECC, 2008).

5.2.4 DISTRIBUTION CAPACITY

Watermains have to be sized to carry the greater of the maximum day plus fire flow or peak hour demand. The range of acceptable pressures under normal conditions (average to peak hour flows) is 275 kPa (40 psi) to 690 kPa (100 psi), while during fire flow conditions pressures may drop to 140 kPa (20 psi) (MOECC, 2008). The maximum allowable water velocity in the distribution system is 3 m/s (MOECC, 2008).

6 FUTURE REQUIREMENTS

6.1 POPULATION PROJECTIONS

As part of the City of Greater Sudbury Master Plan, population forecasts were developed for the 2016, 2021, 2026, 2031, 2036, 2041 and Ultimate Buildout planning years. Ultimate Buildout is defined as an estimate of what the demand from the total population and total number of households in the City of Greater Sudbury would be based on lands that are currently designated for development in the Official Plan within the existing settlement boundaries.

The City supplied planning data sheets with properties and development potential and the vacant residential and ICI land inventory, and Hemson Consultants, on behalf of the City, provided supplementary population projections. Data was provided for each water and wastewater system boundary. This data was used to develop the targeted population growth for each horizon year, as well as development phasing (discussed in the next section and in detail in the *Populations and Unit Rates Technical Memorandum*, WSP 2014).

In cases where the City's planning data sheets and Hemson's population projections forecasted fewer development units than the vacant land inventory for an area, then specific parcels (up to the City's and Hemson's unit projections) of developable units were selected. These parcels were selected based on the rationale provided in the City's Official Plan. That is, the Official Plan prioritizes that development take place in areas that are currently serviced, or where servicing can easily be extended. This focuses growth in existing urban areas until supply is no longer available in these areas.

Based on the planning data, the population of Valley (including Azilda, Capreol, Chelmsford, Hanmer, Val Caron, Val Therese, and rural areas with only water servicing) is projected to increase by 3,382 people in 2041 and 21,259 by Ultimate Buildout, excluding growth due to the development of the Urban Expansion Reserve. Development of the Urban Expansion Reserve was excluded from the infrastructure planning populations since this area may not be developed by the Ultimate Buildout Horizon. As indicated in the Official Plan, all other designated developable areas must be developed before the Urban Expansion Reserve. Including the Urban Expansion Reserve as part of the Ultimate Buildout Horizon would inflate infrastructure planning requirements beyond what would be needed to meet the current Official Plan development.

The population projections to be used in the Master Plan are summarized in Table 6-1 below.

Table 6-1 Valley Population Projections

YEAR	2011	2016	2021	2026	2031	2036	2041	ULTIMATE BUILDOUT
Azilda	4,449	4,624	4,807	4,959	5,050	5,099	5,103	8,361
Capreol	3,392	3,396	3,412	3,435	3,447	3,456	3,450	4,716
Chelmsford	7,400	7,517	7,639	7,763	7,838	7,886	7,891	11,008
Valley East	19,119	19,644	20,219	20,728	21,028	21,205	21,231	31,469
Valley Rural (Water Servicing Only)	2,022	2,054	2,065	2,079	2,087	2,091	2,088	2,088
Total	36,382	37,235	38,142	38,965	39,451	39,737	39,764	57,641

The City's planning data does not specify target years for employment growth. However, vacant lands designated as institutional, commercial, and industrial (ICI) properties have been assigned to different stages of the development process by the City. These stages are described below.

— Draft Approved:

    - These are lands that have draft plan of subdivision approval under the Planning Act or have pending applications with the City. Typically, these lands are close to registration or few years away from development as the required conditions are satisfied
    - Development approvals are near complete, and development could take place at any time. Properties with this designation were set to take place in 2016.
- Legal Lots of Record:
    - These are existing lots, including lots in a registered plan of subdivision. Typically these lands are zoned, serviceable and only require building permit approval for development. In some cases a site plan approval/agreement may also be required.
    - Based on historical trends, development is approximately 15 years away from receiving draft approval. Properties with these designations were assigned to take place in 2026.
- Designated Developable:
    - These lands do not have any development approvals in place but are understood to be areas of future development as they are within the settlement boundary. Designated lands are typically a number of years away from being developed.
    - Based on historical trends, these properties are approximately 10 years away from receiving Legal Lot of Record designation. Designated Developable properties were assumed to take place in 2036.

These land supply categories stem from the land supply requirements that municipalities must maintain under Section 1.4 of the Provincial Policy Statement. In this context, Designated Development Lands would count towards Section 1.4.1 (a) and Legal Lots of record and Draft Approved Lands would count towards 1.4.1 (b). It is also important to note that the total supply is governed by PPS Section 1.1.2.

The targeted ICI development areas for each horizon year are summarized in the table below.

Table 6-2 Valley ICI Projections

	ICI DEVELOPMENT AREAS (HA)						
LAND USE	2016	2021	2026	2031	2036	2041	BUILDOUT
Institutional	0.00	0.00	6.60	0.00	0.00	0.00	0.00
Commercial	0.00	0.00	13.62	0.00	8.71	0.00	0.00
Industrial	0.00	0.00	23.02	0.00	131.07	0.00	0.00
Total	**0.00**	**0.00**	**43.24**	**0.00**	**139.78**	**0.00**	**0.00**

The above assumptions provide an estimate as to the ICI development time line. In reality, development may be more staggered. However, for purposes of infrastructure planning and to ensure that the appropriate infrastructure is in place by the appropriate planning horizon, the above assumptions are considered to be conservative.

6.2 PRIORITY EXTENSION LIST

The City has developed and maintained a Priority Extension List of existing residential and ICI streets that are not currently serviced by either or both municipal water or sewer, but at least one owner on the street has requested servicing. The City's policy on extension of services includes the following conditions:

- Before any project proceeds, the participation rate of benefitting property owners must be 100%, with those benefitting property owners funding 50% of the actual net cost of the project.
- The process must be initiated by property owners submitting a petition to the City of Greater Sudbury.
- At least 80% of the property owners in the project area must sign the petition.

— The project must be on the City's priority list for new servicing schemes, or, there must be demonstrated cause why the project should be included on the City's priority list for new servicing schemes.

In Valley, two streets have been placed on the priority list for sewer servicing. However, to date, the above conditions have not been met and City funding for extension requests is not available. Therefore, these streets have not been included in the demand projections for infrastructure planning as part of the Master Plan.

6.3 PHASING OF FUTURE GROWTH

Growth areas were allocated based on population projections for individual developments and the overall target growth population projections for the Valley area for the horizon years.

Hemson's supplementary tables were used to provide the target population, while the City's planning tables and vacant lot inventory were used to identify phasing of specific properties, and assignment of draft approved, legal lots of record, and designated development properties. In general, priority was given to draft approved properties, followed by legal lots of record and designated developable properties. In accordance with the Official Plan, the City has also assigned a target quantity of legal lots of record and designated developable properties to be developed in each horizon year. That is, legal lots of record should account for approximately 20% of all household growth, and designated developable lots are assigned 20% of the 20 year anticipated growth.

The future growth phasing plans were presented in the *Unit Rates and Population Projections Technical* Memorandum (WSP, 2014).

6.4 FUTURE WATER DEMAND PROJECTIONS AND INFRASTRUCTURE NEEDS

The unit flow criteria listed in Section 5.1 were used to estimate the future water demands in the Valley Water System. In general, the projected flows were calculated by the following formula:

$$\textbf{Projected Average Day Demand} \\ = \textbf{Base Demand} + \textbf{Additional Residential Demand} + \textbf{Additional ICI Demand}$$

The demands corresponding to the population growth forecasts to ultimate buildout (excluding growth from development of the Urban Expansion Reserve) are listed in Table 6-3 below.

Table 6-3 Water Demand Projections for the Valley Water System

YEAR	POPULATION	AVERAGE DAY DEMAND (M3/D)	MAXIMUM DAY DEMAND (M3/D)	PEAK HOUR DEMAND (M3/D)
Base	36,382	10,082	14,760	19,767
2016	37,235	10,295	15,031	22,456
2021	38,142	10,522	15,362	22,950
2026	38,965	12,100	17,665	26,391
2031	39,451	12,221	17,843	26,656
2036	39,737	17,124	25,001	37,350
2041	39,764	17,131	25,011	37,365
Ultimate Buildout	57,641	21,600	31,536	47,113

The Base Demand was the average historical (2009 to 2013) average day demand for the community. The additional residential demand was calculated using the unit flow rate multiplied by the population growth, and similarly, the ICI demand was calculated using the unit flow rate for each type of development (industrial, commercial or institutional), multiplied by the growth in development area.

Maximum day and peak hour demands were calculated by applying the respective peaking factor to the average day demand.

A desktop analysis of historical water demands and future water demand projections is included in **Appendix B**.

6.4.1 SUPPLY CAPACITY

The rated and estimated operating capacities for the Valley Water System are summarized in the table below. The rated capacity is that which is listed in the facility's PTTW, while the estimated actual capacity is the maximum practical capacity, described and detailed previously in Section 4.2. Each is further broken down according to total and firm capacity. The firm capacity and estimated firm capacities are defined as the rated and estimated actual capacities, respectively, less the largest well.

Table 6-4 Supply Capacity of Valley (including Capreol) Water System

SOURCE	RATED CAPACITY (M3/D)	ESTIMATED ACTUAL CAPACITY (M3/D)	TOTAL FIRM CAPACITY (M3/D)	ESTIMATED FIRM CAPACITY (M3/D)
Valley and Capreol Wells	34,285	28,454	30,358	24,579

The value corresponding to the estimated firm production capacity (24,579 m^3/d) was used for comparison against future needs of the Valley Water System. This value provides a realistic and slightly conservative approach to the comparison, and takes into account the potential future loss of production volume equal to the largest well in the system out of service.

The projected maximum day demands are plotted versus the rated, estimated, and firm system capacities on Figure 6-1 below.

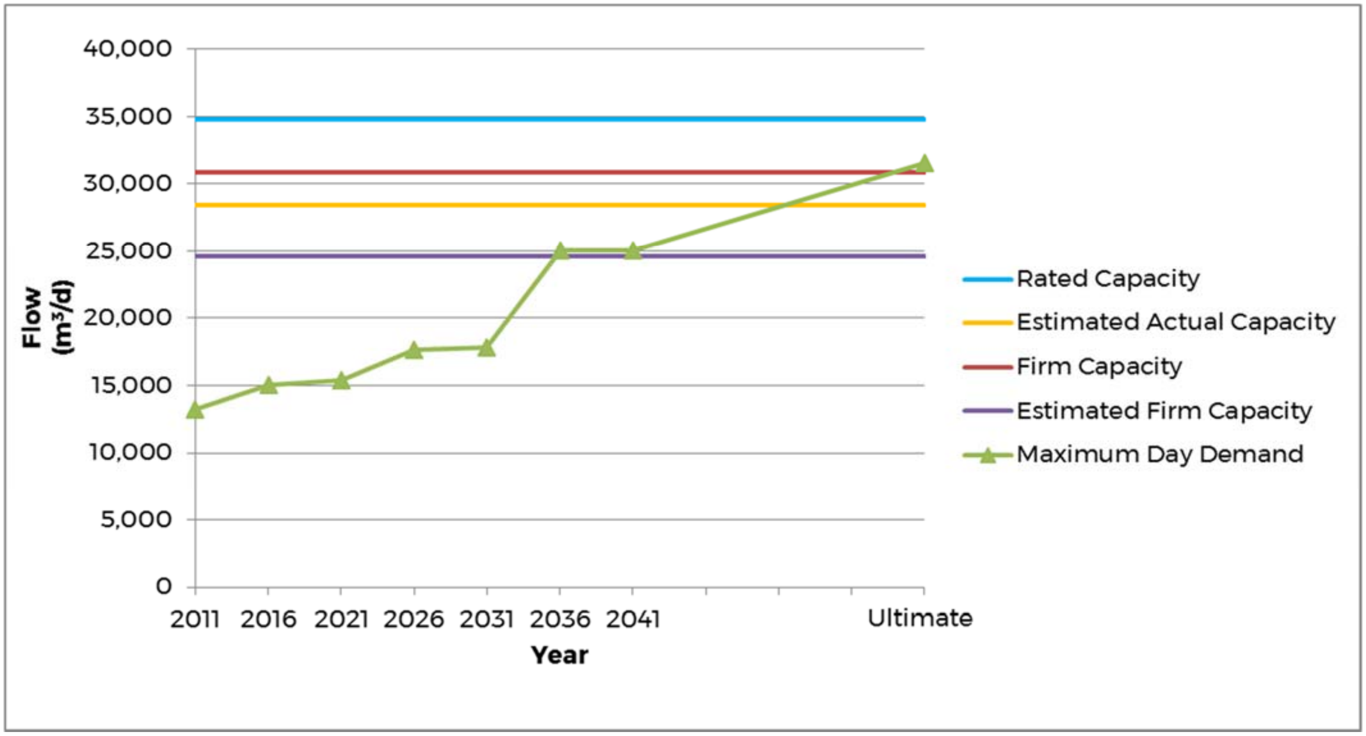

Figure 6-1 Water Demand Projections Compared to Rated Total and Estimated Firm Capacities

Therefore, the Valley Water System (including Capreol) has sufficient firm production capacity to service planned population growth to 2031. Additional supply may be required if the production capacity cannot be increased to meet rated capacity and will be discussed in the Alternative Solutions report.

6.4.2 STORAGE CAPACITY

Storage in the distribution system is provided by three storage tanks, located in Azilda, Chelmsford, and Val Caron. Note that although the entire Valley Water System is interconnected, each tank generally services its own specific community. It is also important to note that the Azilda and Chelmsford tanks are located at the opposite end of the system from the supply (wells), and are connected only by a single trunk watermain. This deficiency will be reviewed in detail in the final Master Plan Report.

Applying the formula to determine storage requirements indicated previously, the corresponding fire storage requirement would be 1,080 m^3. Using the maximum day demand required to service growth to 2041 (25,011 m^3/d), the corresponding equalization storage requirement would be 6,253 m^3 and the emergency storage would be 1,833 m^3. The total required storage to service growth to 2041 would be 9,166 m^3.

The total required storage to service the Ultimate Buildout growth scenario would be 11,205 m^3.

Therefore, the existing total storage volume of 11,519 m^3 provides sufficient storage for the Valley Water System to service growth to Ultimate Buildout.

The amount of storage required for each horizon year is shown in the figure below.

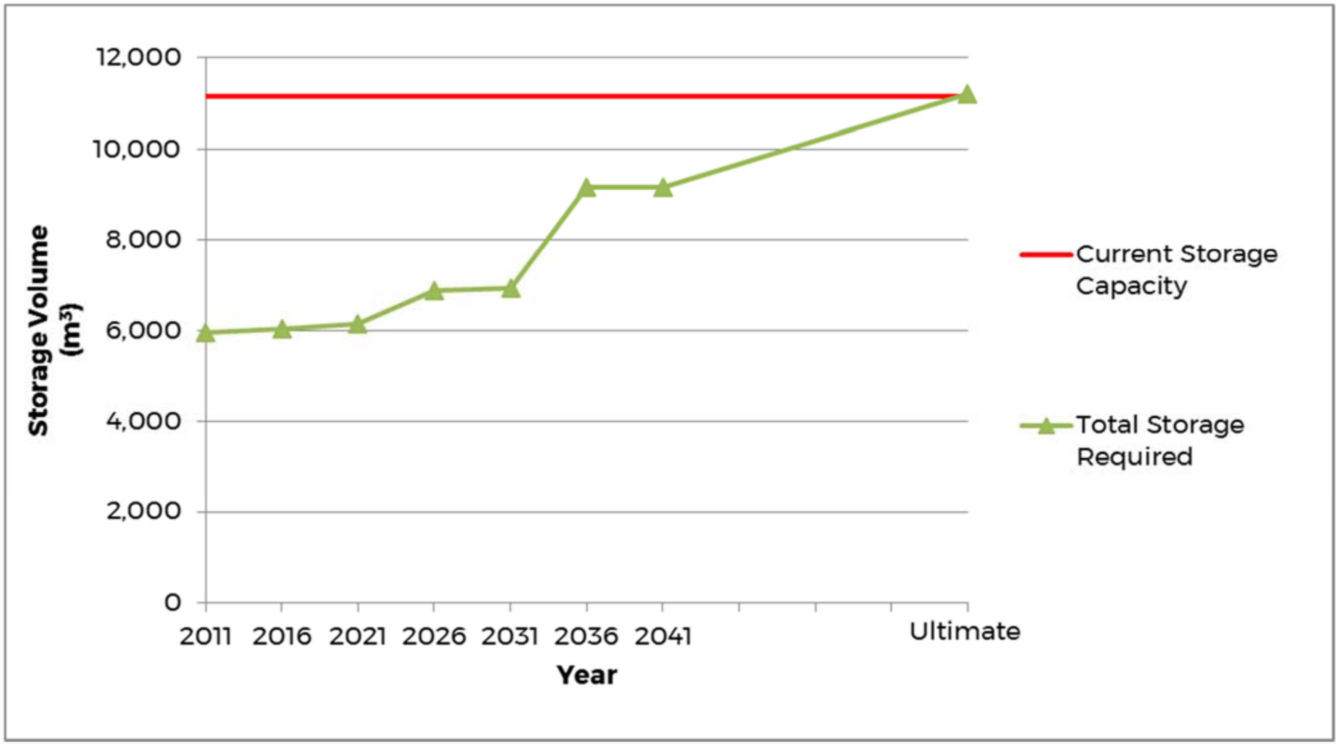

Figure 6-2 Available Storage Capacity Compared to Future Needs (Overall Valley Water System)

6.4.3 DISTRIBUTION NETWORK

The water model was used to identify system elements (i.e. watermains, pumps, storage tank) for which the capacity was exceeded by the projected water demands. The capacity of the system was assessed in terms of the available fire flows and system pressures.

For each planning scenario, watermains of the modelled network were reviewed to assess whether the required minimum fire flows (75 L/s in residential areas or 150 L/s in ICI areas) and pressures (over 20 psi under fire conditions and over 40 psi under normal conditions) were achieved. Furthermore, some new watermains were added to service greenfield areas where development was planned. A simplified watermain layout was assumed for these areas.

Future populations and demands were loaded into the model based on the planning data and flow projections discussed in earlier in Section 6.4. Development that would take place as part of the Urban Expansion Area has been excluded from the Ultimate Buildout modeling scenario to avoid overestimating demands. In general, development in Valley might deviate from the proposed phasing scheme. Thus, it is recommended that the hydraulic water model be updated whenever a development application is submitted.

The findings from the water modeling are discussed in Section 7.2 and presented in **Appendix C**.

7 HYDRAULIC WATER SYSTEM MODELLING

An all-pipe model of the system including pipes, hydrants, storage tanks and system source was developed by the City using Bentley Systems' WaterGEMS hydraulic modeling software. This model was updated based on information provided by the City to reflect current system conditions.

The water model allows for simulations to be conducted that can be used to predict system responses to events under a wide range of conditions. Using simulations, problems can be anticipated in proposed or existing systems, and solutions can be evaluated before time, money, and materials are invested in a real-world project. Simulations can either be steady-state or extended-period. Steady-state simulations represent a snapshot in time and are used to determine the operating behaviour of a system under static conditions. This type of analysis can be useful in determining the short-term effect of fire flows or average demand conditions on the system. Extended period simulations (EPS) are used to evaluate system performance over time. This type of analysis allows modeling the filling and emptying of storage facilities, regulating valves opening and closing, and pressures and flow rates changing throughout the system in response to varying demand conditions and automatic control strategies.

Simulations including steady-state analysis of the Average Day, Maximum Day and Maximum Day + Fire conditions were carried out using the model. Fire flow simulations were carried out throughout the system to determine whether the system could deliver fire flows under the Maximum Day demands.

7.1 WATER MODEL DEVELOPMENT

To model the current scenario, the following steps were taken:

— Total network demand on an average day basis was determined for the current scenario using 2012 water production data.

— The node demand allocations assigned in the model were based on 2012 meter records. Metered flows were assigned to the respective property. In cases where meter records showed zero flow, the value was manually adjusted to reflect a reasonable volume for a respective property, depending on land use.

— The maximum day peaking factor was applied to the average day demand value to determine the maximum day demand.

— The maximum day demand plus fire flow was used to assess the system since it was greater than the peak hour demand.

7.2 MODELLING FINDINGS

7.2.1 FIREFIGHTING CAPACITY

Firefighting capacity was assessed for the distribution system, with exception of areas not designed to convey fire flows. These include areas that were constructed under different design standards; these areas have small diameter (150 mm or less) watermains and no fire hydrants. As such, these were not included in the below assessment.

As noted above, fire flow requirements of 75 L/s for residential areas and 150 L/s for ICI areas were used. Based on these criteria, the model revealed that flows meet current fire flow standards in most areas of the Valley Water System. There are small areas throughout the distribution system that do not meet current fire flow standards, as illustrated in **Appendix C**. Similar trends are observed for 2041 and Ultimate Buildout scenarios, as shown in **Appendix C**.

7.2.2 MODELLED HYDRAULIC CAPACITY UNDER NORMAL CONDITIONS

Based on the system modeling, service pressures throughout the system under the maximum day demand scenario generally range between 40 and 80 psi (276 and 552 kPa) for 2011 to Ultimate Buildout. There are some areas in the Valley Water System with pressures of 80 to100 psi (552 to 689 kPa), including the easternmost end of Chelmsford, southernmost part of Azilda, and parts of Val Caron.

Therefore, flows throughout the system are generally within the range prescribed in the MOECC Guidelines (40 to 100 psi) under normal conditions.

Maps showing pressures at nodes are presented in **Appendix C**.

8 CONCLUSIONS

An assessment of the Valley Water System was completed to identify infrastructure investment requirements to service forecasted growth in the community. The assessment involved a review of previous studies, an analysis of operations and flow data from the water and wastewater facilities, and an evaluation of the capacity of the system.

The conclusions of the assessment are summarized below.

- Based on the estimated firm operating capacity of the Valley Water System as well as historical and projected demands, the wells have sufficient firm production capacity to service growth to 2031. Additional supply may be required if the production capacity cannot be increased to meet rated capacity and will be discussed in the Alternative Solutions report.
- There is enough storage capacity in the system to service growth to Ultimate Buildout.
- The model revealed that flows meet current fire flow standards in most areas of Valley, except along 150 mm diameter watermains that were built to previous standards and certain areas throughout the system.

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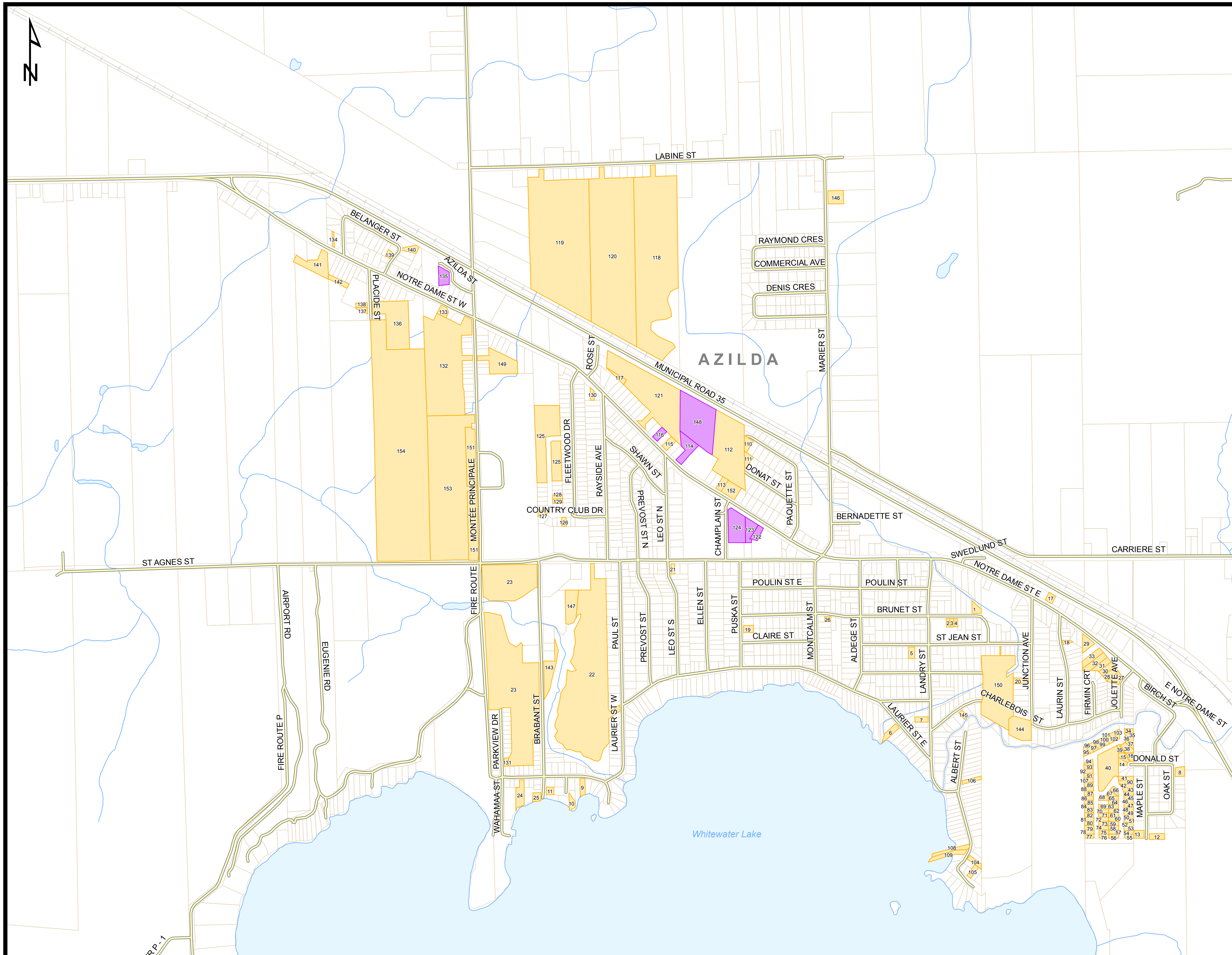
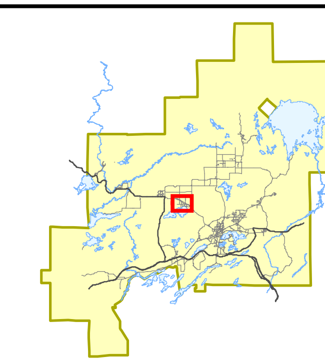
APPENDIX

A RESIDENTIAL AND ICI DEVELOPMENT AREAS



Legend

- Road
- Railway
- Watercourse
- Waterbody
- Parcel
- Vacant ICI
- Vacant Residential



