APPENDIX 3-A

WASTEWATER SYSTEM GAP ANALYSIS & STATUS QUO REPORTS

CITY OF GREATER SUDBURY WATER AND WASTEWATER MASTER PLAN

AZILDA WASTEWATER SYSTEM GAP ANALYSIS AND STATUS QUO REPORT

CITY OF GREATER SUDBURY

DRAFT

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

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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 existing Azilda Wastewater System. Based on population growth projections and design criteria discussed in the *Population Projections and Development of Unit Rates Technical Memorandum* (WSP, 2014), 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 Azilda Wastewater System would continue to be a stand-alone system. Any potential interconnections between Azilda 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

Azilda is located along Regional Road 35 near Whitewater Lake, between Sudbury and Chelmsford. Mapping in Appendix A shows the Azilda 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 *Population Projections and Development of Unit Rates 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 Azilda population is expected to increase from 4,449 in 2011 to 5,103 by 2041 and 8,361 by Ultimate Buildout. ICI growth is expected to be a mix of industrial and commercial. Growth is discussed further in Section 6.1.

3 OVERVIEW OF EXISTING SYSTEM

The Azilda wastewater system services the community of Azilda and includes the Azilda Wastewater Treatment Plant (WWTP), as well as five lift stations and sewer network. The collection system consists of approximately 32.99 km of sewers and forcemains. Additional information on the existing systems is provided in the *Baseline Review Report for Wastewater Systems (WSP, 2014)*.

The Azilda Wastewater System is shown in Appendix B.

3.1 LIFT STATIONS

Table 3-1 below provides a summary of the main features of the lift stations.

Table 3-1 Azilda Sewage Lift Stations

LIFT STATION	YEAR CONSTRUCTED[1]	LAST UPGRADED[2]	WET WELL VOLUME (M^3)[1]	LIFT STATION CAPACITY AND FORCEMAIN INFORMATION[2]
Landry	1973	Unknown	6.1 [3]	Two dry pit pumps with a firm capacity of 41.3 L/s 997 m long, 300 mm diameter forcemain of unknown material
Laurier	1973	1992	22.9 [3]	Three dry pit pumps with a firm capacity of 90.10 L/s 3,108 m long, 450 mm diameter forcemain of unknown material
Maple	1987	None	9.9	Two submersible pumps with a firm capacity of 17.80 L/s 132 m long, 100 mm diameter PVC (SDR 26) forcemain
Marier	1973	Unknown	21.6	Two dry pit pumps with a firm capacity of 10.8 L/s 331 m long, 200 mm diameter forcemain of unknown material
Principale	1973	None	17.5	Two dry pit pumps with a firm design capacity of 32.9 L/s 554 m long, 250 mm diameter forcemain of unknown material

[1] Obtained or estimated from dimensions found in as-built and record, assuming water level does not exceed the High Water Alarm Level or, in absence of this alarm level, the inlet sewer invert.

[2] Obtained from Azilda Wastewater Treatment Plant and Collection System Class Environmental Assessment, Milestone Report #2, Final

[3] Obtained from Azilda Wastewater Treatment Plant and Collection System Class Environmental Assessment, Milestone Report #3, Final

3.2 AZILDA WWTP

The Azilda WWTP is owned and operated by the City of Greater Sudbury and is located at 564 St. Agnes Street. The WWTP is secondary treatment plant with an average day rated capacity of 3,300 m^3/d (MOECC, 2012). The treatment process is illustrated schematically below.

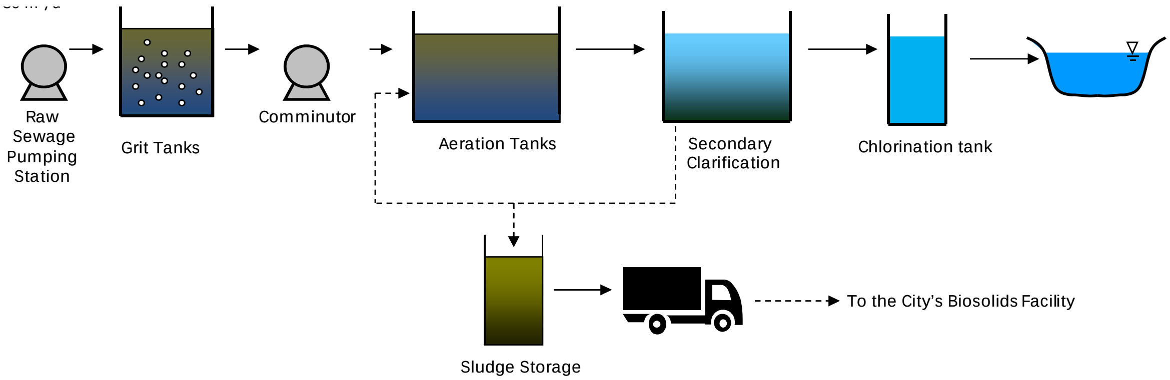

Figure 3-1 Azilda WWTP Process

3.3 KNOWN CHALLENGES

The Azilda Wastewater System has the following known challenges:

- Ground conditions in Azilda pose a challenge for maintaining buried infrastructure. In general, several maintenance holes require resetting annually due to ground conditions.
- From 2011 to 2014, there have been six bypasses at the Azilda WWTP, and one each at Landry, Laurier, and Main lift stations. A bypass occurred at each of these locations on October 16, 2014, while remaining bypasses were on different dates.
- City staff has also noticed that during the spring runoff and heavy rain events, flows to the Azilda WWTP commonly increase by approximately four times the average daily flows. This indicates severe I&I entering the system.
- There is no process redundancy at the Azilda WWTP, making it challenging to stop the flow to any one treatment process.

4 HISTORICAL FLOWS AND REVIEW OF OPERATIONAL DATA

Data reported in the 2009 to 2013 *Annual Reports* for the Azilda WWTP was reviewed and analyzed to determine average day and maximum day flows as well as review effluent parameters.

4.1 FLOW DATA

WWTP flow data from 2009 to 2013 was reviewed. Operational data was not available from the lift stations and so historical peak flow data could not be estimated.

The recorded average day and maximum day flows are summarized in Table 4-1 and plotted in Figure 4-1 below.

Table 4-1 Historical Wastewater Flow Data

YEAR	AVERAGE DAY FLOW (M^3/D)[1]	MAXIMUM DAY FLOW (M^3/D)[1]
2009	2,948	12,806
2010	2,129	12,160
2011	1,657	11,446
2012	1,553	5,606
2013	2,160	13,126

[1] *Annual Reports (2009 - 2013).*

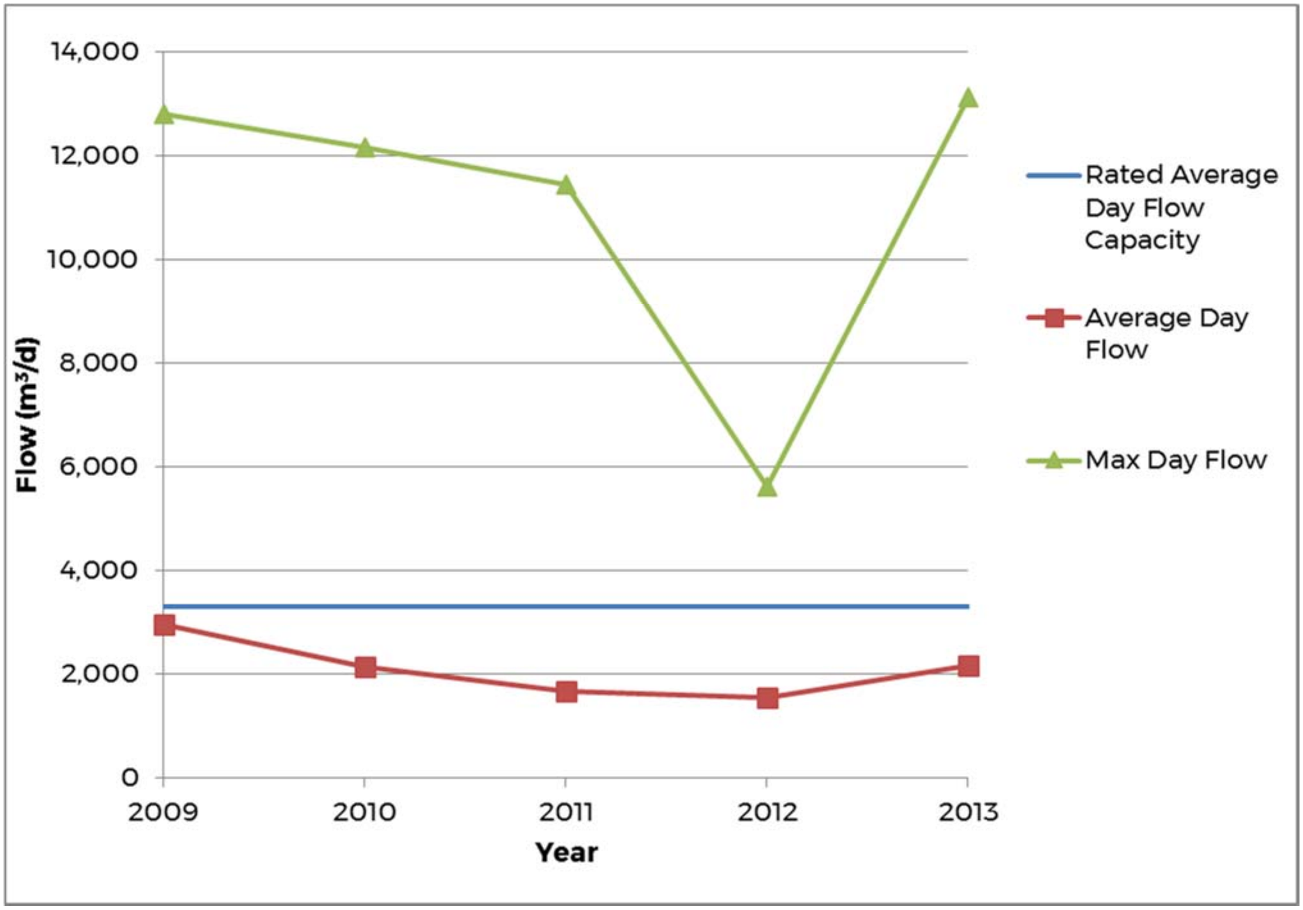

Figure 4-1 Historical Wastewater Flows at the Azilda Wastewater Treatment Plant

The relationship between the different flow regimes was analyzed to compare the maximum day peaking factors derived from historical data to those used in the *City's Engineering Design Manual* and those included in the *MOECC Guidelines.*

The average day flows to the WWTP have shown some variability over the 2009 to 2013 period, averaging 2,089 m^3/d. The variations in historical maximum day flows show no discernible trend when all data is considered. The 2012 maximum day flow was approximately half of the other years'. It is not known why flows were considerably lower in 2012. Assuming 2012 maximum day flow is an outlier, then maximum day flows are fairly consistent, ranging from 11,446 to 13,126 m^3/d, and averaging 12,385 m^3/d. This high max day to average day ratio suggests high rates of I&I in the catchment area.

The wide range in maximum day flows, but stable average day flows, indicates that the system is susceptible to variations in precipitation. This is also consistent with observations by City staff.

The highest maximum day to average day peaking factor based on the maximum day flow recorded in 2011 was 6.91. The average maximum day peaking factor from 2009 to 2013 was 5.76. The City's *Engineering Design Manual* and the *MOECC Guidelines* do not specify recommended maximum day factors and recommend using historical data when available. For future wastewater generation, the average peaking factor was used and based on the assumption that new developments would have less I&I due to more leak tight construction.

4.2 RAW WASTEWATER CHARACTERISTICS

The average raw wastewater characteristics from 2009 to 2012 are summarized in Table 4-2 below. Raw wastewater total Kjeldahl nitrogen (TKN) was reported only from 2009 to June 2010, and temperatures were not reported.

Table 4-2 Average Raw Wastewater Characteristics at the Azilda WWTP (2009-2012)

PARAMETER	AVERAGE VALUE
$CBOD_5$	115 mg/L
Suspended Solids	104 mg/L
Total Phosphorus	3.5 mg/L
TKN	24.9 mg/L
pH	7.3

Wastewater flows to the Azilda WWTP correspond mainly to residential uses, with contributions from commercial and industrial users, and dilution from inflow and infiltration.

4.3 EFFLUENT CRITERIA

The Azilda WWTP is operated in accordance with *MOECC Amended Environmental Compliance Approval (ECA) No. 3498-8XGJVK* dated October 29, 2012.

The *ECA* concentration and loading limits are summarized in Table 4-3.

Table 4-3 Azilda WWTP Effluent Limits and Objectives

EFFLUENT PARAMETER	CONCENTRATION LIMIT	LOADING LIMIT	CONCENTRATION OBJECTIVE
$CBOD_5$	10 mg/L	33 kg/d	7 mg/L
Total Suspended Solids (TSS)	10 mg/L	33 kg/d	7 mg/L
Total Phosphorus (TP)	0.6 mg/L	2.0 kg/d	0.28 mg/L
Total Ammonia Nitrogen (TAN)	5.0 mg/L	16.5 kg/d	2.0 mg/L
pH	6.0 to 9.5	-	6.5 to 8.5
E. coli	200 organisms/100 mL (Monthly Geometric Mean Density)	-	150 organisms/100 mL (Monthly Geometric Mean Density)

Compliance with the concentration and loading limits for $CBOD_5$ and TSS is based on the annual average concentration of each parameter based on all composite samples during any calendar year, whereas compliance for the TP is based on the monthly average concentration.

4.4 OPERATIONAL DATA

The general plant operation was reviewed against the Azilda WWTP ECA requirements and historical data provided in the Annual Reports from 2009 to 2012. Historical data is summarized in Table 4-3.

Table 4-4 Historical Effluent Concentrations

EFFLUENT PARAMETER	ANNUAL AVERAGE			
	2009	2010	2011	2012
CBOD5 (mg/L)	2.0	3.4	2.0	1.8
TSS (mg/L)	5.5	4.4	4.7	5.7
TP (mg/L)	0.32 (all months comply)	0.30 (all months comply)	0.32 (all months comply)	0.34 (all months comply)
pH	7.20	7.00	6.80	6.74
TAN (mg/L)	1.00	2.35	0.30	3.33
E. coli (organisms/100 mL)	60	10	21	19

The Azilda WWTP met all effluent limits. A capacity review of each unit process at the WWTP was not conducted. Instead, the rated capacity was considered the true capacity of the plant.

Biosolids from the Azilda WWTP are hauled to the Biosolids Facility at the Sudbury WWTP, and must meet specific quality requirements. This includes inorganic compounds such as plastics. However, the WWTP does not currently have fine screening to remove such compounds.

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 Development of 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 WASTEWATER DESIGN CRITERIA

The unit flow criteria for growth adopted for this assessment are shown in Table 5-1 below. These values were recommended in the *Population Projections and Development of Unit Rates Technical Memorandum* (WSP, 2014).

Note that the term "extraneous flows" is used interchangeably with "I&I flows".

Table 5-1 Wastewater System Design Criteria

CRITERIA	VALUE	REFERENCE
Average Day Residential Generation	400 L/cap/day	Average of historical values, rounded up to nearest 50 L/cap/day
Average Day Commercial and Institutional Generation	28 m^3/ha/d	Water unit rates, assuming a 1:1 correspondence
Average Day Industrial Generation	35 m^3/ha/d	Water unit rates, assuming a 1:1 correspondence
Average Extraneous Flow	7.5 m^3/ha/d	Peak from City's Engineering Design Manual and assuming a peaking factor of three
Peak Extraneous Flow	22.45 m^3/ha/d	City's Engineering Design Manual
Max Day Peaking Factor	5.76	Average of historical values

Residential average day flows are obtained by multiplying the residential unit rate by the service population. Similarly, average ICI flows were obtained by multiplying the corresponding unit rates to the areas of development, assuming 100% of the area is developed. Maximum day flows to the WWTP are obtained by multiplying the average day flow by the maximum day peaking factor.

5.2 DESIGN CRITERIA FOR WASTEWATER SYSTEM COMPONENTS AND OPERATION

5.2.1 WASTEWATER TREATMENT

Wastewater treatment facilities are rated for average day flows. Plant effluent limits and objectives are established in the C of A or ECA for each facility.

5.2.2 LIFT STATION PUMPING CAPACITY

The firm capacity of the lift station (with the largest pump out of service) must allow pumping of peak wet weather flows corresponding to its catchment area (MOECC, 2008).

Starting limitations on pump motors generally dictate the minimum size of a wet well. The wet well should be large enough to prevent pump motors from overheating due to frequent starting and stopping, but small enough to avoid long retention times leading to septicity and odor problems (Lin & Lee, 2001).

The station wet well shall be sized such that the number of pump starts per hour does not exceed the maximum value recommended by the pump manufacturer. In other words, the time between pump starts and stops (i.e. the pump cycle time) should not exceed that which results in a pump start frequency greater than that recommended by the pump manufacturer. Typically, submersible pumps can cycle four to 10 times per hour with a maximum cycle time not exceeding 30 minutes (Lin & Lee, 2001). A maximum value of four pump starts per hour was assumed to evaluate wet well sizing requirements.

5.2.3 SEWERS

The sewer system is typically sized to convey peak instantaneous (peak wet weather) flows. Sewage flows are made up of wastewater discharges from residential, commercial, institutional and industrial establishments, plus extraneous flow components from such sources as groundwater and surface runoff.

In addition to being able to convey peak flows, sufficient flow velocity should be maintained to transport the sewage solids to avoid deposition and the development of nuisance conditions under lower flow conditions. The minimum acceptable flow velocity in sewers is 0.6 m/s (City of Greater Sudbury, 2012).

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 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 Azilda population with wastewater servicing is projected to increase by 654 people by 2041 and 3,912 by Ultimate Buildout.

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

Table 6-1 Azilda Population Projections

SYSTEM	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

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 the table below.

Table 6-2 Azilda ICI Projections

	ICI DEVELOPMENT AREAS (HA)						
YEAR	2016	2021	2026	2031	2036	2041	BUILDOUT
Institutional	0	0	0	0	0	0	0
Commercial	0	0	1.44	0	0	0	0
Industrial	0	0	0	0	3.16	0	0
Total	**0**	**0**	**1.44**	**0**	**3.16**	**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 Azilda, 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 Azilda 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 WASTEWATER FLOW PROJECTIONS AND INFRASTRUCTURE NEEDS

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

$$\begin{aligned}&\textbf{Projected Average Day Generation}\\ &\quad = \textbf{Base Generation} + \textbf{Additional Residential Generation} + \textbf{Additional ICI Generation}\\ &\quad + \textbf{Average Extraneous Flow}\end{aligned}$$

The flows corresponding to the population growth forecasts to Ultimate Buildout are presented in Table 6-3 below.

Table 6-3 Flow Projections

YEAR	POPULATION	AVERAGE DAY FLOW (M^3/D)	MAXIMUM DAY FLOW (M^3/D)
Base	4,449	2,089	12,385
2016	4,624	2,304	13,273
2021	4,807	2,378	13,695
2026	4,959	2,499	14,393
2031	5,050	2,627	15,129
2036	5,099	2,780	16,014
2041	5,103	2,782	16,024
Ultimate	8,361	4,720	27,189

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 demand was calculated by applying the respective peaking factor to the average day demand. The maximum day demand for the base year was the historical average.

A desktop analysis of historical wastewater flows and future flow projections is included in **Appendix C**.

6.4.1 AZILDA WWTP CAPACITY

The rated average day capacity of the Azilda WWTP is 3,300 m^3/d, and is compared to the current and future flow projections on Figure 6-1 below.

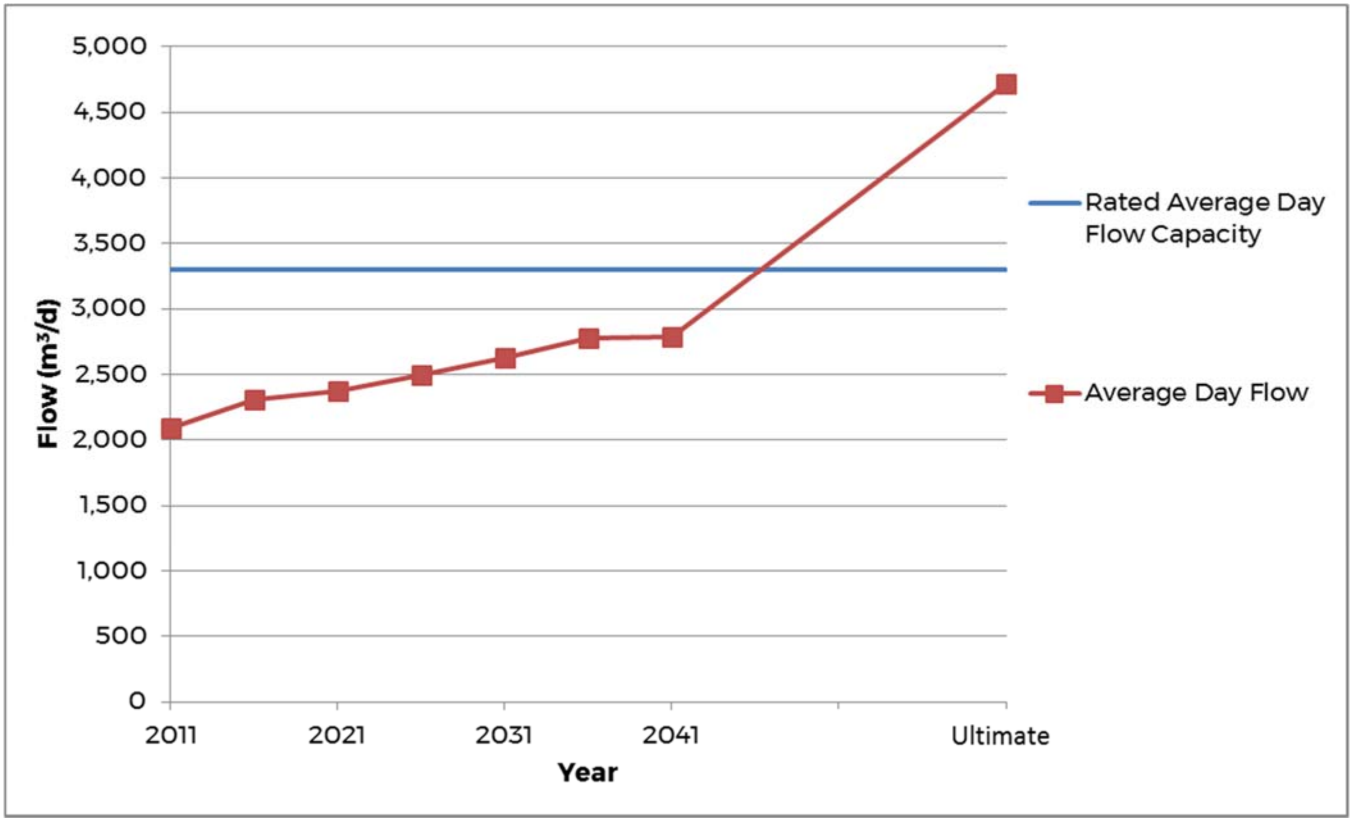

Figure 6-1 Wastewater Flow Projections Compared to Azilda WWTP Rated Capacity

As indicated in the above analysis, the Azilda WWTP can continue operating under its current capacity until approximately 2041. Upgrades will be needed to service future populations through to Ultimate Buildout.

However, it is understood that there have been numerous bypasses at the Azilda WWTP. The inability of the plant to handle peak flows needs to be addressed as part of the Master Plan.

6.4.2 SEWER NETWORK AND LIFT STATIONS

For each of the scenarios modeled, the system was checked for surcharging of sewers and capacity exceedance at the lift stations. The peak flows into each of the lift stations was determined from the computer simulations for the various planning scenarios and is presented in Table 6-4 below. The table also shows the design/rated flow for the pumps, their capacity based on drawdown tests and the computer simulated flow for comparison.

Table 6-4 Lift Station Peak Influent Flow Rates

	CURRENT FIRM CAPACITY	EXISITNG PEAK FLOW	2041 PEAK FLOW	ULTIMATE BUILDOUT
Landry	41.30	106.10	107.1	107.7
Laurier	90.1	296.10	311.20	311.20
Maple	17.8	2.01	2.057	2.139
Marier	10.8	14.7	14.8	15.1
Principale	32.9	12.1	12.5	16.8

When comparing the projected peak inflows against the design capacity for each of the lift stations, Landry, Laurier and Marier stations all require upgrades.

7 HYDRAULIC MODELLING

7.1 APPROACH

A basic sanitary model for the City of Greater Sudbury was received from the City. The model was created in Bentley's SewerGEMS by City staff. The model is an all pipe model of the sanitary network in these systems, but some critical information such as pipe data, invert elevations and lift station characteristics were missing. The model now includes this information as well as key vertical infrastructure in each system, including lift stations and treatment facilities.

The model was loaded with wet weather flow data. A water balance was completed to determine I&I rates for both dry and wet weather flow. The results from the water balance were compared against I&I rates developed through flow monitoring, and the greater of the two values, for each system, was used to load the model.

Current (2011) and future (2016-Ultimate Buildout, in 5 year increments) population data was added to the model using the City's planning data, summarized in previous sections of this report.

Future dry and wet weather flow scenarios were developed for each of the horizon years: 2016, 2021, 2026, 2031, 2036, 2041, and Ultimate Buildout. However, model results did not vary from 2016 to 2041; therefore, this report discusses findings for 2041 and Ultimate Buildout, compared against existing (2011).

7.2 MODELLING FINDINGS

The model was used to check sewer capacity and flow velocity. The majority of the sewers flow at less than 50% of the available capacity through to Ultimate Buildout under the wet weather flow condition.

Flow velocities through most of the Azilda sewer system are generally below the City's standard of 0.6 m/s. This is consistent through to Ultimate Buildout under the wet weather flow condition.

Maps in **Appendix B** illustrate the modeling results for the 2011, 2041, and Ultimate peak flow (wet weather flow plus 2-year storm) scenarios.

8 CONCLUSIONS

An assessment of the Azilda Wastewater System was completed to identify infrastructure requirements to service forecasted growth in the community.

The conclusions of the assessment are summarized below.

- The Azilda catchment area has a high max day to average day ratio suggesting high rates of I&I in the system.
- The WWTP is deemed to have sufficient average day capacity to service growth to 2041, but will need to be expanded for Ultimate Buildout.
- The plant currently does not have a rated maximum day flow.
- The Laurier, Landry and Marier LS all require upgrades to meet the peak capacity requirements.
- The majority of the sewers flow at less than 50% of the available capacity through to Ultimate Buildout under the wet weather flow condition.
- Flow velocities through most of the Azilda sewer system are generally below the City's standard of 0.6 m/s. This is consistent through to Ultimate Buildout under the wet weather flow condition.

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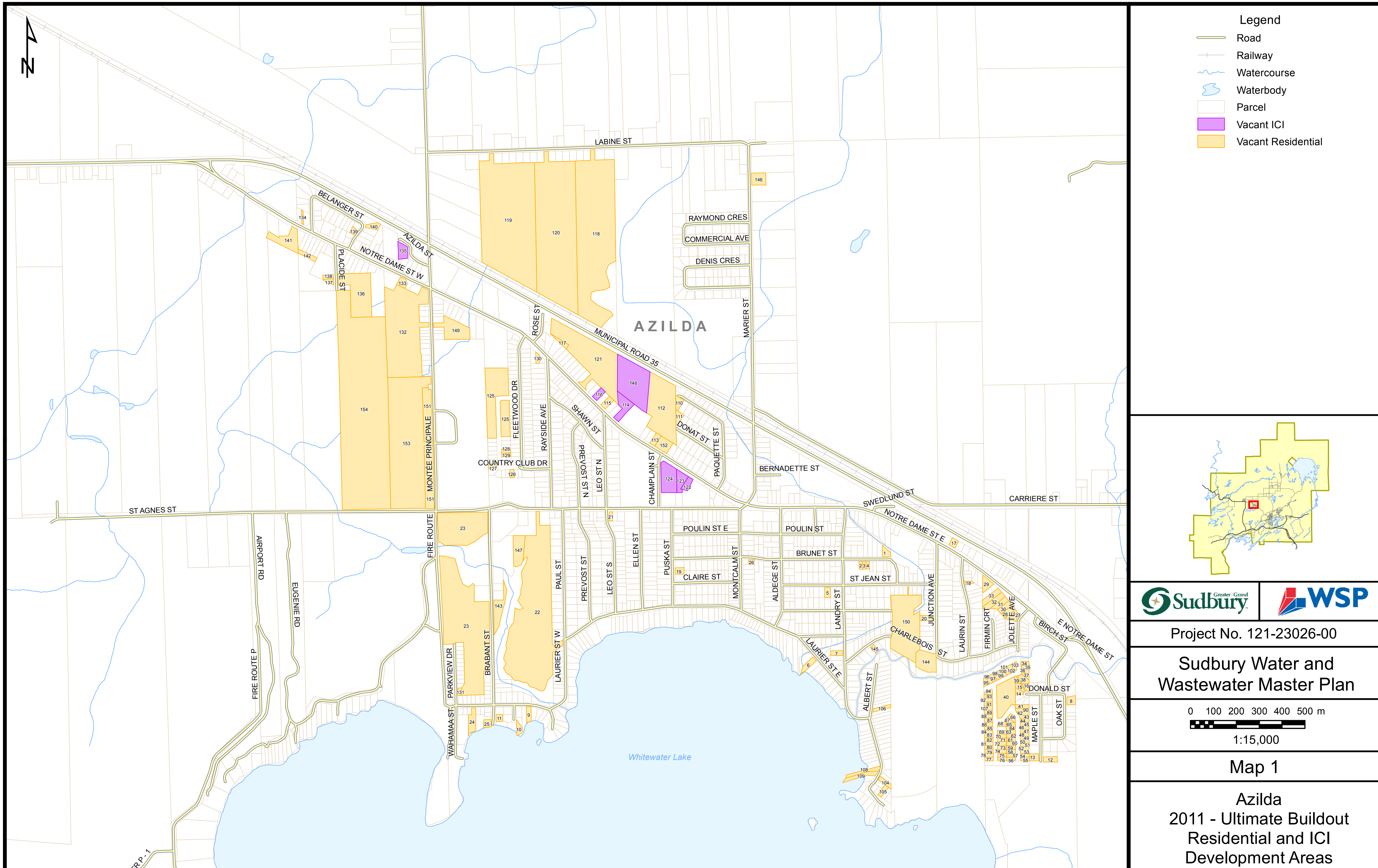
APPENDIX

A RESIDENTIAL AND ICI DEVELOPMENT AREAS



N
Legend
Road
Railway
Watercourse
Waterbody
Parcel
Vacant ICI
Vacant Residential
Greater Grand Sudbury
WSP
Project No. 121-23026-00
Sudbury Water and
Wastewater Master Plan
0 100 200 300 400 500 m
1:15,000
Map 1
Azilda
2011 - Ultimate Buildout
Residential and ICI
Development Areas
AZILDA
Whitewater Lake
LABINE ST
BELANGER ST
AZILDA ST
NOTRE DAME ST W
PLACIDE ST
RAYMOND CRES
COMMERCIAL AVE
DENIS CRES
MARIER ST
ROSE ST
MUNICIPAL ROAD 35
FLEETWOOD DR
RAYSIDE AVE
SHAWN ST
DONAT ST
PAQUETTE ST
MONTÉE PRINCIPALE
COUNTRY CLUB DR
PREVOST ST N
LEO ST N
CHAMPLAIN ST
BERNADETTE ST
SWEDLUND ST
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EUGENIE RD
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LEO ST S
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PUSKA ST
MONTCALM ST
CLAIRE ST
ALDEGE ST
ST JEAN ST
LANDRY ST
JUNCTION AVE
LAURIN ST
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ALBERT ST
DONALD ST
MAPLE ST
OAK ST
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BRABANT ST
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WAHAMAA ST
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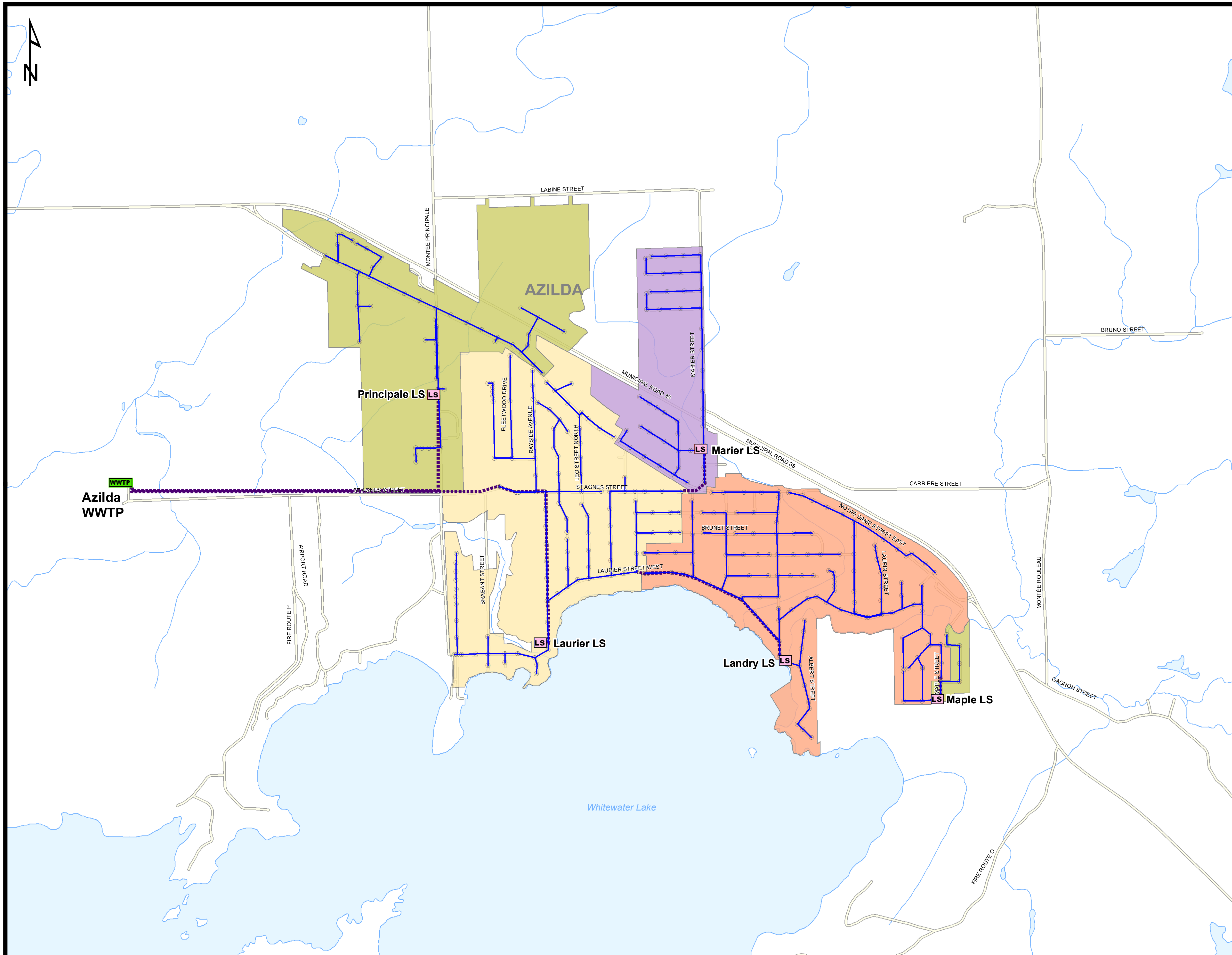
APPENDIX

B WASTEWATER MODEL RESULTS



Legend

- LS Lift Station
- WWTP Sanitary Treatment Plant
- Manhole
- Waterbody
- Road
- Watercourse
- Sanitary Sewer Conduit
- Forcemain

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December 09 2016

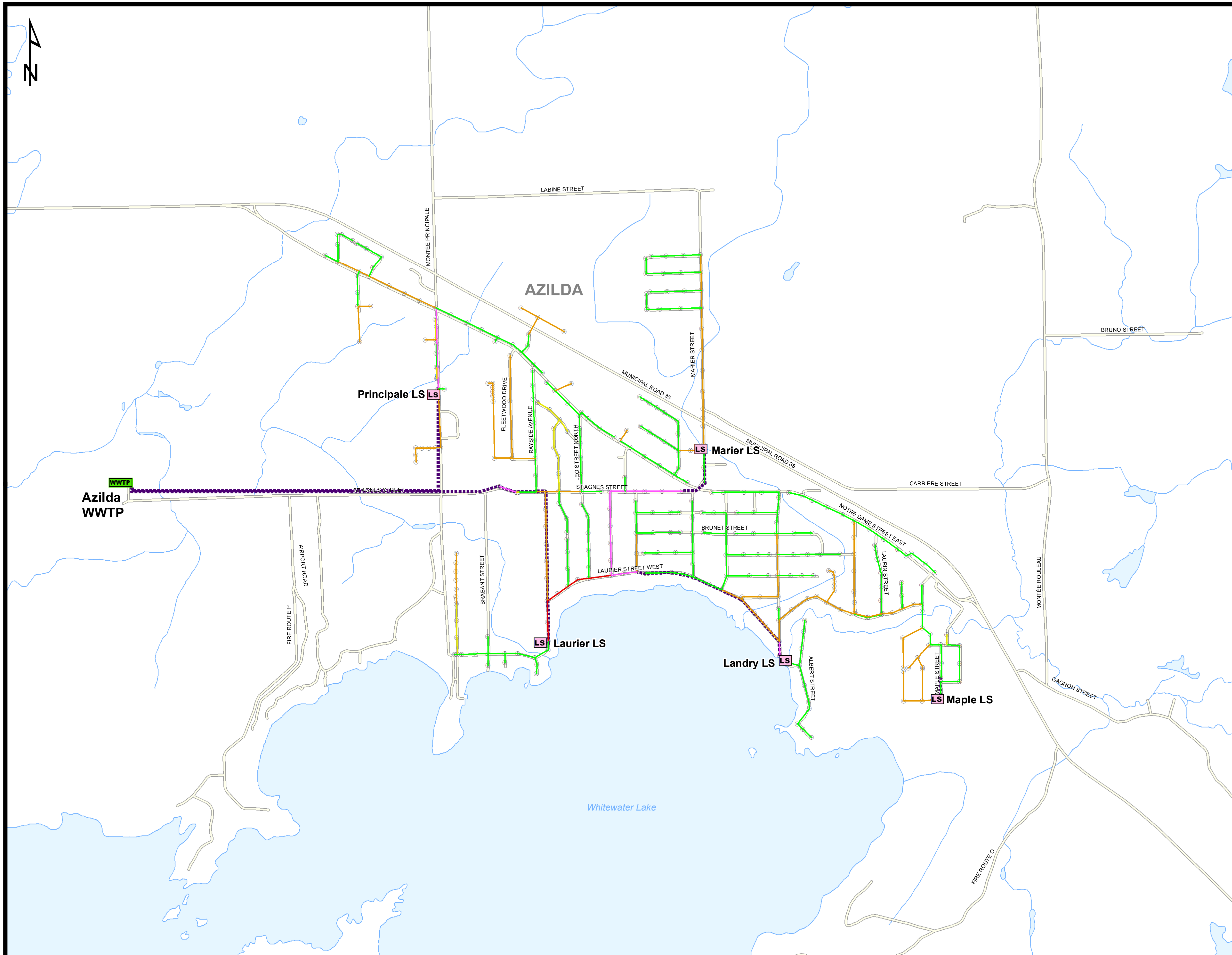
Sudbury Water and Wastewater Master Plan

0 200 400 600 800 1,000 m

1:20,000

Map 1.1

Azilda Wastewater System - Sewershed Map



Legend

- LS Lift Station
- WWTP Sanitary Treatment Plant
- Manhole
- Waterbody
- Road
- Watercourse
- Forcemain

Conduit: Diameter (mm)

- <= 150
- <= 200
- <= 300
- <= 450
- <= 600
- <= 750
- > 750

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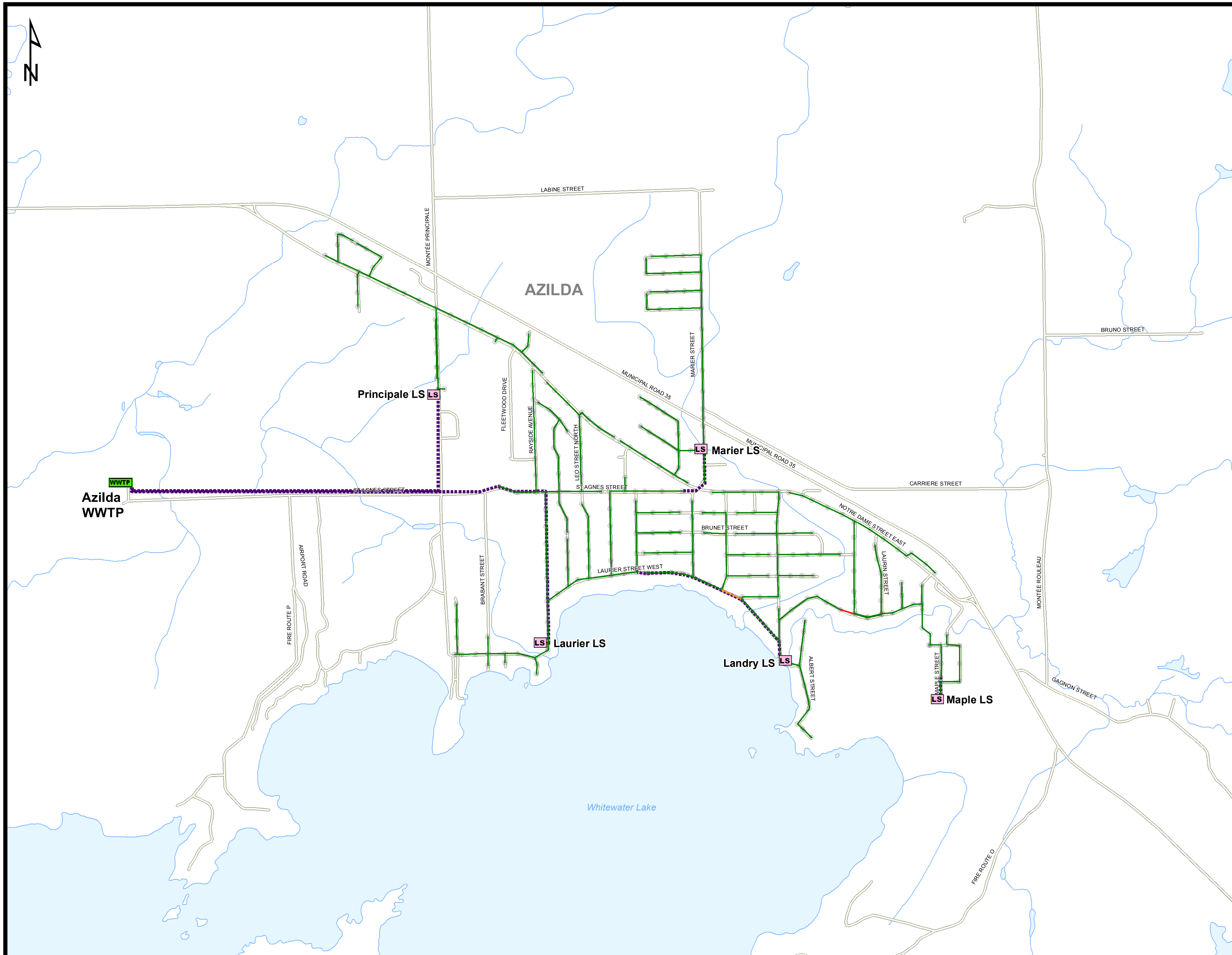
Map 1.2

Azilda Wastewater System - Conduit Diameter



Legend

- LS Lift Station
- WWTP Sanitary Treatment Plant
- Manhole
- Waterbody
- Road
- Watercourse
- Forcemain

Conduit: Flow/Capacity (Design) (%)

- <= 50
- <= 80
- <= 100
- > 100

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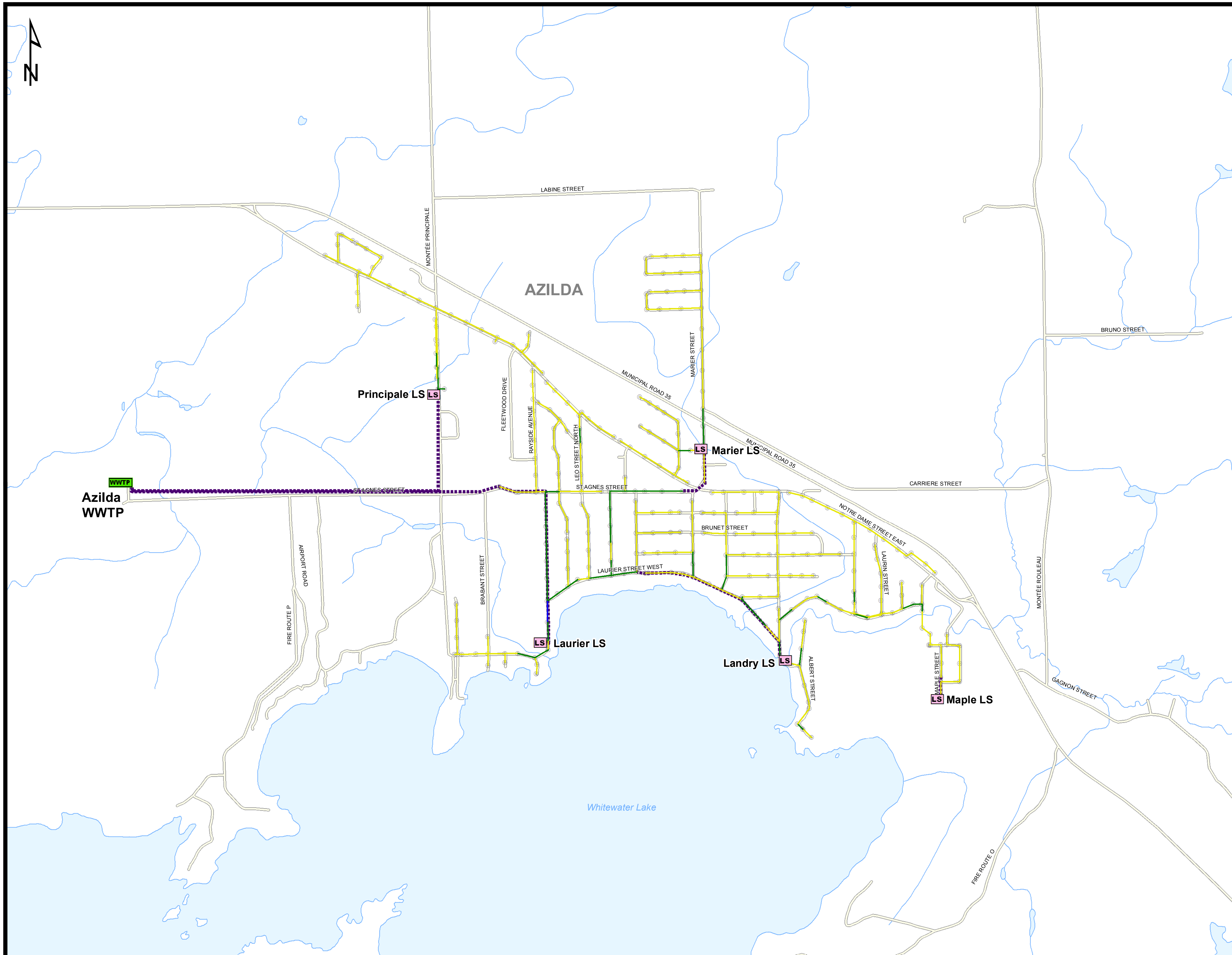
Map 1.3.1

Azilda Wastewater System - Conduit Flow/Capacity (Design)

Scenario: 2011WWF_2yrStorm



Legend

- LS Lift Station
- WWTP Sanitary Treatment Plant
- Manhole
- Waterbody
- Road
- Watercourse
- Forcemain

Conduit: Velocity (m/s)

- <= 0.6
- <= 1.5
- <= 3.0
- > 3.0

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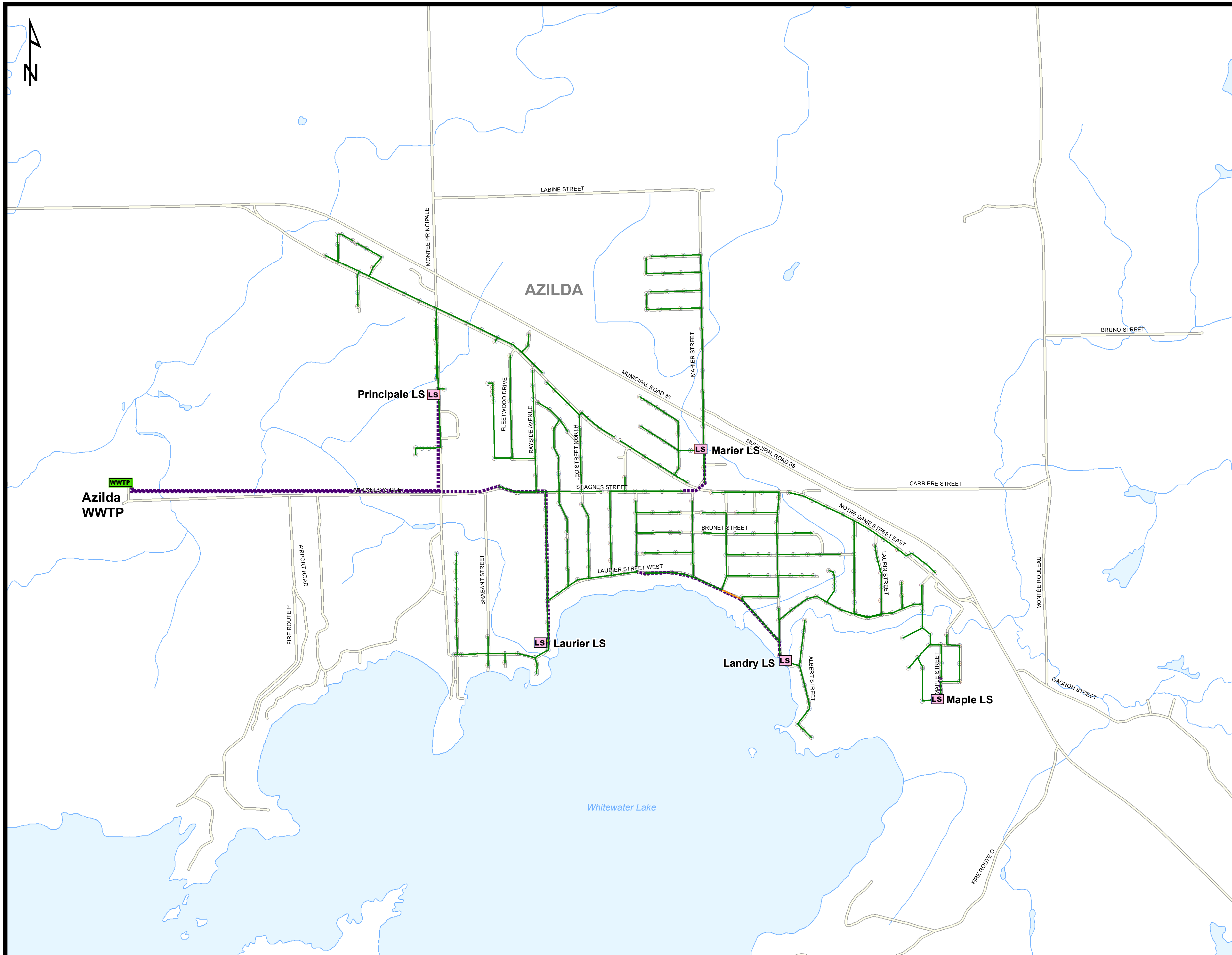
Map 1.3.2

Azilda Wastewater System - Conduit Velocity

Scenario: 2011WWF_2yrStorm



Legend

- LS Lift Station
- WWTP Sanitary Treatment Plant
- Manhole
- Waterbody
- Road
- Watercourse
- Forcemain

Conduit: Flow/Capacity (Design) (%)

- <= 50
- <= 80
- <= 100
- > 100

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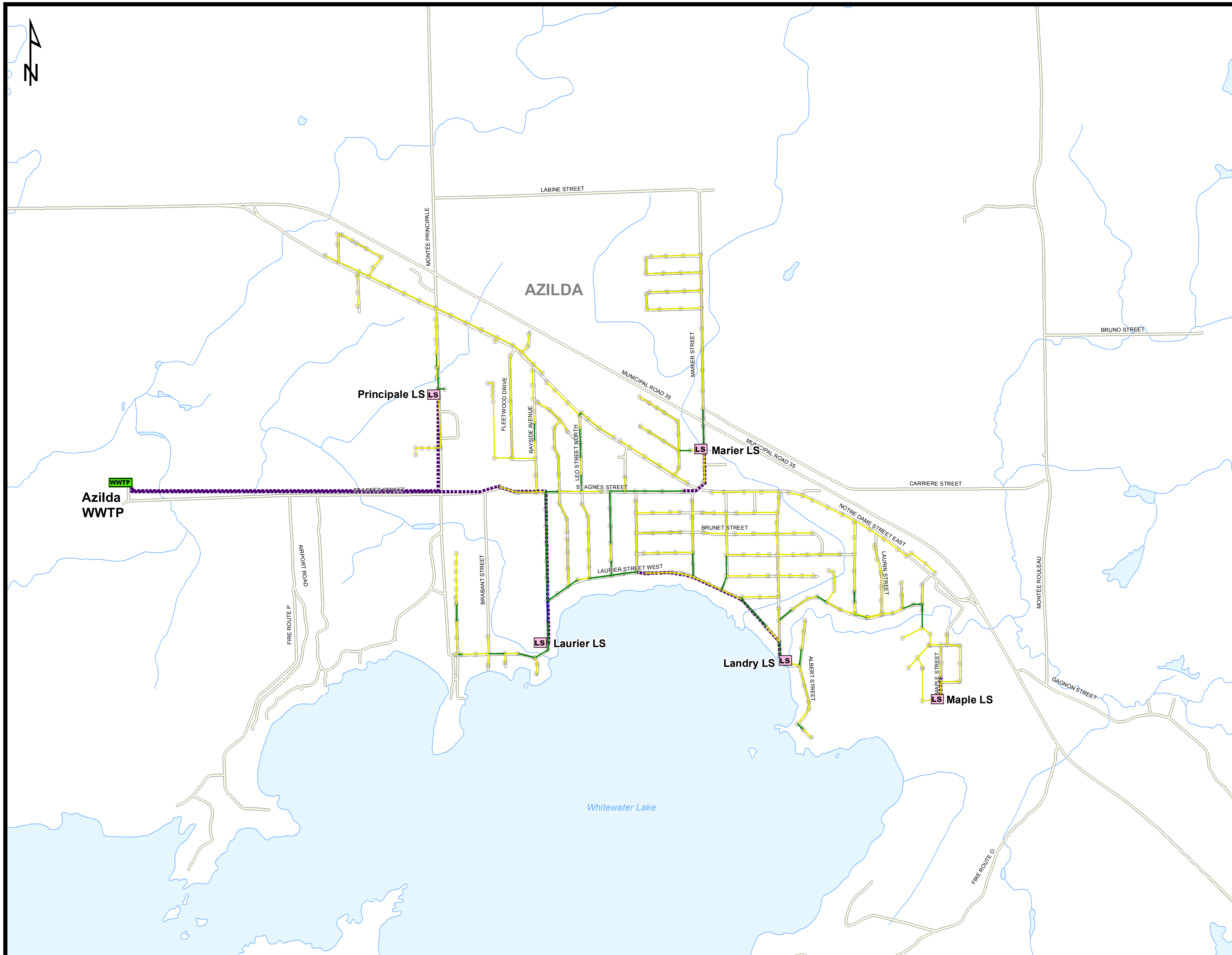
1:20,000

Map 1.4.1

Azilda Wastewater System - Conduit Flow/Capacity (Design)

Scenario: 2041WWF_2yrStorm



Legend

- LS Lift Station
- WWTP Sanitary Treatment Plant
- Manhole
- Waterbody
- Road
- Watercourse
- Forcemain

Conduit: Velocity (m/s)

- <= 0.6
- <= 1.5
- <= 3.0
- > 3.0

Greater Sudbury Grand | WSP

Project No. 121-23026-00

December 09 2016

Sudbury Water and Wastewater Master Plan

0 200 400 600 800 1,000 m

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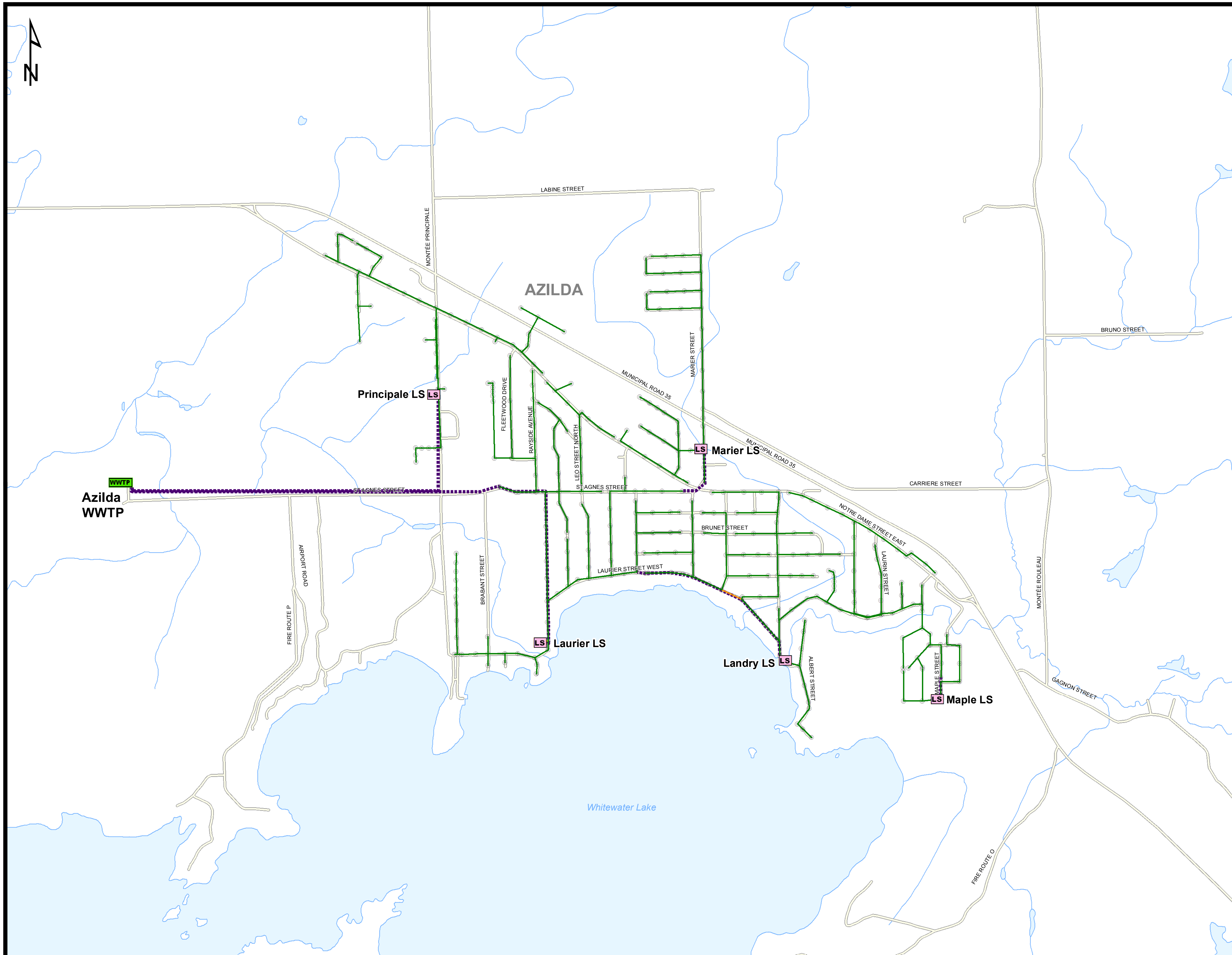
Map 1.4.2

Azilda Wastewater System - Conduit Velocity

Scenario: 2041WWF_2yrStorm



Legend

- LS Lift Station
- WWTP Sanitary Treatment Plant
- Manhole
- Waterbody
- Road
- Watercourse
- Forcemain

Conduit: Flow/Capacity (Design) (%)

- <= 50
- <= 80
- <= 100
- > 100

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Sudbury Water and Wastewater Master Plan

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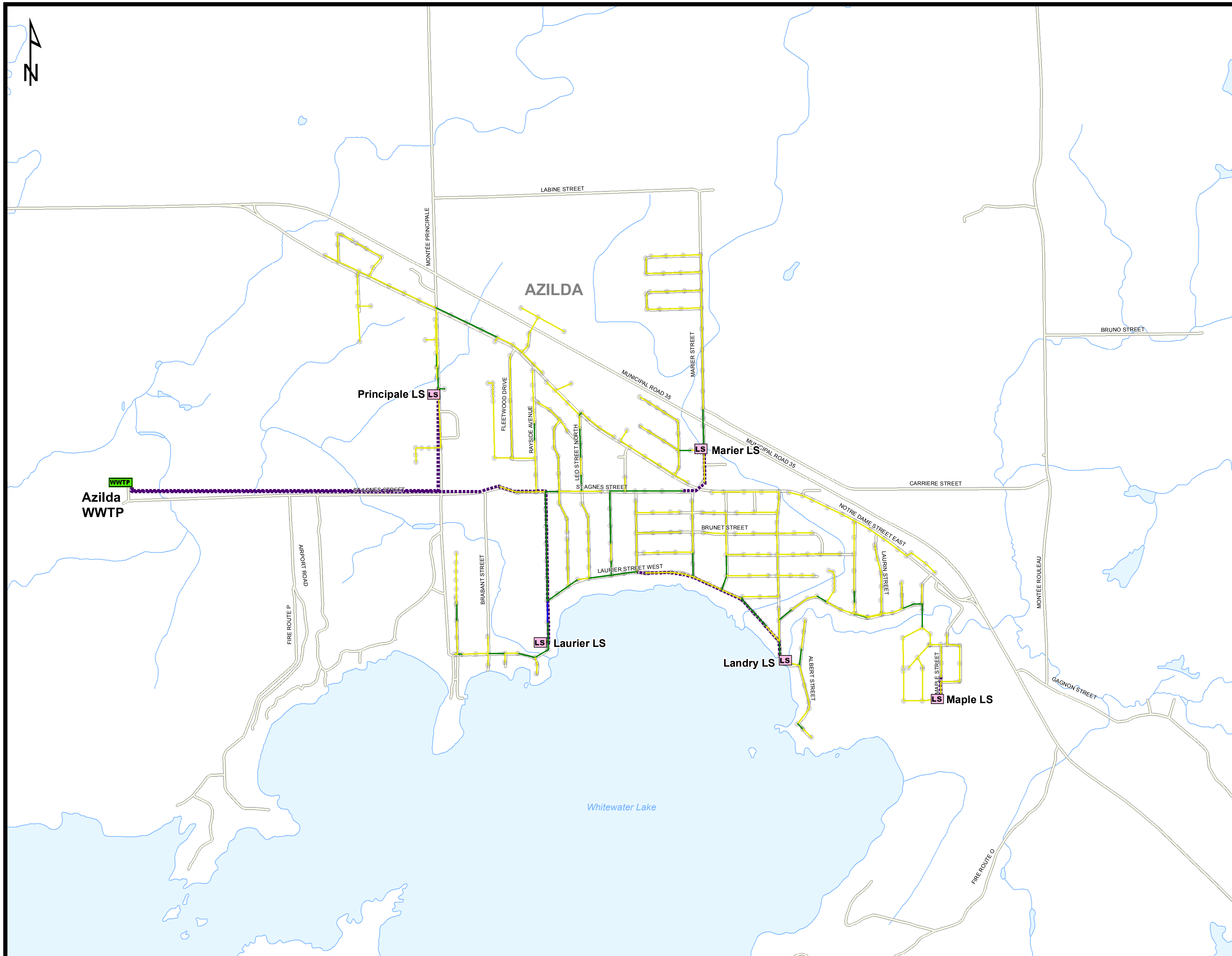
Map 1.5.1

Azilda Wastewater System - Conduit Flow/Capacity (Design)

Scenario: ULTWWF_2yrStorm



Legend

- LS Lift Station
- WWTP Sanitary Treatment Plant
- Manhole
- Waterbody
- Road
- Watercourse
- Forcemain

Conduit: Velocity (m/s)

- <= 0.6
- <= 1.5
- <= 3.0
- > 3.0

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Sudbury Water and Wastewater Master Plan

0 200 400 600 800 1,000 m

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Map 1.5.2

Azilda Wastewater System - Conduit Velocity

Scenario: ULTWWF_2yrStorm

APPENDIX

C WASTEWATER GENERATION AND CAPACITY ASSESSMENTS

Azilda - Wastewater Flow Forecasts

Parameter	Units	2009	2010	2011	2012	2013	Summary	Design Criterion	2016	2021	2026	2031	2036	2041	Ultimate Buildout	Comments
Average Day Flow	(m^3/d)	2,948	2,129	1,657	1,553	2,160	2,089	2,089								From Annual Reports
Max Day Flow	(m^3/d)	12,806	12,160	11,446	5,606	13,126	12,385	12,385								From Annual Reports; the average assumes that the 2012 data is an outlier and is omitted from the analysis.
Max Day Factor		4.34	5.71	6.91	3.61	6.08	5.76	5.76								Calculated - Max Day Flow divided by Average Day Flow. 2012 data was omitted since the Maximum Day Flow was much lower than the remaining years.
Peak Flow	(m^3/d)	Not Available						0								Peak hour flows were not available
Peak Flow	(L/s)							0								
Population (Existing Areas)		4,449	4,449	4,449	4,449	4,449	4,449	4,449	4,449	4,449	4,449	4,449	4,449	4,449	4,449	
Population (Growth Areas)									175	358	510	601	650	654	3,912	
Total Population									4624	4807	4959	5050	5099	5103	8361	Total Population (Hemson)
Residential (ha)									19.37	19.37	20.61	32.81	32.81	32.81	117.72	ICI development areas were assigned to planning years based on the stage of the application. Draft Approved were assigned to 2016, Legal Lots of Record to 2026, and Designated Developable to 2036. Areas are cumulative and carry from the development year, all the way through to Ultimate Buildout
Institutional (ha)																
Commercial (ha)											1.44	1.44	1.44	1.44	1.44	
Industrial (ha)													3.16	3.16	3.16	
ICI (ha)									0.00	0.00	1.44	1.44	4.60	4.60	4.60	
Total (ha)									19.37	19.37	22.05	34.25	37.41	37.41	122.32	
Ratio of Residential to Total Customers		0.80	0.80	0.80	0.80	0.80	0.80									This ratio is based on Water Billing Records for the area and is an approximation of the residential portion of demand.
Residential Share of Average Day Demand	(m^3/d)	2351	1698	1322	1239	1723	1667									
Residential Flow Unit Rate	(m^3/cap/d)	0.529	0.382	0.297	0.278	0.387	0.375	0.400								
Average Institutional Flow Unit Rate	(m^3/ha/d)							28.0								CGS design standards do not suggest average rates for institutional and commercial, but require that rates be developed based on usage. In this case, water demands rates were used and assume a conservative 1:1 ratio of water to sewage.
Average Commercial Flow Unit Rate	(m^3/ha/d)							28.0								CGS design standards do not suggest average rates for institutional and commercial, but require that rates be developed based on usage. In this case, water demands rates were used and assume a conservative 1:1 ratio of water to sewage.
Average Industrial Flow Unit Rate	(m^3/ha/d)							35.0								CGS design standards average rate for light industrial of 35 m^3/ha/d.
Average Extraneous Flow Unit Rate	(m^3/ha/d)							7.48								From CGS Design Standards, peak rate for new developments divided by an assumed peaking factor of 3. This factor would be applied only to new developments, which are assumed to be leak-tight, and have minimal extraneous flow.

Parameter	2016	2021	2026	2031	2036	2041	Ultimate Buildout	Comments
Average Residential Flows (m^3/d) - Existing	2,089	2,089	2,089	2,089	2,089	2,089	2,089	This includes all contribution from existing ICI and infiltration. The base flow was assumed to be the average day flow to the plant for the 2011-2013 period.
Average Residential Flows (m^3/d)	70	143	204	241	260	262	1,565	Obtained by multiplying the projected population growth by the unit rate.
Average Institutional Flow (m^3/d)	0	0	0	0	0	0	0	Institutional growth area multiplied by unit flow rate.
Average Commercial Flow (m^3/d)	0	0	40	40	40	40	40	Commercial growth area multiplied by unit flow rate.
Average Industrial Flow (m^3/d)	0	0	0	0	111	111	111	Industrial growth area multiplied by unit flow rate.
Average Extraneous Flow (m^3/day)	145	145	165	256	280	280	915	
Average Day Flow (m^3/d)	2,304	2,378	2,499	2,627	2,780	2,782	4,720	
Max Day Flow (m^3/d)	13,273	13,695	14,393	15,129	16,014	16,024	27,189	

ALTERNATIVE CALCULATION METHOD

Per Capita Flow (m3/cap/day)	0.663	0.479	0.372	0.349	0.485	0.470	0.470

	2016	2021	2026	2031	2036	2041	Ultimate Buildout
Average Day Flow (m^3/d)	2171.625	2257.521913	2329.042	2371.826	2394.484	2396.692	3926.41819
Max Day Flow (m^3/d)	12,509	13,003	13,415	13,662	13,792	13,805	22,616

Comments

Multiplying the total population by the consolidated per capita flow factor.

SENSITIVITY ANALYSIS

Analyze sensitivity of forecasted flows to unit rate

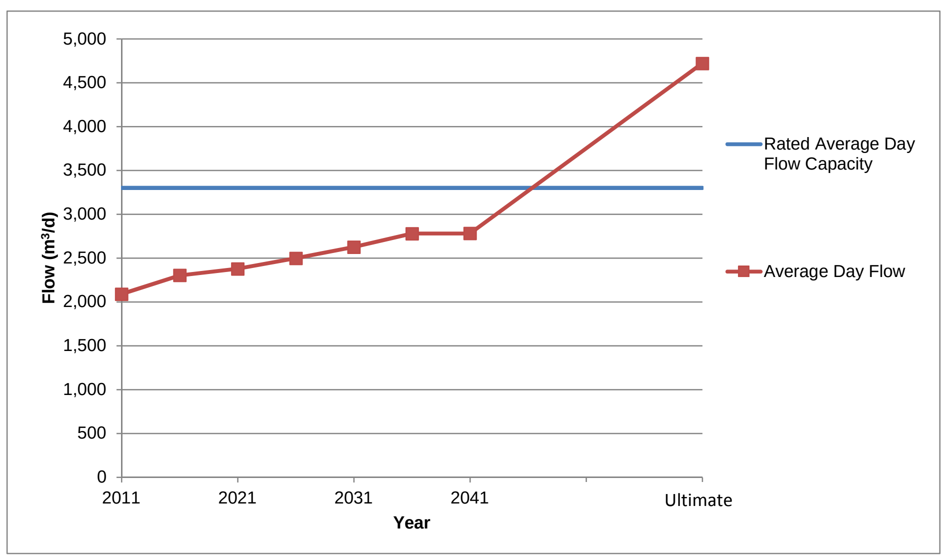
	Unit Rate (m^3/cap/d)	Average Day Flow (m^3/d) 2016	2021	2026	2031	2036	2041	Ultimate Buildout 2061
Base Scenario - Residential Historical Maximum	0.375	2,304	2,378	2,499	2,627	2,780	2,782	4,720
Combined Historical Maximum	0.470	2,317	2,402	2,534	2,668	2,825	2,828	4,993
City Standards	0.360	2,297	2,363	2,478	2,603	2,754	2,756	4,564

Analyze sensitivity of forecasted flows to max day factor

	Max Day Peaking Factor	Max Day Flow (m^3/d) 2016	2021	2026	2031	2036	2041	Ultimate Buildout 2061
Base Scenario - Historical Max	5.58	12,850	13,258	13,935	14,647	15,503	15,514	26,323
Historical Average	6.91	13,273	13,695	14,393	15,129	16,014	16,024	27,189

CAPACITY CHECK

	2011	2016	2021	2026	2031	2036	2041	Ultimate Buildout 2061
Rated Average Day Flow Capacity (m^3/d)	3,300	3,300	3,300	3,300	3,300	3,300	3,300	3,300
Average Day Flow (m^3/d)	2,089	2,304	2,378	2,499	2,627	2,780	2,782	4,720
Maximum Day Flow (m^3/d)	12,385	13,273	13,695	14,393	15,129	16,014	16,024	27,189



CITY OF GREATER SUDBURY WATER AND WASTEWATER MASTER PLAN

CAPREOL WASTEWATER SYSTEM GAP ANALYSIS AND STATUS QUO REPORT

CITY OF GREATER SUDBURY

DRAFT

PROJECT NO.: 121-23026-00
DATE: MARCH 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 existing Capreol Wastewater System. Based on population growth projections and design criteria discussed in the *Population Projections and Development of Unit Rates Technical Memorandum* (WSP, 2014), 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 Capreol Wastewater System would continue to be a stand-alone system. Any potential interconnections between Capreol 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

Capreol is located along Old Highway 69, North of Valley East. Mapping in **Appendix A** shows the Capreol 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 *Population Projections and Development of Unit Rates 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 Capreol population is expected to increase from 3,392 in 2011 to 3,450 by 2041 and 4,716 by Ultimate Buildout. ICI growth is expected to be a mix of institutional, commercial and industrial. Growth is discussed further in Section 6.1.

3 OVERVIEW OF EXISTING SYSTEM

The Capreol Wastewater System services the community of Capreol and includes the Capreol Lagoon, as well as two lift stations and the sewer network. The collection system consists of approximately 21.72 km of sewers and forcemains. On review of bypass event data from 2011 to 2014, there have not been any bypasses in Capreol during this time period. The Capreol Wastewater System is shown in **Appendix B**.

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

3.1 LIFT STATIONS

Table 3-1 below provides a summary of the main features of the lift stations.

Table 3-1 Capreol Sewage Lift Stations

LIFT STATION	YEAR CONSTRUCTED[1]	LAST UPGRADED[2]	WET WELL VOLUME (M^3)[1]	PUMPING STATION CAPACITY AND FORCEMAIN INFORMATION[2]
Lloyd	1976	None	4.4	Two submersible pumps with a firm design capacity of 11.42 L/s 596 m long, 150 mm diameter forcemain of unknown material
Vermilion	c.1961 (per first preliminary approval)	1992	Not known	Two dry pit pumps with a firm design capacity of 100 L/s 152 m long, 250 mm diameter forcemain of unknown material

[1] Obtained or estimated from dimensions found in as-built and record, assuming water level does not exceed the High Water Alarm Level or, in absence of this alarm level, the inlet sewer invert.
[2] Obtained from the City's Wastewater Lift Stations Operations Manual and station as-built drawings.

3.2 CAPREOL LAGOON

The Capreol Lagoon is owned and operated by the City of Greater Sudbury and is located at Lot 11, Concession 6 in Capreol. The lagoon is a two-cell waste stabilization lagoon, operated as a continuous discharge exfiltration system, discharging to the Vermilion River (MOECC, 2001). The treatment process is illustrated schematically below.

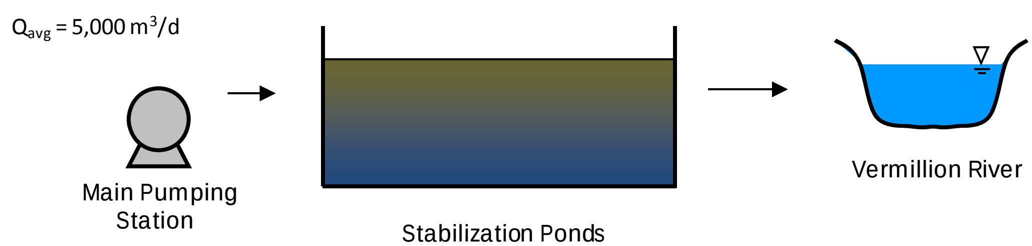

Figure 3-1 Capreol Lagoon System

3.3 KNOWN CHALLENGES

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

— The residential area north of Sellwood Avenue and west of Dennie Street does not have a stormwater collection system, and is prone to basement and overland flooding, as indicated by City staff. In this and other areas of Capreol, some catch basins are connected to the sanitary system, effectively operating as a combined system.

— There is a reverse grade sewer on the northwest end of Oakwood Avenue, necessitating monthly flushing. The area surrounding the intersection of Dennie Street near the railway track is at a low ground elevation and is therefore a source of inflow and infiltration into the system.

— City staff have indicated that the Capreol area is regularly flooded, including flooding in the collection system as well as overland flooding due to blocked catch basins.

4 HISTORICAL FLOWS AND REVIEW OF OPERATIONAL DATA

Data reported in the 2009 to 2013 *Annual Reports* for the Capreol Lagoon was reviewed and analyzed to determine average day and maximum day flows as well as review effluent parameters.

4.1 FLOW DATA

Lagoon flow data from 2009 to 2013 was reviewed. Operational data was not available from the lift stations and so historical peak flow data could not be estimated.

The recorded average day flows are summarized in Table 4-1 and plotted in Figure 4-1 below. Maximum day flows were not available.

Table 4-1 Historical Wastewater Flow Data

YEAR	AVERAGE DAY FLOW (M^3/D)[1]
2009	2,805
2010	1,840
2011	2,708
2012	2,308
2013	2,387

[1] *Annual Reports (2009 - 2013).*

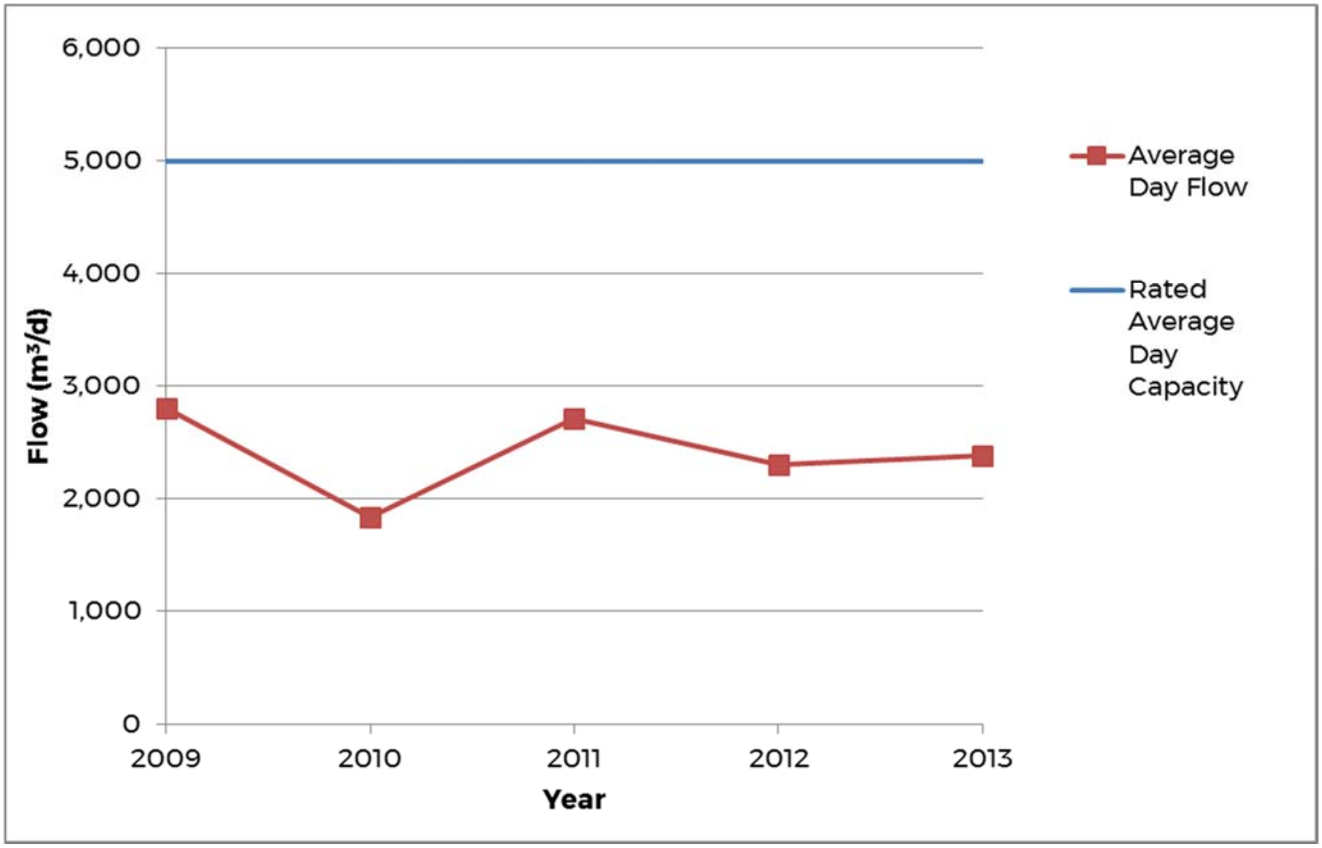

Figure 4-1 Historical Wastewater Flows at the Capreol Lagoon

The relationship between the different flow regimes was analyzed to compare the peaking factors derived from historical data to those used in the *City's Engineering Design Manual* and those included in the *MOECC Guidelines*.

The average day flows to the lagoon have shown some variability over the 2009 to 2013 period, averaging 2,410 m^3/d.

Maximum day to average day peaking factors could not be calculated since maximum day flow records were not available. The City's *Engineering Design Manual* and the *MOECC Guidelines* do not specify recommended maximum day factors and recommend using historical data when available. For future wastewater generation, the maximum day peaking factor was assumed to match that of the neighbouring Valley East Wastewater System.

4.2 RAW WASTEWATER CHARACTERISTICS

The average raw wastewater characteristics from 2009 to 2012 are summarized in the table below. Temperatures and pH were not reported.

Table 4-2 Average Raw Wastewater Characteristics (2009-2012)

PARAMETER	AVERAGE VALUE
$CBOD_5$	83 mg/L
Suspended Solids	93 mg/L
Total Phosphorus	3.0 mg/L
TKN	17.5 mg/L

Wastewater flows to the Capreol Lagoon correspond mainly to residential uses, with contributions from commercial and industrial users.

4.3 EFFLUENT CRITERIA

The Capreol Lagoon is operated in accordance with *MOECC Certificate of Approval for Sewage No. 8214-4UVPUZ* dated March 19, 2001.

The *C of A* concentration limits are summarized in the table below.

Table 4-3 Capreol Lagoon Effluent Limits and Objectives

EFFLUENT PARAMETER	CONCENTRATION LIMIT	CONCENTRATION OBJECTIVE
$CBOD_5$	30 mg/L	25 mg/L
Total Suspended Solids (TSS)	40 mg/L	30 mg/L
Total Phosphorus	1.38 mg/L	N/A

Compliance with the concentration and loading limits for $CBOD_5$, TSS, and Total Phosphorus is based on the annual average concentration of each parameter based on all composite samples during any calendar year.

4.4 OPERATIONAL DATA

The general lagoon operation was reviewed against the Capreol Lagoon *C of A* requirements and historical data provided in the Annual Reports from 2009 to 2012. Historical data is summarized in the table below.

Table 4-4 Historical Effluent Concentrations

HEADING	ANNUAL AVERAGE			
SUB-HEADING	2009	2010	2011	2012
$CBOD_5$ (mg/L)	20.2	27.0	21.5	25.5
TSS (mg/L)	19.2	25.5	31.1	28.5
Total Phosphorus (mg/L)	1.49	1.73	1.54	1.98

The Capreol Lagoon met concentration limits for $CBOD_5$ and TSS in all years. However, the lagoon consistently exceeds Total Phosphorus limits.

A capacity review of each unit process at the lagoon was not conducted. Instead, the rated capacity was considered the true capacity.

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 Development of 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 WASTEWATER DESIGN CRITERIA

The unit flow criteria for growth adopted for this assessment are shown in Table 5-1 below. These values were recommended in the *Population Projections and Development of Unit Rates Technical Memorandum* (WSP, 2014).

Note that the term "extraneous flows" is interchangeable with "I&I flows".

Table 5-1 Wastewater System Design Criteria

CRITERIA	VALUE	REFERENCE
Average Day Residential Generation	650 L/cap/day	Average of historical values, rounded up to nearest 50 L/cap/day
Average Day Commercial and Institutional Generation	28 m^3/ha/d	Water unit rates, assuming a 1:1 correspondence
Average Day Industrial Generation	35 m^3/ha/d	Water unit rates, assuming a 1:1 correspondence
Average Extraneous Flow	11.2 m^3/ha/d	Peak from City's Engineering Design Manual and assuming a peaking factor of three
Peak Extraneous Flow	33.7 m^3/ha/d	City's Engineering Design Manual
Max Day Peaking Factor	2.69	Historical data was not available; matched with nearby Valley East

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

Maximum day flows to the lagoon are obtained by multiplying the average day flow by the maximum day peaking factor.

5.2 DESIGN CRITERIA FOR WASTEWATER SYSTEM COMPONENTS AND OPERATION

5.2.1 WASTEWATER TREATMENT

Wastewater treatment facilities are rated for average day flows. Plant effluent limits and objectives are established in the C of A or ECA for each facility.

5.2.2 LIFT STATION PUMPING CAPACITY

The firm capacity of the lift station (with the largest pump out of service) must allow pumping of peak wet weather flows corresponding to its catchment area (MOECC, 2008).

Starting limitations on pump motors generally dictate the minimum size of a wet well. The wet well should be large enough to prevent pump motors from overheating due to frequent starting and stopping, but small enough to avoid long retention times leading to septicity and odor problems (Lin & Lee, 2001).

The station wet well shall be sized such that the number of pump starts per hour does not exceed the maximum value recommended by the pump manufacturer. In other words, the time between pump starts and stops (i.e. the pump cycle time) should not exceed that which results in a pump start frequency greater than that recommended by the pump manufacturer. Typically, submersible pumps can cycle four to 10 times per hour with a maximum cycle time not exceeding 30 minutes (Lin & Lee, 2001). A maximum value of four pump starts per hour was assumed to evaluate wet well sizing requirements.

5.2.3 SEWERS

The sewer system is typically sized to convey peak instantaneous (peak wet weather) flows. Sewage flows are made up of wastewater discharges from residential, commercial, institutional and industrial establishments, plus extraneous flow components from such sources as groundwater and surface runoff.

In addition to being able to convey peak flows, sufficient flow velocity should be maintained to transport the sewage solids to avoid deposition and the development of nuisance conditions under lower flow conditions. The minimum acceptable flow velocity in sewers is 0.6 m/s (City of Greater Sudbury, 2012).

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 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 *Population Projections and Development of 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 Capreol population with wastewater servicing is projected to increase by 58 people by 2041 and 1,324 people by Ultimate Buildout.

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

Table 6-1 Capreol Population Projections

SYSTEM	2011	2016	2021	2026	2031	2036	2041	ULTIMATE BUILDOUT
Capreol	3,392	3,396	3,412	3,435	3,447	3,456	3,450	4,716

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 the table below.

Table 6-2 Capreol ICI Projections

	ICI DEVELOPMENT AREAS (HA)						
LAND USE	2016	2021	2026	2031	2036	2041	BUILDOUT
Institutional	0	0	0.36	0	0	0	0
Commercial	0	0	3.18	0	0.53	0	0
Industrial	0	0	0	0	2.94	0	0
Total	**0**	**0**	**3.54**	**0**	**3.47**	**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 PHASING OF FUTURE GROWTH

Growth areas were allocated based on population projections for individual developments and the overall target growth population projections for Capreol 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 *Population Projections and Development of Unit Rates Technical Memorandum* (WSP, 2014).

6.3 FUTURE WASTEWATER FLOW PROJECTIONS AND INFRASTRUCTURE NEEDS

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

$$\begin{aligned}&\textbf{Projected Average Day Generation}\\&\quad= \textbf{Base Generation} + \textbf{Additional Residential Generation} + \textbf{Additional ICI Generation}\\&\quad+ \textbf{Average Extraneous Flow}\end{aligned}$$

The flows corresponding to the population growth forecasts to Ultimate Buildout are presented in Table 6-3 below.

Table 6-3 Flow Projections

YEAR	POPULATION	AVERAGE DAY FLOW (M^3/D)	MAXIMUM DAY FLOW (M^3/D)
Base	3,392	2,410	6,473
2016	3,396	2,423	6,509
2021	3,412	2,442	6,561
2026	3,435	2,602	6,990
2031	3,447	2,617	7,030
2036	3,456	2,783	7,476
2041	3,450	2,782	7,473
Ultimate	4,716	4,012	10,778

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 demand was calculated by applying the respective peaking factor to the average day demand.

A desktop analysis of historical wastewater flows and future flow projections is included in **Appendix C**.

6.3.1 CAPREOL LAGOON CAPACITY

Based on the current lagoon rated capacity of 5,000 m^3/d, the wastewater treatment capacity will be sufficient to service growth projections beyond Ultimate Buildout.

The WWTP capacity is plotted with the flow projections on Figure 6-1 below.

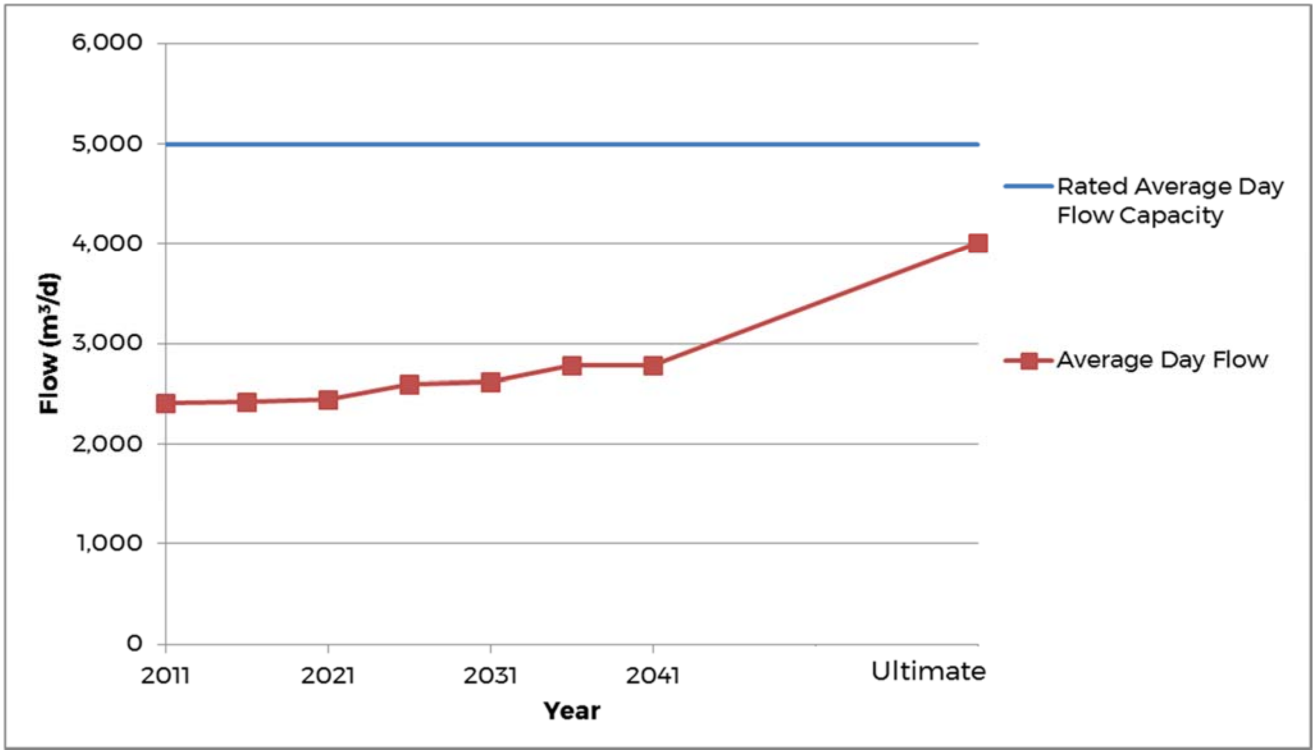

Figure 6-1 Wastewater Flow Projections Compared to Capreol Lagoon Rated Capacity

6.3.2 SEWER NETWORK AND LIFT STATIONS

For each of the scenarios modeled, the system was checked for surcharging of sewers and capacity exceedance at the pumping stations. The peak flows into each of the pumping stations were determined from computer simulations of the various planning scenarios are shown in Table 6-4 below. The table also shows the current pumps' design/rated flow, their capacity based on drawdown tests and the computer simulated flow for comparison.

Table 6-4 Capreol Lift Station Peak Influent Flow Rates

	CURRENT FIRM CAPACITY (L/S)	EXISITING PEAK FLOW (L/S)	2041 PEAK FLOW	ULTIMATE BUILDOUT
Lloyd	11.42	6.23	6.3	6.3
Vermilion	100	75.8	78	81.3

Based on the above table, there is capacity available at both Lloyd LS and Vermilion LS to service growth to Ultimate Buildout.

7 HYDRAULIC MODELLING

7.1 APPROACH

A basic sanitary model for the City of Greater Sudbury was received from the City. The model was created in Bentley's SewerGEMS by City staff. The model is an all pipe model of the sanitary network in these systems, but some critical information such as pipe data, invert elevations and lift station characteristics were missing. The model now includes this information as well as key vertical infrastructure in each system, including lift stations and treatment facilities.