Greater Sudbury Grand

WSP

Project No. 121-23026-00

Sudbury Water and Wastewater Master Plan

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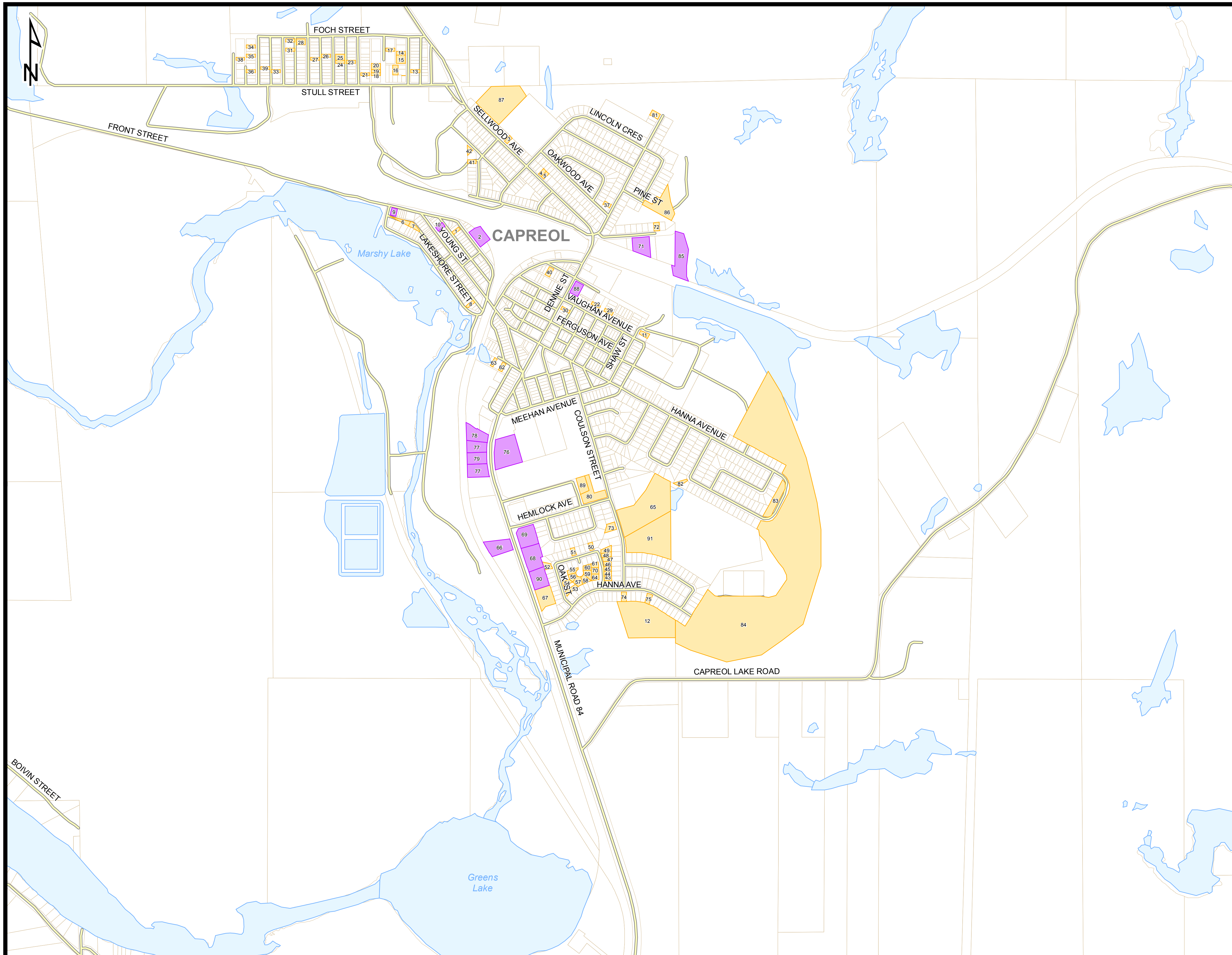
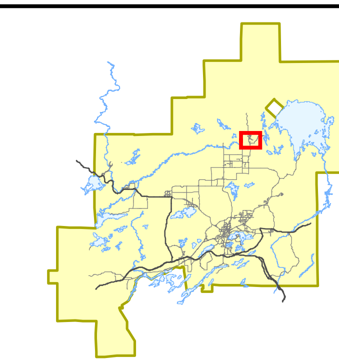
Map 1

Azilda
2011 - Ultimate Buildout Residential and ICI Development Areas



Legend

- Road
- Waterbody
- Parcel
- Vacant ICI
- Vacant Residential



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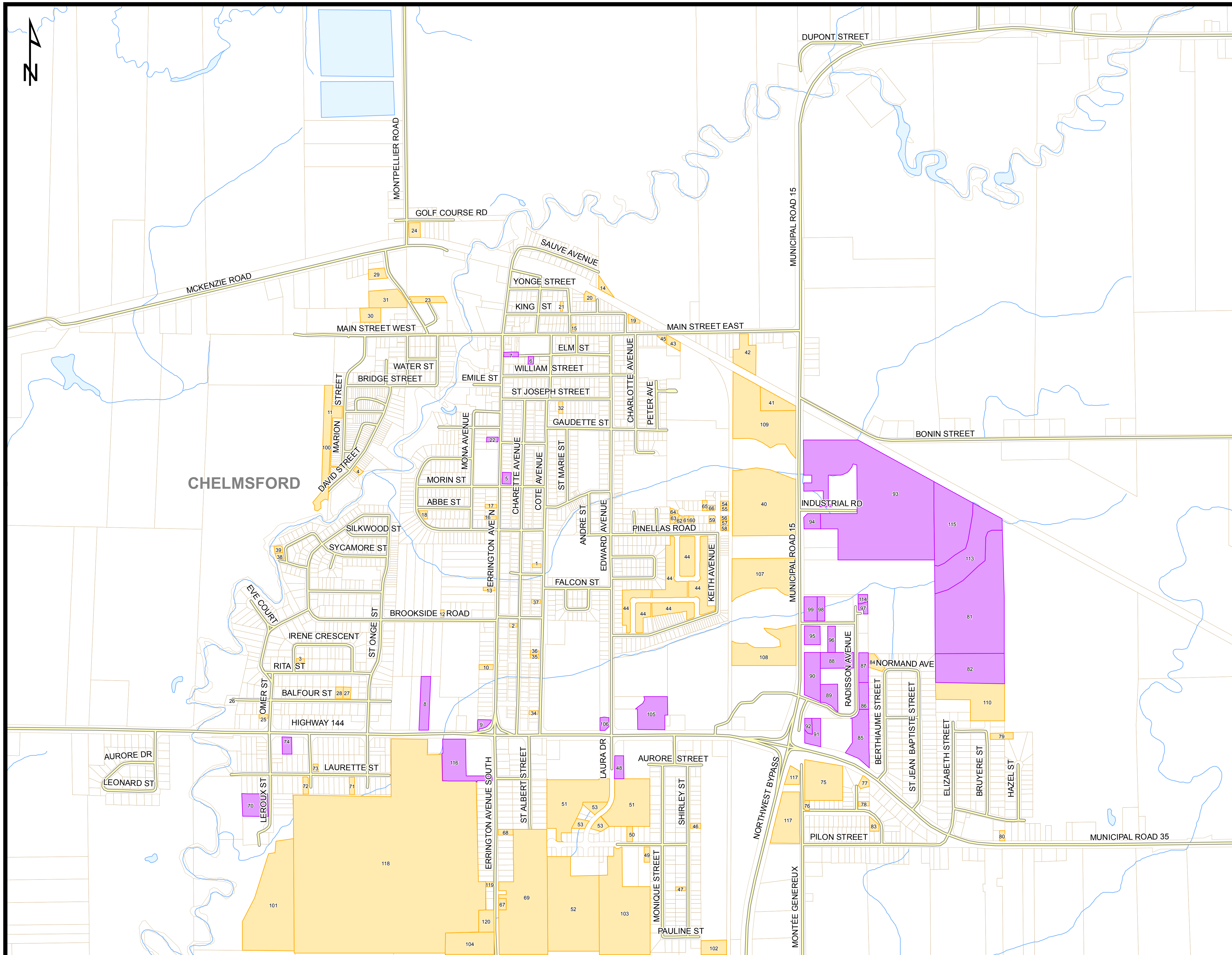
Map 2

Capreol
2011 - Ultimate Buildout Residential and ICI Development Areas



Legend
Road
Watercourse
Waterbody
Parcel
Vacant ICI
Vacant Residential

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Sudbury Water and Wastewater Master Plan

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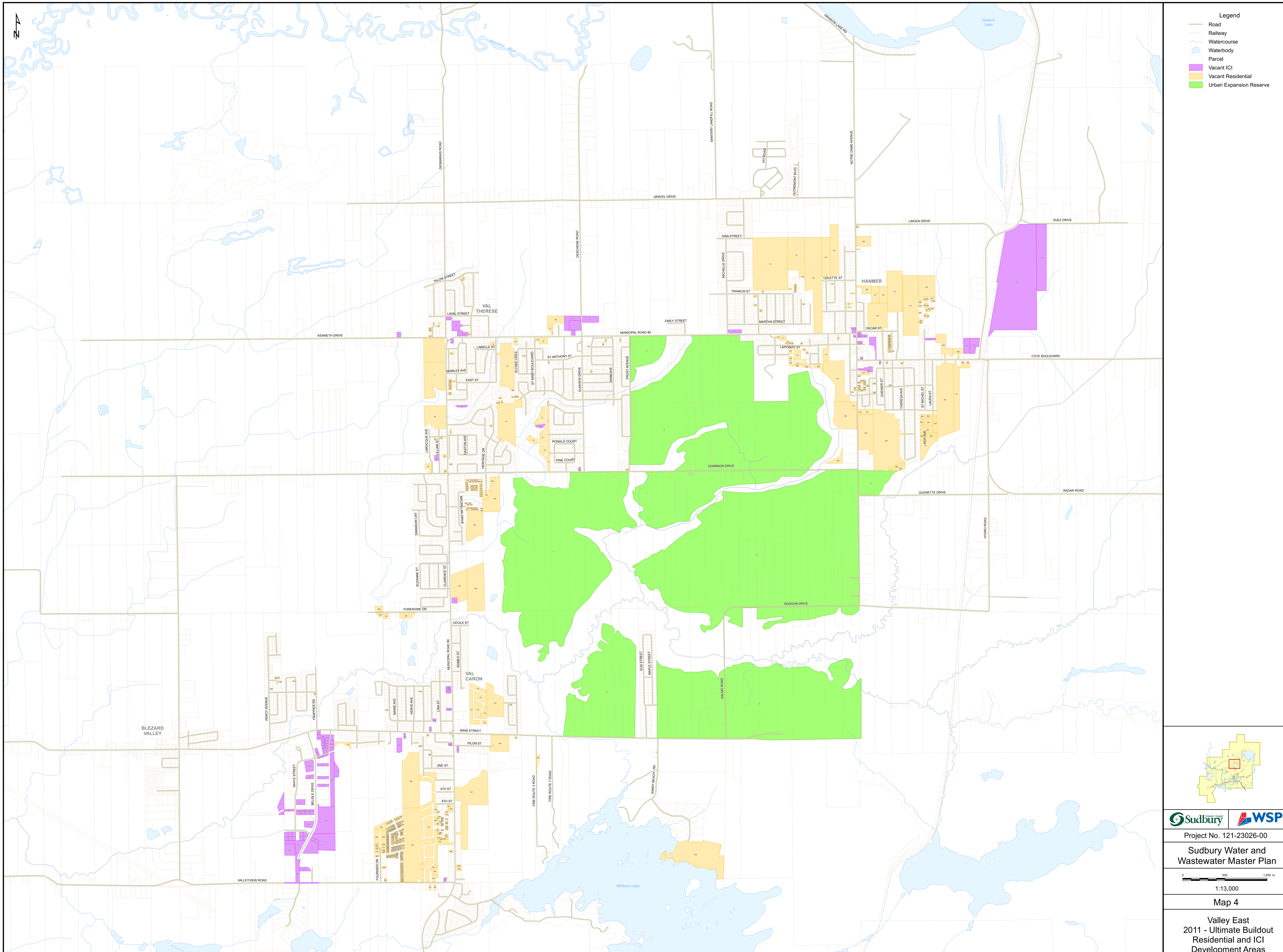
Map 3

Chelmsford
2011 - Ultimate Buildout
Residential and ICI
Development Areas



Legend
Road
Railway
Watercourse
Waterbody
Parcel
Vacant ICI
Vacant Residential
Urban Expansion Reserve
N
VAL THERESE
HANMER
VAL CARON
BLEZARD VALLEY
Whitson Lake
Greens Lake
GRAVEL DRIVE
MUNICIPAL ROAD 80
KENNETH DRIVE
DOMINION DRIVE
GUENETTE DRIVE
RADAR ROAD
COTE BOULEVARD
LINDEN DRIVE
SUEZ DRIVE
BODSON DRIVE
MAIN STREET
VALLEYVIEW ROAD
YORKSHIRE DR
DESMARAIS ROAD
DESCHENE ROAD
NOTRE DAME AVENUE
SANITARY LANDFILL ROAD
HYDRO ROAD
KALMO ROAD
SANDY BEACH RD
Sudbury
WSP
Project No. 121-23026-00
Sudbury Water and Wastewater Master Plan
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Map 4
Valley East
2011 - Ultimate Buildout
Residential and ICI
Development Areas

APPENDIX

B WATER DEMAND AND CAPACITY ASSESSMENTS

Valley - Water Demand Forecasts 1/22/2018

(Includes: Valley East, Capreol, Azilda, and Chelmsford)

DATA ANALYSIS

	2009	2010	2011	2012	2013	Summary	Design Criterion	Comments
Average Day Flow (m^3/d)	Not Available	9,999	10,114	10,080	10,135	10,082	10,082	From Water Historical Production data. The daily production values for each facility were added together to determine the total daily production.
Max Day Flow (m^3/d)		17,601	13,201	14,055	14,182	14,760	14,760	
Max Day Factor		1.76	1.31	1.39	1.40	1.46	1.46	MOE Guidelines recommend a value of 1.80 for populations between 25,001 and 50,000. This is in line with the historical maximum, and so the historical maximum was adopted.
Peak Hour (L/s)	Not Available		220.0	210.5	255.8			Peak values were available only for 2011-2013.
Peak Hour (m^3/d)			19,007	18,189	22,105	19,767		
Peak Hour Factor			1.88	1.80	2.18	1.95	2.18	MOE Guidelines recommend a value of 2.70 for populations between 25,001 and 50,000. However, the historical maximum was consistently lower than 2.70 and so the historical maximum was adopted.

	2010	2011	2012	2013	Design Criterion	2016	2021	2026	2031	2036	2041	Ultimate	Comments
Population (Existing Areas)	36,382	36,382	36,382	36,382	36,382	36,382	36,382	36,382	36,382	36,382	36,382	36,382	From data provided by Hemson grouped by water system.
Population Growth						853	1,760	2,583	3,069	3,355	3,382	21,259	
Total Population						37,235	38,142	38,965	39,451	39,737	39,764	57,641	From data provided by Hemson grouped by water system.
Residential Growth Area (ha)						83.34	9.73	4.62	16.51	1.16	1.17	436.12	From City's GIS database. 2036 and 2041 areas are included with 2031.
Residential Growth Area (ha) - Cumulative						83.34	93.07	97.69	114.20	115.36	116.53	552.65	
Institutional Growth Area (ha)						0.00	0.00	6.60	0.00	0.00	0.00	0.00	
Institutional Growth Area (ha) - Cumulative						0.00	0.00	6.60	6.60	6.60	6.60	6.60	From City's GIS database.
Commercial Growth Area (ha)						0.00	0.00	13.62	0.00	8.71	0.00	0.00	
Commercial Growth Area (ha) - Cumulative						0.00	0.00	13.62	13.62	22.33	22.33	22.33	From City's GIS database.
Industrial Growth Area (ha)						0.00	0.00	23.02	0.00	131.07	0.00	0.00	
Industrial Growth Area (ha) - Cumulative						0.00	0.00	23.02	23.02	154.09	154.09	154.09	From City's GIS database.
ICI (ha) - Cumulative						0.00	0.00	43.24	43.24	183.02	183.02	183.02	Sum of Institutional, Commercial and Industrial areas
Total Growth Area (ha) - Cumulative						83.34	93.07	140.93	157.44	298.38	299.55	735.67	

	2010	2011	2012	2013	Summary	Design Criterion	Comments
Ratio of Residential to Total Water Billed	0.87	0.87	0.87	0.87	0.87		Estimated amount of water consumption related to ICI based on metering data and obtained ratio of residential to total consumption.
Residential Flow (m^3/d)	8683	8782	8753	8800	8,754		Calculated based on ratio of residential consumption to total consumption.
Ratio of ICI to Total Water Billed	0.13	0.13	0.13	0.13	0.132		
	1317	1332	1327	1335	1,328		
Per Capita Residential Demand (m^3/cap/day)	0.239	0.241	0.241	0.242	0.241	0.250	Took average over 2009 to 2013 period. The trend is generally consistent.
Average Institutional Flow Unit Rate (m^3/ha/d)						28.0	*MOE Guidelines recommend a value of 28 m^3/ha/d.*
Average Commercial Flow Unit Rate (m^3/ha/d)						28.0	*MOE Guidelines recommend a value of 28 m^3/ha/d.*
Average Industrial Flow Unit Rate (m^3/ha/d)						35.0	*MOE Guidelines recommend a value of 35 m^3/ha/d for light industry and 55 m^3/ha/d for heavy industry.*

	2016	2021	2026	2031	2036	2041	Ultimate
Average Residential and ICI Flows (m^3/d) - Existing	10,082	10,082	10,082	10,082	10,082	10,082	10,082
Average Residential Flows (m^3/d) - Growth	213	440	646	767	839	846	5,315
Average Residential Flows (m^3/d) - Total	10,295	10,522	10,728	10,849	10,921	10,928	15,397
Average Institutional Flow (m^3/d)	0	0	185	185	185	185	185
Average Commercial Flow (m^3/d)	0	0	381	381	625	625	625
Average Industrial Flow (m^3/d)	0	0	806	806	5,393	5,393	5,393
Average ICI Flow (m^3/d)	0	0	1,372	1,372	6,203	6,203	6,203
Average Day Flow (m^3/d)	10,295	10,522	12,100	12,221	17,124	17,131	21,600
Max Day Flow (m^3/d)	15,031	15,362	17,665	17,843	25,001	25,011	31,536
Peak Hour Flow (m^3/d)	22,456	22,950	26,391	26,656	37,350	37,365	47,113

ALTERNATIVE CALCULATION METHOD This method does not distinguish between Residential and ICI water consumption.

	2010	2011	2012	2013	Design Criterion	Comments
Per Capita Demand (m^3/cap/day)	0.275	0.278	0.277	0.279	0.277	*If ICI is not considered explicitly and demand is divided by total population.*
Max Day Factor					1.76	*The historical per capita consumption is applied for future development.*
Peak Hour Factor					2.18	

	2016	2021	2026	2031	2036	2041	Ultimate
Average Day Flow (m^3/d)	10,318	10,570	10,798	10,933	11,012	11,019	15,973
Max Day Flow (m^3/d)	18,163	18,605	19,007	19,244	19,383	19,397	28,117
Peak Hour Flow (m^3/d)	22,506	23,054	23,552	23,846	24,018	24,035	34,840

SENSITIVITY ANALYSIS

Analyze sensitivity of forecasted average day flows to unit rate

Average Day Flow (m^3/d)

	Unit Rate (m^3/cap/d)	2016	2021	2026	2031	2036	2041	Ultimate
Using a consolidated per capita flow	0.277	10,318	10,570	10,798	10,933	11,012	11,019	15,973
Using estimated average	0.250	10,295	10,522	12,100	12,221	17,124	17,131	21,600
City Standards	0.41	10,432	10,804	12,513	12,712	17,661	17,672	25,002

Analyze sensitivity of forecasted flows to max day peaking factor

Max Day Flow (m^3/d)

	Max Day Peaking Factor	2016	2021	2026	2031	2036	2041	Ultimate
2009-2013 average of peaking factors	1.46	15,081	15,413	17,724	17,902	25,083	25,093	31,640
Maximum historical max day factor	1.76	15,031	15,362	17,665	17,843	25,001	25,011	31,536
MOE Guidelines	1.65	16987	17361	19964	20165	28254	28266	35640

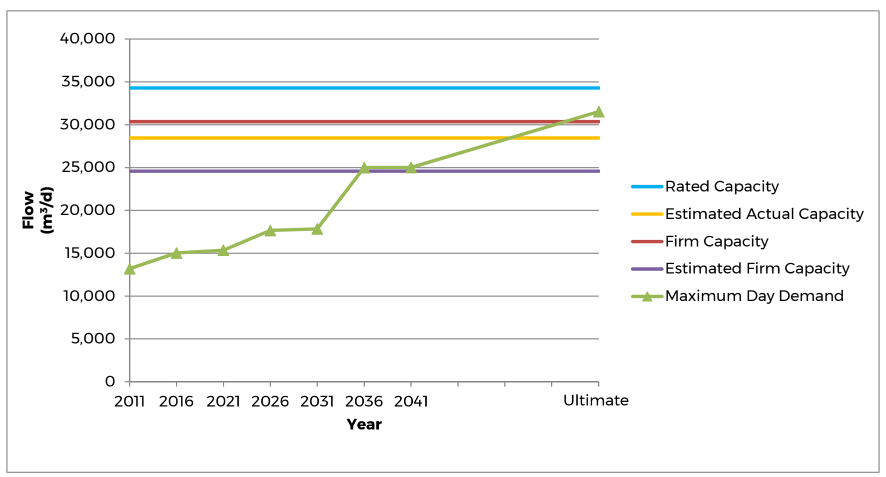
Analyze sensitivity of forecasted flows to peak hour factor

Peak Hour (m^3/d)

	Peak Hour Peaking Factor	2016	2021	2026	2031	2036	2041	Ultimate
Using historical highest peak factor	2.18	22,456	22,950	26,391	26,656	37,350	37,365	47,113
Using average of historical peaking factors	1.95	20,127	20,570	23,655	23,892	33,477	33,490	42,228
MOE Guidelines	2.48	25,532	26,094	30,007	30,309	42,467	42,484	53,568

CAPACITY CHECK

	2011	2016	2021	2026	2031	2036	2041	Ultimate
Rated Capacity	34,285	34,285	34,285	34,285	34,285	34,285	34,285	34,285
Estimated Firm Capacity	24,579	24,579	24,579	24,579	24,579	24,579	24,579	24,579
Maximum Day Demands	13,201	15,031	15,362	17,665	17,843	25,001	25,011	31,536
Peak Hour Demands	19,007	22,456	22,950	26,391	26,656	37,350	37,365	47,113
Firm Capacity	30,358	30,358	30,358	30,358	30,358	30,358	30,358	30,358
Estimated Actual Capacity	28,454	28,454	28,454	28,454	28,454	28,454	28,454	28,454



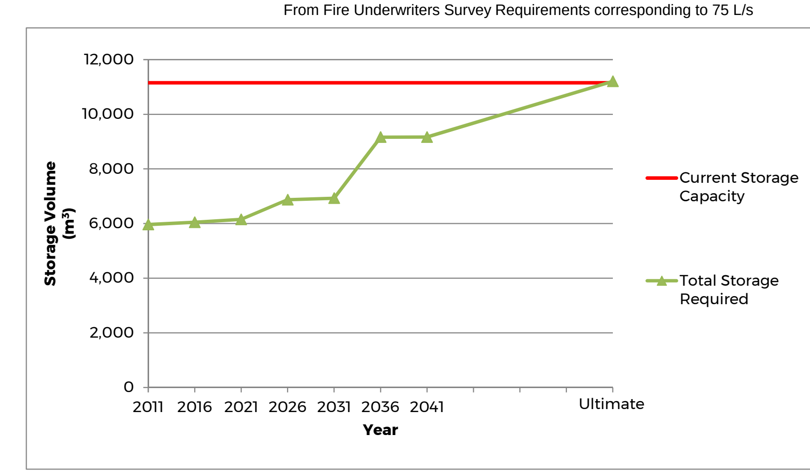
STORAGE REQUIREMENTS

Total Rated System Capacity is the sum of the capacity of all wells (per Drinking Water Licence)

Firm Production Capacity is the sum of the production capacity of all wells, less the one largest. See Baseline Review Report

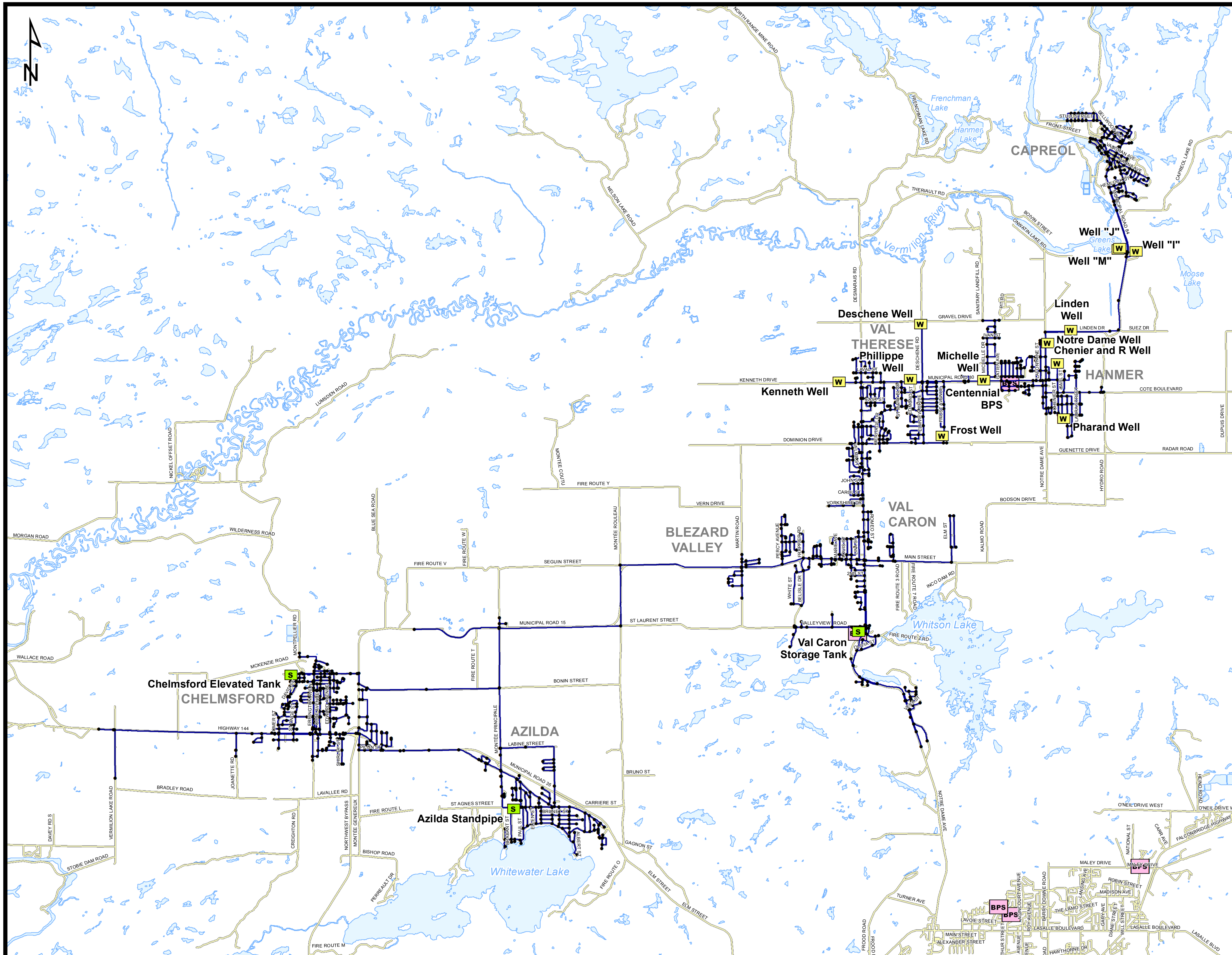
Storage Available		
Val Caron Elevated Tank (m^3)	5,274	From DWWP 016-205, Issue 2 (November 1, 2011)
Azilda Elevated Tank (m^3)	4,524	From DWWP 016-205, Issue 2 (November 1, 2011)
Chelmsford Elevated Tank (m^3)	1,353	From DWWP 016-205, Issue 2 (November 1, 2011)
Total Storage (m^3)	11,151	
Maximum Fire flow Requirements (L/s)	150	From CGS Engineering Design Manual
Fire Duration (hrs)	2	From Fire Underwriters Survey Requirements corresponding to 75 L/s
Minimum Fire Flow Requirement for Residential Areas (L/s)	75	
Fire Duration (hrs)	1.75	

	Max Day Demand (m^3/d)	Required Fire Flow (m^3/d)	Max Day + Fire (m^3/d)	Peak Hour (m^3/d)	A - Fire Storage (m^3)	B - Equalization Storage (m^3)	C - Emergency Storage (m^3)	A + B + C = Storage Required (m^3)	Storage Available (m^3)	Deficit (m^3)
2011	14,760	12,960	27,720	19,767	1,080	3,690	1192	5,962	11,151	0
2016	15,031	12,960	27,991	22,456	1,080	3,758	1209	6,047	11,151	0
2021	15,362	12,960	28,322	22,950	1,080	3,841	1230	6,151	11,151	0
2026	17,665	12,960	30,625	26,391	1,080	4,416	1374	6,870	11,151	0
2031	17,843	12,960	30,803	26,656	1,080	4,461	1385	6,926	11,151	0
2036	25,001	12,960	37,961	37,350	1,080	6,250	1833	9,163	11,151	0
2041	25,011	12,960	37,971	37,365	1,080	6,253	1833	9,166	11,151	0
Ultimate	31,536	12,960	44,496	47,113	1,080	7,884	2241	11,205	11,151	54