The model was loaded with wet weather flow data. A water balance was completed to determine I&I rates for both dry and wet weather flow. The results from the water balance were compared against I&I rates developed through flow monitoring, and the greater of the two values, for each system, was used to load the model.

Current (2011) and future (2016-Ultimate Buildout, in 5 year increments) population data was added to the model using the City's planning data, summarized in previous sections of this report.

Future dry and wet weather flow scenarios were developed for each of the horizon years: 2016, 2021, 2026, 2031, 2036, 2041, and Ultimate Buildout. However, model results did not vary from 2016 to 2041; therefore, this report discusses findings for 2041 and Ultimate Buildout, compared against existing (2011).

7.2 MODELLING FINDINGS

The model was used to check sewer capacity and flow velocity. The majority of the sewers flow at less than 50% of the available capacity through to Ultimate Buildout under the wet weather flow condition.

Flow velocities through most of the Capreol sewer system are generally below the City's standard of 0.6 m/s. This is consistent through to Ultimate Buildout under the wet weather flow condition.

Maps in **Appendix B** illustrate the modeling results for the 2011, 2041, and Ultimate dry and wet weather flow scenarios.

8 CONCLUSIONS

An assessment of the Capreol Wastewater System was completed to identify infrastructure requirements to service forecasted growth in the community.

The conclusions of the assessment are summarized below.

- The Capreol Lagoon is deemed to have sufficient average day capacity to service growth to beyond Ultimate Buildout.
- The Capreol Lagoon does not meet the effluent requirements for TP.
- There is capacity available at both Lloyd LS and Vermilion LS to service growth to Ultimate Buildout.
- The majority of the sewers flow at less than 50% of the available capacity through to Ultimate Buildout under the wet weather flow condition.
- Flow velocities through most of the Capreol sewer system are generally below the City's standard of 0.6 m/s. This is consistent through to Ultimate Buildout under the wet weather flow condition.
- There are concerns with stormwater management and control in the community which has resulted in high levels of I&I in the sewer system.

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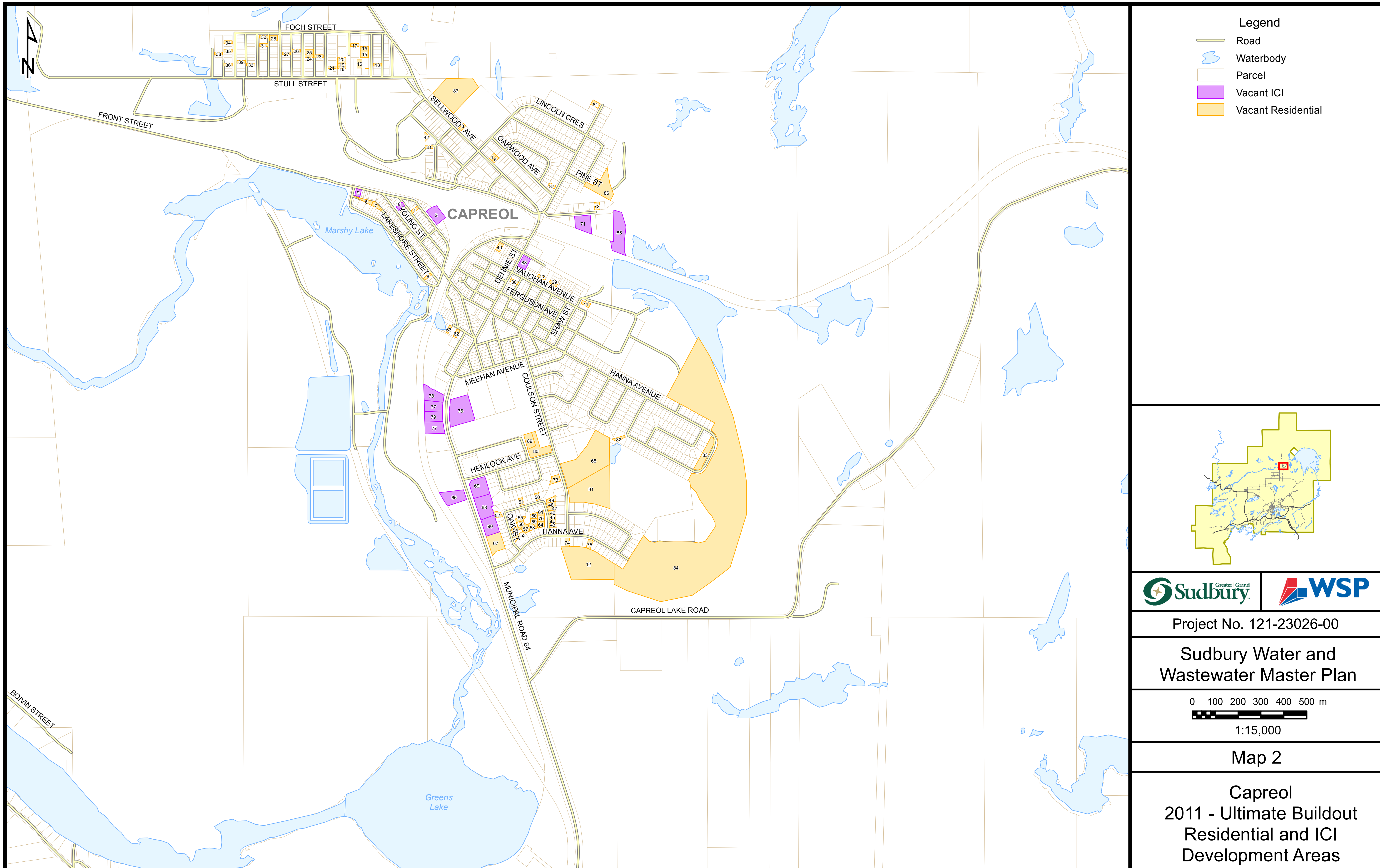
APPENDIX

A RESIDENTIAL AND ICI DEVELOPMENT AREAS



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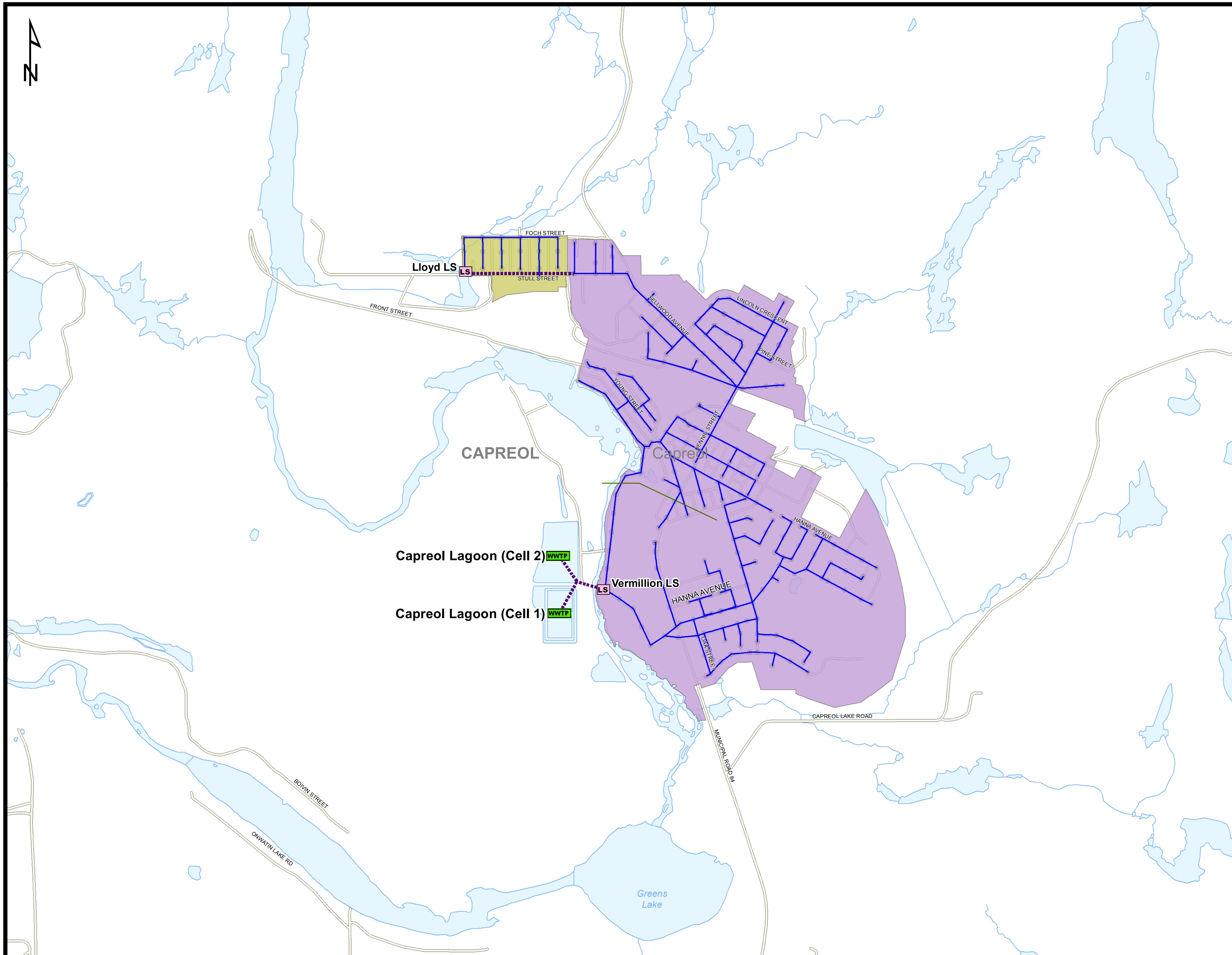
Sudbury Water and Wastewater Master Plan

Map 2

Capreol
2011 - Ultimate Buildout
Residential and ICI
Development Areas

APPENDIX

B WASTEWATER MODEL RESULTS



Legend

- Lift Station
- Sanitary Treatment Plant
- Manhole
- Waterbody
- Road
- Watercourse
- Sanitary Sewer Conduit
- Forcemain

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Sudbury Water and Wastewater Master Plan

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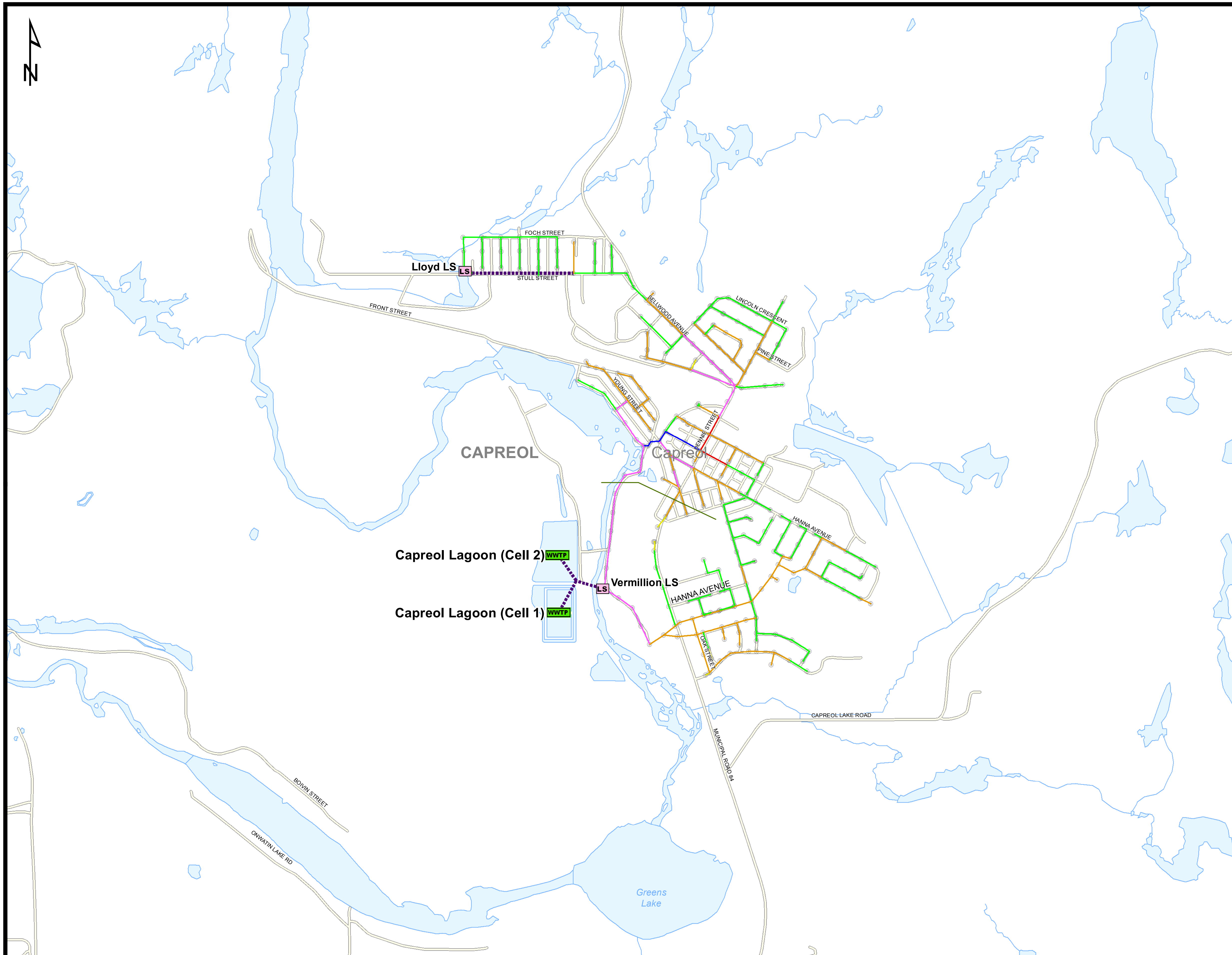
Map 2.1

Capreol Wastewater System - Sewershed Map



Legend

- LS Lift Station
- WWTP Sanitary Treatment Plant
- Manhole
- Waterbody
- Road
- Watercourse
- Forcemain

Conduit: Diameter (mm)

- <= 150
- <= 200
- <= 300
- <= 450
- <= 600
- <= 750
- > 750

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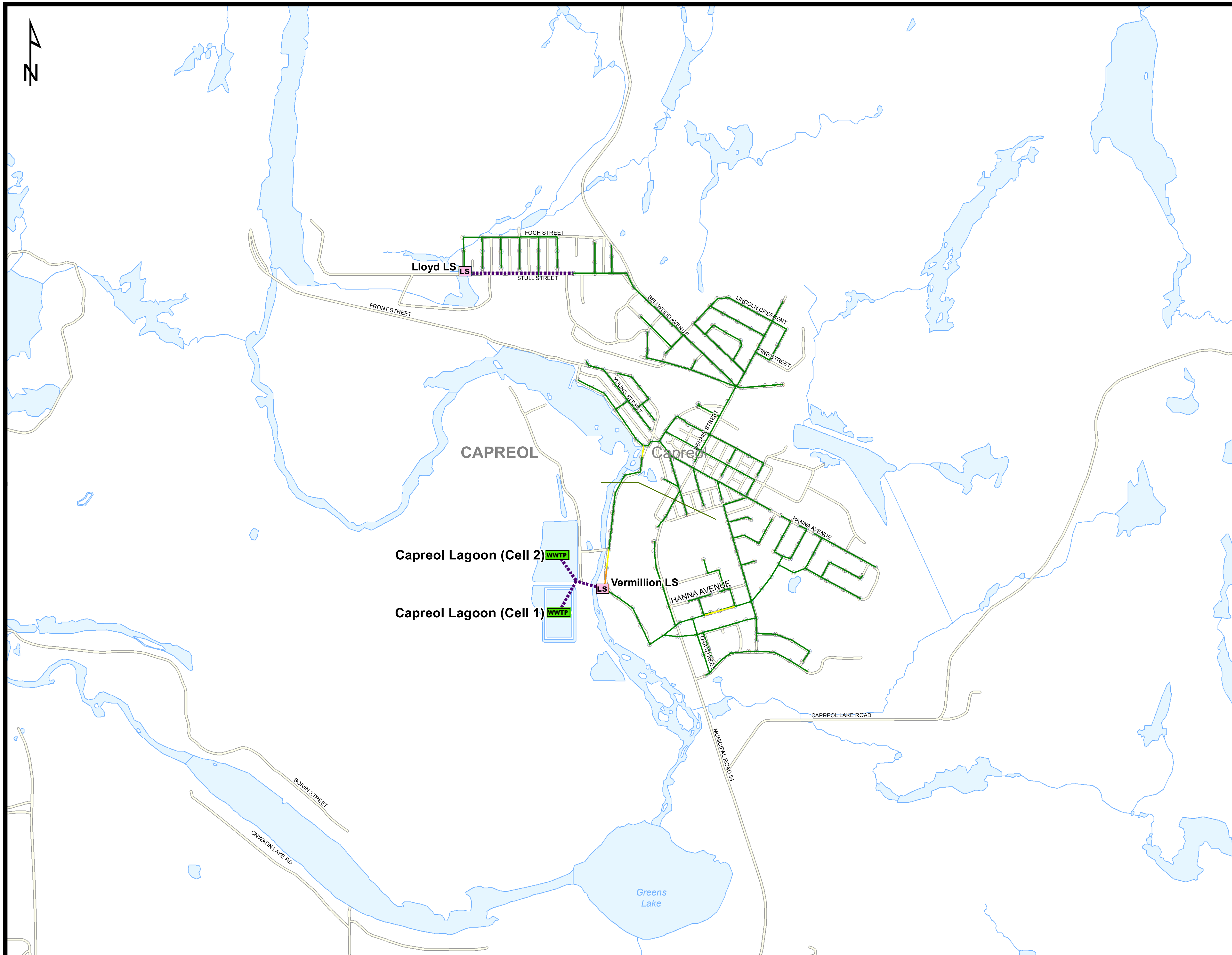
Map 2.2

Capreol Wastewater System - Conduit Diameter



Legend

LS	Lift Station
WWTP	Sanitary Treatment Plant
	Manhole
	Waterbody
	Road
	Watercourse
	Forcemain

Conduit: Flow/Capacity (Design) (%)

- <= 50
- <= 80
- <= 100
- > 100

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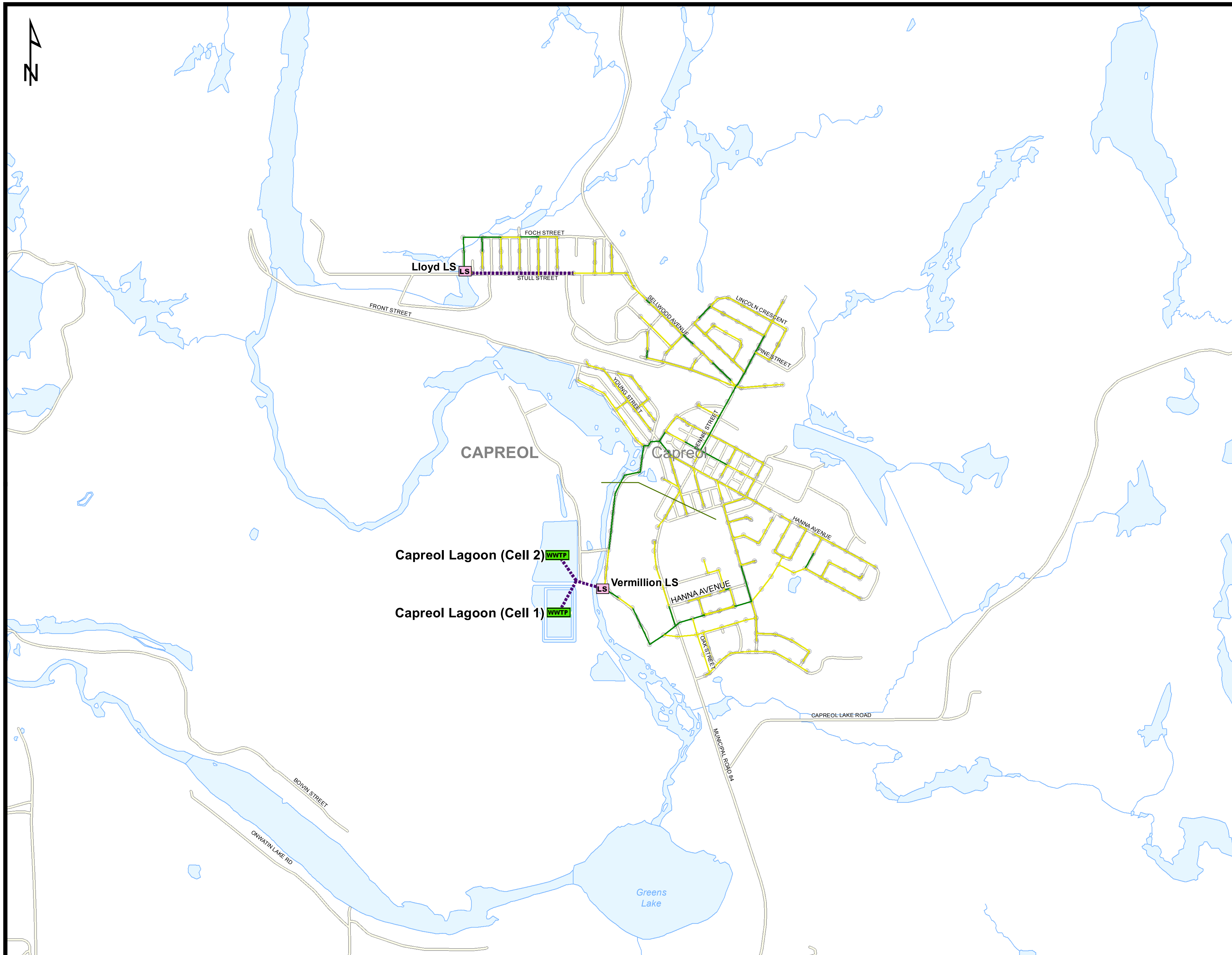
Map 2.3.1

Capreol Wastewater System - Conduit Flow/Capacity (Design)

Scenario: 2011WWF_2yrStorm



Legend

- Lift Station
- Sanitary Treatment Plant
- Manhole
- Waterbody
- Road
- Watercourse
- Forcemain

Conduit: Velocity (m/s)

- <= 0.6
- <= 1.5
- <= 3.0
- > 3.0

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October 04 2017

Sudbury Water and Wastewater Master Plan

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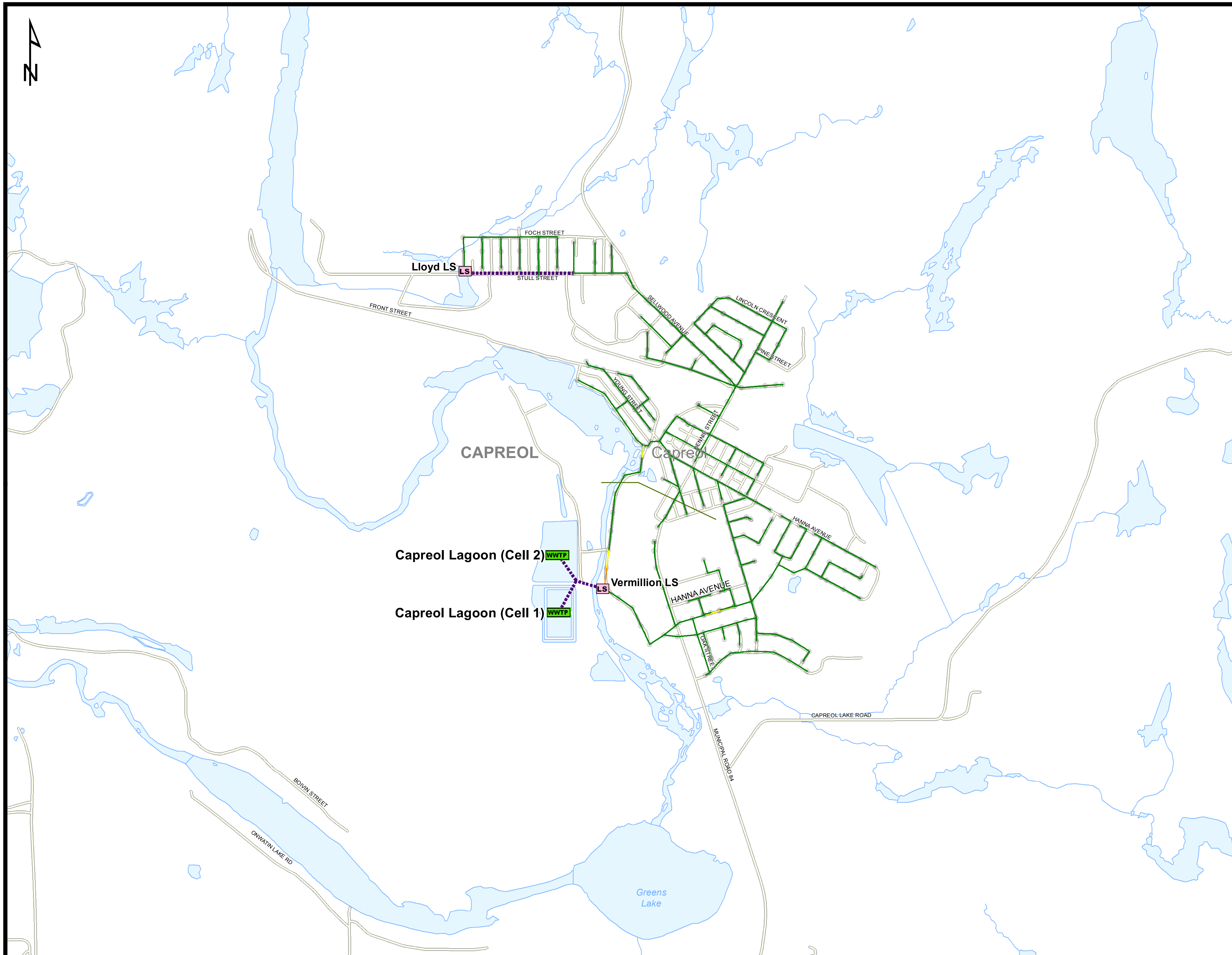
Map 2.3.2

Capreol Wastewater System - Conduit Velocity

Scenario: 2011WWF_2yrStorm



Legend

- LS Lift Station
- WWTP Sanitary Treatment Plant
- Manhole
- Waterbody
- Road
- Watercourse
- Forcemain

Conduit: Flow/Capacity (Design) (%)

- <= 50
- <= 80
- <= 100
- > 100

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Sudbury Water and Wastewater Master Plan

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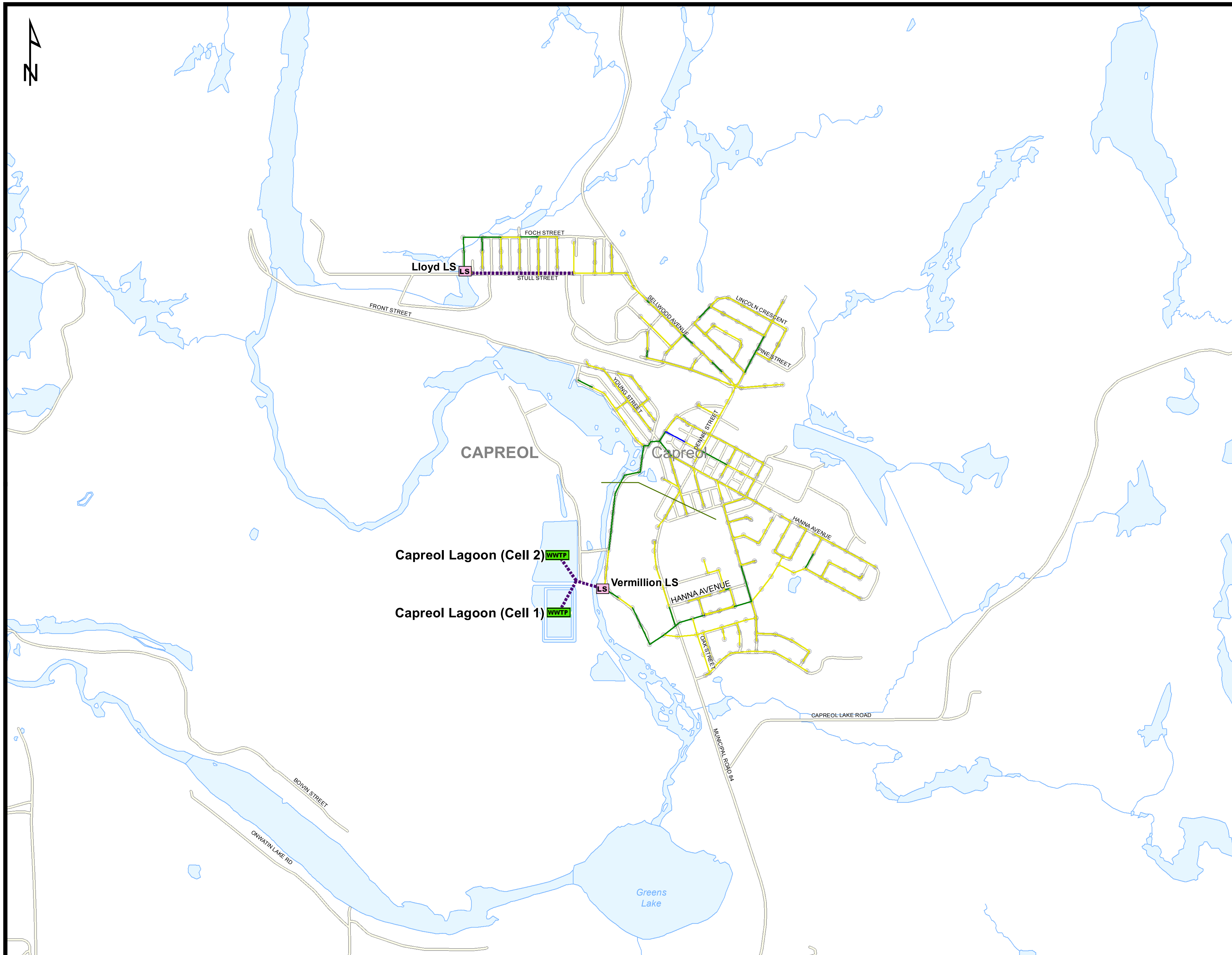
Map 2.4.1

Capreol Wastewater System - Conduit Flow/Capacity (Design)

Scenario: 2041WWF_2yrStorm



Legend

- LS Lift Station
- WWTP Sanitary Treatment Plant
- Manhole
- Waterbody
- Road
- Watercourse
- Forcemain

Conduit: Velocity (m/s)

- <= 0.6
- <= 1.5
- <= 3.0
- > 3.0

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Project No. 121-23026-00

December 09 2016

Sudbury Water and Wastewater Master Plan

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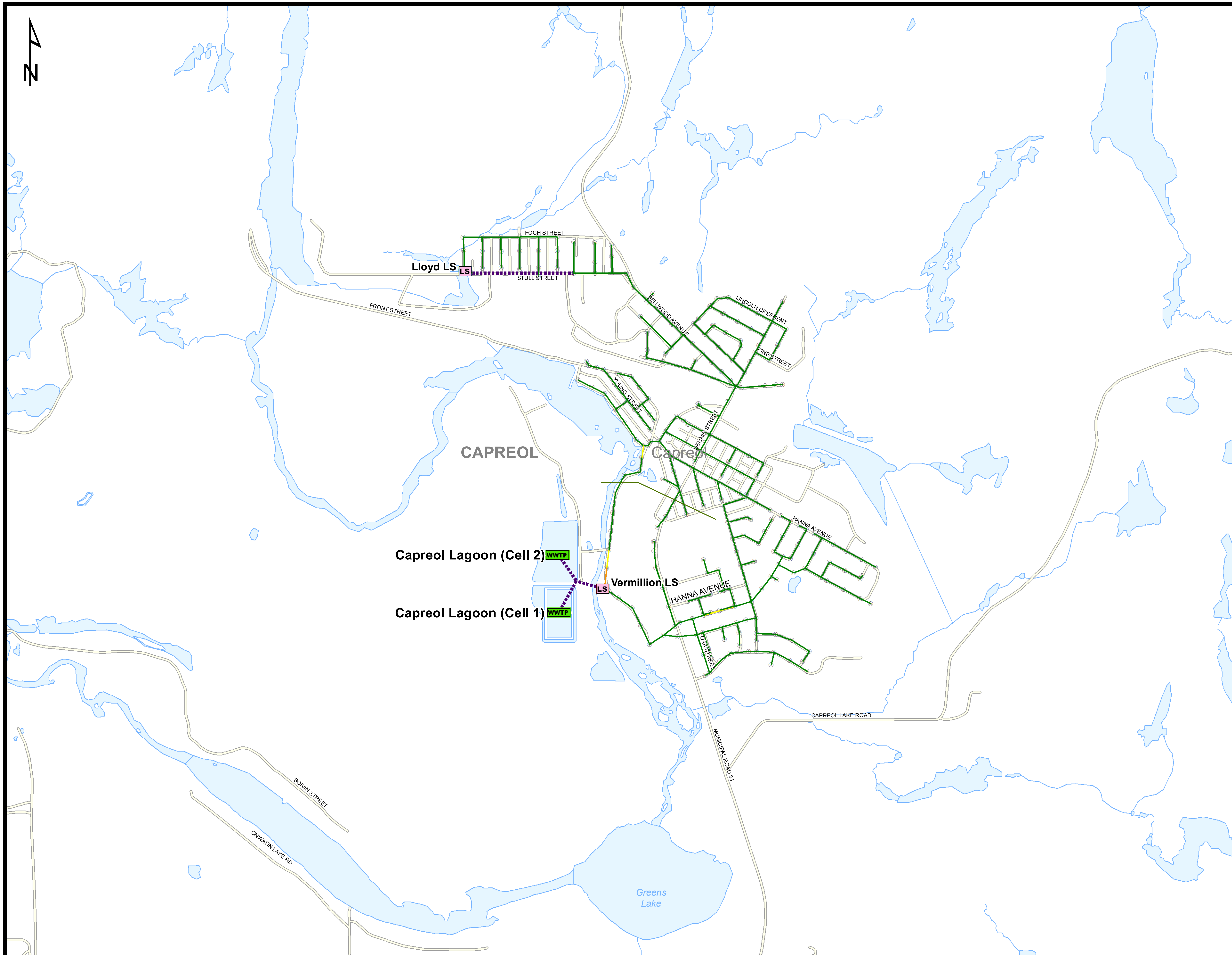
Map 2.4.2

Capreol Wastewater System - Conduit Velocity

Scenario: 2041WWF_2yrStorm



Legend

- LS Lift Station
- WWTP Sanitary Treatment Plant
- Manhole
- Waterbody
- Road
- Watercourse
- Forcemain

Conduit: Flow/Capacity (Design) (%)

- <= 50
- <= 80
- <= 100
- > 100

Sudbury Greater | Grand

WSP

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Sudbury Water and Wastewater Master Plan

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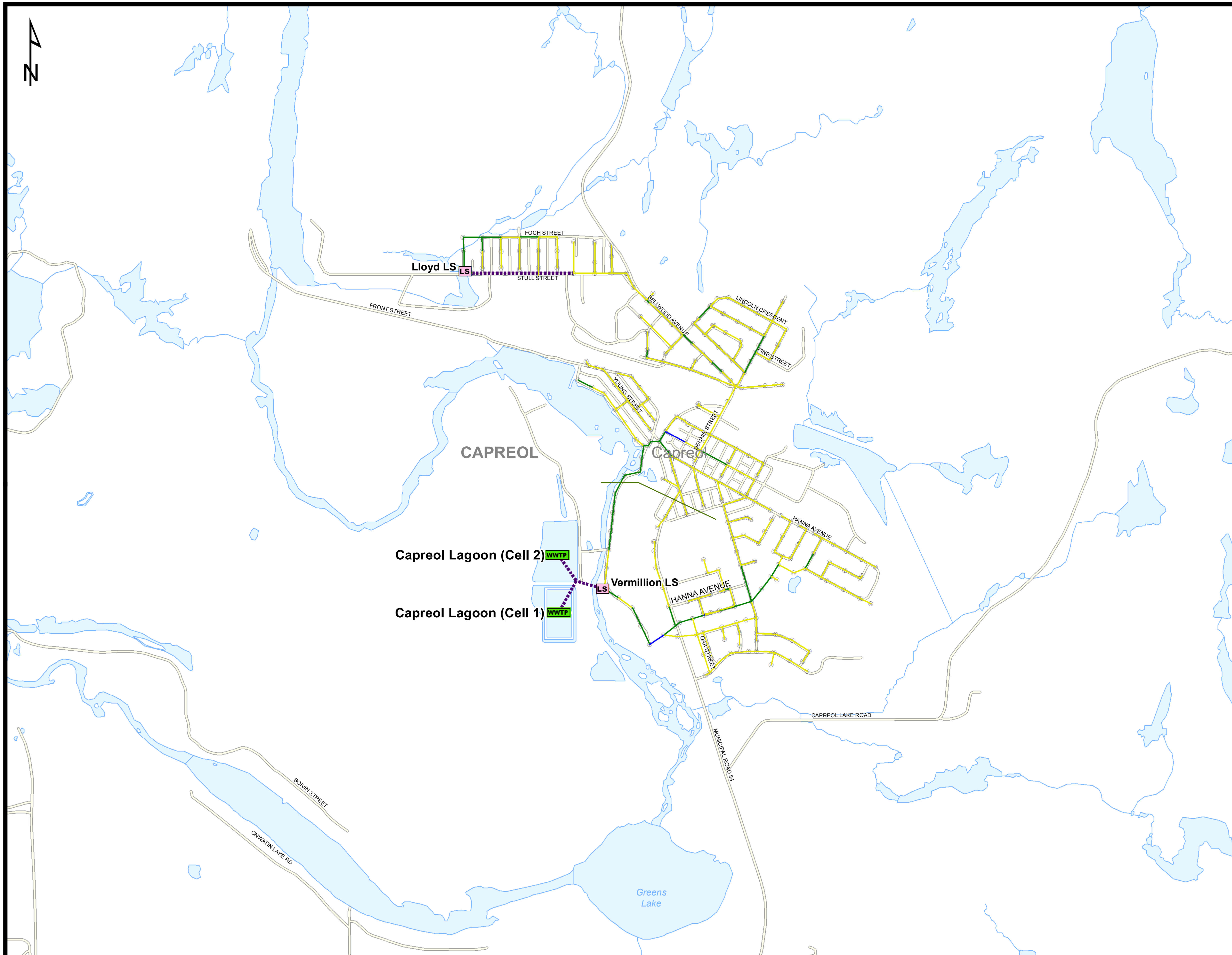
Map 2.5.1

Capreol Wastewater System - Conduit Flow/Capacity (Design)

Scenario: ULTWWF_2yrStorm



Legend

- LS Lift Station
- WWTP Sanitary Treatment Plant
- Manhole
- Waterbody
- Road
- Watercourse
- Forcemain

Conduit: Velocity (m/s)

- <= 0.6
- <= 1.5
- <= 3.0
- > 3.0

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WSP

Project No. 121-23026-00

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Sudbury Water and Wastewater Master Plan

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Map 2.5.2

Capreol Wastewater System - Conduit Velocity

Scenario: ULTWWF_2yrStorm

APPENDIX

C WASTEWATER GENERATION AND CAPACITY ASSESSMENTS

Capreol - Wastewater Flow Forecasts

		2009	2010	2011	2012	2013	Summary	Design Criterion	Comments
Average Day Flow	(m^3/d)	2,805	1,840	2,708	2,308	2,387	2,410	2,410	From Annual Reports
Max Day Flow	(m^3/d)	Not Available						6,473	Estimated by multiplying the estimated maximum day factor to the historical average day flow.
Max Day Factor								2.69	
Peak Flow	(m^3/d)							0	Peak hour flows were not available
Peak Flow	(L/s)							0	
Peak Flow Factor								0	
Population (Existing Areas)		3,392	3,392	3,392	3,392	3,392	3,392	3,392	
Ratio of Residential to Total Customers		0.87	0.87	0.87	0.87	0.87	0.87		This ratio is based on Water Billing Records for the area and is an approximation of the residential portion of demand.
Residential Share of Average Day Demand (m^3/d)		2437	1598	2352	2005	2073	2093		
Residential Flow Unit Rate (m^3/cap/d)		0.718	0.471	0.693	0.591	0.611	0.617	0.650	
Average Institutional Flow Unit Rate (m^3/ha/d)								28.0	CGS design standards do not suggest average rates for institutional and commercial, but require that rates be developed based on usage. In this case, water demands rates were used and assume a conservative 1:1 ratio of water to sewage.
Average Commercial Flow Unit Rate (m^3/ha/d)								28.0	CGS design standards do not suggest average rates for institutional and commercial, but require that rates be developed based on usage. In this case, water demands rates were used and assume a conservative 1:1 ratio of water to sewage.
Average Industrial Flow Unit Rate (m^3/ha/d)								35.0	CGS design standards average rate for light industrial of 35 m^3/ha/d.
Average Extraneous Flow Unit Rate (m^3/ha/d)								11.23	From CGS Design Standards, peak rate for new developments divided by an assumed peaking factor of 3. This factor would be applied only to new developments, which are assumed to be leak-tight, and have minimal extraneous flow.

	2016	2021	2026	2031	2036	2041	Ultimate Buildout	Comments
Population (Existing Areas)	3,392	3,392	3,392	3,392	3,392	3,392	3,392	
Population (Growth Areas)	4	20	43	55	64	58	1,324	
Total Population	3396	3412	3435	3447	3456	3450	4716	Total Population (Hemson)
Residential (ha)	0.97	1.73	2.27	2.89	3.24	3.43	39.69	ICI development areas were assigned to planning years based on the stage of the
Institutional (ha)			0.36	0.36	0.36	0.36	0.36	application. Draft Approved were assigned to 2016, Legal Lots of Record to 2026,
Commercial (ha)			3.18	3.18	3.71	3.71	3.71	and Designated Developable to 2036.
Industrial (ha)					2.94	2.94	2.94	Areas are cumulative and carry from the development year, all the way through
ICI (ha)	0.00	0.00	3.54	3.54	7.01	7.01	7.01	to Ultimate Buildout
Total (ha)	0.97	1.73	5.81	6.43	10.25	10.44	46.70	

	2016	2021	2026	2031	2036	2041	Ultimate Buildout	Comments
Average Residential Flows (m^3/d) - Existing	2,410	2,410	2,410	2,410	2,410	2,410	2,410	This includes all contribution from existing ICI and infiltration. The base flow was assumed to be the average day flow to the plant for the 2011-2013 period.
Average Residential Flows (m^3/d)	2	13	28	36	41	38	861	Obtained by multiplying the projected population growth by the unit rate.
Average Institutional Flow (m^3/d)	0	0	10	10	10	10	10	Institutional growth area multiplied by unit flow rate.
Average Commercial Flow (m^3/d)	0	0	89	89	104	104	104	Commercial growth area multiplied by unit flow rate.
Average Industrial Flow (m^3/d)	0	0	0	0	103	103	103	Industrial growth area multiplied by unit flow rate.
Average Extraneous Flow (m^3/day)	11	19	65	72	115	117	525	
Average Day Flow (m^3/d)	2,423	2,442	2,602	2,617	2,783	2,782	4,012	
Max Day Flow (m^3/d)	6,509	6,561	6,990	7,030	7,476	7,473	10,778	

ALTERNATIVE CALCULATION METHOD

Per Capita Flow (m3/cap/day)	0.827	0.542	0.798	0.680	0.704	0.710	0.710

	2016	2021	2026	2031	2036	2041	Ultimate Buildout
Average Day Flow (m^3/d)	2412	2424	2440	2449	2455	2451	3350
Max Day Flow (m^3/d)	6,481	6,512	6,556	6,579	6,595	6,585	9,001

Comments

Multiplying the total population by the consolidated per capita flow factor.

SENSITIVITY ANALYSIS

Analyze sensitivity of forecasted flows to unit rate

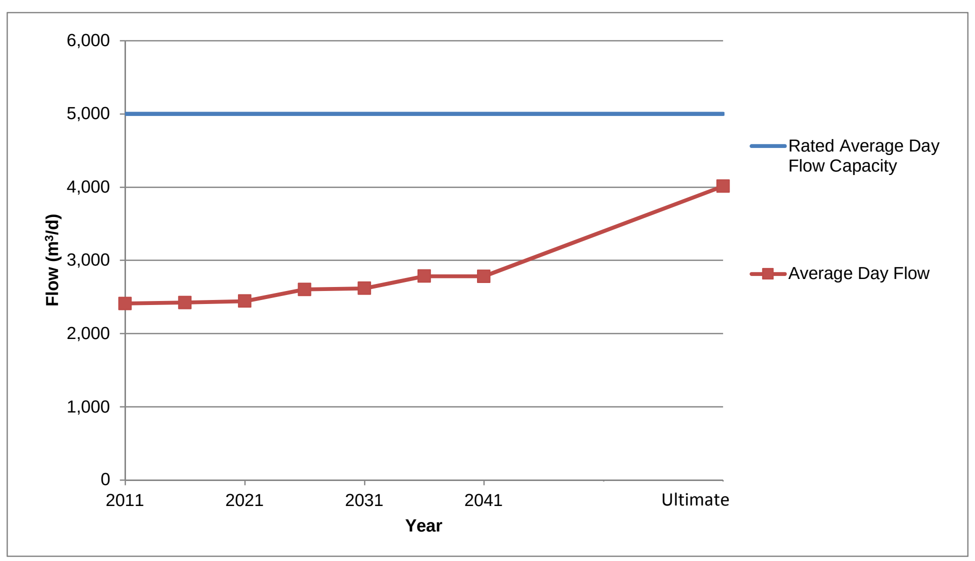
	Unit Rate (m^3/cap/d)	Average Day Flow (m^3/d) 2016	2021	2026	2031	2036	2041	Ultimate Buildout 2061
Base Scenario - Residential Historical Maximum	0.617	2,423	2,442	2,602	2,617	2,783	2,782	4,012
Combined Historical Maximum	0.710	2,423	2,443	2,605	2,620	2,787	2,785	4,092
City Standards	0.500	2,422	2,439	2,596	2,609	2,773	2,773	3,813

Analyze sensitivity of forecasted flows to max day factor

	Max Day Peaking Factor	Max Day Flow (m^3/d) 2016	2021	2026	2031	2036	2041	Ultimate Buildout 2061
Base Scenario - Historical Max	Not Available							
Historical Average	0.00	6,509	6,561	6,990	7,030	7,476	7,473	10,778

CAPACITY CHECK

	2011	2016	2021	2026	2031	2036	2041	Ultimate Buildout 2061
Rated Average Day Flow Capacity (m^3/d)	5,000	5,000	5,000	5,000	5,000	5,000	5,000	5,000
Average Day Flow (m^3/d)	2,410	2,423	2,442	2,602	2,617	2,783	2,782	4,012
Maximum Day Flow (m^3/d)	6,473	6,509	6,561	6,990	7,030	7,476	7,473	10,778



CITY OF GREATER SUDBURY WATER AND WASTEWATER MASTER PLAN

CHELMSFORD WASTEWATER SYSTEM GAP ANALYSIS AND STATUS QUO REPORT

CITY OF GREATER SUDBURY

DRAFT

PROJECT NO.: 121-23026-00
DATE: MARCH 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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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 existing Chelmsford Wastewater System. Based on population growth projections and design criteria discussed in the *Population Projections and Development of Unit Rates Technical Memorandum* (WSP, 2014), 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 Chelmsford Wastewater System would continue to be a stand-alone system. Any potential interconnections between Chelmsford 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

Chelmsford is located along Old Highway 69, North of Valley East. Mapping in **Appendix A** shows the Chelmsford 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 *Population Projections and Development of Unit Rates 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 Chelmsford population is expected to increase from 7,400 in 2011 to 7,891 by 2041 and 11,008 by Ultimate Buildout.

ICI growth is expected to be a mix of institutional, commercial and industrial. Growth is discussed further in Section 6.1.

3 OVERVIEW OF EXISTING SYSTEM

The Chelmsford Wastewater System services the community of Chelmsford and includes the Chelmsford WWTP and the Chelmsford Lagoon. All wastewater generated in Chelmsford is ultimately treated at the WWTP; however, the lagoon is used occasionally in cases of wet weather emergencies. The Chelmsford Wastewater System is shown in **Appendix B**. Additional information on the existing systems is provided in the Baseline Review Report for Wastewater Systems (WSP, 2014).

3.1 LIFT STATIONS

The system also has eight lift stations and a sewer network. The collection system consists of approximately 46.44 km of sewers and forcemains.

Table 3-1 below provides a summary of the main features of the lift stations.

Table 3-1 Chelmsford Sewage Lift Stations

LIFT STATION	YEAR CONSTRUCTED[1]	LAST UPGRADED[2]	WET WELL VOLUME (M^3)[1]	PUMPING STATION CAPACITY AND FORCEMAIN INFORMATION[2]
Belanger	1974	None	6.8	Two dry pit pumps with a firm design capacity of 6.25 L/s 206 m long, 150 mm diameter forcemain of unknown material
Brookside	1976	None	Unknown	Two submersible pumps with a firm design capacity of 13.5 L/s 218 m long, 450 mm diameter forcemain of unknown material
Charette	1973	1999	Unknown	Two submersible pumps with a firm design capacity of 14.9 L/s 138 m long, 250 mm diameter PVC forcemain
Hazel	1988	None	17.6	Two submersible pumps with a firm design capacity of 21.73 L/s 315 m long, 250 mm diameter PVC (SDR 26) forcemain
Keith	1979	None	20.8	Two submersible pumps with a firm design capacity of 45.2 L/s 847 m long, 200 mm diameter plastic forcemain
Main	1979	None	Unknown	Two submersible pumps with a firm design capacity of 40.1 L/s PVC forcemain north to lagoon: 300 mm diameter for 221 m then 300 mm for 46 m, then 200 mm for 1452 m Forcemain south (unknown material): 250 mm for 1513 m, then 300 mm for 7 m

Radisson	1998	None	24.9	Two submersible pumps with a firm design capacity of 6.5 L/s 211 m long, 100 mm diameter forcemain of unknown material
Whitson	1976	None	5.6	Two submersible pumps with a firm design capacity of 20.5 L/s 29 m long, 100 mm diameter forcemain of unknown material

[1]Obtained or estimated from dimensions found in as-built and record drawings, assuming water level does not exceed the High Water Alarm Level or, in absence of this alarm level, the inlet sewer invert.
[2] Obtained from the City's Wastewater Lift Stations Operations Manual and station as-built drawings.

3.2 CHELMSFORD WWTP AND LAGOON

The Chelmsford WWTP is owned and operated by the City of Greater Sudbury and is located at 300 Laurette Street. The WWTP consists of three aeration plants with common preliminary treatment and disinfection. The average day rated capacity is 7,100 m^3/d, rated peak daily flow capacity is 18,200 m^3/d, and peak instantaneous capacity 24,000 m^3/d (MOECC, 2009). The treatment process is illustrated schematically below.

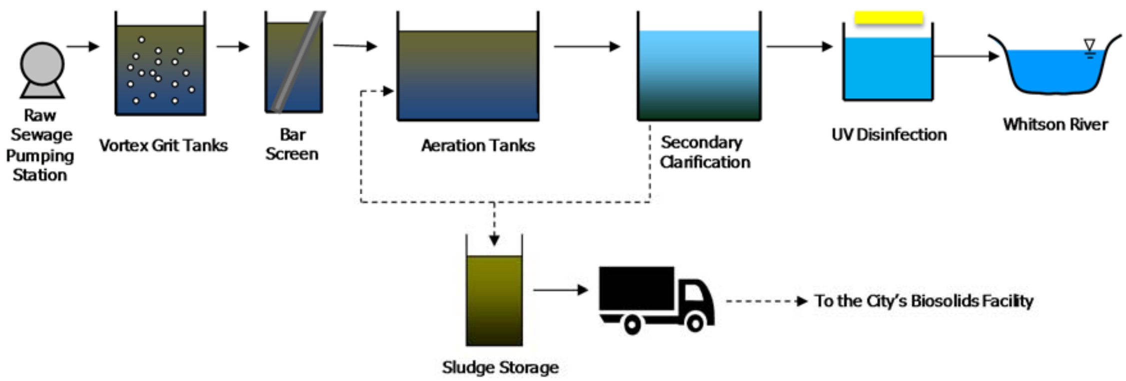

Figure 3-1 Chelmsford WWTP Process

The Chelmsford Lagoon is owned and operated by the City of Greater Sudbury and is located at Concession IV, Lot 2. The lagoon is used for temporary storage of wastewater during high flows. Wastewater is conveyed to the Chelmsford WWTP for treatment when flows subside.

3.3 KNOWN CHALLENGES

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

— City operations staff has reported high levels of infiltration in the residential area north of MR 35 and east of MR 15. Sewers surcharge in several areas of Chelmsford, including near Omer Street and Brookside Road (hydraulic restriction point) and the upstream catchment area of the Charette LS. City staff has also indicated that the area upstream of Charette LS is limited by the amount of flow that can run through the Charette Street sewer entering the lift station, creating a backup of flow in the system. The bottleneck, combined with deep sewers and a high groundwater table, causes surcharging in the area of and surrounding Pinellas Road.

— From 2011 to 2014, there have been two bypasses at the Chelmsford WWTP, one at the Chelmsford Lagoon, and one each at the Belanger and Main Lift Stations. The bypasses seem to be a result of high I&I in the system.

- There is limited land available on the Chelmsford WWTP for expansion.
- Repairs in the collection system are challenging due to ground conditions.

4 HISTORICAL FLOWS AND REVIEW OF OPERATIONAL DATA

Data reported in the 2009 to 2013 *Annual Reports* for the Chelmsford WWTP was reviewed and analyzed to determine average day and maximum day flows as well as review effluent parameters.

4.1 FLOW DATA

WWTP flow data from 2009 to 2013 was reviewed. Operational data was not available from the lift stations and so historical peak flow data could not be estimated.

The recorded average day and maximum day flows are summarized in Table 4-1 and plotted in Figure 4-1 below.

Table 4-1 Historical Wastewater Flow Data

YEAR	AVERAGE DAY FLOW (M^3/D)[1]	MAXIMUM DAY FLOW (M^3/D)[1]
2009	4,753	17,896
2010	3,712	22,090
2011	3,888	14,440
2012	3,704	9,200
2013	4,729	18,210

[1] *Annual Reports (2009 - 2013)*

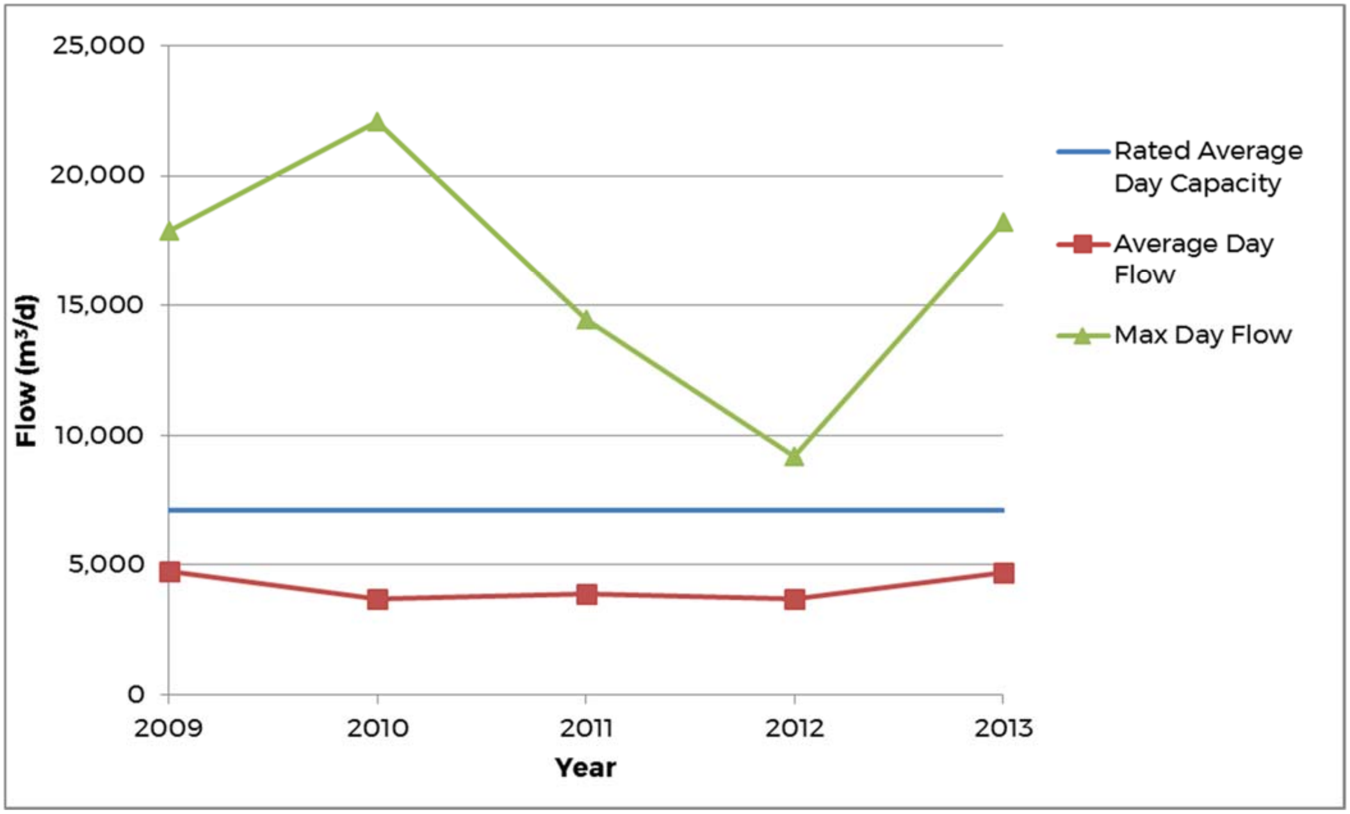

Figure 4-1 Historical Wastewater Flows at the Chelmsford Wastewater Treatment Plant

The relationship between the different flow regimes was analyzed to compare the maximum day peaking factors derived from historical data to those used in the *City's Engineering Design Manual* and those included in the *MOECC Guidelines.*

The average day flows to the WWTP have been consistent over the 2009 to 2013 period, averaging 4,157 m^3/d. The variations in historical maximum day flows show no discernible trend. The greatest maximum day flow occurred in 2010 and the average historical maximum day flow was 16,367 m^3/d.

The wide range in maximum day flows, but stable average day flows, indicates that the system is susceptible to variations in precipitation.

The highest maximum day to average day peaking factor based on the maximum day flow recorded in 2010 was 5.95. The average maximum day peaking factor from 2009 to 2013 was 3.95. The City's *Engineering Design Manual* and the *MOECC Guidelines* do not specify recommended maximum day factors and recommend using historical data when available. For future wastewater generation, the average peaking factor was used and based on the assumption that new developments would have less I&I due to more leak tight construction.

4.2 RAW WASTEWATER CHARACTERISTICS

The average raw wastewater characteristics from 2009 to 2012 are summarized in Table 4-2 below. Temperatures were not reported.

Table 4-2 Average Raw Wastewater Characteristics (2009-2012)

PARAMETER	AVERAGE VALUE
$CBOD_5$	111 mg/L

Suspended Solids	130 mg/L
Total Phosphorus	3.6 mg/L
TKN	25.6 mg/L
pH	7.3

Wastewater flows to the Chelmsford WWTP correspond mainly to residential uses, with contributions from commercial and industrial users, and dilution from inflow and infiltration.

4.3 EFFLUENT CRITERIA

The Chelmsford WWTP is operated in accordance with *MOECC Amended Certificate of Approval for Sewage No. 4370-7QPMGZ* dated December 14, 2009.

The *C of A* concentration and loading limits are summarized in Table 4-3.

Table 4-3 Chelmsford WWTP Effluent Limits and Objectives

EFFLUENT PARAMETER	CONCENTRATION LIMIT	LOADING LIMIT	CONCENTRATION OBJECTIVE / LOADING LIMIT
CBOD5	May – Oct.: 7.0 mg/L Nov. – Apr.: 15 mg/L	May 1 – Oct. 31: 49.7 kg/d Nov. 1 – Apr. 30: 106.5 kg/d	May – Oct.: 5.0 mg/L / 35.5. kg/d Nov. – Apr.: 10 mg/L / 71 kg/d
Total Suspended Solids (TSS)	May – Oct.: 7.0 mg/L Nov. – Apr.: 15 mg/L	May 1 – Oct. 31: 49.7 kg/d Nov. 1 – Apr. 30: 106.5 kg/d	May – Oct.: 5.0 mg/L / 35.5 kg/d Nov. – Apr.: 10 mg/L / 71 kg/d
Total Phosphorus (TP)	May – Oct.: 0.3 mg/L Nov. – Apr.: 0.5 mg/L	May 1 – Oct. 31: 2.13 kg/d Nov. 1 – Apr. 30: 3.55 kg/d	-
Total Ammonia Nitrogen (TAN)	May – Oct.: 2.0 mg/L Nov. – Apr.: 4.0 mg/L	May 1 – Oct. 31: 14.2 kg/d Nov. 1 – Apr. 30: 28.4 kg/d	May – Oct.: 1.0 mg/L / 7.1 kg/d Nov. – Apr.: 2.0 mg/L / 14.2 kg/d
E. coli	-	-	200 organisms/100 mL (Monthly Geometric Mean Density)

Compliance with the concentration and loading limits for $CBOD_5$ and TSS is based on the annual average concentration of each parameter based on all composite samples during any calendar year, whereas compliance for the TP is based on the monthly average concentration.

4.4 OPERATIONAL DATA

The general plant operation was reviewed against the Chelmsford WWTP Amended C of A requirements and historical data provided in the Annual Reports from 2009 to 2012. Historical data is summarized in Table 4-4.

Table 4-4 Historical Effluent Concentrations

EFFLUENT PARAMETER	ANNUAL AVERAGE			
	2009	2010	2011	2012
$CBOD_5$ (mg/L)	4.1	2.8	3.2	3.1
TSS (mg/L)	5.9	4.5	4.9	5.4
TP (mg/L)	0.24 (all months comply)	0.20 (all months comply)	0.22 (all months comply)	0.17 (all months comply)
pH	7.1	7.0	6.9	6.8
E. coli (organisms/100 mL)	17	41	59	14

Historically, the Chelmsford WWTP has met all concentration limits.

A capacity review of each unit process at the WWTP was not conducted. Instead, the rated capacity was considered the true capacity of the plant.

Biosolids from the Chelmsford WWTP are hauled to the Biosolids Facility at the Sudbury WWTP, and must meet specific quality requirements. This includes inorganic compounds such as plastics. However, the WWTP does not currently have fines screening to remove such compounds.

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 WASTEWATER DESIGN CRITERIA

The unit flow criteria for growth adopted for this assessment are shown in Table 5-1 below. These values were recommended in the *Population Projections and Unit Rates Technical Memorandum* (WSP, 2014).

Note that the term "extraneous flows" is interchangeable with "I&I flows".

Table 5-1 Wastewater System Design Criteria

CRITERIA	VALUE	REFERENCE
Average Day Residential Generation	450 L/cap/day	Average of historical values, rounded up to nearest 50 L/cap/day
Average Day Commercial and Institutional Generation	28 m^3/ha/d	Water unit rates, assuming a 1:1 correspondence
Average Day Industrial Generation	35 m^3/ha/d	Water unit rates, assuming a 1:1 correspondence
Average Extraneous Flow	11.2 m^3/ha/d	Peak from City's Engineering Design Manual and assuming a peaking factor of three
Peak Extraneous Flow	33.7 m^3/ha/d	City's Engineering Design Manual
Max Day Peaking Factor	3.95	Average of historical values

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

Maximum day flows to the WWTP are obtained by multiplying the average day flow by the maximum day peaking factor.

5.2 DESIGN CRITERIA FOR WASTEWATER SYSTEM COMPONENTS AND OPERATION

5.2.1 WASTEWATER TREATMENT

Wastewater treatment facilities are rated for average day flows. Plant effluent limits and objectives are established in the C of A or ECA for each facility.

5.2.2 LIFT STATION PUMPING CAPACITY

The firm capacity of the lift station (with the largest pump out of service) must allow pumping of peak wet weather flows corresponding to its catchment area (MOECC, 2008).

Starting limitations on pump motors generally dictate the minimum size of a wet well. The wet well should be large enough to prevent pump motors from overheating due to frequent starting and stopping, but small enough to avoid long retention times leading to septicity and odor problems (Lin & Lee, 2001).

The station wet well shall be sized such that the number of pump starts per hour does not exceed the maximum value recommended by the pump manufacturer. In other words, the time between pump starts and stops (i.e. the pump cycle time) should not exceed that which results in a pump start frequency greater than that recommended by the pump manufacturer. Typically, submersible pumps can cycle four to 10 times per hour with a maximum cycle time not exceeding 30 minutes (Lin & Lee, 2001). A maximum value of four pump starts per hour was assumed to evaluate wet well sizing requirements.

5.2.3 SEWERS

The sewer system is typically sized to convey peak instantaneous (peak wet weather) flows. Sewage flows are made up of wastewater discharges from residential, commercial, institutional and industrial establishments, plus extraneous flow components from such sources as groundwater and surface runoff.

In addition to being able to convey peak flows, sufficient flow velocity should be maintained to transport the sewage solids to avoid deposition and the development of nuisance conditions under lower flow conditions. The minimum acceptable flow velocity in sewers is 0.6 m/s (City of Greater Sudbury, 2012).

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 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 *Population Projections 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 Chelmsford population with wastewater servicing is projected to increase by 491 people by 2041 and 3,608 by Ultimate Buildout.

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

Table 6-1 Chelmsford Population Projections

SYSTEM	2011	2016	2021	2026	2031	2036	2041	ULTIMATE BUILDOUT
Chelmsford	7,400	7,517	7,639	7,763	7,838	7,886	7,891	11,008

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 the table below.

Table 6-2 Chelmsford ICI Projections

	ICI DEVELOPMENT AREAS (HA)						
LAND USE	2016	2021	2026	2031	2036	2041	BUILDOUT
Institutional	0	0	1.60	0	0	0	0
Commercial	0	0	3.75	0	0	0	0
Industrial	0	0	0	0	45.85	0	0
Total	**0**	**0**	**5.35**	**0**	**45.85**	**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 Chelmsford, one streets have been placed on the priority list for sewer and water servicing and two additional 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 Chelmsford 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 WASTEWATER FLOW PROJECTIONS AND INFRASTRUCTURE NEEDS

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

$$\textbf{Projected Average Day Generation} = \textbf{Base Generation} + \textbf{Additional Residential Generation} + \textbf{Additional ICI Generation} + \textbf{Average Extraneous Flow}$$

The flows corresponding to the population growth forecasts to Ultimate Buildout are presented in Table 6-3 below.

Table 6-3 Flow Projections

YEAR	POPULATION	AVERAGE DAY FLOW (M^3/D)	MAXIMUM DAY FLOW (M^3/D)
Base	7,400	4,157	16,367
2016	7,517	4,884	19,305
2021	7,639	4,966	19,631
2026	7,763	5,247	20,742
2031	7,838	5,292	20,918
2036	7,886	7,442	29,419
2041	7,891	7,456	29,471
Ultimate	11,008	9,498	37,545

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 demand was calculated by applying the respective peaking factor to the average day demand. Base maximum day demand was the historical average. A desktop analysis of historical wastewater flows and future flow projections is included in **Appendix C**.

6.4.1 CHELMSFORD WWTP CAPACITY

Based on the current Chelmsford WWTP rated capacity of 7,100 m^3/d, the wastewater treatment capacity will be sufficient to service growth projections until 2031. Additional capacity will be required by 2032.

The WWTP capacity is plotted with the flow projections on Figure 6-1 below.

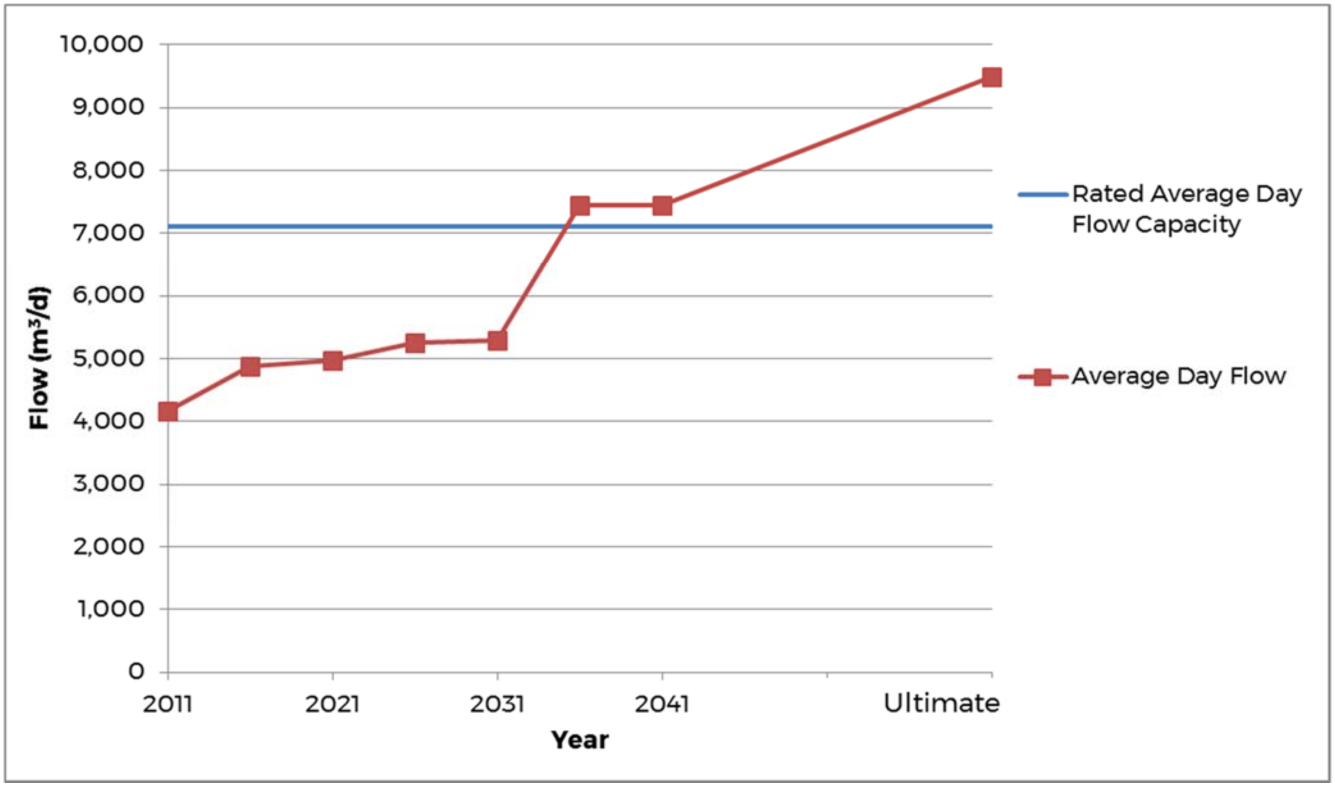

Figure 6-1 Wastewater Flow Projections Compared to Chelmsford WWTP Rated Capacity

6.4.2 SEWER NETWORK AND LIFT STATIONS

For each of the scenarios modeled, the system was checked for surcharging of sewers and capacity exceedance at the pumping stations. The peak flows into each of the pumping stations were determined from computer simulations of the various planning scenarios are shown in Table 6-4 below. The table also shows the current pumps' design/rated flow, their capacity based on drawdown tests and the computer simulated flow for comparison.

Table 6-4 Chelmsford Lift Station Peak Influent Flow Rates

	CURRENT FIRM CAPACITY	EXISTING PEAK FLOW	2041 PEAK FLOW	ULTIMATE BUILDOUT
Belanger	6.25	8.8	9.1	8.8
Brookside	13.5	6.1	6.1	6.1
Charette	14	2.3	2.3	2.4
Hazel	51.7	16.5	30	31
Keith	45.2	4.2	4.2	4.2
Main	40.1	32.8	33	33.7
Radisson	6.5	1.1	9.7	9.8
Whitson	22.5	4.3	4.3	4.3

Based on the above table, a number of lift station issues are apparent:

- Inflows to the Belanger LS exceed the design and drawdown capacities and therefore may not have sufficient capacity to convey peak flows starting in 2011.
- The expected flows to the Radisson LS are projected to be higher than the firm capacity. Additional capacity will be required before 2041.

Based on the above table, the remainder of the pumps' firm design capacity is generally higher than the simulated flow, as expected.

7 HYDRAULIC MODELLING

7.1 APPROACH

A basic sanitary model for the City of Greater Sudbury was received from the City. The model was created in Bentley's SewerGEMS by City staff. The model is an all pipe model of the sanitary network in these systems, but some critical information such as pipe data, invert elevations and lift station characteristics were missing. The model now includes this information as well as key vertical infrastructure in each system, including lift stations and treatment facilities.

The model was loaded with wet weather flow data. A water balance was completed to determine I&I rates for both dry and wet weather flow. The results from the water balance were compared against I&I rates developed through flow monitoring, and the greater of the two values, for each system, was used to load the model.

Current (2011) and future (2016-Ultimate Buildout, in 5 year increments) population data was added to the model using the City's planning data, summarized in previous sections of this report.

Future dry and wet weather flow scenarios were developed for each of the horizon years: 2016, 2021, 2026, 2031, 2036, 2041, and Ultimate Buildout. However, model results did not vary from 2016 to 2041; therefore, this report discusses findings for 2041 and Ultimate Buildout, compared against existing (2011).

7.2 MODELLING FINDINGS

The model was used to check sewer capacity and flow velocity. The majority sewers in Chelmsford flow at less than 50% of the available capacity through to Ultimate Buildout under the wet weather flow condition, with few exceptions.

The following are of note:

— Significant surcharge conditions upstream of Charette LS.

Flow velocities in most of Chelmsford are less than 0.6 m/s through Ultimate Buildout under wet weather flow conditions.

The maps in **Appendix B** illustrate the modeling results for the 2011, 2041, and Ultimate dry and wet weather flow scenarios.

8 CONCLUSIONS

An assessment of the Chelmsford Wastewater System was completed to identify infrastructure requirements to service forecasted growth in the community.

The conclusions of the assessment are summarized below.

— The WWTP is deemed to have sufficient average day capacity to service growth to 2031, but will need to be expanded or Ultimate Buildout. The plant currently does not have a rated maximum day flow.

— Additional pumping capacity may be required at Belanger LS starting from 2011 and Radisson LS starting from 2041.

— Flow velocities through most of the Chelmsford system are below the City's current standard of 0.6 m/s.

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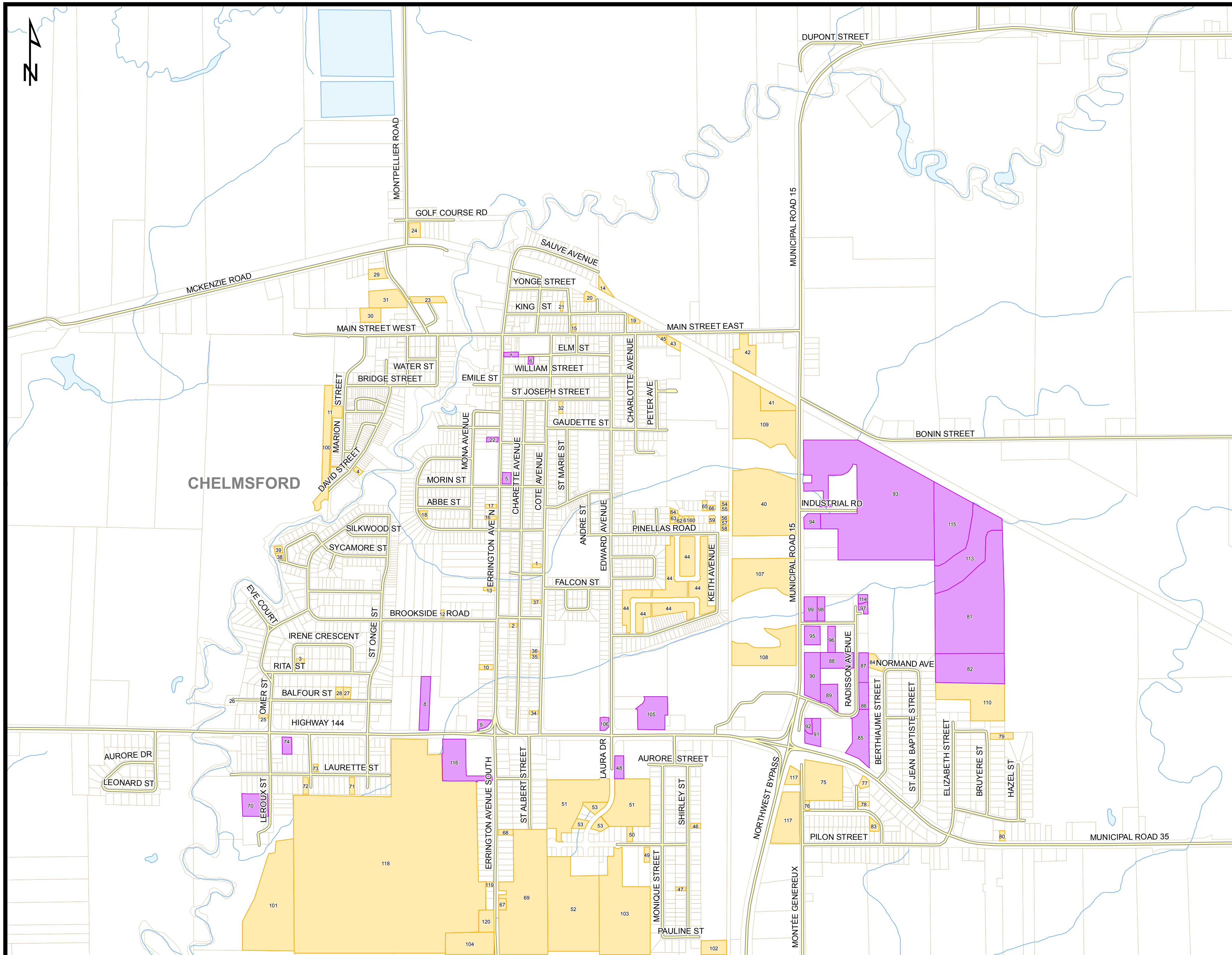
APPENDIX

A RESIDENTIAL AND ICI DEVELOPMENT AREAS



Project No. 121-23026-00

Sudbury Water and Wastewater Master Plan

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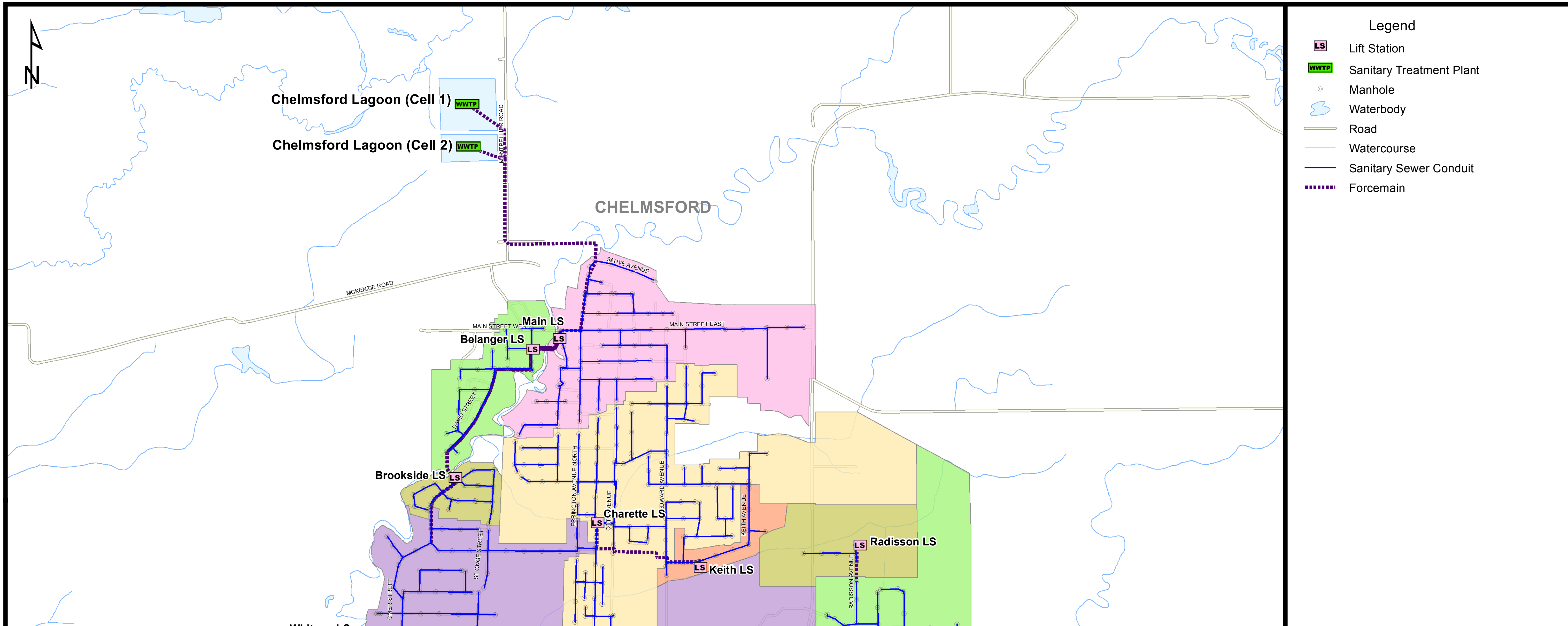
Map 3

Chelmsford
2011 - Ultimate Buildout
Residential and ICI
Development Areas

APPENDIX

B WASTEWATER MODEL RESULTS



Project No. 121-23026-00

December 09 2016

Sudbury Water and Wastewater Master Plan

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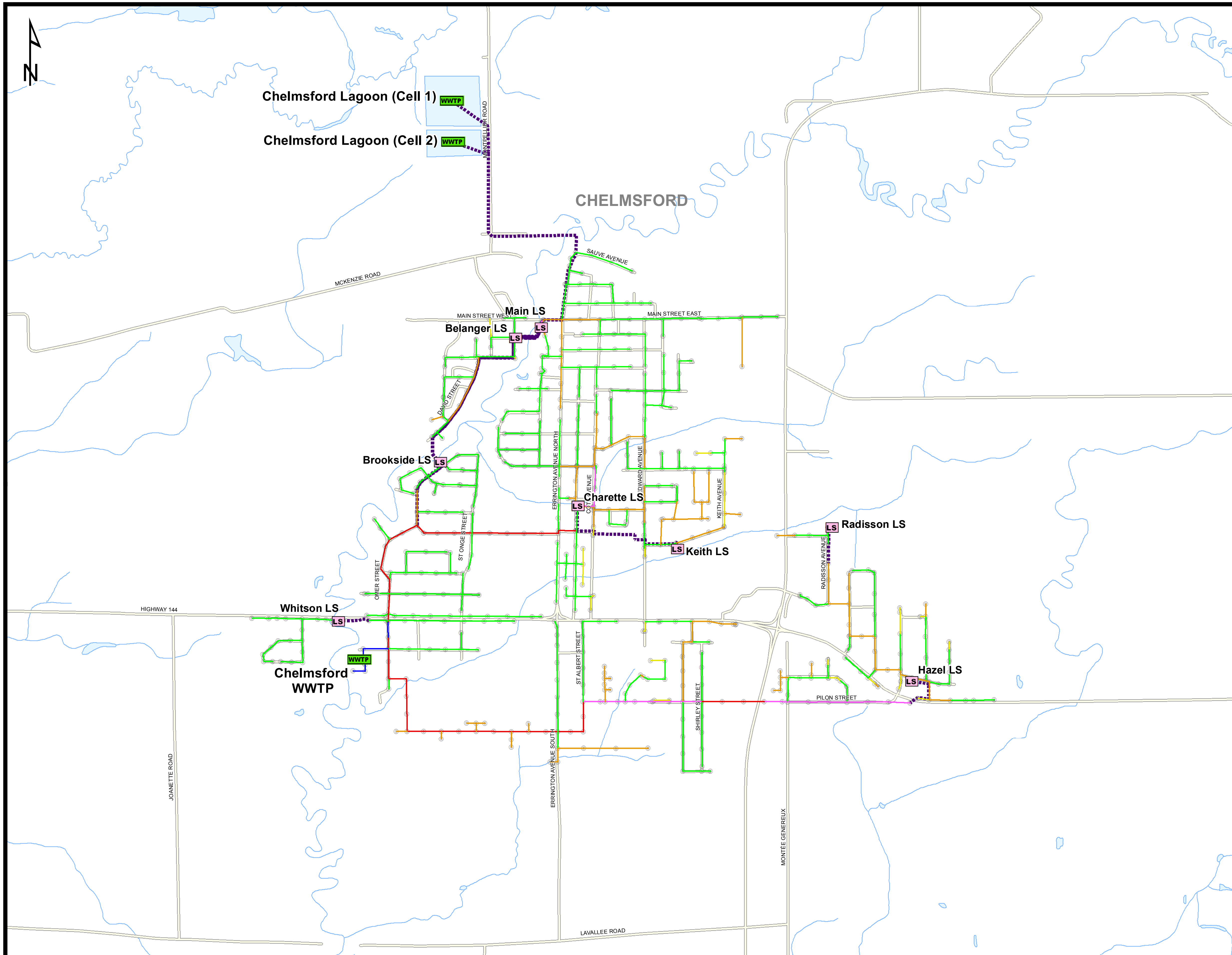
Map 3.1

Chelmsford Wastewater System - Sewershed Map



Legend

- LS Lift Station
- WWTP Sanitary Treatment Plant
- Manhole
- Waterbody
- Road
- Watercourse
- Forcemain

Conduit: Diameter (mm)

- <= 150
- <= 200
- <= 300
- <= 450
- <= 600
- <= 750
- > 750

Greater Sudbury | WSP

Project No. 121-23026-00

December 09 2016

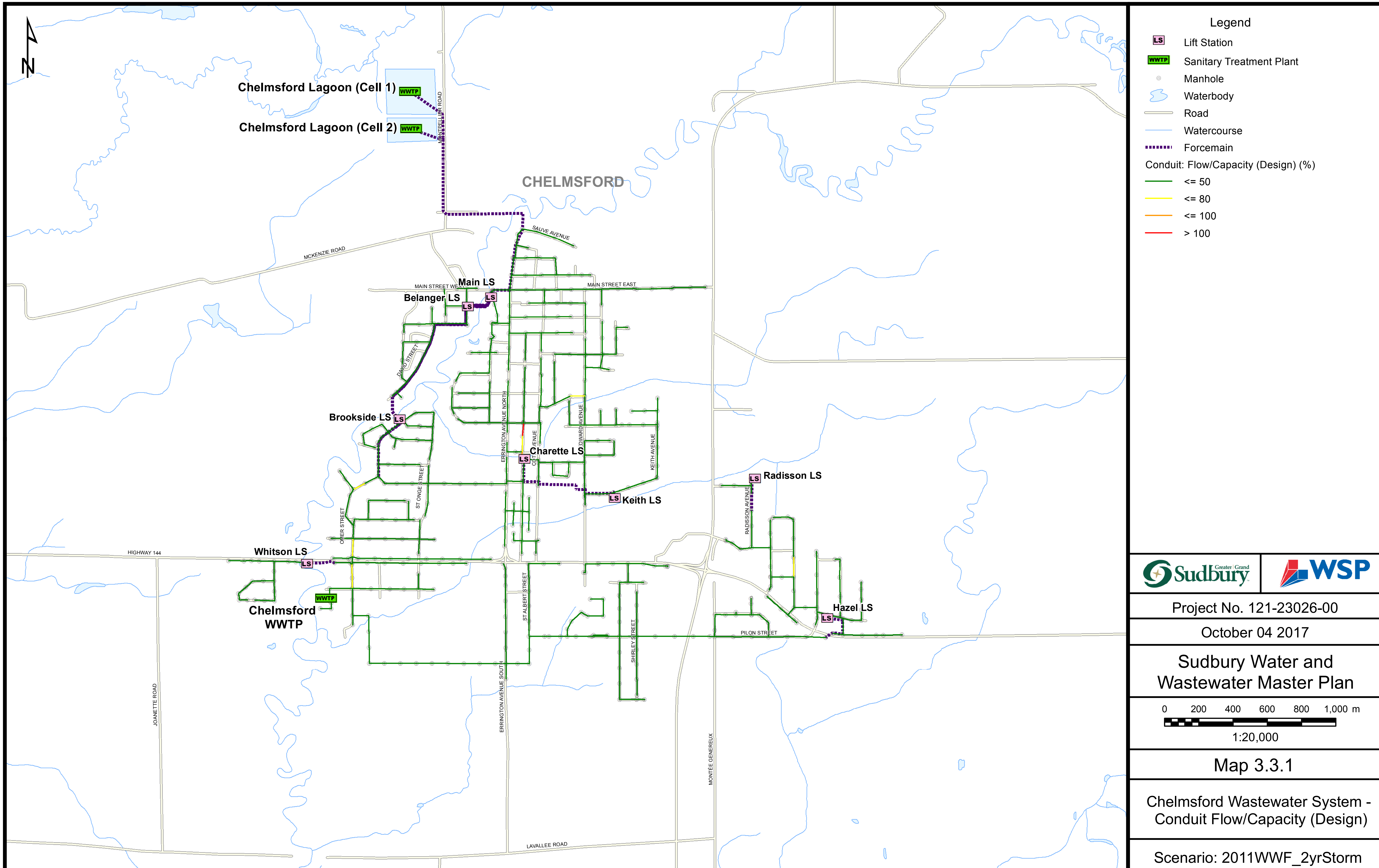
Sudbury Water and Wastewater Master Plan

0 200 400 600 800 1,000 m

1:20,000

Map 3.2

Chelmsford Wastewater System - Conduit Diameter


N
Chelmsford Lagoon (Cell 1)
WWTP
Chelmsford Lagoon (Cell 2)
WWTP
CHELMSFORD
SAUVE AVENUE
MCKENZIE ROAD
MAIN STREET WEST
MAIN STREET EAST
Main LS
Belanger LS
LS
DAVID STREET
Brookside LS
ERRINGTON AVENUE NORTH
Charette LS
KEITH AVENUE
ST ONGE STREET
Radisson LS
Keith LS
RADISSON AVENUE
OMER STREET
HIGHWAY 144
Whitson LS
Chelmsford WWTP
ST ALBERT STREET
Hazel LS
PILON STREET
SHIRLEY STREET
ERRINGTON AVENUE SOUTH
JOANETTE ROAD
MONTÉE GENEREUX
LAVALLEE ROAD
Legend
Lift Station
Sanitary Treatment Plant
Manhole
Waterbody
Road
Watercourse
Forcemain
Conduit: Flow/Capacity (Design) (%)
<= 50
<= 80
<= 100
> 100
Greater Sudbury
WSP
Project No. 121-23026-00
October 04 2017
Sudbury Water and Wastewater Master Plan
0 200 400 600 800 1,000 m
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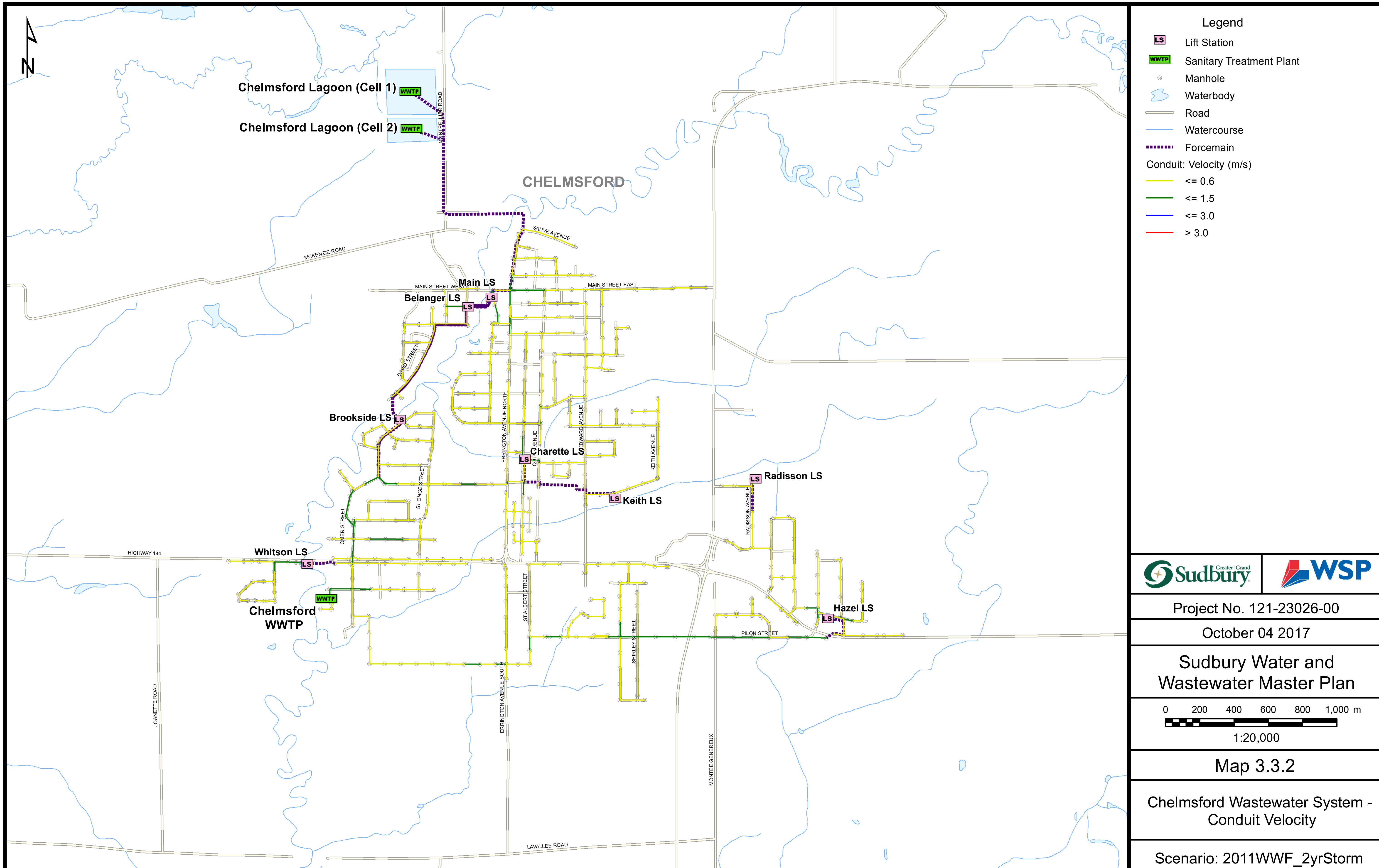
Map 3.3.1
Chelmsford Wastewater System - Conduit Flow/Capacity (Design)
Scenario: 2011WWF_2yrStorm


N
Chelmsford Lagoon (Cell 1)
Chelmsford Lagoon (Cell 2)
WWTP
CHELMSFORD
SAUVE AVENUE
MCKENZIE ROAD
MAIN STREET WEST
MAIN STREET EAST
Main LS
Belanger LS
DAVID STREET
Brookside LS
ERRINGTON AVENUE NORTH
Charette LS
KEITH AVENUE
Radisson LS
Keith LS
RADISSON AVENUE
ST ONGE STREET
OMER STREET
HIGHWAY 144
Whitson LS
Chelmsford WWTP
ST ALBERT STREET
SHIRLEY STREET
Hazel LS
PILON STREET
JOANETTE ROAD
ERRINGTON AVENUE SOUTH
MONTÉE GENEREUX
LAVALLEE ROAD
Legend
Lift Station
Sanitary Treatment Plant
Manhole
Waterbody
Road
Watercourse
Forcemain
Conduit: Velocity (m/s)
<= 0.6
<= 1.5
<= 3.0
> 3.0
Greater Grand Sudbury
WSP
Project No. 121-23026-00
October 04 2017
Sudbury Water and Wastewater Master Plan
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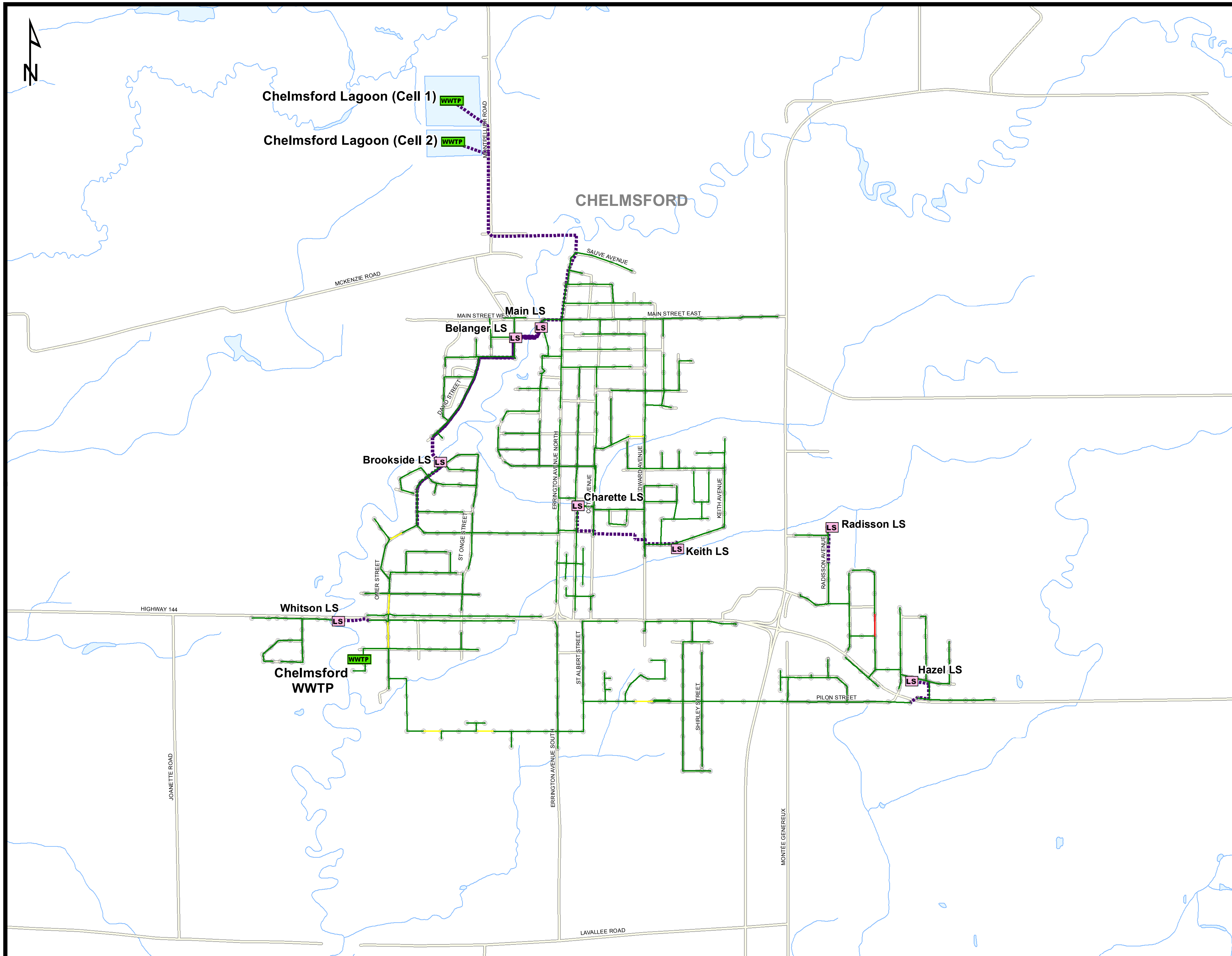
Map 3.3.2
Chelmsford Wastewater System - Conduit Velocity
Scenario: 2011WWF_2yrStorm



Legend

- LS Lift Station
- WWTP Sanitary Treatment Plant
- Manhole
- Waterbody
- Road
- Watercourse
- Forcemain

Conduit: Flow/Capacity (Design) (%)

- <= 50
- <= 80
- <= 100
- > 100

Greater Grand Sudbury | WSP

Project No. 121-23026-00

December 09 2016

Sudbury Water and Wastewater Master Plan

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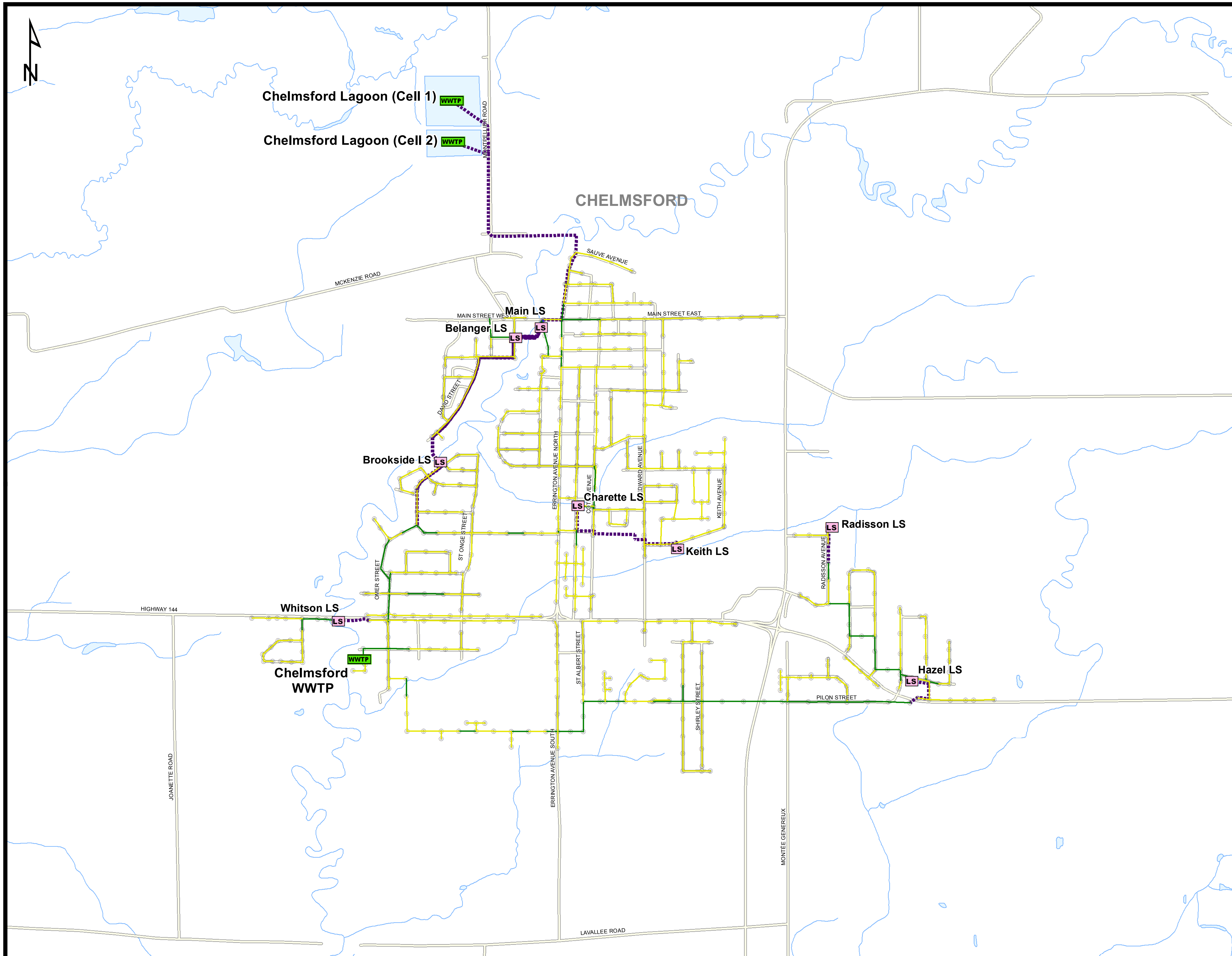
Map 3.4.1

Chelmsford Wastewater System - Conduit Flow/Capacity (Design)

Scenario: 2041WWF_2yrStorm



Legend

- LS Lift Station
- WWTP Sanitary Treatment Plant
- Manhole
- Waterbody
- Road
- Watercourse
- Forcemain

Conduit: Velocity (m/s)

- <= 0.6
- <= 1.5
- <= 3.0
- > 3.0

Sudbury | WSP

Project No. 121-23026-00

December 09 2016

Sudbury Water and Wastewater Master Plan

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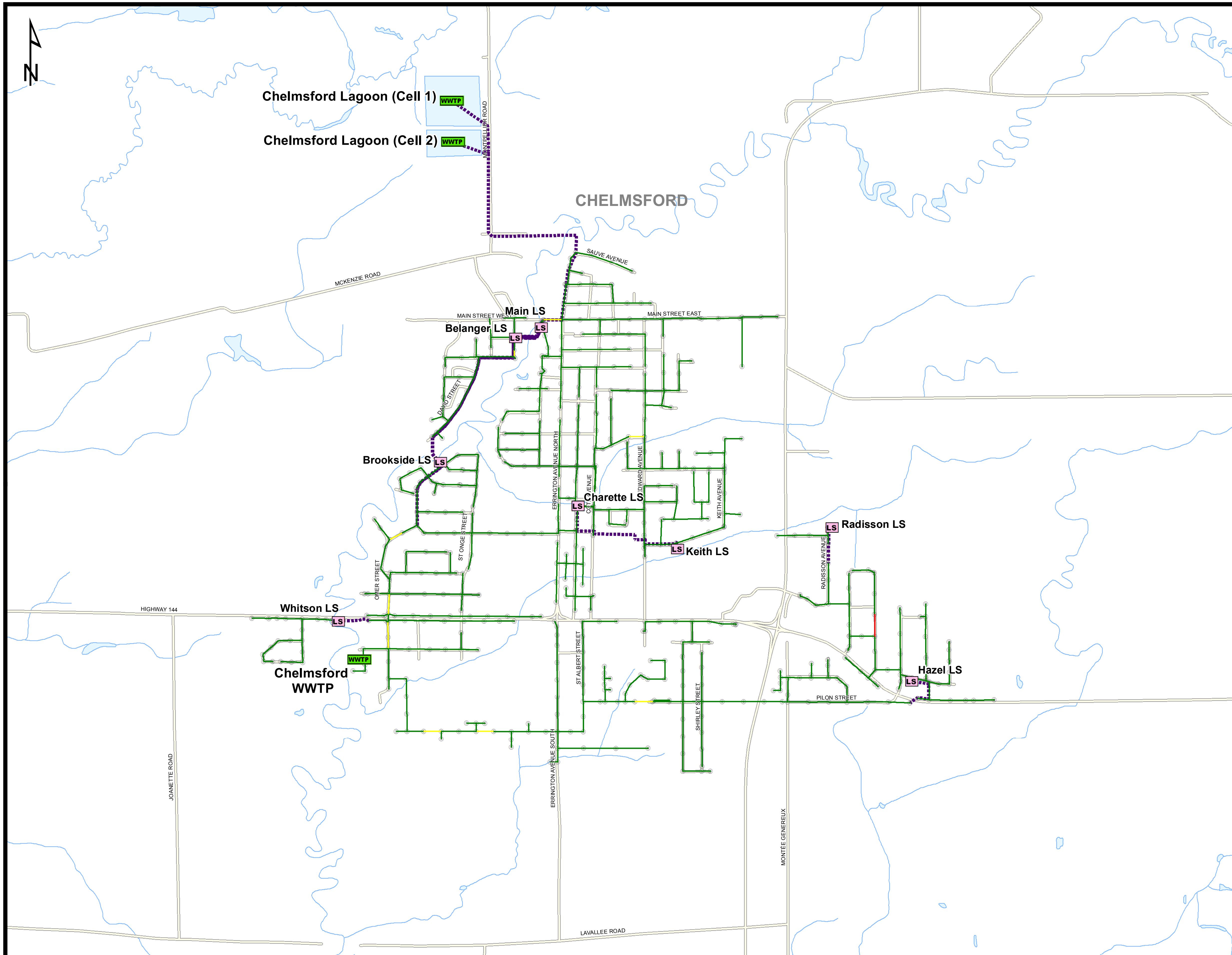
Map 3.4.2

Chelmsford Wastewater System - Conduit Velocity

Scenario: 2041WWF_2yrStorm



Legend

- LS Lift Station
- WWTP Sanitary Treatment Plant
- Manhole
- Waterbody
- Road
- Watercourse
- Forcemain

Conduit: Flow/Capacity (Design) (%)

- <= 50
- <= 80
- <= 100
- > 100

Sudbury Greater Grand | WSP

Project No. 121-23026-00

December 09 2016

Sudbury Water and Wastewater Master Plan

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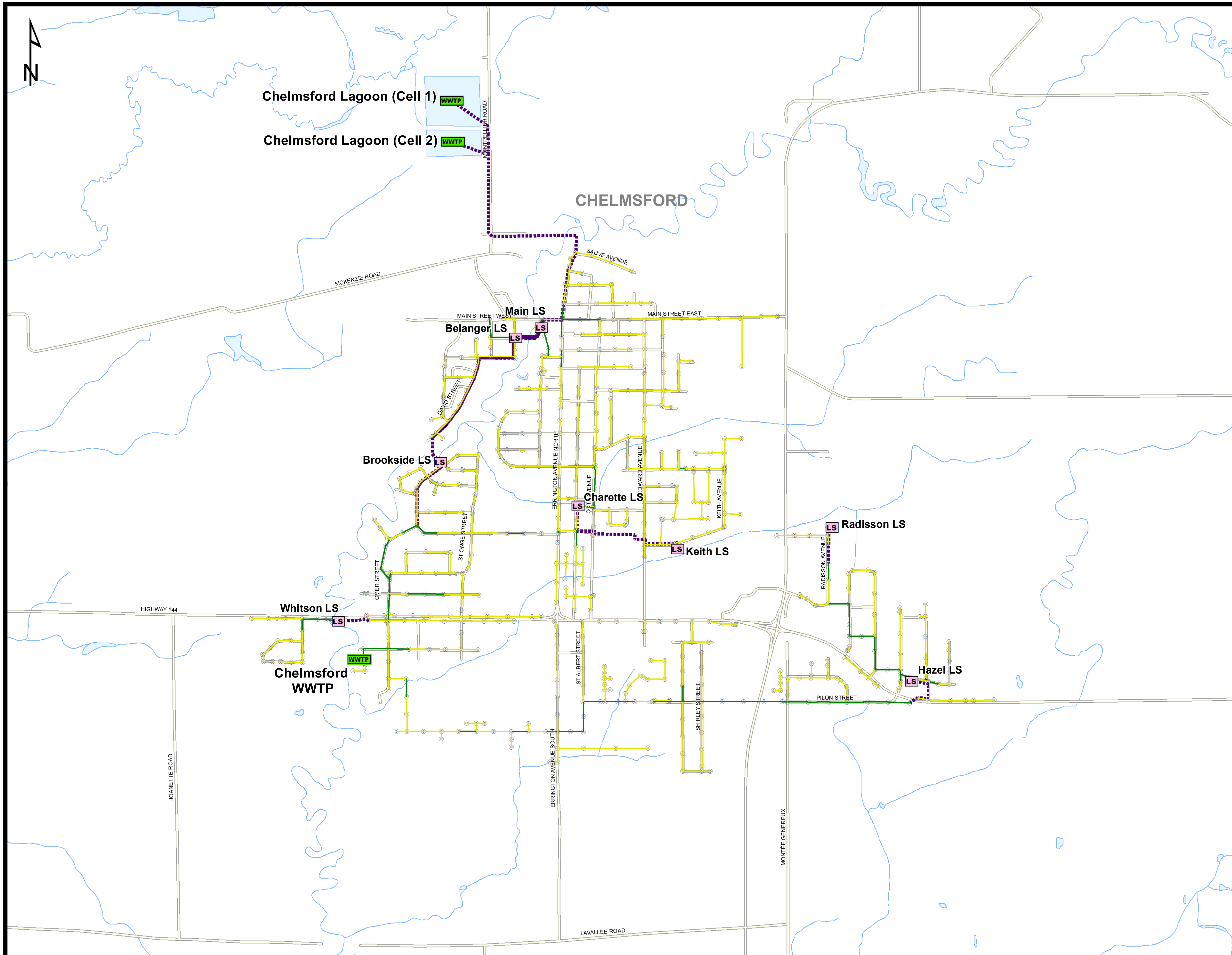
Map 3.5.1

Chelmsford Wastewater System - Conduit Flow/Capacity (Design)

Scenario: ULTWWF_2yrStorm



Legend

- LS Lift Station
- WWTP Sanitary Treatment Plant
- Manhole
- Waterbody
- Road
- Watercourse
- Forcemain

Conduit: Velocity (m/s)

- <= 0.6
- <= 1.5
- <= 3.0
- > 3.0

Sudbury | WSP

Project No. 121-23026-00

December 09 2016

Sudbury Water and Wastewater Master Plan

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Map 3.5.2

Chelmsford Wastewater System - Conduit Velocity

Scenario: ULTWWF_2yrStorm

APPENDIX

C WASTEWATER GENERATION AND CAPACITY ASSESSMENTS

Chelmsford - Wastewater Flow Forecasts

	Units	2009	2010	2011	2012	2013	Summary	Design Criterion	Comments
Average Day Flow	(m^3/d)	4,753	3,712	3,888	3,704	4,729	4,157	4,157	From Annual Reports
Max Day Flow	(m^3/d)	17,896	22,090	14,440	9,200	18,210	16,367	16,367	From Annual Reports
Max Day Factor		3.77	5.95	3.71	2.48	3.85	3.95	3.95	Calculated - Max Day Flow divided by Average Day Flow
Peak Flow	(m^3/d)	Not Available						0	Peak hour flows were not available
Peak Flow	(L/s)							0	
Peak Flow Factor									
Population (Existing Areas)		7,400	7,400	7,400	7,400	7,400	7,400	7,400	
Ratio of Residential to Total Customers		0.79	0.79	0.79	0.79	0.79	0.79		This ratio is based on Water Billing Records for the area and is an approximation of the residential portion of demand.
Residential Share of Average Day Demand (m^3/d)		3759	2936	3075	2929	3740	3288		
Residential Flow Unit Rate (m^3/cap/d)		0.508	0.397	0.416	0.396	0.505	0.444	0.450	
Average Institutional Flow Unit Rate (m^3/ha/d)								28.0	CGS design standards do not suggest average rates for institutional and commercial, but require that rates be developed based on usage. In this case, water demands rates were used and assume a conservative 1:1 ratio of water to sewage.
Average Commercial Flow Unit Rate (m^3/ha/d)								28.0	CGS design standards do not suggest average rates for institutional and commercial, but require that rates be developed based on usage. In this case, water demands rates were used and assume a conservative 1:1 ratio of water to sewage.
Average Industrial Flow Unit Rate (m^3/ha/d)								35.0	CGS design standards average rate for light industrial of 35 m^3/ha/d.
Average Extraneous Flow Unit Rate (m^3/ha/d)								11.23	From CGS Design Standards, peak rate for new developments divided by an assumed peaking factor of 3. This factor would be applied only to new developments, which are assumed to be leak-tight, and have minimal extraneous flow.

	2016	2021	2026	2031	2036	2041	Ultimate Buildout	Comments
Population (Existing Areas)	7,400	7,400	7,400	7,400	7,400	7,400	7,400	
Population (Growth Areas)	117	239	363	438	486	491	3,608	
Total Population	7517	7639	7763	7838	7886	7891	11008	Total Population (Hemson)
Residential (ha)	60.00	62.46	63.81	64.78	65.59	66.57	123.52	ICI development areas were assigned to planning years based on the stage of the application. Draft Approved were assigned to 2016, Legal Lots of Record to 2026, and Designated Developable to 2036.
Institutional (ha)			1.60	1.60	1.60	1.60	1.60	
Commercial (ha)			3.75	3.75	3.75	3.75	3.75	
Industrial (ha)					45.85	45.85	45.85	Areas are cumulative and carry from the development year, all the way through to Ultimate Buildout
ICI (ha)	0.00	0.00	5.35	5.35	51.20	51.20	51.20	
Total (ha)	60.00	62.46	69.16	70.13	116.79	117.77	174.72	

	2016	2021	2026	2031	2036	2041	Ultimate Buildout	Comments
Average Residential Flows (m^3/d) - Existing	4,157	4,157	4,157	4,157	4,157	4,157	4,157	This includes all contribution from existing ICI and infiltration. The base flow was assumed to be the average day flow to the plant for the 2011-2013 period.
Average Residential Flows (m^3/d)	53	107	163	197	219	221	1,623	Obtained by multiplying the projected population growth by the unit rate.
Average Institutional Flow (m^3/d)	0	0	45	45	45	45	45	Institutional growth area multiplied by unit flow rate.
Average Commercial Flow (m^3/d)	0	0	105	105	105	105	105	Commercial growth area multiplied by unit flow rate.
Average Industrial Flow (m^3/d)	0	0	0	0	1,605	1,605	1,605	Industrial growth area multiplied by unit flow rate.
Average Extraneous Flow (m^3/day)	674	702	777	788	1,312	1,323	1,963	
Average Day Flow (m^3/d)	4,884	4,966	5,247	5,292	7,442	7,456	9,498	

	2016	2021	2026	2031	2036	2041	Ultimate Buildout
Max Day Flow (m^3/d)	19,305	19,631	20,742	20,918	29,419	29,471	37,545

ALTERNATIVE CALCULATION METHOD

	2016	2021	2026	2031	2036	2041	Ultimate Buildout	Comments
Per Capita Flow (m3/cap/day)	0.642	0.502	0.525	0.501	0.639	0.562	0.562	
Average Day Flow (m^3/d)	4222.775	4291.276901	4361.277	4403.151	4430.257	4432.898	6183.958276	Multiplying the total population by the consolidated per capita flow factor.
Max Day Flow (m^3/d)	16,692	16,963	17,240	17,405	17,513	17,523	24,445	
Peak Flow (m^3/d)	0	0	0	0	0	0	0	*Multiplying the average flow X the peak hour factor.*
Peak Flow (L/s)	0.0	0.0	0.0	0.0	0.0	0.0	0.0	

REMINDERS:

SENSITIVITY ANALYSIS

Analyze sensitivity of forecasted flows to unit rate

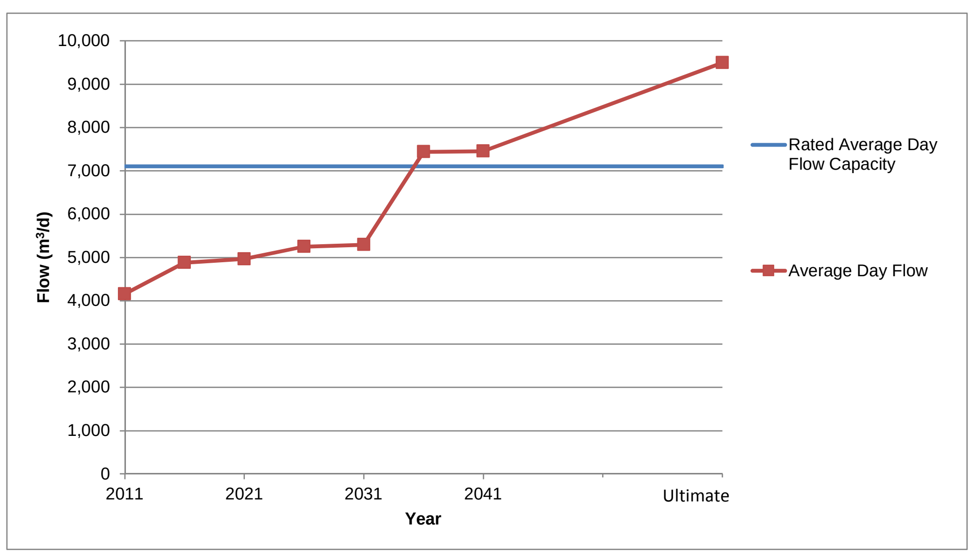
	Unit Rate (m^3/cap/d)	Average Day Flow (m^3/d) 2016	2021	2026	2031	2036	2041	Ultimate Buildout 2061
Base Scenario - Residential Historical Maximum	0.444	4,884	4,966	5,247	5,292	7,442	7,456	9,498
Combined Historical Maximum	0.562	4,897	4,993	5,288	5,341	7,497	7,510	9,901
City Standards	0.360	4,873	4,945	5,215	5,252	7,399	7,411	9,173

Analyze sensitivity of forecasted flows to max day factor

	Max Day Peaking Factor	Max Day Flow (m^3/d) 2016	2021	2026	2031	2036	2041	Ultimate Buildout 2061
Base Scenario - Historical Max	4.00	19,534	19,864	20,989	21,167	29,769	29,821	37,990
Historical Average	5.95	19,305	19,631	20,742	20,918	29,419	29,471	37,545

CAPACITY CHECK

	2011	2016	2021	2026	2031	2036	2041	Ultimate Buildout 2061
Rated WPCP ADF Capacity (m^3/d)	7,100	7,100	7,100	7,100	7,100	7,100	7,100	7,100
Average Day Flow (m^3/d)	4,157	4,884	4,966	5,247	5,292	7,442	7,456	9,498
Maximum Day Flow (m^3/d)	16,367	19,305	19,631	20,742	20,918	29,419	29,471	37,545



CITY OF GREATER SUDBURY WATER AND WASTEWATER MASTER PLAN

CONSITON WASTEWATER SYSTEM GAP ANALYSIS AND STATUS QUO REPORT

CITY OF GREATER SUDBURY

DRAFT

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

WSP
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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 existing Coniston Wastewater System. Based on population growth projections and design criteria discussed in the *Population Projections and Unit Rates Technical Memorandum* (WSP, 2014), 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 Coniston Wastewater System would continue to be a stand-alone system. Any potential interconnections between Coniston 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

Coniston is a community located in the south east end of the City of Greater Sudbury (CGS), just east of Sudbury proper. The community is serviced by a single wastewater system. Mapping in Appendix A shows the Coniston 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 *Population Projections and Unit Rates Technical Memorandum* (WSP, 2014). Existing development in the study area is mixed, and includes residential as well as industrial land uses - notably, Northern Heat Treat Ltd. is located on Smelter Road.

Based on the City's planning data, low growth is expected for Coniston. The area population is expected to increase from 2,225 in 2011 to 3,298 by Ultimate Build-out.

ICI growth is expected to be primarily industrial with very small amounts of institutional. Growth is discussed further in Section 6.1.

3 OVERVIEW OF EXISTING SYSTEM

All wastewater generated in Coniston is collected and treated at the Coniston Wastewater Treatment Plant (WWTP). The Coniston WWTP is located at 121 Government Road. There are two wastewater lift stations that discharge to the Coniston WWTP: the Edward LS and the Government LS, each conveying inflows contributed by gravity sewers. Additional information on the existing system is provided in the Baseline Review Report for Wastewater Systems (WSP, 2014). The Coniston Wastewater System is shown in Appendix B.

3.1 LIFT STATIONS

The Coniston Wastewater System receives sewage from two lift stations, each with its own catchment area. Details regarding the lift stations are summarized in the Baseline Review Report for Wastewater Systems (WSP, 2014).

Table 2-1 provides a summary of the main features of the lift stations.

Table 3-1 Edward and Government Road Lift Stations

LIFT STATION	YEAR CONSTRUCTED[1]	LAST UPGRADED	WET WELL VOLUME (M3)[1]	LIFT STATION CAPACITY AND FORCEMAIN INFORMATION[2]
Edward	1969	1971	26.5 (3.58 sq.m)	Three dry well pumps, one pump is 5 HP and the HP of the two other pumps is unclear. Based on the condition assessment the station capacity is 89.4 L/s (with one pump operational).
Government Road	1983	N/A	116.5 (18.1 sq.m)	Two submersible pumps with a firm design capacity of 18.1 L/s 285 m long, 200 mm diameter PVC

[1] Based on information provided by CGS or the 2013 Wastewater Lift Stations Operating Manual.

[2] Contract ISD14-34 Condition Assessment Edward Wastewater Lift Station

3.2 CONISTON WWTP

The Coniston WWTP is an oxidation ditch system with rated capacity of 3,000 m^3/d. Major system upgrades were completed in the late 1980s. Based on the annual operation reports, the system is well maintained and operating satisfactorily. The effluent is disinfected only during the period of May 1 to October 31.

The treatment process is illustrated schematically in Figure 3-1 below

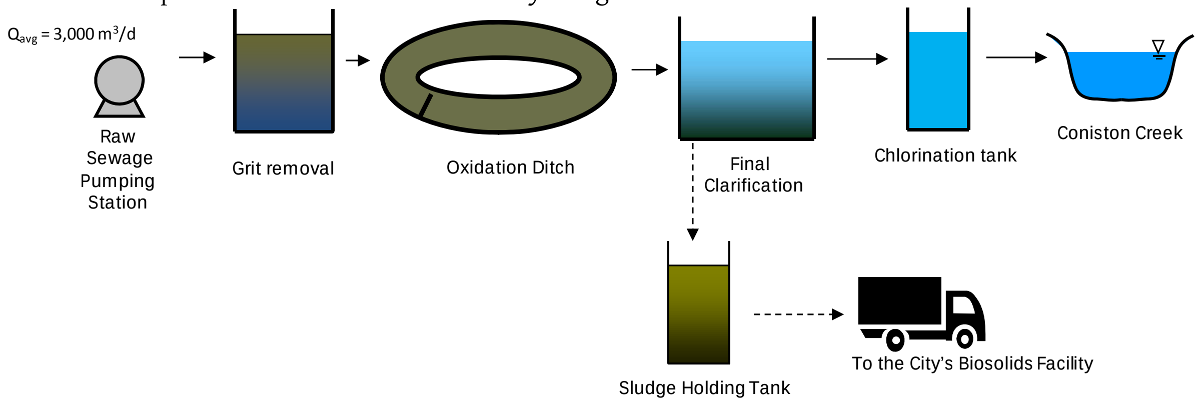

Figure 3-1 Coniston WWTP Process

3.3 KNOWN CHALLENGES

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

— Fifteen plant bypasses were reported at the Coniston WWTP between 2004 and 2012. Most of these occurred at the Coniston WWTP as a result of heavy precipitation and snow melts, contributors to inflow and infiltration concerns. For example, there were 4 bypasses in 2011, 1 in 2012, 2 in 2013 and 5 in 2014. The largest bypass on record occurred on April 14 2014, a rain-on-snow event that caused overflows throughout the greater Sudbury area.

— Both the Edward and Government Road LS's are, according to the City's observations, at capacity during rainstorm events. From 2009-2011, three overflows were reported for the two lift stations; two at the Government Road LS and one at the Edward LS.

— The City has attempted in recent years to decrease the amount of wastewater flow entering the two lift stations. In 2010, the City enforced a by-law which included a provision to regulate what flows can enter the sanitary network. Specifically, any private storm systems were to be disconnected from the municipal sewage system. Additionally, from 2010 to 2012, the City initiated another program to buyout private sump pumps and downspouts to decrease the amount of stormwater infiltration into the sewer system. The City has not noted a significant decrease in wastewater flows as a result of the policies and programs implemented.

4 HISTORICAL FLOWS AND REVIEW OF OPERATIONAL DATA

Data reported in the 2009 to 2013 *Annual Reports* for the Coniston WWTP was reviewed and analyzed to determine average day and maximum day flows as well as review effluent parameters.

4.1 FLOW DATA

WWTP flow data from 2009 to 2013 was reviewed. Historical peak flow data was not available.

The recorded average day and maximum day flows are plotted in Figure 4-1 below.

Table 4-1 Historical Wastewater Flow Data

YEAR	AVERAGE DAY FLOW (M^3/D)[1]	MAXIMUM DAY FLOW (M^3/D)[1]
2009	1,303	3,777
2010	885	2,424
2011	981	3,618
2012	931	3,986
2013	1,239	5,875

[1] Data from Annual Reports (2009 - 2013).

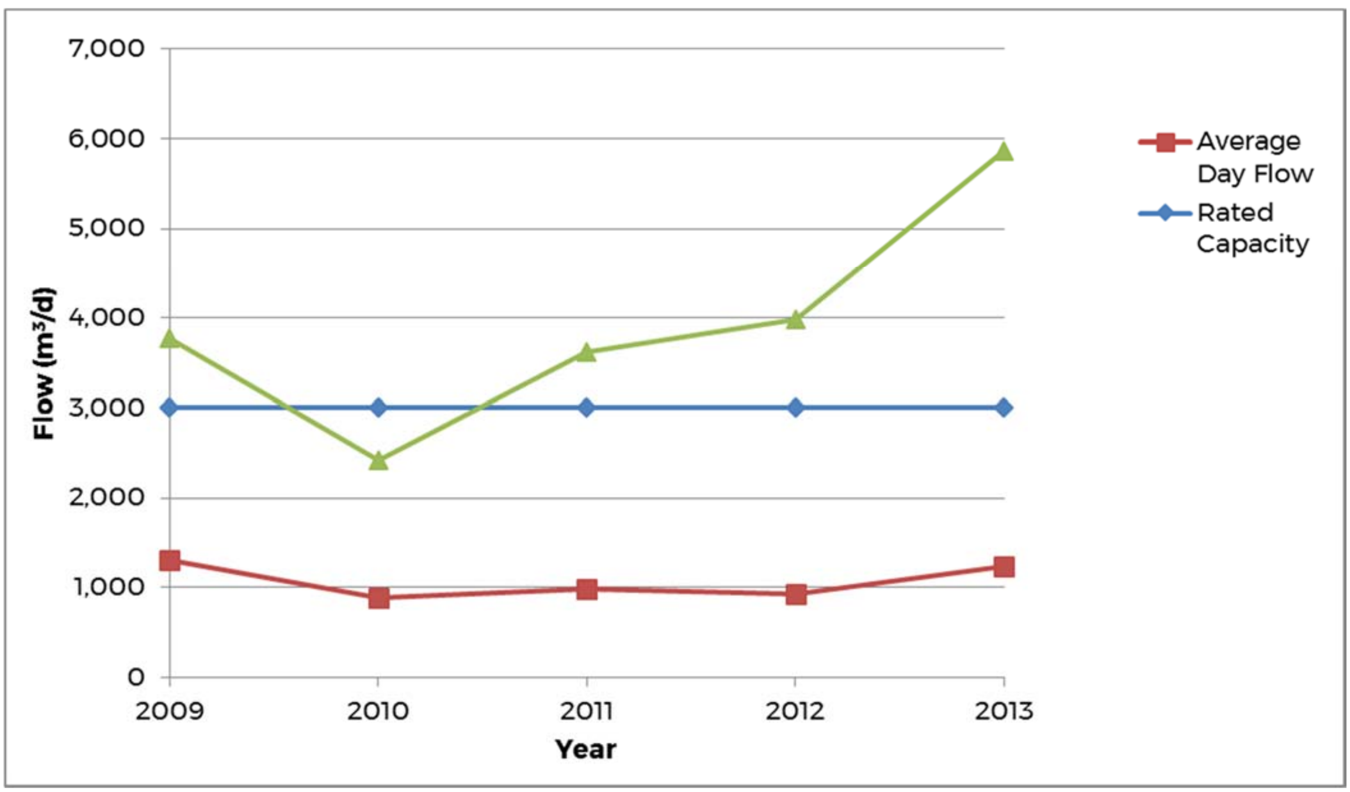

Figure 4-1 Historical Wastewater Flows at the Coniston Wastewater Treatment Plant

The relationship between the different flow regimes was analyzed to compare the maximum day peaking factors derived from historical data to those used in the *City's Engineering Design Manual* and those included in the *MOECC Guidelines.* The average day flows to the WWTP have shown moderate variability over the 2009 to 2013 period, averaging 1,068 m^3/d. The variations in historical maximum day flows show considerable variability.

Upon comparison with historical rainfall data for the Sudbury Station, one finds there is no clear correlation between the maximum day flows and rainfall data. For example, 2009 had the most precipitation (986.4 mm) but flows to the plant were lower than 2012, which had a much lower precipitation (804 mm). This is not to say that the maximum day demands are not being caused by wet weather events. The Sudbury Station rainfall data may not be an accurate representation of the rainfall in Coniston since it is located near the airport, north of Coniston. To confirm the correlation between fluctuations in maximum day flows at the plant and rainfall events, rainfall data collected at a station in Coniston would be required for at least five years consecutively. Such data was not available during the preparation of this gap report. The City did confirm; however, that it is not uncommon for a bypass event to occur under wet weather flow conditions.

The highest maximum day to average day peaking factor identified was 4.74 (2013). The average maximum day peaking factor from 2009 to 2013 was 3.67. The City's *Engineering Design Manual* and the *MOECC Guidelines* do not specify recommended maximum day factors: instead, they recommend using historical data when available.

In calculating future wastewater generation, the average peaking factor (3.67) was used, based on the assumption that new developments would have less I&I due to more leak tight construction.

4.2 RAW WASTEWATER CHARACTERISTICS

The average raw wastewater characteristics from 2009 to 2012 are summarized in Table 4-2 below.

Table 4-2 Average Raw Wastewater Characteristics at the Coniston WWTP (2008-2012)

PARAMETER	AVERAGE VALUE
$CBOD_5$	156.5 mg/L
Suspended Solids	139.5 mg/L
Total Phosphorus	4 mg/L
TKN	24.8 mg/L
pH	6.95

Wastewater flows to the Coniston WWTP correspond to mixed uses, with contributions from residential and ICI users, and dilution from inflow and infiltration.

4.3 EFFLUENT CRITERIA

The Coniston WWTP is operated in accordance with *MOECC Certificate of Approval (C of A) No. 3-0215-86-007* dated April 22, 1986.The C of A for the Coniston WWTP stipulates that the effluent concentrations of $CBOD_5$ and Suspended Solids not exceed 20 mg/L each.

4.4 OPERATIONAL DATA

The general operation of the Coniston WWTP was reviewed against the C of A requirements and historical data provided in the Annual Reports from 2009-2012. Historical data is summarized in the table below.

Table 4-3 Historical Effluent Concentrations

EFFLUENT PARAMETER	ANNUAL AVERAGE			
	2009	2010	2011	2012
CBOD5 (mg/L)	9.7	15.7	8.3	1.8
TSS (mg/L)	5.8	5.8	5.9	4.7
TP (mg/L)	1.45	1.48	1.62	1.34
pH	7.10	7.00	6.40	6.99
TAN (mg/L)	18.62	12.30	14.68	9.28
E. coli (organisms/100 mL)	65	52	109	8

Historically, the Coniston WWTP has met all concentration limits.

A capacity review of each unit process at the WWTP was not conducted. Instead, the rated capacity was considered the true capacity of the plant.

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 wastewater system. The unit rates used to estimate future 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 WASTEWATER DESIGN CRITERIA

The unit flow criteria for growth adopted for this assessment are shown in the table below. These values were recommended in the *Populations and Unit Rates Technical Memorandum* (WSP, 2014).

Note that the term "extraneous flows" is used interchangeably with "I&I flows".

Table 5-1 Wastewater System Design Criteria

CRITERIA	VALUE	REFERENCE
Average Day Residential Generation	400 L/cap/day	City's Engineering Design Manual, rounded down from 410 L/cap/d
Average Day Institutional & Commercial Generation	28 m^3/ha/d	Water unit rates, assuming a 1:1 correspondence
Average Industrial Generation	35 m^3/ha/d	Water unit rates, assuming a 1:1 correspondence
Average Extraneous Flow	11.2 m^3/ha/d	Peak from City's Engineering Design Manual and assuming a peaking factor of three
Peak Extraneous Flow	33.7 m^3/ha/d	City's Engineering Design Manual
Max Day Peaking Factor	3.67	Average of historical values

Residential average day flows were obtained by multiplying the residential unit rate by the service population.

Maximum day flows to the WWTP are obtained by multiplying the average day flow by the maximum day peaking factor.

5.2 DESIGN CRITERIA FOR WASTEWATER SYSTEM COMPONENTS AND OPERATION

5.2.1 WASTEWATER TREATMENT

Wastewater treatment facilities are rated for average day flows. Plant effluent limits and objectives are established in the C of A or ECA for each facility.

5.2.2 LIFT STATION PUMPING CAPACITY

The firm capacity of the lift station (with the largest pump out of service) must allow pumping of peak wet weather flows corresponding to its catchment area (MOECC, 2008).

Starting limitations on pump motors generally dictate the minimum size of a wet well. The wet well should be large enough to prevent pump motors from overheating due to frequent starting and stopping, but small enough to avoid long retention times leading to septicity and odor problems (Lin & Lee, 2001).

The station wet well shall be sized such that the number of pump starts per hour does not exceed the maximum value recommended by the pump manufacturer. In other words, the time between pump starts and stops (i.e. the pump cycle time) should not exceed that which results in a pump start frequency greater than that recommended by the pump manufacturer. Typically, submersible pumps can cycle four to 10 times per hour with a maximum cycle time not exceeding 30 minutes (Lin & Lee, 2001). A maximum value of four pump starts per hour was assumed to evaluate wet well sizing requirements.

5.2.3 SEWERS

The sewer system is typically sized to convey peak instantaneous (peak wet weather) flows. Sewage flows are made up of wastewater discharges from residential, commercial, institutional and industrial establishments, plus extraneous flow components from such sources as groundwater and surface runoff.

In addition to being able to convey peak flows, sufficient flow velocity should be maintained under low (dry weather) flow conditions to transport sewage solids without deposition and the development of nuisance conditions. The minimum acceptable flow velocity in sewers is 0.6 m/s (City of Greater Sudbury, 2012).

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 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 Coniston is projected to increase by 69 people in 2041 and 1,073 by Ultimate Buildout. The population projections to be used in the Master Plan are summarized in Table 6-1 below.

Table 6-1 Coniston Population Projections

SYSTEM	2011	2016	2021	2026	2031	2036	2041	ULTIMATE BUILDOUT
Coniston	2,225	2,242	2,260	2,277	2,287	2,293	2,294	3,298

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 below.

Table 6-2 Coniston ICI Projections

	ICI DEVELOPMENT AREAS (HA)						
LAND USE	2016	2021	2026	2031	2036	2041	BUILDOUT
Institutional	0	0	0.14	0	0	0	0
Commercial	0	0	0	0	0	0	0
Industrial	0	0	0	0	36.36	0	0
Total	**0**	**0**	**0.14**	**0**	**36.36**	**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 Coniston, one street has been placed on the priority list for sewer and water 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 Coniston 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 WASTEWATER FLOW PROJECTIONS AND INFRASTRUCTURE NEEDS

The unit flow criteria indicated in Section 5.2 were used to estimate the future wastewater flows in Coniston. In general, the projected flows were calculated by the following formula:

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

The flows corresponding to the population growth forecasts to Ultimate Buildout are presented in Table 6-3 below.

Table 6-3 Flow Projections

YEAR	POPULATION	AVERAGE DAY FLOW (M^3/D)	MAXIMUM DAY FLOW (M^3/D)
Base	2,225	1,068	3,936
2016	2,242	1,084	3,978
2021	2,260	1,102	4,044
2026	2,277	1,132	4,153
2031	2,287	1,136	4,168
2036	2,293	2,841	10,428
2041	2,294	2,842	10,428
Ultimate	3,298	3,619	13,282

The Base Demand was the average historical (2009 to 2013) average and maximum 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 demand was calculated by applying the respective peaking factor to the average day demand. Base maximum day demand was the historical average.

A desktop analysis of historical wastewater flows and future flow projections is included in Appendix C.

6.4.1 WWTP CAPACITY

The rated average day capacity of the Coniston WWTP is 3,000 m^3/d, and is compared to the current and future flow projections on Figure 6-1 below.

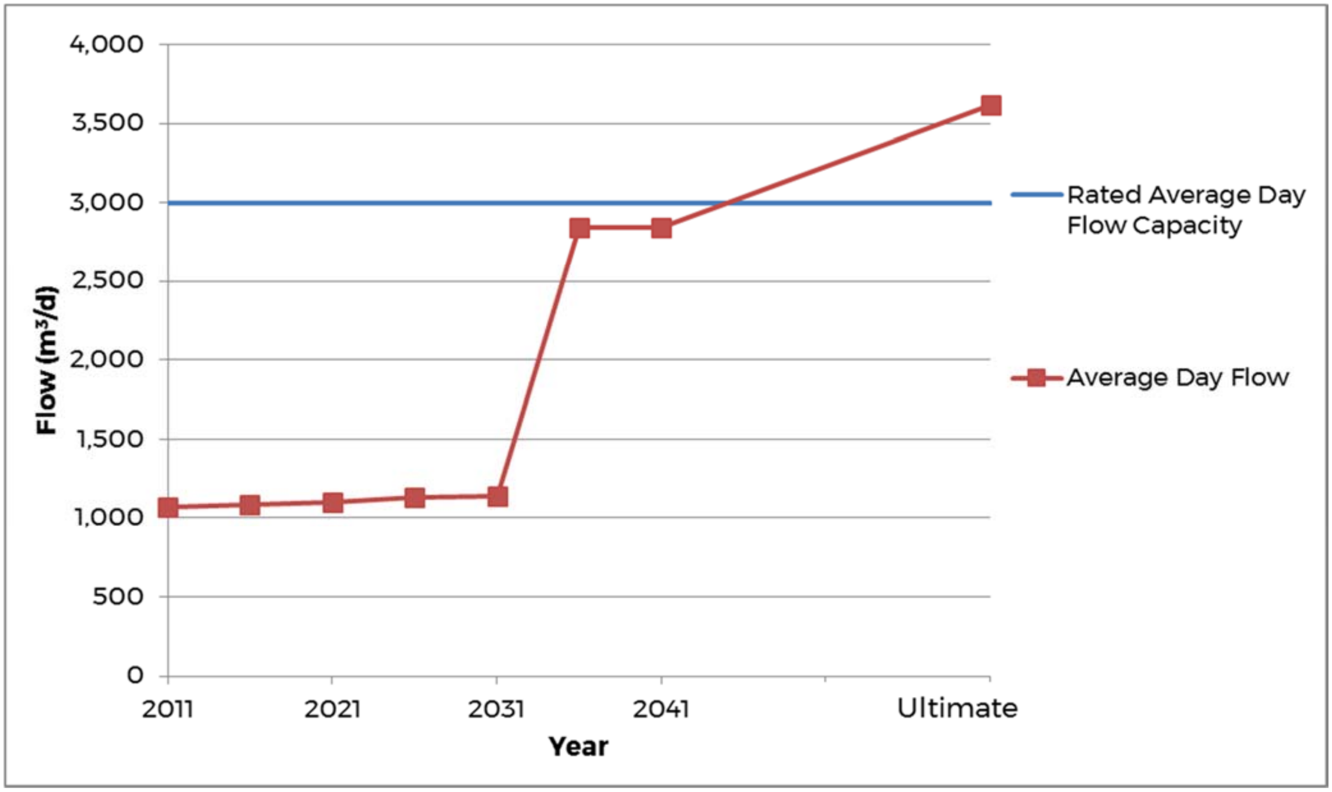

Figure 6-1 Wastewater Flow Projections Compared to Coniston WWTP Rated Capacity Historical

As indicated in the above analysis, the Coniston WWTP can continue operating under its current capacity until approximately 2041. However, maximum day demands are projected to reach 10,248 m^3/d by 2041 and 13,282 m^3/d by Ultimate Buildout and substantially exceed the average day flow plant capacity.

Since the C of A for the plant does not list a maximum day flow capacity, careful consideration will have to be given regarding the extent of the plant expansion. This exercise will be undertaken when considering the servicing solutions.

6.4.2 SEWER NETWORK AND LIFT STATIONS

For each of the scenarios modeled, the system was checked for surcharging of sewers and capacity exceedance at the pumping stations. The peak flows into each of the pumping stations determined through modeling of the various planning scenarios are shown in the table below.

Table 6-4 Coniston Lift Station Peak Inflow Rates during a 2-year Storm

	CURRENT FIRM CAPACITY (L/S)	EXISTING PEAK FLOW (L/S)	2041 PEAK FLOW (L/S)	ULTIMATE BUILDOUT (L/S)
Edward	89.4	106.9	107.2	108.9
Government Road	18.1	125.5	137.3	138.7

The capacity at the Edward LS is not sufficient to convey peak inflows for the 2011, 2041, and Ultimate Buildout scenarios. Similarly, the peak inflows for 2011, 2041, and Ultimate Buildout exceed the Government Road firm capacity. A new lift station is required since the modeled wastewater flows greatly exceed the existing station's capacity. Before undertaking this project, the City should monitor the flows at within the system and at the lift station and further refine the future projected flows.

7 HYDRAULIC MODELLING

7.1 APPROACH

A basic sanitary model for the City of Greater Sudbury was received from the City. The model was created in Bentley's SewerGEMS by City staff and updated by WSP as required. The all-pipe model of the sanitary collection system includes key vertical infrastructure in the system, including lift stations and treatment facilities.

The model was loaded with wet weather flow data. A water balance was completed to determine I&I rates for both dry and wet weather flow, and the average I&I rate from several sewersheds in the greater Sudbury area were also considered and modelled for Coniston. There has been no flow monitoring completed to-date for the Coniston area. Therefore the greater of the average I&I monitored results and the I&I from the water balance were used to load the model.

Current (2011) and future (2016-Ultimate Build-out, in 5 year increments) population data was added to the model using the City's planning data, summarized in previous sections of this report.

Future dry and wet weather flow scenarios were developed for each of the horizon years: 2016, 2021, 2026, 2031, 2036, 2041, and Ultimate Build-out. However, model results varied marginally from 2016 to 2041; therefore, this report discusses findings for 2041 and Ultimate Build-out, compared against existing (2011) – all during a theoretical 2-year storm to determine the system's ability to perform during a significant load case.

7.2 MODELLING FINDINGS

The model was used to check sewer capacity and flow velocity. Maps in Appendix B illustrate the modeling results for the 2011, 2041, and Ultimate wet weather flow scenarios based on a theoretical 2-year storm.

Many of the sewers in the Coniston system operate at less than 50% capacity for 2011, 2041, and Ultimate Buildout scenarios. Some sewer segments, however, operate at or above 100% capacity, as illustrated in the appended maps.

Flow velocity in some sewers in the Coniston system is below the City's standard of 0.6 m/s for 2011, 2041, and Ultimate Buildout, as shown in the appended maps.

8 CONCLUSIONS

An assessment of the Coniston Wastewater System was completed to identify infrastructure requirements to service forecasted growth in the community.

The conclusions of the assessment are summarized below.

— Albeit the WWTP is deemed to have sufficient average day capacity to service growth to 2041, the projected maximum day demands are substantially higher than the current plant average day capacity.

— Both lift stations do not have enough capacity to convey the peak influent from 2011 through to Ultimate Buildout.

— Many of the sewers in Coniston operate at velocities that do not meet the City's current standards (that is, many sewers flow at less than 0.6 L/s). This may cause operational problems, such as solids buildup and odours.

— Some sewers in Coniston are at or above 100% capacity.

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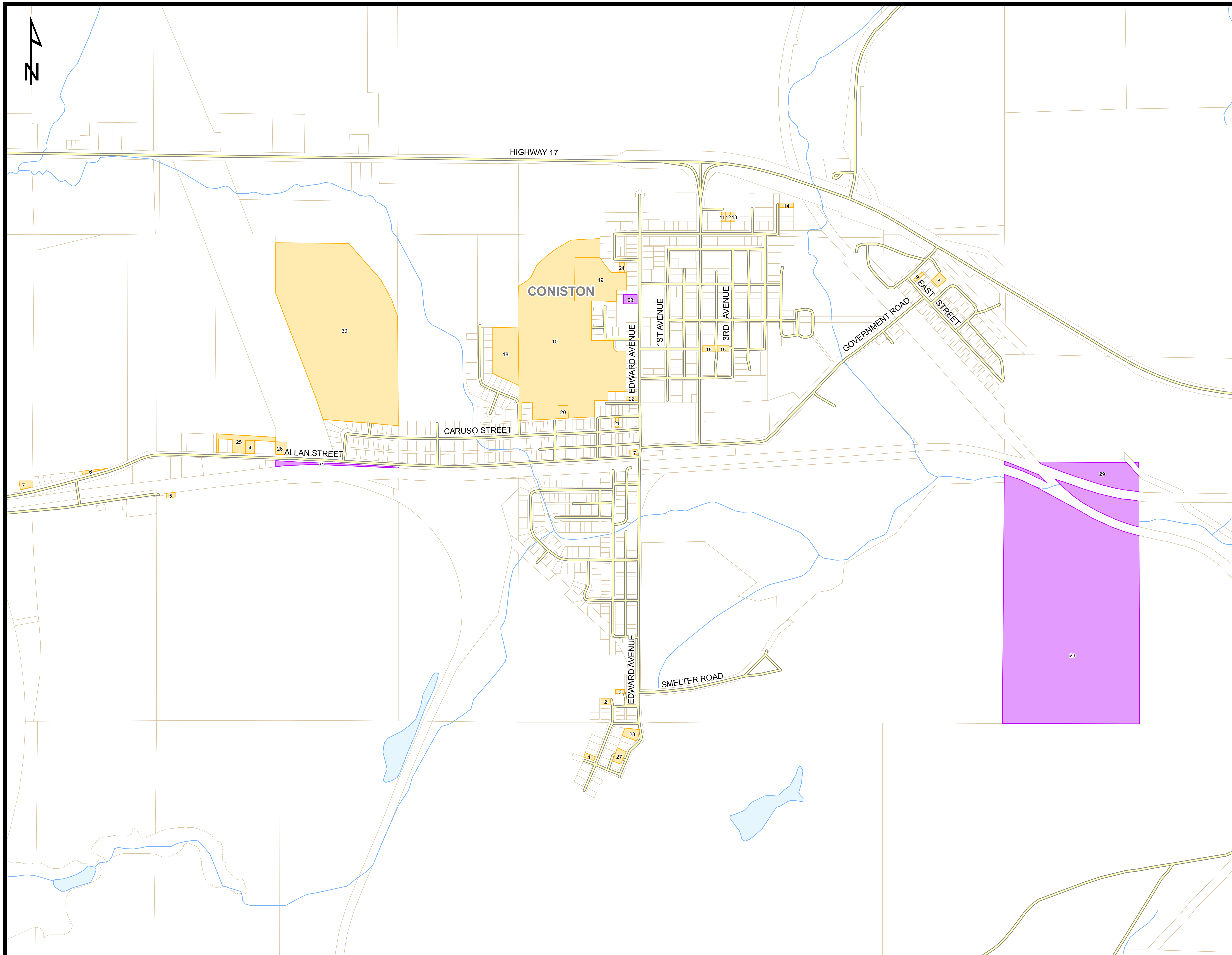
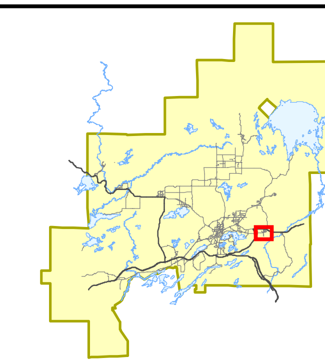
APPENDIX

A RESIDENTIAL AND ICI DEVELOPMENT AREAS



Legend

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



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0 100 200 300 400 500 m

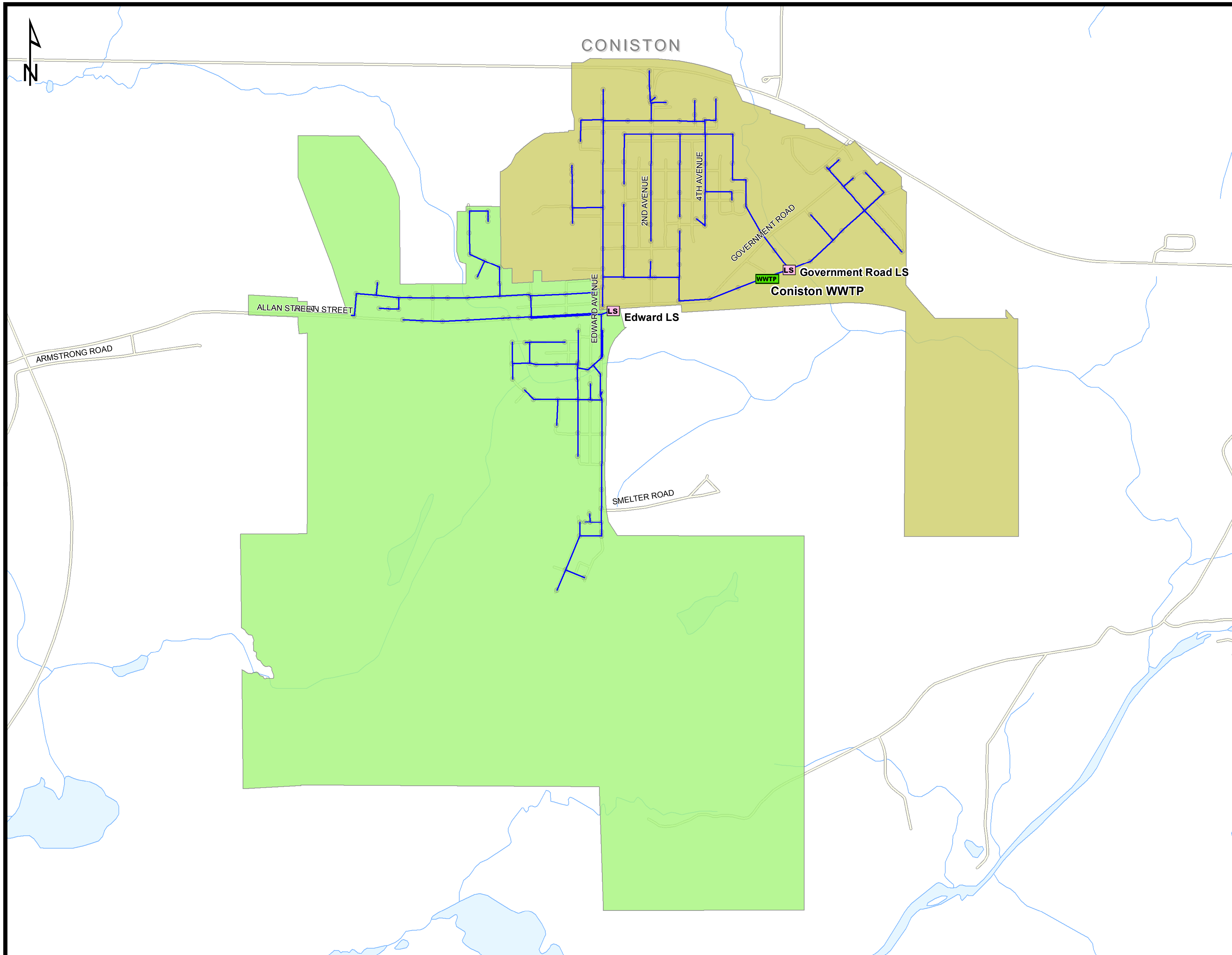
1:12,500

Map 5

Coniston
2011 - Ultimate Buildout Residential and ICI Development Areas

APPENDIX

B WASTEWATER MODEL RESULTS



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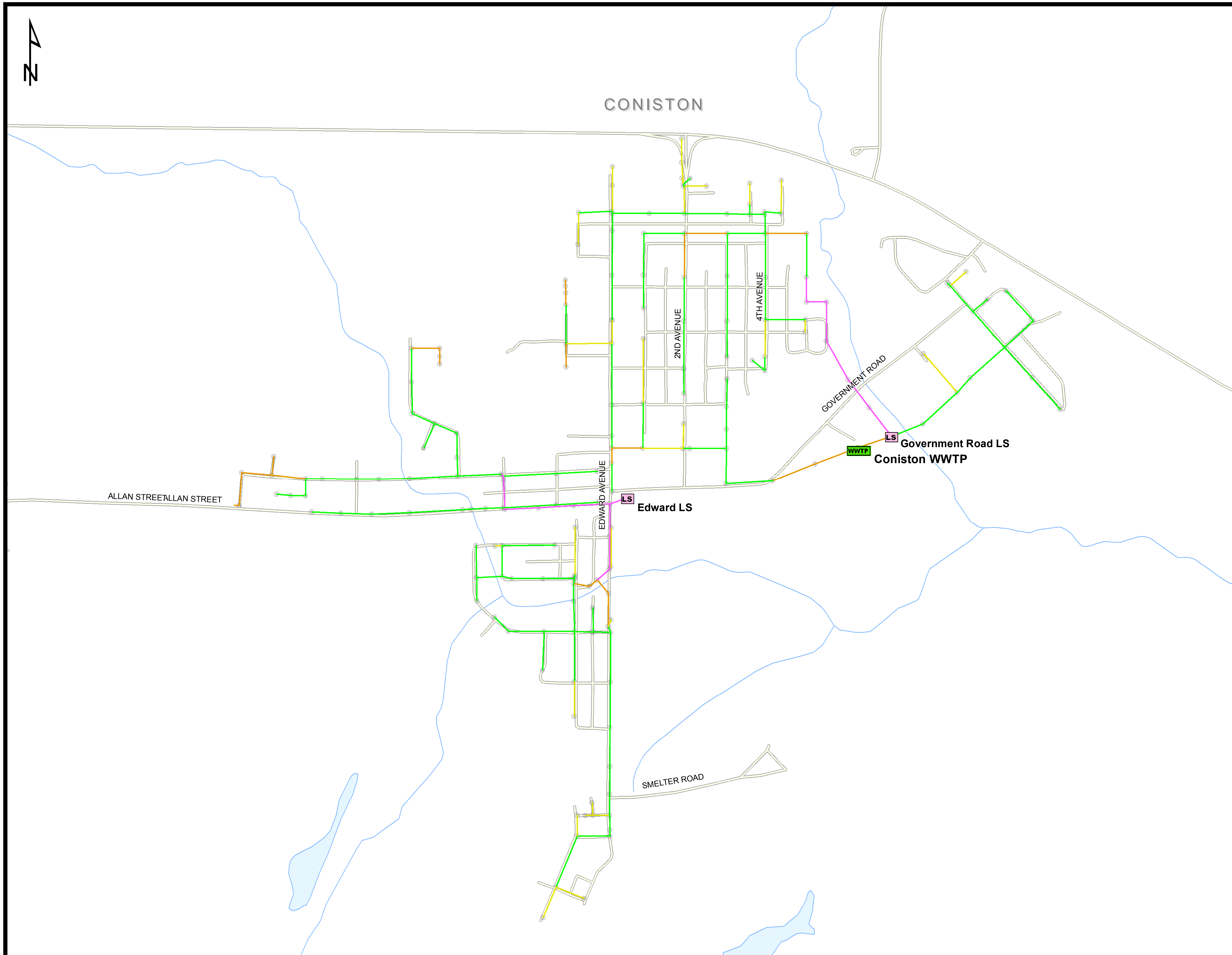
Sudbury Water and Wastewater Master Plan

Map 2.1

Coniston Wastewater System - Sewershed Map



Legend

- LS Lift Station
- WWTP Sanitary Treatment Plant
- Manhole
- Waterbody
- Road
- Watercourse
- Forcemain

Conduit: Diameter (mm)

- <= 150
- <= 200
- <= 300
- <= 450
- <= 600
- <= 750
- > 750

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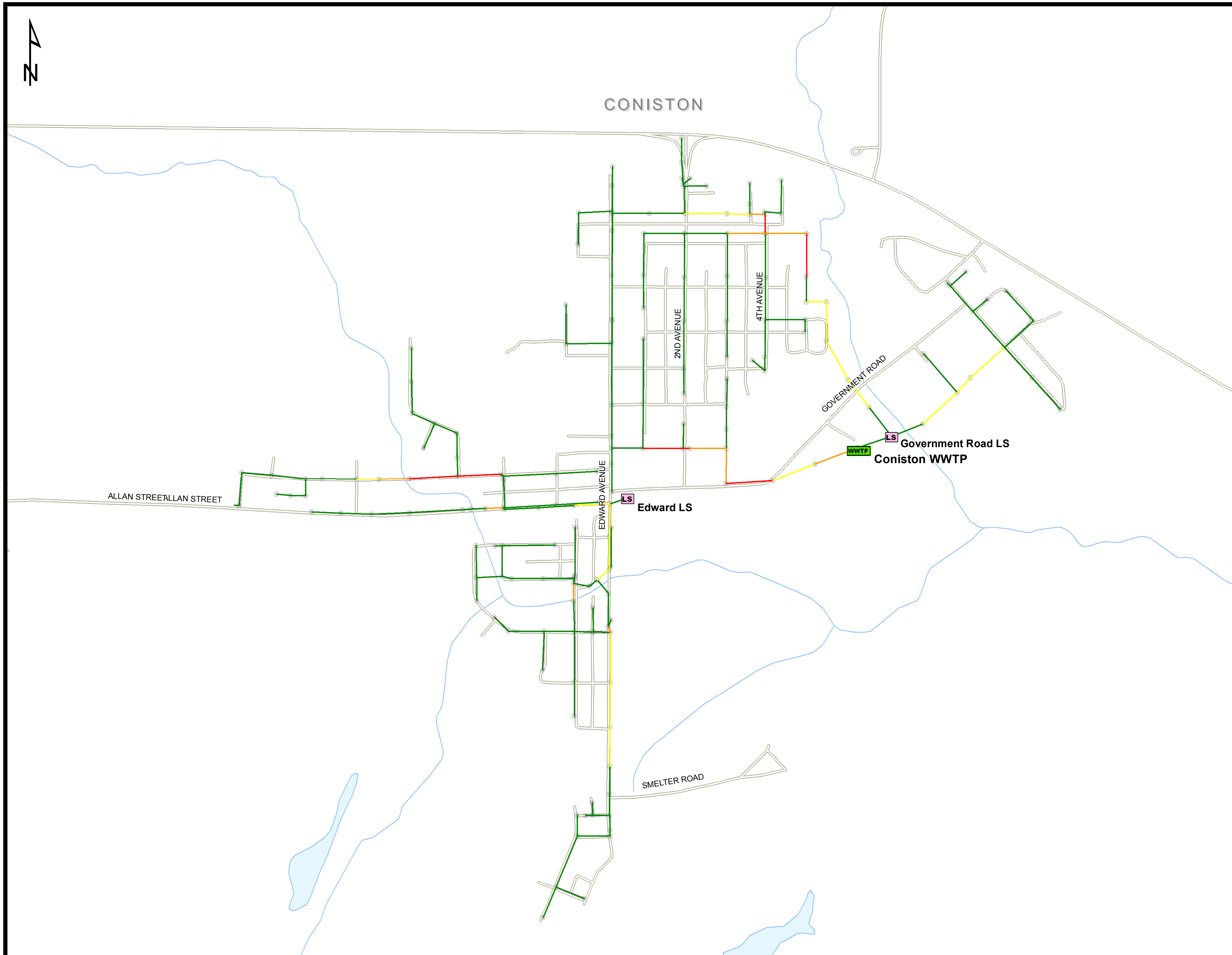
0 200 400 600 m

1:10,000

Map 2.2

Coniston Wastewater System - Conduit Diameter



Legend

- LS Lift Station
- WWTP Sanitary Treatment Plant
- Manhole
- Waterbody
- Road
- Watercourse
- Forcemain

Conduit: Flow/Capacity (Design) (%)

- <= 50
- <= 80
- <= 100
- > 100

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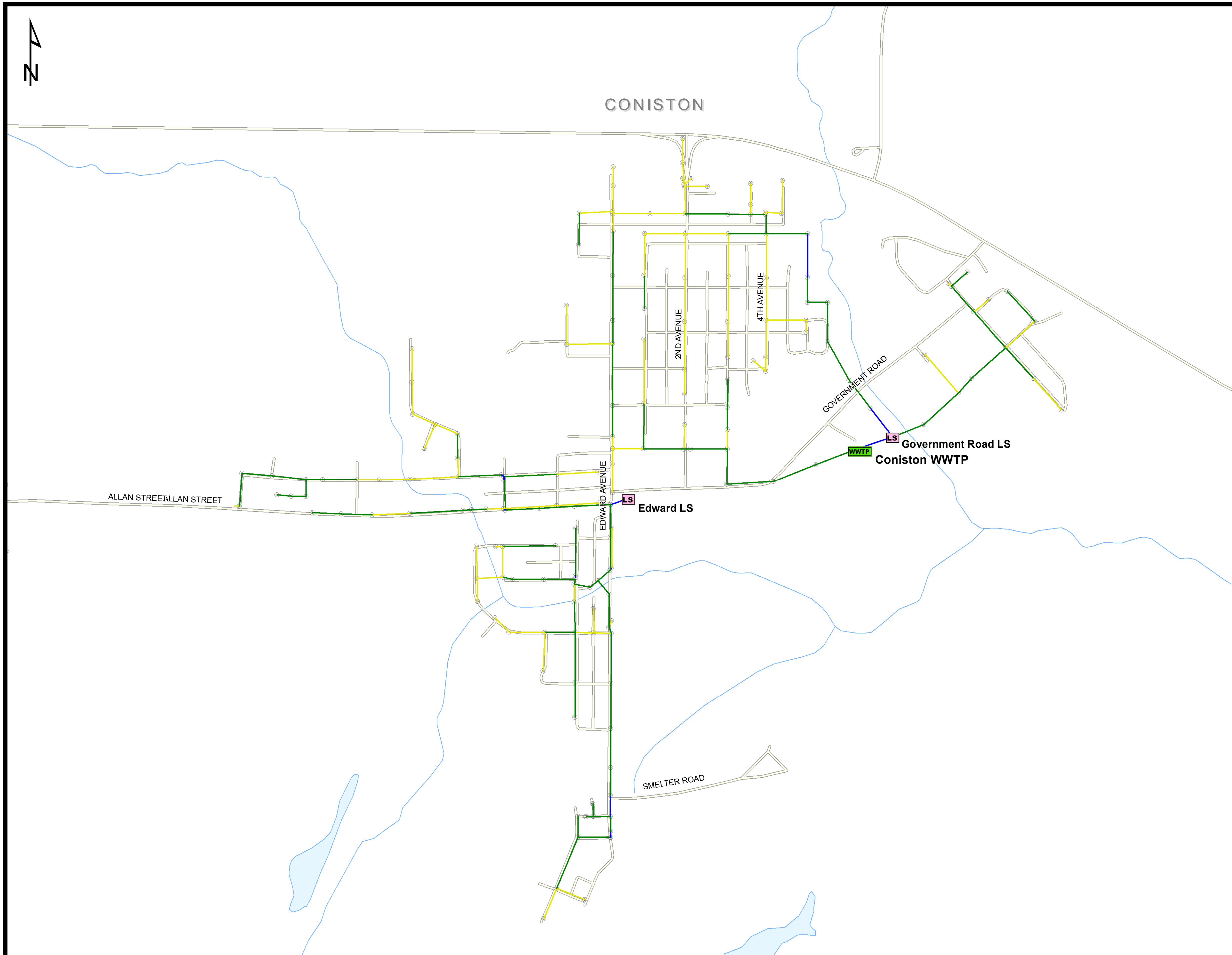
Map 2.3.1

Coniston Wastewater System - Conduit Flow/Capacity (Design)

Scenario: 2011WWF_2yrStorm



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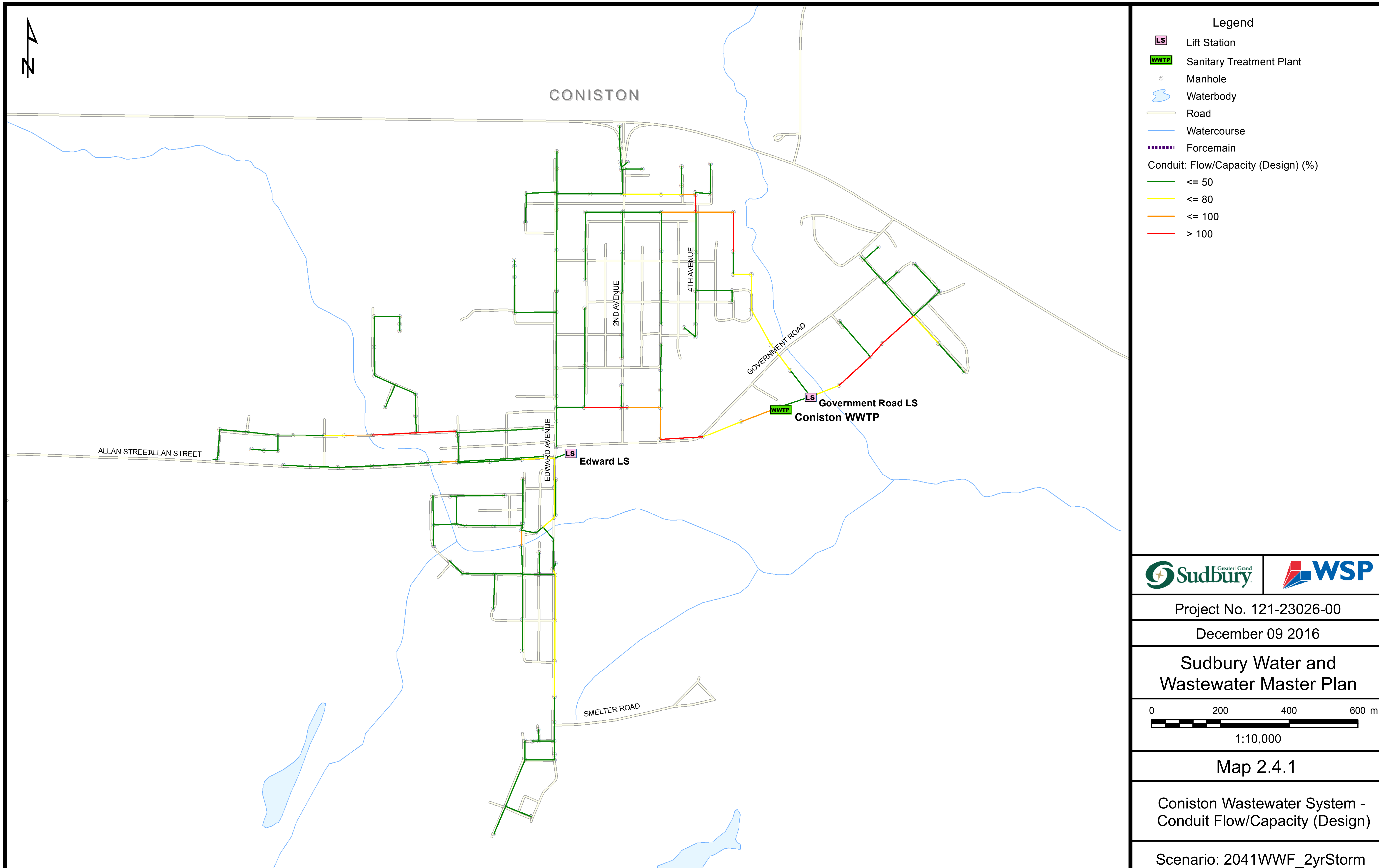
Map 2.3.2

Coniston Wastewater System - Conduit Velocity

Scenario: 2011WWF_2yrStorm



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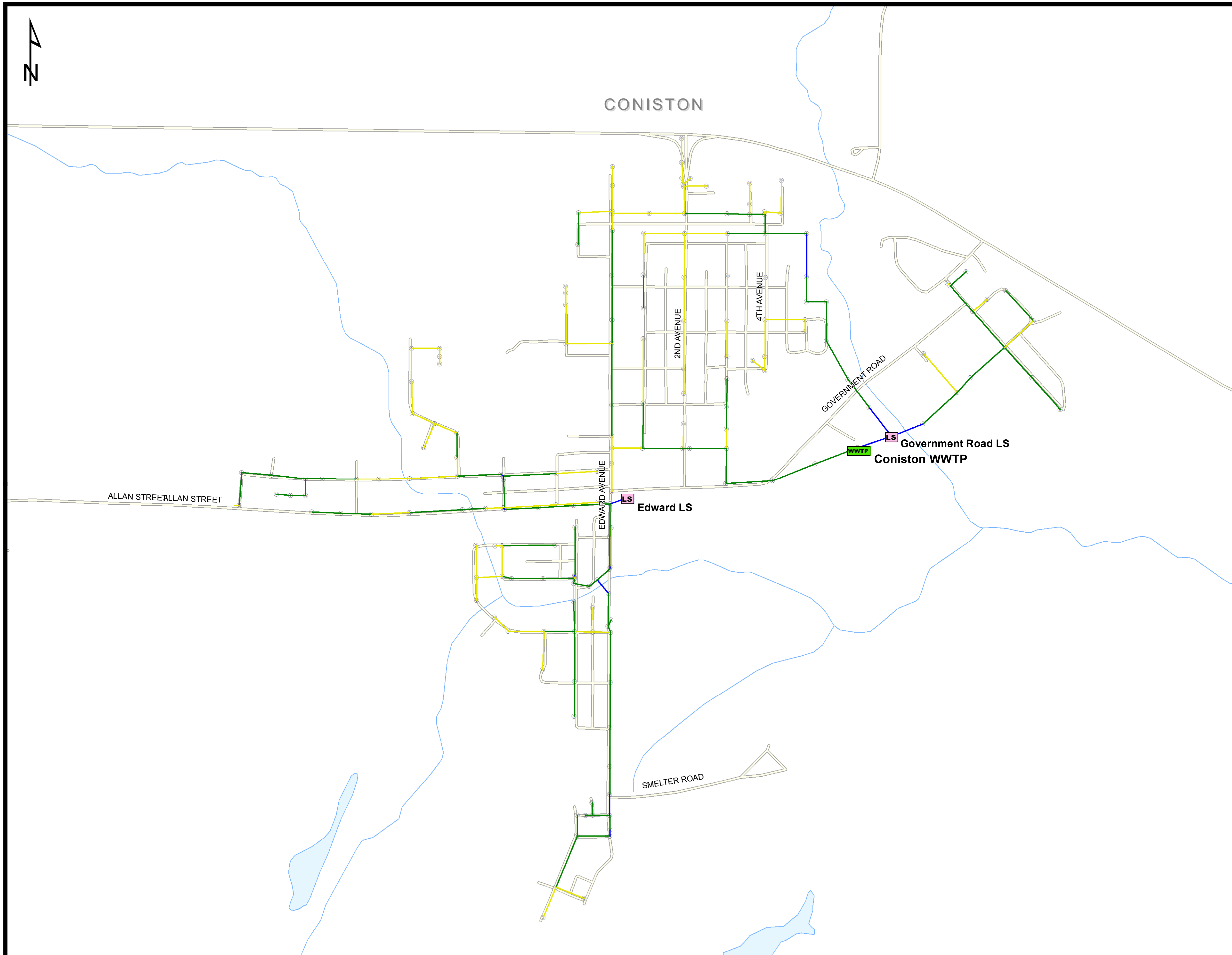
Map 2.4.1

Coniston Wastewater System - Conduit Flow/Capacity (Design)

Scenario: 2041WWF_2yrStorm



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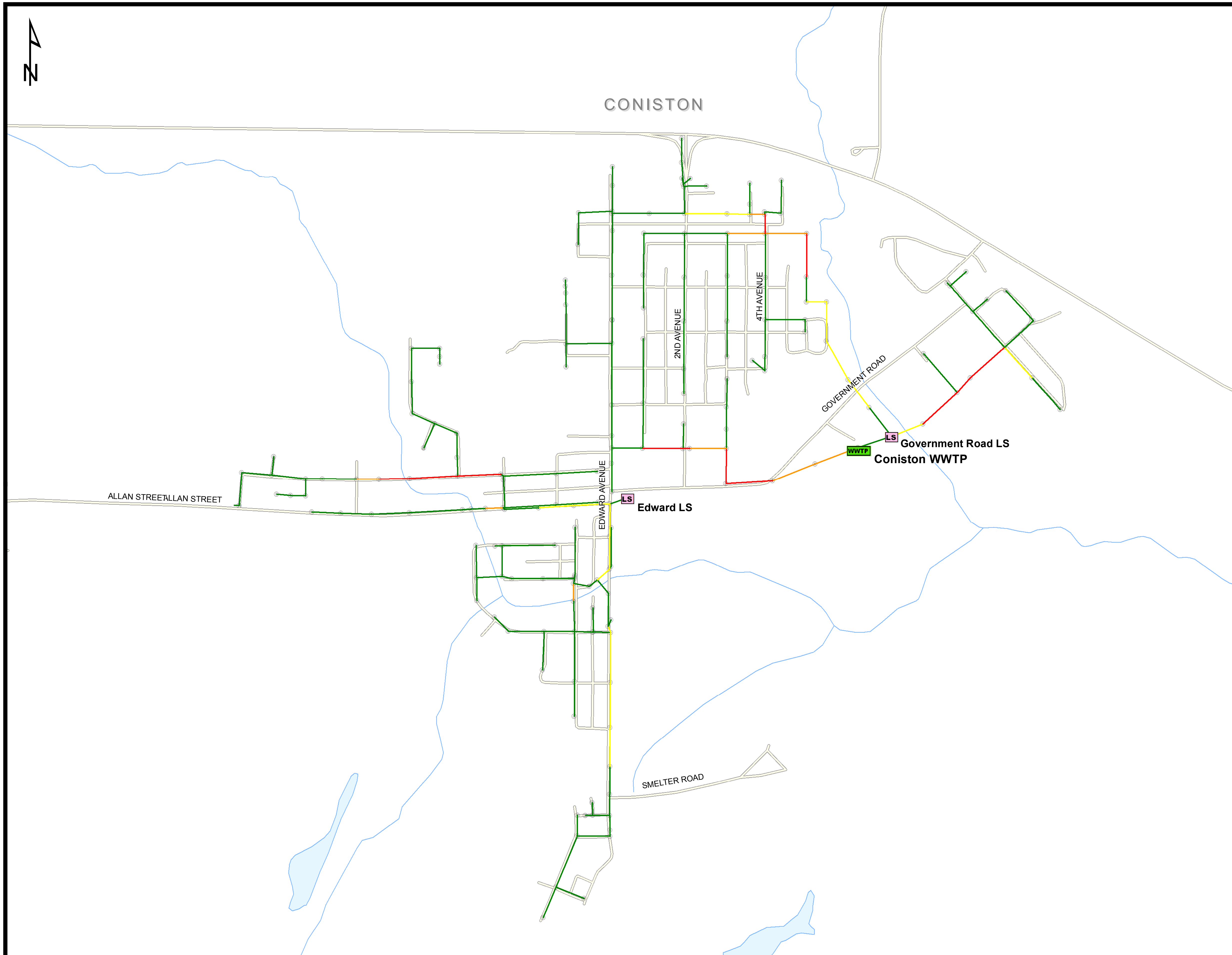
Map 2.4.2

Coniston Wastewater System - Conduit Velocity

Scenario: 2041WWF_2yrStorm



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Sudbury Water and Wastewater Master Plan

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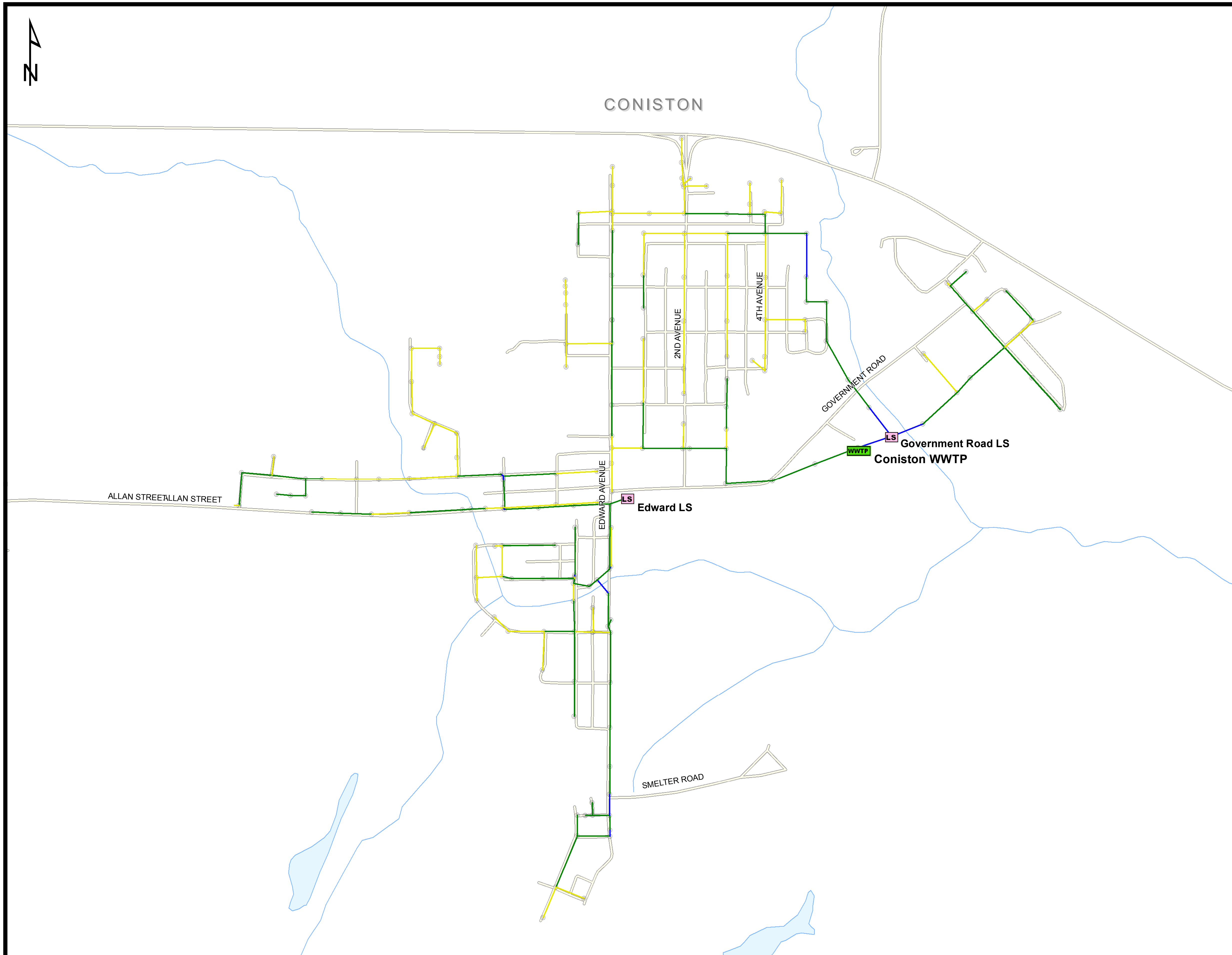
Map 2.5.1

Coniston Wastewater System - Conduit Flow/Capacity (Design)

Scenario: ULTWWF_2yrStorm



Project No. 121-23026-00

December 09 2016

Sudbury Water and Wastewater Master Plan

Map 2.5.2

Coniston Wastewater System - Conduit Velocity

Scenario: ULTWWF_2yrStorm

APPENDIX

C WASTEWATER GENERATION AND CAPACITY ASSESSMENTS

Coniston - Wastewater Flow Forecasts

		2009	2010	2011	2012	2013	Summary	Design Criterion	Comments
Average Day Flow	**(m^3/d)**	1,303	885	981	931	1,239	1,068	1,068	From Annual Reports
Max Day Flow	**(m^3/d)**	3,777	2,424	3,618	3,986	5,875	3,936	3,936	From Annual Reports
Max Day Factor		2.90	2.74	3.69	4.28	4.74	3.67	3.67	Calculated - Max Day Flow divided by Average Day Flow
Peak Flow	**(m^3/d)**	Not Available						0	Peak hour flows were not available
Peak Flow	**(L/s)**							0	
Peak Flow Factor									
Population (Existing Areas)		2,225	2,225	2,225	2,225	2,225	2,225	2,225	
Ratio of Residential to Total Customers		0.80	0.80	0.80	0.80	0.80	0.80		This ratio is based on Water Billing Records for the area and is an approximation of the residential portion of demand.
Residential Share of Average Day Demand (m^3/d)		1049	712	790	749	997	859		
Residential Flow Unit Rate (m^3/cap/d)		0.471	0.320	0.355	0.337	0.448	0.386	0.400	
Average Institutional Flow Unit Rate (m^3/ha/d)								28.0	CGS design standards do not suggest average rates for institutional and commercial, but require that rates be developed based on usage. In this case, water demands rates were used and assume a conservative 1:1 ratio of water to sewage.
Average Commercial Flow Unit Rate (m^3/ha/d)								28.0	CGS design standards do not suggest average rates for institutional and commercial, but require that rates be developed based on usage. In this case, water demands rates were used and assume a conservative 1:1 ratio of water to sewage.
Average Industrial Flow Unit Rate (m^3/ha/d)								35.0	CGS design standards average rate for light industrial of 35 m^3/ha/d.
Average Extraneous Flow Unit Rate (m^3/ha/d)								11.23	From CGS Design Standards, peak rate for new developments divided by an assumed peaking factor of 3. This factor would be applied only to new developments, which are assumed to be leak-tight, and have minimal extraneous flow.