APPENDIX

C WATER MODEL RESULTS



Legend

- W Well
- S Storage Tank
- BPS Booster Pumping Station (BPS)
- WTP Water Treatment Plant (WTP)
- Pipe
- Road
- Waterbody

Greater Sudbury | WSP

Project No. 121-23026-00

April 04 2016

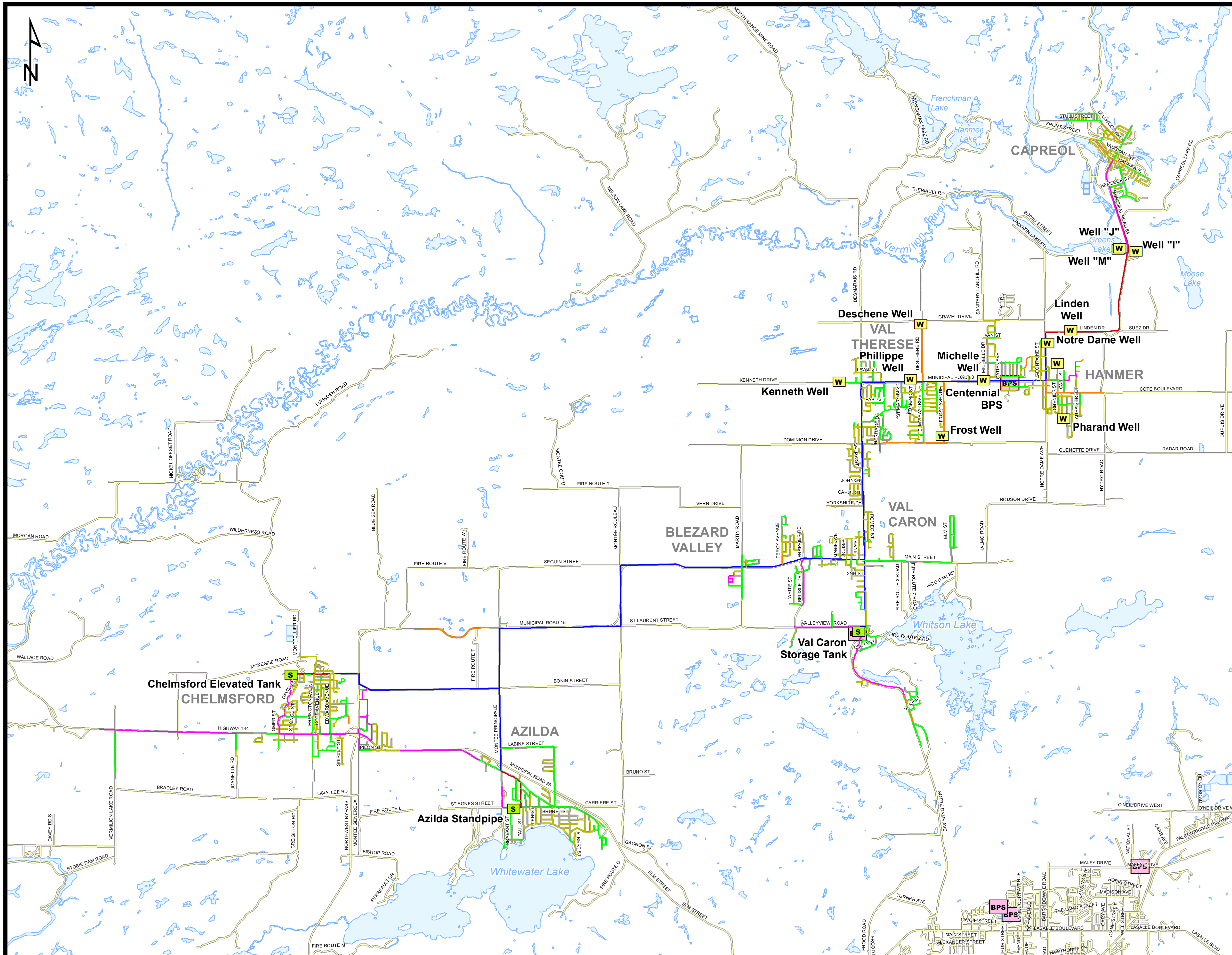
Sudbury Water and Wastewater Master Plan

0 1,000 2,000 3,000 4,000 5,000 m

1:100,000

Map 1

Valley Water System - Key Plan



Legend

- W Well
- S Storage Tank
- BPS Booster Pumping Station (BPS)
- WTP Water Treatment Plant (WTP)
- Road
- Waterbody

Pipe: Diamater (mm)

- <= 150
- <= 200
- <= 250
- <= 300
- <= 400
- <= 600
- > 600

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April 04 2016

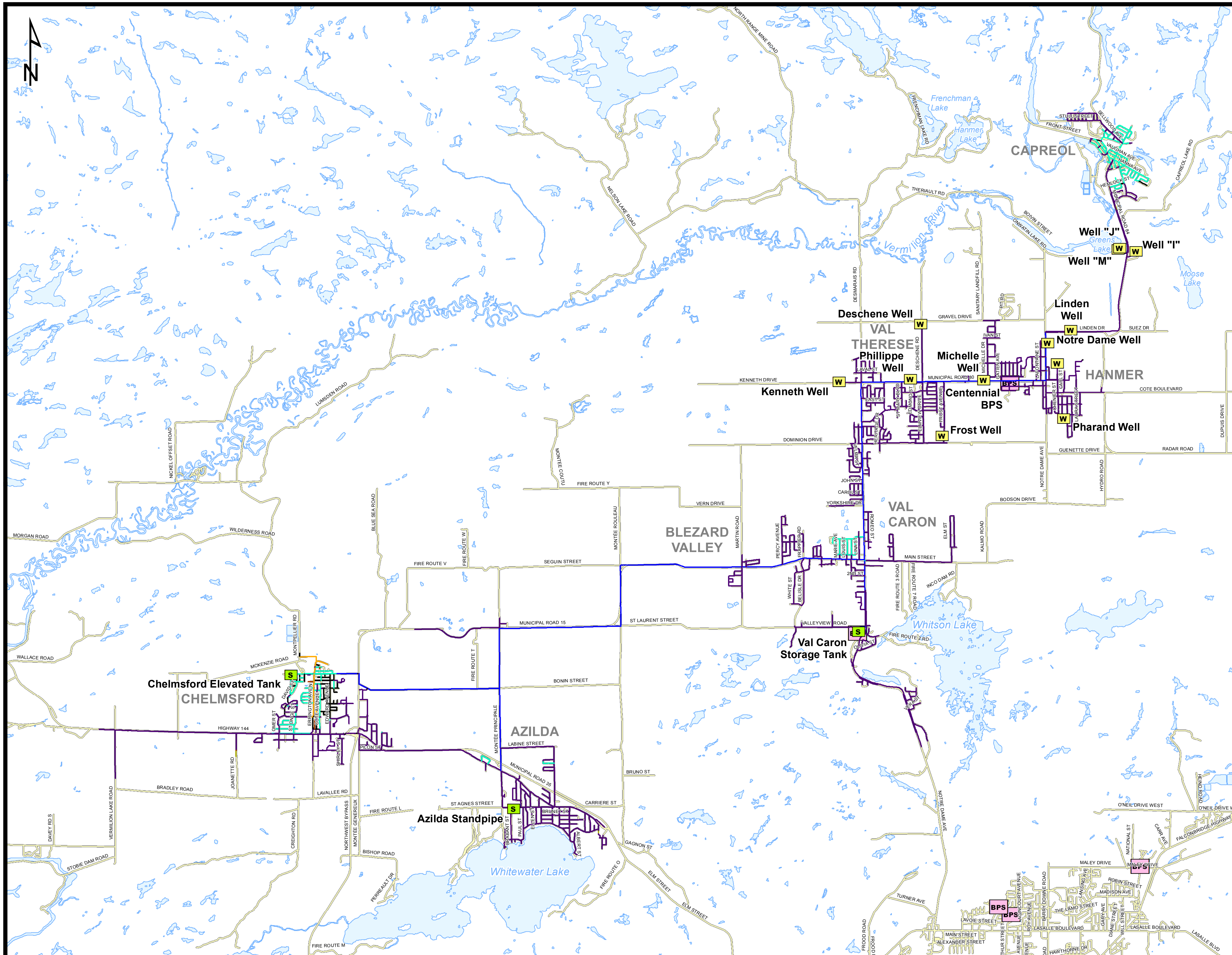
Sudbury Water and Wastewater Master Plan

0 1,000 2,000 3,000 4,000 5,000 m

1:100,000

Map 2

Valley Water System - Pipe Diameter



Legend

- W Well
- S Storage Tank
- BPS Booster Pumping Station (BPS)
- WTP Water Treatment Plant (WTP)
- Road
- Waterbody

Material

- Asbestos Cement
- Cast iron
- Concrete
- Copper
- Ductile Iron
- Galvanized iron
- PVC
- Polyethylene
- Steel

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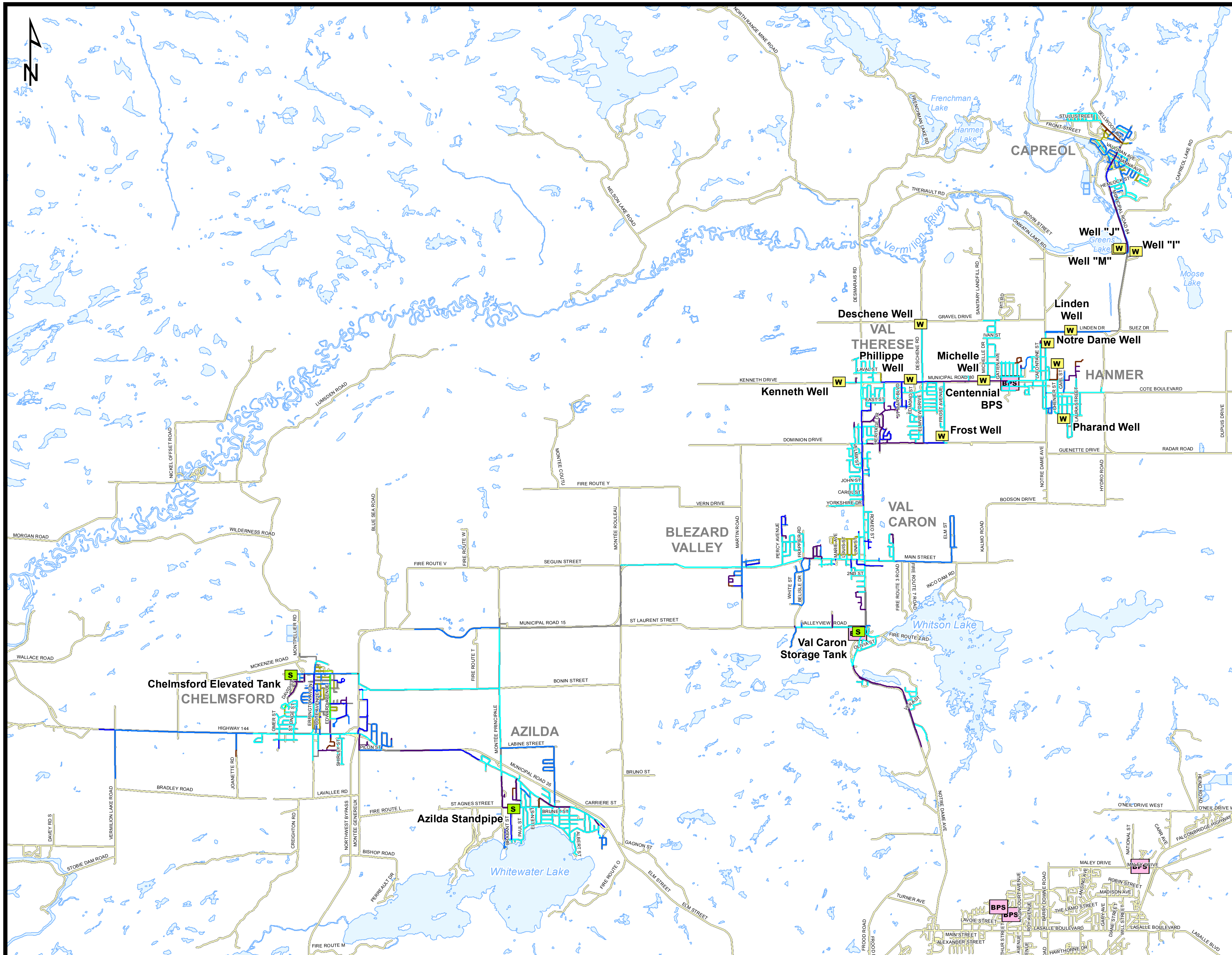
0 1,000 2,000 3,000 4,000 5,000 m

1:100,000

Map 3

Valley Water System - Pipe Material



Legend

- W Well
- S Storage Tank
- BPS Booster Pumping Station (BPS)
- WTP Water Treatment Plant (WTP)
- Road
- Waterbody

Installed Year

- N/A
- <= 1930
- <= 1940
- <= 1950
- <= 1960
- <= 1970
- <= 1980
- <= 1990
- <= 2000
- <= 2010
- > 2011

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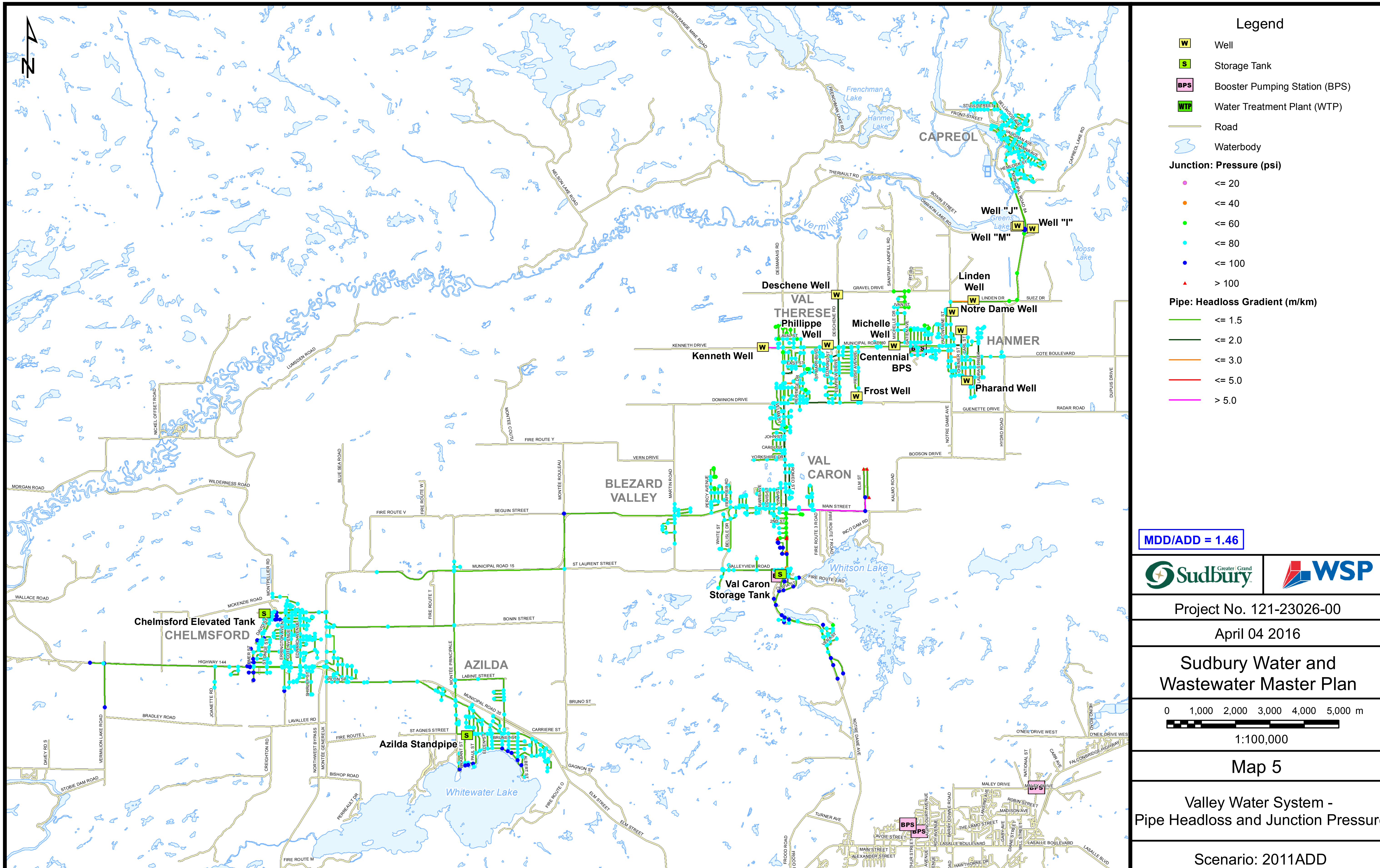
0 1,000 2,000 3,000 4,000 5,000 m

1:100,000

Map 4

Valley Water System - Pipe Age



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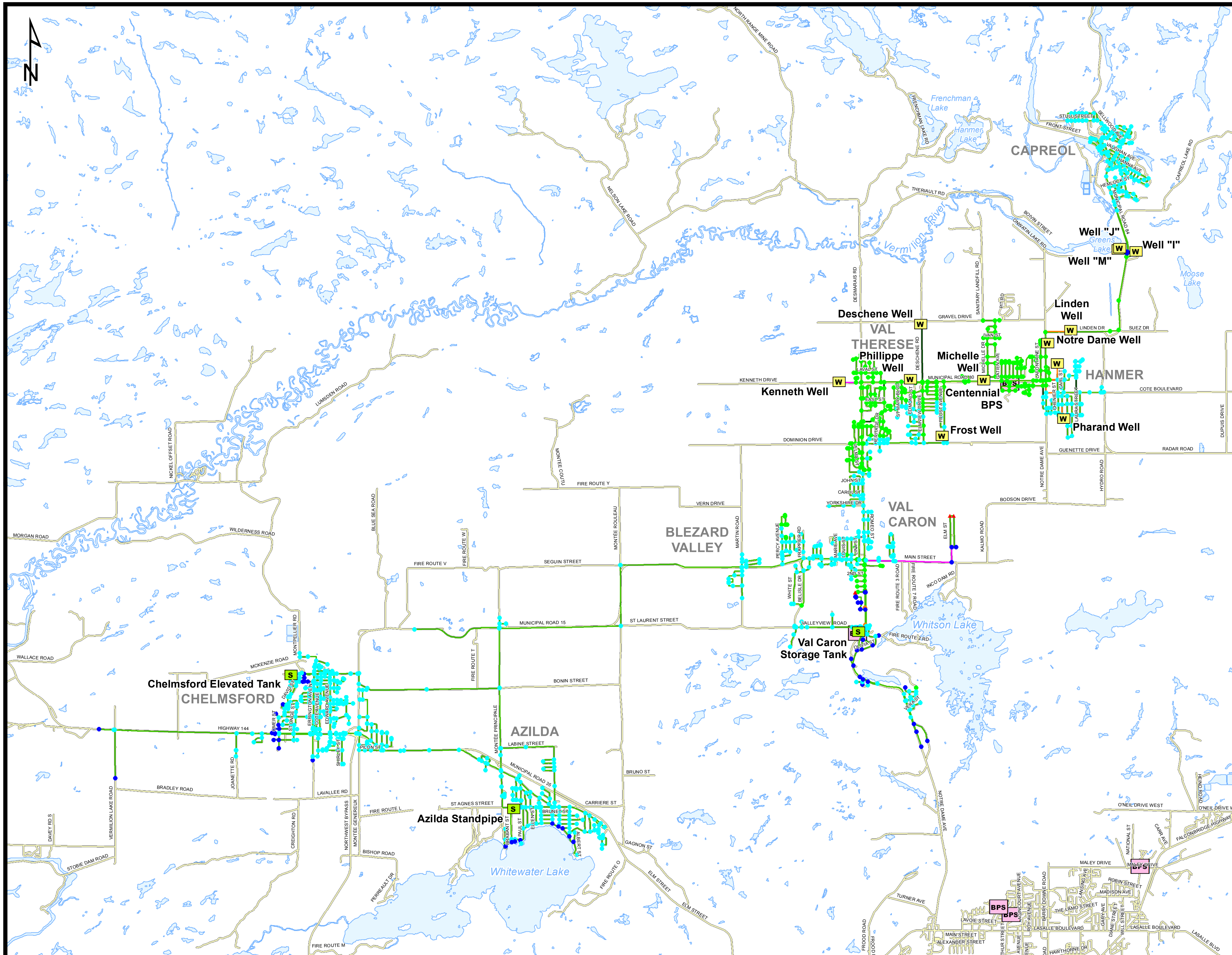
Map 5

Valley Water System - Pipe Headloss and Junction Pressure

Scenario: 2011ADD



Legend

- W Well
- S Storage Tank
- BPS Booster Pumping Station (BPS)
- WTP Water Treatment Plant (WTP)
- Road
- Waterbody

Junction: Pressure (psi)

- <= 20
- <= 40
- <= 60
- <= 80
- <= 100
- > 100

Pipe: Headloss Gradient (m/km)

- <= 1.5
- <= 2.0
- <= 3.0
- <= 5.0
- > 5.0

MDD/ADD = 1.46

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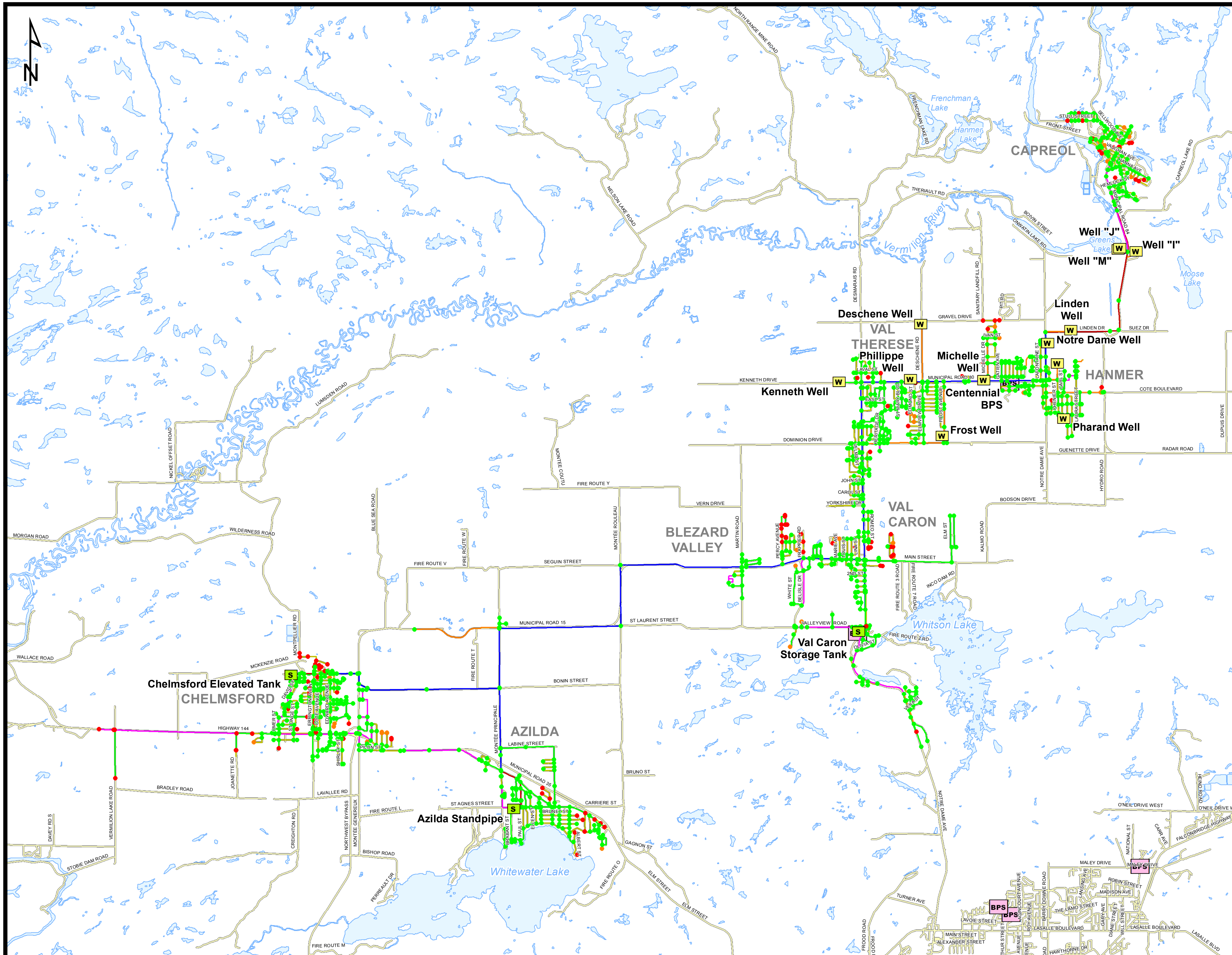
0 1,000 2,000 3,000 4,000 5,000 m

1:100,000

Map 6

Valley Water System - Pipe Headloss and Junction Pressure

Scenario: 2011MDD



Legend

- W — Well
- S — Storage Tank
- BPS — Booster Pumping Station (BPS)
- WTP — Water Treatment Plant (WTP)
- Road
- Waterbody

Junction: Fire Flow (Available) (L/s)

- <= 65 (Res) or 130 (ICI)
- < 75 (Res) or 150 (ICI)
- >= 75 (Res) or 150 (ICI)

Pipe: Diamater (mm)

- <= 150
- <= 200
- <= 250
- <= 300
- <= 400
- <= 600
- > 600

MDD/ADD = 1.46

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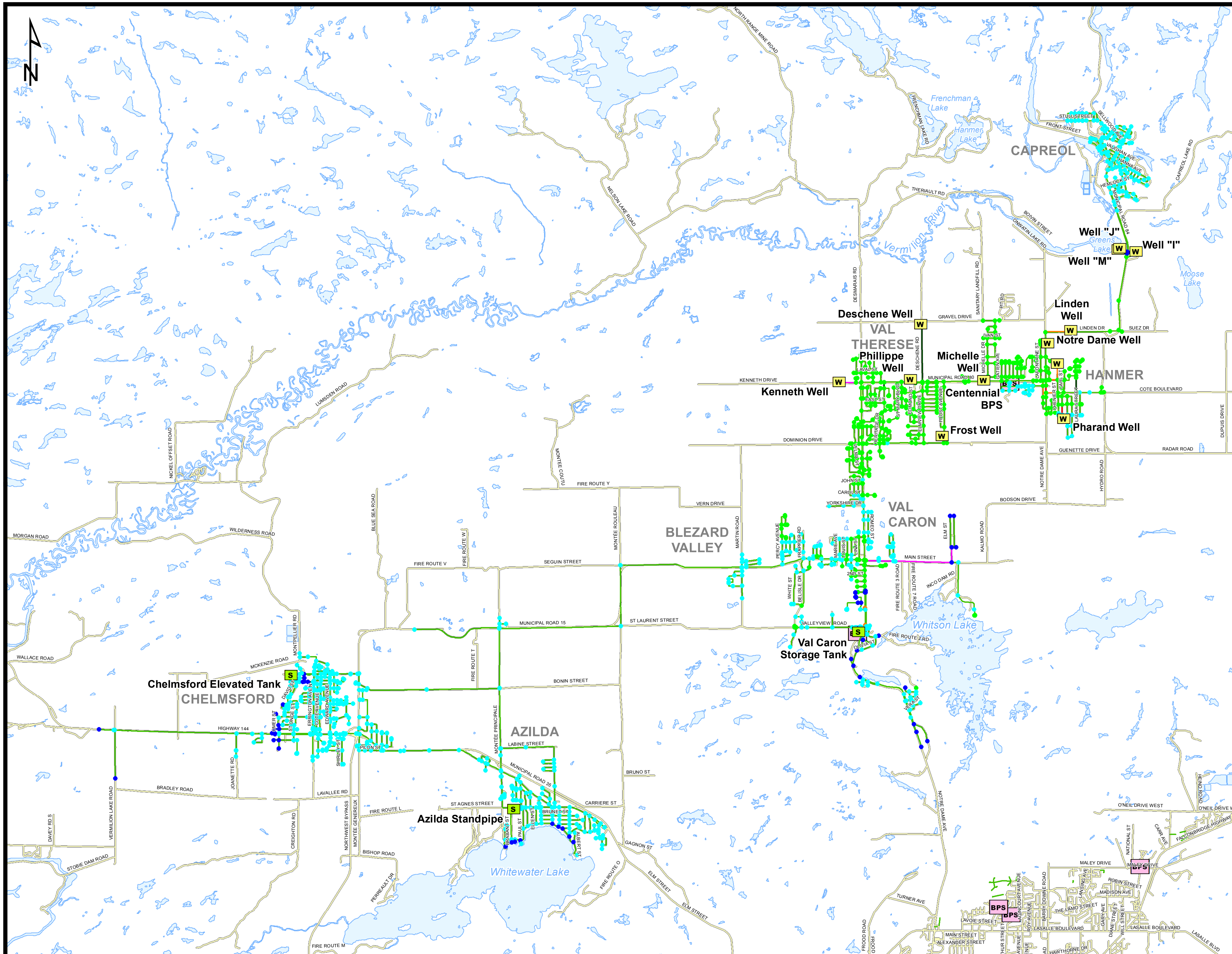
0 1,000 2,000 3,000 4,000 5,000 m