	2016	2021	2026	2031	2036	2041	Ultimate Buildout	Comments
Population (Existing Areas)	2,225	2,225	2,225	2,225	2,225	2,225	2,225	
Population (Growth Areas)	17	35	52	62	68	69	1,073	
Total Population	2242	2260	2277	2287	2293	2294	3298	Total Population (Hemson)
Residential (ha)	0.85	1.78	3.34	3.34	5.33	5.33	38.79	ICI development areas were assigned to planning years based on the stage of the application. Draft Approved were assigned to 2016, Legal Lots of Record to 2026, and Designated Developable to 2036.
Institutional (ha)			0.14	0.14	0.14	0.14	0.14	
Commercial (ha)								
Industrial (ha)					36.36	36.36	36.36	Areas are cumulative and carry from the development year, all the way through to Ultimate Buildout
ICI (ha)	0.00	0.00	0.14	0.14	36.50	36.50	36.50	
Total (ha)	0.85	1.78	3.48	3.48	41.83	41.83	75.29	

	2016	2021	2026	2031	2036	2041	Ultimate Buildout	Comments
Average Residential Flows (m^3/d) - Existing	1,068	1,068	1,068	1,068	1,068	1,068	1,068	This includes all contribution from existing ICI and infiltration. The base flow was assumed to be the average day flow to the plant for the 2011-2013 period.
Average Residential Flows (m^3/d)	7	14	21	25	27	27	429	Obtained by multiplying the projected population growth by the unit rate.
Average Institutional Flow (m^3/d)	0	0	4	4	4	4	4	Institutional growth area multiplied by unit flow rate.
Average Commercial Flow (m^3/d)	0	0	0	0	0	0	0	Commercial growth area multiplied by unit flow rate.
Average Industrial Flow (m^3/d)	0	0	0	0	1,273	1,273	1,273	Industrial growth area multiplied by unit flow rate.
Average Extraneous Flow (m^3/day)	10	20	39	39	470	470	846	
Average Day Flow (m^3/d)	1,084	1,102	1,132	1,136	2,841	2,842	3,619	
Max Day Flow (m^3/d)	3,978	4,044	4,153	4,168	10,428	10,428	13,282	

ALTERNATIVE CALCULATION METHOD

Per Capita Flow (m3/cap/day)	0.586	0.398	0.441	0.418	0.557	0.480	0.480

	2016	2021	2026	2031	2036	2041	Ultimate Buildout	Comments
Average Day Flow (m^3/d)	1075.905	1084.800403	1092.861	1097.664	1100.492	1100.789	1582.864439	Multiplying the total population by the consolidated per capita flow factor.
Max Day Flow (m^3/d)	3,948	3,981	4,011	4,028	4,039	4,040	5,809	

SENSITIVITY ANALYSIS

Analyze sensitivity of forecasted flows to unit rate

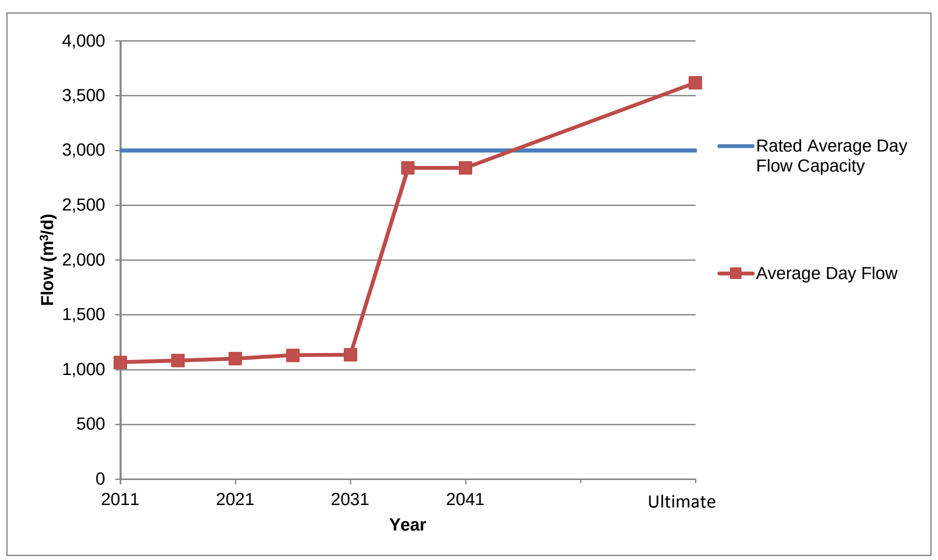
	Unit Rate (m^3/cap/d)	Average Day Flow (m^3/d) 2016	2021	2026	2031	2036	2041	Ultimate Buildout 2061
Base Scenario - Residential Historical Maximum	0.386	1,084	1,102	1,132	1,136	2,841	2,842	3,619
Combined Historical Maximum	0.480	1,085	1,105	1,136	1,141	2,847	2,847	3,705
City Standards	0.410	1,084	1,102	1,132	1,136	2,842	2,842	3,630

Analyze sensitivity of forecasted flows to max day factor

	Max Day Peaking Factor	Max Day Flow (m^3/d) 2016	2021	2026	2031	2036	2041	Ultimate Buildout 2061
Base Scenario - Historical Max	3.86	4,187	4,256	4,371	4,387	10,975	10,976	13,980
Historical Average	4.74	3,978	4,044	4,153	4,168	10,428	10,428	13,282

CAPACITY CHECK

	2011	2016	2021	2026	2031	2036	2041	Ultimate Buildout 2061
Rated WPCP ADF Capacity (m^3/d)	3,000	3,000	3,000	3,000	3,000	3,000	3,000	3,000
Average Day Flow (m^3/d)	1,068	1,084	1,102	1,132	1,136	2,841	2,842	3,619
Maximum Day Flow (m^3/d)	3,936	3,978	4,044	4,153	4,168	10,428	10,428	13,282



CITY OF GREATER SUDBURY WATER AND WASTEWATER MASTER PLAN

COPPER CLIFF WASTEWATER SYSTEM GAP ANALYSIS AND STATUS QUO REPORT

CITY OF GREATER SUDBURY

DRAFT

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

WSP
100 COMMERCE VALLEY DRIVE WEST
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TEL.: +1 905 882-1100
FAX: +1 905 882-0055
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TABLES

FIGURES

NO TABLE OF FIGURES ENTRIES FOUND.

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 existing Copper Cliff Wastewater System. Based on population growth projections and design criteria discussed in the *Population Projections and Development of Unit Rates Technical Memorandum* (WSP, 2014), 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 Copper Cliff Wastewater System would continue to be a stand-alone system. Any potential interconnections between Copper Cliff 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

Copper Cliff is a community located in the center of the City of Greater Sudbury, north-east of the communities of Lively and Walden. Mapping in Appendix A illustrates the Copper Cliff 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.

Based on the City's planning data, little growth is expected for Copper Cliff. The area population is expected to increase from 2,696 in 2011 to 2,736 in 2041 and 2,772 by Ultimate Buildout. ICI growth is expected to be primarily industrial with small amounts of commercial; no institutional growth is planned. Growth is discussed further in Section 6.1.

3 OVERVIEW OF EXISTING SYSTEM

The Copper Cliff Wastewater system is owned and operated both by a third party (Vale) and by the City of Greater Sudbury. The City owns and operates the collection system including sewers and two lift stations (Orford and Nickel), while Vale owns and operates the Copper Cliff Sewage Treatment Plant (STP). Vale also owns and operates a second plant, the Copper Cliff Wastewater Treatment Plant (WWTP), which does not treat municipal sewage.

There were no reported bypasses or spills at the lift stations in recent history. However, information on spills at the Copper Cliff STP are unknown given that the City is not the operating authority for this facility.

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

The Copper Cliff Wastewater System is illustrated in Appendix B.

3.1 LIFT STATIONS

Copper Cliff is serviced by two lift stations: Orford LS and Nickel LS. All wastewater flows generated in the community of Copper Cliff are ultimately conveyed to the Nickel LS and subsequently pumped to the Copper Cliff STP. The Lift Stations' forcemain sizes and other facility details are summarized in the Baseline Review Report for Wastewater Systems (WSP, 2014). Table 3-1 provides a summary of the main features of the lift stations.

Table 3-1 Copper Cliff System Lift Stations

LIFT STATION	YEAR CONSTRUCTED	LAST UPGRADED	WET WELL VOLUME TOTAL (M3)[1]	PUMPING STATION CAPACITY AND FORCEMAIN INFORMATION[2]
Orford	200	2008	N/A	Two submersible pumps with a firm capacity of 18.9 L/s 30 m long, 150 mm diameter forcemain of unknown material
Nickel	N/A	N/A	Approximately 270	Three dry well pumps with a firm design capacity of 100 L/s 597 m long, 400 mm diameter forcemain of unknown material

[1]Obtained or estimated from dimensions found in as-built and record, assuming water level does not exceed the High Water Alarm Level or, in absence of this alarm level, the inlet sewer invert.

[2] Obtained from the City's Wastewater Lift Stations Operations Manual and station as-built drawings.

3.2 WASTEWATER TREATMENT PLANT

The City has entered into an agreement with Vale Canada Ltd. (Vale) regarding the sewage treatment at their Copper Cliff STP. The agreement between the City and Vale extends to 2019. However, the Copper Cliff STP requires significant reinvestment and the City has indicated they intend to construct a new forcemain at the Nickel LS to convey wastewater flows generated in Copper Cliff to the Sudbury WWTP.

3.3 KNOWN CHALLENGES

In addition to concerns discussed in previous sections, the Copper Cliff Wastewater System has the following known challenges:

— I&I is a major concern in the Copper Cliff wastewater system as the system in this community experiences high levels of I&I. The City has noted lots of I&I in the system near the private development located at Power Street and Highway 55. Smoke testing has been undertaken in this area. The parking lot in this development is noted to be often flooded.

— There are several contributors to the high levels of I&I in the system. Manholes throughout the system have been constructed at low elevations. Moreover, many sewers are known to have tree roots growing through them which increase the potential for more infiltration into the system. The subdivision in the north end of the community, adjacent to Godfrey Drive, is an example of an area in which significantly sized tree roots are reported to have grown through the sanitary infrastructure.

— Access to infrastructure is another issue within the Copper Cliff wastewater network. Some sanitary sewers are known to be aligned under residential homes. The 'Little Italy' area is an area in which access to underground infrastructure is particularly challenging. The linear infrastructure is not typically aligned within road rights of way nor within easements. Moreover, in this area, land ownership is generally unclear which also makes accessing infrastructure more challenging.

— The tree roots that are growing through sanitary mains have been noted to impact wastewater flows in the network.

4 HISTORICAL FLOWS AND REVIEW OF OPERATIONAL DATA

Since the Copper Cliff STP is not owned by the City, there is no data for historical flows and operational data.

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 wastewater systems. The unit rates used to estimate future 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 WASTEWATER DESIGN CRITERIA

The unit flow criteria for growth adopted for this assessment are shown in Table 5-1. These values were recommended in the *Populations and Unit Rates Technical Memorandum* (WSP, 2014).

The term "extraneous flows" is used interchangeably with "I&I flows".

Table 5-1 Wastewater System Design Criteria

CRITERIA	VALUE	REFERENCE
Average Day Residential Flow	500 L/cap/day	City's Engineering Design Manual
Average Day Commercial and Institutional Flow	28 m^3/ha/d	Water unit rates, assuming a 1:1 correspondence
Average Day Industrial Flow	35 m^3/ha/d	Water unit rates, assuming a 1:1 correspondence
Average Extraneous Flow	11.2 m^3/ha/d	Peak from City's Engineering Design Manual and assuming a peaking factor of three
Peak Extraneous Flow	33.7 m^3/ha/d	City's Engineering Design Manual
Max Day Peaking Factor	4.05	Estimated by using the same factor as Lively

Historical data was not available for Copper Cliff and the City's current standard of 500 L/cap/day was used instead. The peak flows were estimated by assuming the same factor as used for the Lively wastewater system.

5.2 DESIGN CRITERIA FOR WASTEWATER SYSTEM COMPONENTS AND OPERATION

5.2.1 WASTEWATER TREATMENT

Wastewater treatment facilities are rated for average day flows. Plant effluent limits and objectives are established in the C of A or ECA for each facility.

5.2.2 LIFT STATION PUMPING CAPACITY

The firm capacity of the lift station (with the largest pump out of service) must allow pumping of peak wet weather flows corresponding to its catchment area (MOECC, 2008).

Starting limitations on pump motors generally dictate the minimum size of a wet well. The wet well should be large enough to prevent pump motors from overheating due to frequent starting and stopping, but small enough to avoid long retention times leading to septicity and odor problems (Lin & Lee, 2001).

The station wet well shall be sized such that the number of pump starts per hour does not exceed the maximum value recommended by the pump manufacturer. In other words, the time between pump starts and stops (i.e. the pump cycle time) should not exceed that which results in a pump start frequency greater than that recommended by the pump manufacturer. Typically, submersible pumps can cycle four to 10 times per hour with a maximum cycle time not exceeding 30 minutes (Lin & Lee, 2001). A maximum value of four pump starts per hour was assumed to evaluate wet well sizing requirements.

5.2.3 SEWERS

The sewer system is typically sized to convey peak instantaneous (peak wet weather) flows. Sewage flows are made up of wastewater discharges from residential, commercial, institutional and industrial establishments, plus extraneous flow components from such sources as groundwater and surface runoff.

In addition to being able to convey peak flows, sufficient flow velocity should be maintained to transport the sewage solids to avoid deposition and the development of nuisance conditions under lower flow conditions. The minimum acceptable flow velocity in sewers is 0.6 m/s (City of Greater Sudbury, 2012).

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 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 Projections and Development of 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 Copper Cliff is projected to increase by 39 people in 2041 and 76 by Ultimate Buildout. The population projections to be used in the Master Plan are summarized in Table 6-1.

Table 6-1 Copper Cliff Population Projections

SYSTEM	2011	2016	2021	2026	2031	2036	2041	ULTIMATE BUILDOUT
Copper Cliff	2,696	2,703	2,713	2,724	2,729	2,737	2,736	2,772

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 below.

Table 6-2 Copper Cliff ICI Projections

	ICI DEVELOPMENT AREAS (HA)						
LAND USE	2016	2021	2026	2031	2036	2041	BUILDOUT
Institutional	0	0	0	0	0	0	0
Commercial	0	0	0.16	0	0	0	0
Industrial	0	0	0	0	15.74	0	0
Total	**0**	**0**	**0.16**	**0**	**15.74**	**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 PHASING OF FUTURE GROWTH

Growth areas were allocated based on population projections for individual developments and the overall target growth population projections for Copper Cliff 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 WASTEWATER FLOW PROJECTIONS AND INFRASTRUCTURE NEEDS

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

$$\begin{aligned}&\textbf{Projected Average Day Generation}\\&\quad = \textbf{Base Generation} + \textbf{Additional Residential Generation} + \textbf{Additional ICI Generation}\\&\quad + \textbf{Average Extraneous Flow}\end{aligned}$$

The flows corresponding to the population growth forecasts to Ultimate Buildout are presented in Table 6-3.

Table 6-3 Flow Projections

YEAR	POPULATION	AVERAGE DAY FLOW (M^3/D)	MAXIMUM DAY FLOW (M^3/D)
Base	2,696	1,348	5,460
2016	2,703	1,358	5,501
2021	2,713	1,368	5,542
2026	2,724	1,380	5,588
2031	2,729	1,383	5,600
2036	2,737	2,114	8,563
2041	2,736	2,114	8,560
Ultimate	2,772	2,132	8,633

The Base Demand and 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 demand was calculated by applying the respective peaking factor to the average day demand.

A desktop analysis of historical wastewater flows and future flow projections is included in Appendix C.

6.3.1 COPPER CLIFF STP CAPACITY

The rated average day capacity of the Copper Cliff STP is 231,360 m^3/d. Since the City does not have a record of Vale's flows into the system, we have little insight into the projected capacity limitations at the Copper Cliff STP. Knowledge of Vale's contribution to the STP is important since Vale has a big operation in this area and currently uses the majority the plant's existing capacity. Vale has indicated that the STP is currently approaching its capacity and requires a significant upgrade, which the City would be required to cost-share. The plant also requires condition-based repairs. Hence, the City plans to implement a new forcemain at the Nickel LS to convey flows directly to the Sudbury WWTP.

6.3.2 SEWER NETWORK AND LIFT STATIONS

For each of the scenarios modeled, the system was checked for surcharging of sewers and capacity exceedance at the lift stations. Please note that Nickel LS was not analyzed as it has already been determined that the Nickel LS will be used to pump wastewater to the Sudbury Wastewater Treatment Plant instead of the existing Copper Cliff Plant. The peak flows into the Orford lift station was determined from the computer simulations of the various planning scenarios and are documented in Table 6-4. The table also documents the current pump design rated flow, its capacity based on drawdown tests and the computer simulated flow.

Table 6-4 Copper Cliff System Lift Station Peak Influent Flow Rates

	CURRENT FIRM CAPACITY	EXISTING PEAK FLOW	2041 PEAK FLOW	ULTIMATE BUILDOUT
Orford	18.9	12.4	17.5	17.5

Based on the above table, the peak inflow to the Orford LS does not exceed capacity in 2011; however, the 2041 and Ultimate peak inflows are over 90% of the firm capacity of the LS.

7 HYDRAULIC MODELLING

7.1 APPROACH

A basic sanitary model for the City of Greater Sudbury was received from the City. The model was created in Bentley's SewerGEMS by City staff. The model is an all pipe model of the sanitary network in the system, but critical information such as pipe data, invert elevations and pumping station characteristics were missing. The model now includes key vertical infrastructure in each system, including lift stations and treatment facilities.

The I&I rates for both dry and wet weather flow were developed through flow monitoring and the average value was used to load the model.

Current (2011) and future (2016-Ultimate Buildout, in 5 year increments) population data was added to the model using the City's planning data, summarized in previous sections of this report.

Future dry and wet weather flow scenarios were developed for each of the horizon years: 2016, 2021, 2026, 2031, 2036, 2041, and Ultimate Buildout. However, model results did not vary significantly from 2016 to 2041; therefore, this report discusses findings for 2041 and Ultimate Buildout, compared against existing (2011).

7.2 MODELLING FINDINGS

The model was used to check sewer capacity and flow velocity. Maps of the model findings are presented in Appendix B.

The majority of sewers flow at less than 50% of the available capacity from 2011 to Ultimate Buildout under the wet weather flow conditions. There are a few sewers however, that are flowing at over 100% capacity under the 2041 wet weather flow (2 year storm) condition, as shown on the summary maps.

8 CONCLUSIONS

An assessment of the Copper Cliff Wastewater Systems was completed to identify infrastructure requirements to service forecasted growth in the community.

The conclusions of the assessment are summarized below.

- The Copper Cliff STP is owned and operated by Vale and requires significant reinvestment.
- To limit / reduce the risk associated with long term wastewater treatment in Copper Cliff the City is planning to construct a new forcemain from the Nickel LS to the Sudbury WWTP. This will require significant reinvestment to Nickel LS.
- The peak 2011 to Ultimate inflows to the Nickel LS exceed the capacity. Peak inflow to the Orford LS does not exceed capacity in 2011; however, the 2041 and Ultimate peak inflows are over 90% of the firm capacity of the LS.
- Many of the sewers in Copper Cliff operate at velocities that do not meet the City's current standards (that is, many sewers flow at less than 0.6 L/s). This may cause operational problems, such as solids buildup and odours.
- The majority of sewers in Copper Cliff are flowing at less than 50% full.
- There are a few sewers that are flowing at 100% of their design capacity however none of these sewers are overflowing to ground
- There are a number of sewers which are not located in a road easement. When resources are available the City should attempt to attain easements for the existing sewers. Furthermore, whenever possible sewers should be relocated to public property and/or easements attained.
- There is a bottleneck in the sewer upstream of the Nickel LS.

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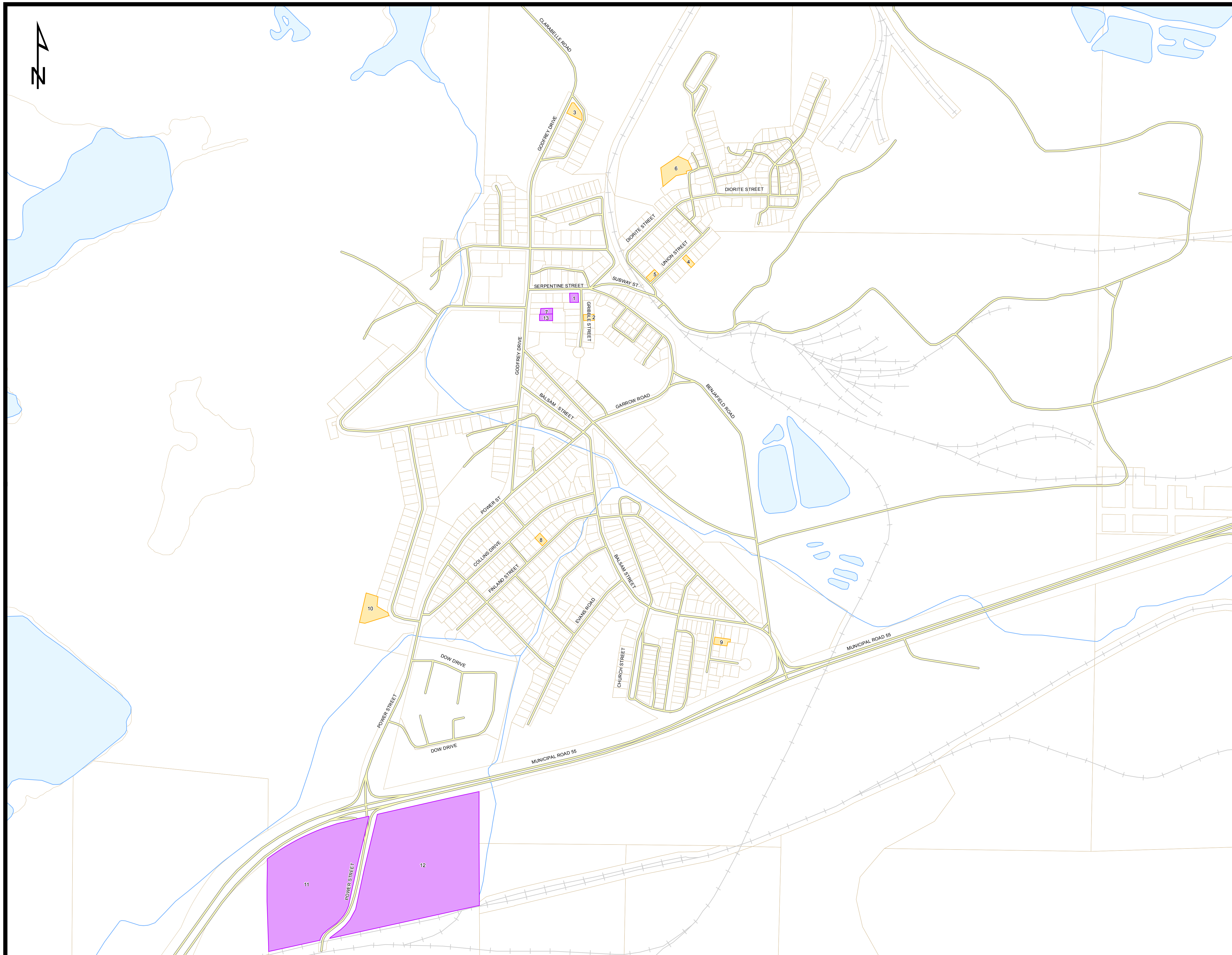
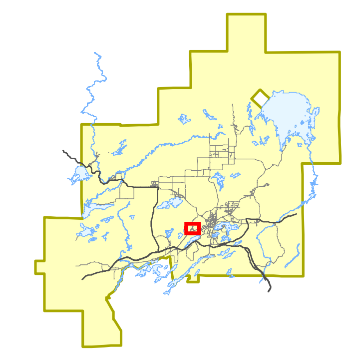
APPENDIX

A RESIDENTIAL AND ICI DEVELOPMENT AREAS



Legend

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



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Sudbury Water and Wastewater Master Plan

0 100 200 300 400 500 m

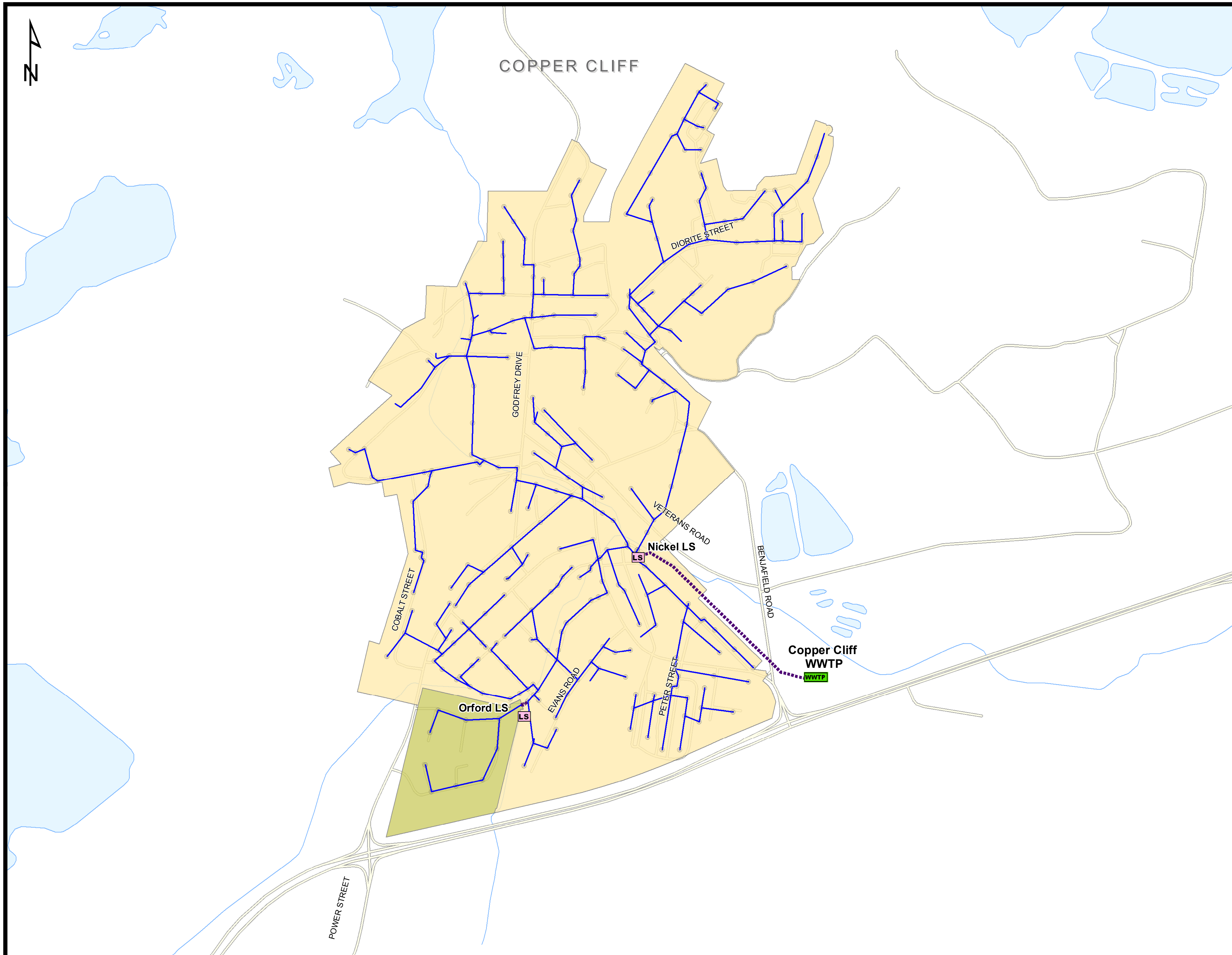
1:10,000

Map 6

Copper Cliff
2011 - Ultimate Buildout Residential and ICI Development Areas

APPENDIX

B WASTEWATER MODEL RESULTS



Legend

- LS Lift Station
- WWTP Sanitary Treatment Plant
- Manhole
- Waterbody
- Road
- Watercourse
- Sanitary Sewer Conduit
- Forcemain

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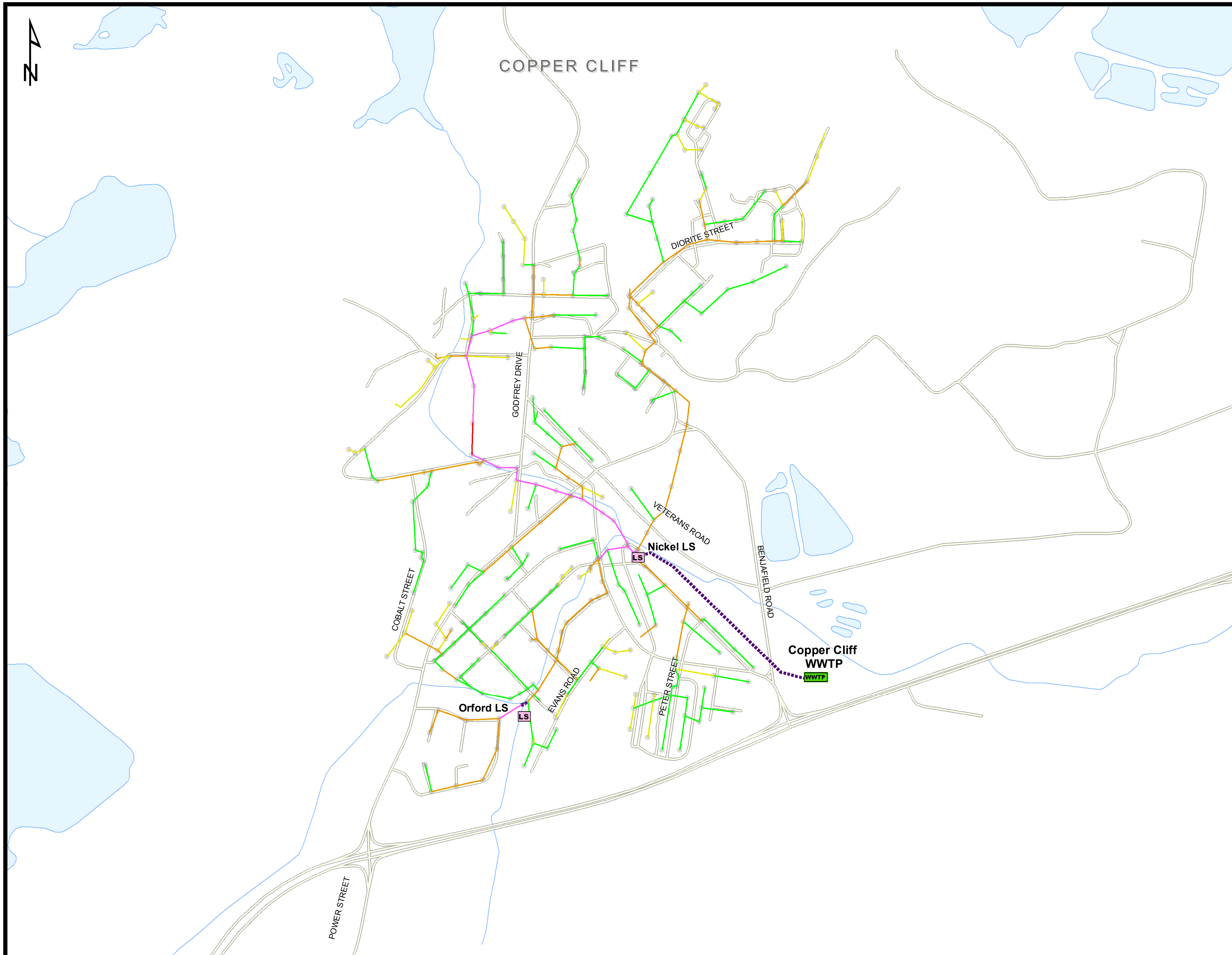
Map 1

Copper Cliff Wastewater System - Sewershed Map



Legend

- LS Lift Station
- WWTP Sanitary Treatment Plant
- Manhole
- Waterbody
- Road
- Watercourse
- Forcemain

Conduit: Diameter (mm)

- <= 150
- <= 200
- <= 300
- <= 450
- <= 600
- <= 750
- > 750

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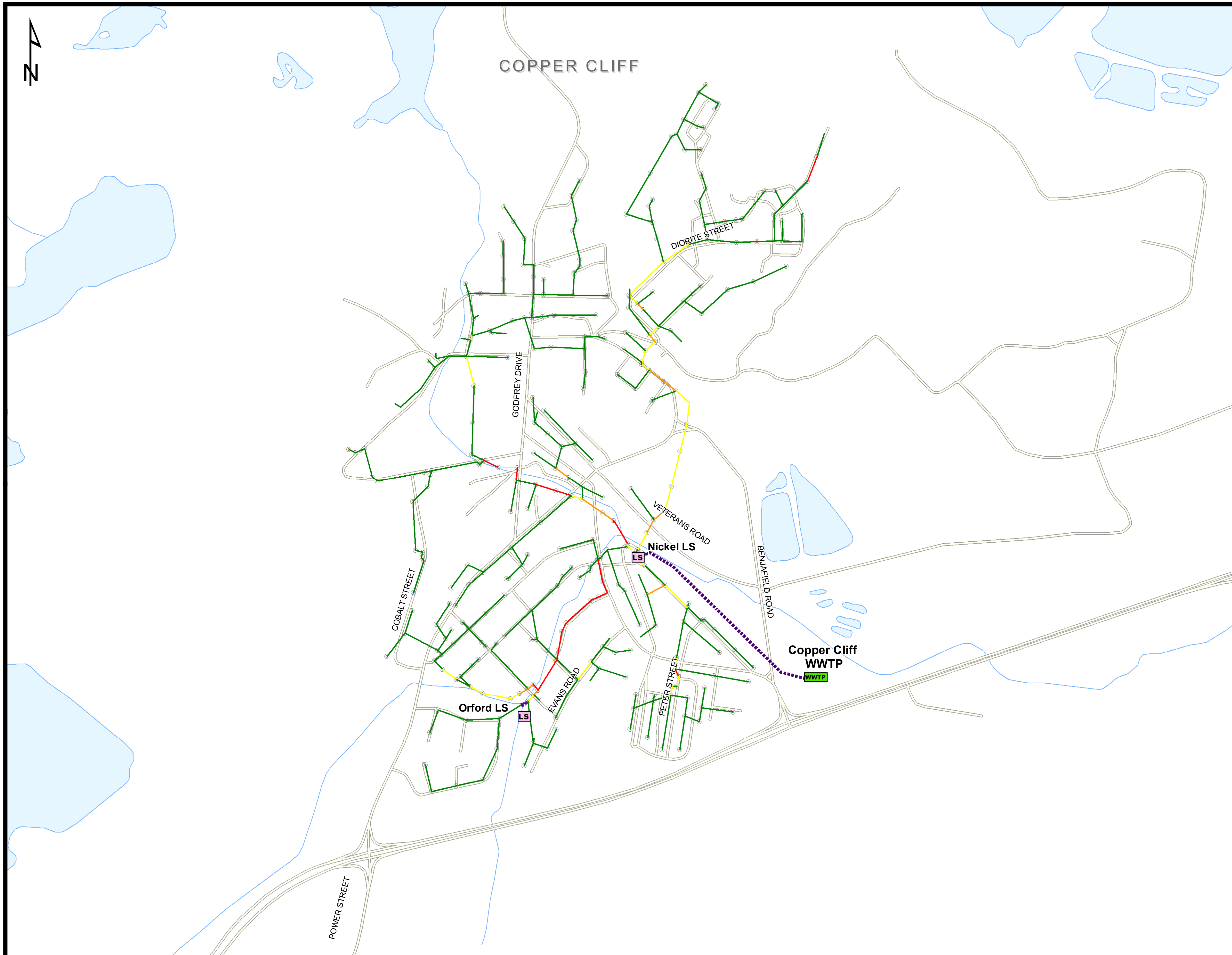
Map 2

Copper Cliff Wastewater System - Conduit Diameter



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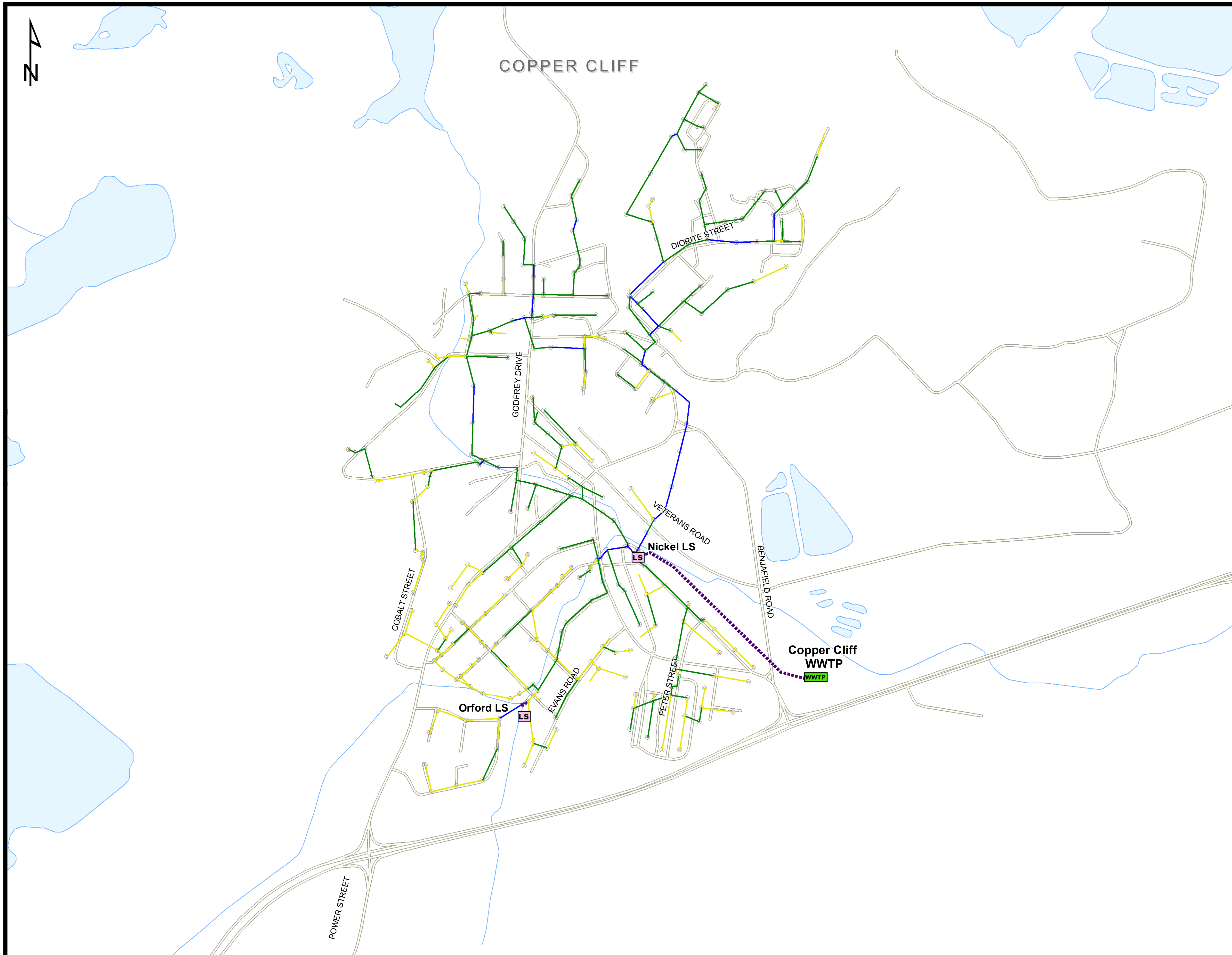
October 04 2017

Sudbury Water and Wastewater Master Plan

Map 3.1

Copper Cliff Wastewater System - Conduit Flow/Capacity (Design)

Scenario: 2011WWF_2yrStorm



Legend

- LS Lift Station
- WWTP Sanitary Treatment Plant
- Manhole
- Waterbody
- Road
- Watercourse
- Forcemain

Conduit: Velocity (m/s)

- <= 0.6
- <= 1.5
- <= 3.0
- > 3.0

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Sudbury Water and Wastewater Master Plan

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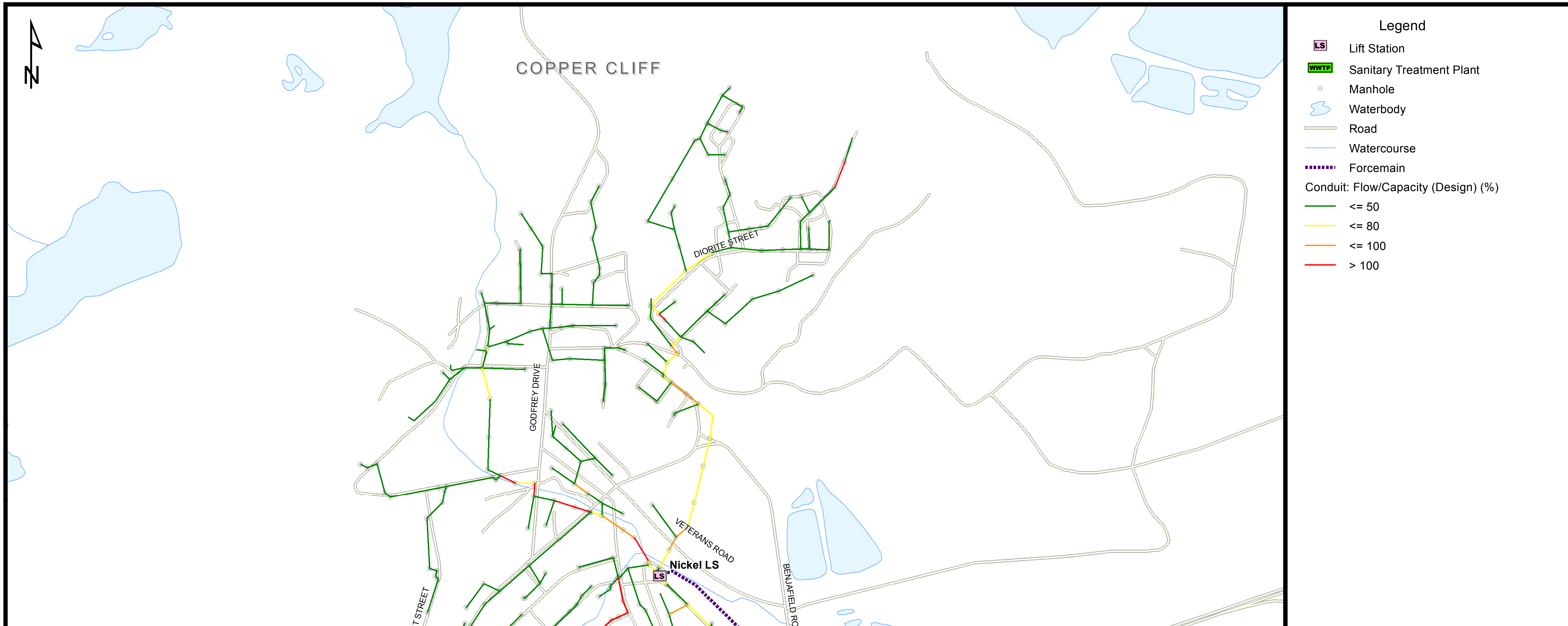
Map 3.2

Copper Cliff Wastewater System - Conduit Velocity

Scenario: 2011WWF_2yrStorm



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WSP

Project No. 121-23026-00

December 09 2016

Sudbury Water and Wastewater Master Plan

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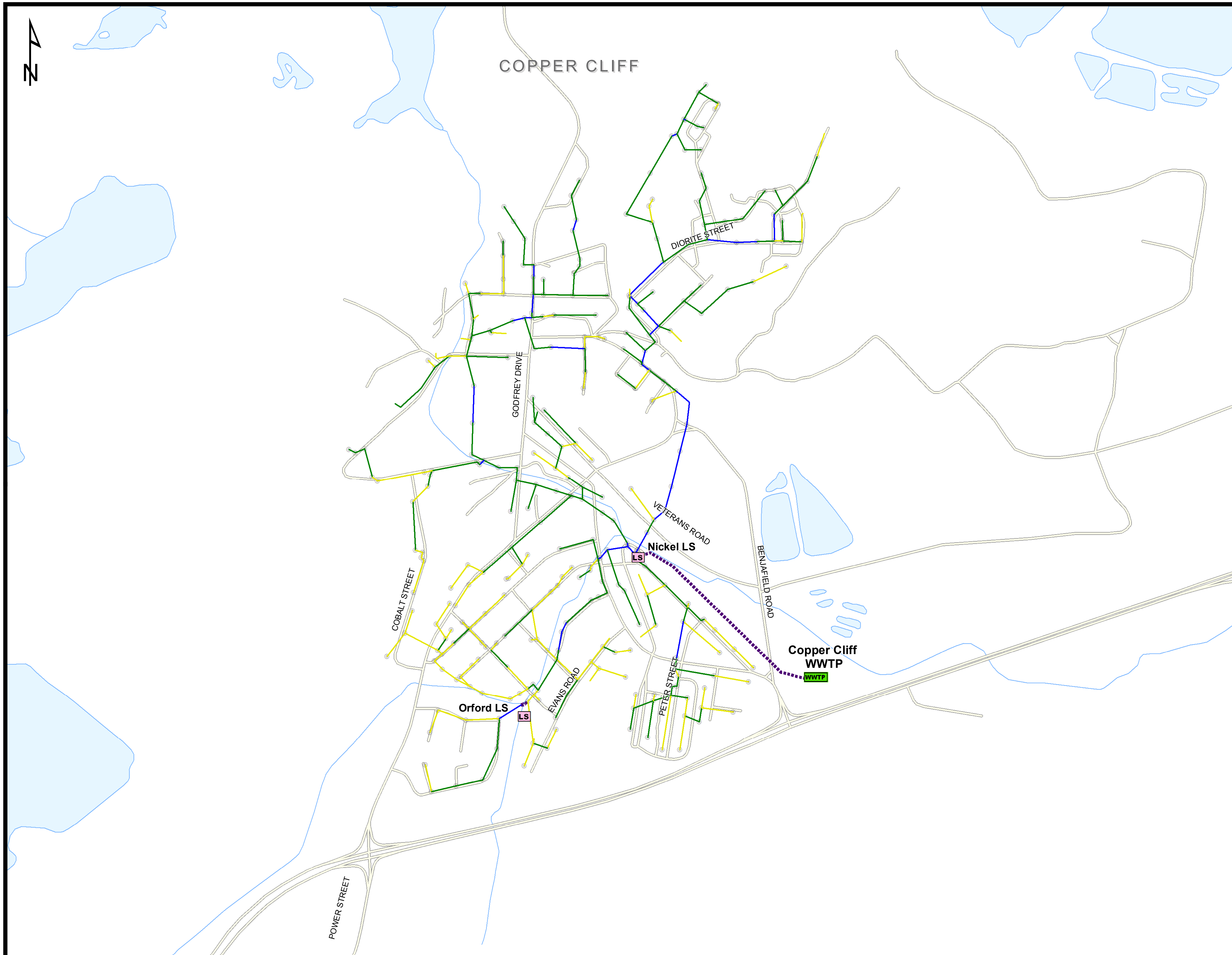
1:10,000

Map 4.1

Copper Cliff Wastewater System - Conduit Flow/Capacity (Design)

Scenario: 2041WWF_2yrStorm



Legend

- Lift Station
- Sanitary Treatment Plant
- Manhole
- Waterbody
- Road
- Watercourse
- Forcemain

Conduit: Velocity (m/s)

- <= 0.6
- <= 1.5
- <= 3.0
- > 3.0

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Sudbury Water and Wastewater Master Plan

0 200 400 600 m

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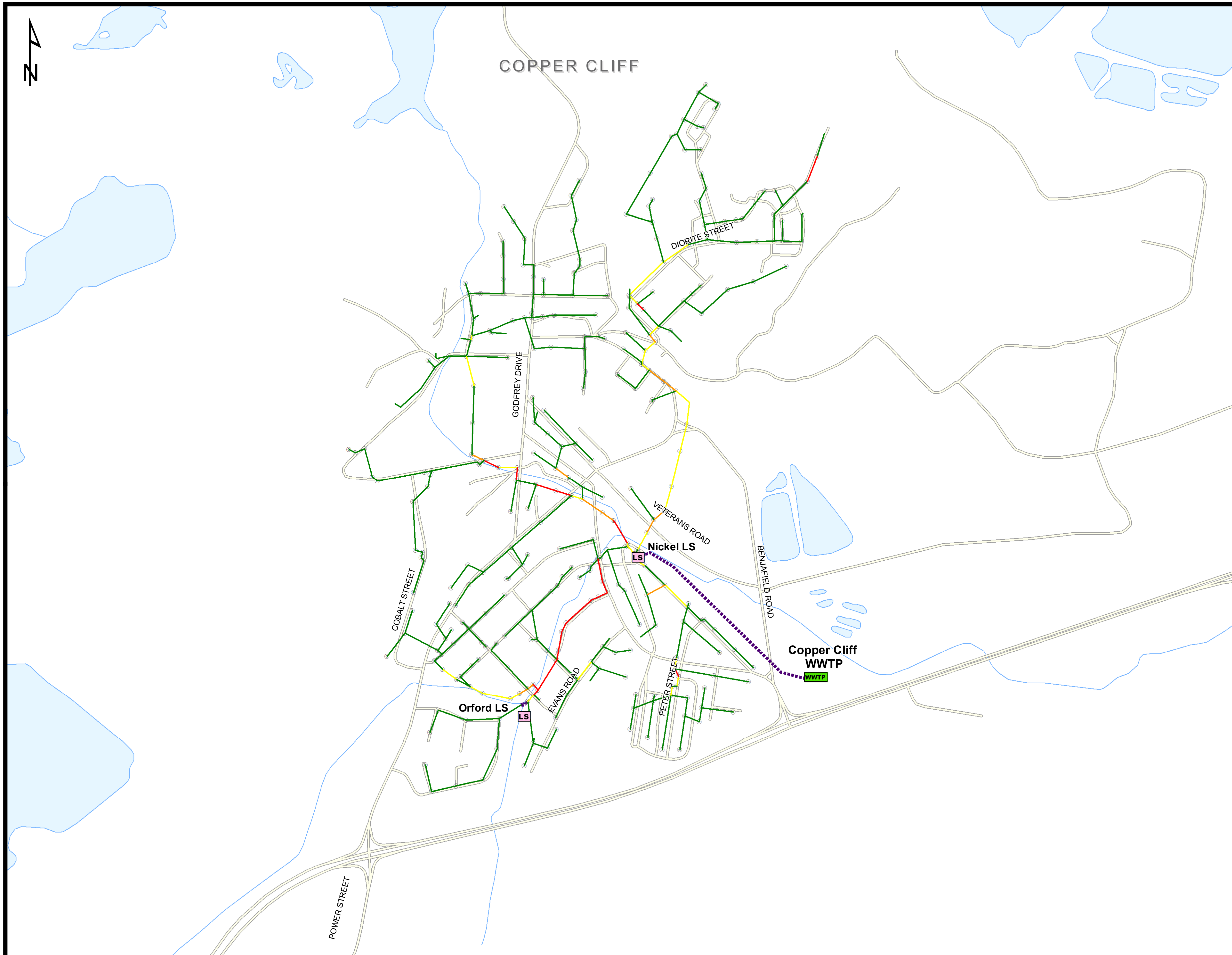
Map 4.2

Copper Cliff Wastewater System - Conduit Velocity

Scenario: 2041WWF_2yrStorm



Legend

- LS Lift Station
- WWTP Sanitary Treatment Plant
- Manhole
- Waterbody
- Road
- Watercourse
- Forcemain

Conduit: Flow/Capacity (Design) (%)

- <= 50
- <= 80
- <= 100
- > 100

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0 200 400 600 m

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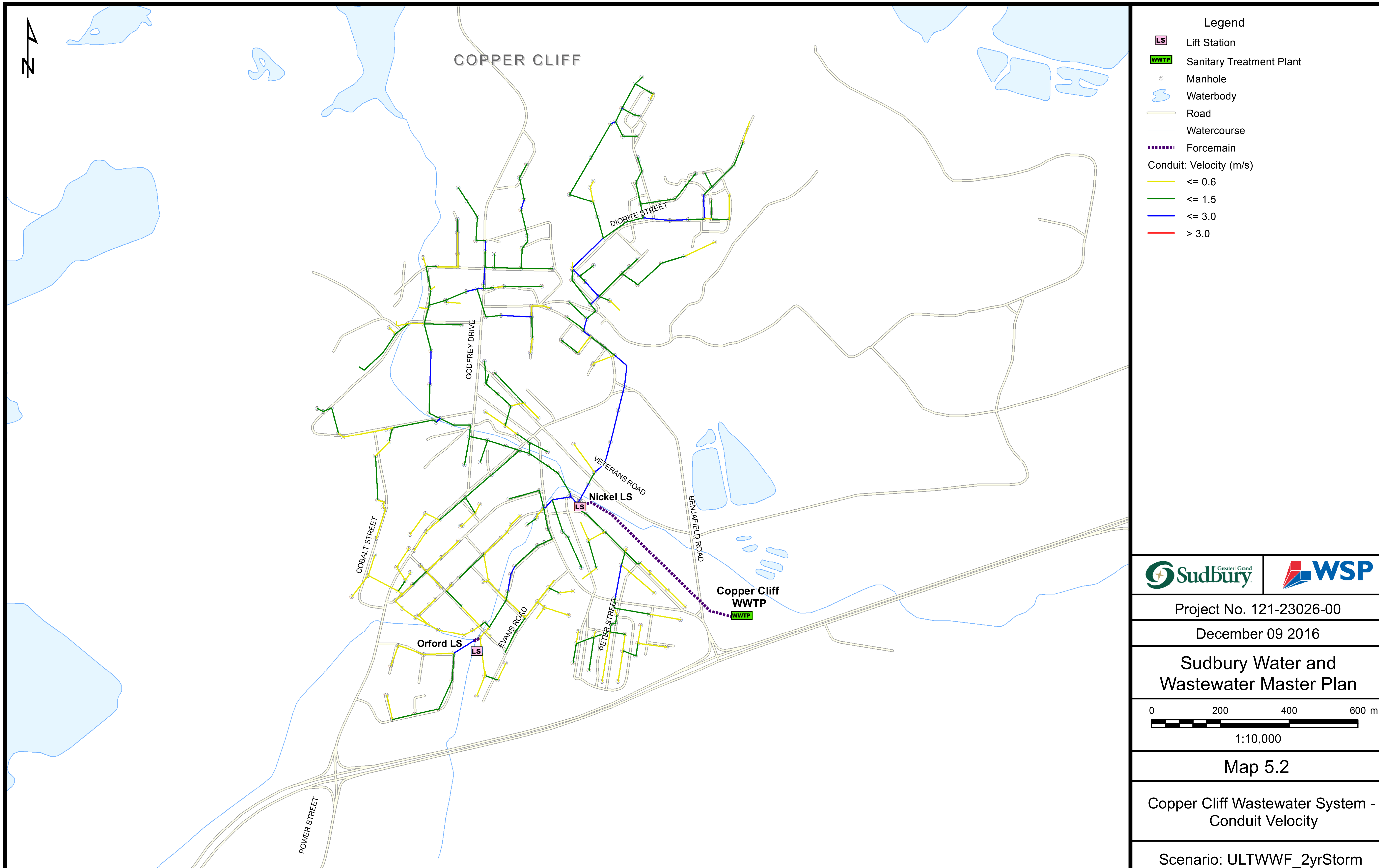
Map 5.1

Copper Cliff Wastewater System - Conduit Flow/Capacity (Design)

Scenario: ULTWWF_2yrStorm



Project No. 121-23026-00

December 09 2016

Sudbury Water and Wastewater Master Plan

Map 5.2

Copper Cliff Wastewater System - Conduit Velocity

Scenario: ULTWWF_2yrStorm

APPENDIX

C WASTEWATER GENERATION AND CAPACITY ASSESSMENTS

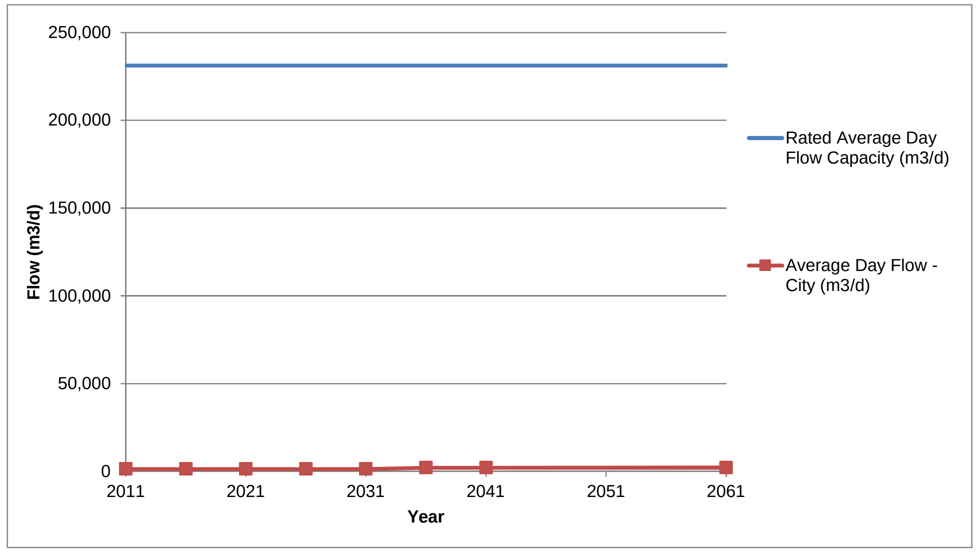
Copper Cliff - Wastewater Flow Forecasts

		2009	2010	2011	2012	2013	Summary	Design Criterion	Comments
Average Day Flow	(m^3/d)	Not Available						1,348	Historical Flow data was not available. The existing design flow was estimated using the existing population and a design unit flow rate.
Max Day Flow	(m^3/d)							5,460	Historical Flow data was not available. The max day flow was estimated by multiplying the average day flow by the max day factor.
Max Day Factor								4.05	Historical Flow data was not available. Value for nearby Lively was used.
Peak Flow	(m^3/d)							0	Historical Flow data was not available.
Peak Flow	(L/s)							0	
Population (Existing Areas)		2,696	2,696	2,696	2,696	2,696	2,696	2,696	
Ratio of Residential to Total Customers		0.84	0.84	0.84	0.84	0.84	0.84		This ratio is based on Water Billing Records for the area and is an approximation of the residential portion of demand.
Residential Share of Average Day Demand (m^3/d)		Not Available							
Residential Flow Unit Rate (m^3/cap/d)								0.500	
Average Institutional Flow Unit Rate (m^3/ha/d)								28.0	CGS design standards do not suggest average rates for institutional and commercial, but require that rates be developed based on usage. In this case, water demands rates were used and assume a conservative 1:1 ratio of water to sewage.
Average Commercial Flow Unit Rate (m^3/ha/d)								28.0	CGS design standards do not suggest average rates for institutional and commercial, but require that rates be developed based on usage. In this case, water demands rates were used and assume a conservative 1:1 ratio of water to sewage.
Average Industrial Flow Unit Rate (m^3/ha/d)								35.0	CGS design standards average rate for light industrial of 35 m^3/ha/d.
Average Extraneous Flow Unit Rate (m^3/ha/d)								11.23	From CGS Design Standards, peak rate for new developments divided by an assumed peaking factor of 3. This factor would be applied only to new developments, which are assumed to be leak-tight, and have minimal extraneous flow.

	2016	2021	2026	2031	2036	2041	Ultimate Buildout	Comments
Population (Existing Areas)	2,696	2,696	2,696	2,696	2,696	2,696	2,696	
Population (Growth Areas)	7	17	27	33	41	39	76	
Total Population	2703	2713	2724	2729	2737	2736	2772	Total Population (Hemson)
Residential (ha)	0.59	1.05	1.05	1.05	1.05	1.05	1.05	ICI development areas were assigned to planning years based on the stage of the application. Draft Approved were assigned to 2016, Legal Lots of Record to 2026, and Designated Developable to 2036.
Institutional (ha)								
Commercial (ha)			0.16	0.16	0.16	0.16	0.16	
Industrial (ha)					15.74	15.74	15.74	Areas are cumulative and carry from the development year, all the way through to Ultimate Buildout
ICI (ha)	0.00	0.00	0.16	0.16	15.90	15.90	15.90	
Total (ha)	0.59	1.05	1.21	1.21	16.95	16.95	16.95	

	2016	2021	2026	2031	2036	2041	Ultimate Buildout	Comments
Average Residential Flows (m^3/d) - Existing	1,348	1,348	1,348	1,348	1,348	1,348	1,348	This includes all contribution from existing ICI and infiltration. The base flow was assumed to be the average day flow to the plant for the 2011-2013 period.
Average Residential Flows (m^3/d)	4	8	14	17	20	20	38	Obtained by multiplying the projected population growth by the unit rate.
Average Institutional Flow (m^3/d)	0	0	0	0	0	0	0	Institutional growth area multiplied by unit flow rate.
Average Commercial Flow (m^3/d)	0	0	4	4	4	4	4	Commercial growth area multiplied by unit flow rate.
Average Industrial Flow (m^3/d)	0	0	0	0	551	551	551	Industrial growth area multiplied by unit flow rate.
Average Extraneous Flow (m^3/day)	7	12	14	14	190	190	190	
Average Day Flow (m^3/d)	1,358	1,368	1,380	1,383	2,114	2,114	2,132	
Max Day Flow (m^3/d)	5,501	5,542	5,588	5,600	8,563	8,560	8,633	

CAPACITY CHECK								Ultimate Buildout
	2011	2016	2021	2026	2031	2036	2041	2061
Rated Average Day Flow Capacity (m^3/d)	231,360	231,360	231,360	231,360	231,360	231,360	231,360	231,360
Average Day Flow - City (m^3/d)	1,348	1,358	1,368	1,380	1,383	2,114	2,114	2,132
Maximum Day Flow - City (m^3/d)	5,460	5,501	5,542	5,588	5,600	8,563	8,560	8,633



CITY OF GREATER SUDBURY WATER AND WASTEWATER MASTER PLAN

DOWLING WASTEWATER SYSTEM GAP ANALYSIS AND STATUS QUO REPORT

CITY OF GREATER SUDBURY

DRAFT

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

WSP
100 COMMERCE VALLEY DRIVE WEST
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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 existing Dowling Wastewater System. Based on population growth projections and design criteria discussed in the *Population Projections and Development of Unit Rates Technical Memorandum* (WSP, 2014), 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 Dowling Wastewater 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. This report assumes that the Dowling Wastewater System will continue to be a stand-alone system. Additional information on the existing wastewater system is provided in the *Baseline Review Report for Wastewater Systems (WSP, 2014).*

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

Dowling is located in the northwest end of Greater Sudbury along Route 144, between the communities of Onaping and Chelmsford. Map 1 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 wastewater collection system in Dowling consists of approximately 15.75 km of sewers and has a single lift station (LS), the Lionel LS. The Lionel LS services all properties on and west of Lionel Avenue, as well as those on Riverside Drive between the lift station and manhole 7-19 and Rheume Street between the lift station and manhole 7-34.

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

The Dowling Wastewater System is shown in **Appendix B**.

3.1 LIFT STATIONS

Table 3-1 below provides a summary of the main features of the lift station.

Table 3-1 Dowling Sewage Lift Stations

LIFT STATION	YEAR CONSTRUCTED	LAST UPGRADED	WET WELL VOLUME[1]	PUMPING STATION CAPACITY
Lionel LS	1979	No upgrades, other than pump replacement (year unknown)	10.9 m^3	2 submersible pumps rated at 18.6 L/s and 21.7 L/s for a total rated capacity of 26.5 L/s. 150 mm diameter forcemain.

[1] Calculated from dimensions found in 1977 Record Drawings prepared by Northland Engineering Ltd., assuming water level does not exceed the High Water Alarm Level.

3.2 DOWLING WWTP

The Dowling Wastewater Treatment Plant (WWTP) is owned and operated by the City of Greater Sudbury and is located approximately 240 m north of the intersection of Riverside Drive and Houle Street. The WWTP is an extended aeration activated sludge facility with an average day rated capacity of 3,200 m^3/d and peak hour capacity of 6,400 m^3/d (MOECC, 1998). The treatment process is illustrated schematically below.

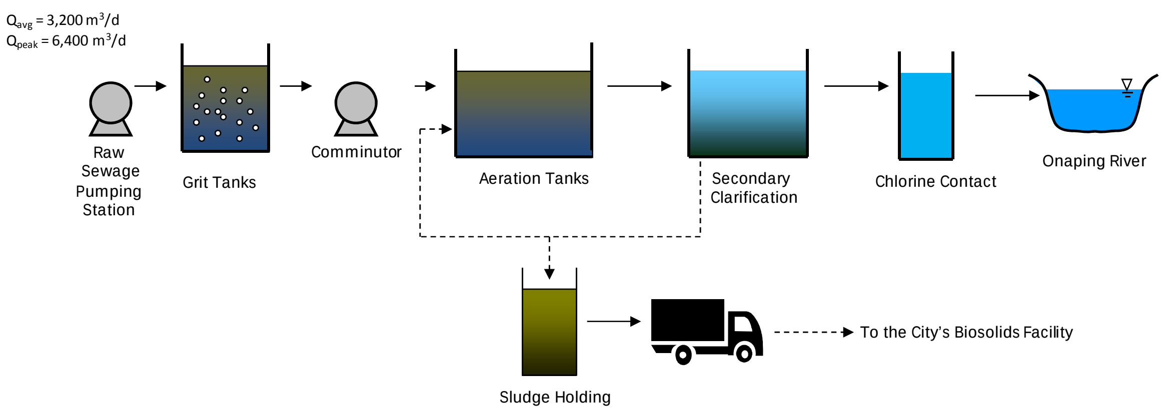

Figure 3-1 Dowling WWTP Process

4 HISTORICAL FLOWS AND REVIEW OF OPERATIONAL DATA

Data reported in the 2009 to 2013 *Annual Reports* for the Dowling WWTP was reviewed and analyzed to determine average day and maximum day flows as well as review effluent parameters.

4.1 FLOW DATA

WWTP flow data from 2009 to 2013 was reviewed. Operational data was not available from the Lionel LS in Dowling and so historical peak flow data could not be estimated.

The recorded average day and maximum day flows are summarized in Table 4-1 and plotted in Figure 4-1 below.

Table 4-1 Historical Wastewater Flow Data

YEAR	AVERAGE DAY FLOW (M^3/D)[1]	MAXIMUM DAY FLOW (M^3/D)[1]
2009	2,033	4,000
2010	1,749	3,180
2011	1,846	2,940
2012	1,837	2,650
2013	2,051	4,338

[1] *Annual Reports (2009 - 2013)*

The relationship between the different flow regimes was analyzed to compare the peaking factors derived from historical data to those used in the *City's Engineering Design Manual* and those included in the *MOECC Guidelines*.

The average day flows to the WWTP have been generally consistent over the 2009 to 2013 period, averaging 1,903 m^3/d. However, the variations in historical maximum day flows show no discernible trend. The greatest maximum day flow occurred in 2013.

A comparison of drinking water production to wastewater flow volumes treated for the period between 2009 to 2013 reveals that on average the flows received at the WWTP are approximately five times greater than the average water flows produced at the wells. This can be explained by substantial inflow and infiltration (I&I) into the sanitary system.

Historically, there have not been any instances in which the rated capacity of the Dowling WWTP has been exceeded, per the *Annual Reports*. Furthermore, no overflows or bypasses have been reported in Dowling over the past five year period. A sludge spill was reported in 2011, but the cause was not identified.

The highest maximum day to average day peaking factor based on the maximum day flow recorded in 2013 was 2.12. The average maximum day peaking factor from 2009 to 2013 was 1.79. The City's *Engineering Design Manual* and the *MOECC Guidelines* do not specify recommended maximum day factors and recommend using historical data when available. For future generation, the average peaking factor was used and based on the assumption that new developments would have less I&I due to more leak tight construction.

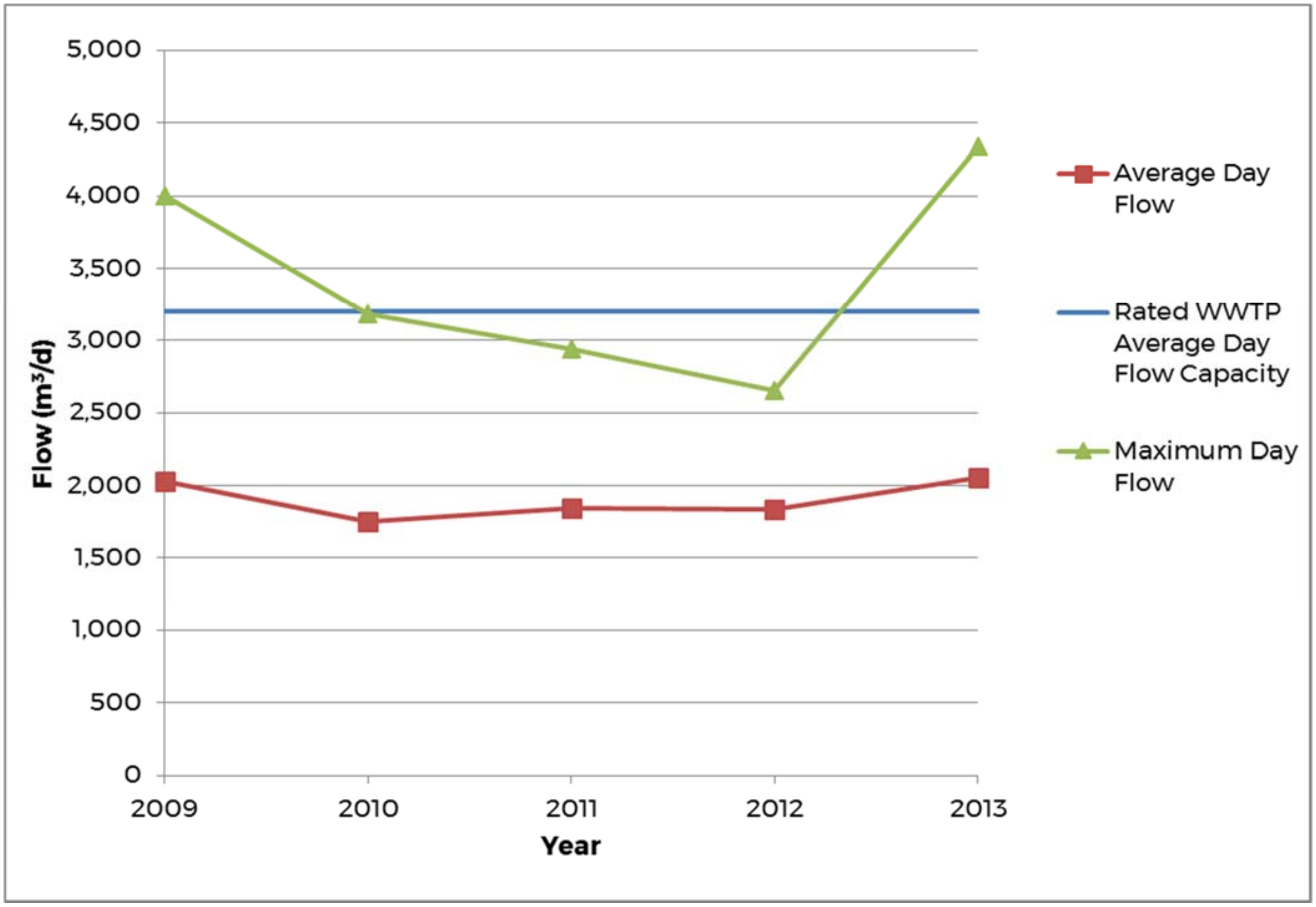

Figure 4-1 Historical Wastewater Flows at the Dowling Wastewater Treatment Plant

4.2 RAW WASTEWATER CHARACTERISTICS

The average raw wastewater characteristics from 2009 to 2012 are summarized in the table below. Raw wastewater temperatures were not reported and raw wastewater total Kjeldahl nitrogen (TKN) was not reported in 2011.

Table 4-2 Average Raw Wastewater Characteristics at the Dowling WWTP (2009-2012)

PARAMETER	AVERAGE VALUE
$CBOD_5$	42 mg/L
Suspended Solids	46 mg/L
Total Phosphorus	1.7 mg/L
TKN	7.3 mg/L
pH	6.7

Wastewater flows to the Dowling WWTP correspond mainly to residential uses, with minor contributions from commercial and industrial users, and dilution from inflow and infiltration.

4.3 EFFLUENT CRITERIA

The Dowling WWTP is operated in accordance with *MOECC Certificate of Approval for Sewage No. 3-0897-98-006* dated November 6, 1998.

The *Certificate of Approval* (*C of A*) concentration and loading limits are summarized in the table below.

Table 4-3 Dowling WWTP Effluent Limits and Objectives

EFFLUENT PARAMETER	CONCENTRATION LIMIT	LOADING LIMIT	CONCENTRATION OBJECTIVE
$CBOD_5$	25 mg/L	80 kg/d	15 mg/L / 48 kg/d
Total Suspended Solids (TSS)	25 mg/L	80 kg/d	15 mg/L / 48 kg/d
Total Phosphorus (TP)	1.0 mg/L	3.2 kg/d[1]	0.5 mg/L / 1.6 kg/d
pH	N/A	N/A	6.0 to 9.5
E. coli	200 organisms/100 mL (Monthly Geometric Mean Density), with disinfection from May 1 to October 31	N/A	N/A

[1] Loading limit to comply with Provincial Water Quality Objective (PWQO) for Total Phosphorus in the Policy 2 ultimate receiver (i.e. Vermilion Lake).

Compliance with the concentration and loading limits for $CBOD_5$ and TSS is based on the annual average concentration of each parameter based on all composite samples during any calendar year, whereas compliance for the TP is based on the monthly average concentration.

4.4 OPERATIONAL DATA

The general operation of the WWTP was reviewed against the C of A requirements and historical data provided in the Annual Reports from 2009-2012. Historical data is summarized in the table below.

Table 4-4 Historical Effluent Concentrations

EFFLUENT PARAMETER	ANNUAL AVERAGE			
	2009	2010	2011	2012
CBOD5 (mg/L)	2.8	3.3	3.4	3.4
TSS (mg/L)	5.5	5.9	6.9	5.4
TP (mg/L)	0.52 (all months comply)	0.56 (all months comply)	0.50 (all months comply)	0.47 (all months comply)
pH	6.4	6.7	6.5	6.6
TAN (mg/L)	1.14	1.65	0.98	0.27
E. coli (organisms/100 mL)	3	10	7	5

The Dowling WWTP was in compliance with the effluent limits for all parameters.

A capacity review of each unit process at the WWTP was not conducted. Instead, the rated capacity was considered the true capacity of the plant.

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 Development of 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 WASTEWATER DESIGN CRITERIA

The unit flow criteria for growth adopted for this assessment are shown in Table 5-1 below. These values were recommended in the *Population Projections and Development of Unit Rates Technical Memorandum* (WSP, 2014).

Note that the term "extraneous flows" is used interchangeably with "I&I flows".

Table 5-1 Wastewater System Design Criteria

CRITERIA	VALUE	REFERENCE
Average Day Residential Flow	900 L/cap/day	Average of historical values
Average Day Commercial and Institutional Flow	28 m^3/ha/d	Water unit rates, assuming a 1:1 correspondence
Average Day Industrial Flow	35 m^3/ha/d	Water unit rates, assuming a 1:1 correspondence
Average Extraneous Flow	11.2 m^3/ha/d	Peak from City's Engineering Design Manual and assuming a peaking factor of three
Peak Extraneous Flow	33.7 m^3/ha/d	City's Engineering Design Manual (rate for new or proposed developments)
Max Day Peaking Factor	1.79	Average of historical values

The historical average day residential flow was very high compared to typical values in the rest of Greater Sudbury (ranging from 250 to 650 L/ca/d). Through discussions with City staff, the cause for this high rate was determined to be the high water table in Dowling.

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

Maximum day flows to the WWTP are obtained by multiplying the average day flow by the maximum day peaking factor.

5.2 DESIGN CRITERIA FOR WASTEWATER SYSTEM COMPONENTS AND OPERATION

5.2.1 WASTEWATER TREATMENT

Wastewater treatment facilities are rated for average day flows. Plant effluent limits and objectives are established in the *C of A* (MOECC, 1998) for the facility.

5.2.2 LIFT STATION PUMPING CAPACITY

The firm capacity of the lift station (with the largest pump out of service) must allow pumping of peak wet weather flows corresponding to its catchment area (MOECC, 2008).

Starting limitations on pump motors generally dictate the minimum size of a wet well. The wet well should be large enough to prevent pump motors from overheating due to frequent starting and stopping, but small enough to avoid long retention times leading to septicity and odor problems (Lin & Lee, 2001).

The station wet well shall be sized such that the number of pump starts per hour does not exceed the maximum value recommended by the pump manufacturer. In other words, the time between pump starts and stops (i.e. the pump cycle time) should not exceed that which results in a pump start frequency greater than that recommended by the pump manufacturer. Typically, submersible pumps can cycle four to 10 times per hour with a maximum cycle time not exceeding 30 minutes (Lin & Lee, 2001). A maximum value of four pump starts per hour was assumed to evaluate wet well sizing requirements.

5.2.3 SEWERS

The sewer system is typically sized to convey peak instantaneous (peak wet weather) flows. Sewage flows are made up of wastewater discharges from residential, commercial, institutional and industrial establishments, plus extraneous flow components from such sources as groundwater and surface runoff.

In addition to being able to convey peak flows, sufficient flow velocity should be maintained to transport the sewage solids to avoid deposition and the development of nuisance conditions under lower flow conditions. The minimum acceptable flow velocity in sewers is 0.6 m/s (City of Greater Sudbury, 2012).

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 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 *Population Projections and Development of 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 Dowling is projected to increase by 244 people by 2041 and 1,949 by Ultimate Buildout. The population projections to be used in the Master Plan are summarized in Table 6-1 below.

Table 6-1 Dowling Population Projections

SYSTEM	2011	2016	2021	2026	2031	2036	2041	ULTIMATE BUILDOUT
Dowling	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 the table below.

Table 6-2 Dowling ICI 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 and water 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 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 were presented in the *Population Projections and Development of Unit Rates Technical Memorandum* (WSP, 2014).

6.4 FUTURE WASTEWATER FLOW PROJECTIONS AND INFRASTRUCTURE NEEDS

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

$$\textbf{Projected Average Day Generation} = \textbf{Base Generation} + \textbf{Additional Residential Generation} + \textbf{Additional ICI Generation} + \textbf{Average Extraneous Flow}$$

The flows corresponding to the population growth forecasts to Ultimate Buildout are presented in Table 6-3 below.

Table 6-3 Flow Projections

YEAR	POPULATION	AVERAGE DAY FLOW (M^3/D)	MAXIMUM DAY FLOW (M^3/D)
Base	1,773	1,903	3,422
2016	1,837	2,106	3,765
2021	1,903	2,267	4,052
2026	1,965	2,367	4,230
2031	1,997	2,435	4,351
2036	2,017	2,452	4,382
2041	2,016	2,452	4,382
Ultimate	3,721	4,582	8,190

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 demand was calculated by applying the respective peaking factor to the average day demand. Base year maximum day demand was the historical average.

A desktop analysis of historical wastewater flows and future flow projections is included in **Appendix C**.

6.4.1 DOWLING WWTP CAPACITY

Based on the current WWTP average day rated capacity of 3,200 m^3/d, the wastewater treatment capacity will be sufficient to service growth projections until 2041. Additional capacity will be required prior to Ultimate Buildout. A Class Environmental Assessment is recommended to evaluate alternatives for expansion or mitigating I&I flows entering the sewage system.

The WWTP capacity is plotted with the flow projections on Figure 6-1 below.

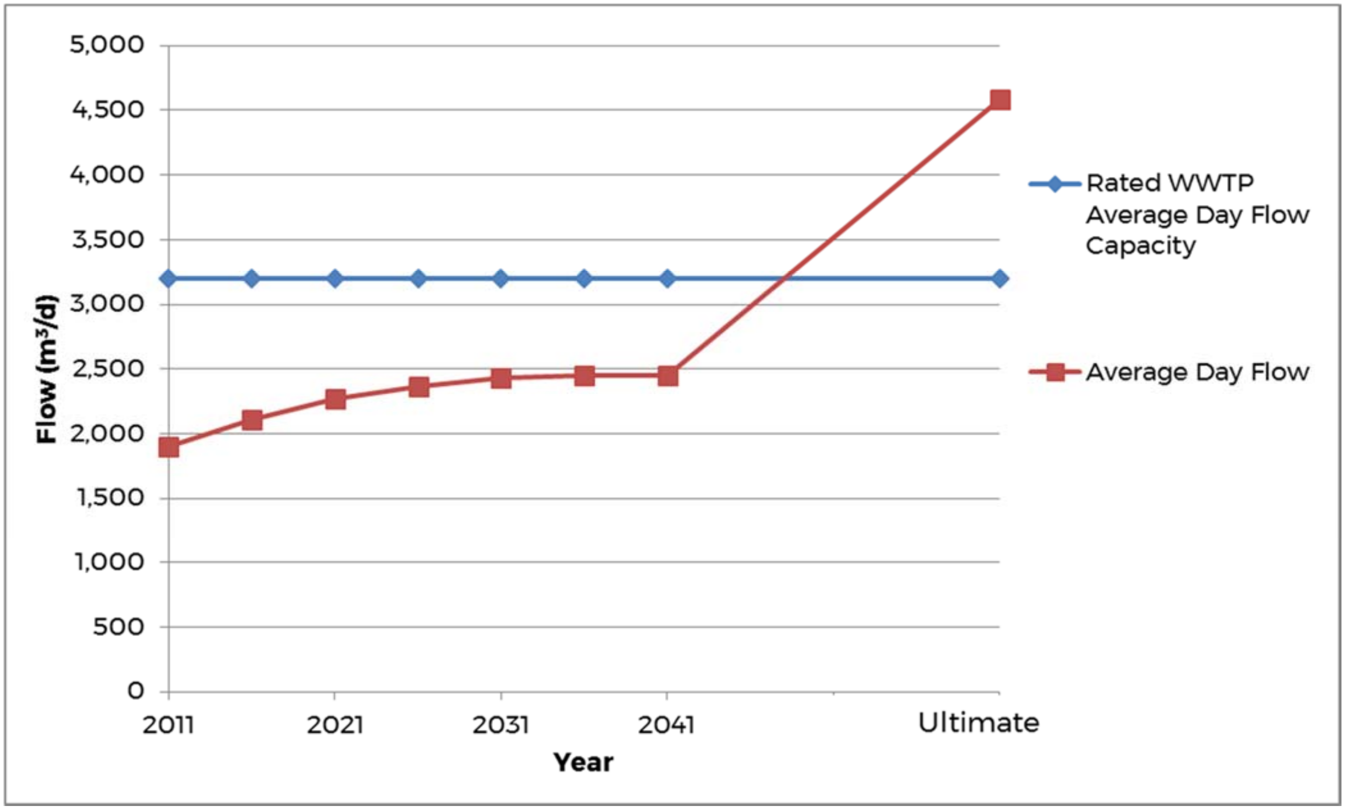

Figure 6-1 Wastewater Flow Projections Compared to WWTP Rated Capacity

6.4.2 SEWER NETWORK AND LIFT STATIONS

For each of the scenarios modeled, the system was checked for surcharging of sewers and capacity exceedance at the lift stations. The peak flows into each of the lift stations was determined from the computer simulations for the various planning scenarios and is presented in Table 6-4 below. The table also shows the design/rated flow for the pumps, their capacity based on drawdown tests and the computer simulated flow for comparison.

Table 6-4 Lift Station Peak Influent Flow Rates

	CURRENT FIRM CAPACITY	EXISTING PEAK FLOW	2041 PEAK FLOW	ULTIMATE BUILDOUT
Lionel	18.61	9.3	9.3	10.4

The Lionel LS has enough capacity to convey flows through Ultimate Buildout.

7 HYDRAULIC MODELLING

7.1 APPROACH

A basic sanitary model for the City of Greater Sudbury was received from the City. The model was created in Bentley's SewerGEMS by City staff. The model is an all pipe model of the sanitary network in these systems, but some critical information such as pipe data, invert elevations and lift station characteristics were missing. The model now includes this information as well as key vertical infrastructure in each system, including lift stations and treatment facilities.

The model was loaded with wet weather flow data. A water balance was completed to determine I&I rates for both dry and wet weather flow, as detailed in Section 0. The results from the water balance were compared against I&I rates developed through flow monitoring, and the greater of the two values, for each system, was used to load the model.

Current (2011) and future (2016-Ultimate Buildout, in 5 year increments) population data was added to the model using the City's planning data, summarized in previous sections of this report.

Future dry and wet weather flow scenarios were developed for each of the horizon years: 2016, 2021, 2026, 2031, 2036, 2041, and Ultimate Buildout. However, model results did not vary from 2016 to 2041; therefore, this report discusses findings for 2041 and Ultimate Buildout, compared against existing (2011).

7.2 MODELLING FINDINGS

The model was used to check sewer capacity and flow velocity. No capacity issues were identified in Dowling, as most of the sewers flow at less than 50% capacity.

Flow velocities through most of the Dowling sewer system are generally below the City's standard of 0.6 m/s. This is consistent through to Ultimate Buildout under the wet weather flow condition.

Maps in **Appendix B** illustrate the modeling results for the 2011, 2041, and Ultimate wet weather flow scenarios.

8 CONCLUSIONS

An assessment of the Dowling Wastewater System was completed to identify infrastructure requirements to service forecasted growth in the community.

The conclusions of the assessment are summarized below.

— The WWTP is deemed to have sufficient average day capacity to service growth to 2041, but will need to be expanded for Ultimate Buildout.
— There are recognized issues with I&I in the system, likely due to the high water table in Dowling.
— No capacity issues were identified in the lift station or sewer system. However, most of the sewers flow at less than the City's current standard flow velocity of 0.6 m/s.

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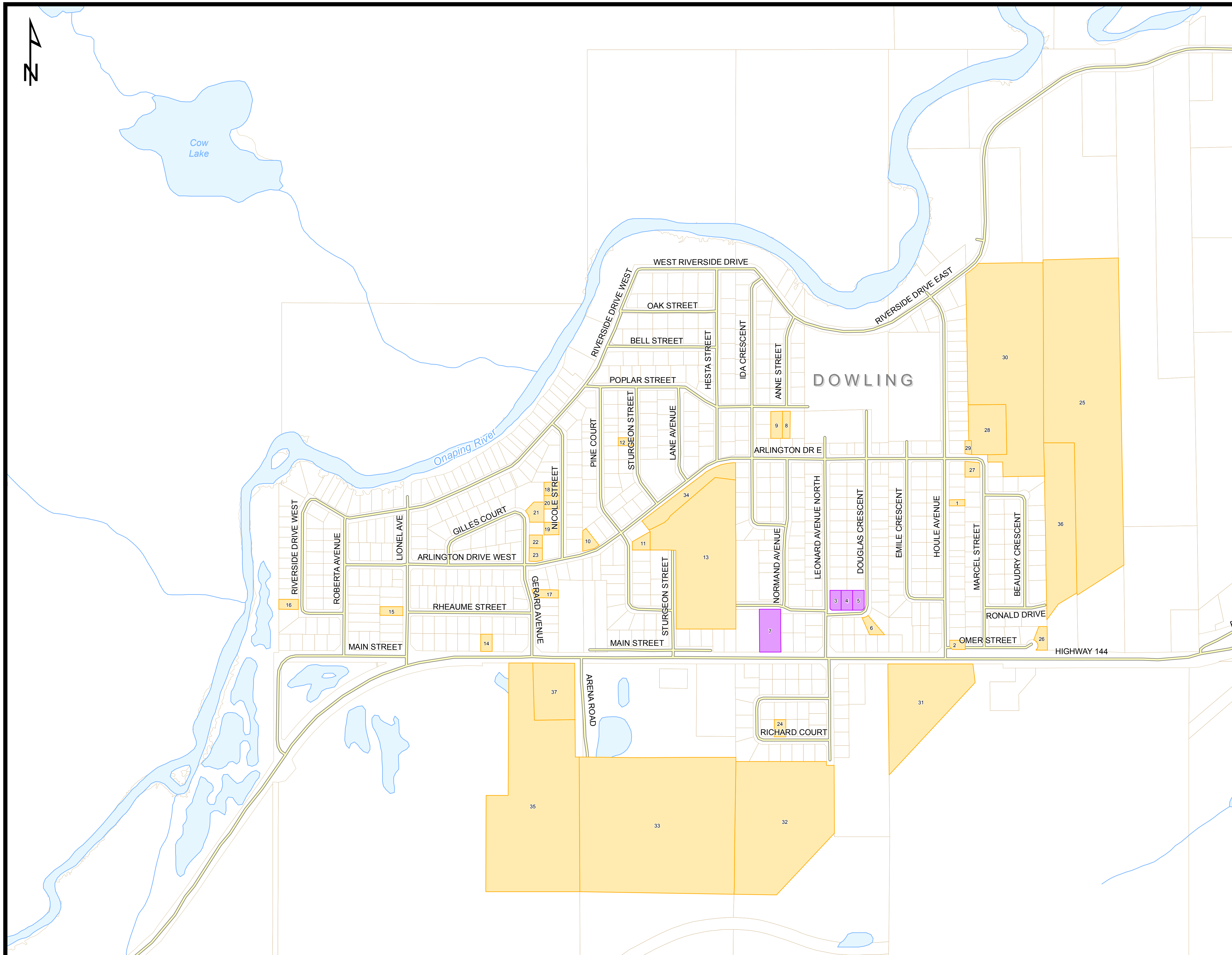
APPENDIX

A RESIDENTIAL AND ICI DEVELOPMENT AREAS



Legend

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

Greater Sudbury Grand | WSP

Project No. 121-23026-00

Sudbury Water and Wastewater Master Plan

0 100 200 300 400 500 m

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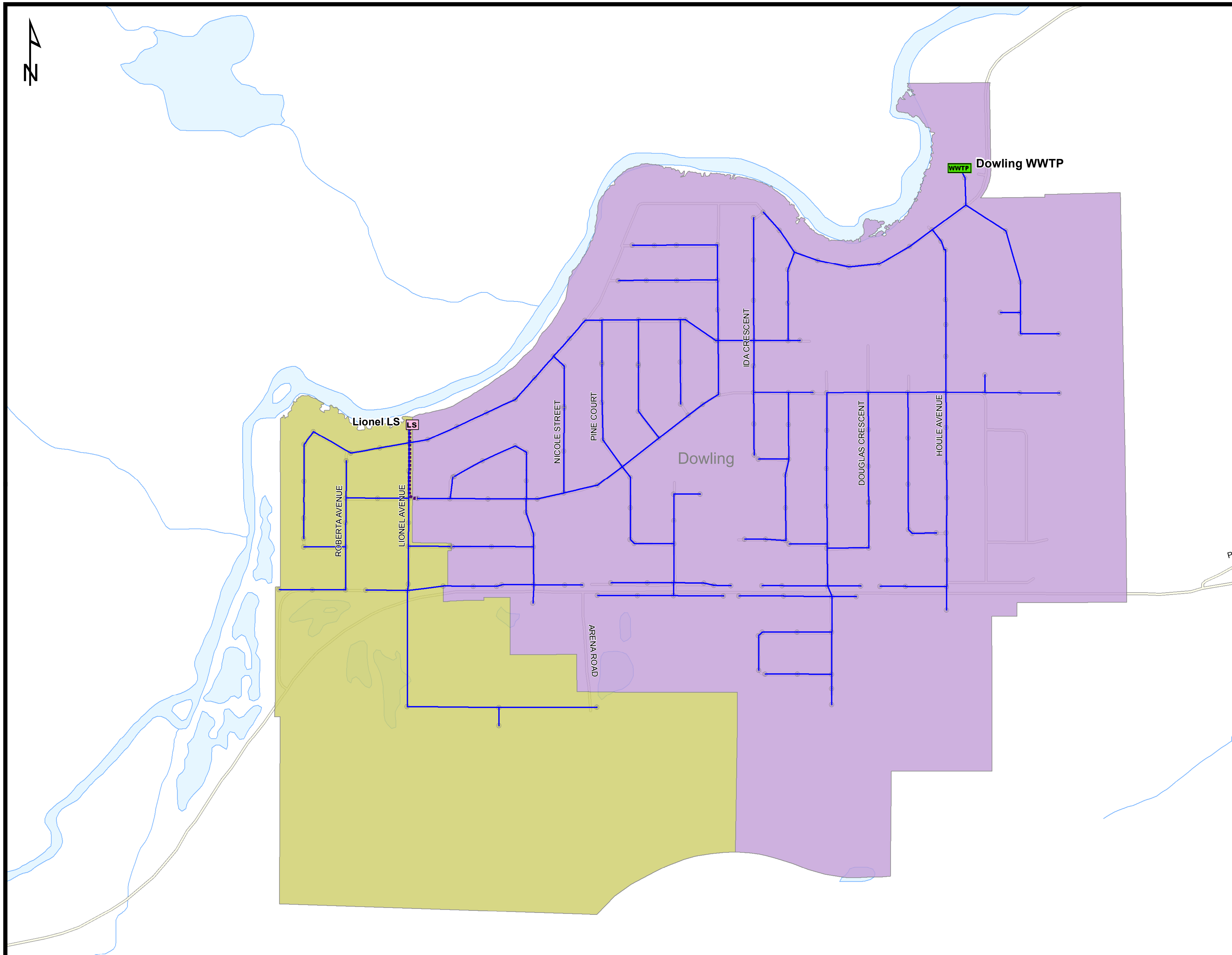
Map 7

Dowling
2011 - Ultimate Buildout
Residential and ICI
Development Areas

APPENDIX

B WASTEWATER MODEL RESULTS



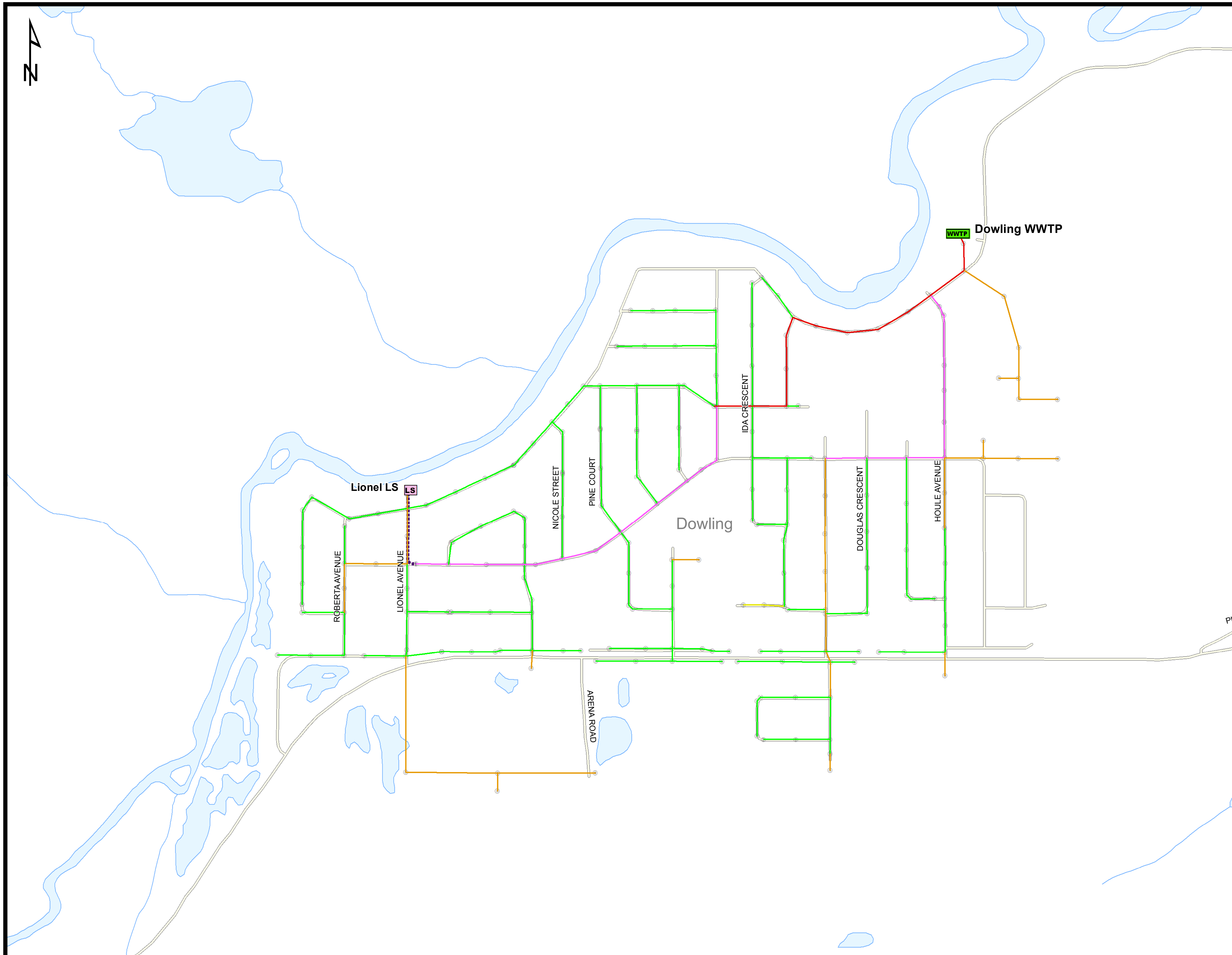
Project No. 121-23026-00

December 09 2016

Sudbury Water and Wastewater Master Plan

Map 1

Dowling Wastewater System - Sewershed Map



Legend

- LS Lift Station
- WWTP Sanitary Treatment Plant
- Manhole
- Waterbody
- Road
- Watercourse
- Forcemain

Conduit: Diameter (mm)

- <= 150
- <= 200
- <= 300
- <= 450
- <= 600
- <= 750
- > 750

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Sudbury Water and Wastewater Master Plan

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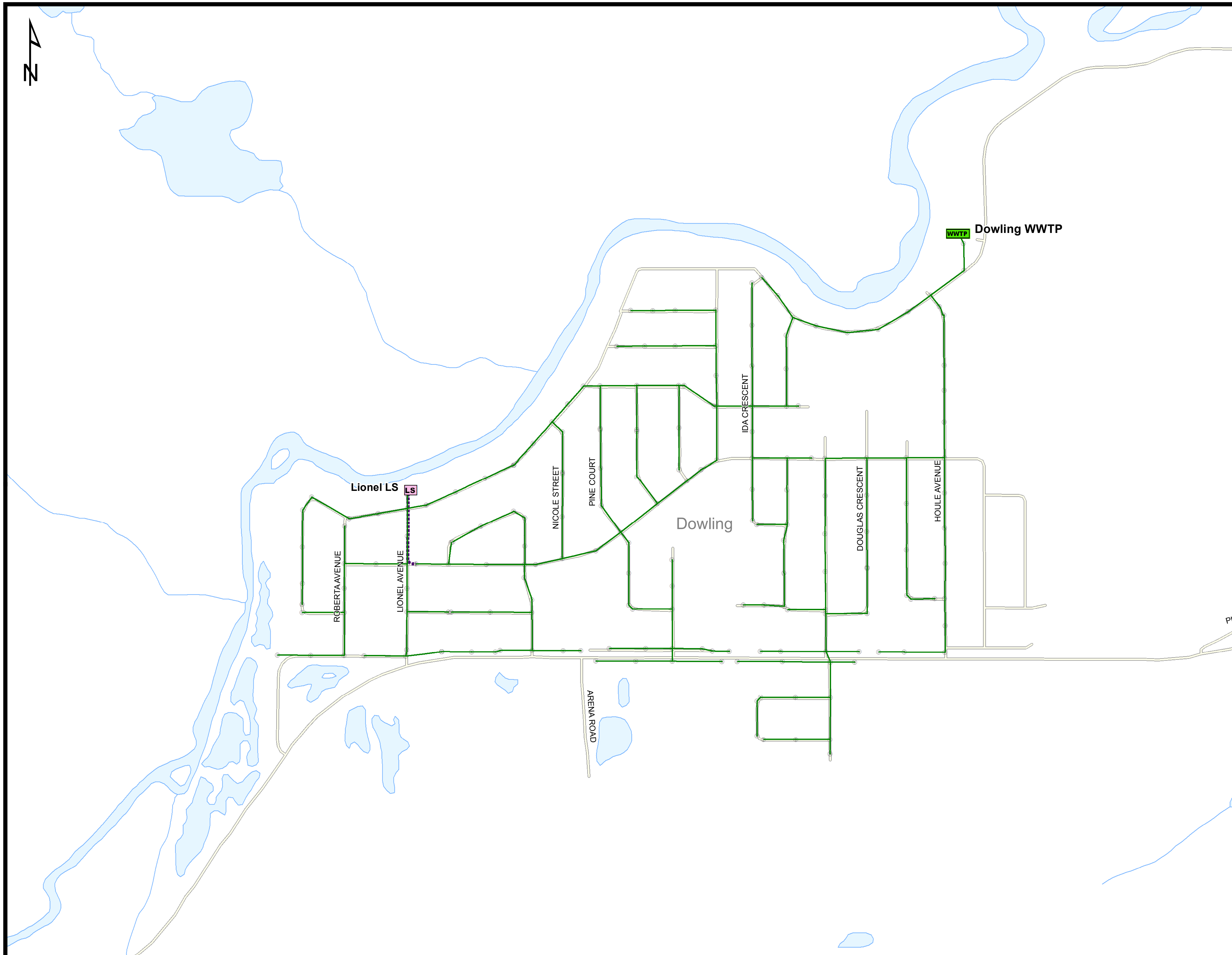
Map 2

Dowling Wastewater System - Conduit Diameter



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October 04 2017

Sudbury Water and Wastewater Master Plan

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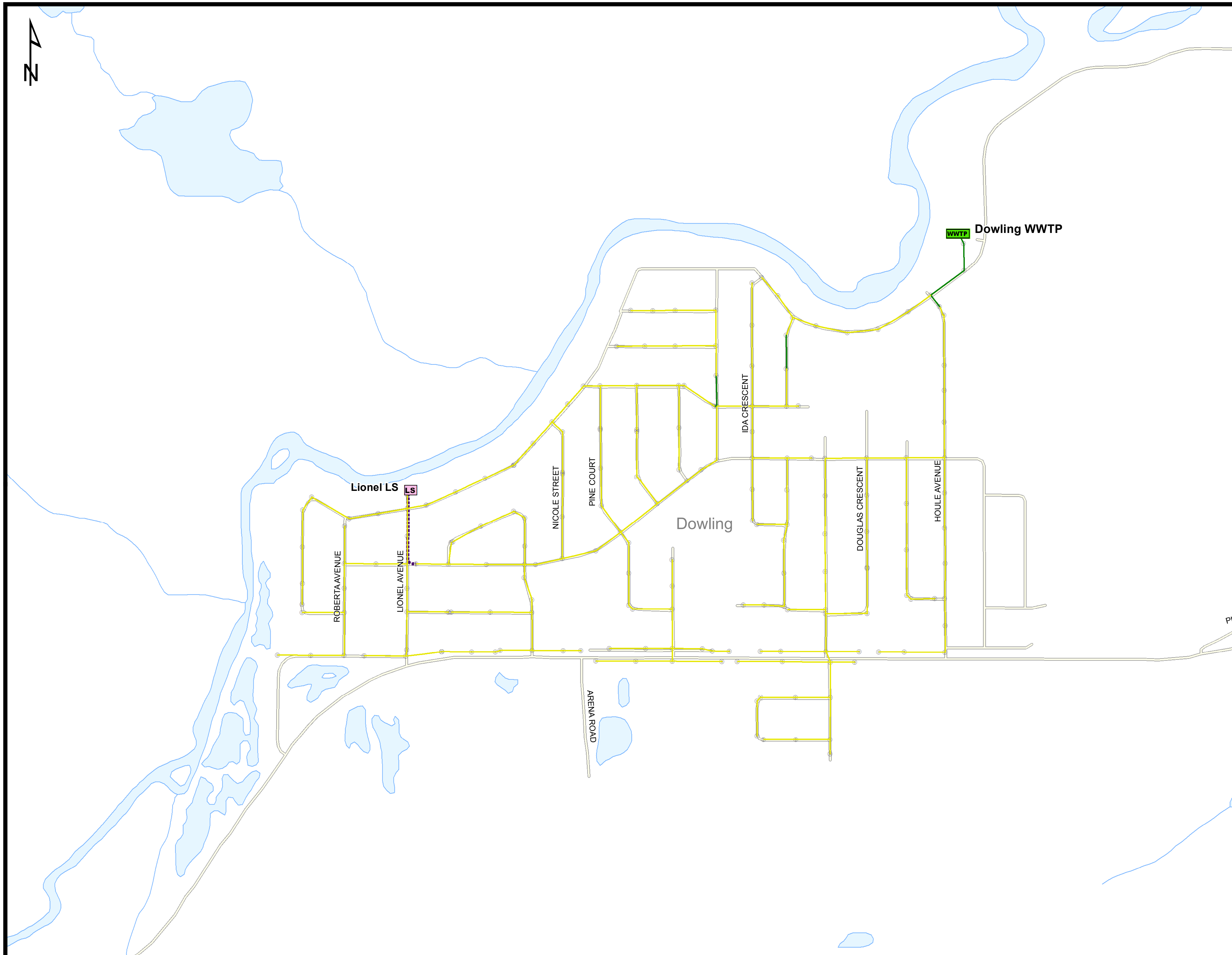
Map 3.1

Dowling Wastewater System - Conduit Flow/Capacity (Design)

Scenario: 2011WWF_2yrStorm



Project No. 121-23026-00

October 04 2017

Sudbury Water and Wastewater Master Plan

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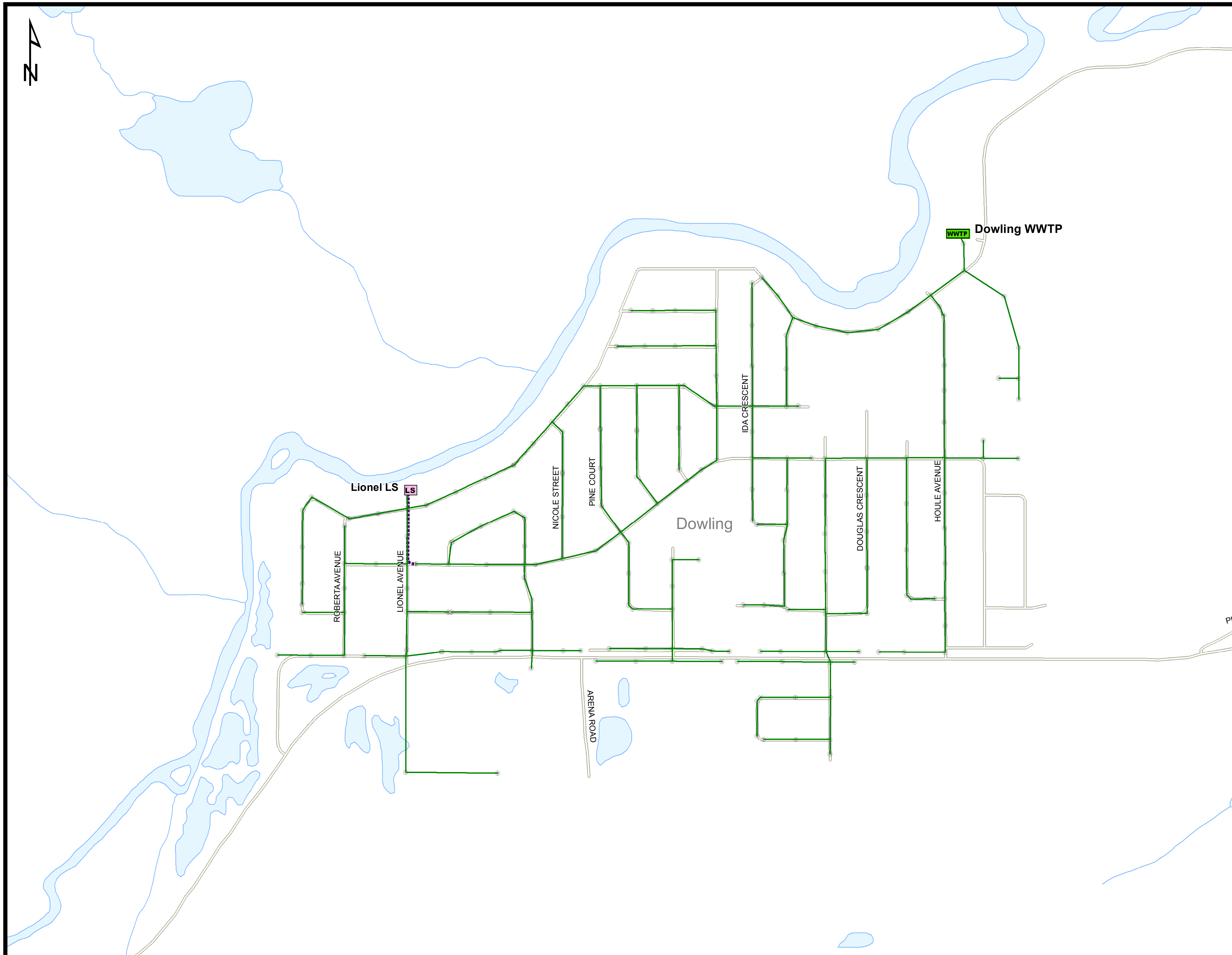
Map 3.2

Dowling Wastewater System - Conduit Velocity

Scenario: 2011WWF_2yrStorm



Legend

- LS Lift Station
- WWTP Sanitary Treatment Plant
- Manhole
- Waterbody
- Road
- Watercourse
- Forcemain

Conduit: Flow/Capacity (Design) (%)

- <= 50
- <= 80
- <= 100
- > 100

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Project No. 121-23026-00

December 09 2016

Sudbury Water and Wastewater Master Plan

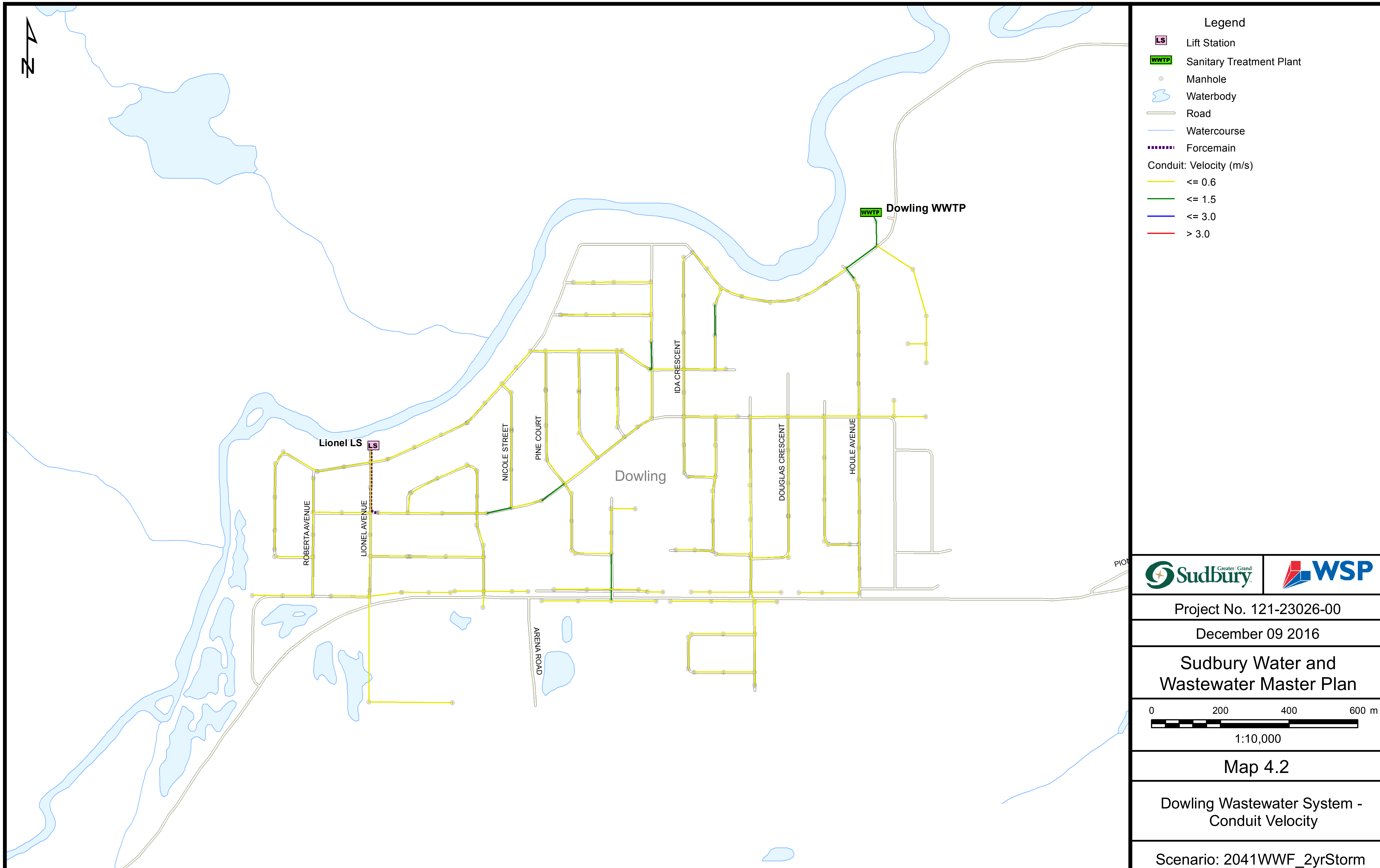
0 200 400 600 m

1:10,000

Map 4.1

Dowling Wastewater System - Conduit Flow/Capacity (Design)

Scenario: 2041WWF_2yrStorm



Project No. 121-23026-00

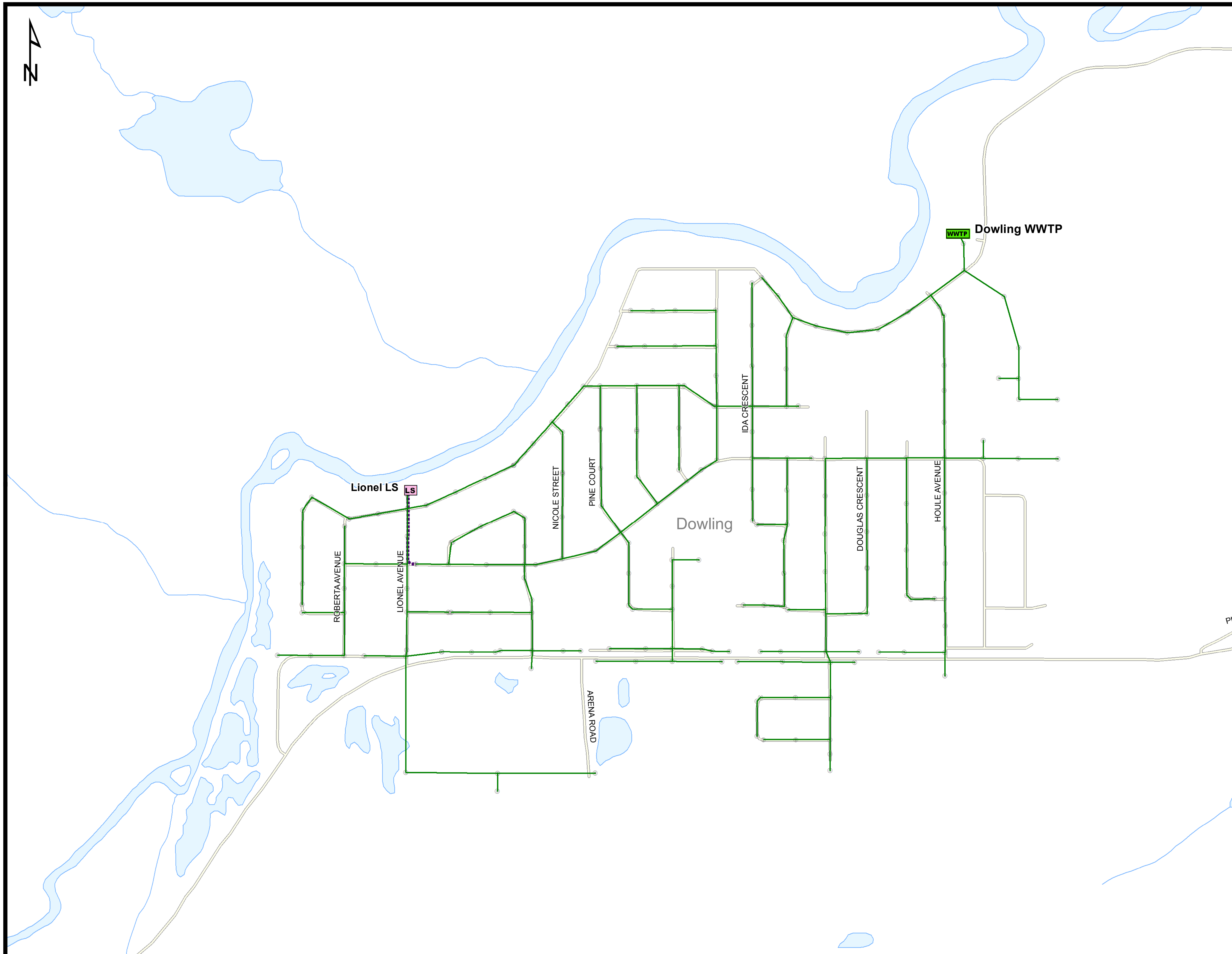
December 09 2016

Sudbury Water and Wastewater Master Plan

Map 4.2

Dowling Wastewater System - Conduit Velocity

Scenario: 2041WWF_2yrStorm



Project No. 121-23026-00

December 09 2016

Sudbury Water and Wastewater Master Plan

0 200 400 600 m

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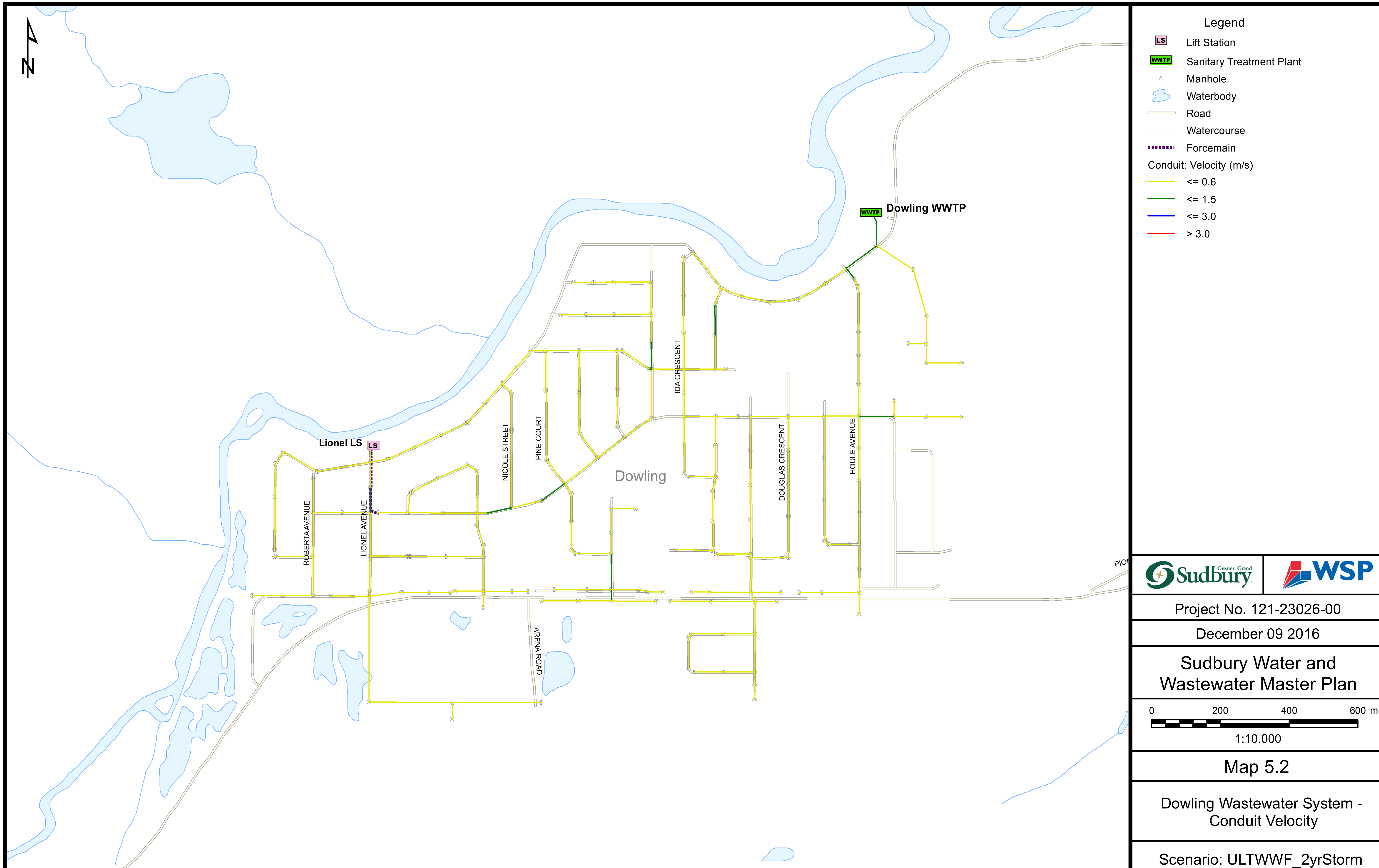
Map 5.1

Dowling Wastewater System - Conduit Flow/Capacity (Design)

Scenario: ULTWWF_2yrStorm



Project No. 121-23026-00

December 09 2016

Sudbury Water and Wastewater Master Plan

Map 5.2

Dowling Wastewater System - Conduit Velocity

Scenario: ULTWWF_2yrStorm

APPENDIX

C WASTEWATER GENERATION AND CAPACITY ASSESSMENTS

Dowling - Wastewater Flow Forecasts

		2009	2010	2011	2012	2013	Summary	Design Criterion	Comments
Average Day Flow	**(m^3/d)**	2,033	1,749	1,846	1,837	2,051	1,903	1,903	From Annual Reports
Max Day Flow	**(m^3/d)**	4,000	3,180	2,940	2,650	4,338	3,422	3,422	From Annual Reports
Max Day Factor		1.97	1.82	1.59	1.44	2.12	1.79	1.79	Calculated - Max Day Flow divided by Average Day Flow
Peak Flow	**(m^3/d)**	Not Available						0	Peak hour flows were not available
Peak Flow	**(L/s)**							0	
Peak Flow Factor									
Population (Existing Areas)		1,773	1,773	1,773	1,773	1,773	1,773	1,773	
Ratio of Residential to Total Customers		0.82	0.82	0.82	0.82	0.82	0.82		This ratio is based on Water Billing Records for the area and is an approximation of the residential portion of demand.
Residential Share of Average Day Demand (m^3/d)		1662	1430	1510	1502	1677	1556		Institutional growth area multiplied by unit flow rate.
Residential Flow Unit Rate (m^3/cap/d)		0.938	0.807	0.852	0.847	0.946	0.878	0.900	Commercial growth area multiplied by unit flow rate.
Average Institutional Flow Unit Rate (m^3/ha/d)								28.0	Industrial growth area multiplied by unit flow rate.
Average Commercial Flow Unit Rate (m^3/ha/d)								28.0	CGS design standards do not suggest average rates for institutional and commercial, but require that rates be developed based on usage. In this case, water demands rates were used and assume a conservative 1:1 ratio of water to sewage.
Average Industrial Flow Unit Rate (m^3/ha/d)								35.0	CGS design standards average rate for light industrial of 35 m^3/ha/d.
Average Extraneous Flow Unit Rate (m^3/ha/d)								11.23	From CGS Design Standards, peak rate for new developments divided by an assumed peaking factor of 3. This factor would be applied only to new developments, which are assumed to be leak-tight, and have minimal extraneous flow.

	2016	2021	2026	2031	2036	2041	Ultimate Buildout	Comments
Population (Existing Areas)	1,773	1,773	1,773	1,773	1,773	1,773	1,773	
Population (Growth Areas)	64	130	193	225	244	244	1,949	
Total Population	1837	1903	1965	1997	2017	2016	3721	Total Population (Hemson)
Residential Growth Area (ha)	12.96	9.01	0.00	3.44	0.00	0.00	53.10	
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	ICI development areas were assigned to planning years based on the stage of the application. Draft Approved were assigned to 2016, Legal Lots of Record to 2026, and Designated Developable to 2036.
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	Areas are cumulative and carry from the development year, all the way through to Ultimate Buildout
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	
ICI (ha) - Cumulative	0.00	0.00	1.11	1.11	1.11	1.11	1.11	
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 Buildout	Comments
Average Residential Flows (m^3/d) - Existing	1,903	1,903	1,903	1,903	1,903	1,903	1,903	This includes all contribution from existing ICI and infiltration. The base flow was assumed to be the average day flow to the plant for the 2011-2013 period.
Average Residential Flows (m^3/d)	58	117	173	202	220	220	1,754	Obtained by multiplying the projected population growth by the unit rate.
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	Multiplying the total population by the consolidated per capita flow factor.
Average Extraneous Flow (m^3/d	146	247	259	298	298	298	894	
Average Day Flow (m^3/d)	2,106	2,267	2,367	2,435	2,452	2,452	4,582	
Max Day Flow (m^3/d)	3,765	4,052	4,230	4,351	4,382	4,382	8,190	

ALTERNATIVE CALCULATION METHOD

Per Capita Flow (m^3/cap/day)	1.147	0.987	1.041	1.036	1.157	1.074	1.074
Average Day Flow (m^3/d)							
Max Day Flow (m^3/d)							

2016	2021	2026	2031	2036	2041	Ultimate Buildout
1972	2043	2110	2145	2165	2165	3995
3,524	3,651	3,771	3,833	3,870	3,869	7,141

Comments

Multiplying the total population X the consolidated per capita flow factor.

SENSITIVITY ANALYSIS

Analyze sensitivity of forecasted flows to unit rate

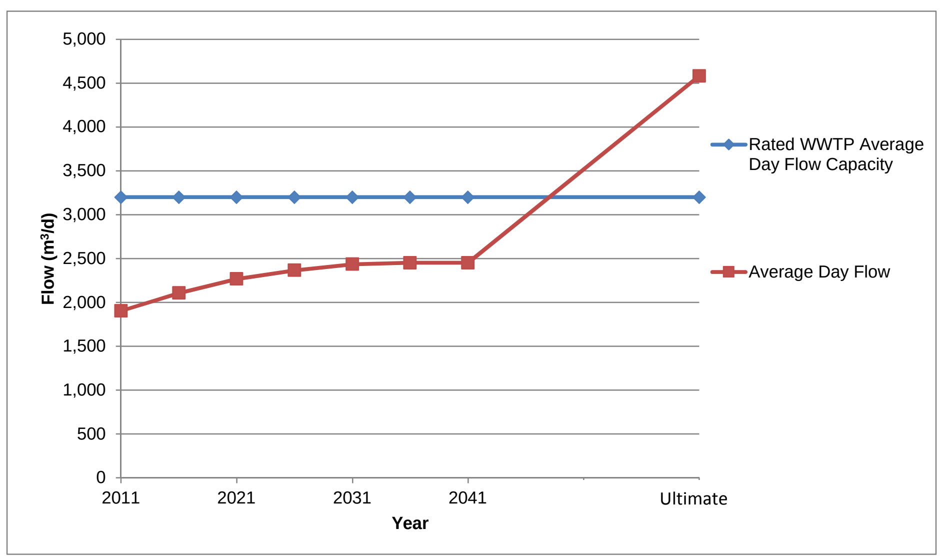
	Average Day Flow (m^3/d)							Ultimate Buildout
	Unit Rate (m^3/cap/d)	2016	2021	2026	2031	2036	2041	2061
Base Scenario - Residential Historical Maximum	0.878	2,106	2,267	2,367	2,435	2,452	2,452	4,582
Combined Historical Maximum	1.074	2,117	2,290	2,400	2,474	2,494	2,494	4,921
City Standards	0.360	2,072	2,197	2,263	2,313	2,320	2,320	3,530

Analyze sensitivity of forecasted flows to max day factor

	Max Day Flow (m^3/d)							Ultimate Buildout
	Max Day Peaking Factor	2016	2021	2026	2031	2036	2041	2061
Base Scenario - Historical Max	1.74	3,670	3,950	4,124	4,241	4,271	4,271	7,983
Historical Average	2.12	3,765	4,052	4,230	4,351	4,382	4,382	8,190

CAPACITY CHECK

								Ultimate Buildout
	2011	2016	2021	2026	2031	2036	2041	2061
Rated WWTP ADF Capacity (m^3/d)	3,200	3,200	3,200	3,200	3,200	3,200	3,200	3,200
Average Day Flow (m^3/d)	1,903	2,106	2,267	2,367	2,435	2,452	2,452	4,582



CITY OF GREATER SUDBURY WATER AND WASTEWATER MASTER PLAN

FALCONBRIDGE WASTEWATER SYSTEM GAP ANALYSIS AND STATUS QUO REPORT

CITY OF GREATER SUDBURY

DRAFT

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

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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 existing Falconbridge Wastewater System. Based on population growth projections and design criteria discussed in the *Population Projections and Development of Unit Rates Technical Memorandum* (WSP, 2014), 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 Wastewater System would continue to be a stand-alone system. 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.

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.

Mapping in **Appendix A** shows 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 *Population Projections and Development of Unit Rates 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 776 by 2041 and 855 by Ultimate Buildout.

ICI growth is expected to be mixed use, but primarily lands with institutional zoning. Growth is discussed further in Section 6.1.

3 OVERVIEW OF EXISTING SYSTEM

All wastewater generated in Falconbridge is collected and treated at the Falconbridge Wastewater Treatment Plant (WWTP). There are no lift stations in Falconbridge and all sewage flows by gravity to the plant. The collection system consists of approximately 7 km of sewers. No bypasses were reported at the plant or in the collection system in the years reviewed (2011-2014). Additional information on the existing system is provided in the Baseline Review Report for Wastewater Systems (WSP, 2014).

The Falconbridge Wastewater System is shown in **Appendix B**.

3.1 FALCONBRIDGE WWTP

The Falconbridge WWTP was constructed in 1978 and originally owned by Falconbridge Nickel Mines Limited. Ownership was transferred to the former Regional Municipality of Sudbury in the 1980s. The plant is a trickling filter plant with an average day capacity of 909 m^3/d, according to a 1979 letter from the MOECC. The treatment process is illustrated schematically below.

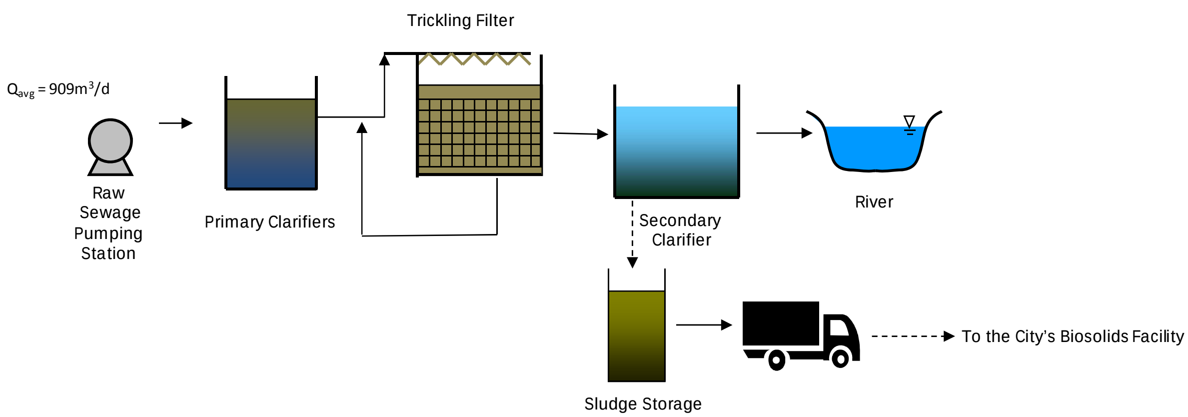

Figure 3-1 Falconbridge WWTP Process

3.2 KNOWN CHALLENGES

The Falconbridge Wastewater System has the following known challenge:

— The wastewater collection system was originally constructed by area mining companies to service their employees' homes, and transferred to the City of Greater Sudbury years later. Much of the linear infrastructure does not meet current standards of construction. For example, some sewers are shallow (less than 1 m deep), while others are located in backyards. For backyard sewers, structures such as pools and fences have been built on top and access is difficult.

4 HISTORICAL FLOWS AND REVIEW OF OPERATIONAL DATA

Data reported in the 2009 to 2013 *Annual Reports* for the Falconbridge WWTP was reviewed and analyzed to determine average day and maximum day flows as well as review effluent parameters.

4.1 FLOW DATA

WWTP flow data from 2009 to 2013 was reviewed. Historical peak flow data was not available.

The recorded average day and maximum day flows are summarized in Table 4-1 and plotted in Figure 4-1 below.

Table 4-1 Historical Wastewater Flow Data

YEAR	AVERAGE DAY FLOW (M^3/D)[1]	MAXIMUM DAY FLOW (M^3/D)[1]
2009	250	342
2010	240	558
2011	264	629
2012	242	765
2013	245	497

[1] *Annual Reports (2009 - 2013).*

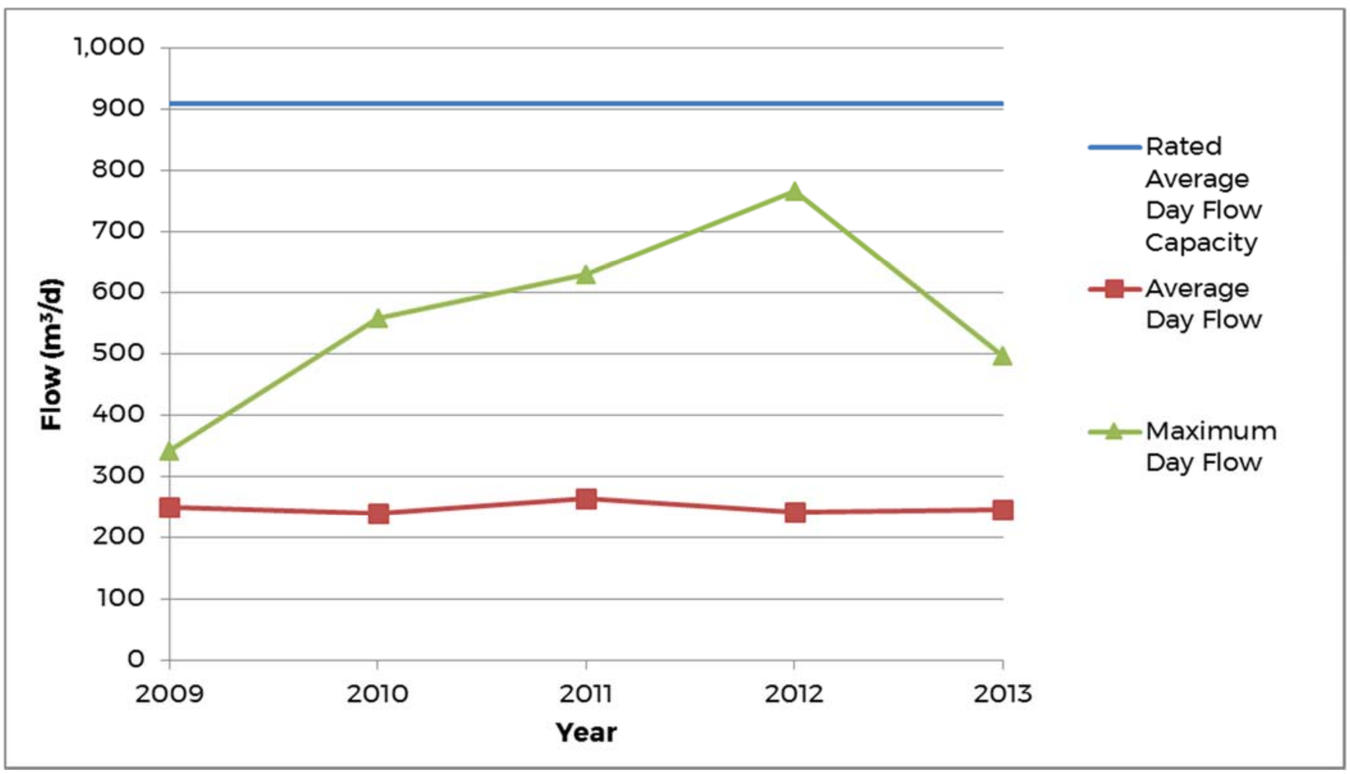

Figure 4-1 Historical Wastewater Flows at the Falconbridge Wastewater Treatment Plant

The relationship between the different flow regimes was analyzed to compare the maximum day peaking factors derived from historical data to those used in the *City's Engineering Design Manual* and those included in the *MOECC Guidelines.*

The average day flows to the WWTP have been steady over the 2009 to 2013 period, averaging 248 m^3/d. The variations in historical maximum day flows show no discernible trend when all flow data is considered. Similarly, on comparison with historical rainfall data for the Sudbury station, there is no clear correlation. For example, 2009 had the most precipitation (986.4 mm) but flows to the plant were lowest, while 2010 had the lowest precipitation (659.8 mm), but flows to the plant were higher than the previous, wetter year.

The highest maximum day to average day peaking factor was 3.16 (2012). The average maximum day peaking factor from 2009 to 2013 was 2.25. The City's *Engineering Design Manual* and the *MOECC Guidelines* do not specify recommended maximum day factors and recommend using historical data when available. For future wastewater generation, the average peaking factor (2.25) was used and based on the assumption that new developments would have less I&I due to more leak tight construction.

4.2 RAW WASTEWATER CHARACTERISTICS

The average raw wastewater characteristics from 2009 to 2012 are summarized in the table below. Raw wastewater temperatures were not reported and raw wastewater total Kjeldahl nitrogen (TKN) was reported only from 2009 to June 2010.

Table 4-2 Average Raw Wastewater Characteristics at the Falconbridge WWTP (2009-2012)

PARAMETER	AVERAGE VALUE
$CBOD_5$	245 mg/L
Suspended Solids	119 mg/L
Total Phosphorus	7.5 mg/L
TKN	61 mg/L (2009-2010 only)
pH	7.0

Wastewater flows to the Falconbridge WWTP correspond to mixed uses, with contributions from residential and ICI users, and dilution from inflow and infiltration.

4.3 EFFLUENT CRITERIA

The Falconbridge WWTP was previously owned and operated by Falconbridge Nickel Mines Limited. The plant was transferred to the City in 1979 and is operated under a Certificate of Approval dated August 23, 1978 and in accordance with a letter from the Ministry of the Environment (MOECC) dated July 17, 1979. The letter indicates that the sewage flows through the plant may not exceed an average of allowable flow of 909 m^3/d. The letter also stipulates that sufficient stream flow is maintained through the effluent receiving area of the existing outfall to provide a 20:1 dilution ratio under average conditions, and 10:1 dilution ratio under worst conditions. Finally, any future increase in flows beyond 909 m^3/d average day flow would require that the plant be expanded.

The letter and C of A do not provide any effluent limits. However, the Annual Reports stipulate compliance limits of 25 mg/L (22.7 kg/d loading) for each $CBOD_5$ and TSS.

4.4 OPERATIONAL DATA

The general operation of the WWTP was reviewed against the C of A requirements and historical data provided in the Annual Reports from 2009-2012. Historical data is summarized in the table below.

Table 4-3 Historical Effluent Concentrations

EFFLUENT PARAMETER	ANNUAL AVERAGE			
	2009	2010	2011	2012
$CBOD_5$ (mg/L)	1.4	1.4	1.3	1.5
TSS (mg/L)	1.9	1.8	2.4	2.6
TP (mg/L)	0.07	0.03	0.02	0.02
pH	7.00	6.90	6.80	6.68
TAN (mg/L)	0.46	0.16	0.25	0.38
E. coli (organisms/100 mL)	220	110	293	73

The Falconbridge WWTP meets compliance parameters for $CBOD_5$ and TSS. Although there are not specific effluent requirements for the remaining parameters, the plant meets typical wastewater effluent limits.

A capacity review of each unit process at the WWTP was not conducted. Instead, the rated capacity was considered the true capacity of the plant.

Biosolids from the Falconbridge WWTP are hauled to the Biosolids Facility at the Sudbury WWTP, and must meet specific quality requirements. This includes inorganic compounds such as plastics. However, the WWTP does not currently have fine screening to remove such compounds.

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 WASTEWATER DESIGN CRITERIA

The unit flow criteria for growth adopted for this assessment are shown in Table 5-1 below. These values were recommended in the *Populations and Unit Rates Technical Memorandum* (WSP, 2014).

Note that the term "extraneous flows" is used interchangeably with "I&I flows".

Table 5-1 Wastewater System Design Criteria

CRITERIA	VALUE	REFERENCE
Average Day Residential Flow	400 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	Water unit rates, assuming a 1:1 correspondence
Average Day Industrial Flow	35 m^3/ha/d	Water unit rates, assuming a 1:1 correspondence
Average Extraneous Flow	11.2 m^3/ha/d	Peak from City's Engineering Design Manual and assuming a peaking factor of three
Peak Extraneous Flow	33.7 m^3/ha/d	City's Engineering Design Manual
Max Day Peaking Factor	2.25	Average of historical values

Residential average day flows were obtained by multiplying the residential unit rate by the service population. However, the unit residential rate for Falconbridge was much lower than typical values (66 L/ca/d) and not recommended for planning purposes. Therefore, the value used in the City's Engineering Design Manual (410 L/ca/d) was used, but rounded down slightly to the nearest hundred.

Maximum day flows to the WWTP are obtained by multiplying the average day flow by the maximum day peaking factor.

5.2 DESIGN CRITERIA FOR WASTEWATER SYSTEM COMPONENTS AND OPERATION

5.2.1 WASTEWATER TREATMENT

Wastewater treatment facilities are rated for average day flows. Plant effluent limits and objectives are established in the C of A or ECA for each facility.

5.2.2 LIFT STATION PUMPING CAPACITY

The firm capacity of the lift station (with the largest pump out of service) must allow pumping of peak wet weather flows corresponding to its catchment area (MOECC, 2008).

Starting limitations on pump motors generally dictate the minimum size of a wet well. The wet well should be large enough to prevent pump motors from overheating due to frequent starting and stopping, but small enough to avoid long retention times leading to septicity and odor problems (Lin & Lee, 2001).

The station wet well shall be sized such that the number of pump starts per hour does not exceed the maximum value recommended by the pump manufacturer. In other words, the time between pump starts and stops (i.e. the pump cycle time) should not exceed that which results in a pump start frequency greater than that recommended by the pump manufacturer. Typically, submersible pumps can cycle four to 10 times per hour with a maximum cycle time not exceeding 30 minutes (Lin & Lee, 2001). A maximum value of four pump starts per hour was assumed to evaluate wet well sizing requirements.

5.2.3 SEWERS

The sewer system is typically sized to convey peak instantaneous (peak wet weather) flows. Sewage flows are made up of wastewater discharges from residential, commercial, institutional and industrial establishments, plus extraneous flow components from such sources as groundwater and surface runoff.

In addition to being able to convey peak flows, sufficient flow velocity should be maintained to transport the sewage solids to avoid deposition and the development of nuisance conditions under lower flow conditions. The minimum acceptable flow velocity in sewers is 0.6 m/s (City of Greater Sudbury, 2012).

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 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 Falconbridge population with wastewater servicing is projected to increase by 776 people by 2041 and 855 people by Ultimate Buildout.

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

Table 6-1 Valley Population Projections

SYSTEM	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 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 the table below.

Table 6-2 Falconbridge ICI Projections

	ICI DEVELOPMENT AREAS (HA)						
LAND USE	2016	2021	2026	2031	2036	2041	BUILDOUT
Institutional	0	0	1.70	0	0.33	0	0
Commercial	0	0	0.17	0	0	0	0
Industrial	0	0	0.36	0	0	0	0
Total	**0**	**0**	**2.23**	**0**	**0.33**	**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 PHASING OF FUTURE GROWTH

Growth areas were allocated based on population projections for individual developments and the overall target growth population projections for Falconbridge 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 WASTEWATER FLOW PROJECTIONS AND INFRASTRUCTURE NEEDS

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

$$\textbf{Projected Average Day Generation} \\ = \textbf{Base Generation} + \textbf{Additional Residential Generation} + \textbf{Additional ICI Generation} \\ + \textbf{Average Extraneous Flow}$$

The flows corresponding to the population growth forecasts to Ultimate Buildout are presented in Table 6-3 below.

Table 6-3 Flow Projections

YEAR	POPULATION	AVERAGE DAY FLOW (M^3/D)	MAXIMUM DAY FLOW (M^3/D)
Base	707	248	558
2016	724	269	606
2021	743	287	647
2026	759	389	876
2031	769	393	885
2036	775	408	919
2041	776	408	920
Ultimate	855	441	994

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 demand was calculated by applying the respective peaking factor to the average day demand. The maximum day demand for the base year was the historical average.

A desktop analysis of historical wastewater flows and future flow projections is included in **Appendix C**.

6.3.1 FALCONBRIDGE WWTP CAPACITY

The rated average day capacity of the Falconbridge WWTP is 909 m^3/d, and is compared to the current and future flow projections on Figure 6-1 below.

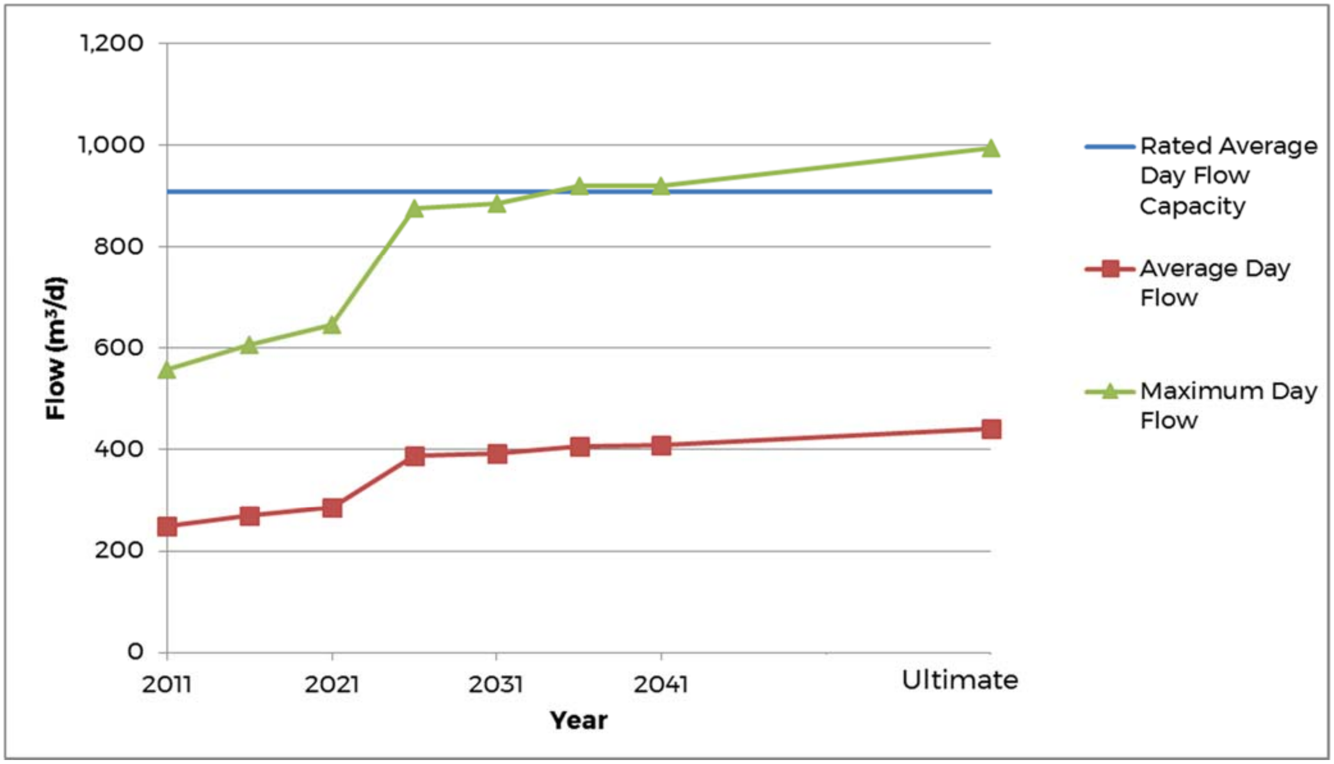

Figure 6-1 Wastewater Flow Projections Compared to Falconbridge WWTP Rated Capacity

As indicated in the above analysis, the Falconbridge WWTP can continue operating under its current capacity until Ultimate Buildout.

7 HYDRAULIC MODELLING

7.1 MODEL DEVELOPMENT

A basic sanitary model for the City of Greater Sudbury was received from the City. The model was created in Bentley's SewerGEMS by City staff. The model is an all pipe model of the sanitary network in these systems, but some critical information such as pipe data, invert elevations and lift station characteristics were missing. The model now includes this information as well as key vertical infrastructure in each system, including lift stations (not applicable in this system) and treatment facilities.

The model was loaded with wet weather flow data. A water balance was completed to determine I&I rates for both dry and wet weather flow. The results from the water balance were compared against I&I rates developed through flow monitoring, and the greater of the two values, for each system, was used to load the model.

Current (2011) and future (2016-Ultimate Buildout, in 5 year increments) population data was added to the model using the City's planning data, summarized in previous sections of this report.

Future dry and wet weather flow scenarios were developed for each of the horizon years: 2016, 2021, 2026, 2031, 2036, 2041, and Ultimate Buildout. However, model results did not vary from 2016 to 2041; therefore, this report discusses findings for 2041 and Ultimate Buildout, compared against existing (2011).

7.2 MODELLING FINDINGS

The model was used to check sewer capacity and flow velocity. No capacity issues were identified in Falconbridge, as most of the sewers flow at less than 50% capacity.

Flow velocities through many areas in the Falconbridge sewer system are below the City's standard of 0.6 m/s. This is consistent through to Ultimate Buildout under the wet weather flow condition. Remaining areas are between 0.6 and 1.5 m/s, therefore in accordance with the standard.

Maps in **Appendix B** illustrate the modeling results for the 2011, 2041, and Ultimate wet weather flow scenarios.

8 CONCLUSIONS

An assessment of the Falconbridge Wastewater System was completed to identify infrastructure requirements to service forecasted growth in the community.

The conclusions of the assessment are summarized below.

- The WWTP is deemed to have sufficient average day capacity to service growth to Ultimate Buildout.
- Many sewers in Falconbridge operate at velocities that do not meet the City's current standards (that is, many sewers flow at less than 0.6 L/s). This may cause operational problems, such as solids buildup and odours.

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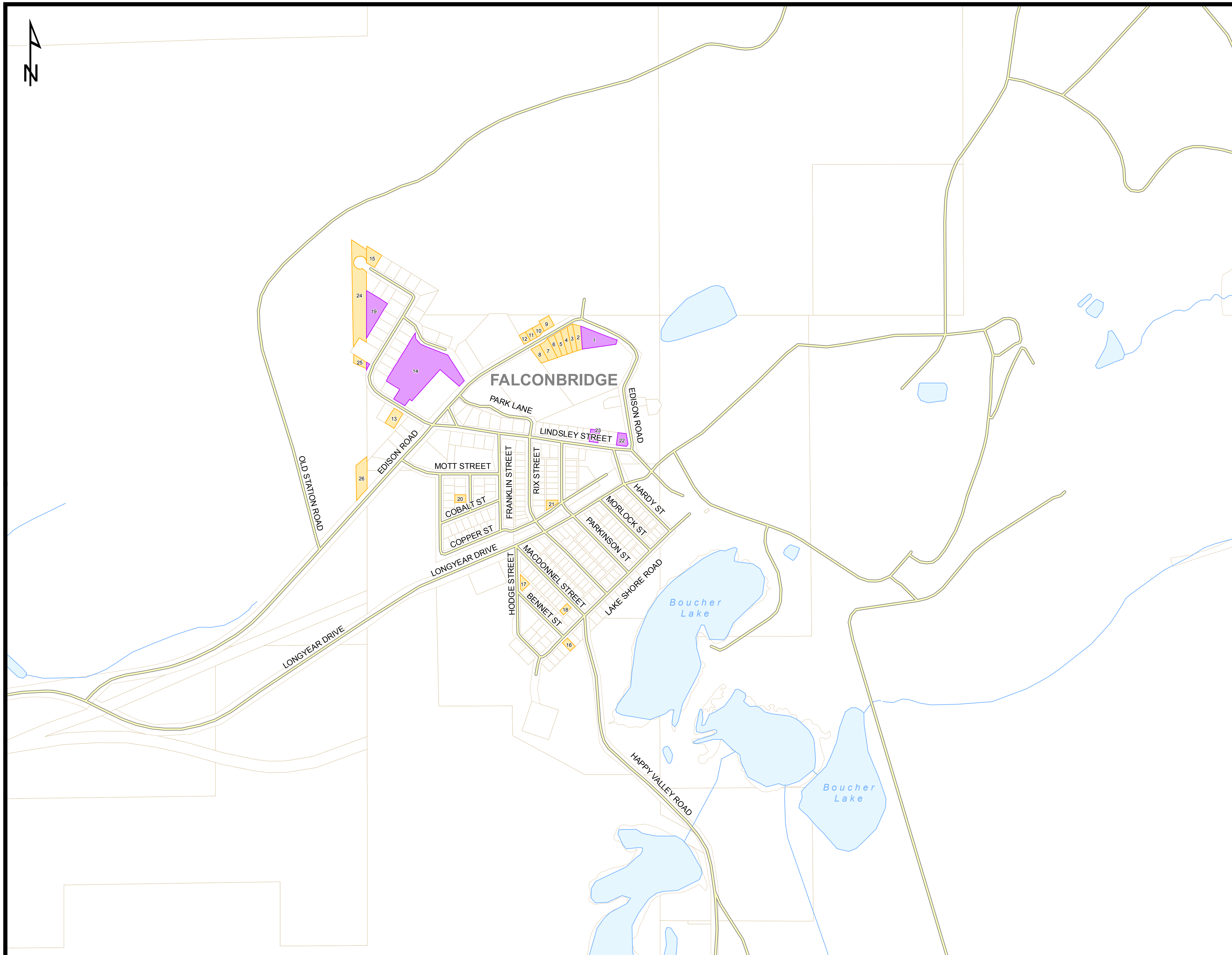
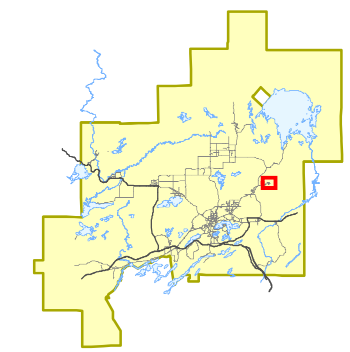
APPENDIX

A RESIDENTIAL AND ICI DEVELOPMENT AREAS



Legend

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



Project No. 121-23026-00

Sudbury Water and Wastewater Master Plan

0 100 200 300 400 500 m

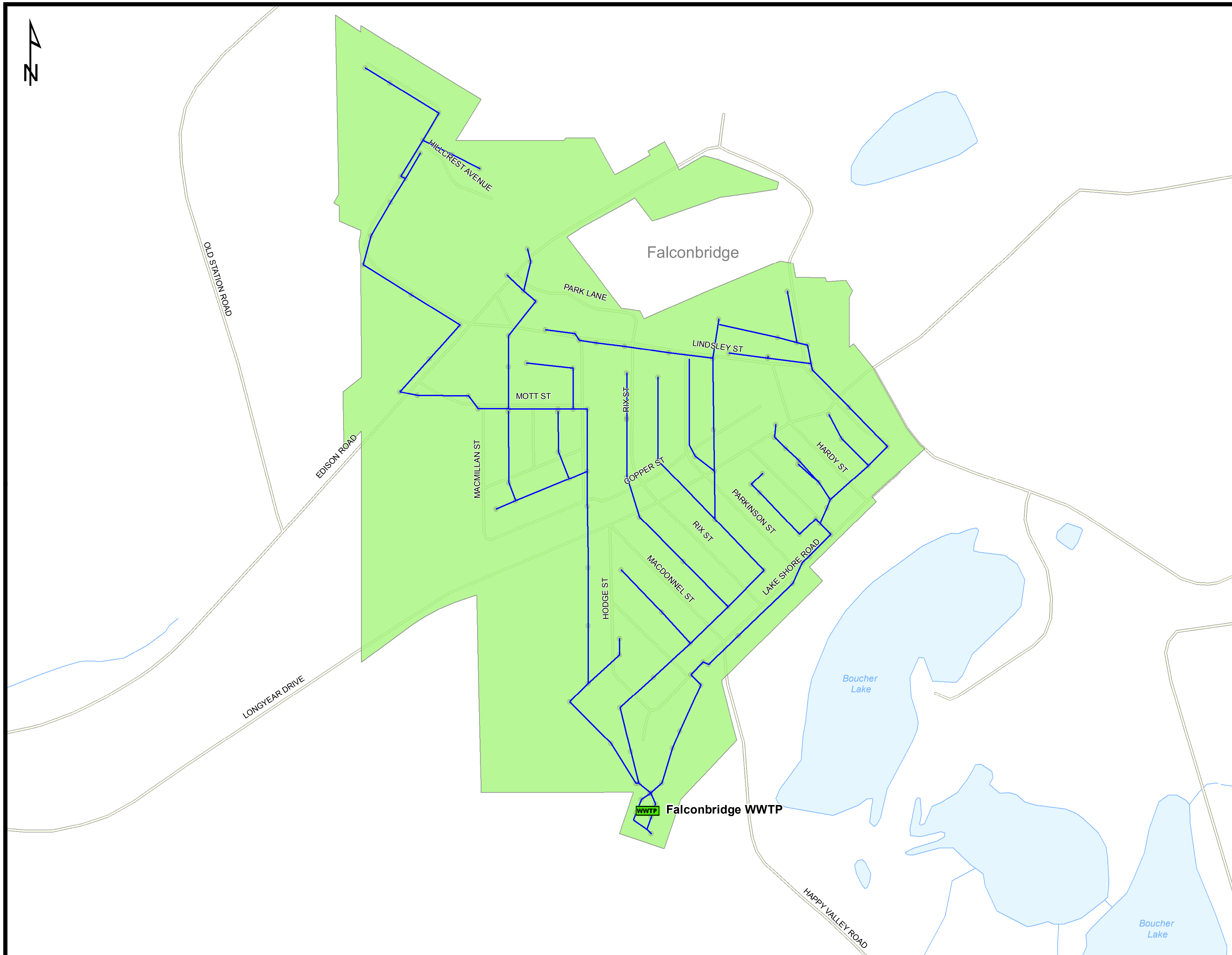
1:10,000

Map 8

Falconbridge
2011 - Ultimate Buildout
Residential and ICI
Development Areas

APPENDIX

B WASTEWATER MODEL RESULTS



Legend

- LS Lift Station
- WWTP Sanitary Treatment Plant
- Manhole
- Waterbody
- Road
- Watercourse
- Sanitary Sewer Conduit
- Forcemain

Project No. 121-23026-00

December 09 2016

Sudbury Water and Wastewater Master Plan

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Map 1

Falconbridge Wastewater System - Sewershed Map

Legend

- LS Lift Station
- WWTP Sanitary Treatment Plant
- Manhole
- Waterbody
- Road
- Watercourse
- Forcemain

Conduit: Diameter (mm)

- <= 150
- <= 200
- <= 300
- <= 450
- <= 600
- <= 750
- > 750

Greater Sudbury Grand | WSP

Project No. 121-23026-00

December 09 2016

Sudbury Water and Wastewater Master Plan

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Map 2

Falconbridge Wastewater System - Conduit Diameter

Legend

- LS Lift Station
- WWTP Sanitary Treatment Plant
- Manhole
- Waterbody
- Road
- Watercourse
- Forcemain

Conduit: Flow/Capacity (Design) (%)

- <= 50
- <= 80
- <= 100
- > 100

Greater Grand Sudbury | WSP

Project No. 121-23026-00

October 04 2017

Sudbury Water and Wastewater Master Plan

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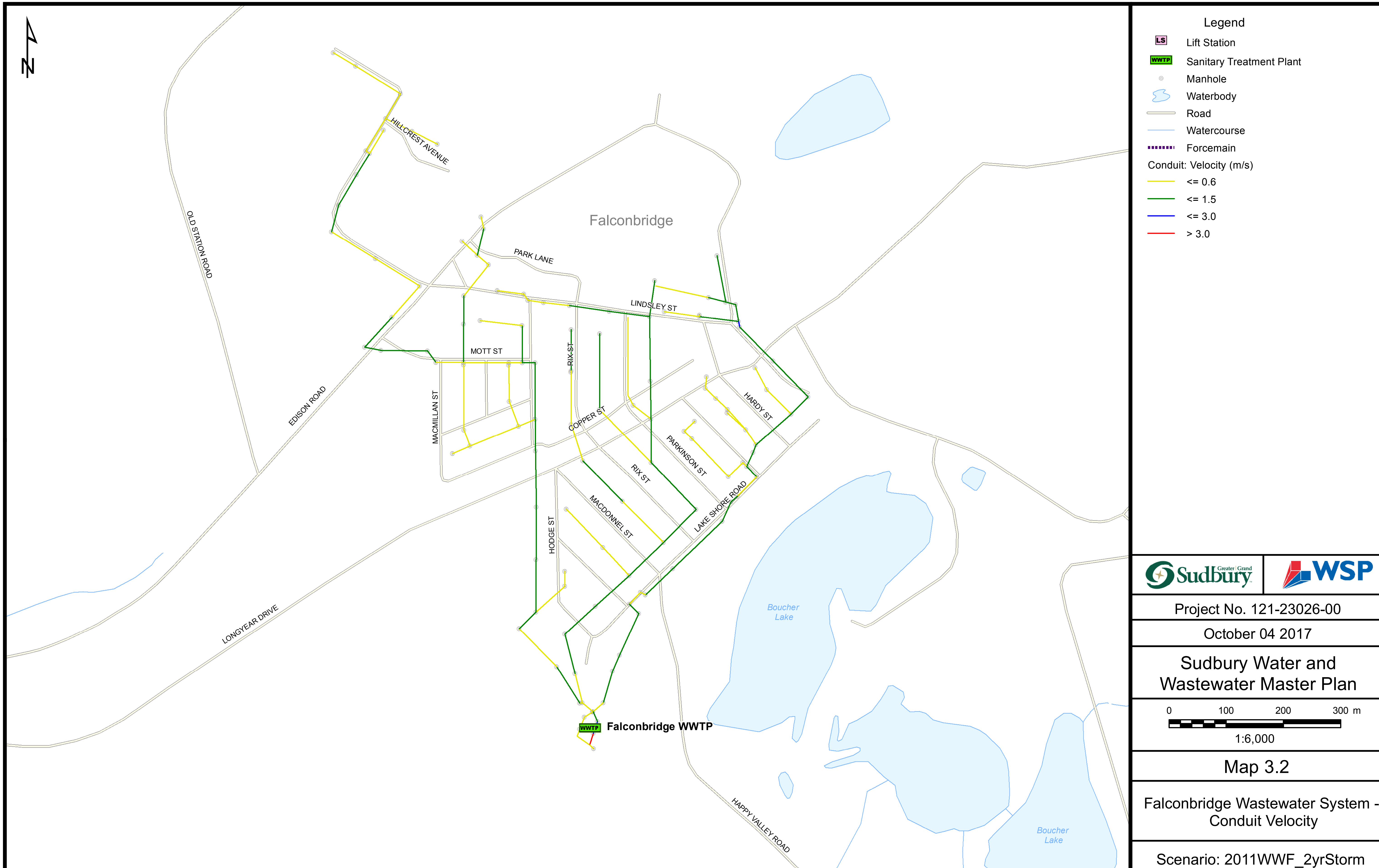
Map 3.1

Falconbridge Wastewater System - Conduit Flow/Capacity (Design)

Scenario: 2011WWF_2yrStorm



Greater Sudbury | WSP

Project No. 121-23026-00

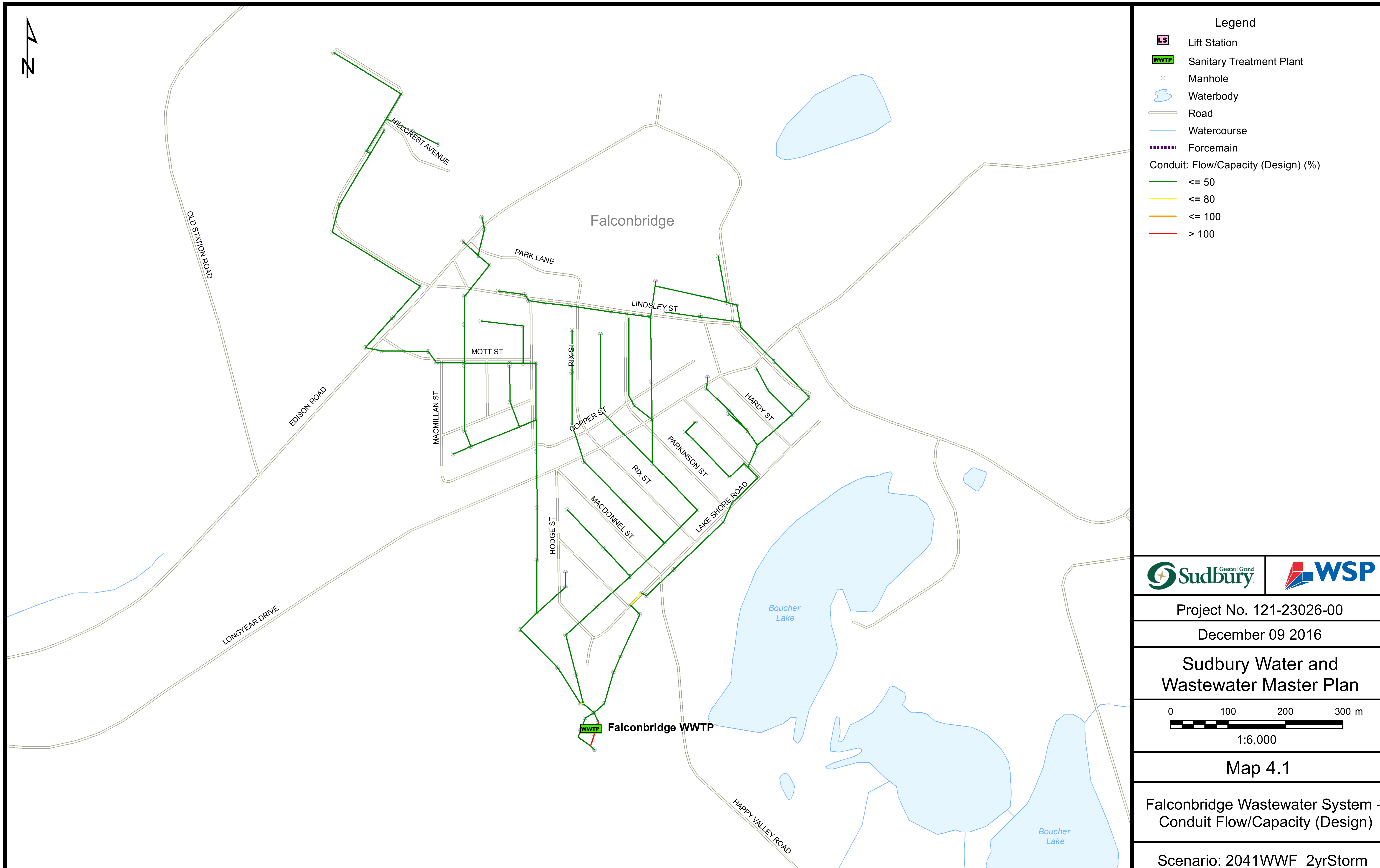
October 04 2017

Sudbury Water and Wastewater Master Plan

Map 3.2

Falconbridge Wastewater System - Conduit Velocity

Scenario: 2011WWF_2yrStorm



Project No. 121-23026-00

December 09 2016

Sudbury Water and Wastewater Master Plan

Map 4.1

Falconbridge Wastewater System - Conduit Flow/Capacity (Design)

Scenario: 2041WWF_2yrStorm

Project No. 121-23026-00

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Sudbury Water and Wastewater Master Plan

Map 4.2

Falconbridge Wastewater System - Conduit Velocity

Scenario: 2041WWF_2yrStorm

Legend

- LS Lift Station
- WWTP Sanitary Treatment Plant
- Manhole
- Waterbody
- Road
- Watercourse
- Forcemain

Conduit: Flow/Capacity (Design) (%)

- <= 50
- <= 80
- <= 100
- > 100

Greater Grand Sudbury | WSP

Project No. 121-23026-00

December 09 2016

Sudbury Water and Wastewater Master Plan

0 100 200 300 m

1:6,000

Map 5.1

Falconbridge Wastewater System - Conduit Flow/Capacity (Design)

Scenario: ULTWWF_2yrStorm

Legend

- Lift Station
- Sanitary Treatment Plant
- Manhole
- Waterbody
- Road
- Watercourse
- Forcemain

Conduit: Velocity (m/s)

- <= 0.6
- <= 1.5
- <= 3.0
- > 3.0

Greater Grand Sudbury

WSP

Project No. 121-23026-00

December 09 2016

Sudbury Water and Wastewater Master Plan

0 100 200 300 m

1:6,000

Map 5.2

Falconbridge Wastewater System - Conduit Velocity

Scenario: ULTWWF_2yrStorm

APPENDIX

C WASTEWATER GENERATION AND CAPACITY ASSESSMENTS

Falconbridge - Wastewater Flow Forecasts

	Units	2009	2010	2011	2012	2013	Summary	Design Criterion	Comments
Average Day Flow	(m^3/d)	250	240	264	242	245	248	248	From Annual Reports
Max Day Flow	(m^3/d)	342	558	629	765	497	558	558	From Annual Reports
Max Day Factor		1.37	2.33	2.38	3.16	2.03	2.25	2.25	Calculated - Max Day Flow divided by Average Day Flow
Peak Flow	(m^3/d)	Not Available						0	Peak hour flows were not available
Peak Flow	(L/s)							0	
Peak Flow Factor									
Population (Existing Areas)		707	707	707	707	707	707	707	
Ratio of Residential to Total Customers		0.19	0.19	0.19	0.19	0.19	0.19		This ratio is based on Water Billing Records for the area and is an approximation of the residential portion of demand.
Residential Share of Average Day Demand (m^3/d)		47	45	50	45	46	47		
Residential Flow Unit Rate (m^3/cap/d)		0.066	0.064	0.070	0.064	0.065	0.066	0.400	
Average Institutional Flow Unit Rate (m^3/ha/d)								28.0	CGS design standards do not suggest average rates for institutional and commercial, but require that rates be developed based on usage. In this case, water demands rates were used and assume a conservative 1:1 ratio of water to sewage.
Average Commercial Flow Unit Rate (m^3/ha/d)								28.0	CGS design standards do not suggest average rates for institutional and commercial, but require that rates be developed based on usage. In this case, water demands rates were used and assume a conservative 1:1 ratio of water to sewage.
Average Industrial Flow Unit Rate (m^3/ha/d)								35.0	CGS design standards average rate for light industrial of 35 m^3/ha/d.
Average Extraneous Flow Unit Rate (m^3/ha/d)								11.23	From CGS Design Standards, peak rate for new developments divided by an assumed peaking factor of 3. This factor would be applied only to new developments, which are assumed to be leak-tight, and have minimal extraneous flow.

	2016	2021	2026	2031	2036	2041	Ultimate Buildout	Comments
Population (Existing Areas)	707	707	707	707	707	707	707	
Population (Growth Areas)	17	35	52	62	68	69	148	
Total Population	724	743	759	769	775	776	855	Total Population (Hemson)
Residential (ha)	1.24	2.20	2.64	2.64	2.64	2.64	2.76	ICI development areas were assigned to planning years based on the stage of the application. Draft Approved were assigned to 2016, Legal Lots of Record to 2026, and Designated Developable to 2036. Areas are cumulative and carry from the development year, all the way through to Ultimate Buildout
Institutional (ha)			1.70	1.70	2.03	2.03	2.03	
Commercial (ha)			0.17	0.17	0.17	0.17	0.17	
Industrial (ha)			0.36	0.36	0.36	0.36	0.36	
ICI (ha)	0.00	0.00	2.23	2.23	2.56	2.56	2.56	
Total (ha)	1.24	2.20	4.87	4.87	5.20	5.20	5.32	

	2016	2021	2026	2031	2036	2041	Ultimate Buildout	Comments
Average Residential Flows (m^3/d) - Existing	248	248	248	248	248	248	248	This includes all contribution from existing ICI and infiltration. The base flow was assumed to be the average day flow to the plant for the 2011-2013 period.
Average Residential Flows (m^3/d)	7	14	21	25	27	27	59	Obtained by multiplying the projected population growth by the unit rate.
Average Institutional Flow (m^3/d)	0	0	48	48	57	57	57	Institutional growth area multiplied by unit flow rate.
Average Commercial Flow (m^3/d)	0	0	5	5	5	5	5	Commercial growth area multiplied by unit flow rate.
Average Industrial Flow (m^3/d)	0	0	13	13	13	13	13	Industrial growth area multiplied by unit flow rate.
Average Extraneous Flow (m^3/day)	14	25	55	55	58	58	60	
Average Day Flow (m^3/d)	269	287	389	393	408	408	441	
Max Day Flow (m^3/d)	606	647	876	885	919	920	994	

ALTERNATIVE CALCULATION METHOD

Per Capita Flow (m3/cap/day)	0.354	0.339	0.373	0.342	0.346	0.351	0.351

	2016	2021	2026	2031	2036	2041	Ultimate Buildout	Comments
Average Day Flow (m^3/d)	254	261	267	270	272	272	300	Multiplying the total population by the consolidated per capita flow factor.
Max Day Flow (m^3/d)	573	587	601	608	613	614	676	

SENSITIVITY ANALYSIS

Analyze sensitivity of forecasted flows to unit rate

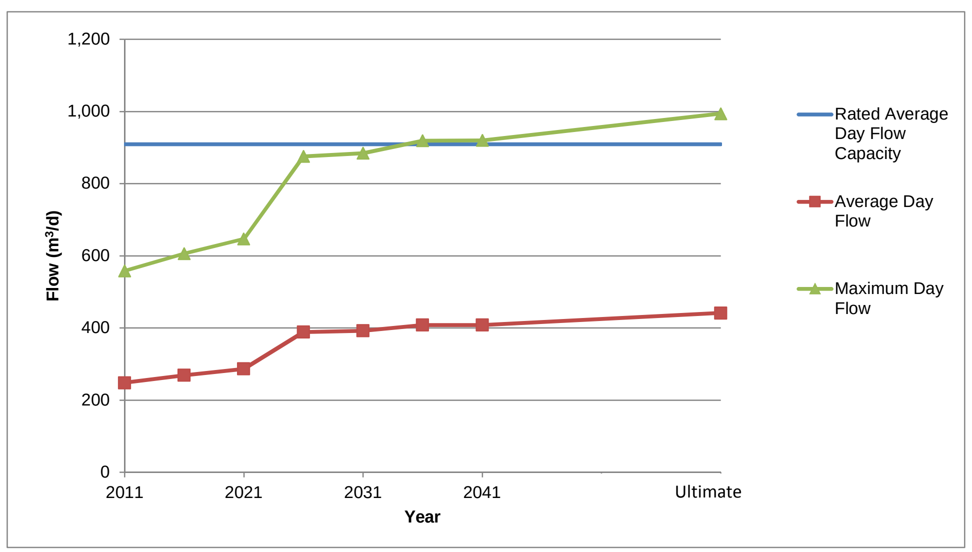
	Unit Rate (m^3/cap/d)	Average Day Flow (m^3/d) 2016	2021	2026	2031	2036	2041	Ultimate Buildout 2061
Base Scenario - Residential Historical Maximum	0.066	263	275	371	372	385	385	392
Combined Historical Maximum	0.351	268	285	386	390	405	405	434
City Standards	0.410	269	287	389	393	409	409	443
Base Scenario - Average of MOE Guidelines Typical Range	0.400	269	287	389	393	408	408	441

Analyze sensitivity of forecasted flows to max day factor

	Max Day Peaking Factor	Max Day Flow (m^3/d) 2016	2021	2026	2031	2036	2041	Ultimate Buildout 2061
Base Scenario - Historical Max	2.47	665	710	962	972	1,010	1,010	1,092
Historical Average	3.16	606	647	876	885	919	920	994

CAPACITY CHECK

	2011	2016	2021	2026	2031	2036	2041	Ultimate Buildout 2061	
Rated WPCP ADF Capacity (m^3/d)	909	909	909	909	909	909	909	909	From C of A (1979), assuming imperial gallons
Average Day Flow (m^3/d)	248	269	287	389	393	408	408	441	
Maximum Day Flow (m^3/d)	558	606	647	876	885	919	920	994	



CITY OF GREATER SUDBURY WATER AND WASTEWATER MASTER PLAN

LEVACK WASTEWATER SYSTEM GAP ANALYSIS AND STATUS QUO REPORT

CITY OF GREATER SUDBURY

DRAFT

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

WSP
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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 existing Levack Wastewater System. Based on population growth projections and design criteria discussed in the *Population Projections and Development of Unit Rates Technical Memorandum* (WSP, 2014), 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 Levack Wastewater System would continue to be a stand-alone system. Any potential interconnections between Levack 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. Additional information on the existing wastewater system is provided in the Baseline Review Report for Wastewater Systems (WSP, 2014).

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

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 wastewater system.

Mapping 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 *Population Projections and Development of Unit Rates Technical Memorandum* (WSP, 2014).

Existing development in the study area is mixed, and includes some residential development and a considerable amount of industrial development.

Based on the City's planning data, significant growth is projected in Levack and Onaping. The area population is planned to increase from 2,112 in 2011 to 2,159 in 2041 and 2,477 by Ultimate Buildout.

ICI growth is expected to be primarily industrial with small amounts of commercial, institutional and industrial development. Growth is discussed further in Section 6.1.

3 OVERVIEW OF EXISTING SYSTEM

All wastewater flows generated in Levack and Onaping is collected and treated at the Levack Wastewater Treatment Plant (WWTP). The plant is located at 45 High Street within the City of Greater Sudbury and operates under amended C of A number 6279-5KKLQA. The Levack Wastewater System includes the Levack WWTP, the Fraser Lift Station and numerous gravity sewers.

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

The Levack Wastewater System is shown in **Appendix B**.

3.1 LIFT STATIONS

The Levack Wastewater System includes one lift station: the Fraser Lift Station. Forcemain sizes and other details for this lift station are summarized in the Baseline Review Report for Wastewater Systems (WSP, 2014).

Table 3-1 provides a summary of the main features of the lift station.

Table 3-1 Levack System Lift Station

LIFT STATION	YEAR CONSTRUCTED[1]	LAST UPGRADED	WET WELL VOLUME (M^3)	PUMPING STATION CAPACITY AND FORCEMAIN INFORMATION[1]
Fraser	1993	N/A	N/A	Two dry well pumps with a firm design capacity of 27 L/s 1737m long, 200 mm diameter forcemain of unknown material

[1]From 2013 Operations Manual. The firm capacity is the draw down test results for one pump.

3.2 LEVACK WWTP

The Levack WWTP is a twin-celled extended aeration plant. The plant has a rated capacity of 2,270 m^3/d, and a peak capacity of 5,670 m^3/d. A process flow schematic of the liquid treatment train at the Levack WWTP is presented in Figure 3-1.

The treatment process is illustrated schematically in Figure 2-2.

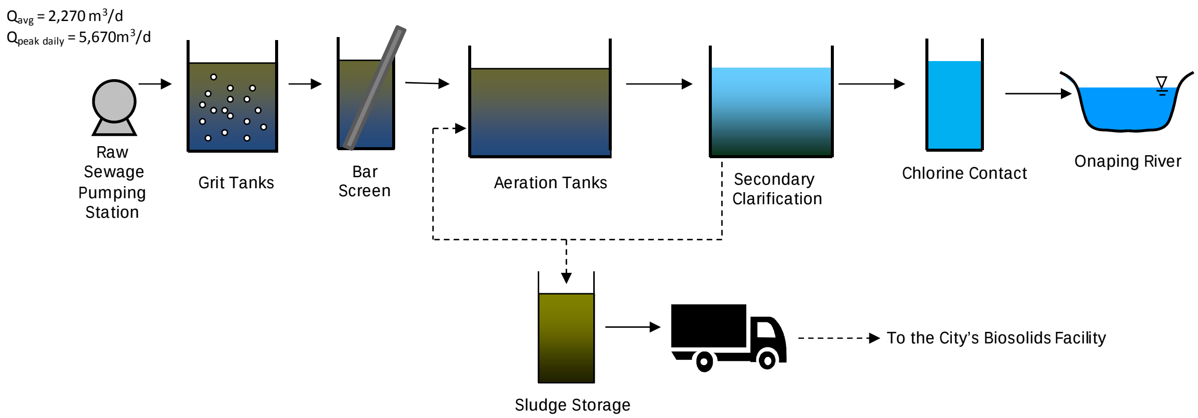

Figure 3-1 Levack WWTP Process

3.3 KNOWN CHALLENGES

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

- The trailer park adjacent to Mountain Avenue in Levack was constructed over top of the existing wastewater linear infrastructure in the area.
- Most of the sewers in Levack pass through the backyards of residential lots.
- In Onaping, the 300 mm diameter trunk main that flows to the Fraser LS is also not located within a road right of way and is therefore not easily accessible.
- The City has noted that although inflow and infiltration does not appear to be of concern in this system, many roof drains are connected to the sanitary network.

4 HISTORICAL FLOWS AND REVIEW OF OPERATIONAL DATA

Data reported in the 2009 to 2013 *Annual Reports* for the Levack WWTP was reviewed and analyzed to determine average day and maximum day flows as well as review effluent parameters.

4.1 FLOW DATA

WWTP flow data from 2009 to 2013 was reviewed. Operational data was not available from the lift stations and so historical peak flow data could not be estimated.

The recorded average day and maximum day flows are summarized in Table 4-1 and plotted in the figure below.

Table 4-1 Historical Wastewater Flow Data

YEAR	AVERAGE DAY FLOW (M^3/D)[1]	MAXIMUM DAY FLOW (M^3/D)[1]
2009	862	2,353
2010	765	3,251
2011	799	1,901
2012	615	1,828
2013	786	2,879

[1] *Annual Reports (2009 - 2013)*

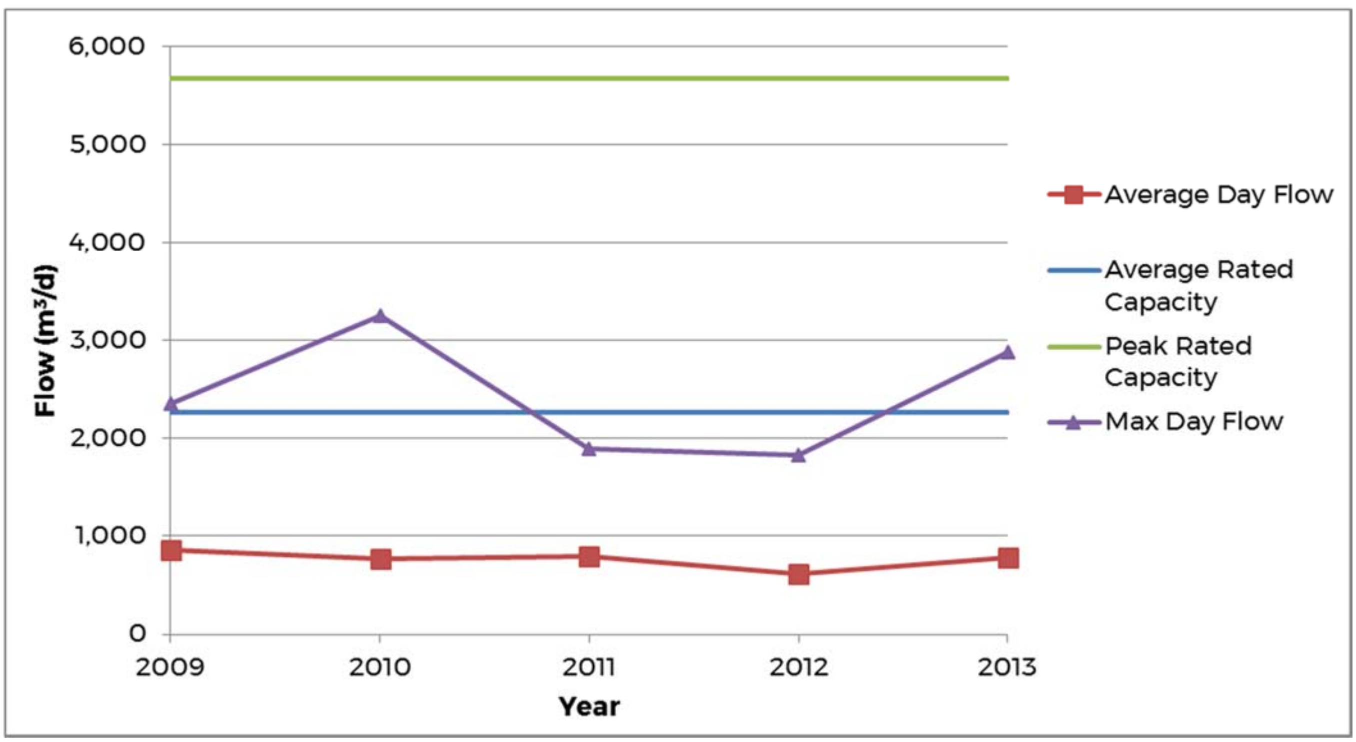

Figure 4-1 Historical Wastewater Flows at the Levack Wastewater Treatment Plant

The relationship between the different flow regimes was analyzed to compare the maximum day peaking factors derived from historical data to those used in the *City's Engineering Design Manual* and those included in the *MOECC Guidelines.*

The average day flows to the WWTP have been relatively steady over the 2009 to 2013 period, averaging 765 m^3/d. Although the historical maximum day flows fluctuate from year to year, there is no discernible trend when all flow data is considered.

The highest maximum day to average day peaking factor was 4.25 (2010). The average maximum day peaking factor from 2009 to 2013 was 3.20. The City's *Engineering Design Manual* and the *MOECC Guidelines* do not specify recommended maximum day factors and recommend using historical data when available. For future wastewater generation, the average peaking factor (3.20) was used and based on the assumption that new developments would have less I&I due to more leak tight construction.

4.2 RAW WASTEWATER CHARACTERISTICS

The average raw wastewater characteristics from 2009 to 2012 are summarized in Table 4-2 below. Raw wastewater temperatures were not reported.

Table 4-2 Average Raw Wastewater Characteristics at the Levack WWTP (2009-2012)

PARAMETER	AVERAGE VALUE
$CBOD_5$	127.8 mg/L
Suspended Solids	134.8 mg/L
Total Phosphorus	4.2 mg/L
TKN	25.3 mg/L
pH	7.0

4.3 EFFLUENT CRITERIA

The Levack WWTP is operated in accordance with the MOECC Certificate of Approval (C of A) No. 6279-5KKLQA dated September 22, 2003.

The *C of A* concentration and loading limits are summarized in Table 4-3.

Table 4-3 Levack WWTP Effluent Limits and Objectives

EFFLUENT PARAMETER	CONCENTRATION LIMIT	LOADING LIMIT	CONCENTRATION OBJECTIVE
$CBOD_5$	25 mg/L	56.75 kg/d	15 mg/
Total Suspended Solids (TSS)	25 mg/L	56.75 kg/d	15 mg/L
Total Phosphorus (TP)	1 mg/L	2.27 kg/d	0.5 mg/L
pH	N/A	N/A	6.0-9.5
E. coli	200 cfu / 100 mL	N/A	N/A

4.4 OPERATIONAL DATA

The general plant operation was reviewed against the Levack WWTP ECA requirements and historical data provided in the Annual Reports from 2009 to 2012. Historical data is summarized in the table below.

Table 4-4 Historical Effluent Concentrations

EFFLUENT PARAMETER	ANNUAL AVERAGE			
	2009	2010	2011	2012
$CBOD_5$ (mg/L)	2.9	1.9	2.2	2.0
TSS (mg/L)	9.9	7.8	7.3	7.1
TP (mg/L)	0.39 (all months comply)	0.35 (all months comply)	0.36 (all months comply)	0.39 (all months comply)
pH	6.30	6.23	5.94	6.26
TAN (mg/L)	4.47	3.00	6.58	3.38
E. coli (organisms/100 mL)	4	14	8	3

Historically, the Levack WWTP has met all concentration limits. However, in June 2009 and October 2010, the pH dropped below the objective range to 5.4 and 5.9, respectively.

A capacity review of each unit process at the WWTP was not conducted. Instead, the rated capacity was considered the true capacity of the plant.

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 WASTEWATER DESIGN CRITERIA

The unit flow criteria for growth adopted for this assessment are shown in Table 5-1 below. These values were recommended in the *Populations and Unit Rates Technical Memorandum* (WSP, 2014).

Note that the term "extraneous flows" is used interchangeably with "I&I flows".