1:100,000

Map 7

Valley Water System - Fire Flow Availability

Scenario: 2011MDD+FF



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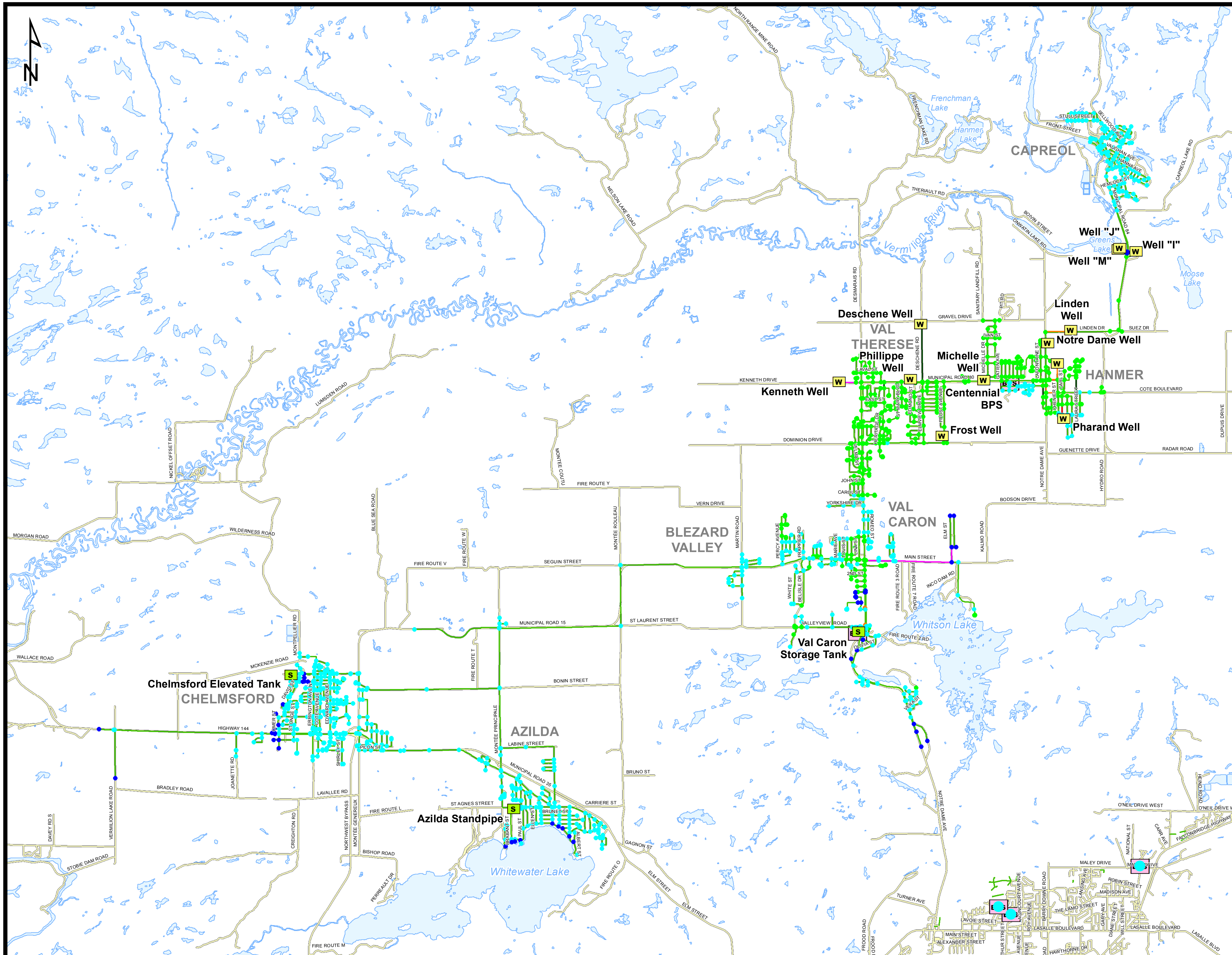
0 1,000 2,000 3,000 4,000 5,000 m

1:100,000

Map 8

Valley Water System - Pipe Headloss and Junction Pressure

Scenario: 2041ADD



Legend

- W Well
- S Storage Tank
- BPS Booster Pumping Station (BPS)
- WTP Water Treatment Plant (WTP)
- Road
- Waterbody

Junction: Pressure (psi)

- <= 20
- <= 40
- <= 60
- <= 80
- <= 100
- > 100

Pipe: Headloss Gradient (m/km)

- <= 1.5
- <= 2.0
- <= 3.0
- <= 5.0
- > 5.0

MDD/ADD = 1.46

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April 04 2016

Sudbury Water and Wastewater Master Plan

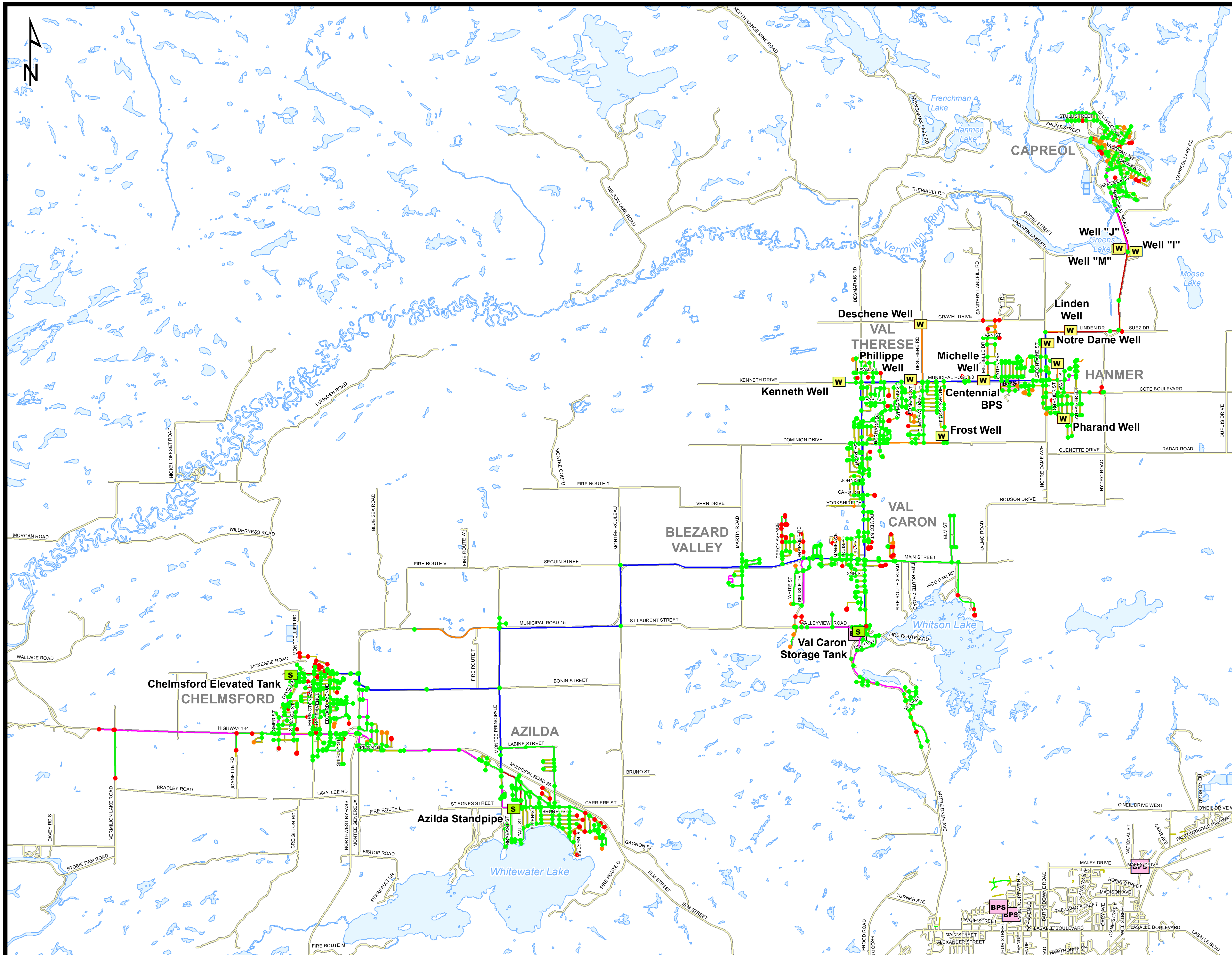
0 1,000 2,000 3,000 4,000 5,000 m

1:100,000

Map 9

Valley Water System - Pipe Headloss and Junction Pressure

Scenario: 2041MDD



Legend

- W — Well
- S — Storage Tank
- BPS — Booster Pumping Station (BPS)
- WTP — Water Treatment Plant (WTP)
- Road
- Waterbody

Junction: Fire Flow (Available) (L/s)

- <= 65 (Res) or 130 (ICI)
- < 75 (Res) or 150 (ICI)
- >= 75 (Res) or 150 (ICI)

Pipe: Diamater (mm)

- <= 150
- <= 200
- <= 250
- <= 300
- <= 400
- <= 600
- > 600

MDD/ADD = 1.46

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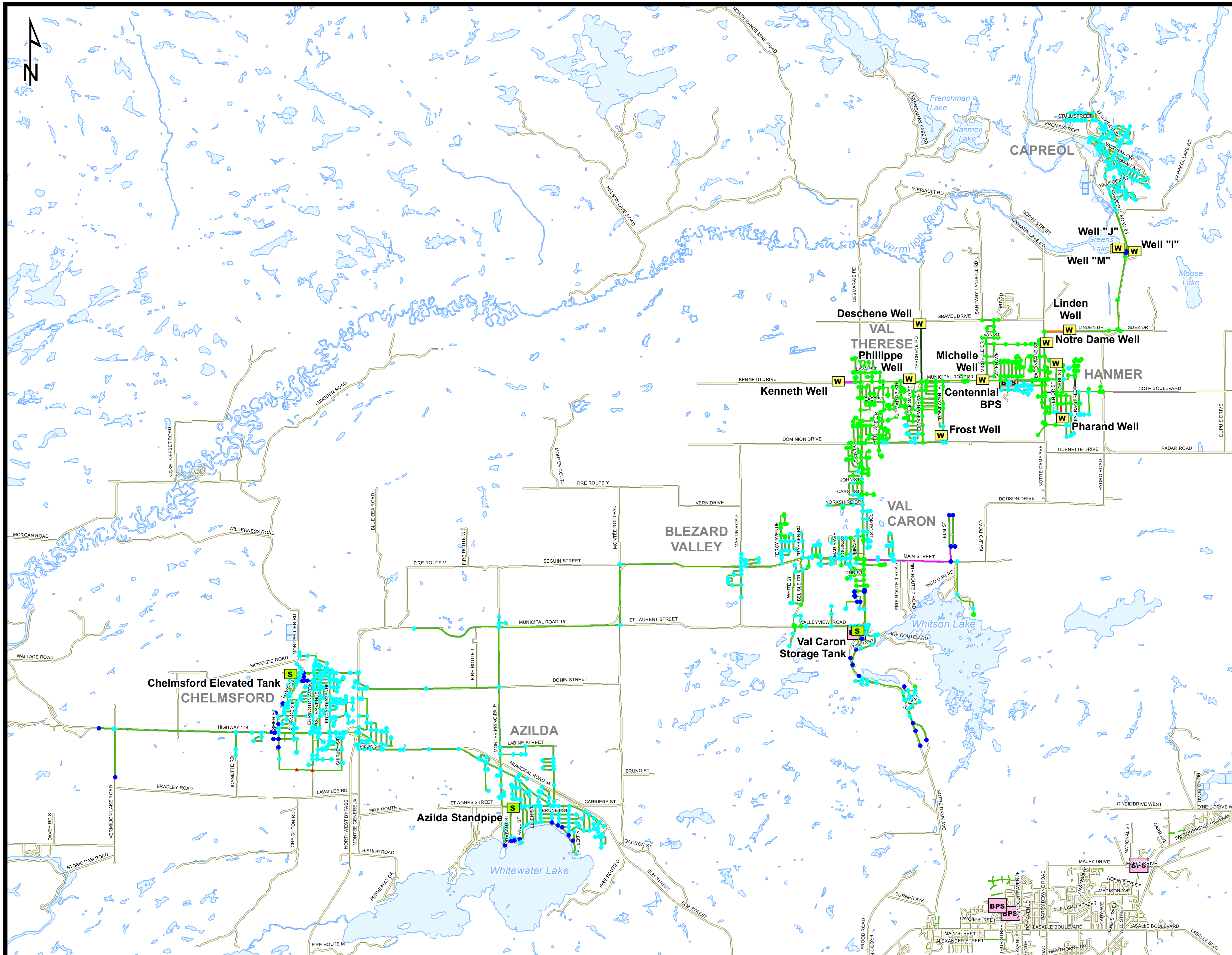
0 1,000 2,000 3,000 4,000 5,000 m

1:100,000

Map 10

Valley Water System - Fire Flow Availability

Scenario: 2041MDD+FF



Legend

- W Well
- S Storage Tank
- BPS Booster Pumping Station (BPS)
- WTP Water Treatment Plant (WTP)
- Road
- Waterbody

Junction: Pressure (psi)

- <= 20
- <= 40
- <= 60
- <= 80
- <= 100
- > 100

Pipe: Headloss Gradient (m/km)

- <= 1.5
- <= 2.0
- <= 3.0
- <= 5.0
- > 5.0

MDD/ADD = 1.46

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April 04 2016

Sudbury Water and Wastewater Master Plan

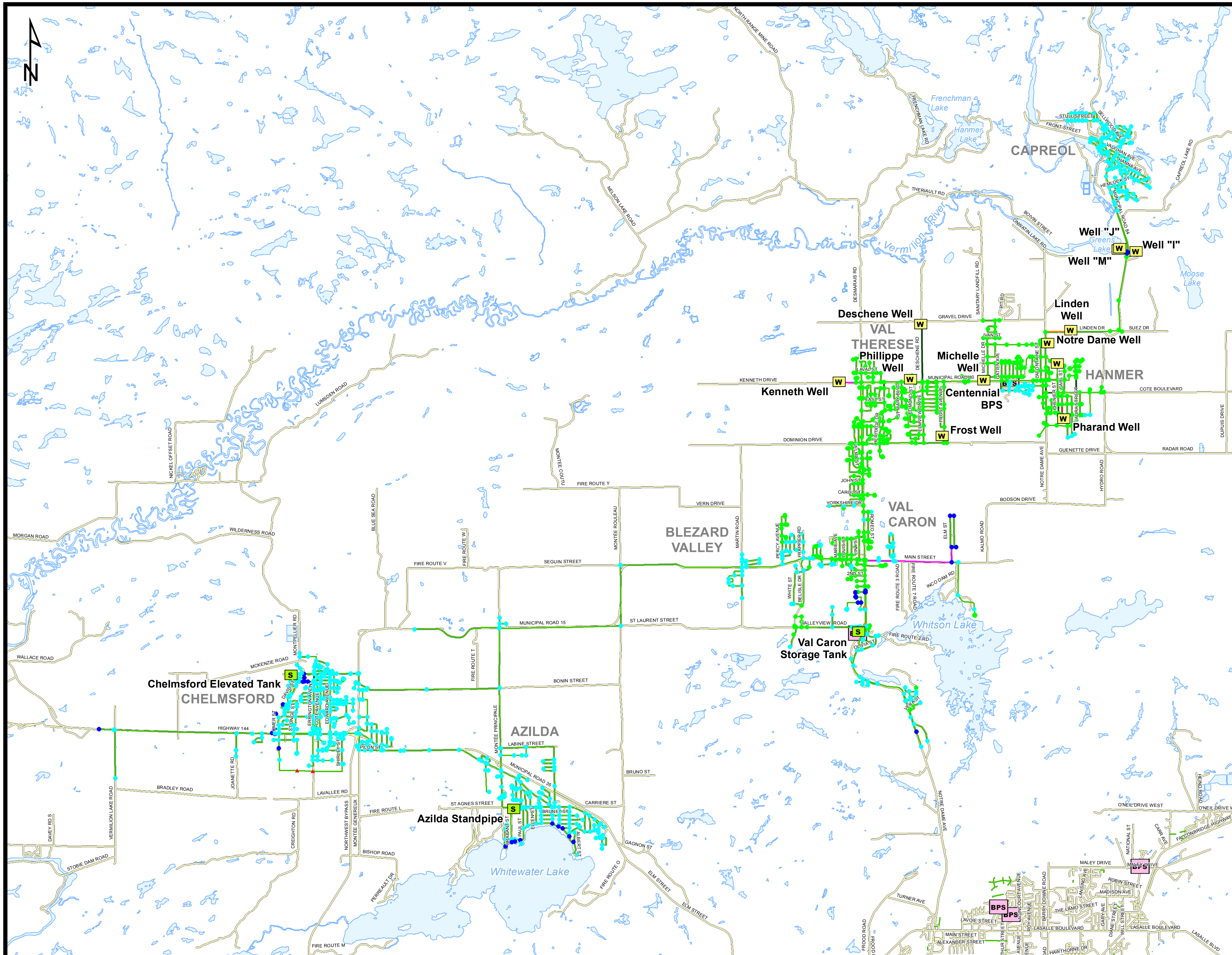
0 1,000 2,000 3,000 4,000 5,000 m

1:100,000

Map 11

Valley Water System - Pipe Headloss and Junction Pressure

Scenario: ULTADD



Legend

- W Well
- S Storage Tank
- BPS Booster Pumping Station (BPS)
- WTP Water Treatment Plant (WTP)
- Road
- Waterbody

Junction: Pressure (psi)

- <= 20
- <= 40
- <= 60
- <= 80
- <= 100
- > 100

Pipe: Headloss Gradient (m/km)

- <= 1.5
- <= 2.0
- <= 3.0
- <= 5.0
- > 5.0

MDD/ADD = 1.46

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Sudbury Water and Wastewater Master Plan

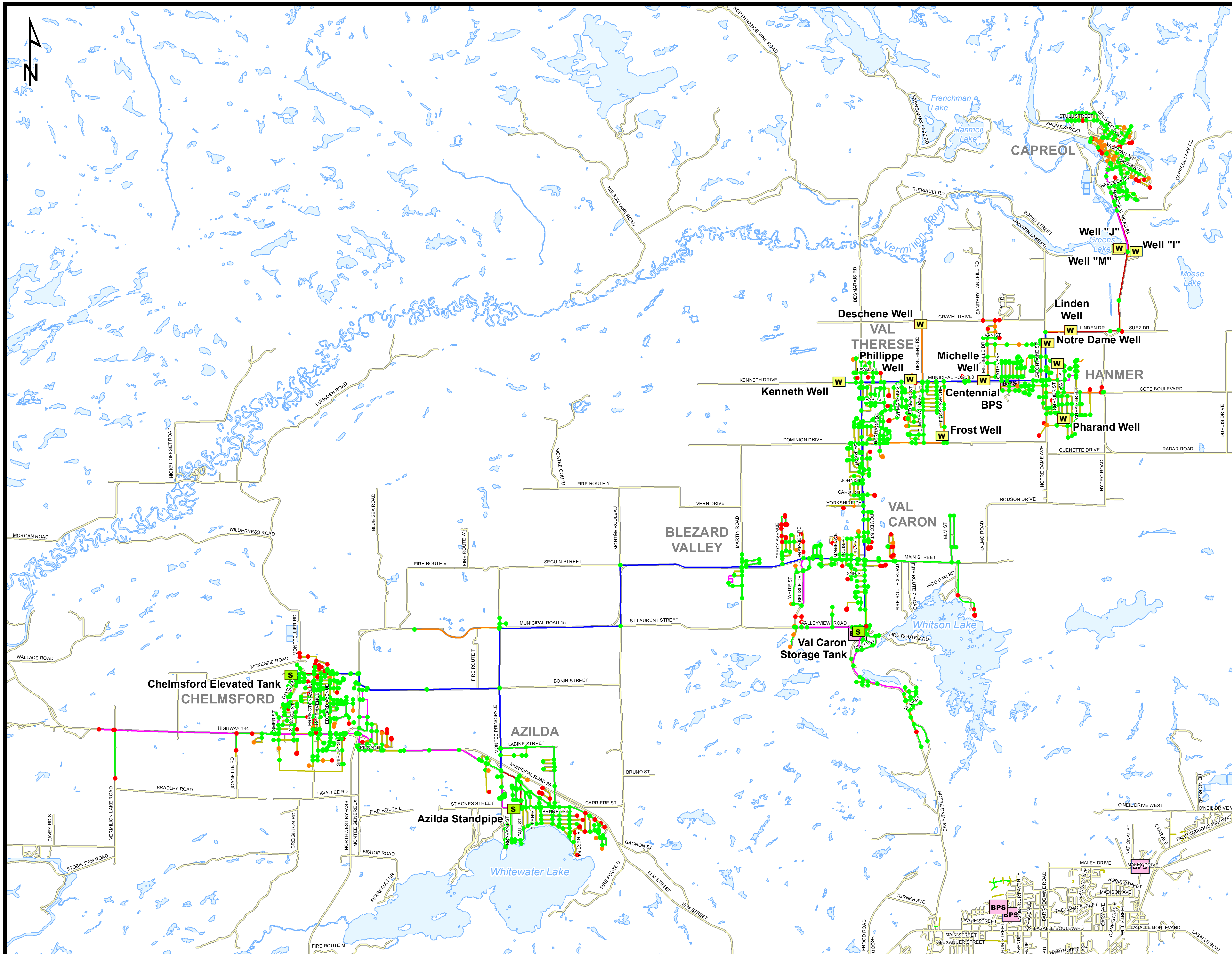
0 1,000 2,000 3,000 4,000 5,000 m

1:100,000

Map 12

Valley Water System - Pipe Headloss and Junction Pressure

Scenario: ULTMDD



Legend

W	Well
S	Storage Tank
BPS	Booster Pumping Station (BPS)
WTP	Water Treatment Plant (WTP)
	Road
	Waterbody

Junction: Fire Flow (Available) (L/s)

- <= 65 (Res) or 130 (ICI)
- < 75 (Res) or 150 (ICI)
- >= 75 (Res) or 150 (ICI)

Pipe: Diamater (mm)

- <= 150
- <= 200
- <= 250
- <= 300
- <= 400
- <= 600
- > 600

MDD/ADD = 1.46

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April 07 2016

Sudbury Water and Wastewater Master Plan

0 1,000 2,000 3,000 4,000 5,000 m

1:100,000

Map 13

Valley Water System - Fire Flow Availability

Scenario: ULTMDD+FF

CITY OF GREATER SUDBURY WATER AND WASTEWATER MASTER PLAN

VERMILION WATER SYSTEM GAP ANALYSIS AND STATUS QUO REPORT

CITY OF GREATER SUDBURY

DRAFT

PROJECT NO.: 121-23026-00
DATE: OCTOBER 2015

WSP
100 COMMERCE VALLEY DRIVE WEST
THORNHILL, ON, CANADA L3T 0A1

TEL.: +1 905 882-1100
FAX: +1 905 882-0055
WSP.COM

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APPENDICES

1 INTRODUCTION

The City of Greater Sudbury (CGS) retained WSP to undertake a Water and Wastewater Master Plan. The purpose of the Master Plan project is to establish servicing strategies for water and wastewater infrastructure for the core urban areas and surrounding communities in the City for the next 20 years, as part of the five-year review of the City's Official Plan. The Master Plan will identify potential projects to address the servicing needs for planned growth within the City. It is being conducted in accordance with the requirements set out in the Municipal Class Environmental Assessment (Class EA) document (June 2000 as amended in 2007 and in 2011).

This report includes a capacity review of the Vermilion existing water system. Based on population growth projections and design criteria discussed in the *Population and Unit Rates Technical Memorandum* (WSP, 2014) water demands and wastewater generation projections were developed and used to determine future infrastructure needs to the 2041 and ultimate buildout planning horizons.

This report assumes that the Vermilion Water System would continue to be a stand-alone system. Any potential interconnections between Vermilion and other systems are not considered as part of this report. Potential interconnections with other communities will be reviewed under separate cover, as part of the Master Plan. It should be noted that the Vermillion WTP is owned by Vale, and the City is in constant communication with Vale regarding water requirements. The City has a long standing and good working relationship with Vale, and expects to maintain it into the future. There is no current indication of a reduction in reliable services. Any change to the relationship would be long term, therefore if any changes to the current agreement should arise, the City would have the required time to plan for an alternate water source.

The conclusions and recommendations provided in this report will be the basis for the problem definition and evaluation of alternatives conducted as part of the Master Plan.

Additional information on the existing water systems is provided in the Baseline Review Reports for Water Systems (WSP, 2014).

2 STUDY AREA

Vermilion is a large community located in the west end of the City of Greater Sudbury. The system is supplied by water from the nearby Vermilion River and distributes it to the greater area of Copper Cliff, Lively, Mikkola, Naughton, and Whitefish.

Maps 1 to 3 in **Appendix A** illustrate the Vermilion study area and identifies future land use and development areas, including vacant residential and industrial, commercial, and institutional (ICI) areas. Additional information on population growth and development phasing is provided in the Unit Rates and Population Projections Technical Memorandum (WSP, 2014).

Existing development in the study area is mixed, and includes residential as well as industrial land uses. Notably, the area's infrastructure is affected by lack of engineering standards when it was constructed. This results in a lack of historical information on watermains.

Based on the City's planning data, the population is expected to increase from 10,359 in 2011 to 12,085 in 2041 to 19,400 by Ultimate Buildout.

ICI growth is expected to be primarily industrial, especially over the 2031-2036 timeframe, with smaller amounts of commercial and institutional development. Growth is discussed further in Section 6.1.

3 OVERVIEW OF EXISTING SYSTEM

3.1 WATER TREATMENT AND DISTRIBUTION

The Vermilion distribution system services Copper Cliff, Lively, Naughton, Mikkola, Whitefish, and Atikameksheng Anishnawbek (previously known as the Whitefish Lake First Nations Reserve). The system receives water from the Vermilion Water Treatment Plant, which is owned and operated by a third party, Vale Limited (Vale). Vale has several mining operations throughout Greater Sudbury and owns and operates this plant to supply its own operations, as well as supplying the City for municipal purposes. The Vermilion WTP complies with all MOECC drinking water quality standards and requirements, and as such, possesses a drinking water works permit, a municipal drinking water licence, and an Operational Plan.

The Vermilion distribution system consists of a network of watermains owned by the City, as well as others owned by Vale. The overall network is interconnected and would be difficult to separate, though as mentioned, the City's long standing relationship with Vale limits the risk to the City regarding facility ownership.

The Vermilion system also consists of a Standpipe on Magill St., called the Walden Standpipe, which is combined with the Walden Metering Chamber at 229 Magill St. to control the tank.

Limited water data is available for the Vermilion system. Until 2015, a designated flow meter was not yet installed to calculate the demand entering the municipal system and the City used customer billing volumes to estimate the average demand. However, a metering chamber complete with a pressure reducing valve and electromagnetic flow meter was commissioned in early 2015. The metering chamber will allow the City to measure flows entering the City's portion of the distribution system.

Appendix C provides an overview of the system.

3.1.1 VERMILION WATER TREATMENT PLANT

The Vermilion WTP, owned and operated by Vale, has a peak capacity of 81,800 m^3/d. The raw surface water comes from the nearby Vermilion River. The Plant uses a conventional treatment process. City operations staff estimate that the City purchases approximately 20% - 30% of the Vermilion WTP annual production.

3.1.2 DISTRIBUTION SYSTEM

The system includes infrastructure owned by both the City and Vale. The City owns:

— 4,732 m^3 Walden Standpipe (Effective Storage Volume of 2,662 m^3)

— A number of watermains

Through discussions with City staff, it is understood that much of the City-owned infrastructure was grandfathered into the municipal system and information such as material and age of construction, as well as existing condition is not available. Staff has also noted that the air release valves on the trunk watermains are aged and require maintenance or replacement.

The following infrastructure is owned by Vale:

— 60,543 m^3 Copper Cliff water storage tank

— Cobalt Booster Pumping Station (BPS)

— C.C. North Mine BPS

— Clarabelle North Mine BPS

— A number of watermains

Using the MOECC A+B+C storage calculation, the total current storage volume required for the Vermilion Water System is approximately 3,760 m^3. The detailed calculation and assumptions are provided in **Appendix B**. This is significantly more than the available effective storage volume owned by the City, 2,662 m^3. However, it is important to note that Vale's 60,543 m^3 Copper Cliff water storage tank could provide some redundant supply in case of an emergency, but the volume dedicated for municipal use cannot be confirmed.

3.1.3 KNOWN CHALLENGES

In addition to concerns discussed in previous sections, the Vermilion Water System has the following known challenges:

— Some of the existing linear infrastructure in the Vermilion water system is undersized since it was not subject to City engineering design standards when it was implemented. As a result of the existing undersized watermains, the system experiences high pressure gradients which result in frequent watermain breaks.

— Watermains in the northern section of Copper Cliff are difficult to access. These watermains were implemented without the use of infrastructure standards.

— Although Vale also has available water storage capacity, the amount dedicated to service municipal users is not defined.

4 HISTORICAL FLOWS AND REVIEW OF OPERATIONAL DATA

Historical water supply data from the Vermilion WTP was available; however, an evaluation of the Maximum Day and Peak Hour factors could not be completed since the plant supplies water to both residential and industrial development in the area and the portion used by the City cannot be determined.

Table 4-1 provides a summary of the data received, and indicates how it was used for the analysis.

Table 4-1 Water Supply Data Reviewed

DATA RECEIVED	PARAMETERS INCLUDED	DATA INTERVAL	USE IN ANALYSIS
Annual Billing Data (2012)	Annual flow per customer in m^3	Annually	To determine the proportion of total water consumption corresponding to residential users

4.1 FLOW DATA

Municipal demand data for the Vermilion system was not available and so peaking factors from the MOECC Design Guidelines for Drinking Water Systems (2008) were used.

4.2 RAW WATER CHARACTERISTICS AND SECURITY OF SUPPLY

Source water protection studies and water budgets have been completed for the watersheds for the Sudbury water facilities, and most recently updated in September 2014. A water budget is a tool to identify the sources of water input to and output from a watershed or water system. They are used to characterize the pathways of water movement through a watershed and help understand water quantity issues, as well as water quality issues.

Through the Tier 1 assessment, the Vermilion River watershed was determined to have a low risk of threats to water quantity. As such, the study for this subwatershed was completed at Tier 1, so water quality threats were not reviewed (water quality is reviewed under Tier 3).

4.2.1 OPERATIONAL DATA

Data reported in the *Annual Reports* for the Sudbury supply network includes effluent chlorine residual, and trihalomethanes (THMs). Since the City does not operate the Vermilion WTP, there is no data available for water testing at the plant. Data was reviewed for 2012 to determine any historical issues within the water distribution network. No exceedances were identified.

5 DESIGN CRITERIA

The following design criteria were used to assess the remaining capacity of the existing systems and to forecast future requirements for the water systems. The unit rates used to estimate future water flows correspond to the values included in the *Population Projections and Unit Rates Technical Memorandum* (WSP, 2014). Otherwise, design criteria recommended in the *MOECC Guidelines* and *City's Engineering Design Manual* were used.

5.1 UNIT WATER DEMAND CRITERIA

The water demand criteria shown in Table 5-1 are from the unit rates recommended in the *Populations and Unit Rates Technical Memorandum* (WSP, 2014) and the *Walden Industrial Park Projected Flows* memorandum (J.L. Richards, 2012). The rates were reviewed against historical data, MOECC *Guidelines* (MOE, 2008), and current standards in the City's *Engineering Design Manual* (City of Greater Sudbury, 2012).

Both the *MOECC Guidelines* and *City Engineering Design Manual* recommend determining demands for institutional, commercial and industrial (ICI) users on a case by case basis. However, the following criteria for ICI demands were used for the purposes of this evaluation.

Table 5-1 Vermilion Water System Design Criteria

CRITERIA	VALUE	REFERENCE
Average Day Residential Demand	250 L/cap/day	MOECC Guidelines
Average Day Institutional & Commercial Demand	28 m^3/ha/d	MOECC Guidelines
Average Industrial Demand	35 m^3/ha/d	MOECC Guidelines
Average Industrial Demand (20% of Walden Industrial Park)	35 m^3/ha/d	Per Methodology in the Lively/Walden Environmental Summary Report (J.L. Richards & Associates Limited, 2013)
Average Industrial Demand (80% of Walden Industrial Park)	3 m^3/ha/d	Per Methodology in the Lively/Walden Environmental Summary Report (J.L. Richards & Associates Limited, 2013)
Average Industrial Demand (Existing Industrial Development in the Walden Industrial Park that is currently not serviced through the City's water supply)	3 m^3/ha/d	Per Methodology in the Lively/Walden Environmental Summary Report (J.L. Richards & Associates Limited, 2013)
Domestic Demand Maximum Day Factor	1.90	MOECC Guidelines
Domestic Demand Peak Hour Factor	2.85	MOECC Guidelines

Notes: 1) Production data was not available for the Vermilion water system. It is recommended that the City meter water entering the Vermilion system in order to better monitor consumption and adjust rates in the future, if needed.
2) The average day water consumption rate was multiplied by the unbilled water rate of 25.8% since the average per capita water consumption was based on billed water rates only.
3) The methodology used by J.L. Richards & Associates in their 2013 ESR for the Lively/Walden EA to calculate projected wastewater flows for the Walden Industrial Park were also used in the Sudbury Water and Wastewater Master Plan. The approach included using a rate of 3 m^3/ha/d for 80% of future developable lands and 35 m^3/ha/d for 20% of future developable lands, based on their analysis of the existing wastewater generation in the Park and the type of industrial development expected to be implemented in the future.

Residential average day demands are obtained by multiplying the residential unit rate by the service population. In the case of the Vermilion system, the unbilled water rate was also factored into the calculation of residential average day water consumption since the per capita water rates were based solely on billed water usage data. Similarly, average ICI demands are obtained by multiplying the corresponding unit rates to the areas of development, assuming 100% of the area would be developed.

Maximum day and peak hour demands are obtained by multiplying the average day demand by the corresponding peaking factor.

For purposes of this study, and in line with City standards and practices, a residential fire flow of 75 L/s over 1.75 hours and ICI fire flow of 150L/s over 2 hours were used.

5.2 DESIGN CRITERIA FOR WATER SYSTEM COMPONENTS AND OPERATION

5.2.1 TREATMENT CAPACITY

Water supply facilities are designed to supply the maximum day demands of the system.

Treatment facilities must be designed in accordance with the *Procedure for Disinfection of Drinking Water in Ontario* (Ontario, 2006). Drinking water treatment systems that obtain water from a surface water or GUDI well supply must achieve an overall performance providing as a minimum a 2-log (99%) removal or inactivation of *Cryptosporidium* oocysts, 3-log (99.9%) removal or inactivation of *Giardia* cysts, and 4-log (99.99%) removal or inactivation of viruses.

At least 0.5-log removal or inactivation of *Giardia* cysts and 2-log removal or inactivation of viruses must be provided through disinfection, while the remaining removal may be achieved through filtration or other equivalent treatment processes.

5.2.2 PUMPING CAPACITY

Pumping stations are rated based on their firm capacity. If sufficient floating storage is available in a particular pressure district, the MOECC defines firm capacity as the capacity of the station with the largest pump out of service. If there is insufficient or no floating storage, firm capacity is defined as the capacity with the two (2) largest pumps out of service (MOE, 2008).

For each pressure district, the pumping stations have to be designed to provide peak hour or maximum day plus fire demands (whichever are greater), if no floating storage is available. If sufficient floating storage is available, then the pumping station only needs to be designed to provide maximum day demands.

5.2.3 STORAGE CAPACITY

Storage requirements are based on the requirement to meet water demands that exceed the capacity of the treatment plant and to satisfy fire flow demands. When the capacity of the supply system is only capable of satisfying maximum day demands, storage requirements are determined using the following formula from the *MOECC Guidelines* (MOE, 2008):

$$\mathbf{Storage = A + B + C}$$

Where: A = Fire Storage, B = Equalization Storage = 25% of maximum day demand, and C = emergency storage = 25% of (A+B).

Fire storage is the product of the maximum fire flow required in the system and the corresponding fire duration based on Fire Underwriters requirements (Fire Underwriters Survey, 1999).

When the system can supply more than just the maximum day demand (but less than the peak demand), the fire storage requirements can be determined using the following formula:

$$\mathbf{A = (Peak\ Demand - Pumping\ Station\ Firm\ Capacity) \times Fire\ Duration}$$

Where: peak demand is the greater of the peak hour demand and the maximum day plus fire demand.

Per *MOECC Guidelines*, floating storage should be designed such that the elevation of the equalization volume (B) is such that a minimum pressure of 275 kPa (40 psi) can be maintained in the system under peak hour flow conditions. The fire (A) and emergency (C) volumes should be at elevations that produce 275 kPa (40 psi) during peak hour demand conditions, and 140 kPa (20 psi) under the maximum day plus fire flow condition (MOE, 2008).

5.2.4 DISTRIBUTION CAPACITY

Watermains have to be sized to carry the greater of the maximum day plus fire flow or peak hour demand. The range of acceptable pressures under normal conditions (average to peak hour flows) is 275 kPa (40 psi) to 690 kPa (100 psi), while during fire flow conditions pressures may drop to 140 kPa (20 psi) (MOE, 2008). The maximum allowable water velocity in the distribution system is 3 m/s (MOE, 2008).

6 FUTURE REQUIREMENTS

6.1 POPULATION PROJECTIONS

As part of the City of Greater Sudbury Master Plan, population forecasts were developed for the 2016, 2021, 2026, 2031, 2036, 2041 and Ultimate Buildout planning years. Ultimate Buildout is defined as an estimate of what the demand from the total population and total number of households in the City of Greater Sudbury would be based on lands that are currently designated for development in the Official Plan within the existing settlement boundaries.

The City supplied planning data sheets with properties and development potential and the vacant residential and ICI land inventory, and Hemson Consultants, on behalf of the City, provided supplementary population projections. Data was provided for each water and wastewater system boundary. These data were used in conjunction to develop the targeted population growth for each horizon year, as well as development phasing (discussed in the next section and in detail in the *Populations and Unit Rates Technical Memorandum*, WSP 2014).

In cases where the City's planning data sheets and Hemson's population projections forecasted fewer development units than the vacant land inventory for an area, then specific parcels (up to the City's and Hemson's unit projections) of developable units were selected. These parcels were selected based on the rationale provided in the City's Official Plan. That is, the Official Plan prioritizes that development take place in areas that are currently serviced, or where servicing can easily be extended. This focuses growth in existing urban areas until supply is no longer available in these areas.

Based on the planning data, the population serviced by the Vermilion Water System is projected to increase by 1,726 people in 2041 and 9,041 by Ultimate Buildout. The population projections to be used in the Master Plan are summarized in Table 6-1.

Table 6-1 Vermilion Water System Population Projections

	2011	2016	2021	2026	2031	2036	2041	ULTIMATE BUILDOUT
Vermilion	10,359	10,845	11,303	11,686	11,912	12,050	12,085	19,400

The City's planning data does not specify target years for employment growth. However, vacant lands designated as institutional, commercial, and industrial (ICI) properties have been assigned to different stages of the development process by the City. These stages are described below.

- Draft Approved:
  - These are lands that have draft plan of subdivision approval under the Planning Act or have pending applications with the City. Typically, these lands are close to registration or few years away from development as the required conditions are satisfied
  - Development approvals are near complete, and development could take place at any time. Properties with this designation were set to take place in 2016.
- Legal Lots of Record:
  - These are existing lots, including lots in a registered plan of subdivision. Typically these lands are zoned, serviceable and only require building permit approval for development. In some cases a site plan approval/agreement may also be required.
  - Based on historical trends, development is approximately 15 years away from receiving draft approval. Properties with these designations were assigned to take place in 2026.
- Designated Developable:

- These lands do not have any development approvals in place but are understood to be areas of future development as they are within the settlement boundary. Designated lands are typically a number of years away from being developed.
- Based on historical trends, these properties are approximately 10 years away from receiving Legal Lot of Record designation. Designated Developable properties were assumed to take place in 2036.

These land supply categories stem from the land supply requirements that municipalities must maintain under Section 1.4 of the Provincial Policy Statement. In this context, Designated Development Lands would count towards Section 1.4.1 (a) and Legal Lots of record and Draft Approved Lands would count towards 1.4.1 (b). It is also important to note that the total supply is governed by PPS Section 1.1.2.

The targeted ICI development areas for each horizon year are summarized in Table 6-2.

Table 6-2 Vermilion Water System ICI Projections

	ICI DEVELOPMENT AREAS (HA)					
LAND USE	2016	2021	2026	2031	2036	2041
Institutional	0.000	0.000	1.120	0.000	0.850	0.000
Commercial	0.000	0.000	1.700	0.000	8.200	0.000
Industrial	0.000	0.000	74.40	100.0[1]	18.93	0.000
Total	**0.000**	**0.000**	**77.22**	**100.0[1]**	**27.98**	**0.000**

**The 100 ha of industrial lands developing by 2031 is current development that is not serviced by the City's water system (per the development areas used in the Walden Industrial Park Projected Flows memorandum (J.L. Richards, 2012)).*

The above assumptions provide an estimate as to the ICI development time line. In reality, development may be more staggered. However, for purposes of infrastructure planning and to ensure that the appropriate infrastructure is in place by the appropriate planning horizon, the above assumptions are considered to be conservative.

6.2 PRIORITY EXTENSION LIST

The City has developed and maintained a Priority Extension List of existing residential and ICI streets that are not currently serviced by either or both municipal water or sewer, but at least one owner on the street has requested servicing. The City's policy on extension of services includes the following conditions:

- Before any project proceeds, the participation rate of benefitting property owners must be 100%, with those benefitting property owners funding 50% of the actual net cost of the project.
- The process must be initiated by property owners submitting a petition to the City of Greater Sudbury.
- At least 80% of the property owners in the project area must sign the petition.
- The project must be on the City's priority list for new servicing schemes, or, there must be demonstrated cause why the project should be included on the City's priority list for new servicing schemes.

In Vermilion, one street has been placed on the priority list for sewer servicing. However, to date, the above conditions have not been met and City funding for extension requests is not available. Therefore, this street has not been included in the demand projections for infrastructure planning as part of the Master Plan.

6.3 PHASING OF FUTURE GROWTH

Growth areas were allocated based on population projections for individual developments and the overall target growth population projections for the Vermilion Water System service area for the horizon years.

Hemson's supplementary tables were used to provide the target population, while the City's planning tables and vacant lot inventory were used to identify phasing of specific properties, and assignment of draft approved, legal lots of record, and designated development properties. In general, priority was given to draft approved properties, followed by legal lots of record and designated developable properties. In accordance with the Official Plan, the City has also assigned a target quantity of legal lots of record and designated developable properties to be developed in each horizon year. That is, legal lots of record should account for approximately 20% of all household growth, and designated developable lots are assigned 20% of the 20 year anticipated growth.

The future growth phasing plans were presented in the *Unit Rates and Population Projections Technical* Memorandum (WSP, 2014).

6.4 FUTURE WATER DEMAND PROJECTIONS AND INFRASTRUCTURE NEEDS

The unit flow criteria listed in Section 5.1 were used to estimate the future water demands in the Vermilion Water System. In general, the projected flows were calculated by the following formula:

$$\textbf{Projected Average Day Demand} = \textbf{Base Demand} + \textbf{Additional Residential Demand} + \textbf{Additional ICI Demand}$$

The demands corresponding to the population growth forecasts to ultimate buildout are listed in Table 6-3.

Table 6-3 Water Demand Projections for the Vermilion Water System

YEAR	POPULATION	AVERAGE DAY DEMAND (M3/D)	MAXIMUM DAY DEMAND (M3/D)	PEAK HOUR DEMAND (M3/D)
Base	10,359	4,059	7,712	11,569
2016	10,845	4,212	8,003	12,004
2021	11,303	4,356	8,276	12,414
2026	11,686	5,315	10,098	15,148
2031	11,912	5,686	10,804	16,206
2036	12,050	6,646	12,627	18,941
2041	12,085	6,657	12,648	18,972
Ultimate Buildout	19,400	8,957	17,019	25,529

The Base Demand was the average historical (2009 to 2013) average day, maximum day, and peak hour demand for the community. The additional residential demand was calculated using the unit flow rate multiplied by the population growth, and similarly, the ICI demand was calculated using the unit flow rate for each type of development (industrial, commercial or institutional), multiplied by the growth in development area.

Maximum day and peak hour demands were calculated by applying the respective peaking factor to the average day demand.

A desktop analysis of historical water demands and future water demand projections is included in **Appendix B**.

6.4.1 SUPPLY CAPACITY

The rated capacity for the Vermilion Water plant is 81,800 m^3. Details on the pumping capacities for the plant are unknown since the City does not own or operate the plant. The projected maximum day demands are plotted versus the total rated and firm production system capacities in Figure 6-1.

Figure 6-1 includes an assumption for the volume of water utilized by Vale. The industrial water consumption is based on Vale's water consumption data from 2013. Therefore the curve that includes Vale's water projections should be interpreted with caution since additional coordination with Vale is required to understand their future water requirements – this curve assumes their 2013 consumption will remain constant. That said, based on Vale's water consumption remaining constant, there wouldn't be a need for an expansion to the plant since it could supply flows to meet Ultimate Buildout demands.

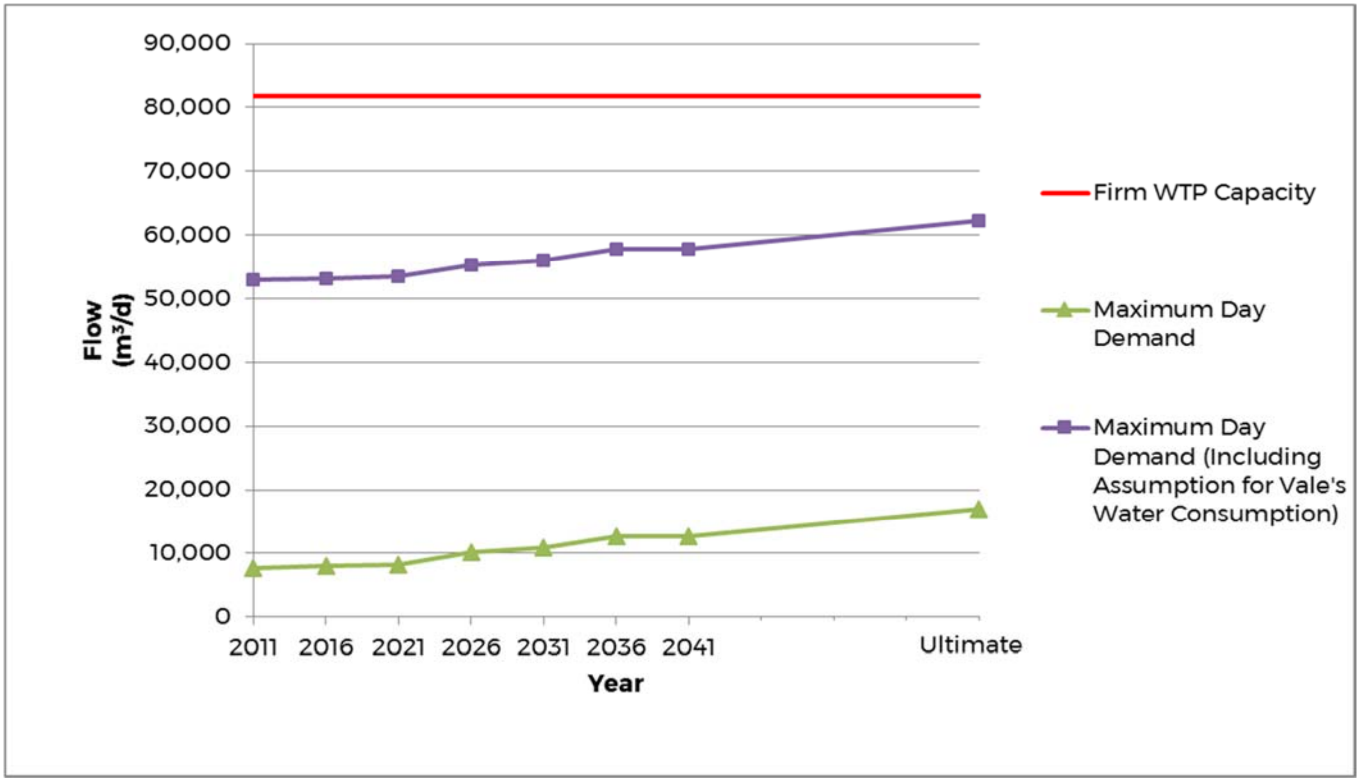

Figure 6-1 Water Demand Projections Compared to Firm Capacity

6.4.2 STORAGE CAPACITY

Storage in the distribution system is provided by the Walden (City owned) and Copper Cliff Water Storage (Vale owned) Tanks.

Applying the formula to determine storage requirements indicated previously, the corresponding fire storage requirement would be 1,080 m^3. Using the maximum day demand required to service growth to 2041 (12,648 m^3/d), the corresponding equalization storage requirement would be 3,162 m^3 and the emergency storage would be 1,061 m^3. The total required storage to service growth to 2041 would be 5,303 m^3.

The total required storage to service the Ultimate Buildout growth scenario would be 6,668 m^3.

Therefore, the existing total storage volume of 2,662 m^3 may not provide sufficient storage capacity for the Vermilion Water System.

The amount of storage required for each horizon year is shown in Figure 6-2.

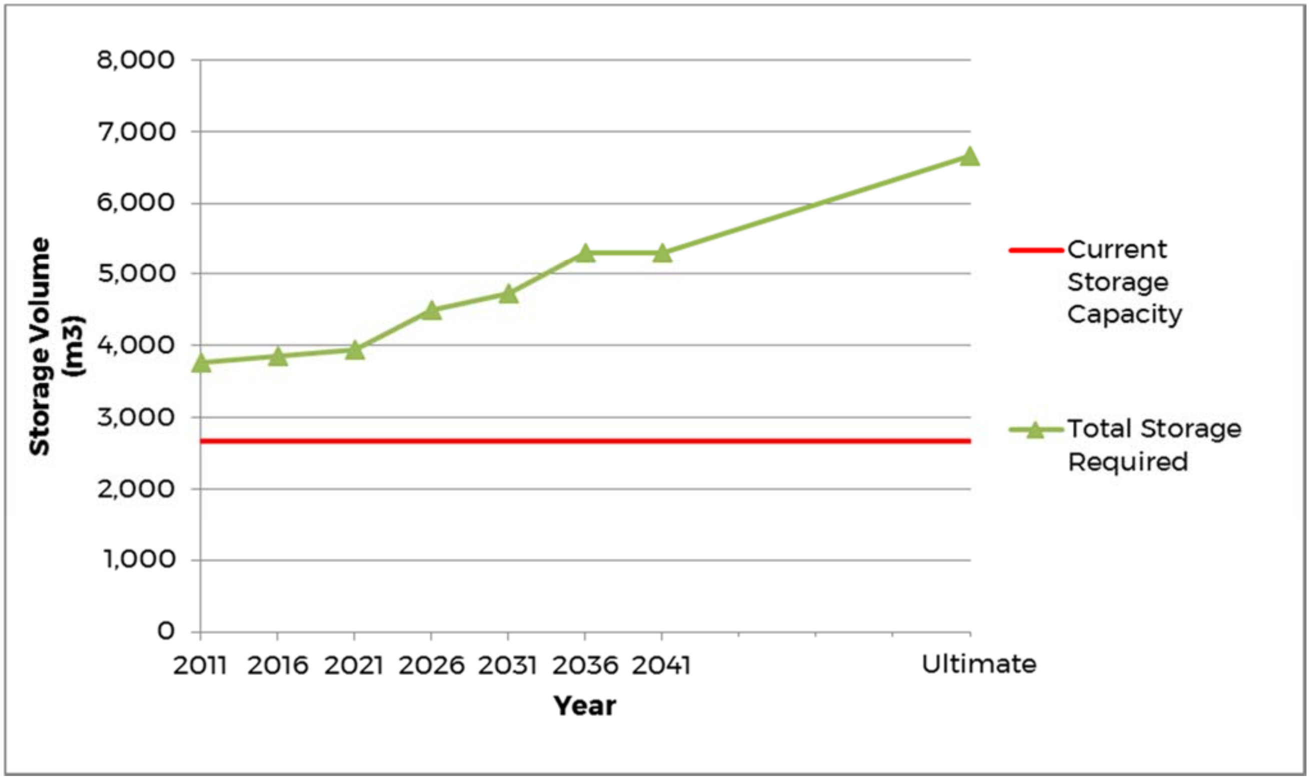

Figure 6-2 Available Storage Capacity Compared to Future Needs

6.4.3 DISTRIBUTION NETWORK

The water model was used to identify system elements (i.e. watermains, pumps, storage tank) for which the capacity was exceeded by the projected water demands. The capacity of the system was assessed in terms of the available fire flows and system pressures.

For each planning scenario, watermains of the modelled network were reviewed to assess whether the required minimum fire flows (75 L/s in residential areas or 150 L/s in ICI areas) and pressures (over 20 psi under fire conditions and over 40 psi under normal conditions) were achieved. Furthermore, some new watermains were added to service greenfield areas where development was planned. A simplified watermain layout was assumed for these areas.

Future populations and demands were loaded into the model based on the planning data and flow projections discussed in earlier in Section 6.3. In general, development in Vermilion might deviate from the proposed phasing scheme. Thus, it is recommended that the hydraulic water model be updated whenever a development application is submitted.

The findings from the water modeling are discussed in Section 7.2 and presented in **Appendix C**.

7 HYDRAULIC WATER SYSTEM MODELING

An all-pipe model of the system including pipes, hydrants, storage tanks and system source was developed by the City using Bentley Systems' WaterGEMS hydraulic modeling software. This model was updated based on information provided by the City to reflect current system conditions.

The water model allows for simulations to be conducted that can be used to predict system responses to events under a wide range of conditions. Using simulations, problems can be anticipated in proposed or existing systems, and solutions can be evaluated before time, money, and materials are invested in a real-world project. Simulations can either be steady-state or extended-period. Steady-state simulations represent a snapshot in time and are used to determine the operating behaviour of a system under static conditions. This type of analysis can be useful in determining the short-term effect of fire flows or average demand conditions on the system. Extended period simulations (EPS) are used to evaluate system performance over time. This type of analysis allows modeling the filling and emptying of storage facilities, regulating valves opening and closing, and pressures and flow rates changing throughout the system in response to varying demand conditions and automatic control strategies.

Simulations including steady-state analysis of the Average Day, Maximum Day and Maximum Day + Fire conditions were carried out using the model. Fire flow simulations were carried out throughout the system to determine whether the system could deliver fire flows under the Maximum Day demands.

7.1 WATER MODEL DEVELOPMENT

To model the current scenario, the following steps were taken:

- Total network demand on an average day basis was determined for the current scenario using 2012 water production data.
- The node demand allocations assigned in the model were based on 2012 meter records. Metered flows were assigned to the respective property. In cases where meter records showed zero flow, the value was manually adjusted to reflect a reasonable volume for a respective property, depending on land use.
- The maximum day peaking factor was applied to the average day demand value to determine the maximum day demand.
- The maximum day demand plus fire flow was used to assess the system since it was greater than the peak hour demand.