Table 5-1 Wastewater System Design Criteria

CRITERIA	VALUE	REFERENCE
Average Day Residential Flow	200 L/cap/day	Rounded up from average of historical values (156 L/cap/day)
Average Day Commercial and Institutional Flow	28 m^3/ha/d	Water unit rates, assuming a 1:1 correspondence
Average Day Industrial Flow	35 m^3/ha/d	Water unit rates, assuming a 1:1 correspondence
Average Extraneous Flow	7.48 m^3/ha/d	Peak from City's Engineering Design Manual and assuming a peaking factor of three
Peak Extraneous Flow	22.5 m^3/ha/d	City's Engineering Design Manual
Max Day Peaking Factor	3.20	Average of historical values

Residential average day flows were obtained by multiplying the residential unit rate by the service population.

Maximum day flows to the WWTP are obtained by multiplying the average day flow by the maximum day peaking factor.

5.2 DESIGN CRITERIA FOR WASTEWATER SYSTEM COMPONENTS AND OPERATION

5.2.1 WASTEWATER TREATMENT

Wastewater treatment facilities are rated for average day flows. Plant effluent limits and objectives are established in the C of A or ECA for each facility.

5.2.2 LIFT STATION PUMPING CAPACITY

The firm capacity of the lift station (with the largest pump out of service) must allow pumping of peak wet weather flows corresponding to its catchment area (MOECC, 2008).

Starting limitations on pump motors generally dictate the minimum size of a wet well. The wet well should be large enough to prevent pump motors from overheating due to frequent starting and stopping, but small enough to avoid long retention times leading to septicity and odor problems (Lin & Lee, 2001).

The station wet well shall be sized such that the number of pump starts per hour does not exceed the maximum value recommended by the pump manufacturer. In other words, the time between pump starts and stops (i.e. the pump cycle time) should not exceed that which results in a pump start frequency greater than that recommended by the pump manufacturer. Typically, submersible pumps can cycle four to 10 times per hour with a maximum cycle time not exceeding 30 minutes (Lin & Lee, 2001). A maximum value of four pump starts per hour was assumed to evaluate wet well sizing requirements.

5.2.3 SEWERS

The sewer system is typically sized to convey peak instantaneous (peak wet weather) flows. Sewage flows are made up of wastewater discharges from residential, commercial, institutional and industrial establishments, plus extraneous flow components from such sources as groundwater and surface runoff.

In addition to being able to convey peak flows, sufficient flow velocity should be maintained to transport the sewage solids to avoid deposition and the development of nuisance conditions under lower flow conditions. The minimum acceptable flow velocity in sewers is 0.6 m/s (City of Greater Sudbury, 2012).

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 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 *Population Projections 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 Levack population with wastewater servicing is projected to increase by 47 people by 2041 and 365 by Ultimate Buildout.

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

Table 6-1 Levack-Onaping Population Projections

SYSTEM	2011	2016	2021	2026	2031	2036	2041	ULTIMATE BUILDOUT
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 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 the table below.

Table 6-2 Levack and Onaping Falls ICI Projections

	ICI DEVELOPMENT AREAS (HA)						
LAND USE	2016	2021	2026	2031	2036	2041	BUILDOUT
Institutional	0	0	0	0	2.78	0	0
Commercial	0	0	0.71	0	0	0	0
Industrial	0	0	0	0	1.86	0	0
Total	**0**	**0**	**0.71**	**0**	**4.64**	**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 PHASING OF FUTURE GROWTH

Growth areas were allocated based on population projections for individual developments and the overall target growth population projections for Levack 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 WASTEWATER FLOW PROJECTIONS AND INFRASTRUCTURE NEEDS

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

$$\textbf{Projected Average Day Generation} \\ = \textbf{Base Generation} + \textbf{Additional Residential Generation} + \textbf{Additional ICI Generation} \\ + \textbf{Average Extraneous Flow}$$

The flows corresponding to the population growth forecasts to Ultimate Buildout are presented in Table 6-3 below.

Table 6-3 Flow Projections

YEAR	POPULATION	AVERAGE DAY FLOW (M^3/D)	MAXIMUM DAY FLOW (M^3/D)
Base	2,112	765	2,442
2016	2,123	777	2,485
2021	2,135	780	2,494
2026	2,146	813	2,600
2031	2,154	815	2,607
2036	2,159	1,028	3,290
2041	2,159	1,028	3,290
Ultimate	2,477	1,143	3,655

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 demand was calculated by applying the respective peaking factor to the average day demand. The maximum day demand for the base year was the historical average.

A desktop analysis of historical wastewater flows and future flow projections is included in Appendix C.

6.3.1 LEVACK WWTP CAPACITY

The rated average day capacity of the Levack WWTP is 2,270 m^3/d, and is compared to the current and future flow projections on Figure 6-1 below.

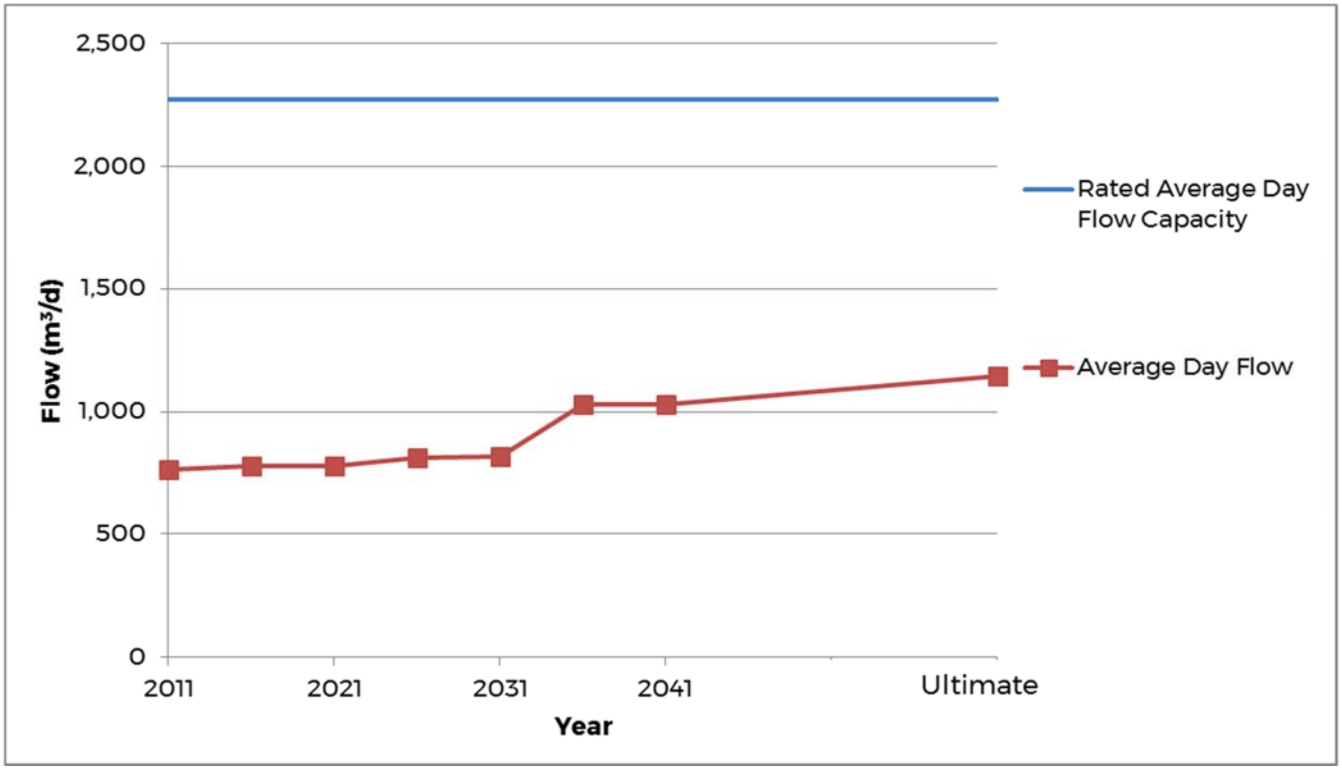

Figure 6-1 Wastewater Flow Projections Compared to Levack WWTP Rated Capacity

As indicated in the above analysis, the Levack WWTP can continue operating under its current capacity until beyond Ultimate Buildout.

6.3.2 SEWER NETWORK AND LIFT STATIONS

For each of the scenarios modeled, the system was checked for surcharging of sewers and capacity exceedance at the lift stations. The peak flows into each of the lift stations was determined from the computer simulations for the various planning scenarios and is presented in Table 6-4 below. The table also shows the design/rated flow for the pumps, their capacity based on drawdown tests and the computer simulated flow for comparison.

Table 6-4 Lift Station Peak Influent Flow Rates

	CURRENT FIRM CAPACITY	EXISTING PEAK FLOW	2041 PEAK FLOW	ULTIMATE BUILDOUT
Fraser	27	36.8	38.1	38.7

Therefore, the Fraser LS does not have enough capacity to convey flows from 2011 and beyond.

7 HYDRAULIC MODELLING

7.1 APPROACH

A basic sanitary model for the City of Greater Sudbury was received from the City. The model was created in Bentley's SewerGEMS by City staff. The model is an all pipe model of the sanitary network in these systems, but some critical information such as pipe data, invert elevations and lift station characteristics were missing. The model now includes this information as well as key vertical infrastructure in each system, including lift stations and treatment facilities.

The model was loaded with wet weather flow data. A water balance was completed to determine I&I rates for both dry and wet weather flow. The results from the water balance were compared against I&I rates developed through flow monitoring, and the greater of the two values, for each system, was used to load the model.

Current (2011) and future (2016-Ultimate Buildout, in 5 year increments) population data was added to the model using the City's planning data, summarized in previous sections of this report.

Future dry and wet weather flow scenarios were developed for each of the horizon years: 2016, 2021, 2026, 2031, 2036, 2041, and Ultimate Buildout. However, model results did not vary from 2016 to 2041; therefore, this report discusses findings for 2041 and Ultimate Buildout, compared against existing (2011).

7.2 MODELLING FINDINGS

The model was used to check sewer capacity and flow velocity. Most of the sewers flow at less than 50% of the available capacity from 2011 to Ultimate Buildout scenarios under the wet weather flow condition.

Flow velocities through many sewers in the Levack system are generally below the City's standard of 0.6 m/s. This is consistent through to Ultimate Buildout under the wet weather flow condition.

Maps in **Appendix B** illustrate the modeling results for the 2011, 2041, and Ultimate wet weather flow scenarios.

8 CONCLUSIONS

An assessment of the Levack Wastewater System was completed to identify infrastructure requirements to service forecasted growth in the community.

The conclusions of the assessment are summarized below.

— The WWTP is deemed to have sufficient average day capacity to service growth to Ultimate Buildout.

— Many of the sewers in Levack and Onaping operate at velocities that do not meet the City's current standards (that is, many sewers flow at less than 0.6 L/s). This may cause operational problems, such as solids buildup and odours.

— The Fraser LS requires upgrades to meet existing peak capacity requirements.

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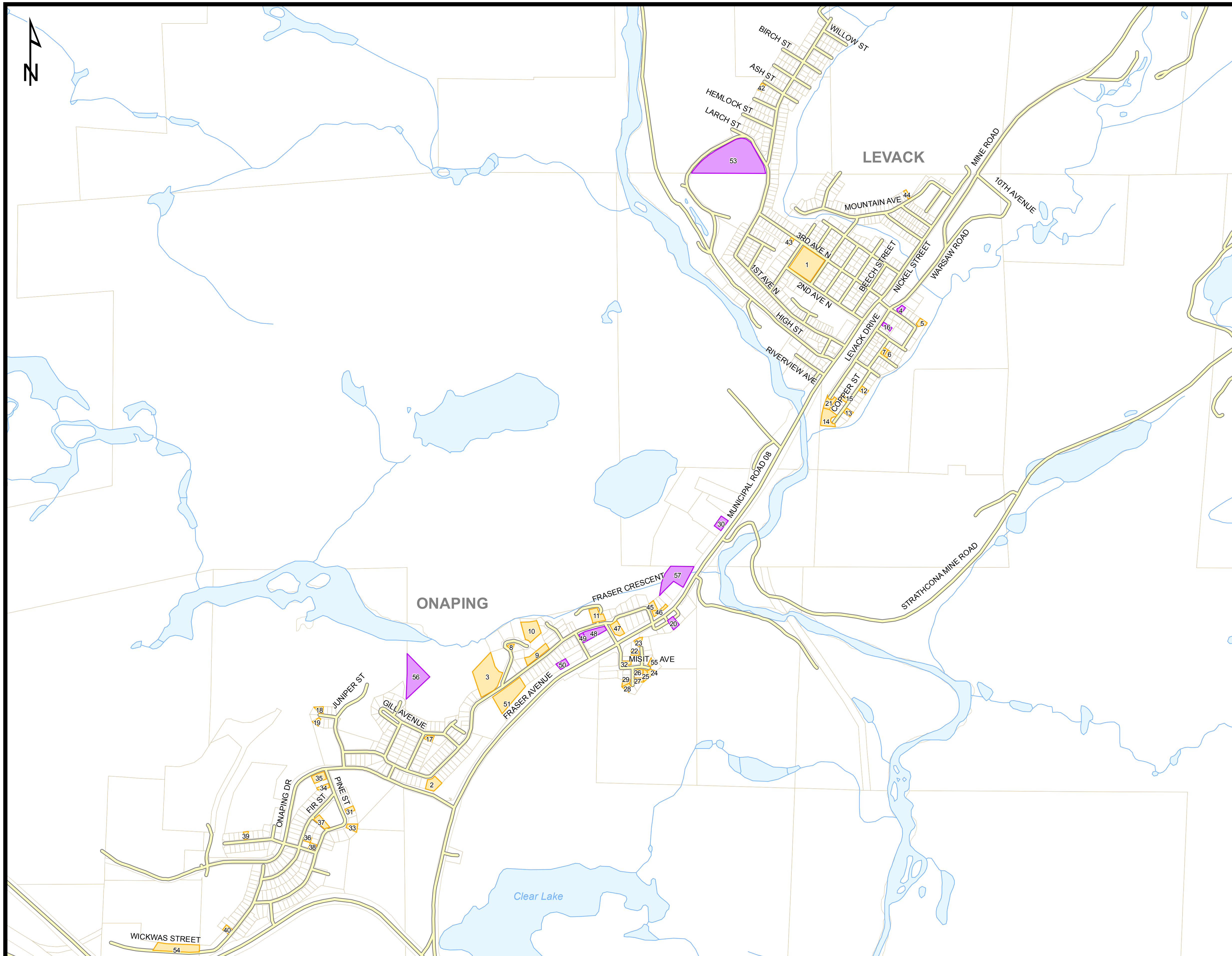
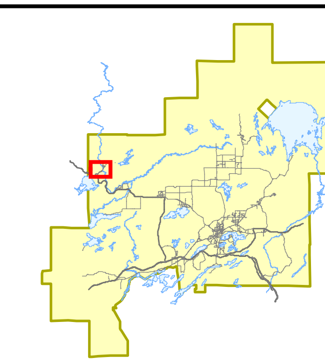
APPENDIX

A RESIDENTIAL AND ICI DEVELOPMENT AREAS



Legend

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



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WSP

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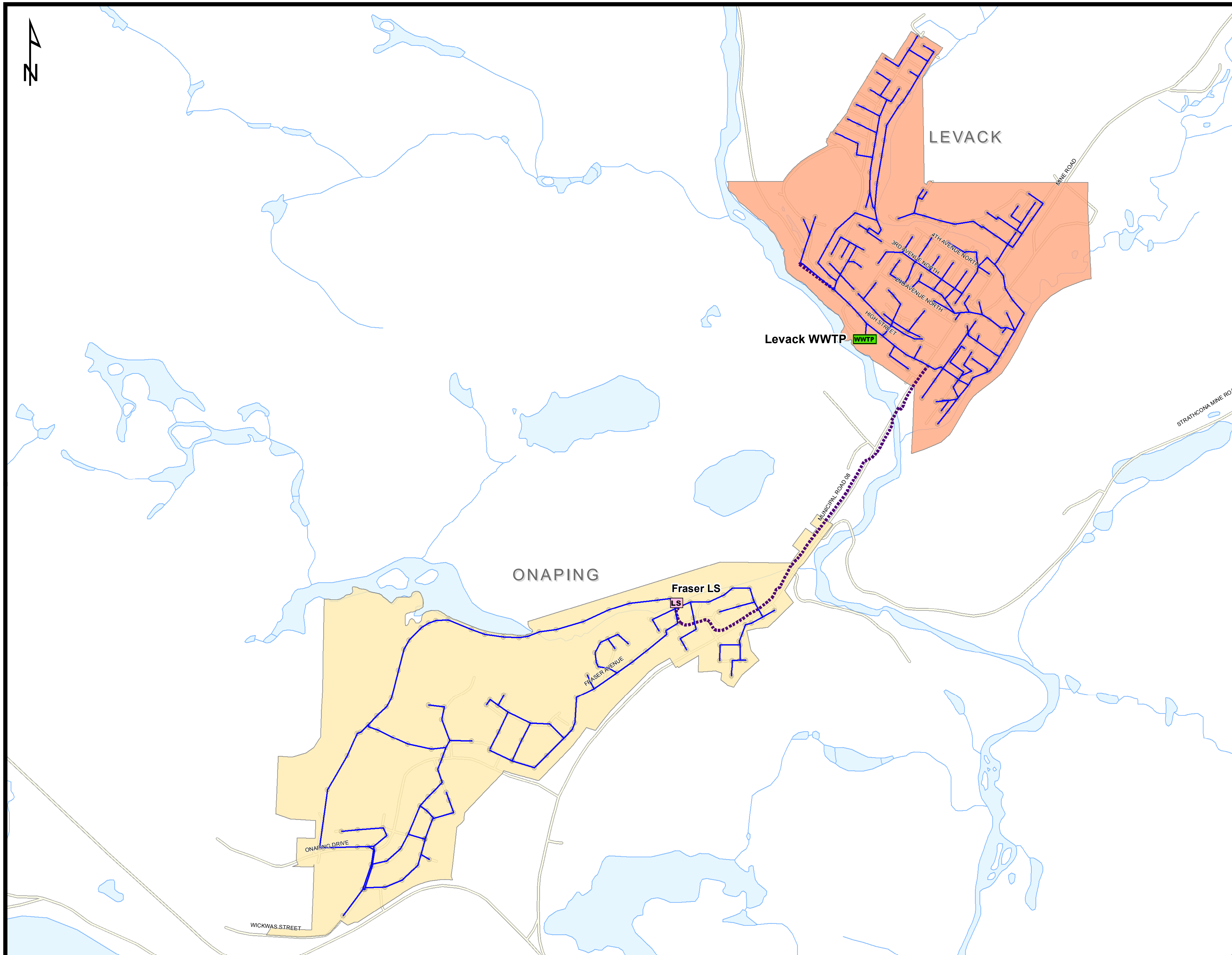
Map 10

Levack
2011 - Ultimate Buildout
Residential and ICI
Development Areas

APPENDIX

B WASTEWATER MODEL RESULTS



Legend

- LS Lift Station
- WWTP Sanitary Treatment Plant
- Manhole
- Waterbody
- Road
- Watercourse
- Sanitary Sewer Conduit
- Forcemain

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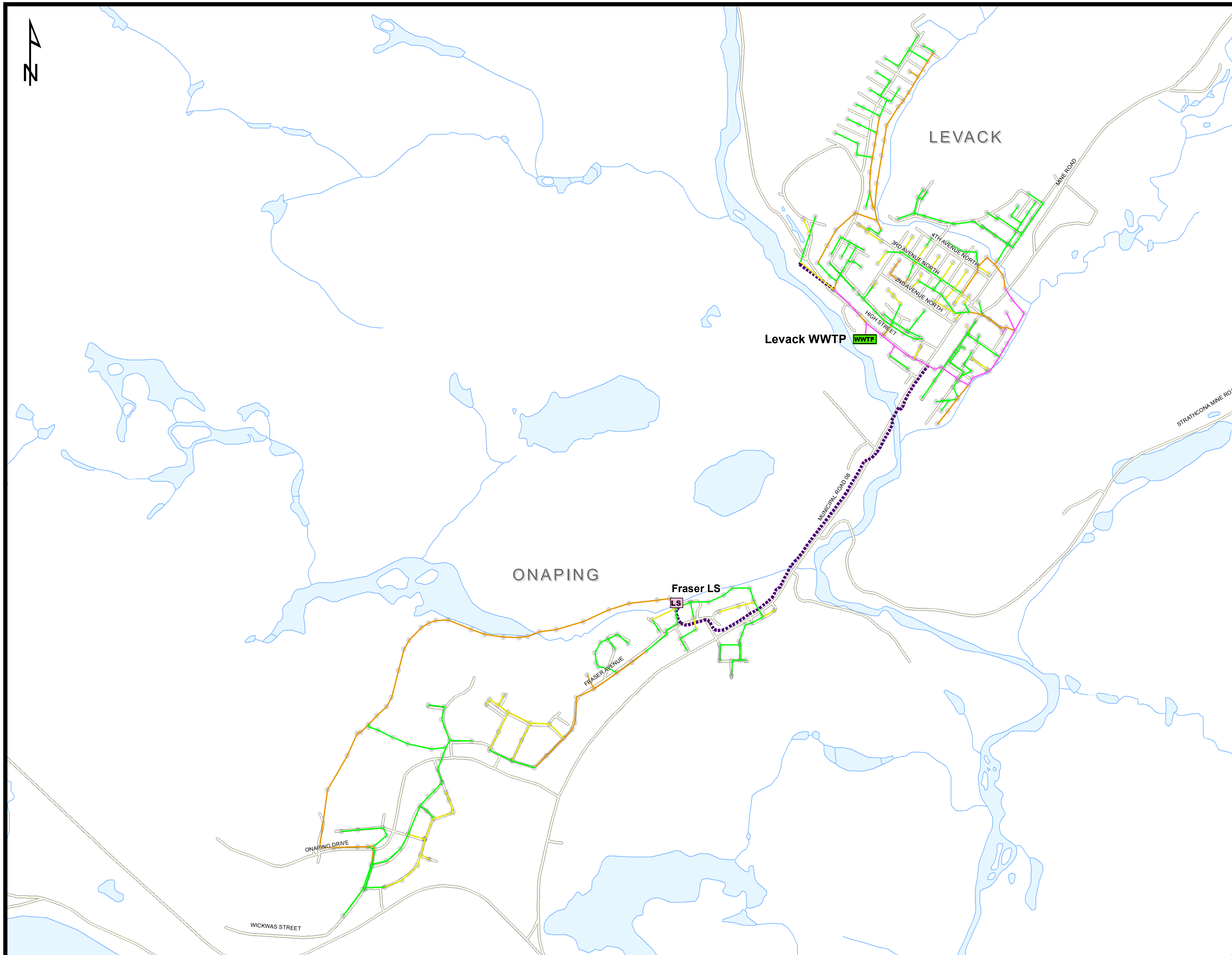
Map 1

Onaping and Levack Wastewater System - Sewershed Map



Legend

- LS Lift Station
- WWTP Sanitary Treatment Plant
- Manhole
- Waterbody
- Road
- Watercourse
- Forcemain

Conduit: Diameter (mm)

- <= 150
- <= 200
- <= 300
- <= 450
- <= 600
- <= 750
- > 750

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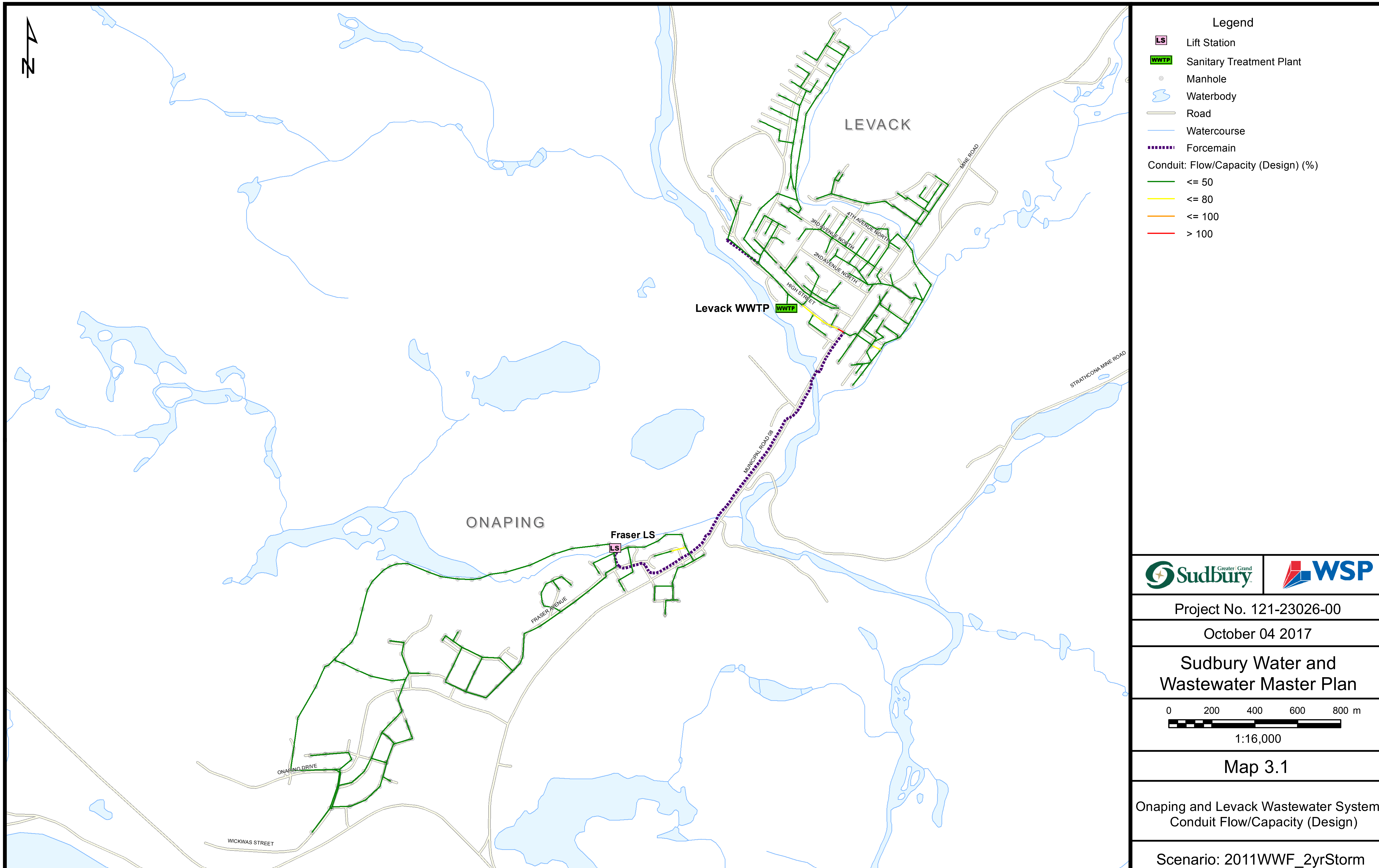
Map 2

Onaping and Levack Wastewater System - Conduit Diameter



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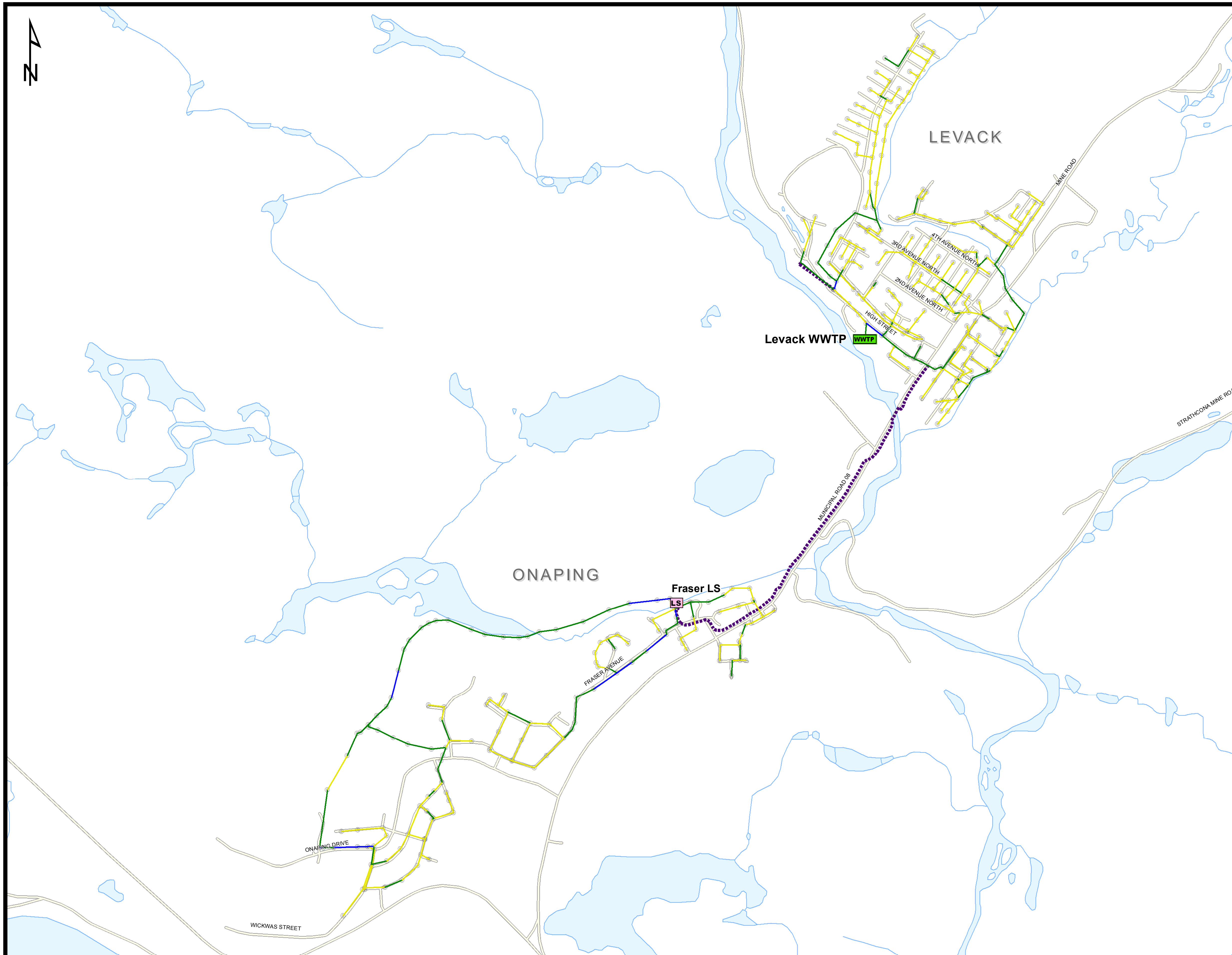
Map 3.1

Onaping and Levack Wastewater System - Conduit Flow/Capacity (Design)

Scenario: 2011WWF_2yrStorm



Legend

- LS Lift Station
- WWTP Sanitary Treatment Plant
- Manhole
- Waterbody
- Road
- Watercourse
- Forcemain

Conduit: Velocity (m/s)

- <= 0.6
- <= 1.5
- <= 3.0
- > 3.0

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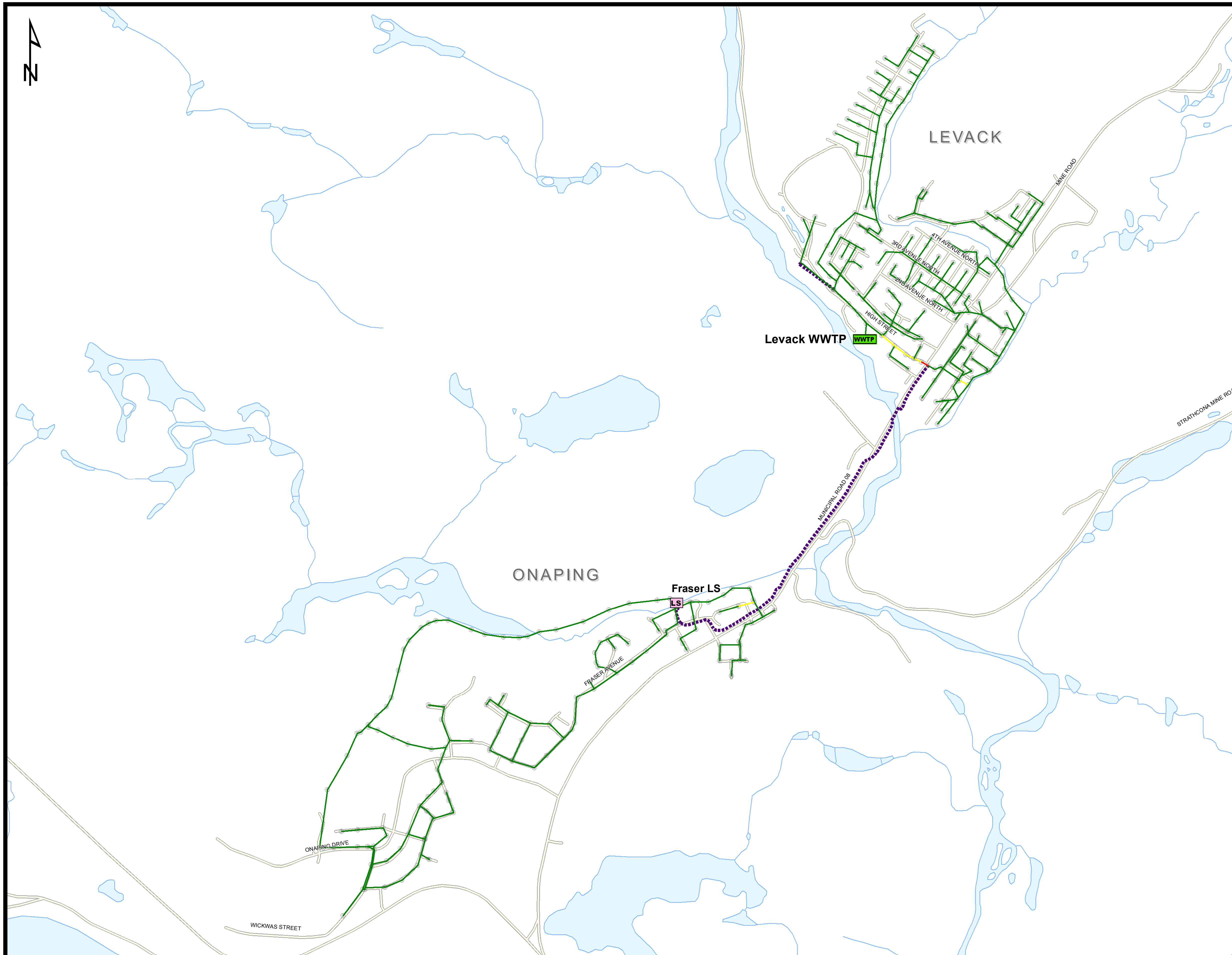
Map 3.2

Onaping and Levack Wastewater System - Conduit Velocity

Scenario: 2011WWF_2yrStorm



Legend

- LS Lift Station
- WWTP Sanitary Treatment Plant
- Manhole
- Waterbody
- Road
- Watercourse
- Forcemain

Conduit: Flow/Capacity (Design) (%)

- <= 50
- <= 80
- <= 100
- > 100

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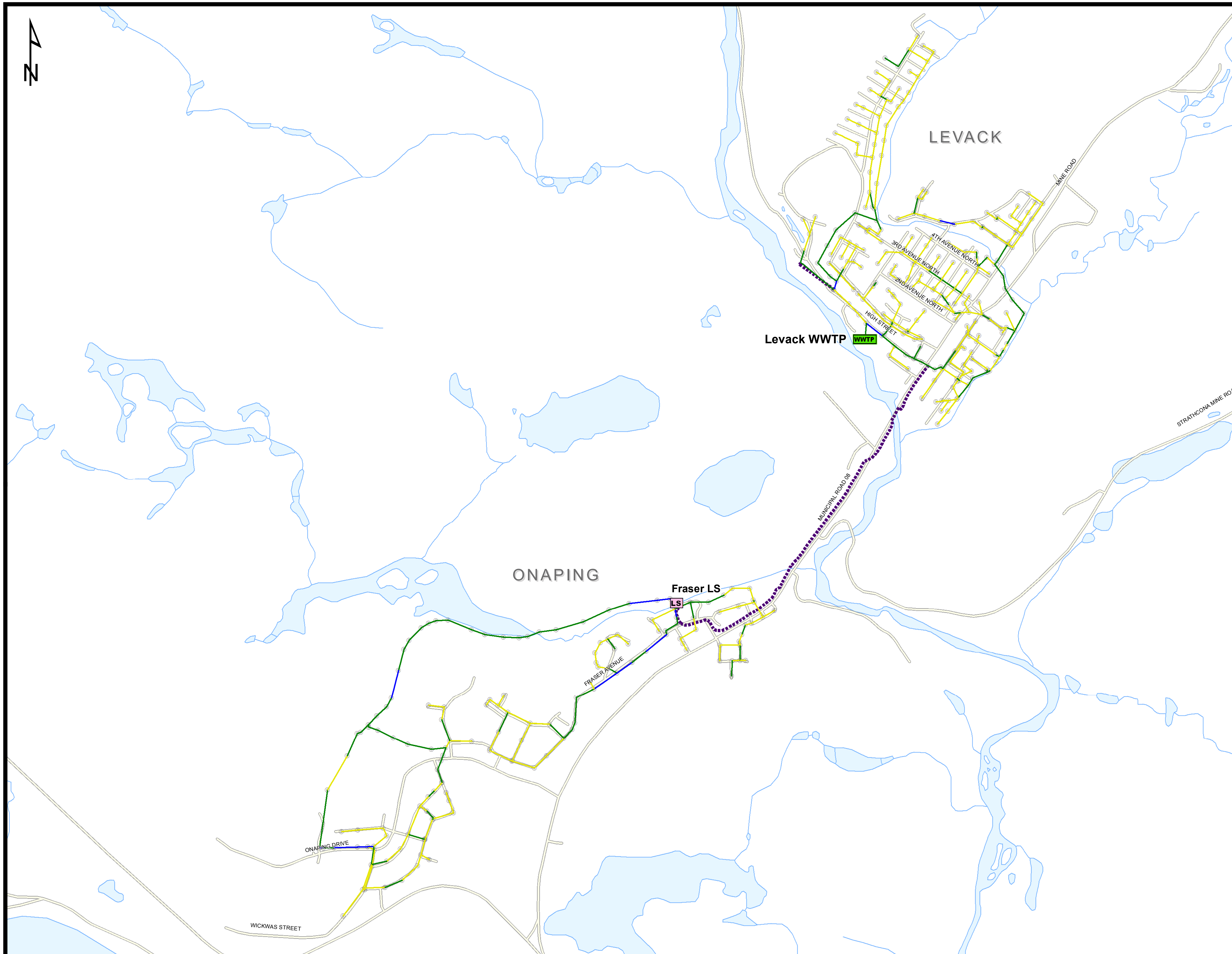
Map 4.1

Onaping and Levack Wastewater System - Conduit Flow/Capacity (Design)

Scenario: 2041WWF_2yrStorm



Legend

- LS Lift Station
- WWTP Sanitary Treatment Plant
- Manhole
- Waterbody
- Road
- Watercourse
- Forcemain

Conduit: Velocity (m/s)

- <= 0.6
- <= 1.5
- <= 3.0
- > 3.0

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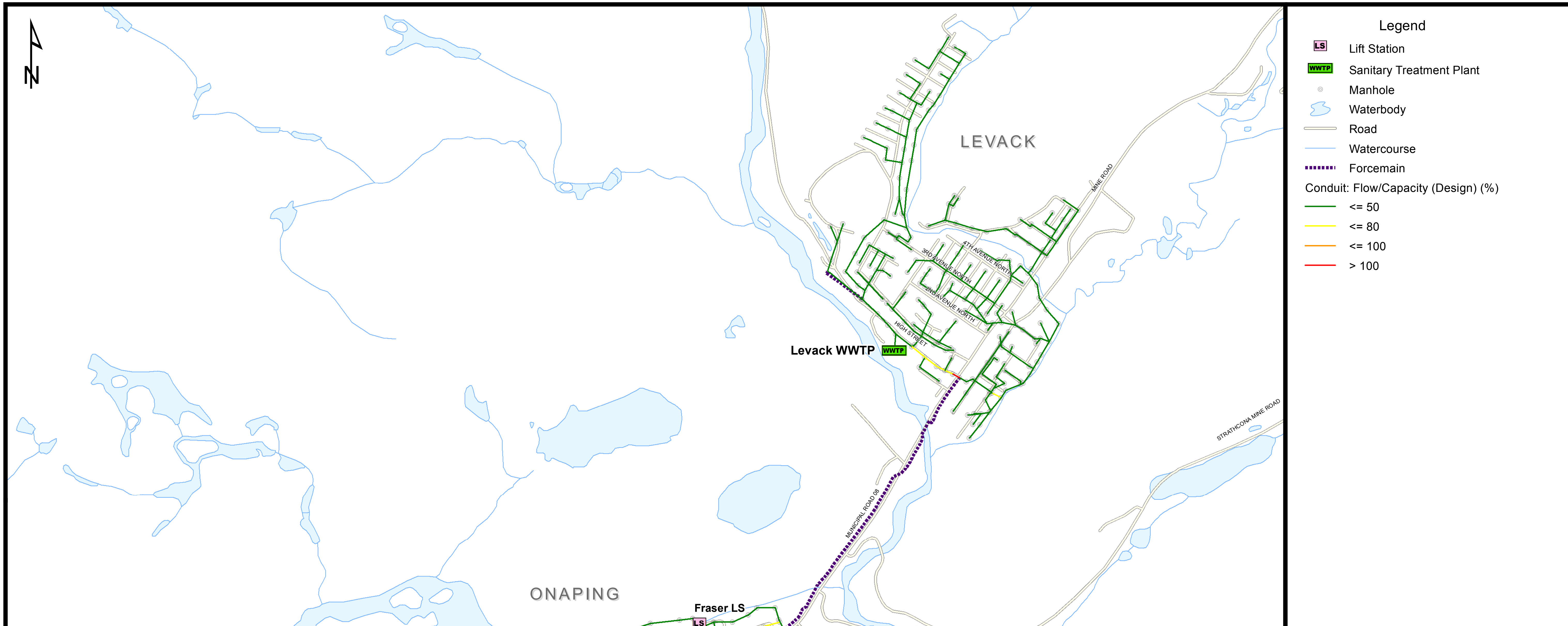
Map 4.2

Onaping and Levack Wastewater System - Conduit Velocity

Scenario: 2041WWF_2yrStorm



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Sudbury Water and Wastewater Master Plan

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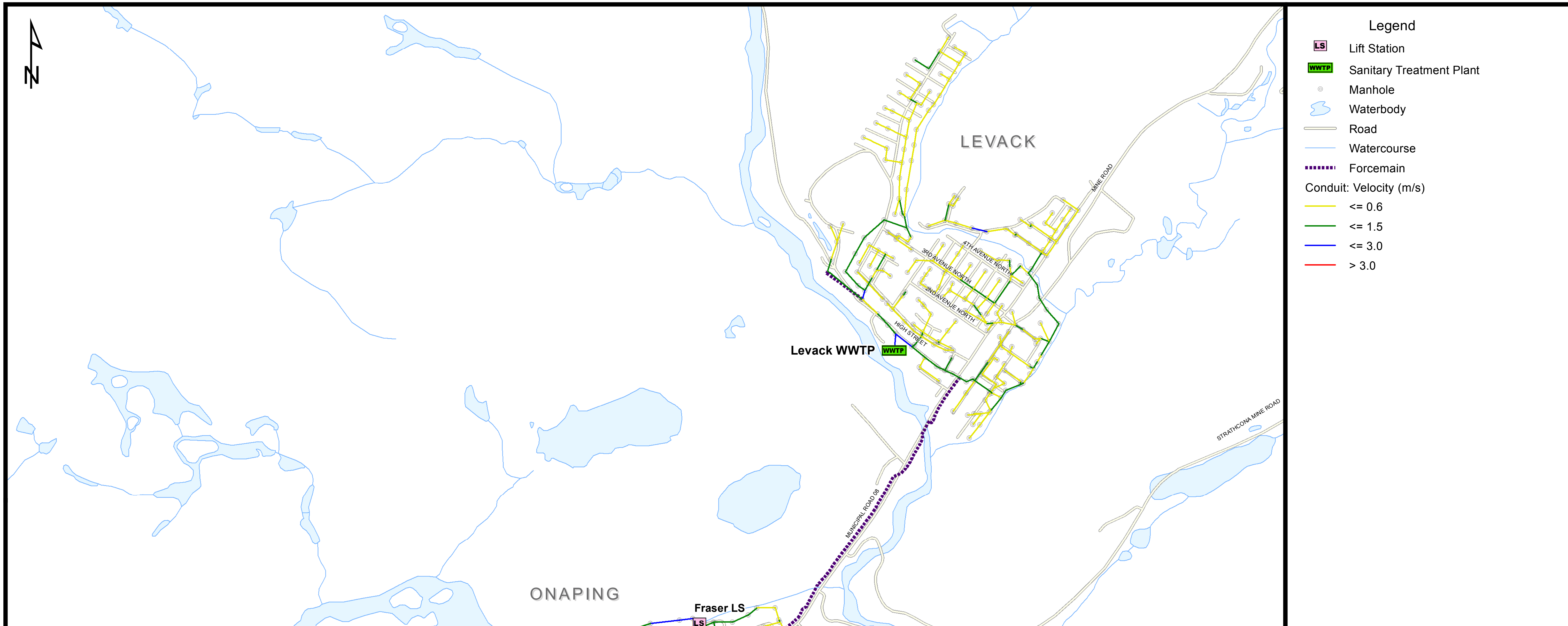
Map 5.1

Onaping and Levack Wastewater System - Conduit Flow/Capacity (Design)

Scenario: ULTWWF_2yrStorm



Greater Sudbury | WSP

Project No. 121-23026-00

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Sudbury Water and Wastewater Master Plan

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Map 5.2

Onaping and Levack Wastewater System - Conduit Velocity

Scenario: ULTWWF_2yrStorm

APPENDIX

C WASTEWATER GENERATION AND CAPACITY ASSESSMENTS

Levack - Wastewater Flow Forecasts

		2009	2010	2011	2012	2013	Summary	Design Criterion	Comments
Average Day Flow	**(m^3/d)**	862	765	799	615	786	765	765	From Annual Reports
Max Day Flow	**(m^3/d)**	2,353	3,251	1,901	1,828	2,879	2,442	2,442	From Annual Reports
Max Day Factor		2.73	4.25	2.38	2.97	3.66	3.20	3.20	Calculated - Max Day Flow divided by Average Day Flow
Peak Flow	**(m^3/d)**	Not Available						0	Peak hour flows were not available
Peak Flow	**(L/s)**							0	
Peak Flow Factor									
Population (Existing Areas)		2,112	2,112	2,112	2,112	2,112	2,112	2,112	
Ratio of Residential to Total Customers		0.43	0.43	0.43	0.43	0.43	0.43		This ratio is based on Water Billing Records for the area and is an approximation of the residential portion of demand.
Residential Share of Average Day Demand (m^3/d)		372	330	344	265	339	330		
Residential Flow Unit Rate (m^3/cap/d)		0.176	0.156	0.163	0.126	0.160	0.156	0.200	
Average Institutional Flow Unit Rate (m^3/ha/d)								28.0	CGS design standards do not suggest average rates for institutional and commercial, but require that rates be developed based on usage. In this case, water demands rates were used and assume a conservative 1:1 ratio of water to sewage.
Average Commercial Flow Unit Rate (m^3/ha/d)								28.0	CGS design standards do not suggest average rates for institutional and commercial, but require that rates be developed based on usage. In this case, water demands rates were used and assume a conservative 1:1 ratio of water to sewage.
Average Industrial Flow Unit Rate (m^3/ha/d)								35.0	CGS design standards average rate for light industrial of 35 m^3/ha/d.
Average Extraneous Flow Unit Rate (m^3/ha/d)								7.48	From CGS Design Standards, peak rate for new developments divided by an assumed peaking factor of 3. This factor would be applied only to new developments, which are assumed to be leak-tight, and have minimal extraneous flow.

	2016	2021	2026	2031	2036	2041	Ultimate Buildout	Comments
Population (Existing Areas)	2,112	2,112	2,112	2,112	2,112	2,112	2,112	
Population (Growth Areas)	11	23	34	42	47	47	365	
Total Population	2123	2135	2146	2154	2159	2159	2477	Total Population (Hemson)
Residential (ha)	1.25	1.30	2.07	2.15	6.79	6.79	13.55	ICI development areas were assigned to planning years based on the stage of the application. Draft Approved were assigned to 2016, Legal Lots of Record to 2026, and Designated Developable to 2036.
Institutional (ha)					2.78	2.78	2.78	
Commercial (ha)			0.71	0.71	0.71	0.71	0.71	
Industrial (ha)					1.86	1.86	1.86	Areas are cumulative and carry from the development year, all the way through to Ultimate Buildout
ICI (ha)	0.00	0.00	0.71	0.71	5.35	5.35	5.35	
Total (ha)	1.25	1.30	2.78	2.86	12.14	12.14	18.90	

	2016	2021	2026	2031	2036	2041	Ultimate Buildout	Comments
Average Residential Flows (m^3/d) - Existing	765	765	765	765	765	765	765	This includes all contribution from existing ICI and infiltration. The base flow was assumed to be the average day flow to the plant for the 2011-2013 period.
Average Residential Flows (m^3/d)	2	5	7	8	9	9	73	Obtained by multiplying the projected population growth by the unit rate.
Average Institutional Flow (m^3/d)	0	0	0	0	78	78	78	Institutional growth area multiplied by unit flow rate.
Average Commercial Flow (m^3/d)	0	0	20	20	20	20	20	Commercial growth area multiplied by unit flow rate.
Average Industrial Flow (m^3/d)	0	0	0	0	65	65	65	Industrial growth area multiplied by unit flow rate.
Average Extraneous Flow (m^3/day)	9	10	21	21	91	91	141	
Average Day Flow (m^3/d)	777	780	813	815	1,028	1,028	1,143	
Max Day Flow (m^3/d)	2,485	2,494	2,600	2,607	3,290	3,290	3,655	

ALTERNATIVE CALCULATION METHOD

Per Capita Flow (m3/cap/day)	0.408	0.362	0.378	0.291	0.372	0.362	0.362

	2016	2021	2026	2031	2036	2041	Ultimate Buildout	Comments
Average Day Flow (m^3/d)	769	774	778	781	782	782	898	Multiplying the total population by the consolidated per capita flow factor.
Max Day Flow (m^3/d)	2,461	2,475	2,488	2,497	2,503	2,503	2,871	

SENSITIVITY ANALYSIS

Analyze sensitivity of forecasted flows to unit rate

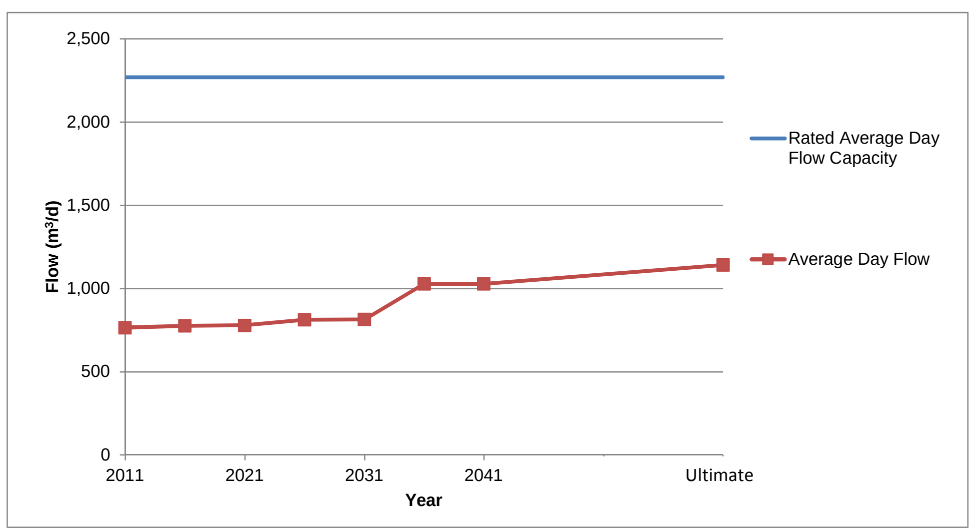
	Unit Rate (m^3/cap/d)	Average Day Flow (m^3/d) 2016	2021	2026	2031	2036	2041	Ultimate Buildout 2061
Base Scenario - Residential Historical Maximum	0.156	777	780	813	815	1,028	1,028	1,143
Combined Historical Maximum	0.362	779	783	818	822	1,036	1,036	1,202
City Standards	0.410	779	785	820	824	1,038	1,038	1,219

Analyze sensitivity of forecasted flows to max day factor

	Max Day Peaking Factor	Max Day Flow (m^3/d) 2016	2021	2026	2031	2036	2041	Ultimate Buildout 2061
Base Scenario - Historical Max	3.32	2,577	2,586	2,696	2,703	3,410	3,410	3,789
Historical Average	4.25	2,485	2,494	2,600	2,607	3,290	3,290	3,655

CAPACITY CHECK

	2011	2016	2021	2026	2031	2036	2041	Ultimate Buildout 2061
Rated WPCP ADF Capacity (m^3/d)	2,270	2,270	2,270	2,270	2,270	2,270	2,270	2,270
WPCP Peak Capacity (m^3/d)	5,675	5,675	5,675	5,675	5,675	5,675	5,675	5,675
Average Day Flow (m^3/d)	765	777	780	813	815	1,028	1,028	1,143
Maximum Day Flow (m^3/d)	2,442	2,485	2,494	2,600	2,607	3,290	3,290	3,655



CITY OF GREATER SUDBURY WATER AND WASTEWATER MASTER PLAN

LIVELY-WALDEN WASTEWATER SYSTEM GAP ANALYSIS AND STATUS QUO REPORT

CITY OF GREATER SUDBURY

TYPE OF DOCUMENT (VERSION)

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

WSP
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FIGURES

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 existing Lively/Walden Wastewater System. Based on population growth projections and design criteria discussed in the *Population Projections and Development of Unit Rates Technical Memorandum* (WSP, 2014), 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 Lively/Walden Wastewater System would continue to be a stand-alone system. Any potential interconnections between Lively/Walden 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 study area includes the areas of Lively, Walden, Naughton, and Mikkola. The area is serviced by a single wastewater system that includes two wastewater treatment plants. Mapping in Appendix A illustrates the study area and identify 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 *Population Projections and Development of Unit Rates Technical Memorandum* (WSP, 2014).

Existing development in the study area is mixed, and includes residential as well as industrial, commercial and institutional land uses. Based on the City's planning data, significant growth is expected in both Lively and Walden in the long term, that is to Ultimate Buildout, but not to 2041. The area population serviced in Lively is expected to increase from 2,197 in 2011 to 2,728 in 2041 and 5,154 by Ultimate Buildout and the area population serviced in Walden is expected to increase from 5,178 in 2011 to 6,324 in 2041 and 11,177 by Ultimate Buildout.

ICI growth is expected to be primarily industrial with smaller amounts of commercial and very small amounts of institutional. Growth is discussed further in Section 6.1.

3 OVERVIEW OF EXISTING SYSTEM

The Lively/Walden Wastewater System includes two wastewater treatment plants, seven wastewater lift stations and a network of sewers. The two wastewater treatment plants, the Lively WWTP and Walden WWTP, service the Lively and Naughton communities, the Mikkola and Oja subdivisions, and the surrounding developed areas. Additional information is provided in the Baseline Review Report (WSP, 2014).

The Lively/Walden Wastewater System is shown in Appendix B.

3.1 LIFT STATIONS

The Lively/Walden wastewater system has seven lift stations; one located in Lively and six located in Walden. Flow from the Lively WWTP can be diverted to the Jacob LS in the Walden WWTP catchment via manhole MR24. Table 3-1 below provides a summary of the main features of the lift stations.

Table 3-1 Lively/Walden Sewage Lift Stations

LIFT STATION	YEAR CONSTRUCTED	LAST UPGRADED	WET WELL VOLUME (M^3)[1]	PUMPING STATION CAPACITY AND FORCEMAIN INFORMATION
Lively-Walden Lift Stations				
Anderson	1976	2004	51.21	Two dry pit pumps with a firm design capacity of 97.80 L/s 300 mm for 7.6 m then increasing to 350 mm diameter HDPE DR17 forcemain for 727 m.
Jacob	1980	1992	N/A	Three submersible pumps with a firm capacity of 138.90 L/s 1993 m long, 400 mm diameter forcemain of unknown material.
Magill	1976	N/A	60.62	Two submersible pumps with a firm capacity of 20.10 L/s 150 mm diameter forcemain for 256 m then increasing to 250 mm diameter cast iron forcemain for 823 m.
Oja	1986	N/A	41.59	Two submersible pumps with a firm capacity of 15.39 L/s 1451 m long, 150 mm diameter PVC forcemain.
Simon Lake East	1984	1997	17.79	Two submersible pumps with a firm capacity of 39.4 L/s 777 m long, 250 mm diameter PVC forcemain.
Simon Lake West	1987	1997	29.91	Two submersible pumps with a firm design capacity of 37.85 L/s 1089 m long, 200 mm diameter PVC forcemain.

Vagnini	1977	N/A	39.10	Two submersible pumps with a firm design capacity of 32.50 L/s 150mm dia. forcemain for 936 m then increasing to 250mm dia. forcemain for 120 m of unknown material.

[1] Obtained or estimated from dimensions found in as-built and record, assuming water level does not exceed the High Water Alarm Level or, in absence of this alarm level, the inlet sewer invert.
[2] Obtained from the City's Wastewater Lift Stations Operations Manual and station as-built drawings.

3.2 LIVELY WWTP

The Lively WWTP provides wastewater servicing to the community of Lively. It is located at Lot 7, Concession V within the City of Greater Sudbury and operated under Certificate of Approval number 6339-7W6JAJ. The plant has an average rated capacity of 1,600 m^3/d, and a peak capacity of 3,000 m^3/d.

The plant was originally commissioned in 1950 by Vale Limited. The ownership of the plant was transferred to what is now the City of Greater Sudbury in 1973. The Lively WWTP uses a conventional wastewater treatment process. A process flow schematic of the liquid treatment train at the Lively WWTP is presented in Figure 3-1.

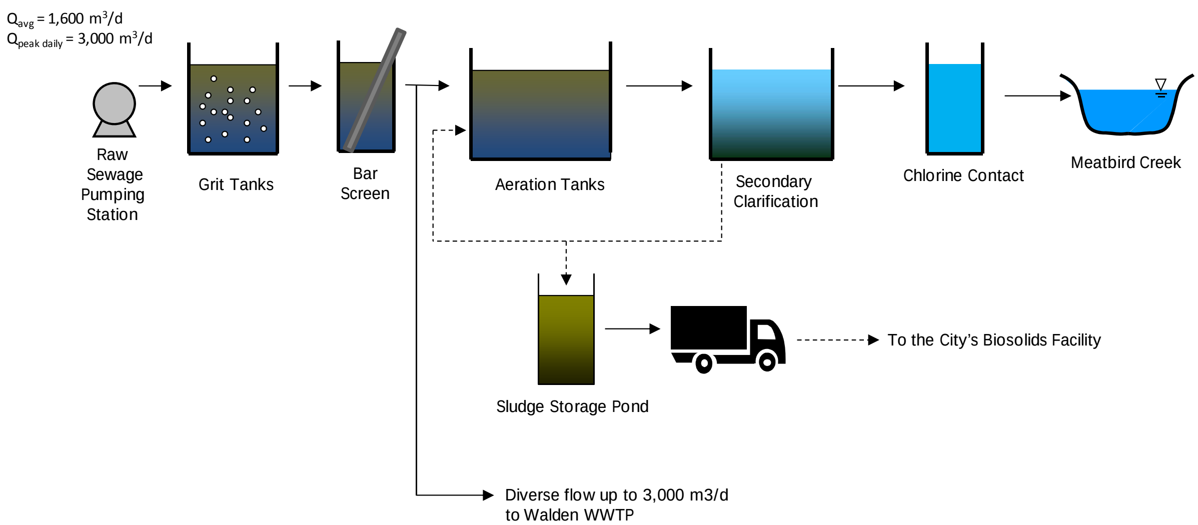

Figure 3-1 Lively WWTP Process

3.3 WALDEN WWTP

The Walden WWTP provides wastewater servicing to the community of Walden and the surrounding area. It is located at Lot 10, Concession III within the City of Greater Sudbury. The plant was originally commissioned in 1982 and has gradually expanded to accommodate growth of the area.

The Walden WWTP is an extended aeration plant. The plant has an average rated capacity of 4,500 m^3/d, and a peak capacity of 8,000 m^3/d. A process flow schematic of the liquid treatment train at the Walden WWTP is presented in Figure 3-2.

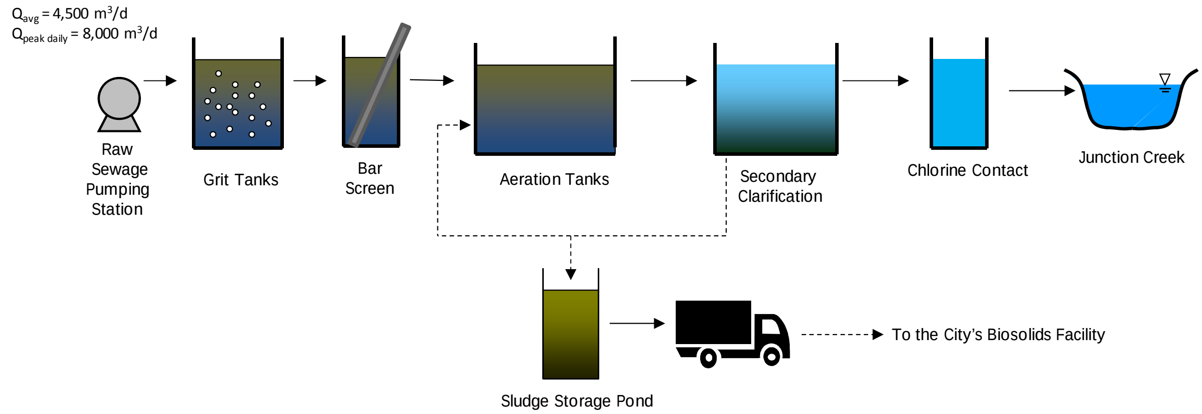

Figure 3-2 Walden WWTP Process

3.4 KNOWN CHALLENGES

In addition to concerns discussed in previous sections, the Lively/Walden Wastewater System has the following known challenges:

— There have been numerous bypasses and spills in the Lively/Walden system since 2004, many of which occurred at the Lively WWTP. Discharges at the Lively WWTP were mainly primary bypasses due to heavy precipitation and/or snow melt, indicating inflow and infiltration problems. There have been four bypasses at the Walden WWTP since 2004; all bypasses were primary bypasses, and three were due to heavy precipitation. There have not been any bypasses at the plant since 2009.

— The 300 mm diameter trunk sewer that runs parallel to 3rd Avenue in Lively is not easily accessible since it is not aligned within the roadway. As such, maintenance and cleaning of this line is less frequently undertaken due to access issues.

— Some sanitary sewers in Lively have been cracked by tree roots and are difficult to access due to the infrastructure being aligned in the backyards of homes in the area. The Sewer on 4th Avenue is also noted to have a very flat grade, which may result in not reaching the required cleansing velocities in the pipe.

— Access to 600 diameter trunk sewer that conveys flows from Mikkola subdivision to the Jacob LS in Walden is not easily accessible due to the manholes in this area being sealed with tar. As such, maintenance and cleaning of this line is less frequently undertaken due to access issues.

— The City has noted that there have been cases of known inflow into the system in the part of the network adjacent to Mud Lake in Walden. A similar situation exists in part of the network near the Oja LS, in which water from the McCharles Lake flows into one of the manholes in this part of the system.

4 HISTORICAL FLOWS AND REVIEW OF OPERATIONAL DATA

Data reported in the 2009 to 2013 *Annual Reports* for Lively and Walden WWTPs was reviewed and analyzed to determine average day and maximum day flows as well as review effluent parameters.

4.1 FLOW DATA

WWTP flow data from 2009 to 2013 was reviewed. Operational data was not available from the lift stations and so historical peak flow data could not be estimated.

The recorded average day and maximum day flows are summarized in Table 4-1 and plotted in Figure 4-1 for the Lively WWTP, and Table 4-2 and Figure 4-2 for the Walden WWTP.

Table 4-1 Historical Wastewater Flow Data for Lively WWTP Service Area

YEAR	AVERAGE DAY FLOW (M^3/D)[1]	MAXIMUM DAY FLOW (M^3/D)[1]
2009	1,143	2,860
2010	923	5,190
2011	1,025	3,912
2012	1,129	5,227
2013	1,693	6,207

[1] *Annual Reports (2009 - 2013).*

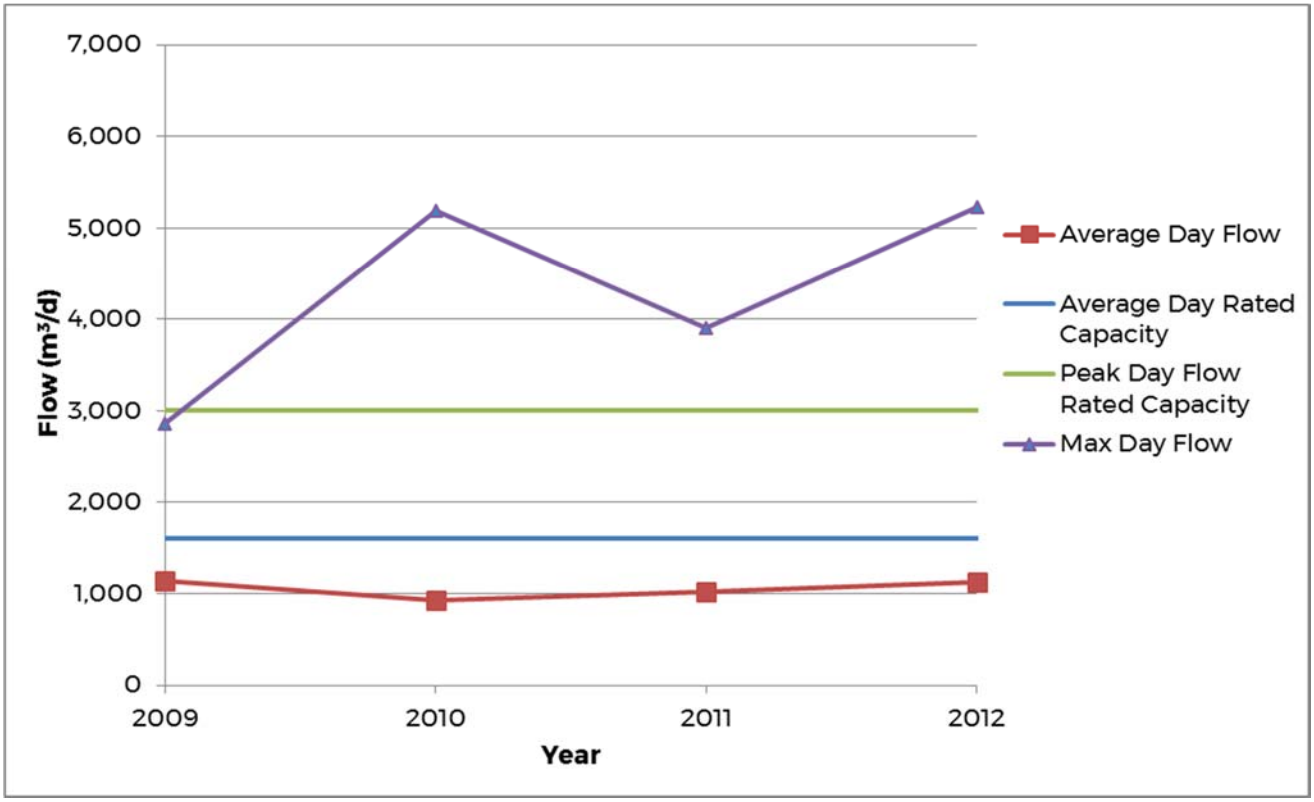

Figure 4-1 Historical Wastewater Flows at the Lively Wastewater Treatment Plant

Table 4-2 Historical Wastewater Flow Data for Walden WWTP Service Area

YEAR	AVERAGE DAY FLOW (M^3/D)[1]	MAXIMUM DAY FLOW (M^3/D)[1]
2009	3,979	11,610
2010	3,079	10,106
2011	3,258	11,199
2012	3,317	12,693
2013	4,252	14,273

[1] *Annual Reports (2009 - 2013)*

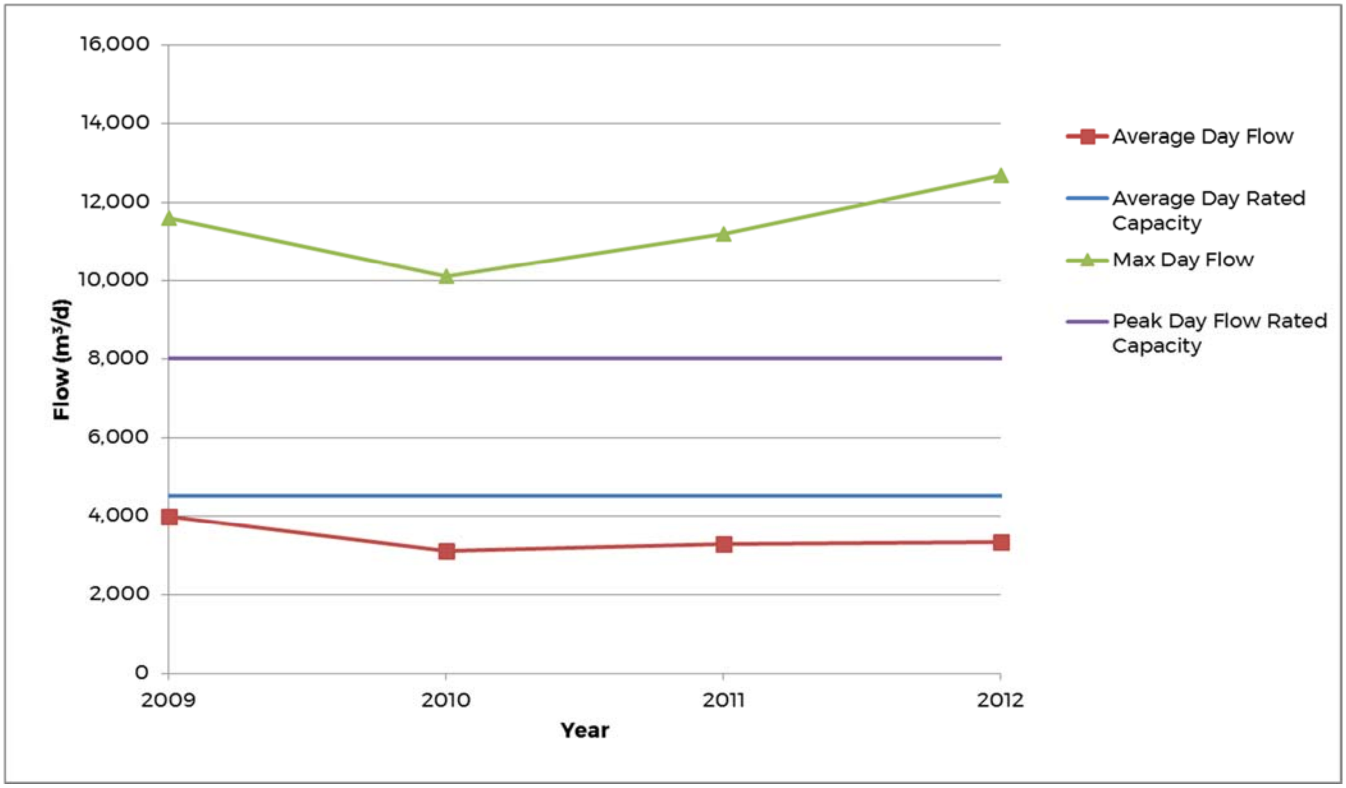

Figure 4-2 Historical Wastewater Flows at the Walden Wastewater Treatment Plant

The relationship between the different flow regimes was analyzed to compare the maximum day peaking factors derived from historical data to those used in the *City's Engineering Design Manual*.

The average day flows to the WWTPs have been relatively steady over the 2009 to 2013 period, averaging 1,183 m^3/d for the Lively WWTP service area and 3,577 m^3/d for the Walden WWTP service area. The variations in historical maximum day flows show no discernible trend when all flow data is considered.

The highest maximum day to average day peaking factor was 5.62 for Lively (2010) and 3.83 for Walden (2012). The average maximum day peaking factor from 2009 to 2013 was 4.05 for Lively and 3.36 for Walden. The City's *Engineering Design Manual* and the *MOE Guidelines* do not specify recommended maximum day factors and recommend using historical data when available. For future wastewater generation, the average peaking factors (4.05 and 3.36 for Lively and Walden respectively) were used and based on the assumption that new developments would have less I/I due to more leak tight construction.

4.2 RAW WASTEWATER CHARACTERISTICS

The average raw wastewater characteristics from 2009 to 2012 for the Lively and Walden WWTPs are summarized in the tables below.

Table 4-3 Average Raw Wastewater Characteristics at the Lively WWTP (2008-2012)

PARAMETER	AVERAGE VALUE
$CBOD_5$	125 mg/L
Suspended Solids	134 mg/L
Total Phosphorus	4.4 mg/L

TKN	31.9 mg/L
pH	7.1

Table 4-4 Average Raw Wastewater Characteristics at the Walden WWTP (2008-2012)

PARAMETER	AVERAGE VALUE
$CBOD_5$	233 mg/L
Suspended Solids	271 mg/L
Total Phosphorus	4.3 mg/L
TKN	31.1 mg/L
pH	6.9

Wastewater flows to the both the Lively and Walden WWTPs correspond to mixed uses, with contributions from residential and ICI users.

4.3 EFFLUENT CRITERIA

The Lively WWTP is operated in accordance with MOE Certificate of Approval (C of A) No. 6339-7W6JAJ dated December 1, 2009 and the Walden WWTP is operated in accordance with MOE Certificate of Approval (C of A) No. 5318-7W6J9Y dated December 1, 2009.

The C of A concentration and loading limits for both the Lively WWTP and Walden WWTP are summarized in Table 4-5 and Table 4-6.

Table 4-5 Lively WWTP Effluent Limits and Objectives

EFFLUENT PARAMETER	CONCENTRATION LIMIT	LOADING LIMIT (ANNUAL AVERAGE)	CONCENTRATION OBJECTIVE/LOADING LIMIT
$CBOD_5$	25 mg/L	40 kg/d	25 mg/L
TSS	25 mg/L	40 kg/d	25 mg/L
Total P June 1 to August 31 September 1 to May 31	1 mg/L	1.6 kg/d	 0.5 mg/L <1.0 mg/L
E. coli	200 cfu / 100 ml	N/A	N/A
pH	N/A	N/A	6.0-9.5

Table 4-6 Walden WWTP Effluent Limits and Objectives

PARAMETER	CONCENTRATION LIMIT	LOADIN LIMIT (ANNUAL AVERAGE)	CONCENTRATION OBJECTIVE / LOADING LIMIT
$CBOD_5$	25 mg/L	112.5 kg/d	15 mg/L
TSS	25 mg/L	112.5 kg/d	15 mg/L

PARAMETER	CONCENTRATION LIMIT	LOADIN LIMIT (ANNUAL AVERAGE)	CONCENTRATION OBJECTIVE / LOADING LIMIT
Total P June 1 to August 31 September 1 to May 31	1 mg/L	4.5 kg/d	 0.5 mg/L <1.0 mg/L
E. coli	200 cfu / 100 ml	N/A	N/A
pH	N/A	N/A	6.0-9.5

4.4 OPERATIONAL DATA

The general plant operation was reviewed against the Lively and Walden WWTPs C of A requirements and historical data provided in the Annual Reports from 2009 to 2012, respectively for each. Historical data is summarized in Table 4-7 and Table 4-8.

Table 4-7 Historical Effluent Concentrations for the Lively WWTP

EFFLUENT PARAMETER	ANNUAL AVERAGE 2009	2010	2011	2012
$CBOD_5$ (mg/L)	7.2	2.0	3.0	2.0
TSS (mg/L)	5.6	5.8	7.1	7.3
TP (mg/L)	0.60 (all months comply)	0.46 (all months comply)	0.34 (all months comply)	0.93 (exceedance in July – 6.50 mg/L)
pH	6.80	6.60	6.90	6.93
TAN (mg/L)	6.26	4.33	10.05	13.69
E. coli (organisms/100 mL)	May, June, Sept exceedance (TNTC)	5	6	5

The Lively WWTP was in compliance for all parameters except E.Coli in 2009 and TP in July 2012.

Table 4-8 Historical Effluent Concentrations for the Walden WWTP

EFFLUENT PARAMETER	ANNUAL AVERAGE 2009	2010	2011	2012
CBOD5 (mg/L)	12.0	4.6	1.6	2.9
TSS (mg/L)	13.7	10.1	8.6	9.5
TP (mg/L)	0.41 (all months comply)	0.42 (all months comply)	0.39 (all months comply)	0.34 (all months comply)
pH	6.50	6.60	6.80	6.55
TAN (mg/L)	3.70	5.75	0.22	1.79
E. coli (organisms/100 mL)	55	7	13	7

The Walden WWTP has met all concentration limits.

A capacity review of each unit process at the WWTP was not conducted. Instead, the rated capacity was considered the true capacity of the plant.

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 *Pop Population Projections and Development of Unit Rates Technical Memorandum* (WSP, 2014). Otherwise, design criteria recommended in the *MOE Guidelines* and *City's Engineering Design Manual* were used.

5.1 UNIT WASTEWATER DESIGN CRITERIA

The unit flow criteria for growth adopted for this assessment are shown in Table 5-1 below. These values were recommended in the *Population Projections and Development of Unit Rates Technical Memorandum* (WSP, 2014).

Note that the term "extraneous flows" is used interchangeably with "I/I flows".

Table 5-1 Wastewater System Design Criteria

CRITERIA	VALUE	REFERENCE
Average Day Residential Flow	450 L/cap/day	City's Engineering Design Manual (for flow in Walden)
Average Day Commercial and Institutional Flow	28 m^3/ha/d	Water unit rates, assuming a 1:1 correspondence
Average Day Industrial Flow (Balance of projected industrial lands, over and above the 172 ha considered in J.L. Richards' work)	35 m^3/ha/d	Water unit rates, assuming a 1:1 correspondence
Average Industrial Flow (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 Flow (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 Flow (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)
Average Extraneous Flow	11.2 m^3/ha/d for Lively 6.02 m^3/ha/d for Walden	Peak from City's Engineering Design Manual and assuming a peaking factor of three
Peak Extraneous Flow	33.7 m^3/ha/d for Lively 18.05 m^3/ha/d for Walden	City's Engineering Design Manual
Max Day Peaking Factor	4.05 for Lively 3.36 for Walden	Average of historical values

I/I Rate DWF	0.0014 – 0.0019 L/s/m(pipe)	City of Greater Sudbury Lively and Walden Inflow and Infiltration Study – Report 1 (J.L. Richards & Associates Limited, 2011)
I/I Rate WWF	0.0009 – 0.0405 L/s/m(pipe)	City of Greater Sudbury Lively and Walden Inflow and Infiltration Study – Report 1 (J.L. Richards & Associates Limited, 2011)

Residential average day flows were obtained by multiplying the residential unit rate by the service population.

Maximum day flows to the WWTP are obtained by multiplying the average day flow by the maximum day peaking factor.

5.2 DESIGN CRITERIA FOR WASTEWATER SYSTEM COMPONENTS AND OPERATION

5.2.1 WASTEWATER TREATMENT

Wastewater treatment facilities are rated for average day flows. Plant effluent limits and objectives are established in the C of A or ECA for each facility.

5.2.2 LIFT STATION PUMPING CAPACITY

The firm capacity of the lift station (with the largest pump out of service) must allow pumping of peak wet weather flows corresponding to its catchment area (MOECC, 2008).

Starting limitations on pump motors generally dictate the minimum size of a wet well. The wet well should be large enough to prevent pump motors from overheating due to frequent starting and stopping, but small enough to avoid long retention times leading to septicity and odor problems (Lin & Lee, 2001).

The station wet well shall be sized such that the number of pump starts per hour does not exceed the maximum value recommended by the pump manufacturer. In other words, the time between pump starts and stops (i.e. the pump cycle time) should not exceed that which results in a pump start frequency greater than that recommended by the pump manufacturer. Typically, submersible pumps can cycle four to 10 times per hour with a maximum cycle time not exceeding 30 minutes (Lin & Lee, 2001). A maximum value of four pump starts per hour was assumed to evaluate wet well sizing requirements.

5.2.3 SEWERS

The sewer system is typically sized to convey peak instantaneous (peak wet weather) flows. Sewage flows are made up of wastewater discharges from residential, commercial, institutional and industrial establishments, plus extraneous flow components from such sources as groundwater and surface runoff.

In addition to being able to convey peak flows, sufficient flow velocity should be maintained to transport the sewage solids to avoid deposition and the development of nuisance conditions under lower flow conditions. The minimum acceptable flow velocity in sewers is 0.6 m/s (City of Greater Sudbury, 2012).

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 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 Lively WWTP is projected to increase by 531 people in 2041 and 2,957 by Ultimate Buildout and the population service by the Walden WWTP is projected to increase by 1,146 people in 2041 and 5,998 by Ultimate Buildout. The population projections to be used in the Master Plan are summarized in the table below.

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

Table 6-1 Lively and Walden Population Projections

SYSTEM	2011	2016	2021	2026	2031	2036	2041	ULTIMATE BUILDOUT
Lively	2,197	2,348	2,491	2,607	2,676	2,716	2,728	5,154
Walden	5,178	5,501	5,804	6,059	6,209	6,299	6,324	11,177

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 the table below.

Table 6-2 Walden ICI Projections

	ICI DEVELOPMENT AREAS (HA)						
LAND USE	2016	2021	2026	2031	2036	2041	BUILDOUT
Institutional	0	0	1.12	0	0.85	0	0
Commercial	0	0	1.50	0	8.21	0	0
Industrial	0	0	74.41	100.00	3.19	0	0
Total	**0**	**0**	**77.03**	**100.00**	**12.25**	**0**	**0**

Lively has no expected ICI growth.

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 Lively and Walden, 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, 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 Lively/Walden 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 WASTEWATER FLOW PROJECTIONS AND INFRASTRUCTURE NEEDS

The unit flow criteria indicated in Section 5.1 were used to estimate the future wastewater flows in Lively/Walden. In general, the projected flows were calculated by the following formula:

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

The flows corresponding to the population growth forecasts to Ultimate Buildout are presented in the tables below.

Table 6-3 Flow Projections for Lively WWTP Service Area

YEAR	POPULATION	AVERAGE DAY FLOW (M³/D)	MAXIMUM DAY FLOW (M³/D)
Base	2,197	1,183	4,679
2016	2,348	1,357	5,493
2021	2,491	1,421	5,753
2026	2,607	2,159	8,739
2031	2,676	2,190	8,865
2036	2,716	2,208	8,939
2041	2,728	2,214	8,960
Ultimate	5,154	3,305	13,379

Table 6-4 Flow Projections for Walden WWTP Service Area

YEAR	POPULATION	AVERAGE DAY FLOW (M³/D)	MAXIMUM DAY FLOW (M³/D)
Base	5,178	3,577	11,976
2016	5,501	4,159	13,991
2021	5,804	4,319	14,530
2026	6,059	6,125	20,605
2031	6,209	7,028	23,644

YEAR	POPULATION	AVERAGE DAY FLOW (M^3/D)	MAXIMUM DAY FLOW (M^3/D)
2036	6,299	7,515	25,282
2041	6,324	7,273	24,466
Ultimate	11,177	10,208	34,341

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 demand was calculated by applying the respective peaking factor to the average day demand. The maximum day demand for the base year was the average historical.

A desktop analysis of historical wastewater flows and future flow projections is included in Appendix C.

6.4.1 LIVELY/WALDEN WWTP CAPACITY

The rated average day capacity of the Lively WWTP is 1,600 m^3/d, and is compared to the current and future flow projections in Figure 6-1. As is illustrated in Figure 6-1, the plant is very close to reaching its rated capacity and therefore additional wastewater treatment servicing is required for the Lively area. That said, in the Lively/Walden Environmental Servicing Report (J.L. Richards 2013), this very issue was assessed and the recommendation proposed as part of the Study was to decommission the existing Lively WWTP and to expand the Walden WWTP such that it could support wastewater treatment requirements for wastewater generated in both Lively and Walden. As such, the remainder of this report will focus on the requirements at the Walden WWTP, on the basis that wastewater flows generated in Lively will be conveyed to the Walden WWTP for treatment.

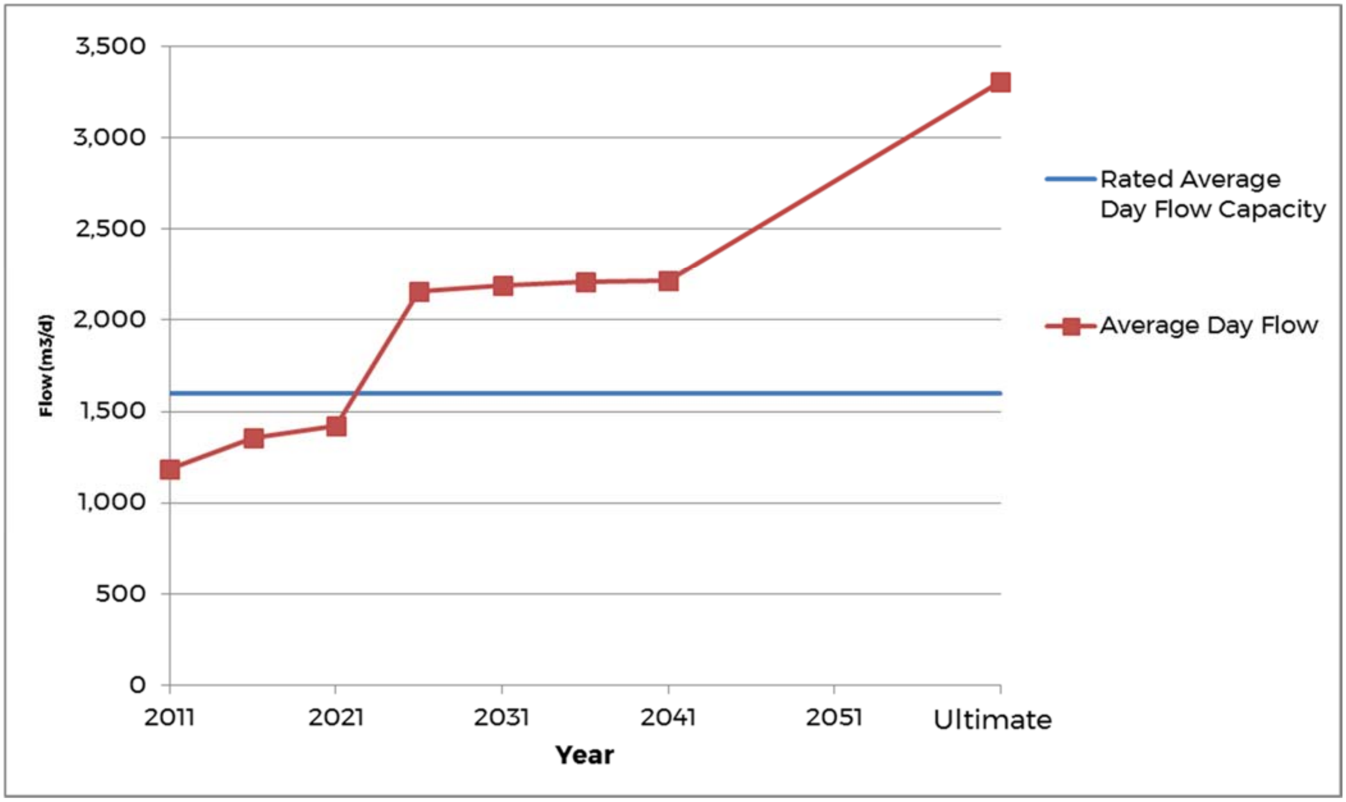

Figure 6-1 Wastewater Flow Projections Compared to Lively WWTP Rated Capacity Historical

Figure 6-2 illustrates the combined projected average day flows generated in Walden, as well as in Lively from 2021 onwards. The assumption is that by 2021 all the wastewater flows generated in Lively will be treated at the Walden WWTP. It appears that, although the wastewater flows from Lively will add to the treatment requirements in the long term, the expansion of the plant will be driven first by the additional wastewater treatment capacity required to treat wastewater flows generated in Walden. The plant has already reached about 85% of its capacity in 2013, based on the projections.

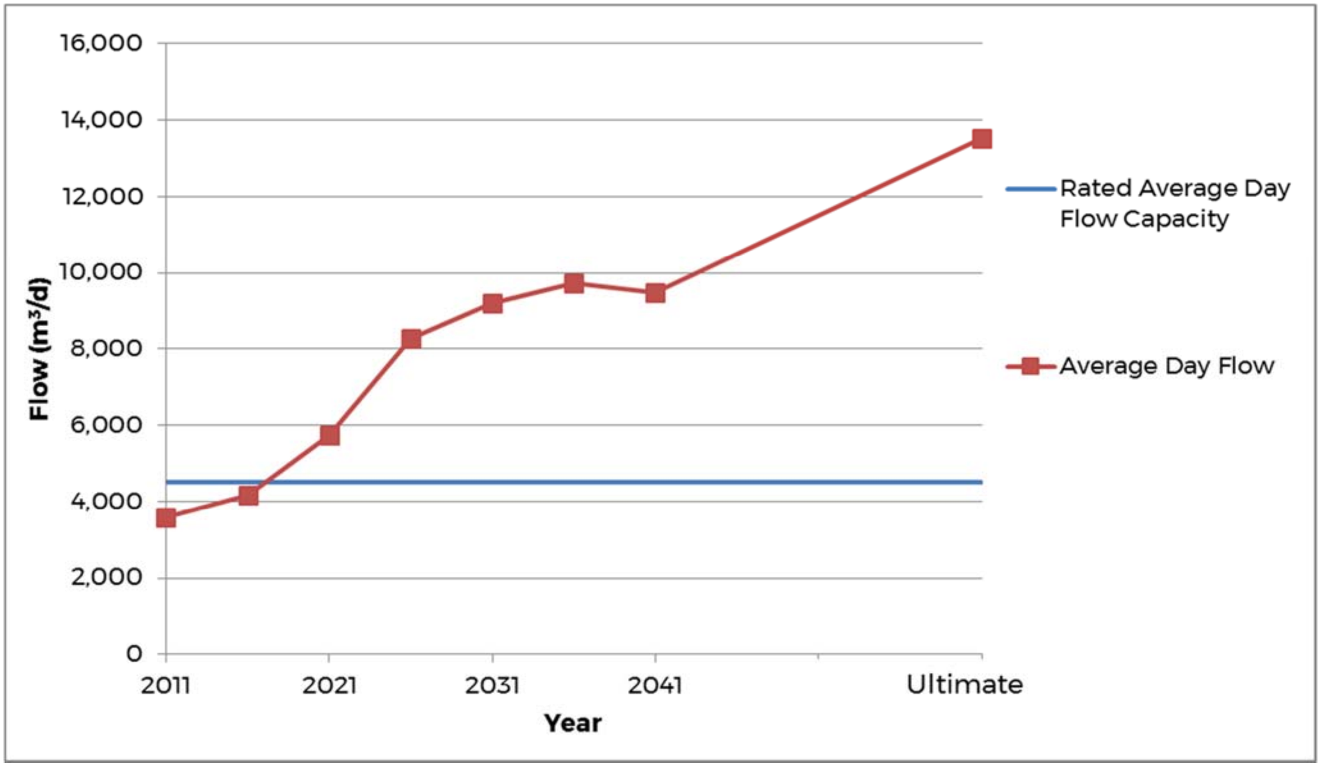

Figure 6-2 **Wastewater Flow Projections Compared to Walden WWTP Rated Cap2acity**

6.4.2 SEWER NETWORK AND LIFT STATIONS

For each of the scenarios modeled, the system was checked for surcharging of sewers and capacity exceedance at the lift stations. The peak flows into each of the lift stations was determined from the computer simulations for the various planning scenarios and is presented in Table 6-5 below. The table also shows the design/rated flow for the pumps, their capacity based on drawdown tests and the computer simulated flow for comparison.

Table 6-5 **Lift Station Peak Influent Flow Rates**

	CURRENT FIRM CAPACITY (L/S)	EXISITING PEAK FLOW (L/S)	2041 PEAK FLOW	ULTIMATE BUILDOUT
Anderson	97.8	173.2	174.1	177.9
Jacob (includes the flows from Lively)	138.9	622.5	638.9	651.1
Magill	20.1	0.4	2.66	2.66
Oja	15.39	5.26	6.12	6.47
Simon Lake East	39.4	34.1	35.9	36.3
Simon Lake West	37.85	13.5	14.5	14.9
Vagnini	32.50	2.4	10.3	10.3

Based on the above table, peak inflows exceed station capacity at Anderson and Jacob Lift Stations. Peak inflows also exceed simulated station capacity at Simon Lake East LS, but do not exceed the drawdown capacity.

7 HYDRAULIC MODELLING

7.1 APPROACH

A basic sanitary model for the City of Greater Sudbury was received from the City. The model was created in Bentley's SewerGEMS by City staff. The model is an all pipe model of the sanitary network in these systems, but some critical information such as pipe data, invert elevations and lift station characteristics were missing. The model now includes this information as well as key vertical infrastructure in each system, including lift stations and treatment facilities.

The model was loaded with wet weather flow data. A water balance was completed to determine I/I rates for both dry and wet weather flow. The results from the water balance were compared against I/I rates developed through flow monitoring, and the greater of the two values, for each system, was used to load the model.