7.2 MODELING FINDINGS

7.2.1 FIREFIGHTING CAPACITY

An assessment of the available fire flows was conducted using the hydraulic model. As noted above, a fire flow requirement of 150 L/s was estimated for ICI areas, while a value of 75 L/s was adopted for residential areas.

The model revealed that, in many areas of Vermilion, the watermains are 150 mm diameter or smaller and were sized in accordance with previous standards. Therefore, such watermains may not have capacity to deliver fire flows that meet current standards. Similarly, areas with dead end watermains could not deliver flows that meet current fire flow standards. This gap is most prevalent in the system's extremities – in Whitefish and Copper Cliff.

Water model outputs, including maps showing fire flow analysis, are provided in **Appendix C.**

7.2.2 MODELED HYDRAULIC CAPACITY UNDER NORMAL CONDITIONS

Based on the system modeling, service pressures throughout the system under the maximum day demand scenario generally range between 40 and 100 psi for all scenarios, with one exception. High pressures, greater than 100 psi, are noted along Municipal Road 24 near Hill Road in all growth scenarios, under average day demand and maximum day demand scenarios. These high pressures result due to the low elevation of this area. Apart from this exception, flows throughout the system are within the range prescribed in the MOECC Guidelines (40 to 100 psi) under normal conditions.

Maps showing pressures at nodes are presented in **Appendix C**.

8 CONCLUSIONS AND RECOMMENDATIONS

An assessment of the Vermilion Water Systems was completed to identify infrastructure investment requirements to service forecasted growth in the community. The assessment involved a review of previous studies, an analysis of operations and flow data from the water facilities, and an evaluation of the capacity of the system.

The conclusions of the assessment are summarized below.

8.1 WATER SYSTEM

— Based on the estimated firm capacity of the Vermilion WTP as well as historical and projected maximum day demands, additional water supply will not be required to service growth up to Ultimate Buildout. This conclusion is based on Vale's water consumption not increasing in future. Coordination with Vale will be required to confirm future industrial water usage and determine if additional water supply is required in the system and the timing requirements for the additional water supply.

— There is not enough storage capacity in the system to service the current or future system. The current (2011) storage deficit is 1.1 ML, growing to 9.2 ML to service growth to Ultimate Buildout.

— The model revealed that flows do not meet current fire flow standards in several areas of Vermilion and mostly in the system's extremities, and is attributed mainly to small diameter watermains (150 mm or smaller) and dead end watermains. 150 mm diameter and smaller watermains typically do not have capacity to deliver fire flows.

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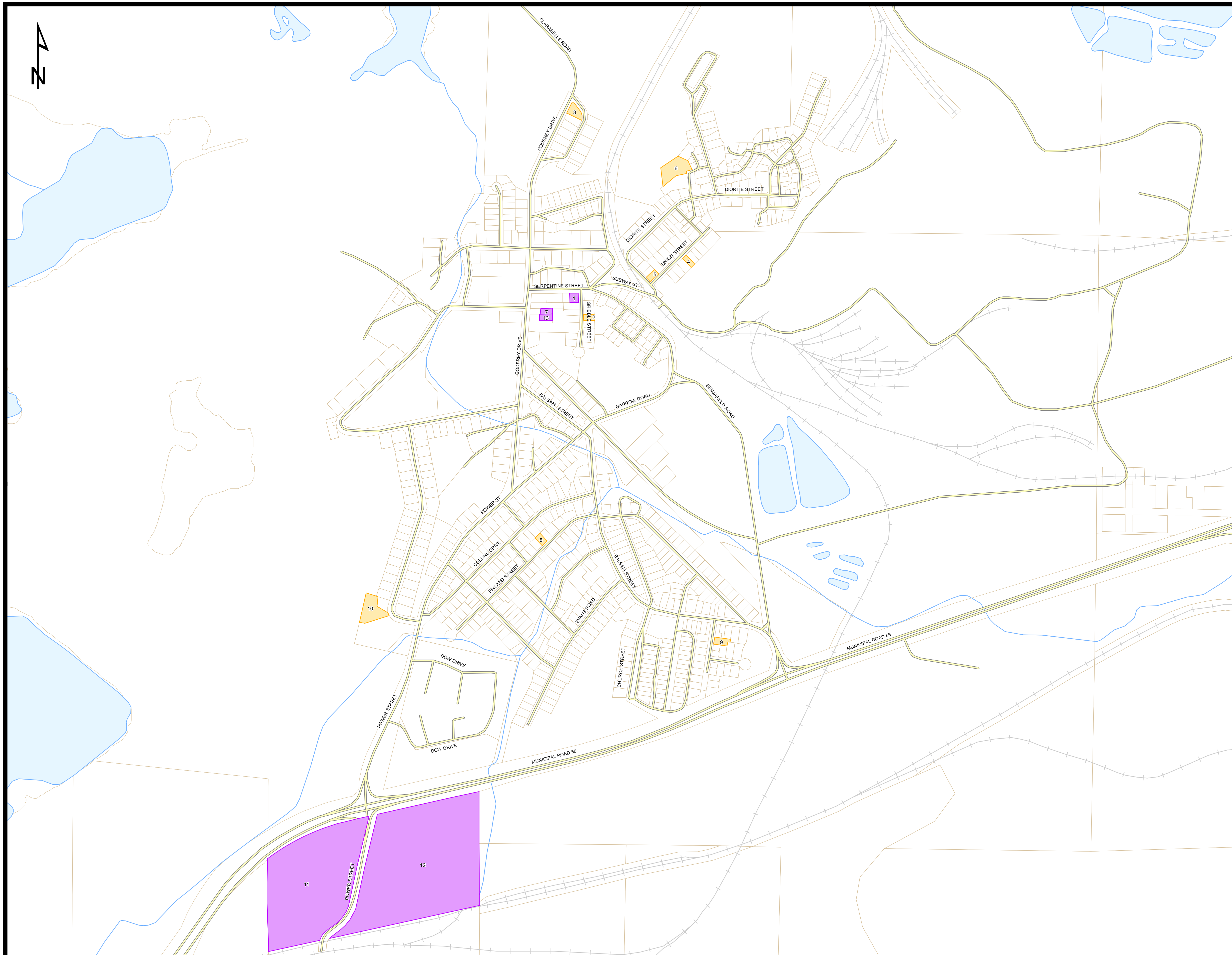
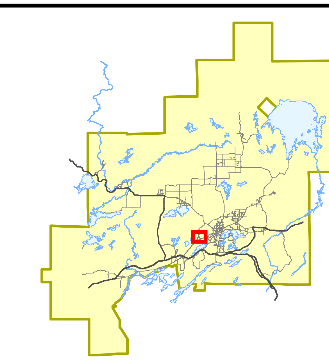
APPENDIX

A RESIDENTIAL AND ICI DEVELOPMENT AREAS



Legend

- Road
- Railway
- Watercourse
- Waterbody
- Parcel
- Vacant ICI
- Vacant Residential



Sudbury Greater Grand | WSP

Project No. 121-23026-00

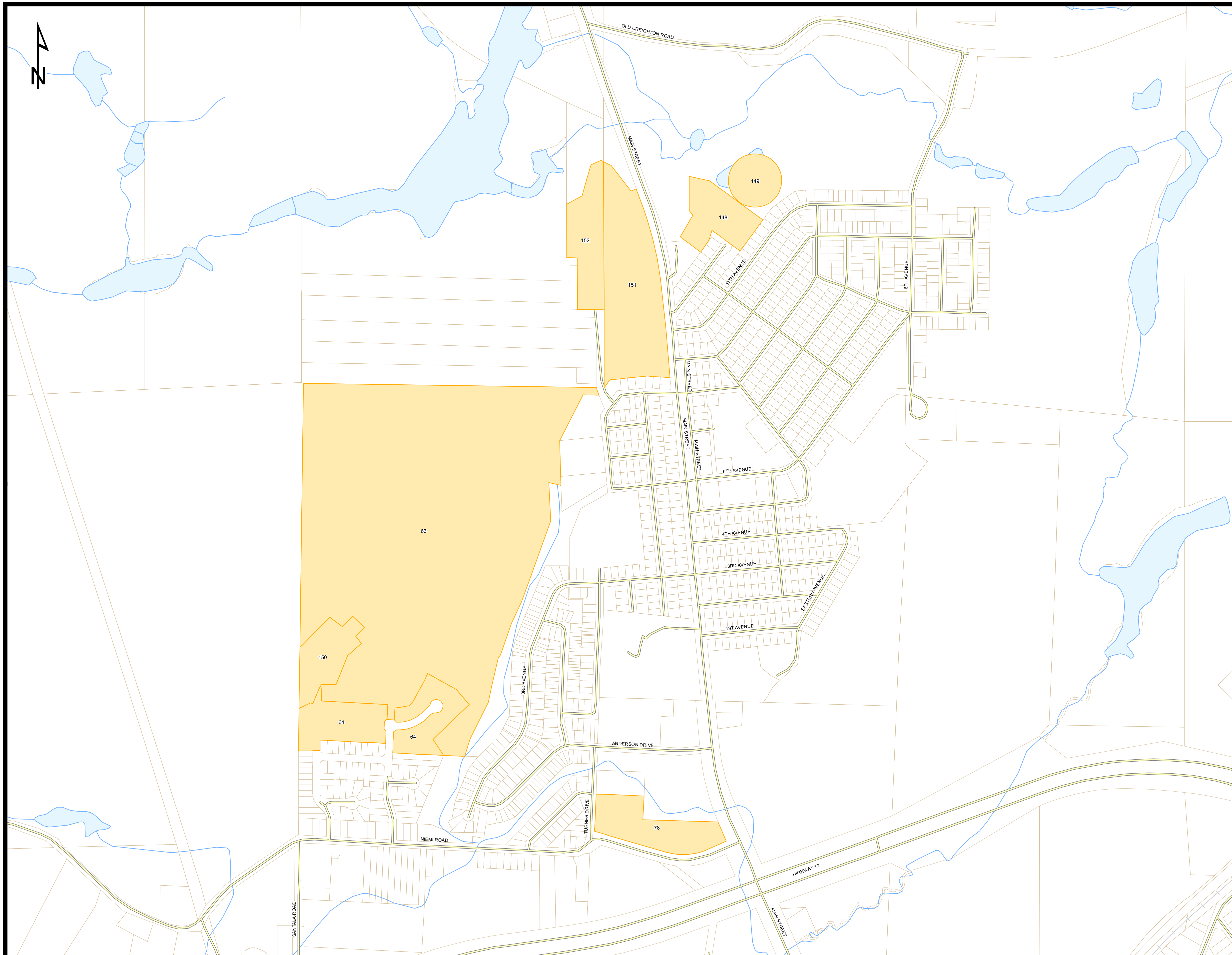
Sudbury Water and Wastewater Master Plan

0 100 200 300 400 500 m

1:10,000

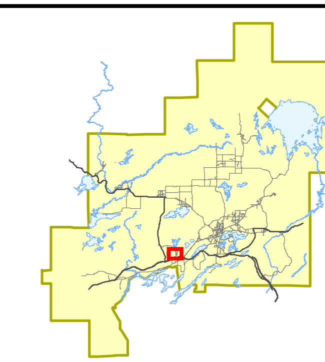
Map 1

Copper Cliff
2011 - Ultimate Buildout Residential and ICI Development Areas



Legend

- Road
- Railway
- Watercourse
- Waterbody
- Parcel
- Vacant ICI
- Vacant Residential



Project No. 121-23026-00

Sudbury Water and Wastewater Master Plan

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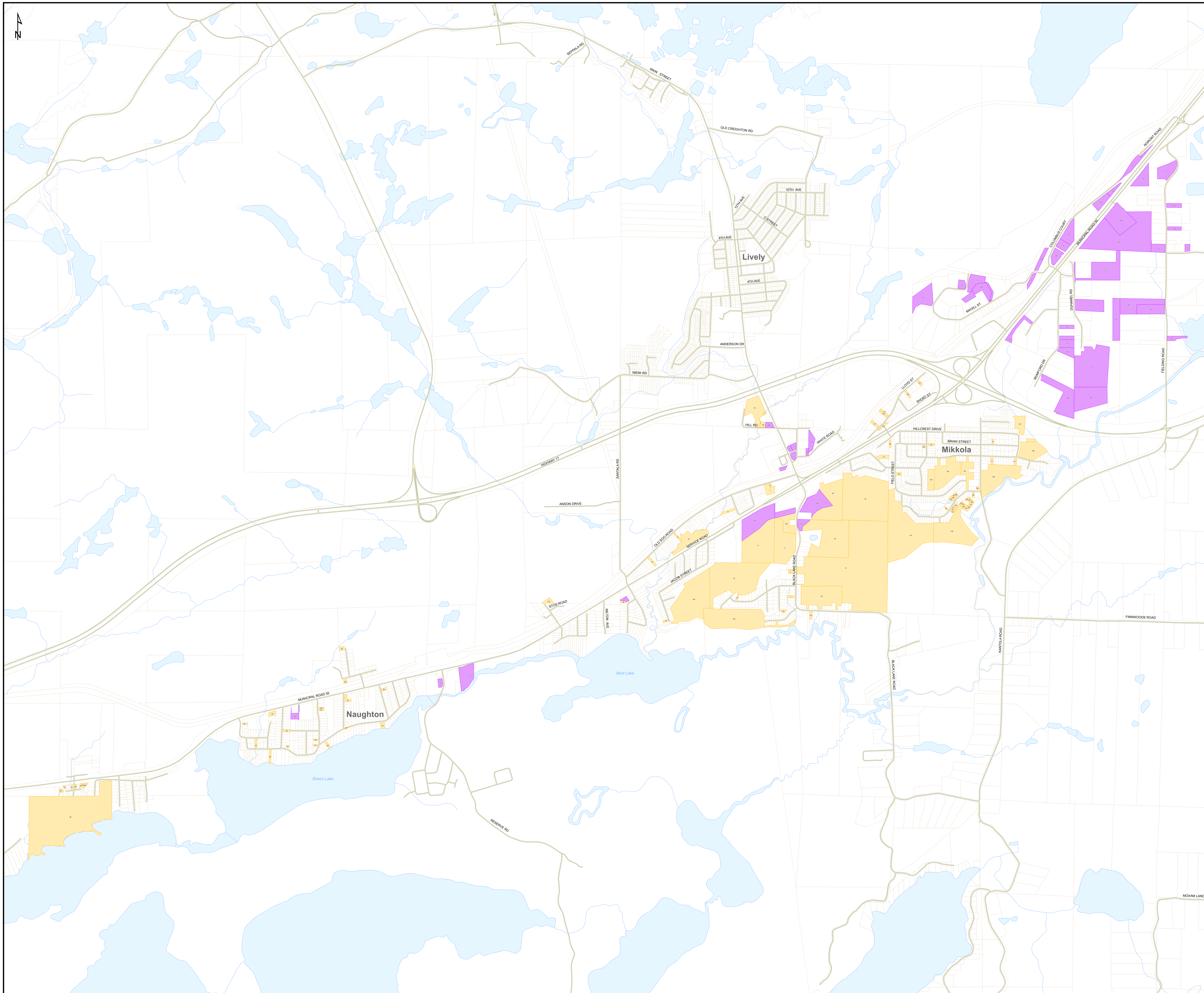
Map 2

Lively
2011 - Ultimate Buildout
Residential and ICI
Development Areas



Legend

- Road
- Watercourse
- Waterbody
- Parcel
- Vacant ICI
- Vacant Residential

Sudbury | WSP

Project No. 121-23026-00

Sudbury Water and Wastewater Master Plan

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Map 3

Walden
2011 - Ultimate Buildout
Residential and ICI
Development Areas

APPENDIX

B WATER DEMAND CAPACITY ASSESSMENTS

Vermilion - Water Demand Forecasts 5/11/2017

(Includes: Copper Cliff, Lively, Walden, Naughton, Whitefish)

DATA ANALYSIS

	2009	2010	2011	2012	2013	Summary	Design Criterion	Comments
Average Day Flow (m^3/d)				3,227			4,059	Production data is not available. These values are from the 2012 billing data. Average existing flows were estimated using historical billing records for 2012 and multiplied with an unbilled water rate. The billed water rates do not account for any leakage or unmetered water in the system.
Max Day Flow (m^3/d)							7,712	Data is not available. Estimated using the estimate average day demand (based on billing data) and the max day factor from the MOE Guidelines.
Max Day Factor							1.90	MOE Guidelines recommend a value of 1.90 for populations between 10,001 and 25,000. In the absence of historical production data, this value was adopted.
Peak Hour (L/s)				Not Available				Data is not available.
Peak Hour (m^3/d)							11,569	
Peak Hour Factor							2.85	MOE Guidelines recommend a value of 2.85 for populations between 10,001 and 25,000. In the absence of historical production data, this value was adopted.

	2009	2010	2011	2012	2013	Design Criterion	2016	2021	2026	2031	2036	2041	Ultimate	Comments
Population (Existing Areas)	10,359	10,359	10,359	10,359	10,359	10,359	10,359	10,359	10,359	10,359	10,359	10,359	10,359	From data provided by Hemson grouped by water system.
Population Growth							486	944	1,327	1,553	1,691	1,726	9,041	
Total Population							10,845	11,303	11,686	11,912	12,050	12,085	19,400	From data provided by Hemson grouped by water system.
Residential Growth Area (ha)							3.5	8.9	5.7	1.6	0.1	0.7	178.7	From City's GIS database.
Residential Growth Area (ha) - Cumulative							3.5	12.4	18.1	19.7	19.7	20.4	199.1	
Institutional Growth Area (ha)							0.0	0.0	1.12	0.00	0.85	0.0	0.0	
Institutional Growth Area (ha) - Cumulative							0.0	0.0	1.12	1.12	1.97	1.97	1.97	From City's GIS database.
Commercial Growth Area (ha)							0.0	0.0	1.7	0.0	8.2	0.0	0.0	
Commercial Growth Area (ha) - Cumulative							0.0	0.0	1.66	1.66	9.87	9.87	9.87	From City's GIS database.
Industrial Growth Area - Walden Industrial Area - Total Undeveloped and Unserviced Land (ha)							0.0	0.0	72.0	0.0	0.0	0.0	0.0	
Industrial Growth Area - Walden Industrial Area - 20% (ha)							0.0	0.0	14.4	0.0	0.0	0.0	0.0	
Industrial Growth Area - Walden Industrial Area - 80% (ha)							0.0	0.0	57.6	0.0	0.0	0.0	0.0	
Industrial Growth Area - Walden Industrial Area - Total Developed and Unserviced Land (ha)							0.0	0.0	0.0	100.0	0.0	0.0	0.0	
Industrial Growth Area - Balance of Industrial Area (ha)							0.0	0.0	2.4	0.0	18.93	0.0	0.0	
Industrial Growth Area (ha)*							0.0	0.0	74.4	100.0	18.93	0.0	0.0	
Industrial Growth Area (ha) - Walden 20% Cumulative							0.0	0.0	14.4	14.4	14.4	14.4	14.4	
Industrial Growth Area (ha) - Walden 80% Cumulative							0.0	0.0	57.6	57.6	57.6	57.6	57.6	
Industrial Growth Area - Walden Industrial Area - Developed and Unserviced Land (ha) Cumulative							0.0	0.0	0.0	100.0	100.0	100.0	100.0	
Industrial Growth Area (ha) - Balance Cumulative							0.0	0.0	2.4	2.4	21.3	21.3	21.3	
Industrial Growth Area (ha) - Cumulative							0.0	0.0	74.4	174.4	193.3	193.3	193.3	From City's GIS database.
**Includes 100 ha of land that is currently developed but not serviced (to be serviced by 2031)*														
							0.0	0.0	77.2	100.0	28.0	0.0	0.0	
ICI (ha) - Cumulative							0.0	0.0	77.2	177.2	205.2	205.2	205.2	Sum of Institutional, Commercial and Industrial areas
Total Growth Area (ha) - Cumulative							3.5	12.4	95.3	196.8	224.9	225.6	404.3	

	2009	2010	2011	2012	2013	Summary	Design Criterion	Comments
Ratio of Residential to Total Water Billed	0.80	0.80	0.80	0.80	0.80	0.80		Estimated amount of water consumption related to ICI based on metering data and obtained ratio of residential to total consumption.
Residential Flow (m^3/d)				2581				Calculated based on ratio of residential consumption to total consumption.
Ratio of ICI to Total Water Billed	0.20	0.20	0.20	0.20	0.20	0.200		
				646				
Per Capita Residential Demand (m^3/cap/day)				0.249		0.249	0.250	Average day demand from production data was not available. The historical demand was estimated using the approximate amount of water billed per capita per day in 2012. The MOE suggests using between 0.27 and 0.45 m^3/cap/d. Therefore, 0.27 m3/cap/d was adopted in this case since it would accould for slight variation in historical production values, compared to billing data.
Unbilled Rate							25.8%	Rates determined for the Vermilion Water System were based on billed water rates, which do not account for any leakage or unmetered water in the network. The average day rates were therefore multiplied by this unbilled water rate.
Average Institutional Flow Unit Rate (m^3/ha/d)							28.0	*MOE Guidelines recommend a value of 28 m^3/ha/d.*
Average Commercial Flow Unit Rate (m^3/ha/d)							28.0	*MOE Guidelines recommend a value of 28 m^3/ha/d.*
Average Industrial Flow Unit Rate - Balance of Industrial Lands (m^3/ha/d)							35.0	*MOE Guidelines recommend a value of 35 m^3/ha/d for light industry and 55 m^3/ha/d for heavy industry.*
Average Industrial Flow Unit Rate - Walden 20% (m^3/ha/d)							35.0	
Average Industrial Flow Unit Rate - Walden 80% (m^3/ha/d)							3.0	
Average Industrial Flow Unit Rate - Currently Developed but Unserviced Area (m^3/ha/d)							3.0	

Vermilion - Water Demand Forecasts 5/11/2017

	2016	2021	2026	2031	2036	2041	Ultimate
Average Residential and ICI Flows (m^3/d) - Existing	4,059	4,059	4,059	4,059	4,059	4,059	4,059
Average Residential Flows (m^3/d) - Growth	153	297	417	488	532	543	2,843
Average Residential Flows (m^3/d) - Total	4,212	4,356	4,476	4,548	4,591	4,602	6,903
Average Institutional Flow (m^3/d)	0	0	31	31	55	55	55
Average Commercial Flow (m^3/d)	0	0	46	46	276	276	276
Average Industrial Flow - Walden Industrial Park - 20% (m^3/d)	0	0	504	504	504	504	504
Average Industrial Flow - Walden Industrial Park - 80% (m^3/d)	0	0	173	173	173	173	173
Average Industrial Flow - Walden Industrial Park - Currently Developed but Locally Serviced Area (m^3/d)	0	0	0	300	300	300	300
Average Industrial Flow - Balance of Industrial Area (m^3/d)	0	0	84	84	747	747	747
Average ICI Flow (m^3/d)	0	0	839	1,139	2,055	2,055	2,055
Average Day Flow (m^3/d)	4,212	4,356	5,315	5,686	6,646	6,657	8,957
Max Day Flow (m^3/d)	8,003	8,276	10,098	10,804	12,627	12,648	17,019
Peak Hour Flow (m^3/d)	12,004	12,414	15,148	16,206	18,941	18,972	25,529

Average existing flows were estimated using historical billing records for 2012 and therefore multiplied with an unbilled water rate. The billed water rates do not account for any leakage or unmetered water in the system.

Average existing flows were estimated using historical billing records for 2012 and therefore multiplied with an unbilled water rate. The billed water rates do not account for any leakage or unmetered water in the system.

ALTERNATIVE CALCULATION METHOD This method does not distinguish between Residential and ICI water consumption.

Per Capita Demand (m^3/cap/day)	0.000	0.000	0.000	0.392	0.000	0.392	0.392
Max Day Factor							1.90
Peak Hour Factor							2.85

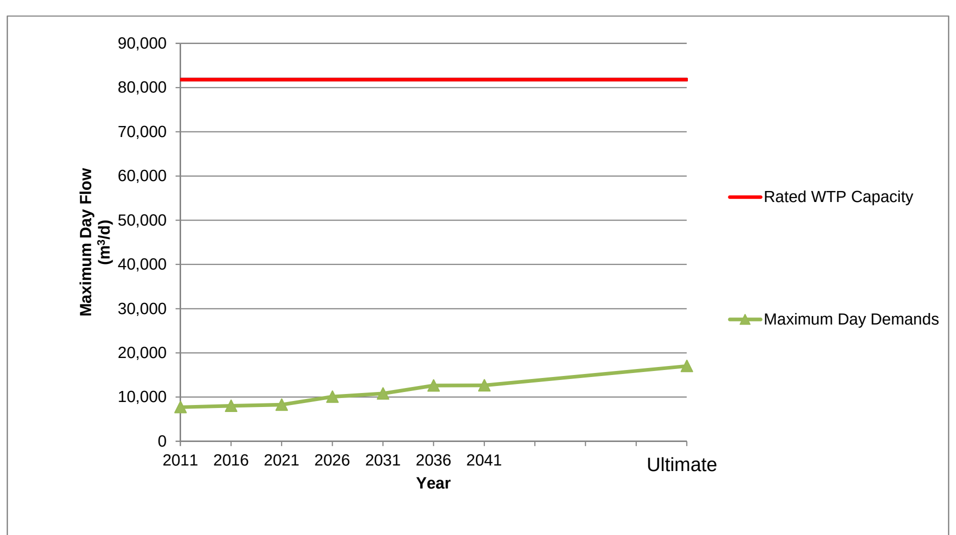
If ICI is not considered explicitly and demand is divided by total population.
The historical per capita consumption is applied for future development.

	2016	2021	2026	2031	2036	2041	Ultimate
Average Day Flow (m^3/d)	4,249	4,429	4,579	4,668	4,722	4,735	7,602
Max Day Flow (m^3/d)	8,074	8,415	8,700	8,868	8,971	8,997	14,443
Peak Hour Flow (m^3/d)	12,111	12,622	13,050	13,303	13,457	13,496	21,665

SENSITIVITY ANALYSIS Not applicable - limited data available.

CAPACITY CHECK Note: The Vermilion WTP is owned by Vale and is used by Vale operations. Vale's use is not included in the demands.

	2011	2016	2021	2026	2031	2036	2041	Ultimate
Rated WTP Capacity	81,800	81,800	81,800	81,800	81,800	81,800	81,800	81,800
Actual WTP Capacity	81,800	81,800	81,800	81,800	81,800	81,800	81,800	81,800
Maximum Day Demands	7,712	8,003	8,276	10,098	10,804	12,627	12,648	17,019
Peak Hour Demands	11,569	12,004	12,414	15,148	16,206	18,941	18,972	25,529

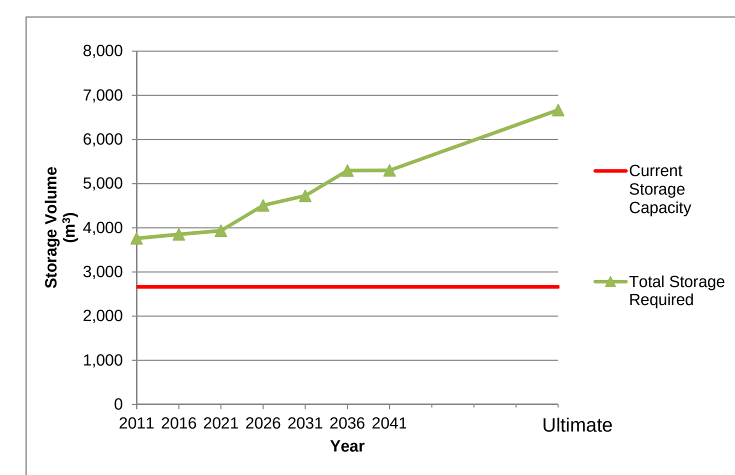


STORAGE REQUIREMENTS

Storage Available

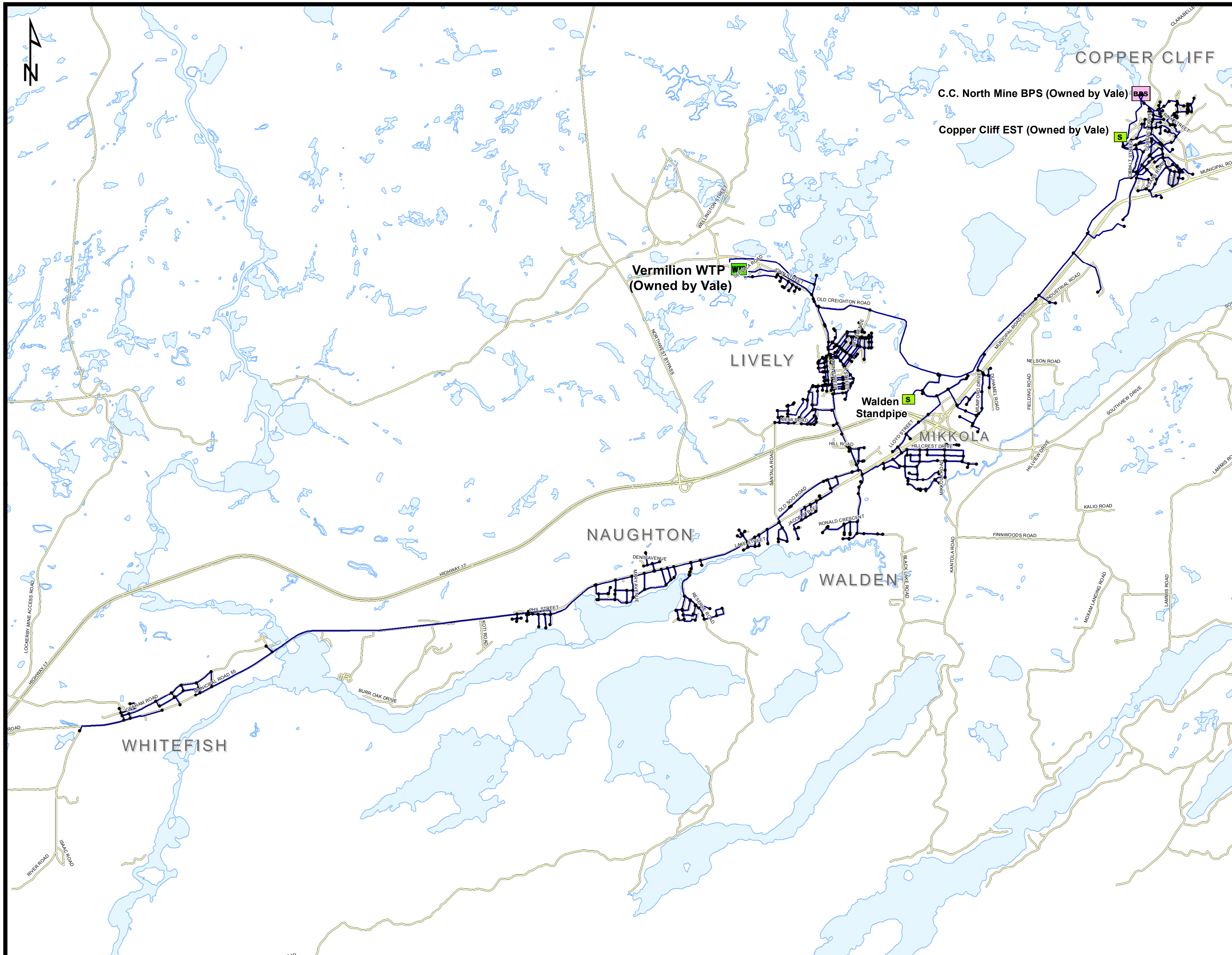
Standpipe Volume (m^3)	4,732	From DWWP 016-204, Issue 2 (November 3, 2011) for Walden tank. In addition, Vale's Copper Cliff tank would provide some protection, but the volume available for City use cannot be confirmed.
		Historical Maximum Day and Peak Hour Demands are not available for 2011, but are estimated using billing data and MOE Design Guidelines Max Day and Peaking Factors.
Total Effective Storage (m^3)*	2,662	Total effective storage is based on a LWL and HWL of 308.61m and 320.34m respectively and a raidus of 8.5m, per the engineering drawings for the Walden Standpipe (by Horton CBI Limited).
Maximum Fire flow Requirements (L/s)	150	
Fire Duration (hrs)	2	
Minimum Fire Flow Requirement for Residential Areas (L/s)	75	From CGS Engineering Design Manual
Fire Duration (hrs)	1.75	From Fire Underwriters Survey Requirements corresponding to 75 L/s

	Max Day Demand (m^3/d)	Required Fire Flow (m^3/d)	Max Day + Fire (m^3/d)	Peak Hour (m^3/d)	A - Fire Storage (m^3)	B - Equalization Storage (m^3)	C - Emergency Storage (m^3)	A + B + C = Storage Required (m^3)	Storage Available (m^3)	Deficit (m^3)
2011	7,712	12,960	20,672	11,569	1,080	1,928	752.0	3,760	2,662	1,098
2016	8,003	12,960	20,963	12,004	1,080	2,001	770.2	3,851	2,662	1,188
2021	8,276	12,960	21,236	12,414	1,080	2,069	787.3	3,936	2,662	1,274
2026	10,098	12,960	23,058	15,148	1,080	2,525	901.2	4,506	2,662	1,843
2031	10,804	12,960	23,764	16,206	1,080	2,701	945.2	4,726	2,662	2,064
2036	12,627	12,960	25,587	18,941	1,080	3,157	1059.2	5,296	2,662	2,633
2041	12,648	12,960	25,608	18,972	1,080	3,162	1060.5	5,302	2,662	2,640
Ultimate	17,019	12,960	29,979	25,529	1,080	4,255	1333.7	6,668	2,662	4,006



APPENDIX

C WATER MODEL RESULTS



Legend

- Well
- Storage Tank
- Booster Pumping Station (BPS)
- Water Treatment Plant (WTP)
- Pipe
- Road
- Waterbody

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0 1,000 2,000 3,000 4,000 m

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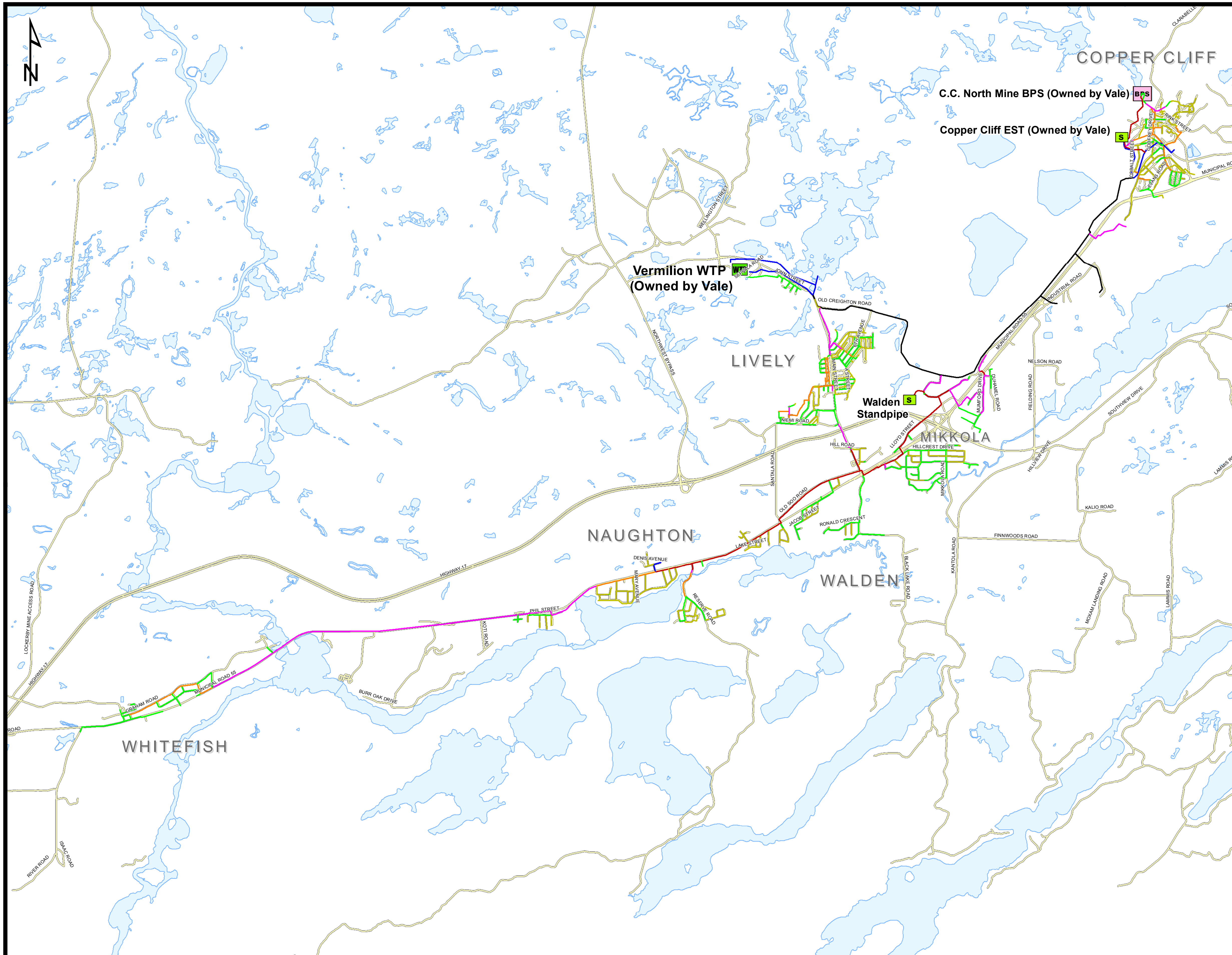
Map 1

Vermilion Water System - Key Plan



Legend

- W Well
- S Storage Tank
- BPS Booster Pumping Station (BPS)
- WTP Water Treatment Plant (WTP)
- Road
- Waterbody

Pipe: Diamater (mm)

- <= 150
- <= 200
- <= 250
- <= 300
- <= 400
- <= 600
- > 600

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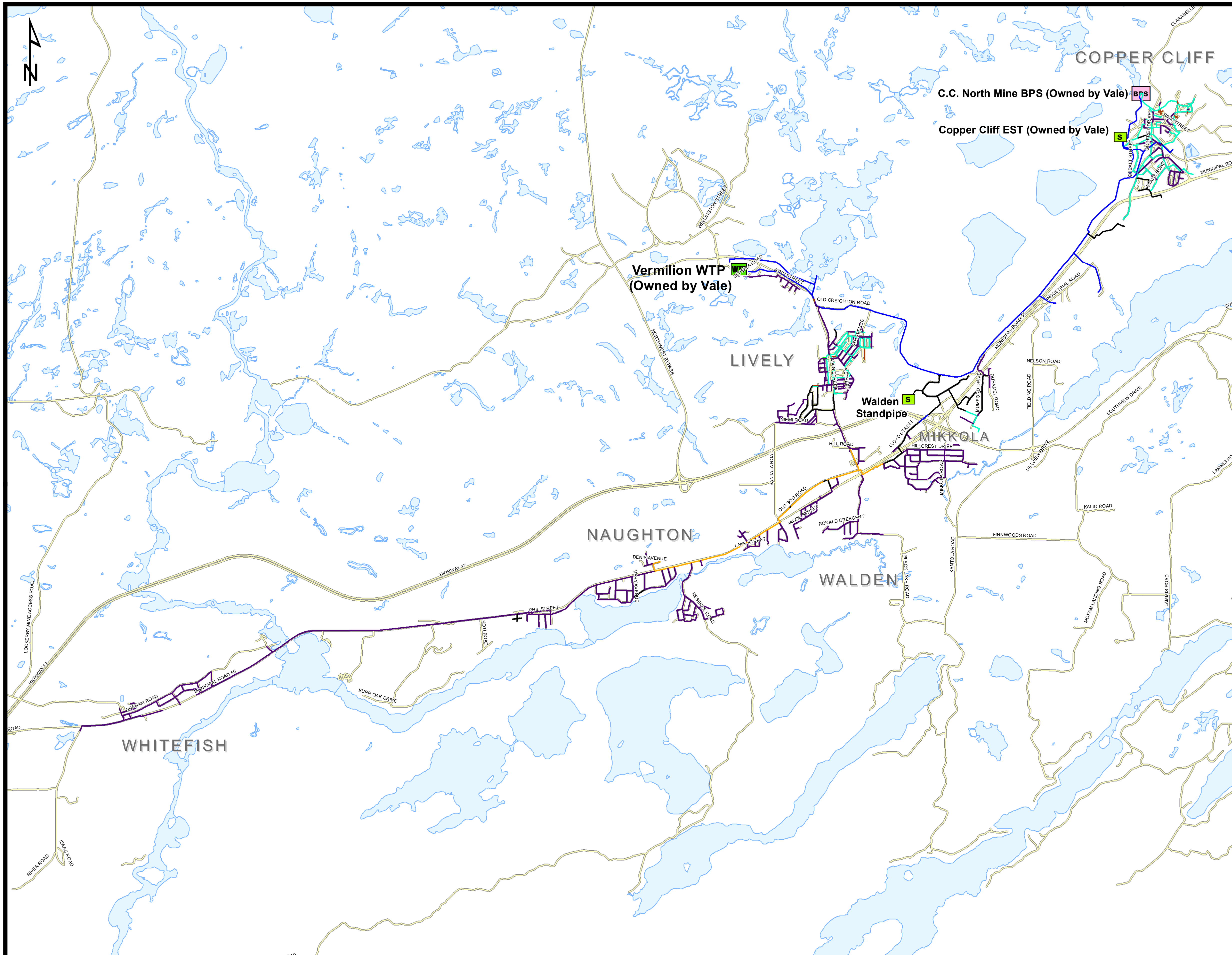
Map 2

Vermilion Water System - Pipe Diameter



Legend

- W Well
- S Storage Tank
- BPS Booster Pumping Station (BPS)
- WTP Water Treatment Plant (WTP)
- Road
- Waterbody

Material

- Asbestos Cement
- Cast iron
- Concrete
- Copper
- Ductile Iron
- Galvanized iron
- PVC
- Polyethylene
- Steel

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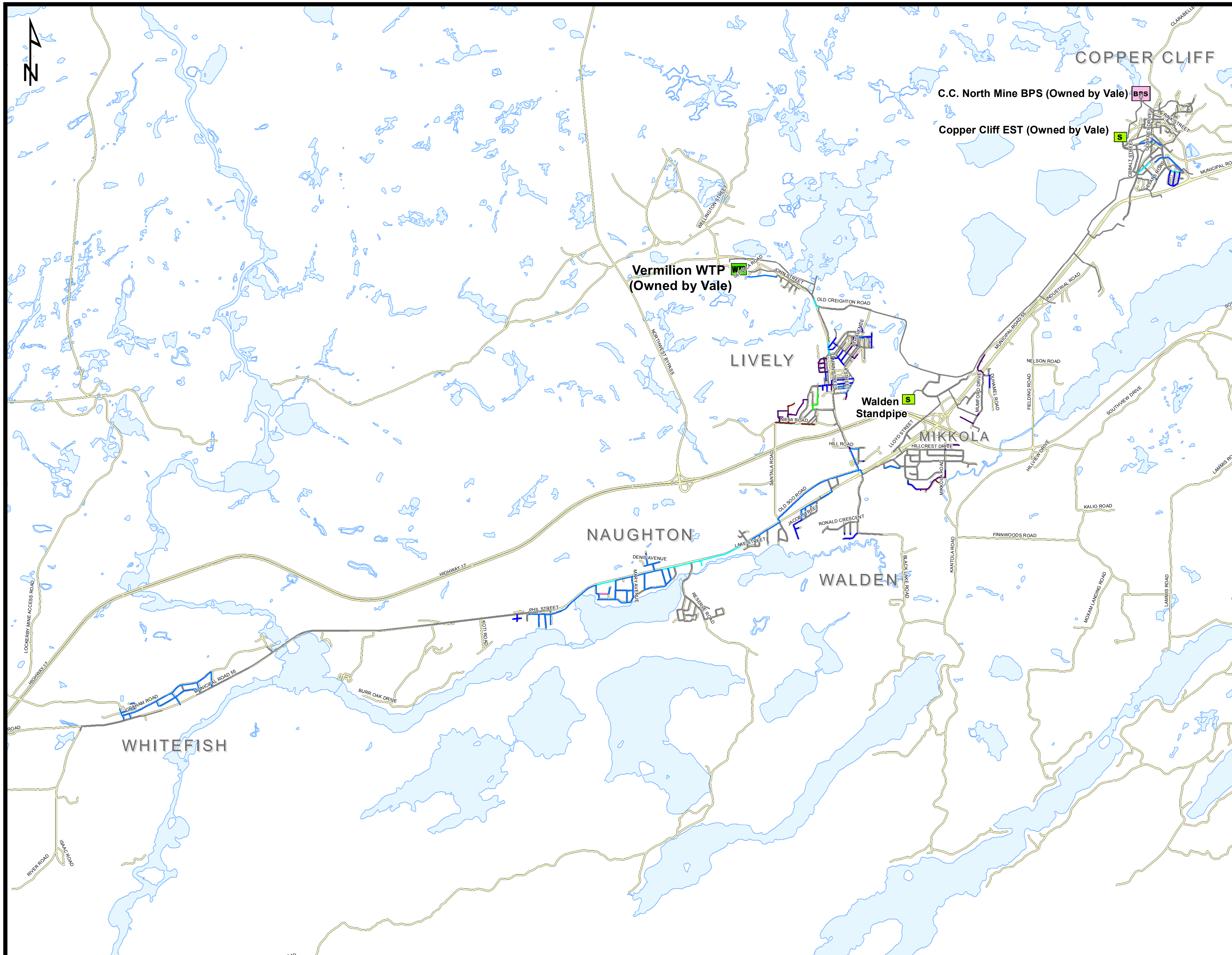
Map 3

Vermilion Water System - Pipe Material



Legend

- W Well
- S Storage Tank
- BPS Booster Pumping Station (BPS)
- WTP Water Treatment Plant (WTP)
- Road
- Waterbody

Installed Year

- N/A
- <= 1930
- <= 1940
- <= 1950
- <= 1960
- <= 1970
- <= 1980
- <= 1990
- <= 2000
- <= 2010
- > 2011

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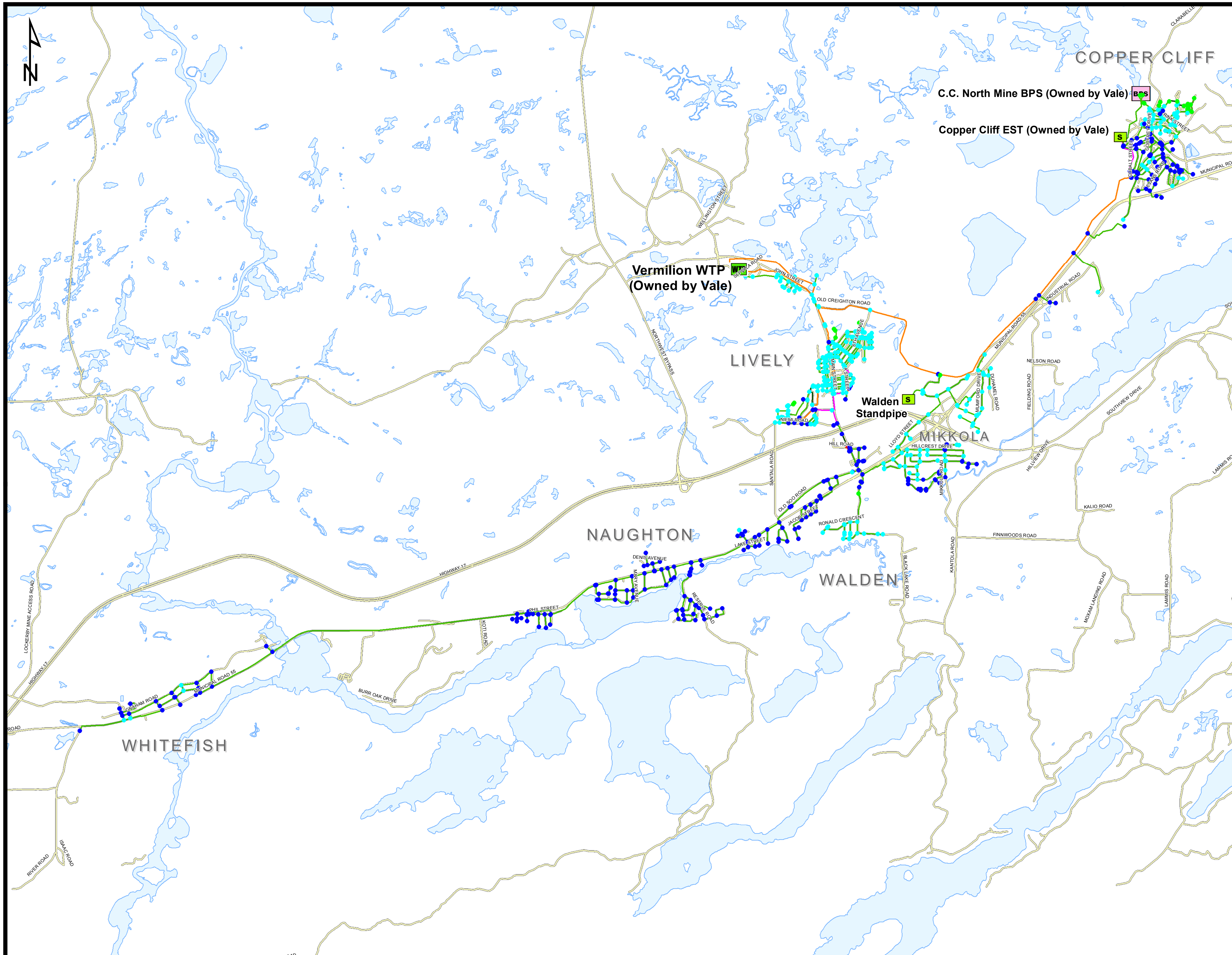
Map 4

Vermilion Water System - Pipe Age



Legend

- W Well
- S Storage Tank
- BPS Booster Pumping Station (BPS)
- WTP Water Treatment Plant (WTP)
- Road
- Waterbody

Junction: Pressure (psi)

- <= 20
- <= 40
- <= 60
- <= 80
- <= 100
- > 100

Pipe: Headloss Gradient (m/km)

- <= 1.5
- <= 2.0
- <= 3.0
- <= 5.0
- > 5.0

MDD/ADD = 1.9

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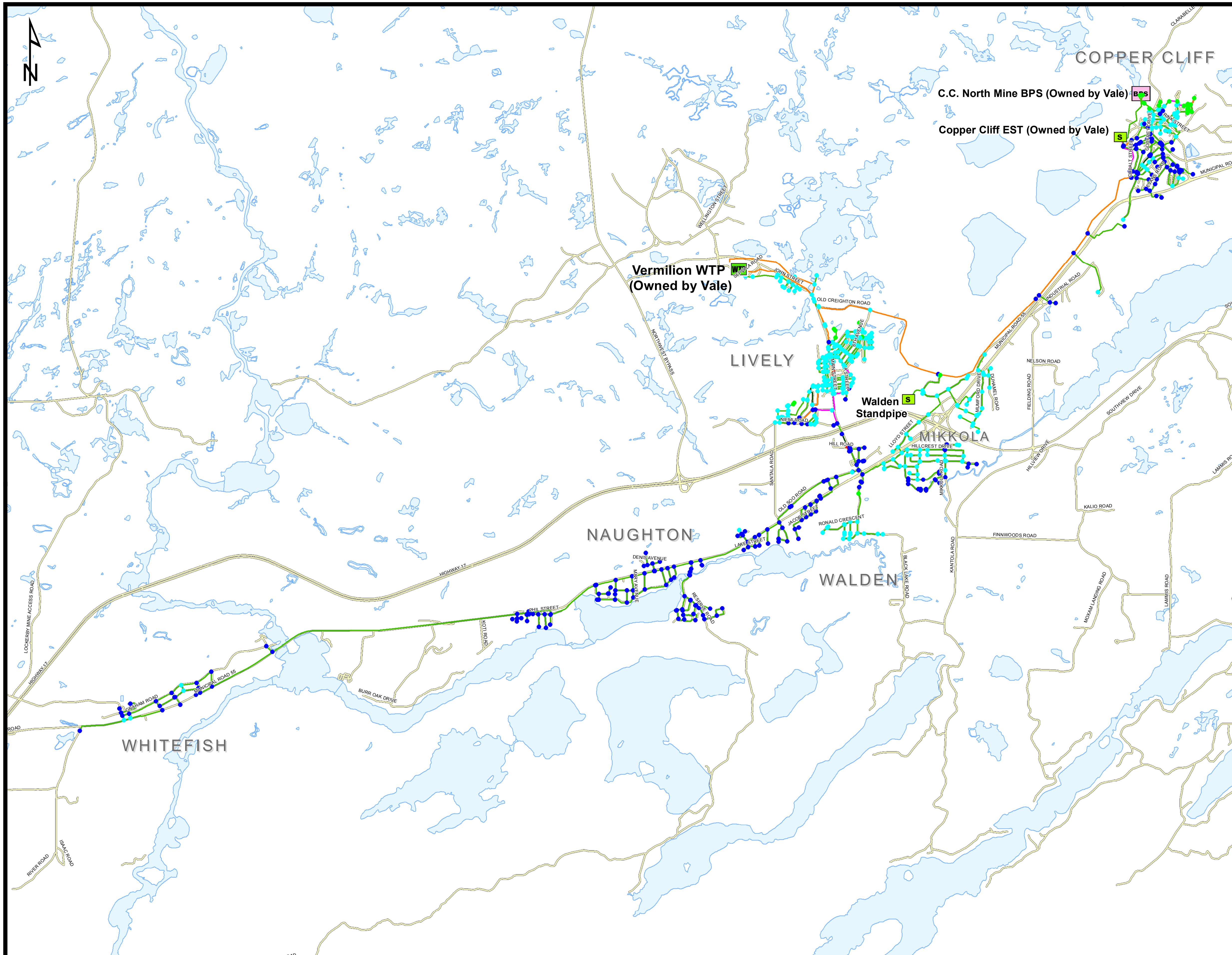
Map 5

Vermilion Water System - Pipe Headloss and Junction Pressure

Scenario: 2011ADD



Legend

- W Well
- S Storage Tank
- BPS Booster Pumping Station (BPS)
- WTP Water Treatment Plant (WTP)
- Road
- Waterbody

Junction: Pressure (psi)

- <= 20
- <= 40
- <= 60
- <= 80
- <= 100
- > 100

Pipe: Headloss Gradient (m/km)