Current (2011) and future (2016-Ultimate Buildout, in 5 year increments) population data was added to the model using the City's planning data, summarized in previous sections of this report.

Future dry and wet weather flow scenarios were developed for each of the horizon years: 2016, 2021, 2026, 2031, 2036, 2041, and Ultimate Buildout. However, model results did not vary from 2016 to 2041; therefore, this report discusses findings for 2041 and Ultimate Buildout, compared against existing (2011).

7.2 MODELLING FINDINGS

The model was used to check sewer capacity and flow velocity.

Many of the sewers flow at less than 50% of the available capacity from 2011 to Ultimate Buildout scenarios under the wet weather flow condition. However, the sewer along MR24 from Lively to Walden does not have sufficient capacity to convey 2041 and Ultimate buildout peak flows. The sewer along 3rd Avenue does not have sufficient capacity to convey flows from 2011 through Ultimate.

Flow velocities through many sewers in the Lively and Walden system are generally below the City's standard of 0.6 m/s. This is consistent through to Ultimate Buildout under the wet weather flow condition.

Maps in Appendix B illustrate the modeling results for the 2011, 2041, and Ultimate dry and wet weather flow scenarios.

8 CONCLUSIONS

An assessment of the Lively/Walden Wastewater System was completed to identify infrastructure requirements to service forecasted growth in the community.

The conclusions of the assessment are summarized below.

- The Walden WWTP does not have sufficient average day capacity in the short term and therefore does not have capacity to treat wastewater flows generated in Lively.
- Many of the sewers in Lively and Walden operate at velocities that do not meet the City's current standards (that is, many sewers flow at less than 0.6 L/s). This may cause operational problems, such as solids buildup and odours. The sewer along MR24 exceeds capacity in the 2041 and Ultimate Buildout scenarios.
- Both Anderson and Jacob LSs do not have sufficient capacity however based on the Lively/Walden Environmental Servicing Report (J.L. Richards 2013) the Anderson LS is recommended to be decommissioned.

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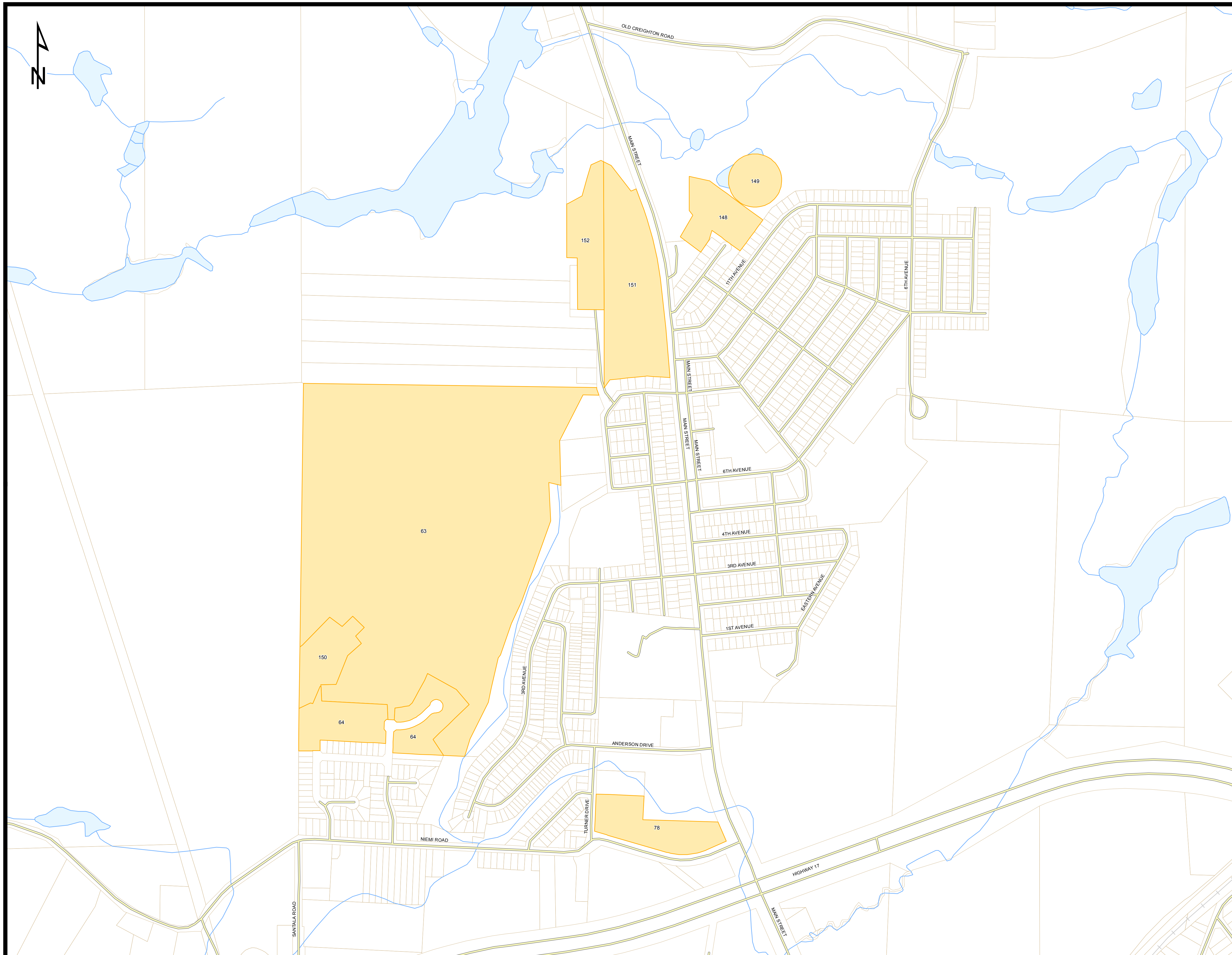
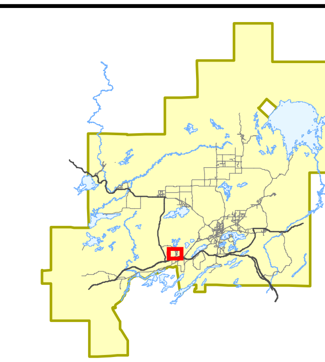
APPENDIX

A RESIDENTIAL AND ICI DEVELOPMENT AREAS



Legend

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



Greater Sudbury | WSP

Project No. 121-23026-00

Sudbury Water and Wastewater Master Plan

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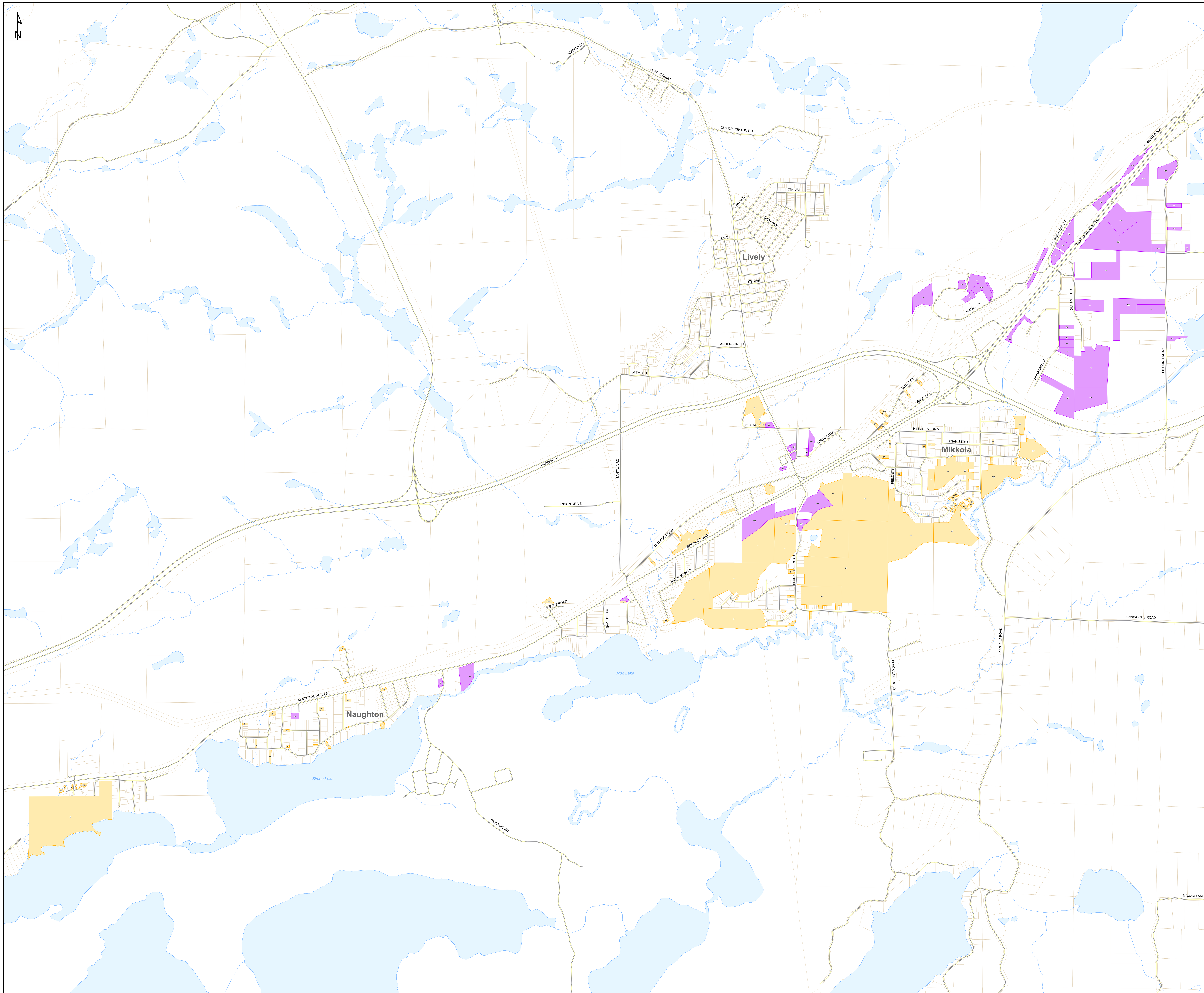
Map 11

Lively
2011 - Ultimate Buildout Residential and ICI Development Areas



Project No. 121-23026-00

Sudbury Water and Wastewater Master Plan

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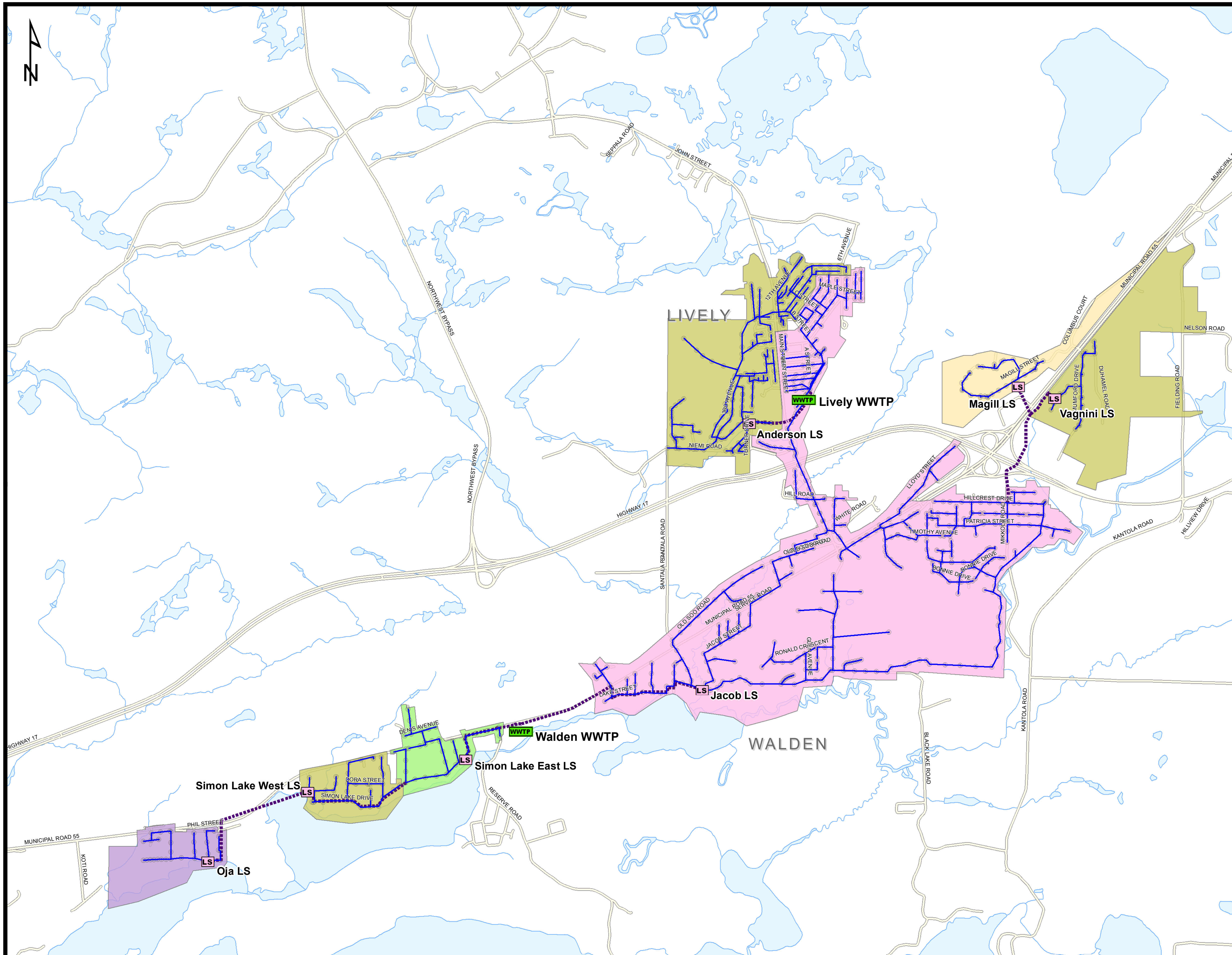
Map 11

Walden
2011 - Ultimate Buildout
Residential and ICI
Development Areas

APPENDIX

B WASTEWATER MODEL RESULTS



Legend

- LS Lift Station
- WWTP Sanitary Treatment Plant
- Manhole
- Waterbody
- Road
- Watercourse
- Sanitary Sewer Conduit
- Forcemain

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December 09 2016

Sudbury Water and Wastewater Master Plan

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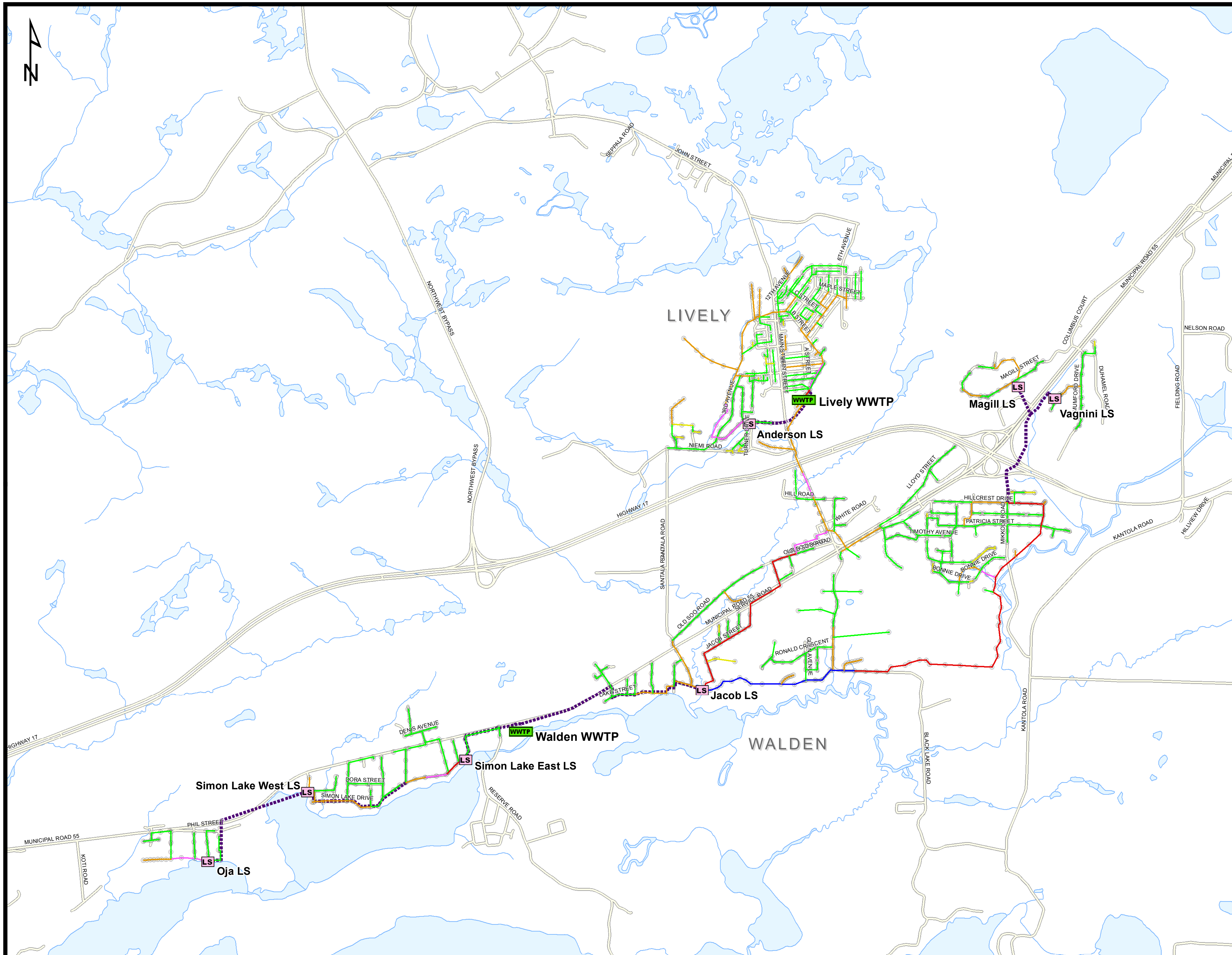
Map 1

Lively and Walden Wastewater System - Sewershed Map



Legend

- LS Lift Station
- WWTP Sanitary Treatment Plant
- Manhole
- Waterbody
- Road
- Watercourse
- Forcemain

Conduit: Diameter (mm)

- <= 150
- <= 200
- <= 300
- <= 450
- <= 600
- <= 750
- > 750

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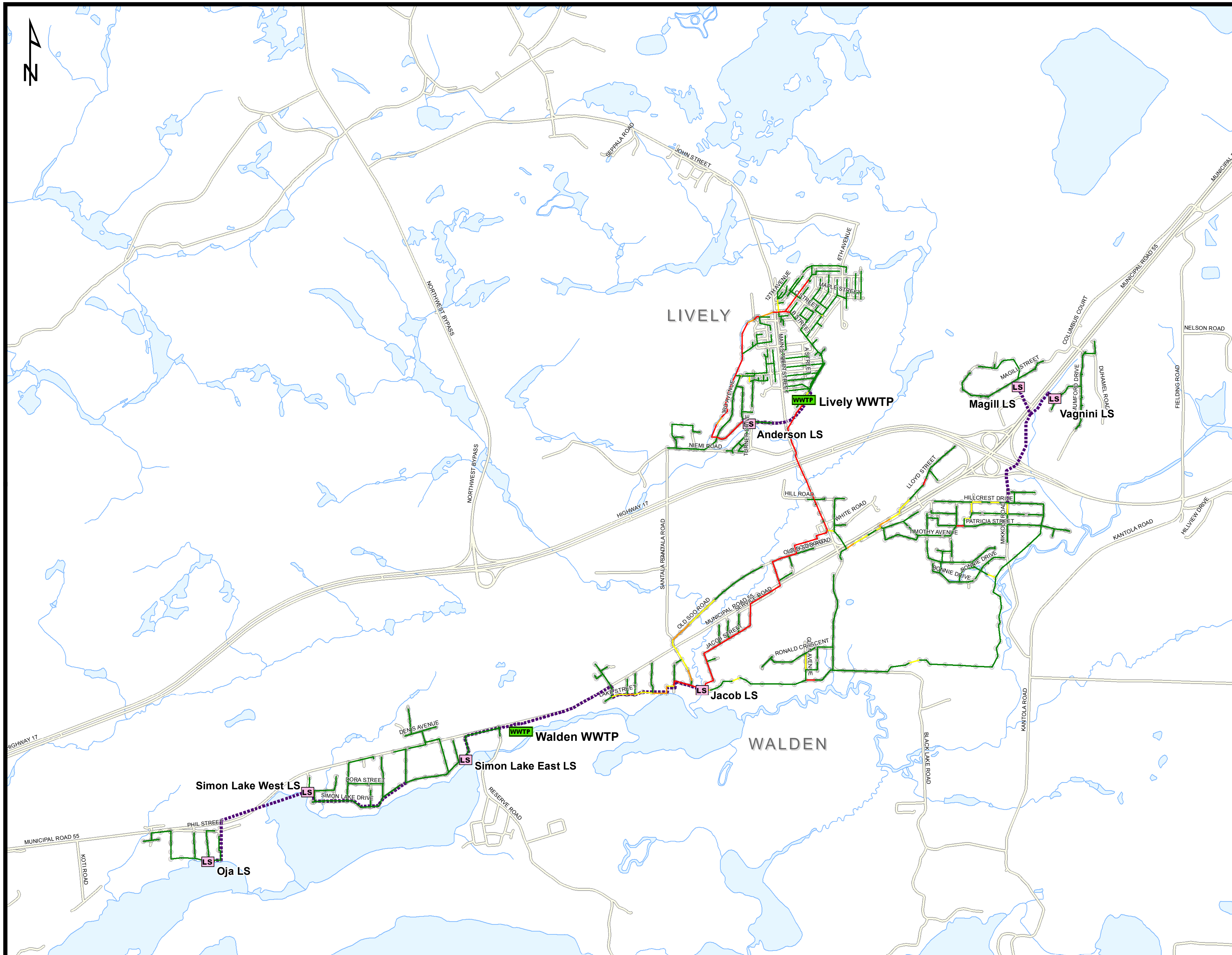
Map 2

Lively and Walden Wastewater System - Conduit Diameter



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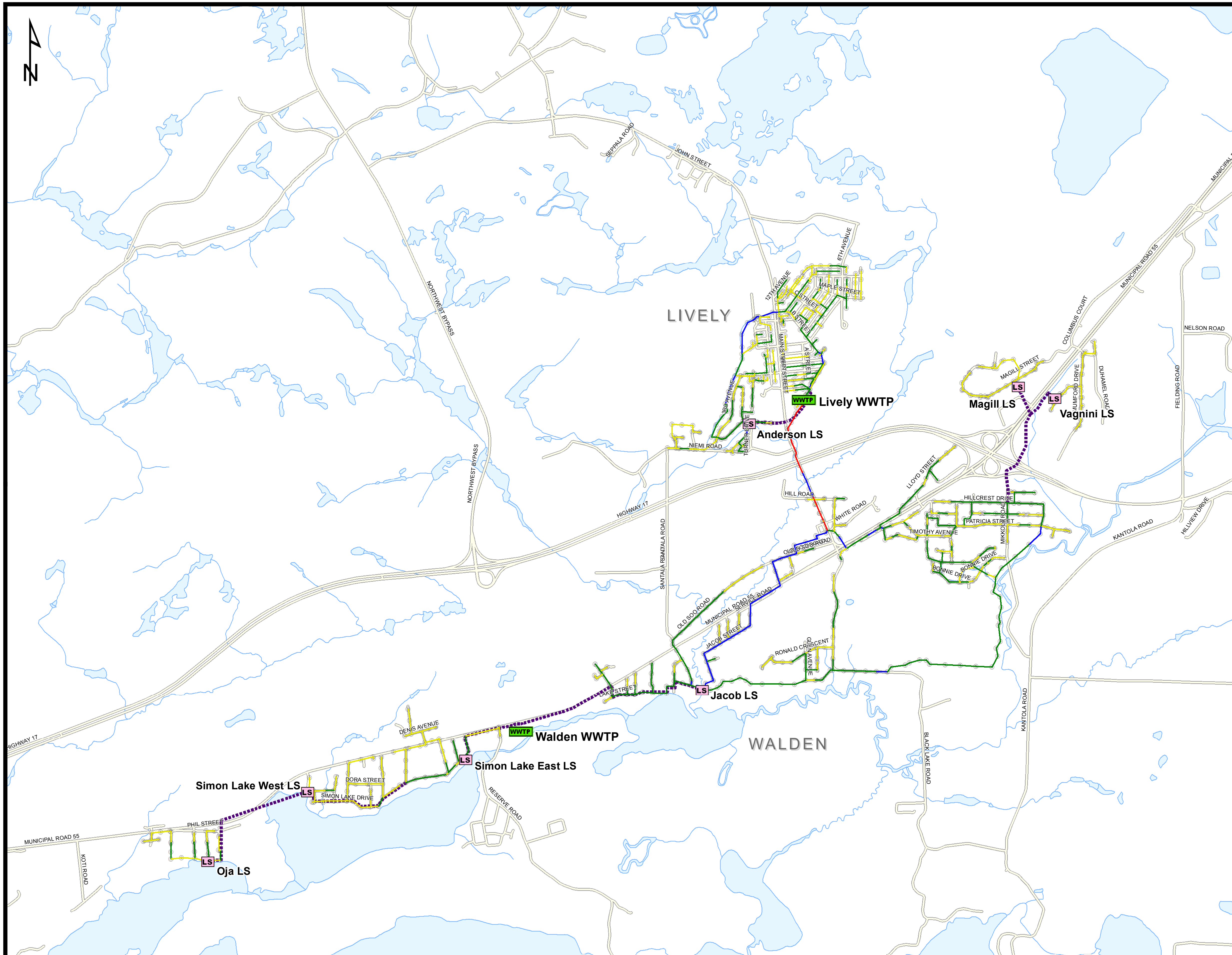
Map 3.1

Lively and Walden Wastewater System - Conduit Flow/Capacity (Design)

Scenario: 2011WWF_2yrStorm



Legend

- LS Lift Station
- WWTP Sanitary Treatment Plant
- Manhole
- Waterbody
- Road
- Watercourse
- Forcemain

Conduit: Velocity (m/s)

- <= 0.6
- <= 1.5
- <= 3.0
- > 3.0

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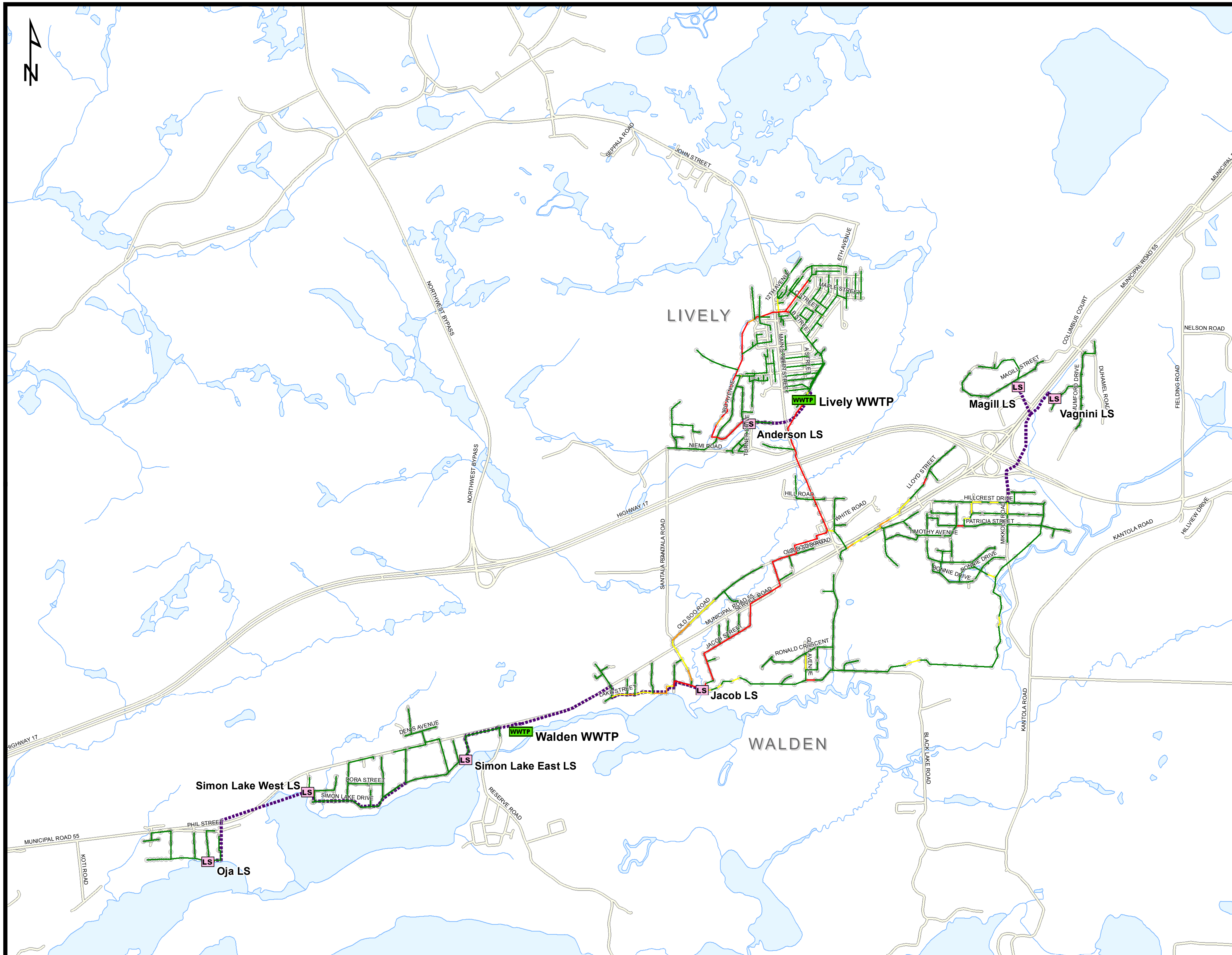
Map 3.2

Lively and Walden Wastewater System - Conduit Velocity

Scenario: 2011WWF_2yrStorm



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Sudbury Water and Wastewater Master Plan

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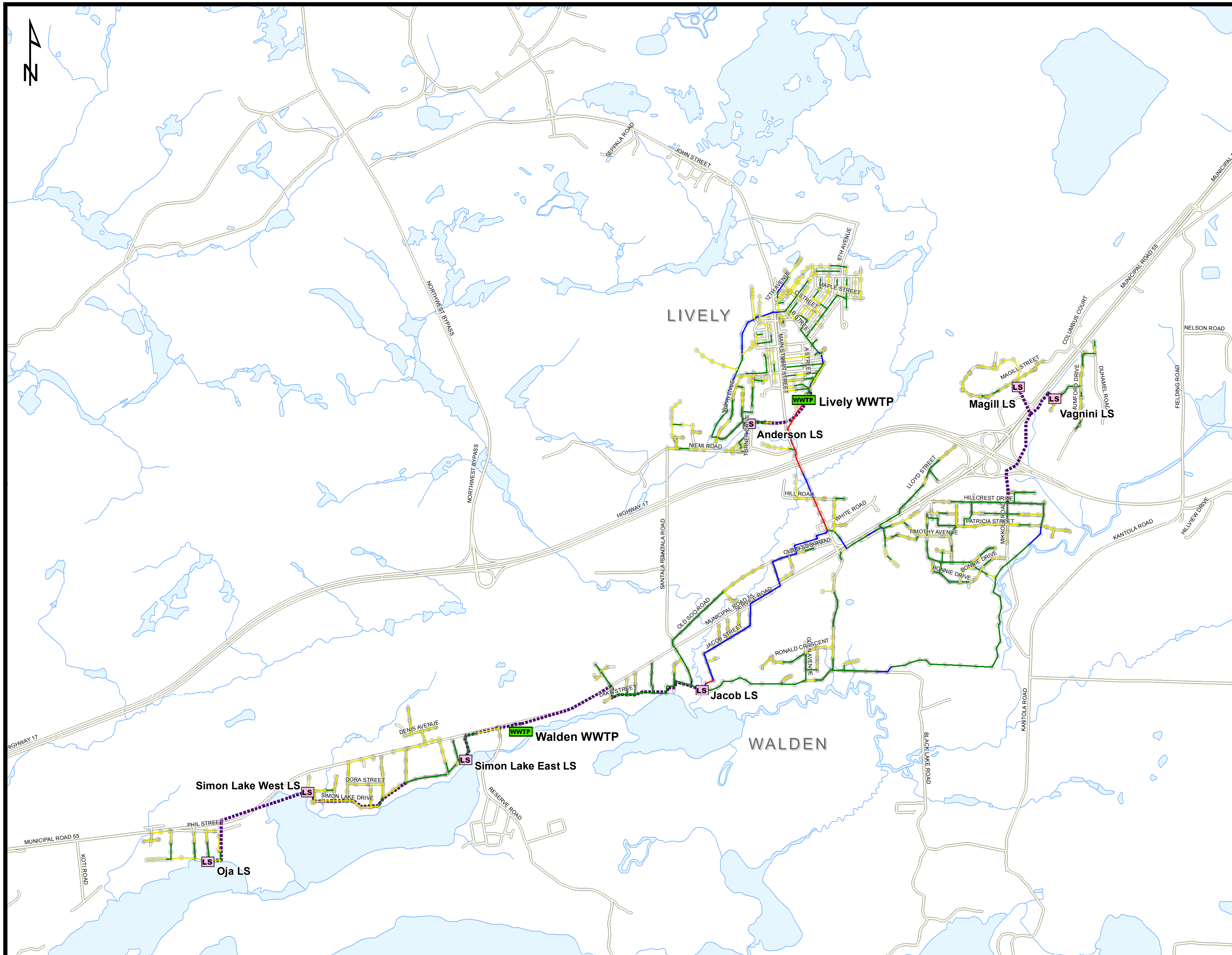
Map 4.1

Lively and Walden Wastewater System - Conduit Flow/Capacity (Design)

Scenario: 2041WWF_2yrStorm



Legend

LS Lift Station

WWTP Sanitary Treatment Plant

Manhole

Waterbody

Road

Watercourse

Forcemain

Conduit: Velocity (m/s)

<= 0.6

<= 1.5

<= 3.0

> 3.0

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Sudbury Water and Wastewater Master Plan

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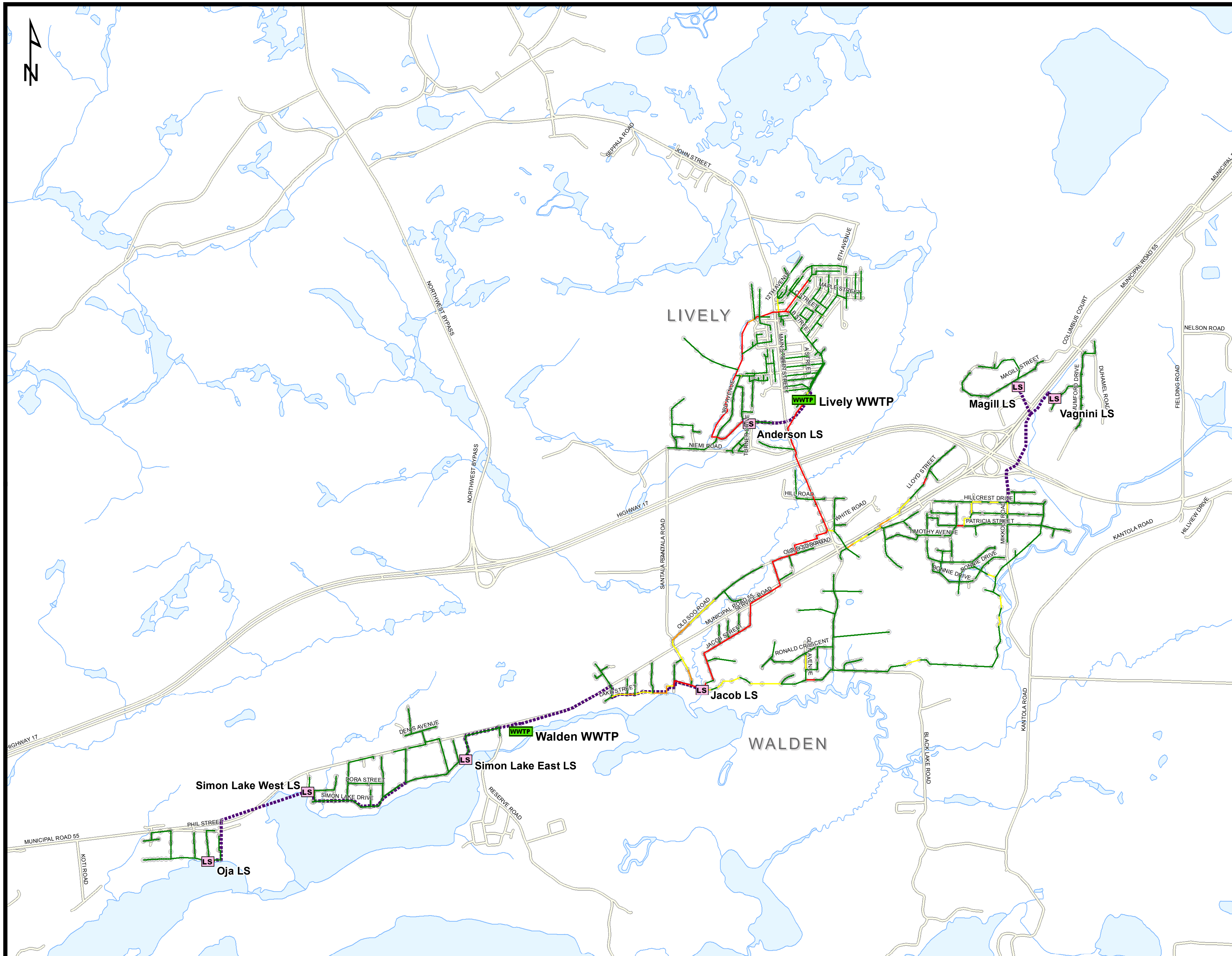
Map 4.2

Lively and Walden Wastewater System - Conduit Velocity

Scenario: 2041WWF_2yrStorm



Legend

- LS Lift Station
- WWTP Sanitary Treatment Plant
- Manhole
- Waterbody
- Road
- Watercourse
- Forcemain

Conduit: Flow/Capacity (Design) (%)

- <= 50
- <= 80
- <= 100
- > 100

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Sudbury Water and Wastewater Master Plan

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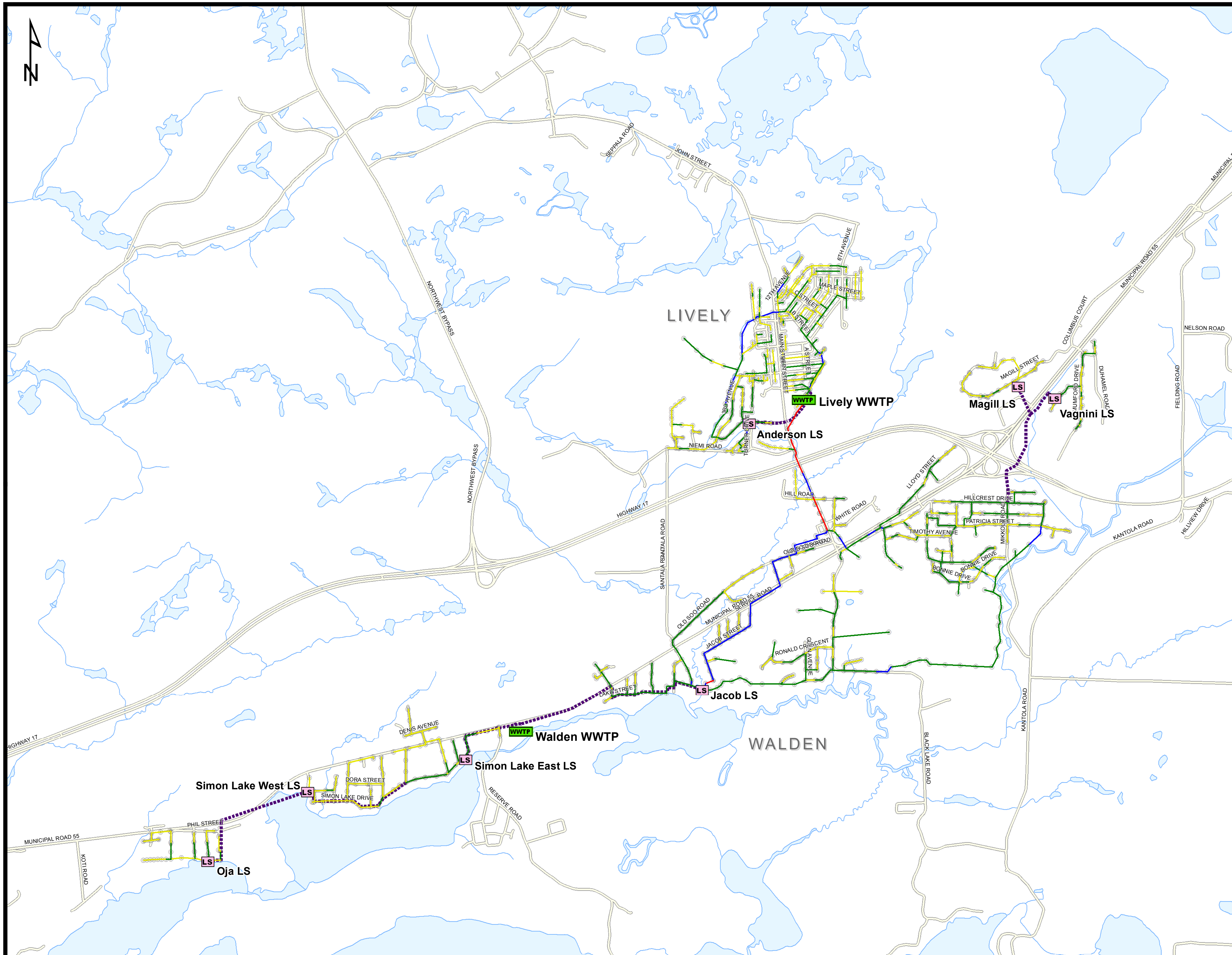
Map 5.1

Lively and Walden Wastewater System - Conduit Flow/Capacity (Design)

Scenario: ULTWWF_2yrStorm



Legend

- LS Lift Station
- WWTP Sanitary Treatment Plant
- Manhole
- Waterbody
- Road
- Watercourse
- Forcemain

Conduit: Velocity (m/s)

- <= 0.6
- <= 1.5
- <= 3.0
- \> 3.0

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Project No. 121-23026-00

December 09 2016

Sudbury Water and Wastewater Master Plan

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

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Map 5.2

Lively and Walden Wastewater System - Conduit Velocity

Scenario: ULTWWF_2yrStorm

APPENDIX

C WASTEWATER GENERATION AND CAPACITY ASSESSMENTS

Lively - Wastewater Flow Forecasts

(Lively WWTP Service Area)

		2009	2010	2011	2012	2013	Summary	Design Criterion	Comments
Average Day Flow	(m^3/d)	1,143	923	1,025	1,129	1,693	1,183	1,183	From Annual Reports
Max Day Flow	(m^3/d)	2,860	5,190	3,912	5,227	6,207	4,679	4,679	From Annual Reports
Max Day Factor		2.50	5.62	3.82	4.63	3.67	4.05	4.05	Calculated - Max Day Flow divided by Average Day Flow
Peak Flow	(m^3/d)	Not Available						0	Peak hour flows were not available
Peak Flow	(L/s)							0	
Peak Flow Factor									
Population (Existing Areas)		2,197	2,197	2,197	2,197	2,197	2,197	2,197	
Ratio of Residential to Total Customers		0.77	0.77	0.77	0.77	0.77	0.77		This ratio is based on Water Billing Records for the area and is an approximation of the residential portion of demand.
Residential Share of Average Day Demand (m^3/d)		879	710	788	868	1302	909		
Residential Flow Unit Rate (m^3/cap/d)		0.400	0.323	0.359	0.395	0.593	0.414	0.450	
Average Institutional Flow Unit Rate (m^3/ha/d)								28.0	CGS design standards do not suggest average rates for institutional and commercial, but require that rates be developed based on usage. In this case, water demands rates were used and assume a conservative 1:1 ratio of water to sewage.
Average Commercial Flow Unit Rate (m^3/ha/d)								28.0	CGS design standards do not suggest average rates for institutional and commercial, but require that rates be developed based on usage. In this case, water demands rates were used and assume a conservative 1:1 ratio of water to sewage.
Average Industrial Flow Unit Rate (m^3/ha/d)								35.0	CGS design standards average rate for light industrial of 35 m^3/ha/d.
Average Extraneous Flow Unit Rate (m^3/ha/d)								11.23	From CGS Design Standards, peak rate for new developments divided by an assumed peaking factor of 3. This factor would be applied only to new developments, which are assumed to be leak-tight, and have minimal extraneous flow.

	2016	2021	2026	2031	2036	2041	Ultimate Buildout	Comments
Population (Existing Areas)	2,197	2,197	2,197	2,197	2,197	2,197	2,197	
Population (Growth Areas)	152	294	410	479	520	531	2,957	
Total Population	2348	2491	2607	2676	2716	2728	5154	Total Population (Hemson)
Residential (ha)	9.46	9.46	70.50	70.50	70.50	70.50	70.50	
Institutional (ha)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	
Commercial (ha)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	No ICI development is expected within the Lively WWTP catchment area.
Industrial (ha)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	
ICI (ha)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	
Total (ha)	9.46	9.46	70.50	70.50	70.50	70.50	70.50	

	2016	2021	2026	2031	2036	2041	Ultimate Buildout	Comments
Average Residential Flows (m^3/d) - Existing	1,183	1,183	1,183	1,183	1,183	1,183	1,183	This includes all contribution from existing ICI and infiltration. The base flow was assumed to be the average day flow to the plant for the 2011-2013 period.
Average Residential Flows (m^3/d)	68	132	185	216	234	239	1,331	Obtained by multiplying the projected population growth by the unit rate.
Average Institutional Flow (m^3/d)	0	0	0	0	0	0	0	Institutional growth area multiplied by unit flow rate.
Average Commercial Flow (m^3/d)	0	0	0	0	0	0	0	Commercial growth area multiplied by unit flow rate.
Average Industrial Flow (m^3/d)	0	0	0	0	0	0	0	Industrial growth area multiplied by unit flow rate.
Average Extraneous Flow (m^3/day)	106	106	792	792	792	792	792	
Average Day Flow (m^3/d)	1,357	1,421	2,159	2,190	2,208	2,214	3,305	
Max Day Flow (m^3/d)	5,493	5,753	8,739	8,865	8,939	8,960	13,379	

ALTERNATIVE CALCULATION METHOD

Per Capita Flow (m3/cap/day)	0.520	0.420	0.467	0.514	0.771	0.538	0.538

	2016	2021	2026	2031	2036	2041	Ultimate Buildout	Comments
Average Day Flow (m^3/d)	1264	1341	1404	1440	1462	1469	2775	Multiplying the total population by the consolidated per capita flow factor.
Max Day Flow (m^3/d)	5,117	5,428	5,681	5,830	5,919	5,944	11,231	

SENSITIVITY ANALYSIS

Analyze sensitivity of forecasted flows to unit rate

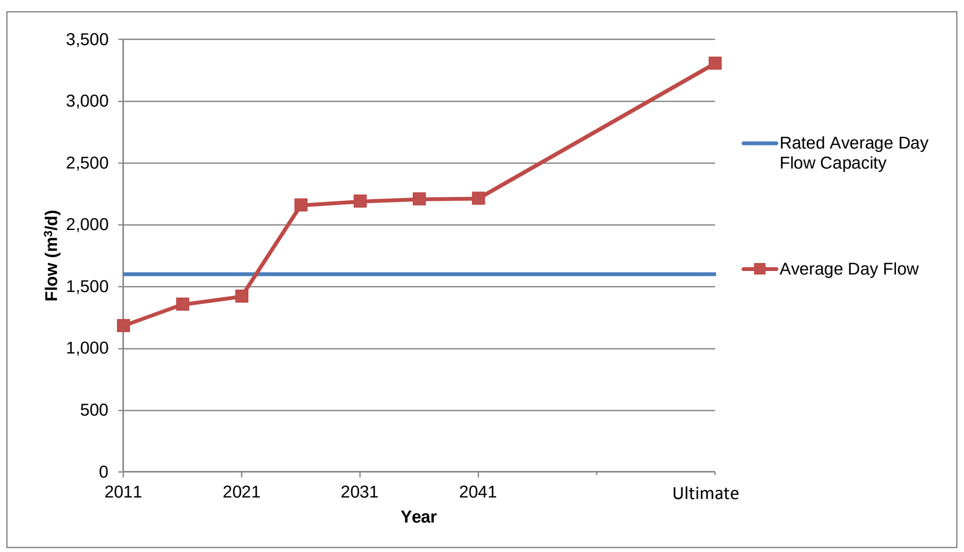
	Unit Rate ($m^3/cap/d$)	Average Day Flow (m^3/d) 2016	2021	2026	2031	2036	2041	Ultimate Buildout 2061
Base Scenario - Residential Historical Maximum	0.414	1,357	1,421	2,159	2,190	2,208	2,214	3,305
Combined Historical Maximum	0.538	1,370	1,447	2,195	2,232	2,254	2,260	3,567
City Standards for Lively	0.410	1,351	1,410	2,143	2,171	2,188	2,192	3,187
City Standards for Walden	0.450	1,357	1,421	2,159	2,190	2,208	2,214	3,305

Analyze sensitivity of forecasted flows to max day factor

	Max Day Peaking Factor	Max Day Flow (m^3/d) 2016	2021	2026	2031	2036	2041	Ultimate Buildout 2061
Base Scenario - Historical Max	4.43	6,017	6,302	9,574	9,711	9,792	9,815	14,656
Historical Average	5.62	5,493	5,753	8,739	8,865	8,939	8,960	13,379

CAPACITY CHECK

	2011	2016	2021	2026	2031	2036	2041	Ultimate Buildout 2061	Comments
Rated WPCP ADF Capacity (m^3/d)	1,600	1,600	1,600	1,600	1,600	1,600	1,600	1,600	Proposed capacity for the Walden WWTP from Lively/Walden Class EA ESR, page 75
WPCP Peak Capacity (m^3/d)	3,000	3,000	3,000	3,000	3,000	3,000	3,000	3,000	
Average Day Flow (m^3/d)	1,183	1,357	1,421	2,159	2,190	2,208	2,214	3,305	
Maximum Day Flow (m^3/d)	4,679	5,493	5,753	8,739	8,865	8,939	8,960	13,379	



Walden - Wastewater Flow Forecasts

(Walden WWTP Service Area)

		2009	2010	2011	2012	2013	Summary	Design Criterion	2016	2021	2026	2031	2036	2041	Ultimate Buildout	Comments
Average Day Flow	**(m^3/d)**	3,979	3,079	3,258	3,317	4,252	3,577	3,577								From Annual Reports
Max Day Flow	**(m^3/d)**	11,610	10,106	11,199	12,693	14,273	11,976	11,976								From Annual Reports
Max Day Factor		2.92	3.28	3.44	3.83	3.36	3.36	3.36								Calculated - Max Day Flow divided by Average Day Flow
Peak Flow	**(m^3/d)**	Not Available						0								Peak hour flows were not available
Peak Flow	**(L/s)**							0								
Peak Flow Factor																
Population (Existing Areas)		5,178	5,178	5,178	5,178	5,178	5,178	5,178	5,178	5,178	5,178	5,178	5,178	5,178	5,178	
Population (Growth Areas)									322	626	880	1,031	1,120	1,146	5,998	
Total Population									5,501	5,804	6,059	6,209	6,299	6,324	11,177	Total Population (Hemson)
Residential (ha)									72.60	76.50	141.90	143.10	144.34	144.34	269.24	ICI development areas were assigned to planning years based on the stage of the application. Draft Approved were assigned to 2016, Legal Lots of Record to 2026, and Designated Developable to 2036. Areas are cumulative and carry from the development year, all the way through to Ultimate Buildout
Institutional (ha)									0.00	0.00	1.12	0.00	0.85	0.00	0.00	
Institutional Cummulative (ha)									0.00	0.00	1.12	1.12	1.97	1.97	1.97	
Commercial (ha)									0.00	0.00	1.50	0.00	8.21	0.00	0.00	
Commercial Cummulative (ha)									0.00	0.00	1.50	1.50	9.71	9.71	9.71	
Industrial - Walden Industrial Area - Total Undeveloped and Unserviced Land (ha)									0.00	0.00	72.0	0.0	0.0	0.0	0.0	
Industrial - Walden Industrial Area - 20% (ha)									0.00	0.00	14.4	0.0	0.0	0.0	0.0	
Industrial - Walden Industrial Area - 80% (ha)									0.00	0.00	57.6	0.0	0.0	0.0	0.0	
Industrial - Walden Industrial Area - Total Developed and Unserviced Land (ha)									0.00	0.00	0.0	100.0	0.0	0.0	0.0	
Industrial - Balance of Industrial Area (ha)									0.00	0.00	2.4	0.0	3.2	0.0	0.0	
Total Industrial (ha)*									0.00	0.00	74.41	100.00	3.19	0.00	0.00	
Industrial Cummulative - Walden Industrial Area - Total Undeveloped and Unserviced Land (ha)									0.0	0.0	72.0	72.0	72.0	72.0	72.0	
Industrial Cummulative - Walden Industrial Area - 20% (ha)									0.0	0.0	14.4	14.4	14.4	14.4	14.4	
Industrial Cummulative - Walden Industrial Area - 80% (ha)									0.0	0.0	57.6	57.6	57.6	57.6	57.6	
Industrial Cummulative - Walden Industrial Area - Total Developed and Unserviced Land (ha)									0.0	0.0	0.0	100.0	100.0	100.0	100.0	
Industrial Cummulative - Balance of Industrial Area (ha)									0.0	0.0	2.4	2.4	5.6	5.6	5.6	
Industrial Growth Area (ha) - Cumulative									0.0	0.0	74.4	174.4	177.6	177.6	177.6	

**Includes 100 ha of land that is currently developed but not serviced (to be serviced by 2031)*

		2009	2010	2011	2012	2013	Summary	Design Criterion	2016	2021	2026	2031	2036	2041	Ultimate Buildout	Comments
ICI Cummulative (ha)									0.00	0.00	77.03	177.03	189.28	189.28	189.28	
Total Cummulative (ha)									72.60	76.50	218.93	320.13	333.62	333.62	458.52	
Ratio of Residential to Total Customers		0.77	0.77	0.77	0.77	0.77	0.77									This ratio is based on Water Billing Records for the area and is an approximation of the residential portion of demand.
Residential Share of Average Day Demand (m^3/d)		3059	2367	2505	2550	3269	2750									
Residential Flow Unit Rate (m^3/cap/d)		0.591	0.457	0.484	0.492	0.631	0.531	0.450								
Average Institutional Flow Unit Rate (m^3/ha/d)								28.0								CGS design standards do not suggest average rates for institutional and commercial, but require that rates be developed based on usage. In this case, water demands rates were used and assume a conservative 1:1 ratio of water to sewage.
Average Commercial Flow Unit Rate (m^3/ha/d)								28.0								CGS design standards do not suggest average rates for institutional and commercial, but require that rates be developed based on usage. In this case, water demands rates were used and assume a conservative 1:1 ratio of water to sewage.
Average Industrial Flow Unit Rate - Balance of Industrial Lands (m^3/ha/d)								35.0								CGS design standards average rate for light industrial of 35 m^3/ha/d.
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								
Average Extraneous Flow Unit Rate (m^3/ha/d)								6.02								From CGS Design Standards, peak rate for new developments divided by an assumed peaking factor of 3. This factor would be applied only to new developments, which are assumed to be leak-tight, and have minimal extraneous flow.

	2016	2021	2026	2031	2036	2041	Ultimate Buildout	Comments
Average Residential Flows (m^3/d) - Existing	3,577	3,577	3,577	3,577	3,577	3,577	3,577	This includes all contribution from existing ICI and infiltration. The base flow was assumed to be the average day flow to the plant for the 2011-2013 period.
Average Residential Flows (m^3/d)	145	282	396	464	504	515	2,699	Obtained by multiplying the projected population growth by the unit rate.
Average Institutional Flow (m^3/d)	0	0	31	0	24	0	0	Institutional growth area multiplied by unit flow rate.
Average Commercial Flow (m^3/d)	0	0	42	0	230	0	0	Commercial growth area multiplied by unit flow rate.

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	196	196	196	Industrial growth area multiplied by unit flow rate.
Average Extraneous Flow (m^3/day)	437	460	1,317	1,926	2,007	2,007	2,759	
Average Day Flow (m^3/d)	4,159	4,319	6,125	7,028	7,515	7,273	10,208	
Max Day Flow (m^3/d)	13,991	14,530	20,605	23,644	25,282	24,466	34,341	

ALTERNATIVE CALCULATION METHOD

Per Capita Flow (m3/cap/day)	0.768	0.595	0.629	0.641	0.821	0.691	0.691

	2016	2021	2026	2031	2036	2041	Ultimate Buildout	Comments
Average Day Flow (m^3/d)	3800	4009	4185	4289	4351	4368	7720	Multiplying the total population by the consolidated per capita flow factor.
Max Day Flow (m^3/d)	12,782	13,489	14,080	14,430	14,637	14,696	25,972	

SENSITIVITY ANALYSIS

Analyze sensitivity of forecasted flows to unit rate

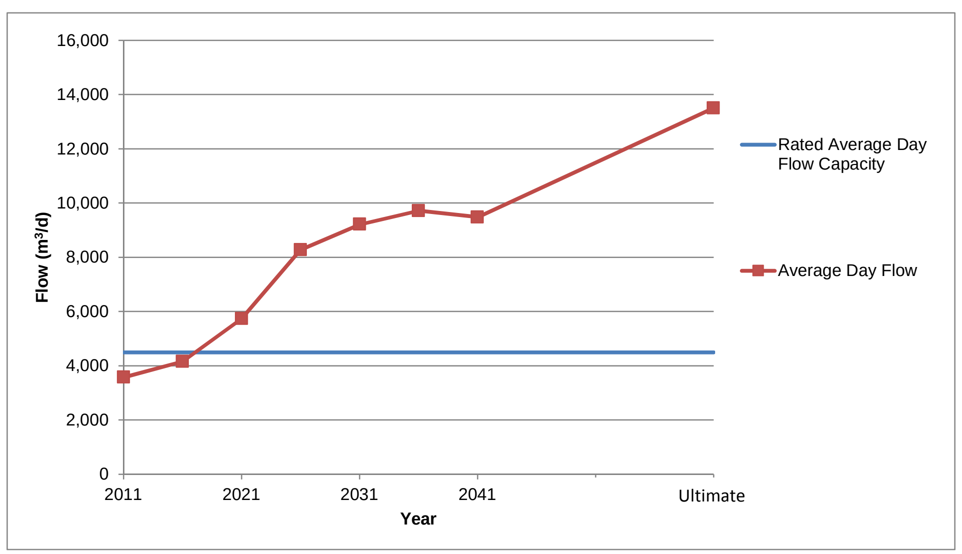
	Unit Rate (m^3/cap/d)	Average Day Flow (m^3/d) 2016	2021	2026	2031	2036	2041	Ultimate Buildout 2061
Base Scenario - Residential Historical Maximum	0.531	4,159	4,319	6,125	7,028	7,515	7,273	10,208
Combined Historical Maximum	0.691	4,236	4,470	6,337	7,276	7,785	7,548	11,652
City Standards for Lively	0.410	4,146	4,294	6,090	6,987	7,470	7,227	9,968
City Standards for Walden	0.450	4,159	4,319	6,125	7,028	7,515	7,273	10,208

Analyze sensitivity of forecasted flows to max day factor

	Max Day Peaking Factor	Max Day Flow (m^3/d) 2016	2021	2026	2031	2036	2041	Ultimate Buildout 2061
Base Scenario - Historical Max	3.48	14,455	15,012	21,289	24,429	26,120	25,278	35,480
Historical Average	3.83	13,991	14,530	20,605	23,644	25,282	24,466	34,341

CAPACITY CHECK

	2011	2016	2021	2026	2031	2036	2041	Ultimate Buildout 2061	Comments
Rated WPCP ADF Capacity (m^3/d)	4,500	4,500	4,500	4,500	4,500	4,500	4,500	4,500	Proposed capacity for the Walden WWTP from Lively/Walden Class EA ESR, page 75
WPCP Peak Capacity (m^3/d)	8,000	8,000	8,000	8,000	8,000	8,000	8,000	8,000	
Average Day Flow (m^3/d) - Walden	3,577	4,159	4,319	6,125	7,028	7,515	7,273	10,208	
Average Day Flow (m3/d) - Lively			1,421	2,159	2,190	2,208	2,214	3,305	
Total Average Day Flow To Walden WWTP	3,577	4,159	5,740	8,284	9,218	9,723	9,486	13,513	
Maximum Day Flow (m^3/d) - Walden	11,976	13,991	14,530	20,605	23,644	25,282	24,466	34,341	



CITY OF GREATER SUDBURY WATER AND WASTEWATER MASTER PLAN

SUDBURY WASTEWATER SYSTEM GAP ANALYSIS AND STATUS QUO REPORT

CITY OF GREATER SUDBURY

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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 existing Sudbury Wastewater System. Based on population growth projections and design criteria discussed in the *Population Projections and Development of Unit Rates Technical Memorandum* (WSP, 2014), 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 Sudbury Wastewater System will continue to be a stand-alone system. Additional information on the existing wastewater system is provided in the Baseline Review Report for Wastewater Systems (WSP, 2014).

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

Sudbury is located in the central portion of the City of Greater Sudbury and it is the most populated community. Wastewater generated in Garson and Sudbury is treated at the Sudbury Wastewater Treatment Plant (WWTP). However, during wet weather emergencies, flows generated in Garson may be diverted to the Garson Lagoons.

Mapping 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 *Population Projections and Development of Unit Rates Technical Memorandum* (WSP, 2014). An analysis of future wastewater flows is provided in Appendix C.

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

Based on the City's planning data, the Sudbury and Garson wastewater servicing area population is expected to increase from 91,246 in 2011 to 95,739 by 2041 and 121,886 by Ultimate Buildout.

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

3 OVERVIEW OF EXISTING SYSTEM

The Sudbury Wastewater System services the community of Sudbury and Garson, including the Sudbury WWTP and the Garson Lagoons. Wastewater generated in Garson is typically treated at the Sudbury WWTP; however, the lagoons are used occasionally in cases of wet weather emergencies.

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

The Sudbury Wastewater System is shown in Appendix B.

3.1 LIFT STATIONS AND SEWER NETWORK

The system includes twenty-seven lift stations (LS) in the Sudbury Wastewater System and three in Garson, as well as approximately 433 km of sewers and forcemains. There are also several recently decommissioned lift stations:

— Old-Burwash LS
— Paris LS
— Steward LS
— Walford West LS
— Green LS
— Oriole LS

Within the Garson conveyance system, all wastewater flows generated in within the community are conveyed to the O'Neil LS. The O'Neil LS normally operates by allowing the wet well to overflow; the overflow pipe conveys flows by gravity to the Sudbury WWTP. During wet weather emergencies, the wet well is pumped out and the forcemain discharges to the Garson Lagoons. When the wet weather emergency subsides, a valve is manually opened and the lagoon is allowed to drain by gravity back to the O'Neil LS, and wastewater is conveyed to the Sudbury WWTP for treatment.

Table 3-1 below provides a summary of the main features for the 30 lift stations in use within Sudbury and Garson. From left to right, the contents of the table are as follows:

— Lift Station (Status): Name of the facility and whether it is in operation, seasonal or decommissioned.
— Year Built: Year the wet well and primary works were originally constructed based on the 2013 Operations Manual and/or Certificate of Approval.
— Last Upgrade: Year the LS or forcemain were last upgraded or refurbished, if available.
— Wet Well Volume: Active volume available in the wet well without endangering pumps (low level, below which volume is ineffective) or causing an overflow to a pipe or ground. In cases where LS have a top slab, about 0.3 m below that level was assumed to be the maximum level.
— LS Capacity and Forcemain Data: Number of pumps and their flow capacity at the heads expected based on the 2013 Operations Manual and/or as-builts; as modelled in SewerGEMS. Key forcemain data includes length, nominal diameter and material.

Table 3-1 Sudbury and Garson Wastewater Lift Stations

LIFT STATION (STATUS)	YEAR BUILT[1]	LAST UPGRADE[2]	WET WELL VOLUME (M^3)[1]	LIFT STATION CAPACITY AND FORCEMAIN DATA[2]
Garson Lift Stations				

Gar-Con (In Operation)	1978	N/A	363.8	Two submersible pumps with a firm design capacity of 24.3 L/s 1,073 m long, 150 mm diameter PVC forcemain
O'Neil (Seasonal / Used under high flow conditions)	1965[3]	1975	485.1	Two dry well pumps with a firm design capacity of 98.6 L/s 674 m long, 250 mm diameter forcemain of unknown material
Penman (In Operation)	1975[3]	N/A	478.78	One submersible pumps with a design capacity of 8.3 L/s 258 m long, 150 mm diameter forcemain of unknown material
Sudbury Lift Stations				
Bell Park (Seasonal only used when park is open)	1978	N/A	108.5	Two submersible pumps[1] 448 m long, 100 mm diameter polyethylene forcemain
Beverly (In Operation)	1960	N/A	364.5	Two dry well pumps with a firm design capacity of 28.8 L/s 450 m long, 200 mm diameter forcemain of unknown material
Brenda (In Operation)	1988	N/A	432.9	Two submersible pumps with a firm design capacity of 13.30 L/s 629 m long, 150 mm diameter forcemain of unknown material
Cerilli (In Operation)	1979	N/A	227.8	Two submersible pumps with a firm design capacity of 14 L/s 181 m long, 100 mm diameter forcemain of unknown material
Countryside[4] (In Operation)	1991	N/A	212.43	Two submersible pumps with a firm design capacity of 7.6 L/s 507 m long, 75 mm diameter forcemain of unknown material 507 m long, 200 mm diameter forcemain of unknown material.
Don Lita (In Operation)	1967	N/A	547.2	Two dry well pumps with a firm design capacity of 30.3 L/s 753 m long, 200 mm diameter cement-lined cast iron forcemain
Dufferin (In Operation)	1974[3]	N/A	≥ 95.3[4]	One submersible pumps with a design capacity of 6.4 L/s 71 m long, 100 mm diameter forcemain of unknown material

Ester (In Operation)	1980	N/A	312.2	Two submersible pumps with a firm design capacity of 28.4 L/s 211 m long, 150 mm diameter forcemain of unknown material
Fourth (In Operation)	1980	2000	538.6	Two submersible pumps with a firm design capacity of 15.2 L/s 262 m long, 150 mm diameter DR-26 PVC forcemain
Helen's Point (In Operation)	1979	N/A	216.9	Two submersible pumps with a firm design capacity of 7.6 L/s 383 m long, 100 mm diameter DR-26 PVC forcemain
Kincora (In Operation)	1970***	N/A	96.1	Two submersible pumps with a firm design capacity of 8.7 L/s 308 m long, 100 mm diameter asbestos cement forcemain
Lagace (In Operation)	1946[3]	N/A	324.1	Two dry well pumps with a firm design capacity of 14 L/s 3 m long, 100 mm diameter cast iron forcemain
Lakeview (In Operation)	1946[3]	N/A	362.83	Two dry well pumps with a firm design capacity of 20.9 L/s 91 m long, 150 mm diameter asbestos cement forcemain
Levesque (In Operation)	1967	N/A	846.0	Two dry well pumps with a firm design capacity of 167.6 L/s 1,043 m long, 400 mm diameter forcemain of unknown material
Loach's Road (In Operation)	1960	N/A	≥ 137.2[4]	Two submersible pumps with a firm design capacity of 12.1 L/s 275 m long, 100 mm diameter DR-26 PVC forcemain
Marcel Bouchard (Decommissioned; used for emergencies only; flows bypassing this LS are conveyed directly to the tunnel)	1972	N/A	N/A	Two submersible pumps with a firm design capacity of 303.3 L/s 61 m long, 750 mm diameter forcemain of unknown material
Mark (In Operation)	1966[3]	1999	425.0	Two dry well pumps with a firm design capacity of 41.7 L/s 384 m long, 150 mm diameter forcemain of unknown material

Moonlight (In Operation)	1967	N/A	≥ 182.9[4]	Two dry well pumps with a firm design capacity of 16.3 L/s 415 m long, 150 mm diameter forcemain of unknown material
Moonlight Beach (Seasonal)	1970***	N/A	N/A	Two submersible pumps 514 m long, 150 mm diameter asbestos cement forcemain
Northshore (In Operation)	1962***	N/A	≥ 10.97[4]	Two submersible pumps with a firm design capacity of 11.4 L/s 383 m long, 150 mm diameter forcemain of unknown material
Ramsey (In Operation)	1984	N/A	592.0	Two submersible pumps with a firm design capacity of 32.2 L/s 200 mm for 2.4 m then increasing to 250 mm for 789 m DR-26 forcemain
St. Charles (In Operation)	1930	2012[3] (odour control)	≥ 166.7[4]	Two submersible pumps with a firm design capacity of 383 L/s 888 m long, 400 mm diameter cast iron forcemain
Selkirk (In Operation)	1984[3]	1994	490.1	Two dry well pumps with a firm design capacity of 38.7 L/s 330 m long, 150 mm diameter cast iron (with mechanical joints) forcemain
Sherwood (In Operation)	1966[3]	1974	373.1	Two dry well pumps with a firm design capacity of 30 L/s 330 m long, 150 mm diameter forcemain of unknown material
Southview (In Operation)	1964[3]	N/A	380.4	Two dry well pumps with a firm design capacity of 58.8 L/s 589 m long, 300 mm diameter forcemain of unknown material
Walford East (In Operation)	1960	1971	580.34	Two dry well pumps with a firm design capacity of 127 L/s 449 m long, 300 mm diameter forcemain of unknown material
York (In Operation)	1972[3]	1980	446.23	Two submersible pumps with a firm design capacity of 13.2 L/s 244 m long, 100 mm diameter cast iron forcemain

[1] Obtained or estimated from dimensions found in as-built and record drawings, assuming water level does not exceed the High Water Alarm Level or, in absence of this alarm level, the inlet sewer invert.

[2] Obtained from the City's Wastewater Lift Stations Operations Manual and station as-built drawings.

[3] Obtained from the City's PSAB Database.

[4] Based on 2013 Operations Manual. Likely for a 0.3 m operating range. Effective capacity likely 3 to 10 times larger.

3.2 WASTEWATER ROCK TUNNEL SYSTEM

Sudbury has varied topography and bedrock geology. As such, it is challenging to convey flows by gravity to the WWTP using conventional sewers due to the potential need for deep construction in rock. Over time, as development in Sudbury progressed, the need for wastewater collection and treatment became necessary. In the 1930s and 1940s, the City's first collection system was constructed and it included gravity sewers as well as lift stations discharging directly to Junction Creek. As environmental awareness rose in the 1960s, the City's first wastewater tunnel was constructed, followed by the Sudbury WWTP in the 1970s.

Over time, more and more lift stations were constructed to pump flows to the WWTP. Expansion of the rock tunnel system allowed the City to collect and convey flows by gravity to the WWTP and provided the opportunity to decommission lift stations that were no longer needed. Currently, there are four rock tunnels servicing Sudbury:

- Main Tunnel: the original tunnel constructed in the 1960s
- Minnow Lake Tunnel: an expansion of the original tunnel, constructed in the 1970s
- Lockerby Tunnel: an expansion of the original tunnel, constructed in the 1970s
- South End Tunnel: latest expansion, connected to the end of the Lockerby Tunnel, constructed in the late 2000s.

The location of each tunnel is shown in Figure 3-1.

As tunnels were brought online, some lift stations were eliminated. This allowed for more streamlined and efficient operation of the sanitary system.

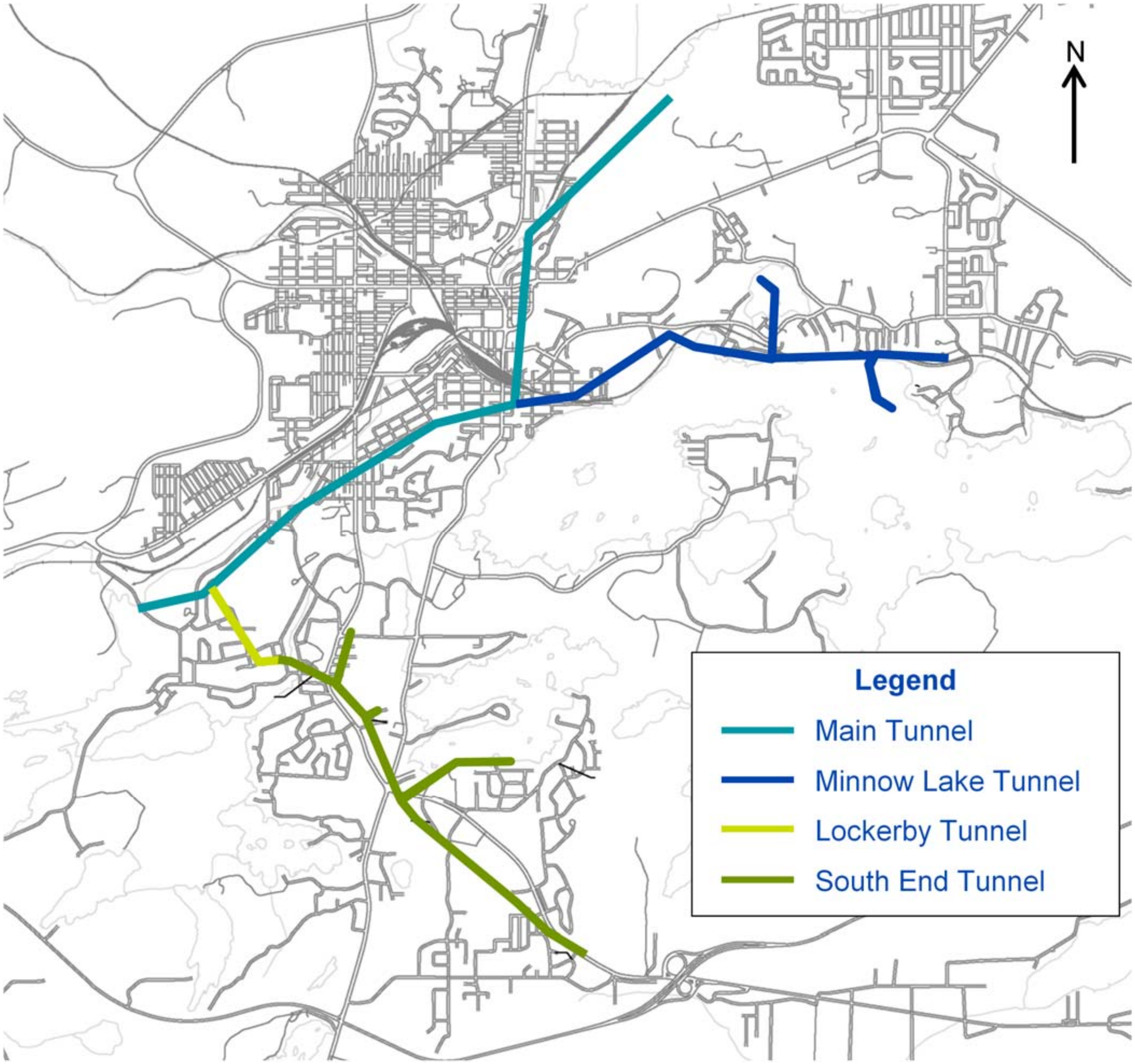

Figure 3-1 Locations of Rock Tunnels

3.3 GARSON LAGOONS

Historically, the Garson Wastewater Lagoons provided wastewater treatment for flows generated by the community of Garson. It is located at Lot 7 Concession 2 within the City of Greater Sudbury. The original lagoons have been in operation since the 1960s. However, no effluent has been discharged from the lagoons since the fall of 2007 due to the lagoons' inability to treat wastewater flows to the required effluent criteria. Specifically, the lagoons were not able to meet acceptable phosphorus and suspended solids levels. Odour concerns at the lagoons were also a problem.

Since 2007 all wastewater generated in the Garson system has been diverted to the Sudbury Wastewater Treatment Plant, with the option to send flow to the Garson Lagoons in the summer in case of emergency or during wet weather flow events. This option will be further discussed in Volume 5 as simulations suggest significant advantages in the event of a significant wet weather flow event.

3.4 SUDBURY WWTP

The Sudbury WWTP is a conventional activated sludge plant. Currently, Phase 1 upgrades have been implemented, resulting in a rated capacity of 79,625 m^3/d and peak flow rate of 159,250 m^3/d.

There are plans (included in the WWTP's Certificate of Approval) to upgrade the plant using moving bed biofilm reactor (MBBR) technology and add tertiary treatment to ensure the plant can treat an increased influent volume while meeting more stringent phosphorus limit requirements. Once the Phase 2 expansion is complete, the plant's rated capacity will increase to 102,375 m^3/d and peak flow rate of 204,750 m^3/d. A process flow schematic of the liquid treatment train after Phase 2 expansion at the Sudbury WWTP is presented in Figure 3-2.

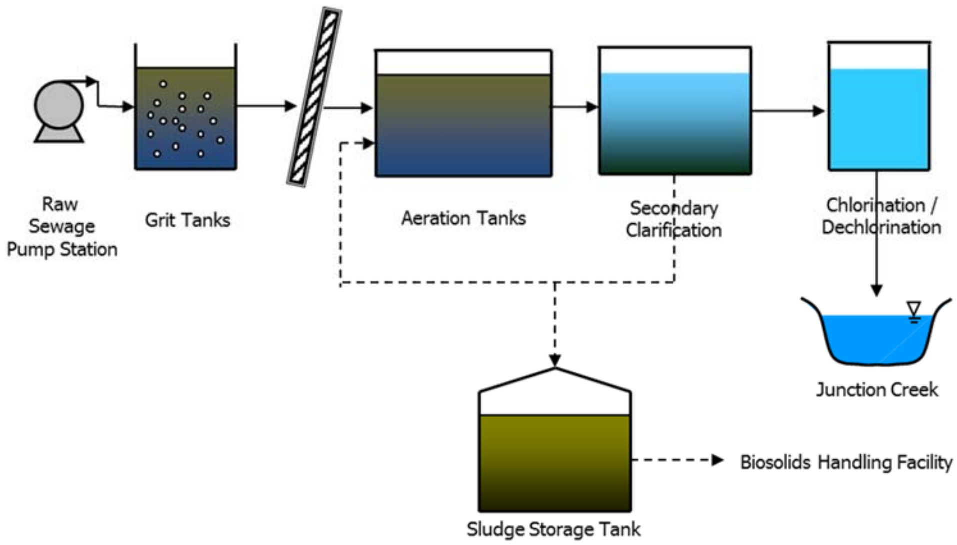

Figure 3-2 Sudbury WWTP Process

3.5 KNOWN CHALLENGES

3.5.1 SUDBURY SYSTEM

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

- Sudbury is known to experience sewer surcharging and overland flooding events that can cause flow to enter the sewer. Sewer surcharges were mostly concentrated in the north-central and north-east areas, whereas the flooding events were evenly distributed, with some clusters concentrated in the north-west area. The City has also noted that the downtown and the Flour Mill area are known to experience flooding events.
- The City noted that there are many challenges with regards to the linear infrastructure within Sudbury.
  - Sewers in older neighborhoods tend to be in poor condition; some have tree roots growing inside of them.
  - Access to many sewers in the downtown area is limited.
  - There are numerous old, clay sewer mains and laterals. Their condition contributes to I/I.

- There are direct connections to the sewers in downtown and older areas of Sudbury.
  - Several large diameter sewers are located in easements and access for maintenance can be challenging or not possible.
  - There have been issues with regards to commercial and industrial users within the network. The City has reported garbage and grease build up at the Sudbury WWTP plant and has in recent years implemented a sewer use by-law to enforce limitations on industrial/commercial inputs into the sewer system.
- Several bypasses have been reported at facilities within Sudbury. From 2004 to 2011, 41 spills were reported at the Sudbury WWTP and from 2009 to 2011 four overflow/sewer bypass events were reported at both the Stewart LS and Moonlight LS as well as one bypass at the Green LS. However, the Stewart and Green LS's have since been decommissioned.
- The hydraulic capacity of the rock tunnel is not fully understood. The City is completing surveys but they are costly and potentially dangerous due to confined space entry requirements.
- There are recognized issues with I/I in the system. The difference between dry weather flows and wet weather flows is significant.

3.5.2 GARSON SYSTEM

There have been no reported spills at the LSs or at the lagoons in Garson. The City has indicated that the area near the Gar-Con LS has experienced flooding and sewer backups, a result of high inflow and infiltration in this part of the system.

3.6 BIOSOLIDS MANAGEMENT FACILITY AT THE SUDBURY WWTP

The City has recently commissioned a Biosolids Management Facility located at the Sudbury WWTP. The facility receives sludge hauled from each of the City's wastewater treatment plants and converts the material to biosolids. The project is a public-private partnership. The facility will be maintained and operated by a third party, N-Viro, for 20 years, the City retains full ownership of the facility for this entire duration.

4 HISTORICAL FLOWS AND REVIEW OF OPERATIONAL DATA

Data reported in the 2009 to 2013 *Annual Reports* for the Sudbury WWTP was reviewed and analyzed to determine average day and maximum day flows as well as review effluent parameters. A review of the historical and operational data for the Garson Lagoons was not completed because data was not available, and because all flows are ultimately treated at the Sudbury WWTP.

4.1 FLOW DATA

Flow data for the Sudbury WWTP from 2009 to 2013 was reviewed. Operational data was not available from the lift stations.

The recorded average day and maximum day flows are summarized in Table 4-1 and plotted in Figure 4-1 below.

Table 4-1 Historical Wastewater Flow Data

YEAR	AVERAGE DAY FLOW (M^3/D)[1]	MAXIMUM DAY FLOW (M^3/D)[1]
2009	65,033	228,500
2010	55,181	231,700
2011	58,207	258,400
2012	56,233	179,200
2013	64,710	257,300

[1]*Annual Reports (2009 - 2013).*

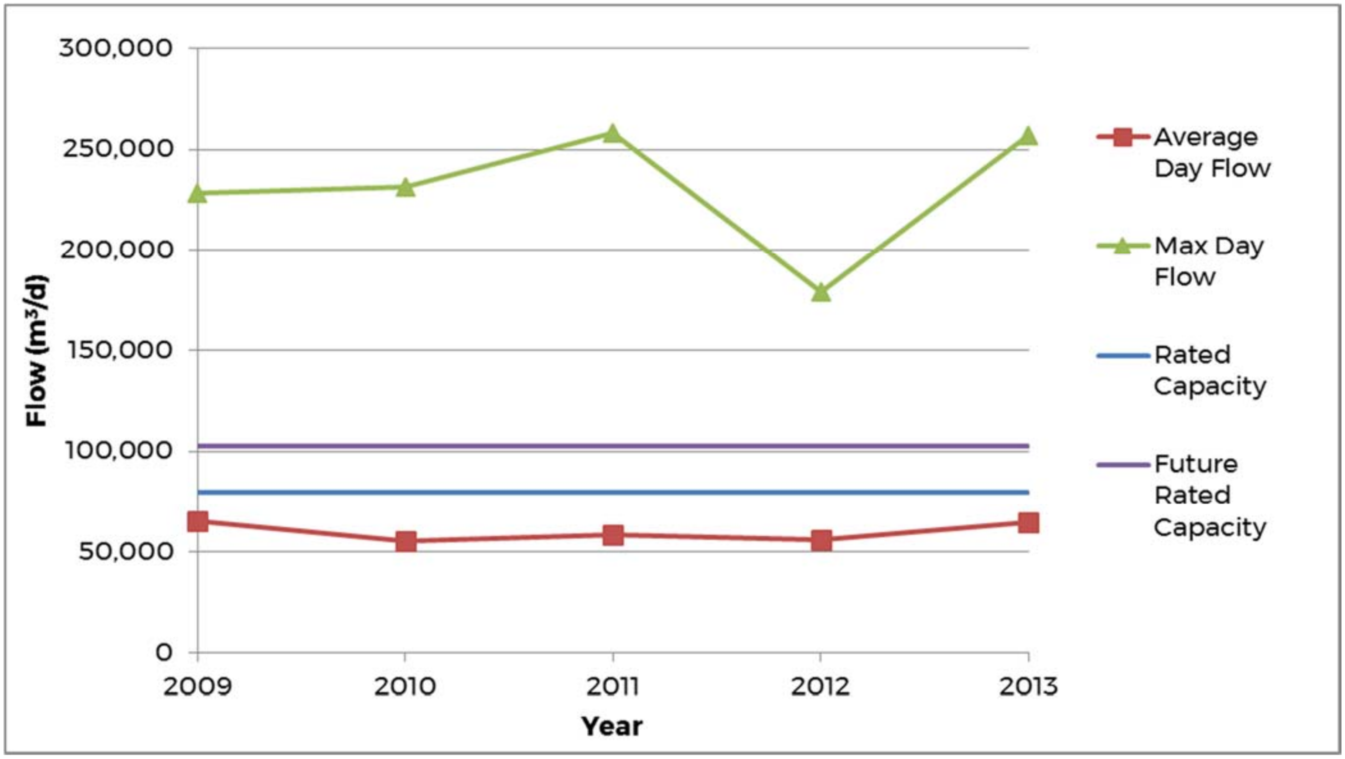

Figure 4-1 Historical Wastewater Flows at the Sudbury WWTP

The relationship between the different flow regimes was analyzed to compare the maximum day peaking factors derived from historical data to those used in the *City's Engineering Design Manual* and those included in the *MOE Guidelines*.

The average day flows to the WWTP have been consistent over the 2009 to 2013 period, averaging 59,873 m^3/d. The variations in historical maximum day flows show no discernible trend. The greatest maximum day flow occurred in 2011 and the average historical maximum day flow was 231,020 m^3/d.

The wide range in maximum day flows, but stable average day flows, indicates that the system is susceptible to variations in precipitation.

The highest maximum day to average day peaking factor based on the maximum day flow recorded in 2011 was 4.44. The average maximum day peaking factor from 2009 to 2013 was 3.86. The City's *Engineering Design Manual* and the *MOE Guidelines* do not specify recommended maximum day factors and recommend using historical data when available. For future wastewater generation, the average peaking factor was used and based on the assumption that new developments would have less I/I due to more leak tight construction.

4.2 RAW WASTEWATER CHARACTERISTICS

The average raw wastewater characteristics from 2009 to 2012 are summarized in Table 4-2 below. Raw wastewater temperatures were not reported.

Table 4-2 Average Raw Wastewater Characteristics (2009-2012)

PARAMETER	AVERAGE VALUE
$CBOD_5$	142 mg/L
Suspended Solids	206 mg/L

Total Phosphorus	3.0 mg/L
TKN	24.0 mg/L
pH	7.0

Wastewater flows to the Sudbury WWTP correspond mainly to residential uses, with contributions from commercial and industrial users.

4.3 EFFLUENT CRITERIA

The Sudbury WWTP is operated in accordance with *MOE Amended Environmental Compliance Approval (ECA) No. 1978-9CXQJL* dated May 27, 2014.

The *ECA* concentration and loading limits are summarized in Table 4-3 for the current plant operation. Future (Phase II) requirements, based on an average day flow of 102,375 m^3/day are listed in Table 4-4.

Table 4-3 Current (Phase I) Sudbury WWTP Effluent Limits and Objectives

EFFLUENT PARAMETER	CONCENTRATION LIMIT	LOADING LIMIT (ANNUAL AVERAGE)	CONCENTRATION OBJECTIVE / LOADING LIMIT
$CBOD_5$	25.0 mg/L (annual average)	1990.6 kg/d	15.0 mg/L / 1194.4 kg/d
Total Suspended Solids (TSS)	25.0 mg/L (annual average)	1990.6 kg/d	15.0 mg/L / 1194.4 kg/d
Total Phosphorus (TP)	Oct. to May: 1.0 mg/L Jun. to Sep.: 0.5 mg/L (monthly average)	Oct. to May: 79.6 kg/d Jun. to Sep.: 39.8 kg/d	0.5 mg/L / 39.8 kg/d
Total Ammonia Nitrogen (TAN)	-	-	7.0 mg/L / 557.4 kg/d
Total Residual Chlorine	0.02 mg/L (monthly average)	1.6 kg/d	Non-detectable
E. coli	200 organisms/100 mL (Monthly Geometric Mean Density)	-	100 organisms/100 mL (Monthly Geometric Mean Density)
pH	6.0 to 9.5	-	6.5 to 8.5

Table 4-4 Future (Phase II) Sudbury WWTP Effluent Limits and Objectives

EFFLUENT PARAMETER	CONCENTRATION LIMIT (MONTHLY AVERAGE)	LOADING LIMIT (ANNUAL AVERAGE)	CONCENTRATION OBJECTIVE / LOADING LIMIT
$CBOD_5$	5.0 mg/L	511.8 kg/d	4.0 mg/L / 409.5 kg/d
Total Suspended Solids (TSS)	10.0 mg/L	1023.8 kg/d	8.0 mg/L / 819.0 kg/d
Total Phosphorus (TP)	0.25 mg/L	25.6 kg/d	0.2 mg/L / 20.5 kg/d

Total Ammonia Nitrogen (TAN)	9.0 mg/L	921.4 mg/L	7.0 mg/L / 716.6 kg/d
Total Residual Chlorine	0.02 mg/L	2.0 kg/d	Non-detectable
E. coli	200 organisms/100 mL (Monthly Geometric Mean Density)	-	100 organisms/100 mL (Monthly Geometric Mean Density)
pH	6.0 to 9.5	-	6.5 to 8.5

4.4 OPERATIONAL DATA

The general plant operation was reviewed against the Sudbury WWTP Amended C of A requirements and historical data provided in the Annual Reports from 2009 to 2012. Historical data is summarized in Table 4-5.

Table 4-5 Historical Effluent Concentrations

EFFLUENT PARAMETER	ANNUAL AVERAGE			
	2009	2010	2011	2012
$CBOD_5$ (mg/L)	6.2	6.0	4.9	4.5
TSS (mg/L)	9.4	8.2	6.6	6.1
TP (mg/L)	0.42 (exceedance July – 0.52 mg/L)	0.31 (all months comply)	0.32 (all months comply)	0.30 (all months comply)
TAN (mg/L)	11.42	13.20	12.69	12.40
pH	7.20	6.80	6.90	6.88
E. coli (organisms/100 mL)	141	72	11	117

Historically, the Sudbury WWTP has met all concentration limits. However, based on historical data, the plant would not meet future Phase II effluent requirements without treatment process upgrades to manage $CBOD_5$, TSS, TP, and TAN.

A capacity review of each unit process at the WWTP was not conducted. Instead, the rated capacity was considered the true capacity of the plant.

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 wastewater system. The unit rates used to estimate future water and wastewater flows correspond to the values included in the *Population Projections and Development of Unit Rates Technical Memorandum* (WSP, 2014). Otherwise, design criteria recommended in the *MOE Guidelines* and *City's Engineering Design Manual* were used.

Note that the total wastewater flows considered to analyze the capacity of the Sudbury WWTP include wastewater flow generated in Sudbury and Garson (as the two systems are already connected) as well as the wastewater flow generated in Copper Cliff (based on the fact that the City is currently planning to interconnect the Copper Cliff wastewater system to the Sudbury wastewater system, by means of pumping all wastewater flows generated in Copper Cliff into the Sudbury wastewater network via the Nickel LS).

5.1 UNIT WASTEWATER DESIGN CRITERIA

The unit flow criteria for growth adopted for this assessment, as it relates to wastewater generated in Sudbury, are shown in Table 5-1 below. Unit flow criteria used to calculate projected flows within Copper Cliff are documented in the Copper Cliff Wastewater System Gap Analysis and Status Quo Report. These values were recommended in the *Population Projections and Development of Unit Rates Technical Memorandum* (WSP, 2014). Note that the term "extraneous flows" is interchangeable with "I/I flows".

Table 5-1 Wastewater System Design Criteria

CRITERIA	VALUE	REFERENCE
Average Day Residential Flow	500 L/cap/day	Average of historical values, rounded up to nearest 50 L/cap/day
Average Day Commercial and Institutional Flow	28 m^3/ha/d	Water unit rates, assuming a 1:1 correspondence
Average Day Industrial Flow	35 m^3/ha/d	Water unit rates, assuming a 1:1 correspondence
Average Extraneous Flow	5.76 m^3/ha/d	Peak from City's Engineering Design Manual and assuming a peaking factor of three
Peak Extraneous Flow (Future Development Areas).	17.28 m^3/ha/d	City's Engineering Design Manual
Max Day Peaking Factor	3.86	Average of historical values

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

Maximum day flows to the WWTP are obtained by multiplying the average day flow by the maximum day peaking factor.

5.2 DESIGN CRITERIA FOR WASTEWATER SYSTEM COMPONENTS AND OPERATION

5.2.1 WASTEWATER TREATMENT

Wastewater treatment facilities are rated for average day flows. Plant effluent limits and objectives are established in the C of A or ECA for each facility.

5.2.2 LIFT STATION PUMPING CAPACITY

The firm capacity of the lift station (with the largest pump out of service) must allow pumping of peak wet weather flows corresponding to its catchment area (MOE, 2008).

Starting limitations on pump motors generally dictate the minimum size of a wet well. The wet well should be large enough to prevent pump motors from overheating due to frequent starting and stopping, but small enough to avoid long retention times leading to septicity and odor problems (Lin & Lee, 2001).