- <= 1.5
- <= 2.0
- <= 3.0
- <= 5.0
- > 5.0

MDD/ADD = 1.9

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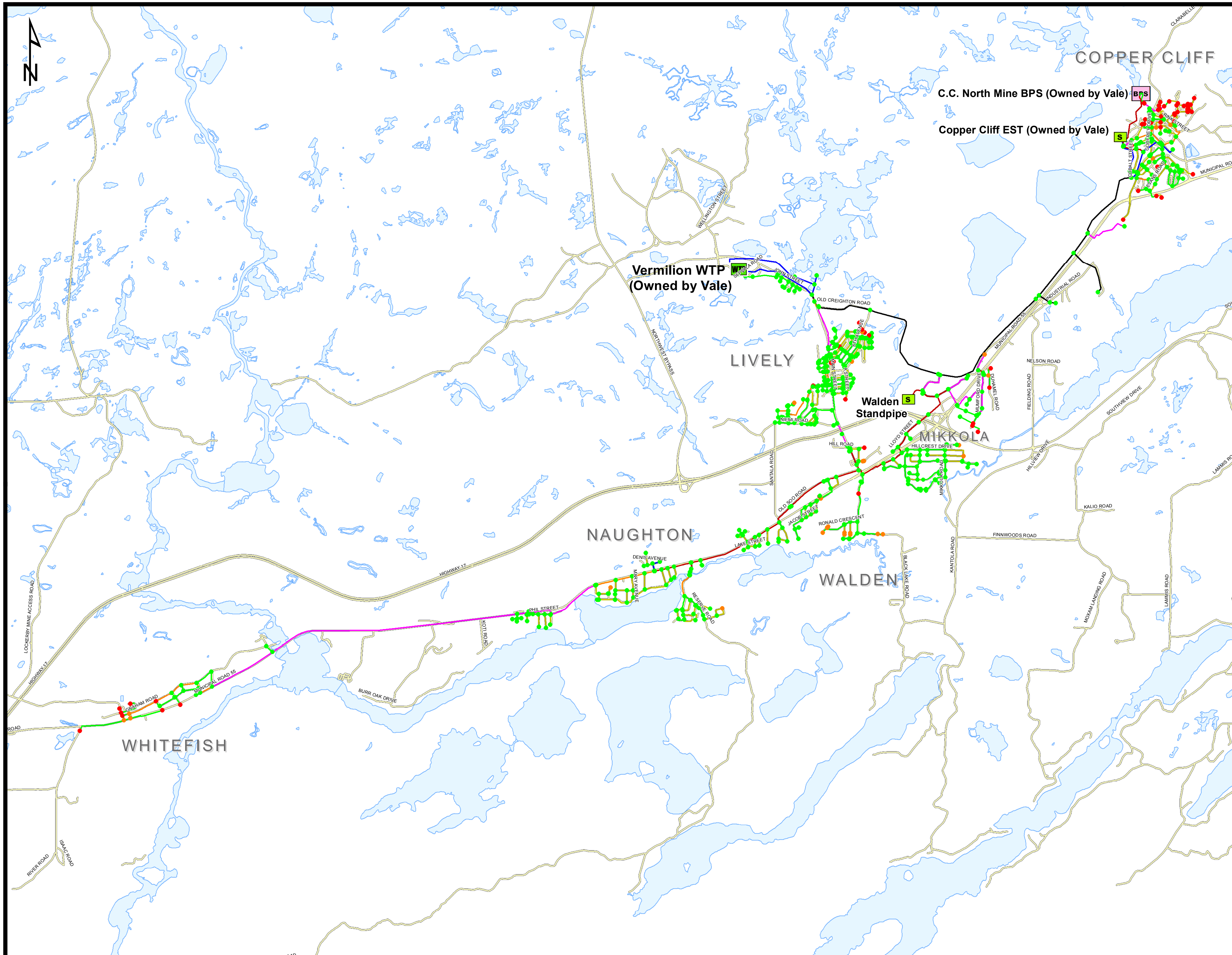
Map 6

Vermilion Water System - Pipe Headloss and Junction Pressure

Scenario: 2011MDD



Legend

- W Well
- S Storage Tank
- BPS Booster Pumping Station (BPS)
- WTP Water Treatment Plant (WTP)
- Road
- Waterbody

Junction: Fire Flow (Available) (L/s)

- <= 65 (Res) or 130 (ICI)
- < 75 (Res) or 150 (ICI)
- >= 75 (Res) or 150 (ICI)

Pipe: Diamater (mm)

- <= 150
- <= 200
- <= 250
- <= 300
- <= 400
- <= 600
- > 600

MDD/ADD = 1.9

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0 1,000 2,000 3,000 4,000 m

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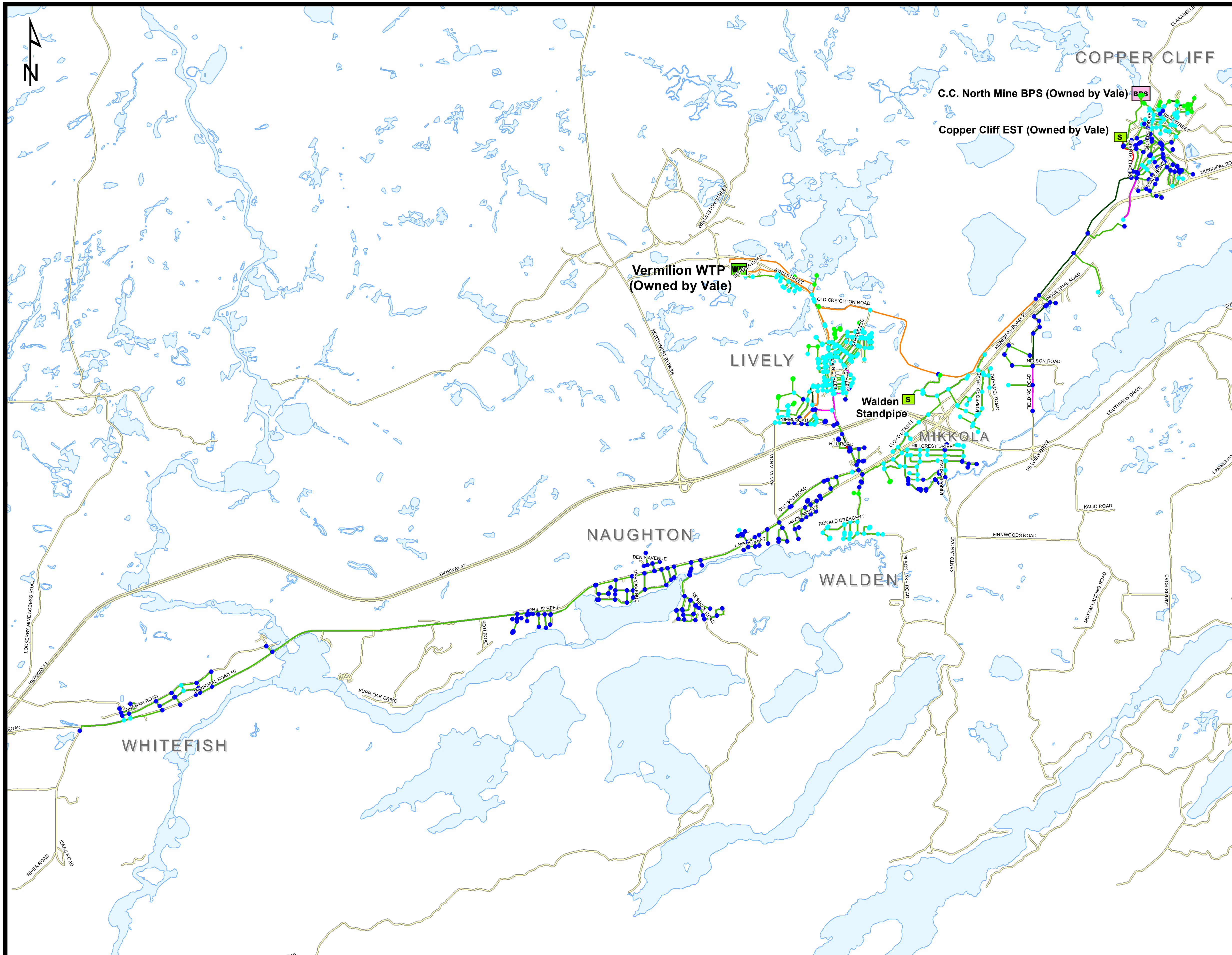
Map 7

Vermilion Water System - Fire Flow Availability

Scenario: 2011MDD+FF



Legend

- W Well
- S Storage Tank
- BPS Booster Pumping Station (BPS)
- WTP Water Treatment Plant (WTP)
- Road
- Waterbody

Junction: Pressure (psi)

- <= 20
- <= 40
- <= 60
- <= 80
- <= 100
- > 100

Pipe: Headloss Gradient (m/km)

- <= 1.5
- <= 2.0
- <= 3.0
- <= 5.0
- > 5.0

MDD/ADD = 1.9

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0 1,000 2,000 3,000 4,000 m

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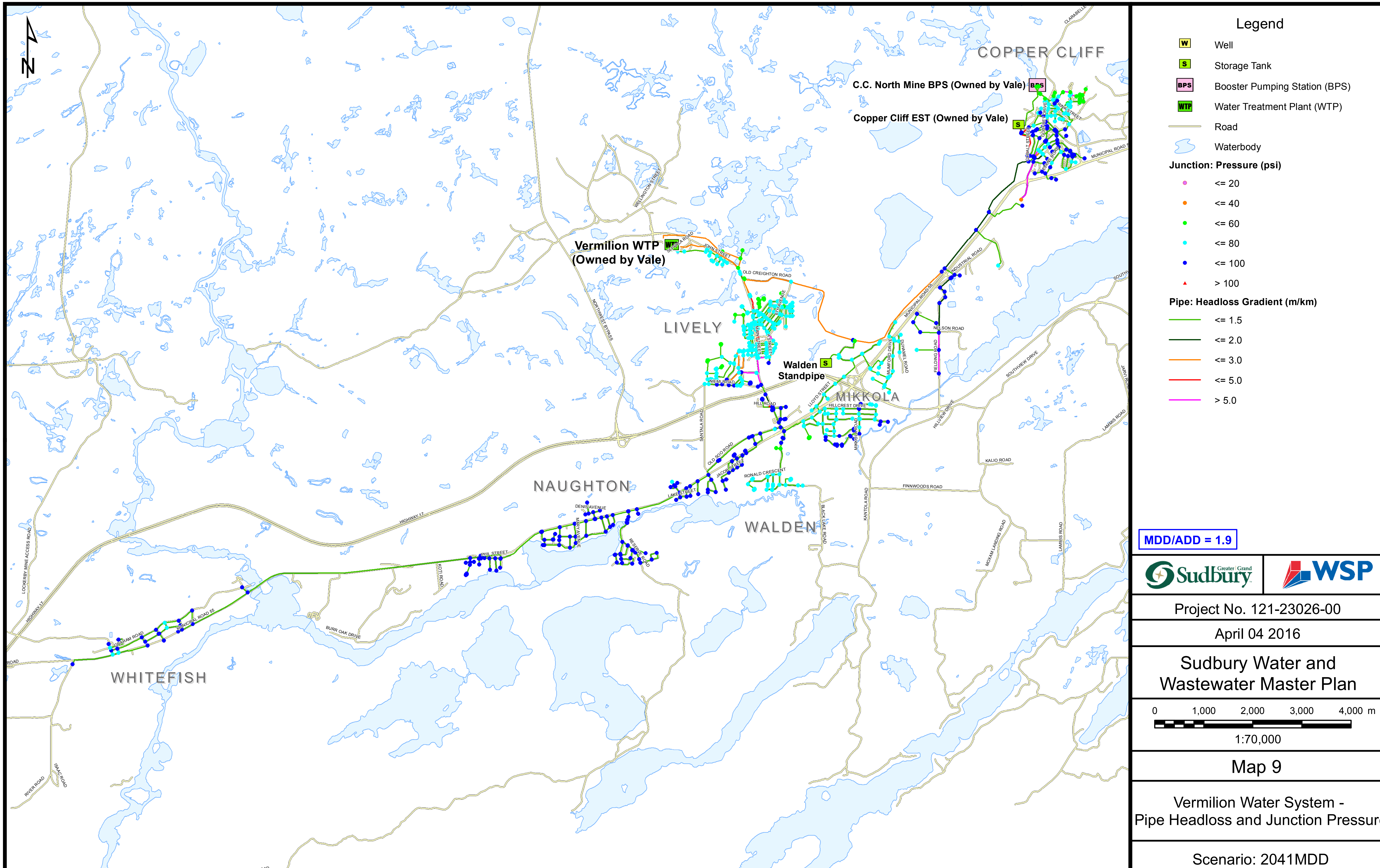
Map 8

Vermilion Water System - Pipe Headloss and Junction Pressure

Scenario: 2041ADD



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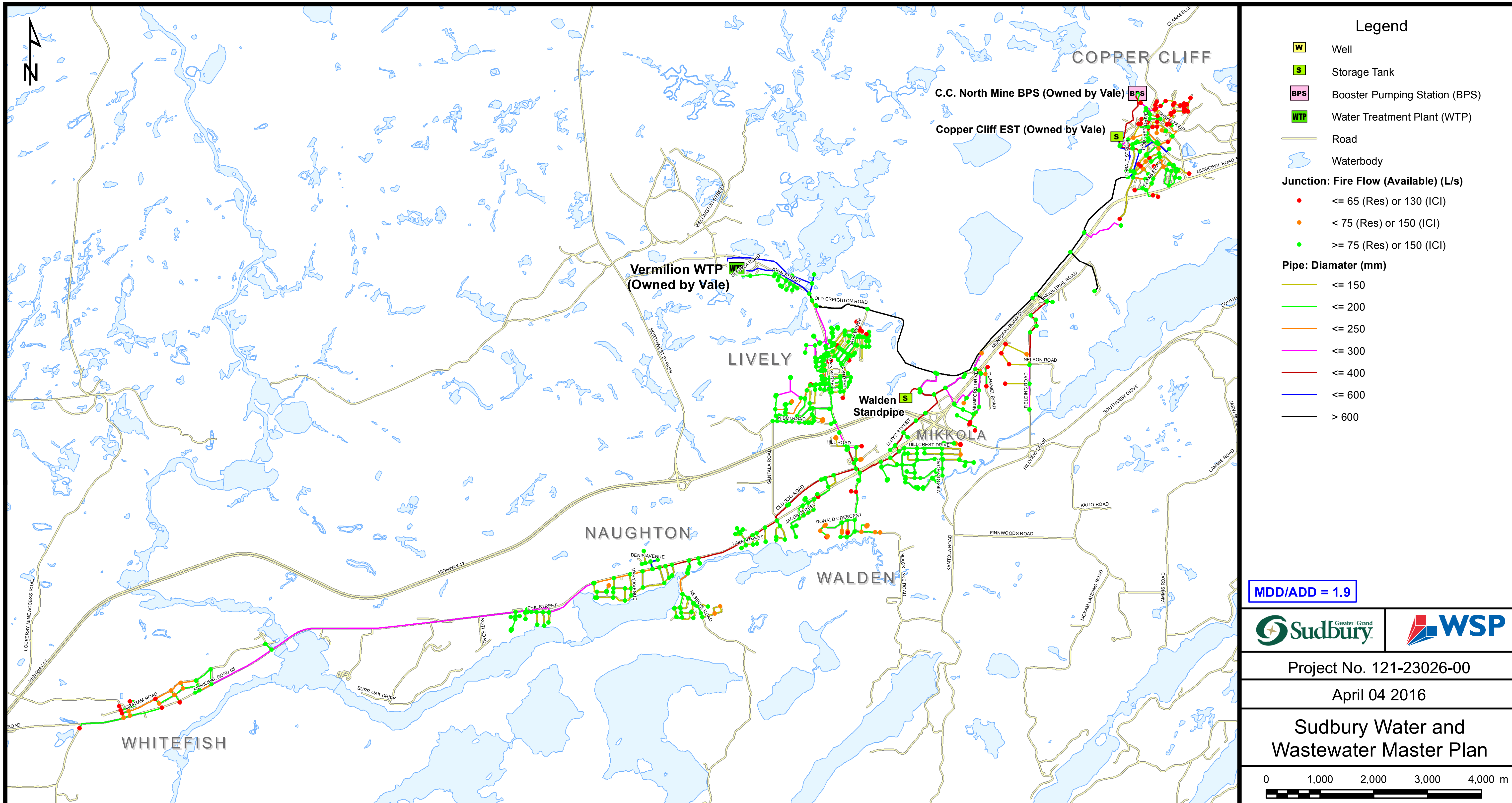
Map 9

Vermilion Water System - Pipe Headloss and Junction Pressure

Scenario: 2041MDD



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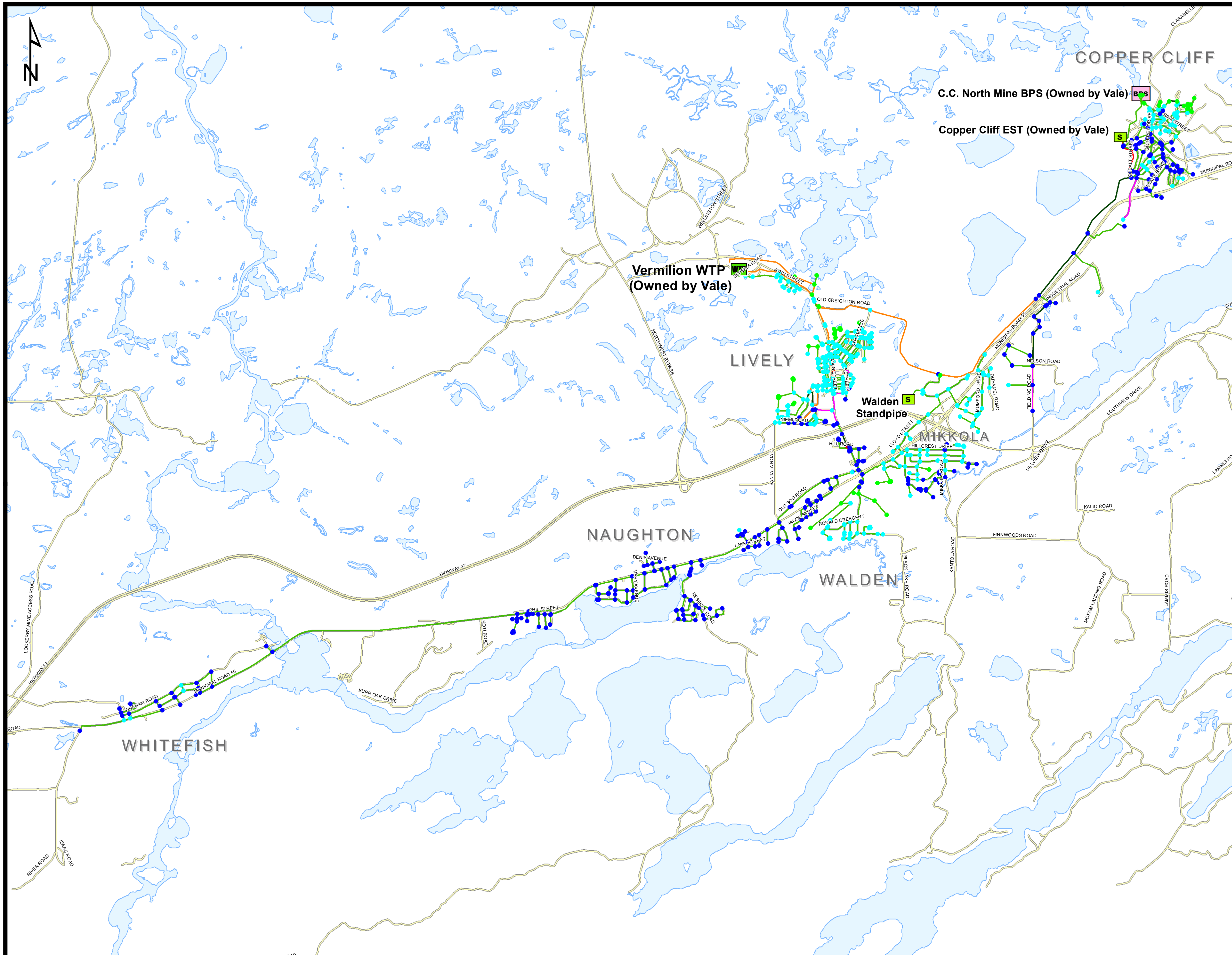
Map 10

Vermilion Water System - Fire Flow Availability

Scenario: 2041MDD+FF



Legend

- W Well
- S Storage Tank
- BPS Booster Pumping Station (BPS)
- WTP Water Treatment Plant (WTP)
- Road
- Waterbody

Junction: Pressure (psi)

- <= 20
- <= 40
- <= 60
- <= 80
- <= 100
- > 100

Pipe: Headloss Gradient (m/km)

- <= 1.5
- <= 2.0
- <= 3.0
- <= 5.0
- > 5.0

MDD/ADD = 1.9

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0 1,000 2,000 3,000 4,000 m

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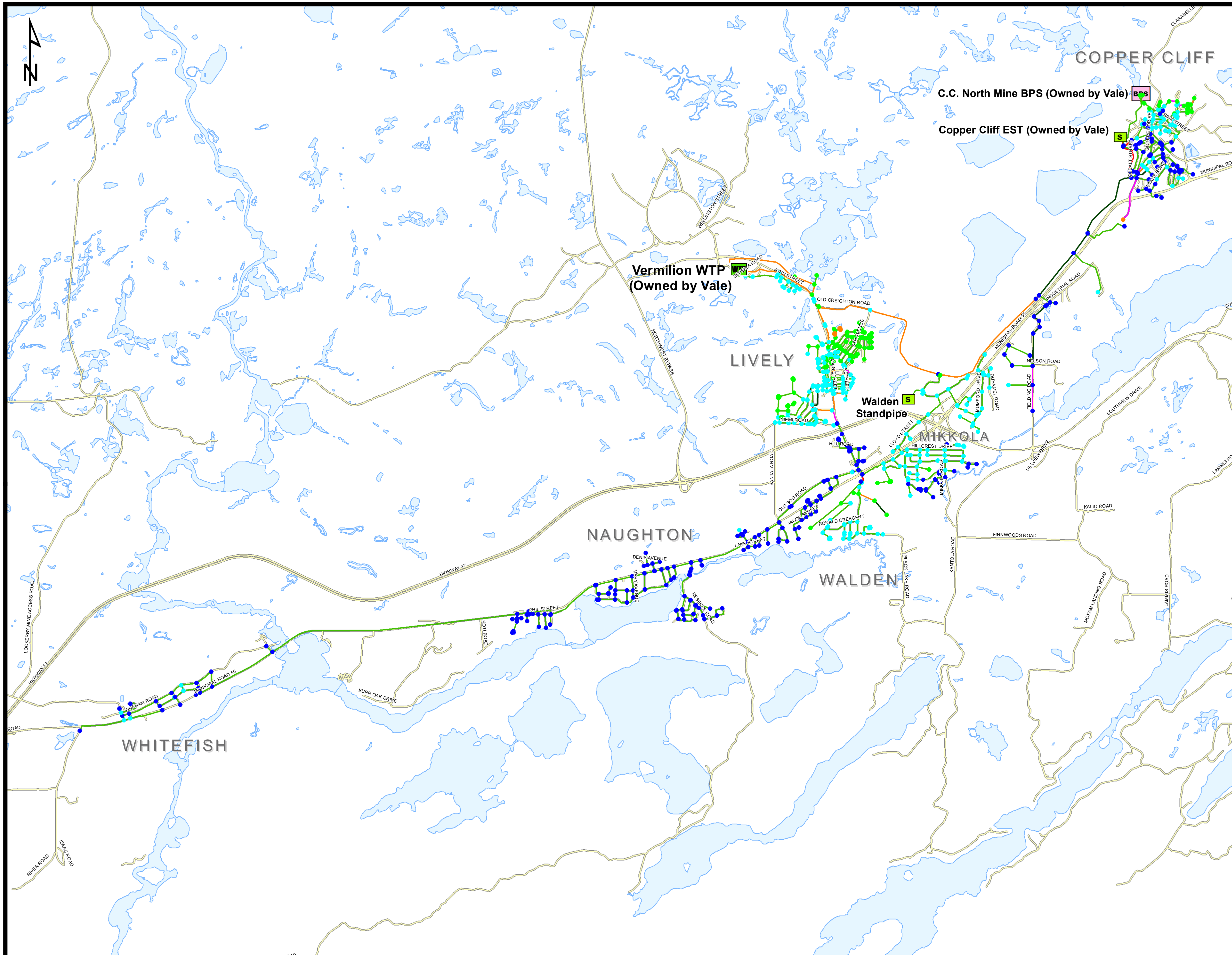
Map 11

Vermilion Water System - Pipe Headloss and Junction Pressure

Scenario: ULTADD



Legend

- Well
- Storage Tank
- Booster Pumping Station (BPS)
- Water Treatment Plant (WTP)
- Road
- Waterbody

Junction: Pressure (psi)

- <= 20
- <= 40
- <= 60
- <= 80
- <= 100
- > 100

Pipe: Headloss Gradient (m/km)

- <= 1.5
- <= 2.0
- <= 3.0
- <= 5.0
- > 5.0

MDD/ADD = 1.9

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0 1,000 2,000 3,000 4,000 m

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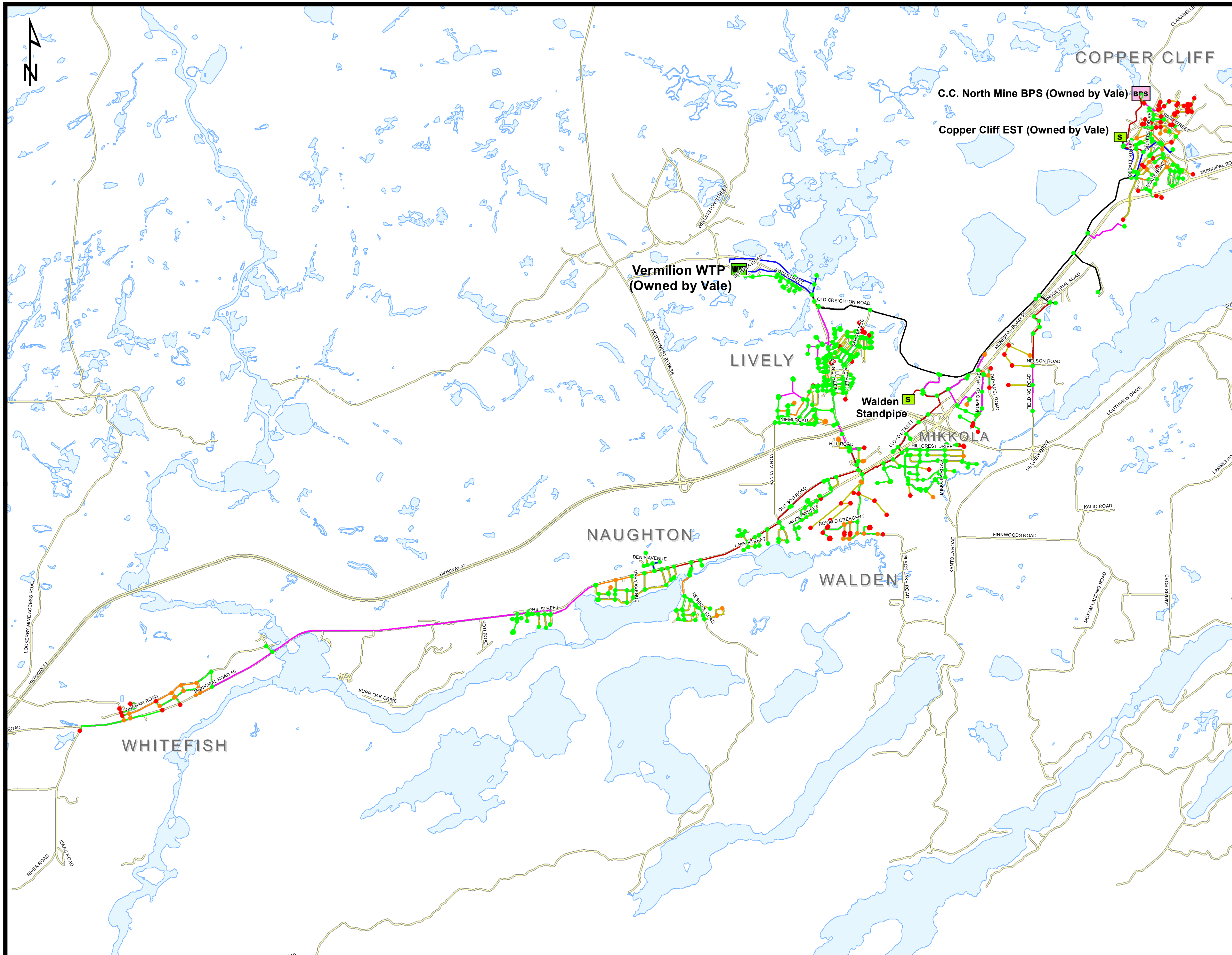
Map 12

Vermilion Water System - Pipe Headloss and Junction Pressure

Scenario: ULTMDD



Legend

- W Well
- S Storage Tank
- BPS Booster Pumping Station (BPS)
- WTP Water Treatment Plant (WTP)
- Road
- Waterbody

Junction: Fire Flow (Available) (L/s)

- <= 65 (Res) or 130 (ICI)
- < 75 (Res) or 150 (ICI)
- >= 75 (Res) or 150 (ICI)

Pipe: Diamater (mm)

- <= 150
- <= 200
- <= 250
- <= 300
- <= 400
- <= 600
- > 600

MDD/ADD = 1.9

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0 1,000 2,000 3,000 4,000 m

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Map 13

Vermilion Water System -
Fire Flow Availability

Scenario: ULTMDD+FF