The station wet well shall be sized such that the number of pump starts per hour does not exceed the maximum value recommended by the pump manufacturer. In other words, the time between pump starts and stops (i.e. the pump cycle time) should not exceed that which results in a pump start frequency greater than that recommended by the pump manufacturer. Typically, submersible pumps can cycle four to 10 times per hour with a maximum cycle time not exceeding 30 minutes (Lin & Lee, 2001). A maximum value of four pump starts per hour was assumed to evaluate wet well sizing requirements.

5.2.3 SEWERS

The sewer system is typically sized to convey peak instantaneous (peak wet weather) flows. Wastewater flows are made up of wastewater discharges from residential, commercial, institutional and industrial establishments, plus extraneous flow components from I/I sources.

Due to the high levels of inflow and infiltration observed in the Sudbury wastewater network, the service level for the existing system has been established as the sanitary flow plus I/I corresponding to a 2-year rain on snow event.

Therefore, the wastewater system must be able to adequately service the existing customers plus approved development. The capacity in the existing network has been assessed based on using I/I rates (peak extraneous flow rates) from a 2 year rain on snow event whereas new sewers within the network will be based on the I/ I rates stipulated in the City's Engineering Design Manual.

Additionally, for sewers to convey peak flows, sufficient flow velocity need to be maintained to transport the wastewater solids to avoid deposition and the development of nuisance conditions under lower flow conditions. The minimum acceptable flow velocity in sewers is 0.6 m/s (City of Greater Sudbury, 2012).

Not all sewer sections achieve scouring velocity during dry weather flows (including dry weather I/I), resulting in the possibility of accumulations that reduce capacity over the long term. Targeted inspections and possibly flushing can identify such areas and recover capacity.

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 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 *Population Projections and Development of 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 Sudbury population with wastewater servicing is projected to increase by 4,493 people by 2041 and 30,641 by Ultimate Buildout.

The population projections for Sudbury and Garson to be used in the Master Plan are summarized in Table 6-1 below. Population projections for Copper Cliff are documented in the Copper Cliff Wastewater System Gap Analysis and Status Quo Report. Existing and future wastewater flow generated by residential development in Copper Cliff will be added to the wastewater flow generated in Sudbury and Garson to evaluate the capacity of the Sudbury WWTP. Wastewater flow generated in Copper Cliff will be considered as part of the capacity analysis of the Sudbury WWTP since the City is planning to pump all wastewater flows generated in Copper Cliff into the Sudbury wastewater network via the Nickel LS.

Table 6-1 Sudbury Population Projections

YEAR	POPULATION
2011	91,246
2016	92,182
2021	93,391
2026	94,640
2031	95,352
2036	95,795
2041	95,739
Ultimate Buildout	121,886

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 in Sudbury for each horizon year are summarized in the table below. The targeted ICI development areas in Copper Cliff are documented in the Copper Cliff Wastewater System Gap Analysis and Status Quo Report. Existing and future wastewater flow generated by ICI development in Copper Cliff will be added to the wastewater flow generated in Sudbury and Garson to evaluate the capacity of the Sudbury WWTP. Wastewater flow generated in Copper Cliff will be considered as part of the capacity analysis of the Sudbury WWTP since the City is planning to pump all wastewater flows generated in Copper Cliff into the Sudbury wastewater network via the Nickel LS.

Table 6-2 Sudbury ICI Projections for Areas with Wastewater Servicing

	ICI DEVELOPMENT AREAS (HA)						
LAND USE	2016	2021	2026	2031	2036	2041	BUILDOUT
Institutional	0	0	9.54	0	0	0	0
Commercial	0	0	23.09	0	76.84	0	0
Industrial	146.32	0	70.96	0	171.71	0	0
Total	146.32	0	103.59	0	248.55	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 the Sudbury/Garson service area, two streets have been placed on the priority list for sewer and water servicing and an additional seven for sewer only. 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 WASTEWATER FLOW PROJECTIONS AND INFRASTRUCTURE NEEDS

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

$$\begin{aligned}\textbf{Projected Average Day Generation}&\\ &= \textbf{Base Generation} + \textbf{Additional Residential Generation} + \textbf{Additional ICI Generation}\\ &+ \textbf{Average Extraneous Flow}\end{aligned}$$

The flows corresponding to the population growth forecasts in Sudbury and Garson to Ultimate Buildout are presented in Table 6-3 below, following the procedure described in Section 5.1.

Table 6-3 Flow Projections – Sudbury & Garson

YEAR	POPULATION	AVERAGE DAY FLOW (M^3/D)	MAXIMUM DAY FLOW (M^3/D)
Base	91,246	59,873	231,020
2016	92,182	66,531	257,006
2021	93,391	67,185	259,534
2026	94,640	71,923	277,836

2031	95,352	72,474	279,966
2036	95,795	82,424	318,399
2041	95,739	82,453	318,513
Ultimate Buildout	121,886	97,431	376,371

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 demand was calculated by applying the respective peaking factor to the average day demand. The maximum day demand for the base year was the historical average.

The flows corresponding to the population growth forecasts in Sudbury, Garson, and Copper Cliff to Ultimate Buildout are presented in Table 6-4 below. These flow projections were used to ascertain the remaining capacity at the Sudbury WWTP. Wastewater flow generated in Copper Cliff was considered as part of the capacity analysis of the Sudbury WWTP since the City is planning to pump all wastewater flows generated in Copper Cliff into the Sudbury wastewater network via the Nickel LS.

Table 6-4 Flow Projections – Sudbury, Garson & Copper Cliff

YEAR	AVERAGE DAY FLOW (M^3/D)	MAXIMUM DAY FLOW (M^3/D)
2016[1]	66,531	257,006
2021	68,553	265,076
2026	73,303	283,424
2031	73,857	285,566
2036	84,538	326,962
2041	84,567	327,073
Ultimate Buildout	99,563	385,004

[1] *Flow projections in 2016 include wastewater flows generated in Sudbury and Garson only. The City is in the process of planning the infrastructure required to pump wastewater flows from Copper Cliff into the Sudbury wastewater network.*

A desktop analysis of historical wastewater flows and future flow projections is included in Appendix C.

6.4.1 SUDBURY WWTP CAPACITY

Based on the current Sudbury WWTP rated capacity of 79,625 m^3/d, the wastewater treatment capacity will be sufficient to service growth projections until 2031. Therefore, the Phase 2 upgrades will be required to service 2041 projected populations. The WWTP capacity is plotted with the flow projections on Figure 6-1 below.

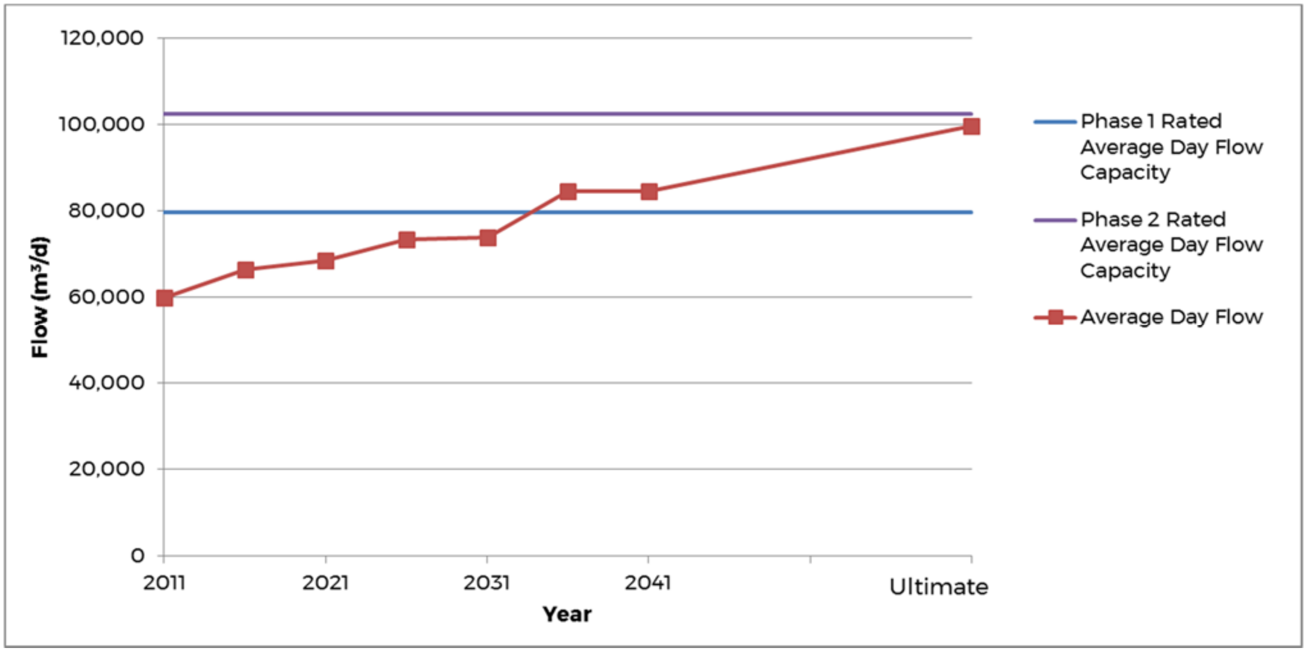

Figure 6-1 Wastewater Flow Projections Compared to Sudbury WWTP Current and Future Capacities

6.4.2 GARSON LAGOONS

Although the Garson Lagoons are have not been used for treating wastewater flows generated in Garson since 2011, they are frequently used for storage during peak wet weather flow events as measure to not surcharge the wastewater conveyance system in Sudbury as well as the Sudbury WWTP.

6.4.3 SEWER NETWORK AND LIFT STATIONS

For each of the scenarios modeled, the system was checked for surcharging of sewers and capacity exceedance at the lift stations. The interconnections between the various lift stations in this system are illustrated in the figure below.

The peak flows into each of the lift stations was determined from the computer simulations for the various planning scenarios and is presented in the table below. The table also shows the design/rated flow for the pumps, their capacity based on drawdown tests and the computer simulated flow for comparison.

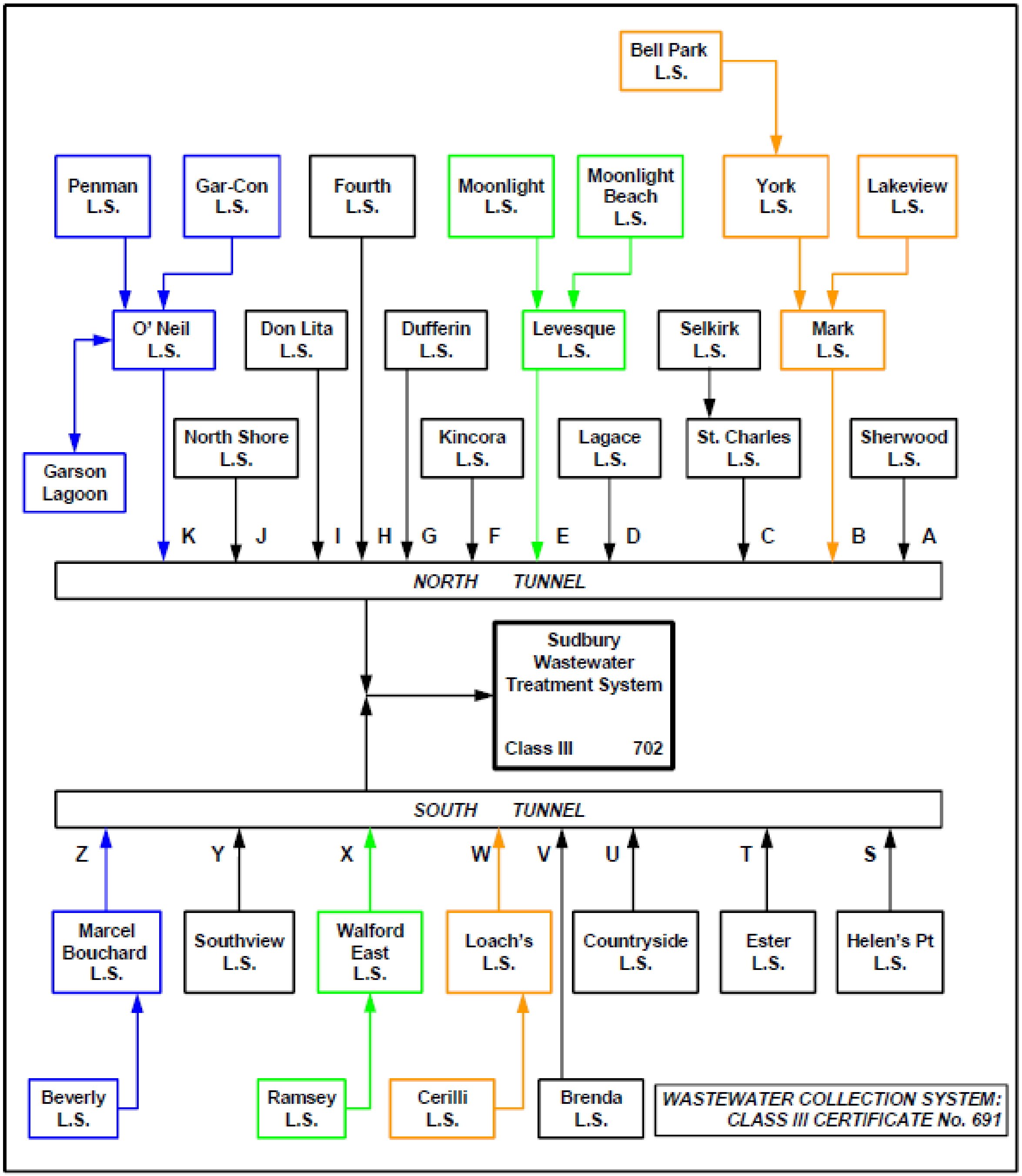

Figure 6-2 Lift Station Schematic Showing the Chains Leading to the Tunnel

Table 6-5 Sudbury Lift Station Capacities and Peak Inflow Rates

NAME	FLOW TO TUNNEL	CURRENT FIRM CAPACITY	EXISITING PEAK FLOW	2041 PEAK FLOW	ULTIMATE BUILDOUT
Bell Park[5]	B	N/A	N/A	N/A	N/A
Beverly	Z	28.8	36.62	36.62	37
Brenda	V	13.3	7.28	7.28	7.29
Cerilli	C	14	2.33	2.33	2.35
Countryside	U	7.6	3.79	9	13
Don Lita	I	30.3	52.06	55	72
Dufferin	G	6.4	4.8	4.8	4.8
Ester	T	28.4	13.98	14.96	17.99
Fourth	H	15.2	31.24	31	32
Gar-Con	K	24.3	18.52	18.97	18.97
Helen's Point	S	7.6	5.99	5.99	5.99
Kincora	F	8.7	2.91	2.92	2.92
Lagace	D	14	56.95	56.95	56.95
Lakeview	B	20.9	0.64	0.65	0.65
Levesque	E	167.6	176.83	191.09	195.52
Loach's Road	W	12.1	5.44	5.44	5.45
Marcel – Bouchard[6]	Z	303.3	N/A	N/A	N/A
Mark	B	41.7	17.22	17.27	17.27
Moonlight	E	16.3	19.73	20.20	20.20
Moonlight Beach[5]	E	N/A	N/A	N/A	N/A
Northshore	J	11.4	4.23	4.2	4.2
O'Neil[5]	K	98.6	N/A	N/A	N/A
Penman	K	8.3	6.5	7.1	9.2
Ramsey	X	32.2	46.43	48.63	50.63
Selkirk	C	38.7	31.65	31.8	31.8
Sherwood	A	30	24.68	53.2	53.2
Southview	Y	58.8	108.13	108.13	108.82
St. Charles	C	383	254.44	520	520
Walford East	X	127	77.98	80.18	82.24
York	B	13.2	25	25	25

[1] *Simulated using the pump curves obtained from pump manufacturer.*

[2] *Based on CGS lift station operating manual information (May, 2013).*

[3] Based on CGS drawdown tests from 2010 to 2015 (varies for each LS).

[4] Pump updated in 2013 after drawdown test.

[5] Seasonal operation.

[6] Decommissioned. Used in emergency cases only.

The theoretical peak inflow is based on the sewer system monitoring for the observed 2-year storm or, where no monitoring was available, computer simulations of the same storm event and I/I rates.

Based on the computer simulations and City documentation summarised in

Table 6-5, a number of lift station issues are apparent:

- Levesque
- Lagace
- Moonlight
- Beverley
- Don Lita
- Fourth
- Ramsey
- Sherwood
- Southview
- St. Charles
- York
- Penman

7 HYDRAULIC MODELLING

7.1 APPROACH

A basic sanitary model for the City of Greater Sudbury was received from the City. The model was created in Bentley's SewerGEMS by City staff. The model is an all pipe model of the sanitary network in these systems, but some critical information such as pipe data, invert elevations and lift station characteristics were missing. The model now includes this information as well as key vertical infrastructure in each system, including lift stations and treatment facilities.

The model was loaded with wet weather flow data. A water balance was completed to determine I/I rates for both dry and wet weather flow. The results from the water balance were compared against I/I rates developed through flow monitoring, and the greater of the two values, for each system, was used to load the model.

Current (2011) and future (2016-Ultimate Buildout, in 5 year increments) population data was added to the model using the City's planning data, summarized in previous sections of this report.

Future dry and wet weather flow scenarios were developed for each of the horizon years: 2016, 2021, 2026, 2031, 2036, 2041, and Ultimate Buildout. However, model results did not vary from 2016 to 2041; therefore, this report discusses findings for 2041 and Ultimate Buildout, compared against existing (2011).

7.2 MODELLING FINDINGS

The model was used to check sewer capacity and flow velocity. Maps in Appendix B illustrate the modeling results for the 2011, 2041, and Ultimate dry and wet weather flow scenarios.

The majority of the sewers flow at less than 50% of the available capacity through to Ultimate Buildout under the wet weather flow condition, with some exceptions. Of note, the trunk sewer that generally parallels Junction Creek would operate at over 100% of its capacity by Ultimate Buildout.

Flow velocities through most of the Sudbury sewer system are generally under 1.5 m/s throughout the network. Many areas also flow at less than the city's current standard of 0.6 m/s.

8 CONCLUSIONS

An assessment of the Sudbury Wastewater System was completed to identify infrastructure requirements to service forecasted growth in the community.

The conclusions of the assessment are summarized below.

— The WWTP is deemed to have sufficient average day capacity to service growth to 2031. The Phase 2 Rated Capacity would be sufficient to service flows to 2041 and Ultimate Buildout.

— There are wet weather flow concerns at the Sudbury WWTP

— There are capacity concerns at the following lift stations: Levesque, Lagace, Moonlight, Beverley, Don Lita, Fourth, Ramsey, Sherwood, Southview, St. Charles, York and Penman

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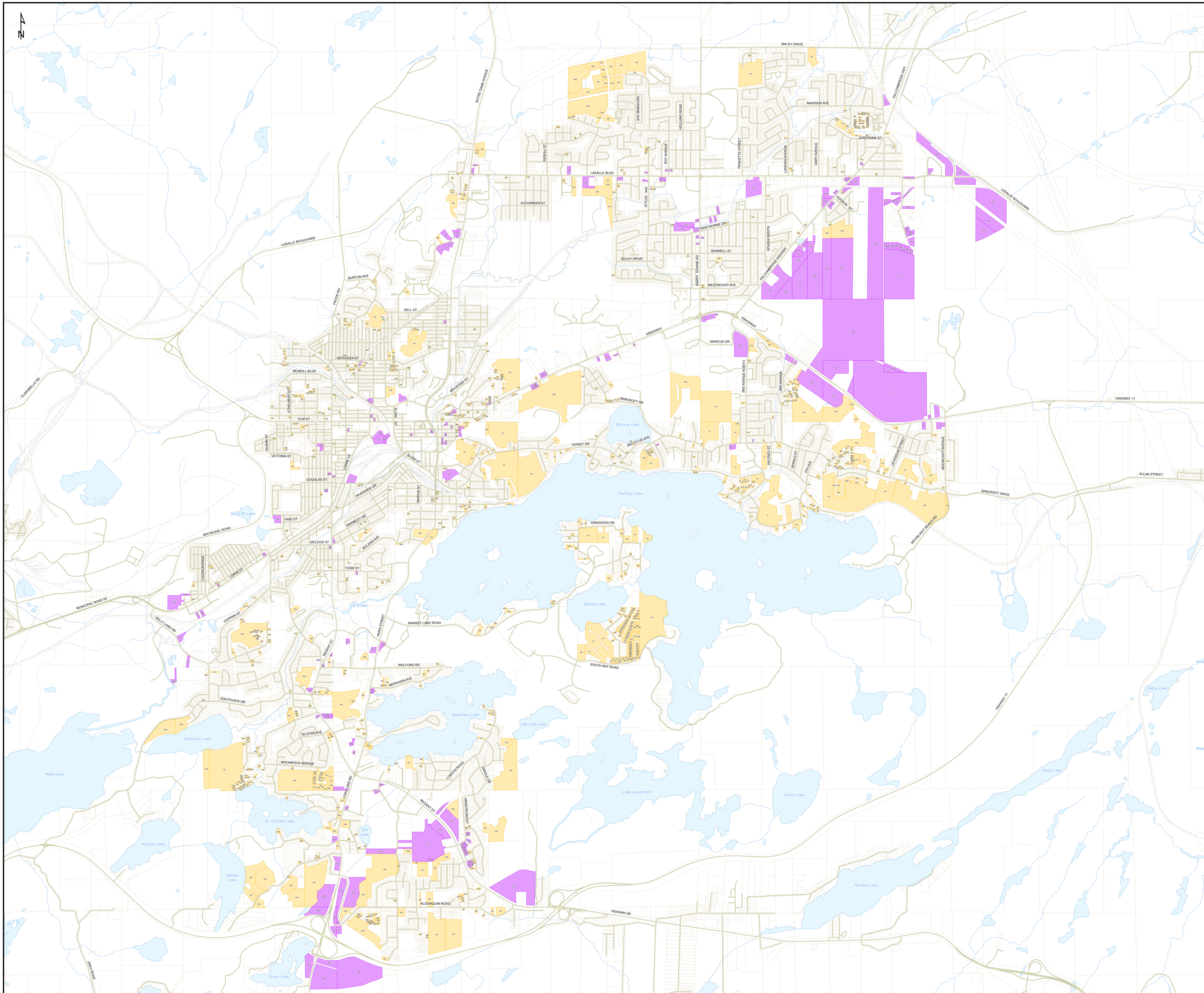
APPENDIX

A RESIDENTIAL AND ICI DEVELOPMENT AREAS



Sudbury | WSP

Project No. 121-23026-00

Sudbury Water and Wastewater Master Plan

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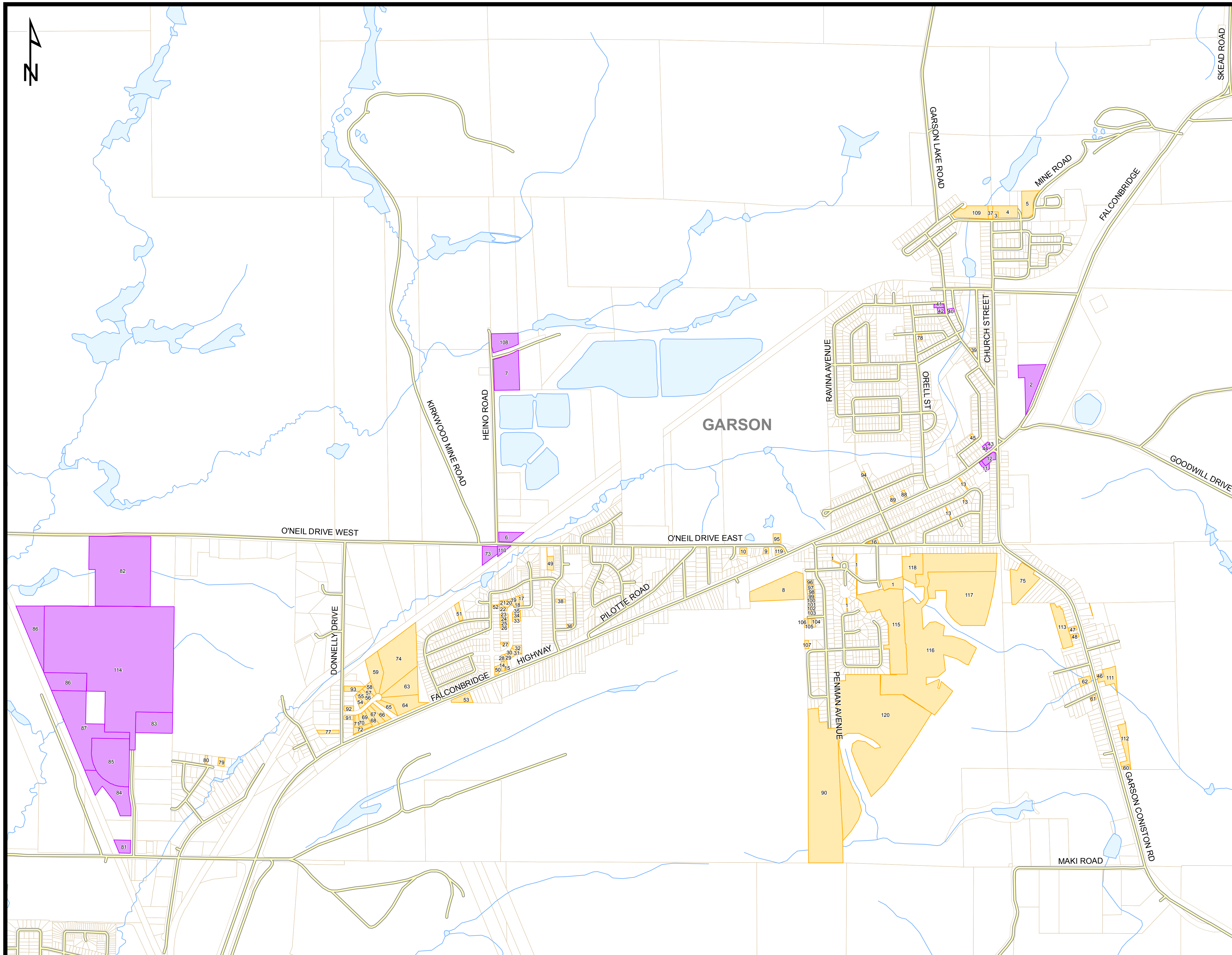
Map 5

Sudbury
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

0 200 400 600 800 1,000 m

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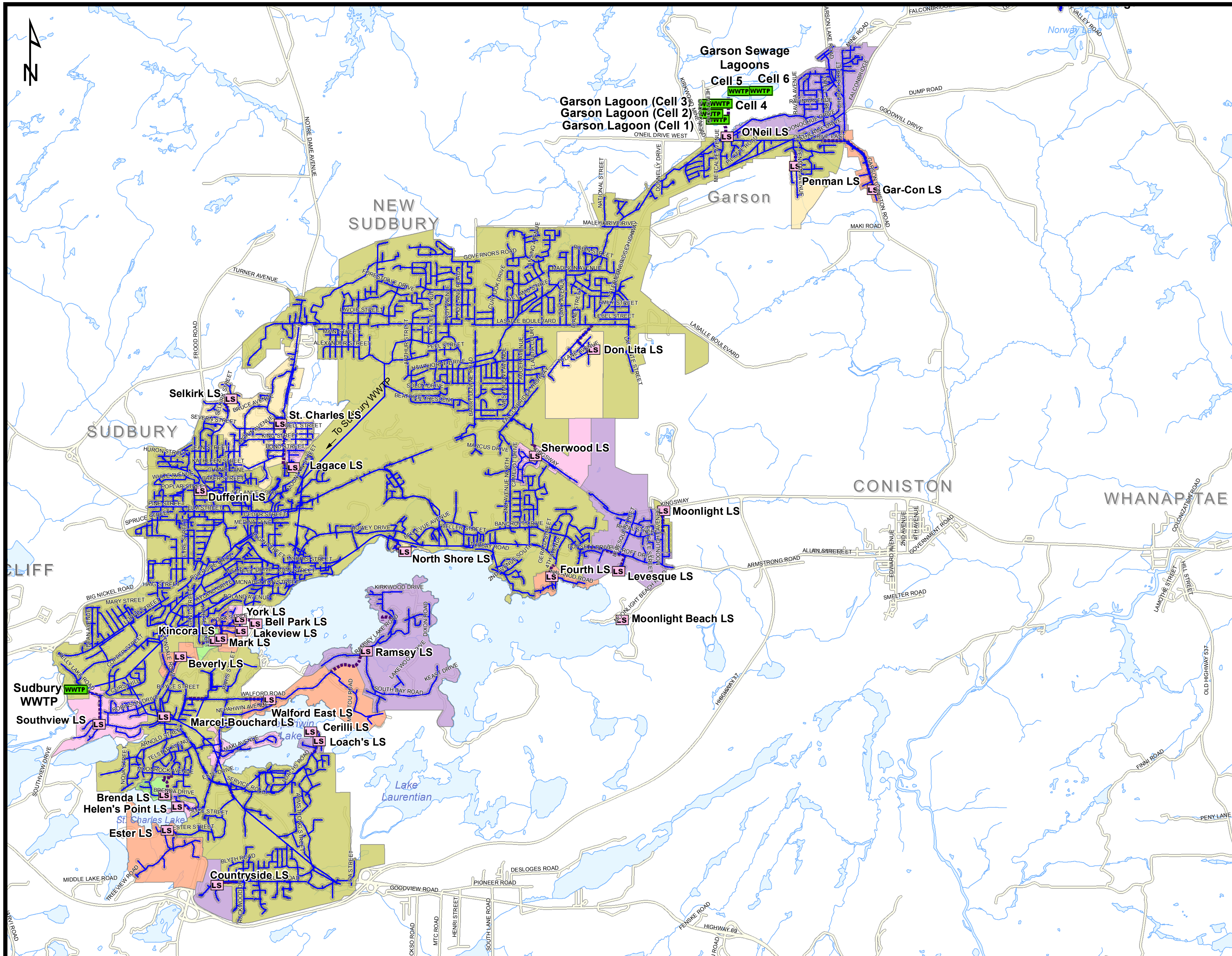
Map 9

Garson
2011 - Ultimate Buildout
Residential and ICI
Development Areas

APPENDIX

B WASTEWATER MODEL RESULTS



Legend

LS Lift Station

WWTP Sanitary Treatment Plant

Manhole

Waterbody

Road

Watercourse

Sanitary Sewer Conduit

Forcemain

Greater Sudbury Grand

WSP

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December 09 2016

Sudbury Water and Wastewater Master Plan

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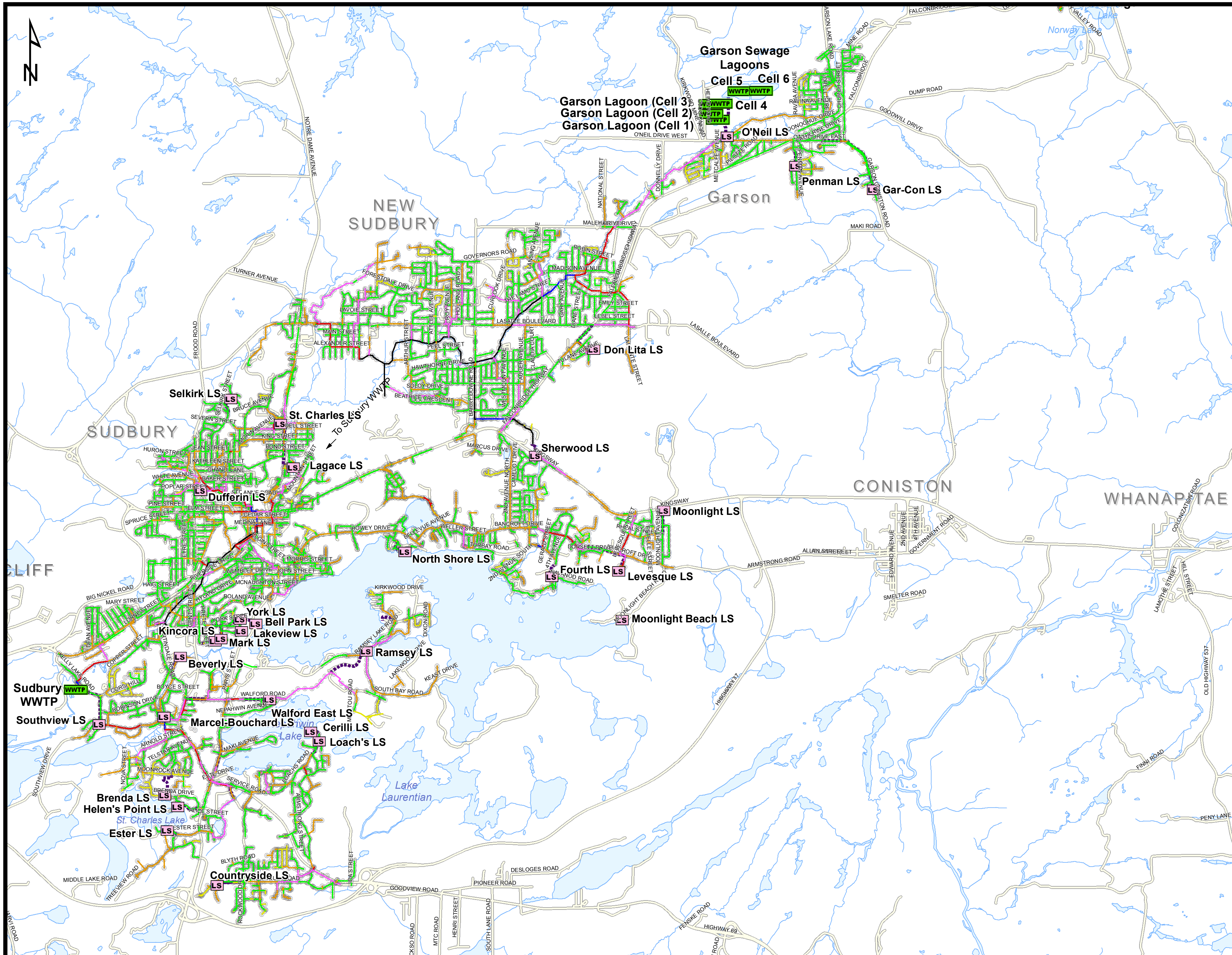
Map 1.1

Sudbury Wastewater System - Sewershed Map



Legend

LS	Lift Station
WWTP	Sanitary Treatment Plant
	Manhole
	Waterbody
	Road
	Watercourse
	Forcemain

Conduit: Diameter (mm)

- <= 150
- <= 200
- <= 300
- <= 450
- <= 600
- <= 750
- > 750

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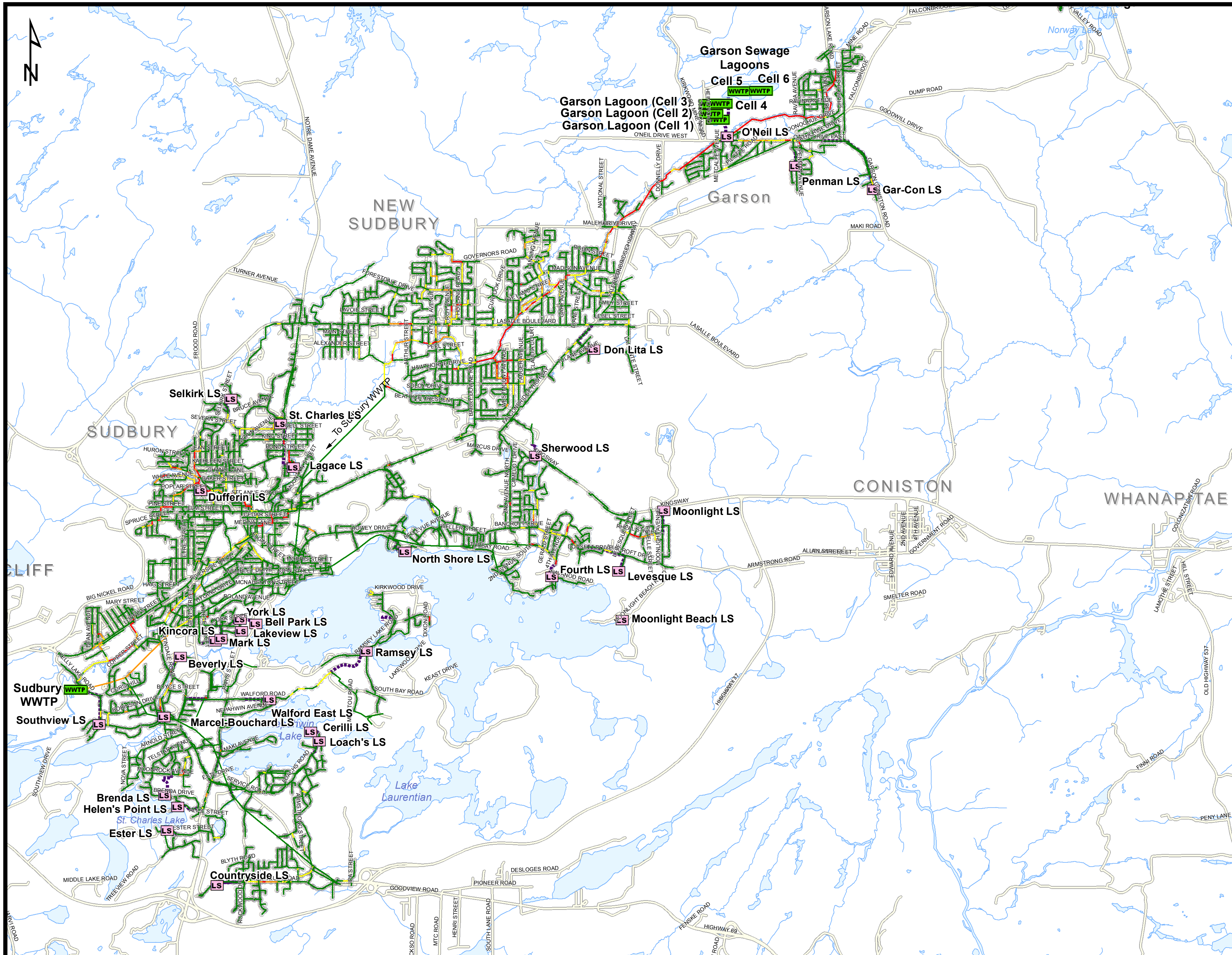
Map 1.2

Sudbury Wastewater System - Conduit Diameter



Legend

- LS Lift Station
- WWTP Sanitary Treatment Plant
- Manhole
- Waterbody
- Road
- Watercourse
- Forcemain

Conduit: Flow/Capacity (Design) (%)

- <= 50
- <= 80
- <= 100
- > 100

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October 04 2017

Sudbury Water and Wastewater Master Plan

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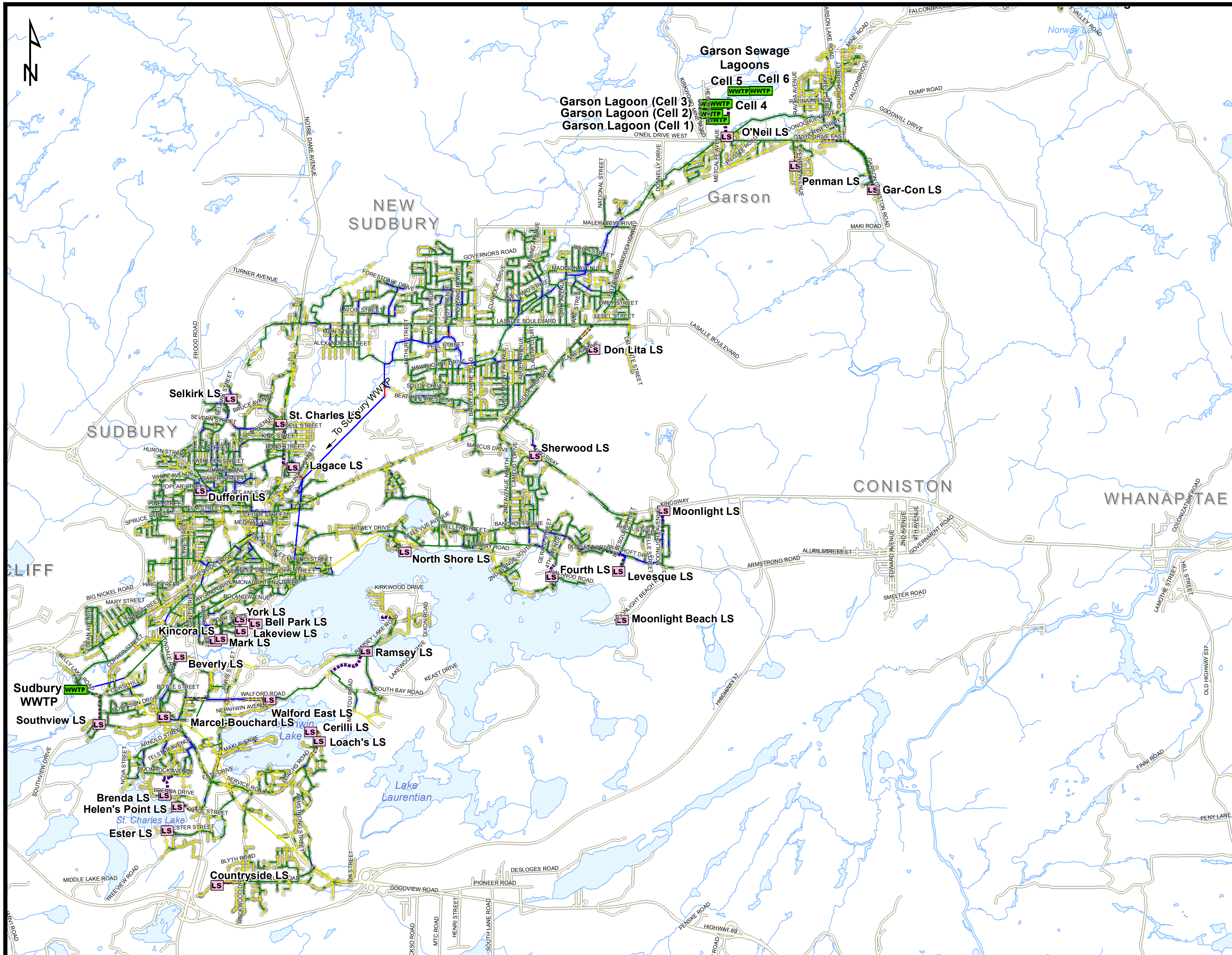
Map 1.3.1

Sudbury Wastewater System - Conduit Flow/Capacity (Design)

Scenario: 2011WWF_2yrStorm



Legend

- Lift Station
- Sanitary Treatment Plant
- Manhole
- Waterbody
- Road
- Watercourse
- Forcemain

Conduit: Velocity (m/s)

- <= 0.6
- <= 1.5
- <= 3.0
- > 3.0

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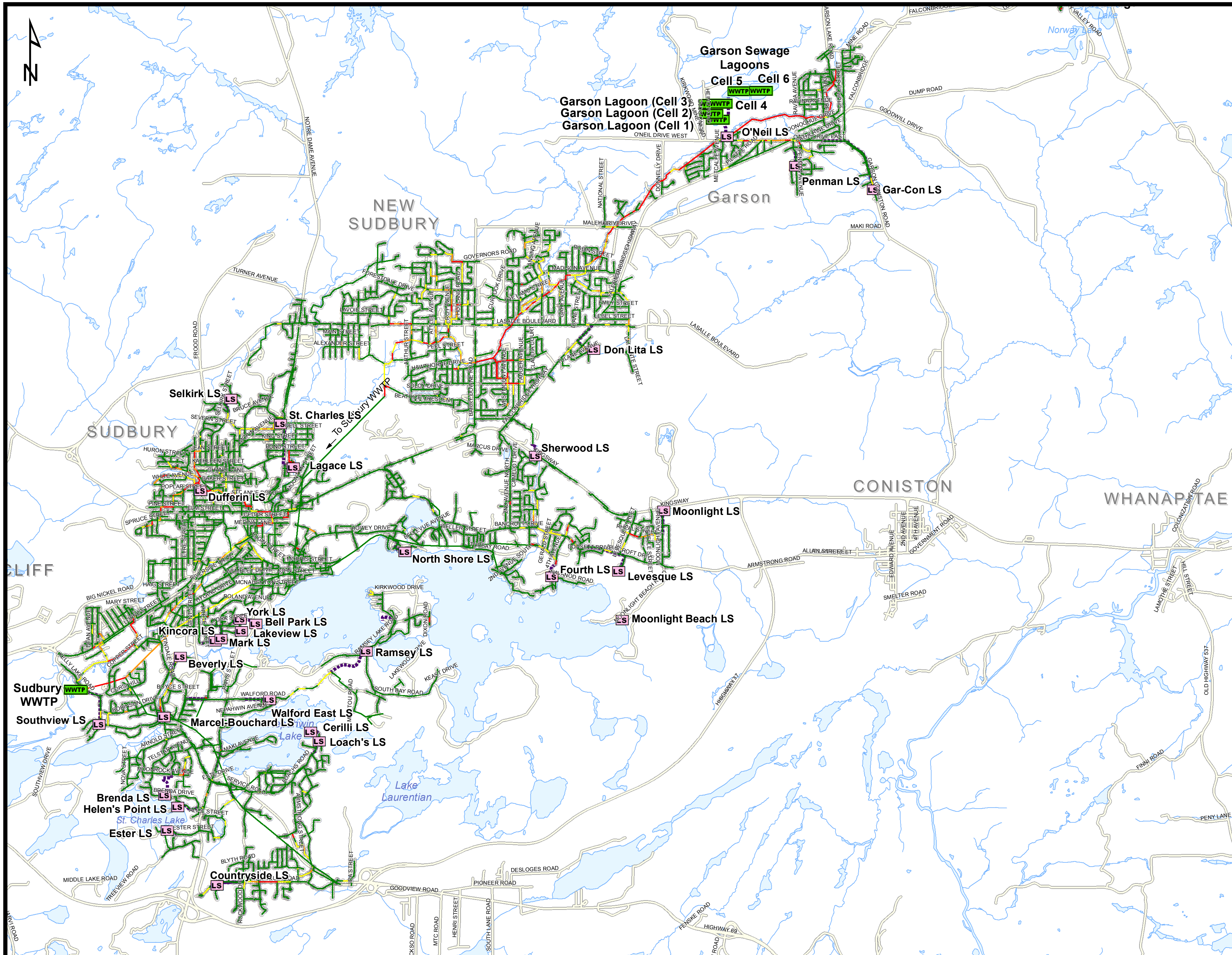
Map 1.3.2

Sudbury Wastewater System - Conduit Velocity

Scenario: 2011WWF_2yrStorm



Legend

- LS Lift Station
- WWTP Sanitary Treatment Plant
- Manhole
- Waterbody
- Road
- Watercourse
- Forcemain

Conduit: Flow/Capacity (Design) (%)

- <= 50
- <= 80
- <= 100
- > 100

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Sudbury Water and Wastewater Master Plan

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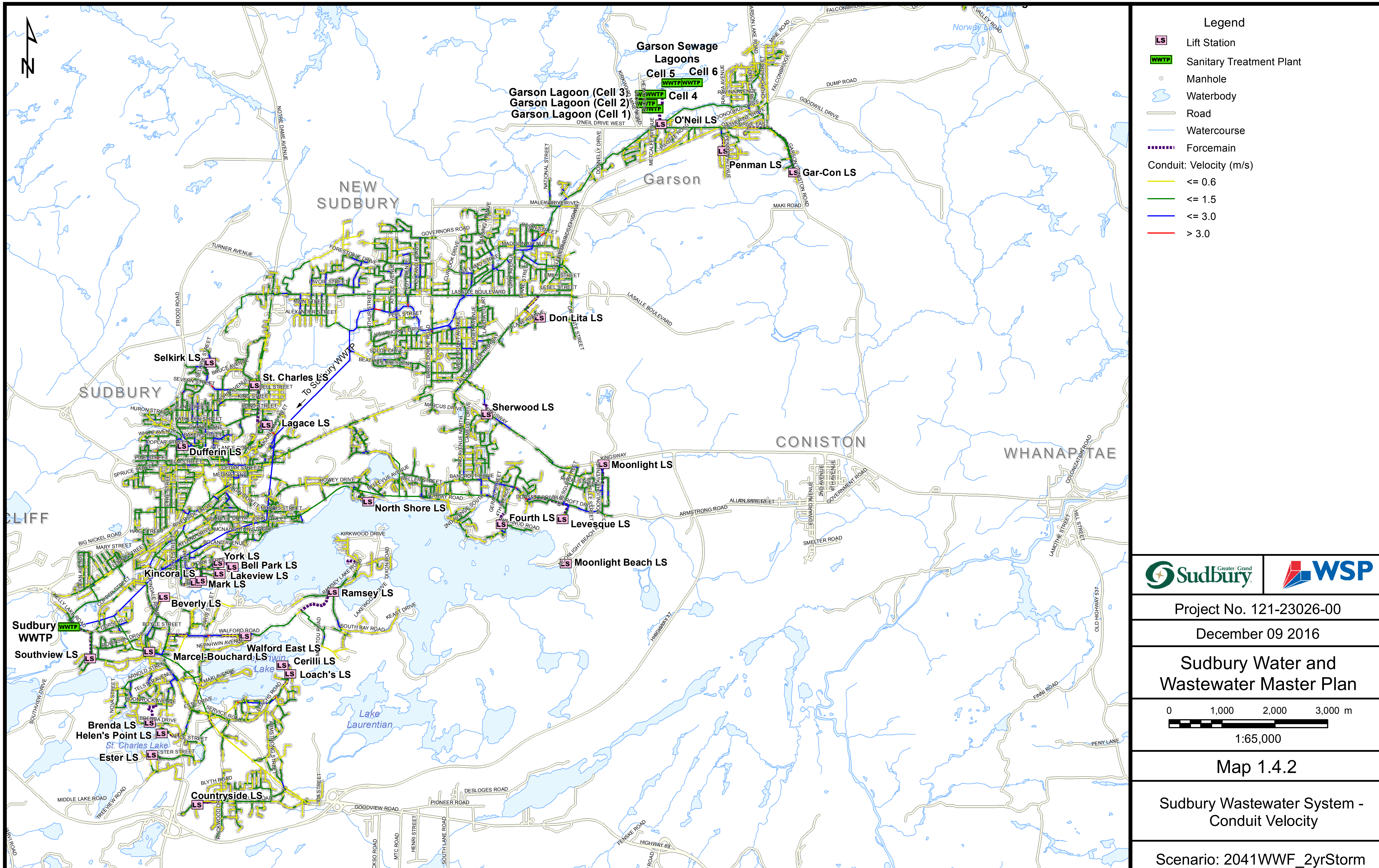
Map 1.4.1

Sudbury Wastewater System - Conduit Flow/Capacity (Design)

Scenario: 2041WWF_2yrStorm



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Sudbury Water and Wastewater Master Plan

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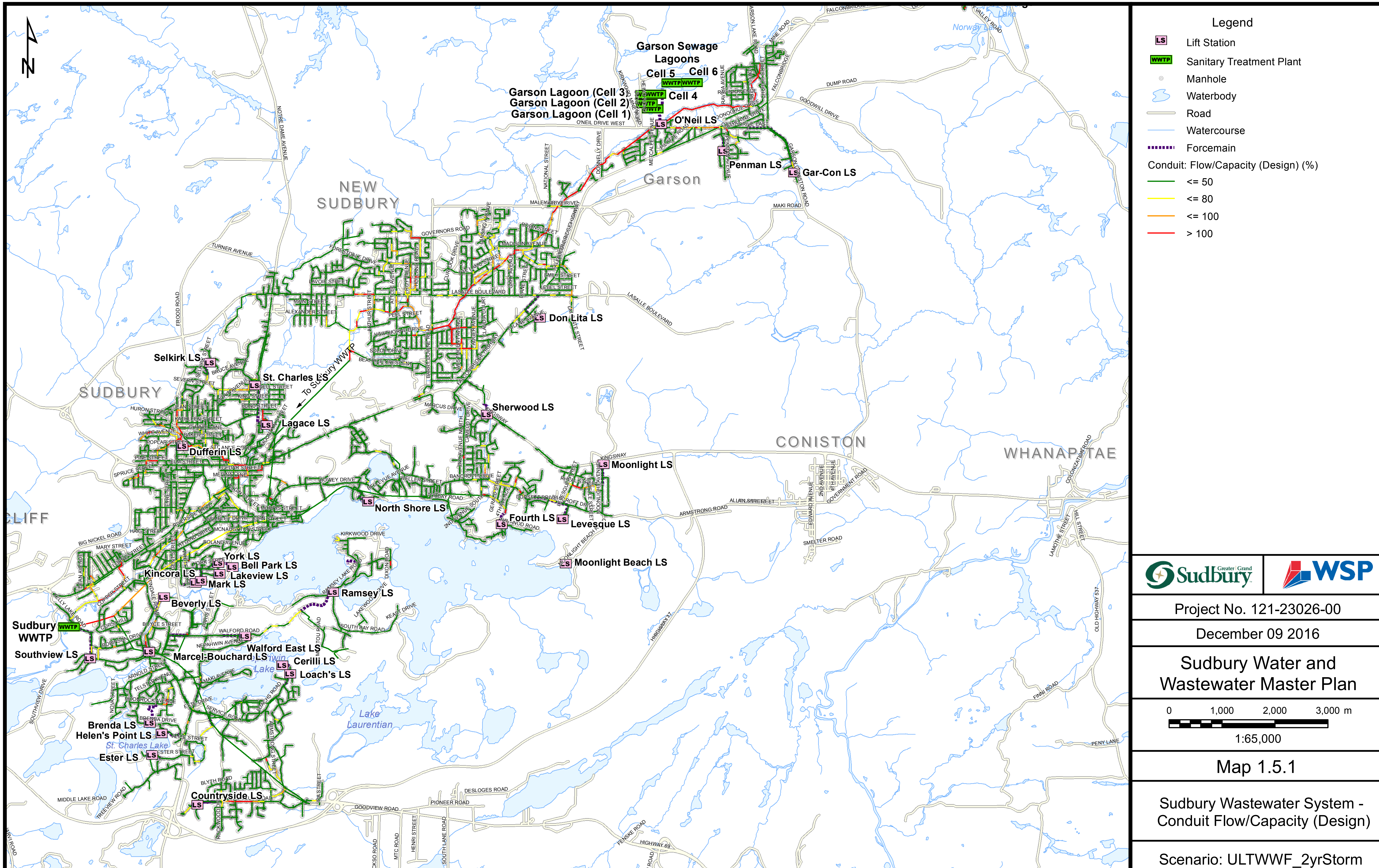
Map 1.4.2

Sudbury Wastewater System - Conduit Velocity

Scenario: 2041WWF_2yrStorm



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Sudbury Water and Wastewater Master Plan

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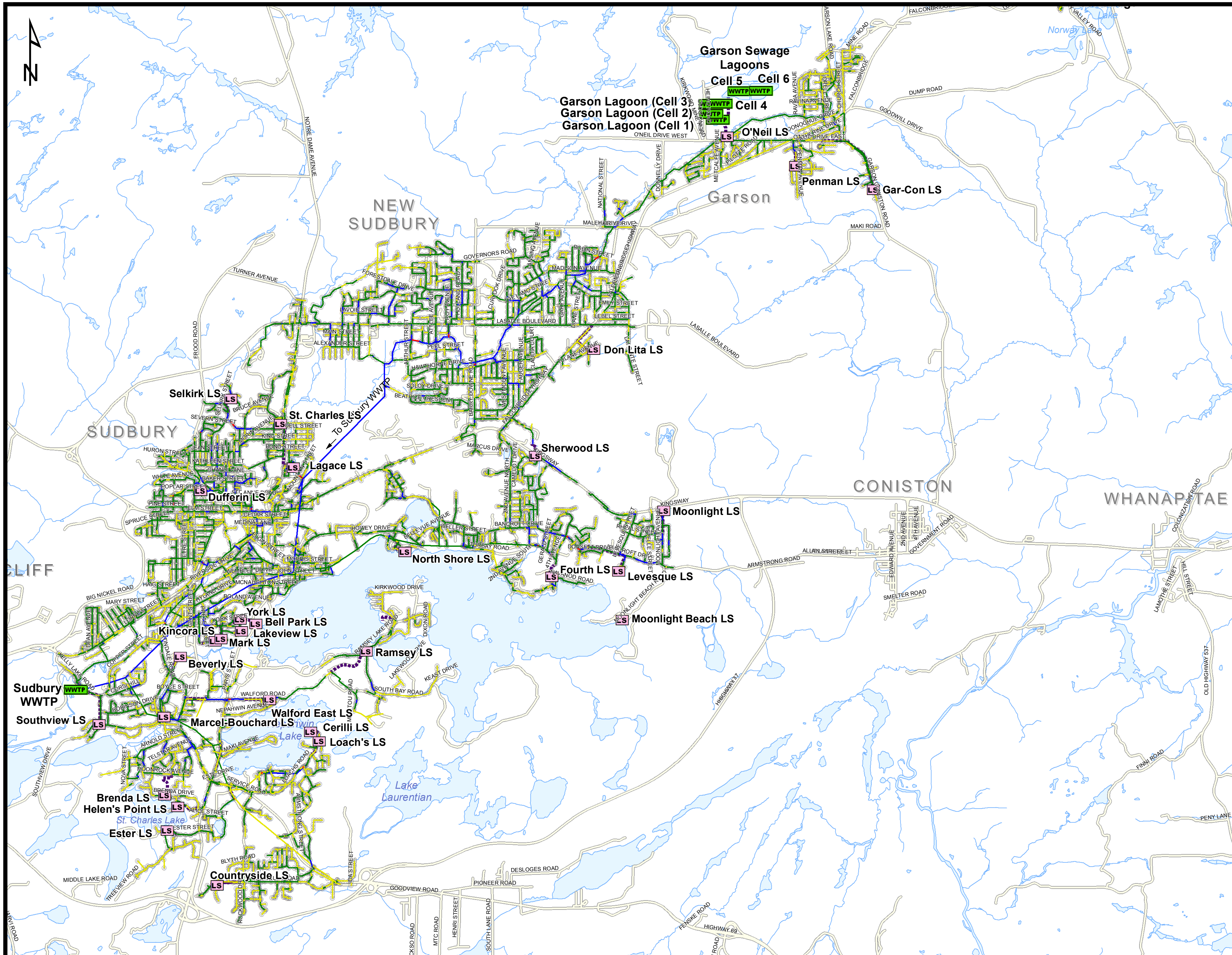
Map 1.5.1

Sudbury Wastewater System - Conduit Flow/Capacity (Design)

Scenario: ULTWWF_2yrStorm



Legend

- LS Lift Station
- WWTP Sanitary Treatment Plant
- Manhole
- Waterbody
- Road
- Watercourse
- Forcemain

Conduit: Velocity (m/s)

- <= 0.6
- <= 1.5
- <= 3.0
- > 3.0

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December 09 2016

Sudbury Water and Wastewater Master Plan

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1:65,000

Map 1.5.2

Sudbury Wastewater System - Conduit Velocity

Scenario: ULTWWF_2yrStorm

APPENDIX

C WASTEWATER GENERATION AND CAPACITY ASSESSMENTS

Sudbury (incl. Garson & Copper Cliff) - Wastewater Flow Forecasts

		2009	2010	2011	2012	2013	Summary	Design Criterion	Comments
Average Day Flow	**(m^3/d)**	65,033	55,181	58,207	56,233	64,710	59,873	59,873	From Annual Reports
Max Day Flow	**(m^3/d)**	228,500	231,700	258,400	179,200	257,300	231,020	231,020	From Annual Reports
Max Day Factor		3.51	4.20	4.44	3.19	3.98	3.86	3.86	Calculated - Max Day Flow divided by Average Day Flow
Peak Flow	**(m^3/d)**	Not Available							Peak hour flows were not available
Peak Flow	**(L/s)**								
Peak Flow Factor									
Population (Existing Areas)		91,246	91,246	91,246	91,246	91,246	91,246	91,246	
Ratio of Residential to Total Customers		0.73	0.73	0.73	0.73	0.73	0.73		This ratio is based on Water Billing Records for the area and is an approximation of the residential portion of demand.
Residential Share of Average Day Demand (m^3/d)		47690	40466	42685	41237	47453	43906		
Residential Flow Unit Rate (m^3/cap/d)		0.523	0.443	0.468	0.452	0.520	0.481	0.500	
Average Institutional Flow Unit Rate (m^3/ha/d)								28.0	CGS design standards do not suggest average rates for institutional and commercial, but require that rates be developed based on usage. In this case, water demands rates were used and assume a conservative 1:1 ratio of water to sewage.
Average Commercial Flow Unit Rate (m^3/ha/d)								28.0	CGS design standards do not suggest average rates for institutional and commercial, but require that rates be developed based on usage. In this case, water demands rates were used and assume a conservative 1:1 ratio of water to sewage.
Average Industrial Flow Unit Rate (m^3/ha/d)								35.0	CGS design standards average rate for light industrial of 35 m^3/ha/d.
Average Extraneous Flow Unit Rate (m^3/ha/d)								5.76	From CGS Design Standards, peak rate for new developments divided by an assumed peaking factor of 3. This factor would be applied only to new developments, which are assumed to be leak-tight, and have minimal extraneous flow.

	2016	2021	2026	2031	2036	2041	Ultimate Buildout	Comments
Population (Existing Areas)	91,246	91,246	91,246	91,246	91,246	91,246	91,246	All populations includes Garson
Population (Growth Areas)	936	2,145	3,394	4,107	4,550	4,493	30,641	
Total Population	92182	93391	94640	95352	95795	95739	121886	Total Population (Hemson)
Residential (ha)	39.27	47.94	68.66	102.51	125.88	135.96	466.44	ICI development areas were assigned to planning years based on the stage of the application. Draft Approved were assigned to 2016, Legal Lots of Record to 2026, and Designated Developable to 2036.
Institutional (ha)	0.00	0.00	9.54	9.54	9.54	9.54	9.54	
Commercial (ha)	0.00	0.00	23.09	23.09	99.93	99.93	99.93	
Industrial (ha)	146.32	146.32	217.28	217.28	388.99	388.99	388.99	Areas are cumulative and carry from the development year, all the way through to Ultimate Buildout
ICI (ha)	146.32	146.32	249.91	249.91	498.46	498.46	498.46	
	185.59	194.26	318.57	352.42	624.34	634.42	964.90	

	2016	2021	2026	2031	2036	2041	Ultimate Buildout	Comments
Average Residential Flows (m^3/d) - Existing	59,873	59,873	59,873	59,873	59,873	59,873	59,873	This includes all contribution from existing ICI and infiltration. The base flow was assumed to be the average day flow to the plant for the 2011-2013 period.
Average Residential Flows (m^3/d)	468	1,072	1,697	2,053	2,275	2,246	15,320	Obtained by multiplying the projected population growth by the unit rate.
Average Institutional Flow (m^3/d)	0	0	267	267	267	267	267	Institutional growth area multiplied by unit flow rate.
Average Commercial Flow (m^3/d)	0	0	647	647	2,798	2,798	2,798	Commercial growth area multiplied by unit flow rate.
Average Industrial Flow (m^3/d)	5,121	5,121	7,605	7,605	13,615	13,615	13,615	Industrial growth area multiplied by unit flow rate.
Average Extraneous Flow (m^3/day)	1,069	1,119	1,835	2,030	3,596	3,654	5,558	
Average Day Flow (m^3/d)	66,531	67,185	71,923	72,474	82,424	82,453	97,431	
Max Day Flow (m^3/d)	257,006	259,534	277,836	279,966	318,399	318,513	376,371	

ALTERNATIVE CALCULATION METHOD

	2016	2021	2026	2031	2036	2041	Ultimate Buildout	Comments
Per Capita Flow (m^3/cap/day)	0.713	0.605	0.638	0.616	0.709	0.656	0.656	
Average Day Flow (m^3/d)	60487	61280	62100	62567	62858	62821	79978	Multiplying the total population by the consolidated per capita flow factor.
Max Day Flow (m^3/d)	233,659	236,723	239,888	241,695	242,818	242,674	308,952	

SENSITIVITY ANALYSIS

Analyze sensitivity of forecasted flows to unit rate

	Unit Rate (m^3/cap/d)	Average Day Flow (m^3/d) 2016	2021	2026	2031	2036	2041	Ultimate Buildout 2061
Base Scenario - Residential Historical Maximum	0.500	66,531	67,185	71,923	72,474	82,424	82,453	97,431
Combined Historical Maximum	0.656	66,677	67,520	72,453	73,116	83,134	83,155	102,216
City Standards	0.410	66,447	66,992	71,618	72,105	82,014	82,049	94,673

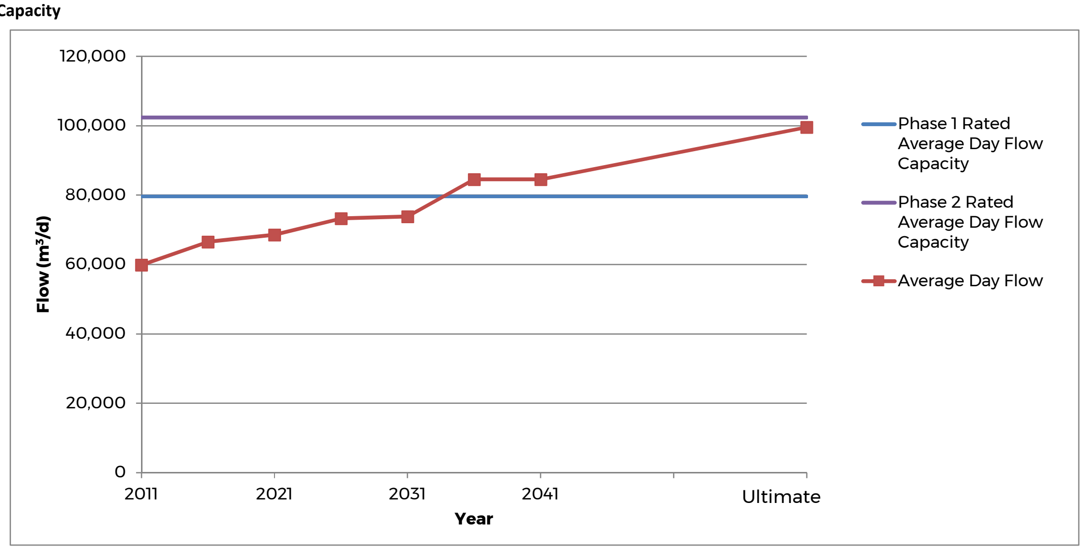
Analyze sensitivity of forecasted flows to max day factor

	Max Day Peaking Factor	Max Day Flow (m^3/d) 2016	2021	2026	2031	2036	2041	Ultimate Buildout 2061
Base Scenario - Historical Max	3.86	257,006	259,534	277,836	279,966	318,399	318,513	376,371
Historical Average	3.86	257,006	259,534	277,836	279,966	318,399	318,513	376,371

CAPACITY CHECK

	2011	2016	2021	2026	2031	2036	2041	Ultimate Buildout 2061
Rated WPCP ADF Capacity (m^3/d)	79,625	79,625	79,625	79,625	79,625	79,625	79,625	79,625
Average Day Flow (m^3/d) - Sudbury/Garson	59,873	66,531	67,185	71,923	72,474	82,424	82,453	97,431
Average Day Flow (m3/d) - Copper Cliff	-	-	1,368	1,380	1,383	2,114	2,114	2,132
Total Average Day Flow (m3/d)	59,873	66,531	68,554	73,303	73,857	84,538	84,567	99,562
Max Day Flow (m^3/d) - Sudbury/Garson.	231,020	257,006	259,534	277,836	279,966	318,399	318,513	376,371
Max Day Flow (m^3/d) - Copper Cliff	-	-	5,542	5,588	5,600	8,563	8,560	8,633
Total Average Day Flow (m3/d)	231,020	257,006	265,076	283,424	285,566	326,961	327,073	385,004
Phase 2 WPCP ADF Capacity (m^3/d)	102,375	102,375	102,375	102,375	102,375	102,375	102,375	102,375

Capacity



CITY OF GREATER SUDBURY WATER AND WASTEWATER MASTER PLAN

VALLEY EAST WASTEWATER SYSTEM GAP ANALYSIS AND STATUS QUO REPORT

CITY OF GREATER SUDBURY

DRAFT

PROJECT NO.: 121-23036-00
DATE: MARCH 2015

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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 existing Valley East Wastewater System. Based on population growth projections and design criteria discussed in the *Population Projections and Development of Unit Rates Technical Memorandum* (WSP, 2014), 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 Valley East Wastewater System would continue to be a stand-alone system. Any potential interconnections between Valley East 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

Valley East is located in the north end of the City of Greater Sudbury and is serviced by the Valley East Wastewater System. Mapping in **Appendix A** shows the Valley East study area 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 deemed to be in the path of 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 *Population Projections and Development of Development of Unit Rates 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 East service area population (including Hanmer, Val Caron and Val Therese) is expected to increase from 19,119 in 2011 to 21,231 by 2041 and 31,469 by Ultimate Buildout, excluding the Urban Expansion Reserve. 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 SYSTEM

The Valley East Wastewater System services the communities of Hanmer, Val Caron, and Val Therese, and includes the Valley East WWTP, as well as nine lift stations and sewer network. The collection system consists of approximately 130.14 km of sewers and forcemains. Additional information on the existing systems is provided in the Baseline Review Report for Wastewater Systems (WSP, 2014). The Valley East Wastewater System is shown in **Appendix B**.

3.1 LIFT STATIONS

Table 3-1 below provides a summary of the main features of the lift stations.

Table 3-1 Valley East Sewage Lift Stations

LIFT STATION	YEAR CONSTRUCTED[1]	LAST UPGRADED[2]	WET WELL VOLUME (M3)[1]	PUMPING STATION CAPACITY AND FORCEMAIN INFORMATION[2]
Fleming	c.1980	None	12.1	Two submersible pumps with a firm design capacity of 25.1 L/s 661 m long, 200 mm diameter polyethylene (series 60) forcemain
Helene	Unknown	Unknown	Unknown	Two dry pit pumps with a firm design capacity of 40.30 L/s 860 m long, 300 mm diameter forcemain of unknown material
Hillsdale	Pre-1981	1981	30.3	Two dry pit pumps with a firm design capacity of 52.20 L/s 2,013 m long, 250 mm diameter forcemain of unknown material
Jeanne D'arc	1976	None	Unknown	Two dry pit pumps with a firm design capacity of 110 L/s 406 m long, 400 mm diameter forcemain of unknown material
Madeleine	1979	None	7.2	Two submersible pumps with a firm design capacity of 15.18 L/s 98 m long, 150 mm diameter PVC (series 125) forcemain
St. Isidore	Unknown	Unknown	Unknown	Two submersible pumps with a firm design capacity of 27.9 L/s 20 m long, 200 mm diameter forcemain of unknown material
Spruce	1974	Unknown	43.6	Two dry pit pumps the firm capacity is 54.38 1,682 m long, 350 mm diameter forcemain of unknown material

Tena	c.1975	1992	Unknown	Two submersible pumps the firm capacity is 22 L/s 491 m long, 150 mm diameter forcemain of unknown material
Tupper	c.1975	1999	Unknown	Two submersible pumps with a firm design capacity of 9.4 L/s 167 m long, 150 mm diameter forcemain of unknown material

[1] Obtained or estimated from dimensions found in as-built and record, assuming water level does not exceed the High Water Alarm Level or, in absence of this alarm level, the inlet sewer invert.

[2] Obtained from the City's Wastewater Lift Stations Operations Manual and station as-built drawings.

3.2 VALLEY EAST WWTP

The Valley East WWTP is owned and operated by the City of Greater Sudbury and is located at 1317 Yorkshire Drive in Val Caron. The WWTP is a conventional activated sludge treatment plant with an average day rated capacity of 11,365 m^3/d (MOECC, 2010).The treatment process is illustrated schematically below.

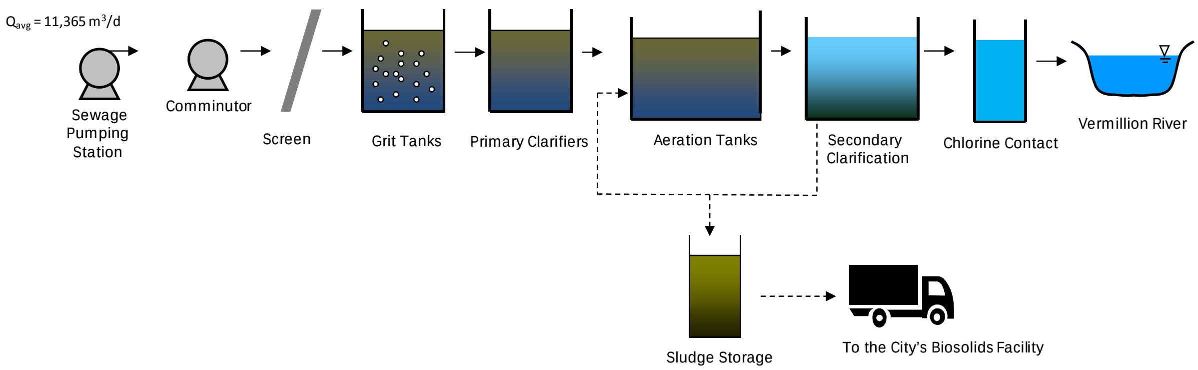

Figure 3-1 Valley East WWTP Process

3.3 KNOWN CHALLENGES

The Valley Wastewater System has the following known challenges:

— There have been reported by-passes at the WWTP during wet weather events.

— The influent pumping station at the WWTP is a bottleneck under high flow conditions. To mitigate this City operations staff set up a temporary forcemain and an auxiliary diesel pump to bypass the bottleneck.

— Equipment at the WWTP is reaching the end of its useful service life in the short to medium term. For additional information please review the City of Greater Sudbury Water and Wastewater Asset Management Plan.

— City staff have indicated that the WWTP electrical system requires upgrades, such as installation of variable frequency drives (VFDs) and replacement of emergency power. Projects to address these challenges have been included in the City of Greater Sudbury Water and Wastewater Asset Management Plan.

4 HISTORICAL FLOWS AND REVIEW OF OPERATIONAL DATA

Data reported in the 2009 to 2013 *Annual Reports* for the Valley East WWTP was reviewed and analyzed to determine average day and maximum day flows as well as review effluent parameters.

4.1 FLOW DATA

WWTP flow data from 2009 to 2013 was reviewed. Operational data was not available from the lift stations and so historical peak flow data could not be estimated.

The recorded average day and maximum day flows are summarized in Table 4-1 and plotted in Figure 4-1 below.

Table 4-1 Historical Wastewater Flow Data

YEAR	AVERAGE DAY FLOW (M^3/D)[1]	MAXIMUM DAY FLOW (M^3/D)[1]
2009	6,893	23,060
2010	4,958	6,370
2011	4,928	14,450
2012	4,566	8,580
2013	5,724	22,844

[1] *Annual Reports (2009 - 2013).*

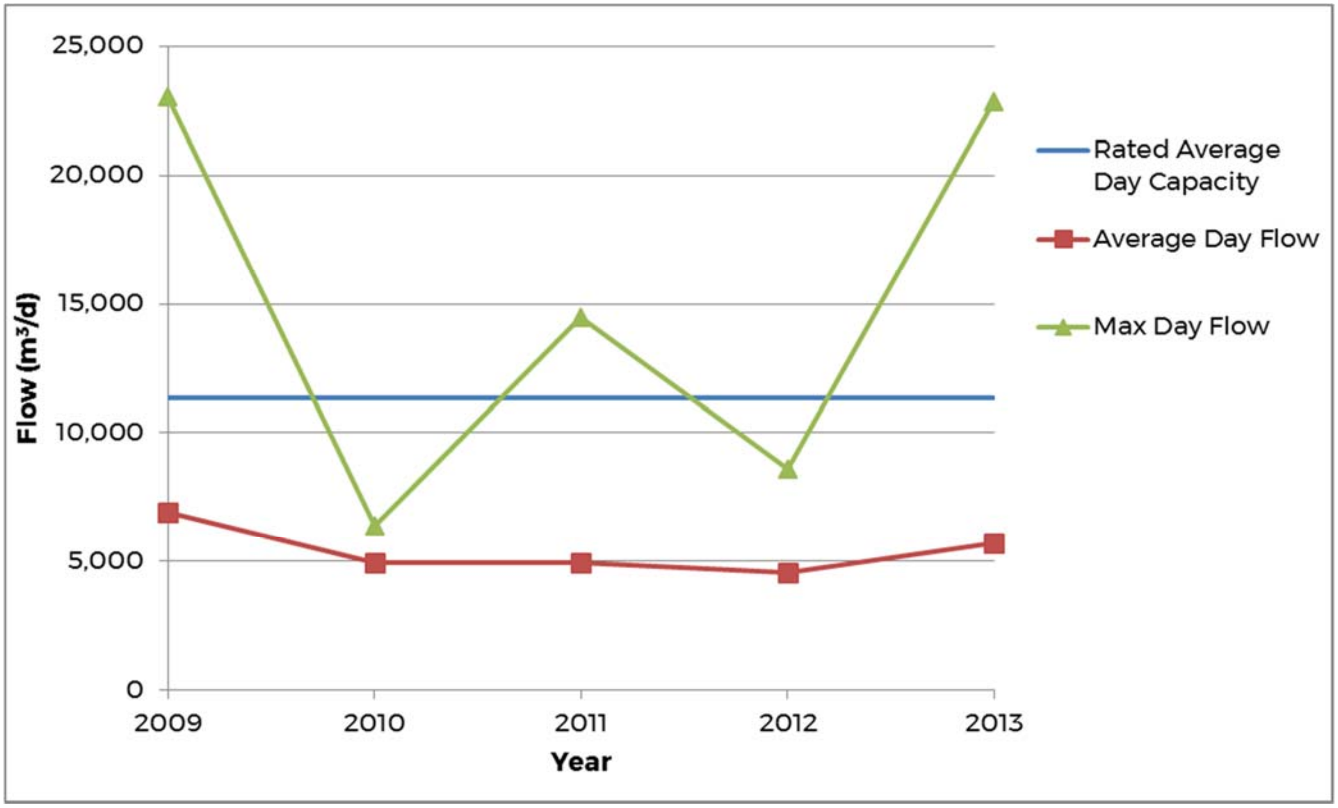

Figure 4-1 Historical Wastewater Flows at the Valley East Wastewater Treatment Plant

The relationship between the different flow regimes was analyzed to compare the peaking factors derived from historical data to those used in the *City's Engineering Design Manual* and those included in the *MOECC Guidelines*.

The average day flows to the WWTP have been consistent over the 2009 to 2013 period, averaging 5,414 m^3/d. The variations in historical maximum day flows show no discernible trend. The greatest maximum day flow occurred in 2009 and the average historical maximum day flow was 15,061 m^3/d.

The wide range in maximum day flows, but stable average day flows, indicates that the system is susceptible to variations in precipitation. There has been one overflow/bypass due to heavy precipitation or snow melt at the Valley East WWTP between 2009 and 2013 (City of Greater Sudbury, 2009-2013).

The highest maximum day to average day peaking factor based on the maximum day flow recorded in 2013 was 3.99. The average maximum day peaking factor from 2009 to 2013 was 2.69. The City's *Engineering Design Manual* and the *MOECC Guidelines* do not specify recommended maximum day factors and recommend using historical data when available. For future wastewater generation, the average peaking factor was used and based on the assumption that new developments would have less I & I due to more leak tight construction

4.2 RAW WASTEWATER CHARACTERISTICS

The average raw wastewater characteristics from 2009 to 2012 are summarized in Table 4-2 below. Raw wastewater temperatures were not reported.

Table 4-2 Average Raw Wastewater Characteristics (2009-2012)

PARAMETER	AVERAGE VALUE
$CBOD_5$	171 mg/L

Suspended Solids	169 mg/L
Total Phosphorus	3.0 mg/L
TKN	35.8 mg/L
pH	7.2

Wastewater flows to the Valley East WWTP correspond mainly to residential uses, with contributions from commercial and industrial users, and dilution from inflow and infiltration.

4.3 EFFLUENT CRITERIA

The Valley East WWTP is operated in accordance with *MOECC Amended Certificate of Approval for Sewage No. 5864-7E5RLV* dated May 9, 2008. The *C of A* concentration and loading limits are summarized in Table 4-3.

Table 4-3 Valley East Effluent Limits and Objectives

EFFLUENT PARAMETER	CONCENTRATION LIMIT	LOADING LIMIT	CONCENTRATION OBJECTIVE / LOADING LIMIT
$CBOD_5$	25 mg/L	284 kg/d	15 mg/L
Total Suspended Solids (TSS)	25 mg/L	284 kg/d	15 mg/L
Total Phosphorus (TP)	1.0 mg/L	11.4 kg/d	0.8 mg/L
pH	6.0 to 9.5	-	6.5 to 9.5
E. coli	200 organisms/100 mL (Monthly Geometric Mean Density)	-	150 organisms/100 mL (Monthly Geometric Mean Density)

Compliance with the concentration and loading limits for $CBOD_5$ and TSS is based on the annual average concentration of each parameter based on all composite samples during any calendar year, whereas compliance for the TP is based on the monthly average concentration.

4.4 OPERATIONAL DATA

The general plant operation was reviewed against the Valley East WWTP Amended C of A requirements and historical data provided in the Annual Reports from 2009 to 2012. Historical data is summarized in the table below.

Table 4-4 Historical Effluent Concentrations

EFFLUENT PARAMETER	ANNUAL AVERAGE			
	2009	2010	2011	2012
$CBOD_5$ (mg/L)	4.7	4.8	3.1	3.2
TSS (mg/L)	9.4	11.2	10.6	11.1
TP (mg/L)	0.45 (all months comply)	0.40 (all months comply)	0.45 (all months comply)	0.47 (all months comply)

EFFLUENT PARAMETER	ANNUAL AVERAGE			
	2009	2010	2011	2012
pH	7.00	7.10	7.00	6.71
E. coli (organisms/100 mL)	12	19	13	24

The Valley East WWTP met all effluent limits. A capacity review of each unit process at the WWTP was not conducted. Instead, the rated capacity was considered the true capacity of the plant.

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 WASTEWATER DESIGN CRITERIA

The unit flow criteria for growth adopted for this assessment are shown in Table 5-1 below. These values were recommended in the *Population Projections and Unit Rates Technical Memorandum* (WSP, 2014).

Note that the term "extraneous flows" is used interchangeably with "I&I flows".

Table 5-1 Wastewater System Design Criteria

CRITERIA	VALUE	REFERENCE
Average Day Residential Flow	250 L/cap/day	Average of historical values, rounded up to nearest 50 L/cap/day
Average Day Commercial and Institutional Flow	28 m^3/ha/d	Water unit rates, assuming a 1:1 correspondence
Average Day Industrial Flow	35 m^3/ha/d	Water unit rates, assuming a 1:1 correspondence
Average Extraneous Flow	11.2 m^3/ha/d	Peak from City's Engineering Design Manual and assuming a peaking factor of three
Peak Extraneous Flow	33.7 m^3/ha/d	City's Engineering Design Manual
Max Day Peaking Factor	2.69	Average of historical values

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

Maximum day flows to the WWTP are obtained by multiplying the average day flow by the maximum day peaking factor.

5.2 DESIGN CRITERIA FOR WASTEWATER SYSTEM COMPONENTS AND OPERATION

5.2.1 WASTEWATER TREATMENT

Wastewater treatment facilities are rated for average day flows. Plant effluent limits and objectives are established in the C of A or ECA for each facility.

5.2.2 LIFT STATION PUMPING CAPACITY

The firm capacity of the lift station (with the largest pump out of service) must allow pumping of peak wet weather flows corresponding to its catchment area (MOECC, 2008).

Starting limitations on pump motors generally dictate the minimum size of a wet well. The wet well should be large enough to prevent pump motors from overheating due to frequent starting and stopping, but small enough to avoid long retention times leading to septicity and odor problems (Lin & Lee, 2001).

The station wet well shall be sized such that the number of pump starts per hour does not exceed the maximum value recommended by the pump manufacturer. In other words, the time between pump starts and stops (i.e. the pump cycle time) should not exceed that which results in a pump start frequency greater than that recommended by the pump manufacturer. Typically, submersible pumps can cycle four to 10 times per hour with a maximum cycle time not exceeding 30 minutes (Lin & Lee, 2001). A maximum value of four pump starts per hour was assumed to evaluate wet well sizing requirements.

5.2.3 SEWERS

The sewer system is typically sized to convey peak instantaneous (peak wet weather) flows. Sewage flows are made up of wastewater discharges from residential, commercial, institutional and industrial establishments, plus extraneous flow components from such sources as groundwater and surface runoff.

In addition to being able to convey peak flows, sufficient flow velocity should be maintained to transport the sewage solids to avoid deposition and the development of nuisance conditions under lower flow conditions. The minimum acceptable flow velocity in sewers is 0.6 m/s (City of Greater Sudbury, 2012).

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 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 Development of 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 Valley East population with wastewater servicing is projected to increase by 2,113 people by 2041 and 12,350 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 East Population Projections

SYSTEM	2011	2016	2021	2026	2031	2036	2041	ULTIMATE BUILDOUT
Valley East	19,119	19,644	20,219	20,728	21,028	21,205	21,231	31,469

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 the table below.

Table 6-2 Valley East ICI Projections

	ICI DEVELOPMENT AREAS (HA)						
LAND USE	2016	2021	2026	2031	2036	2041	BUILDOUT
Institutional	0	0	4.64	0	0	0	0
Commercial	0	0	5.25	0	8.18	0	0
Industrial	0	0	23.02	0	79.12	0	0
Total	**0**	**0**	**32.91**	**0**	**87.3**	**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 the Valley East servicing area, one street has been placed on the priority list for sewer and water servicing and an additional one street for sewer servicing only. 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 Valley East 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 *Population Projections and Development of Unit Rates Technical* Memorandum (WSP, 2014).

6.4 FUTURE WASTEWATER FLOW PROJECTIONS AND INFRASTRUCTURE NEEDS

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

$$\textbf{Projected Average Day Generation} \\ = \textbf{Base Generation} + \textbf{Additional Residential Generation} + \textbf{Additional ICI Generation} \\ + \textbf{Average Extraneous Flow}$$

The flows corresponding to the population growth forecasts to Ultimate Buildout are presented in Table 6-3 below.

Table 6-3 Flow Projections

YEAR	POPULATION	AVERAGE DAY FLOW (M^3/D)	MAXIMUM DAY FLOW (M^3/D)
Base	19,119	5,414	15,061
2016	19,644	5,579	14,988
2021	20,219	5,796	15,570
2026	20,728	7,392	19,858
2031	21,028	7,498	20,142
2036	21,205	11,521	30,950
2041	21,231	11,527	30,968
Ultimate	31,469	16,985	45,630

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 demand was calculated by applying the respective peaking factor to the average day demand. The maximum day demand for the base year was the historical average.

A desktop analysis of historical wastewater flows and future flow projections is included in **Appendix C**.

6.4.1 VALLEY EAST WWTP CAPACITY

Based on the current WWTP rated capacity of 11,365 m^3/d, the wastewater treatment capacity will be sufficient to service growth projections until 2031. The Valley East WWTP has sufficient capacity to treat Average Day Wastewater Flows generated by both existing and 2041 projected populations in Valley East. Albeit in 2036 projected average day flows surpass the WWTP's rated capacity by 1.4%, this flow average is not deemed to be significant enough to require planning for additional Average Day treatment capacity. Instead, the wastewater flow rates collected at the plant would simply be monitored over time to ensure that actual Average Day flows are not surpassing the flow trends calculated.

The WWTP capacity is plotted with the flow projections on Figure 6-1 below.

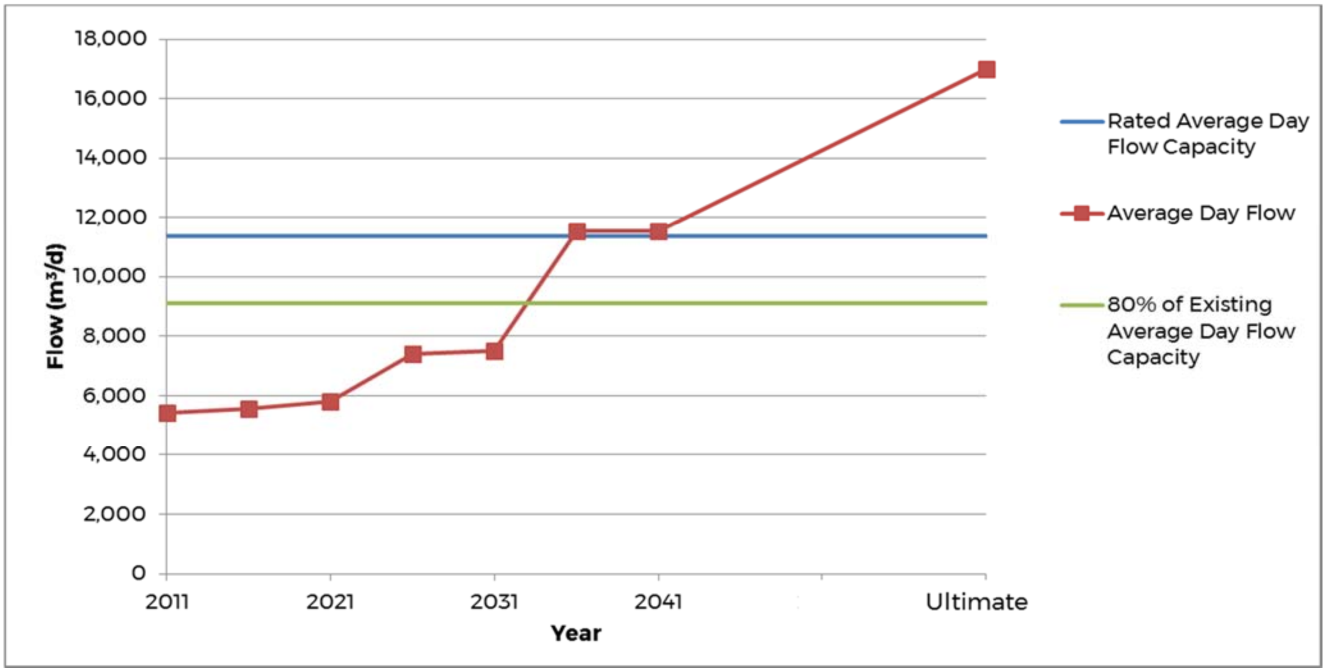

Figure 6-1 Wastewater Flow Projections Compared to Valley East WWTP Rated Capacity

As indicated in the above analysis, the Valley East WWTP can continue operating under its current capacity until approximately 2031.

6.4.2 SEWER NETWORK AND LIFT STATIONS

For each of the scenarios modeled, the system was checked for surcharging of sewers and capacity exceedance at the pumping stations. The peak flows into each of the pumping stations determined through modeling of the various planning scenarios are shown in Table 6-4 below.

Table 6-4 Valley East Lift Station Peak Influent Flow Rates

	CURRENT FIRM CAPACITY	EXISITNG PEAK FLOW	2041 PEAK FLOW	ULTIMATE BUILDOUT
Fleming	25.1	6.6	6.6	6.6
Helene	40.3	92.4	111.8	122.4
Hillsdale	52.2	9.1	21.1	21.1

	CURRENT FIRM CAPACITY	EXISITNG PEAK FLOW	2041 PEAK FLOW	ULTIMATE BUILDOUT
Jeanne D'Arc	110	170.1	171.8	180
Madeleine	15.2	3.0	3.0	3.1
Spruce	74	119.3	126.2	144
St. Isidore	27.9	18	18.5	21.7
Tena	22	1.75	2	2.1
Tupper	9.4	0.94	2.7	3.0

7 HYDRAULIC MODELLING

7.1 APPROACH

A basic sanitary model for the City of Greater Sudbury was received from the City. The model was created in Bentley's SewerGEMS by City staff. The model is an all pipe model of the sanitary network in these systems, but some critical information such as pipe data, invert elevations and lift station characteristics were missing. The model now includes this information as well as key vertical infrastructure in each system, including lift stations and treatment facilities.

The model was loaded with wet weather flow data. A water balance was completed to determine I&I rates for both dry and wet weather flow. The results from the water balance were compared against I&I rates developed through flow monitoring, and the greater of the two values, for each system, was used to load the model.

Current (2011) and future (2016-Ultimate Buildout, in 5 year increments) population data was added to the model using the City's planning data, summarized in previous sections of this report.

Future dry and wet weather flow scenarios were developed for each of the horizon years: 2016, 2021, 2026, 2031, 2036, 2041, and Ultimate Buildout. However, model results did not vary from 2016 to 2041; therefore, this report discusses findings for 2041 and Ultimate Buildout, compared against existing (2011).

7.2 MODELLING FINDINGS

The model was used to check sewer capacity and flow velocity. The majority of sewers in Valley East flow at less than 50% of the available capacity through to Ultimate Buildout under the wet weather flow condition.

Flow velocities in Valley East are less than 0.6 m/s in most areas through to Ultimate Buildout under the wet weather flow condition.

Maps in **Appendix B** illustrate the modeling results for the 2011, 2041, and Ultimate dry and wet weather flow scenarios.

8 CONCLUSIONS

An assessment of the Valley East Wastewater System was completed to identify infrastructure requirements to service forecasted growth in the community.

The conclusions of the assessment are summarized below.

— The WWTP is deemed to have sufficient average day capacity to service growth to 2041; however, in 2036 the projected average day flow exceeds the rated capacity by 1.4%. The flows to the plant should be monitored in case per capita wastewater generation rates increase.

— The capacities at Helene LS, Jeanne D'Arc LS, and Spruce LS are exceeded by peak inflow from 2011 through to Ultimate Buildout.

— Most sewers in the Valley East system operate at less than 50% capacity, but have flow velocities less than the City's standard of 0.6 m/s. This is the case for all horizon years.

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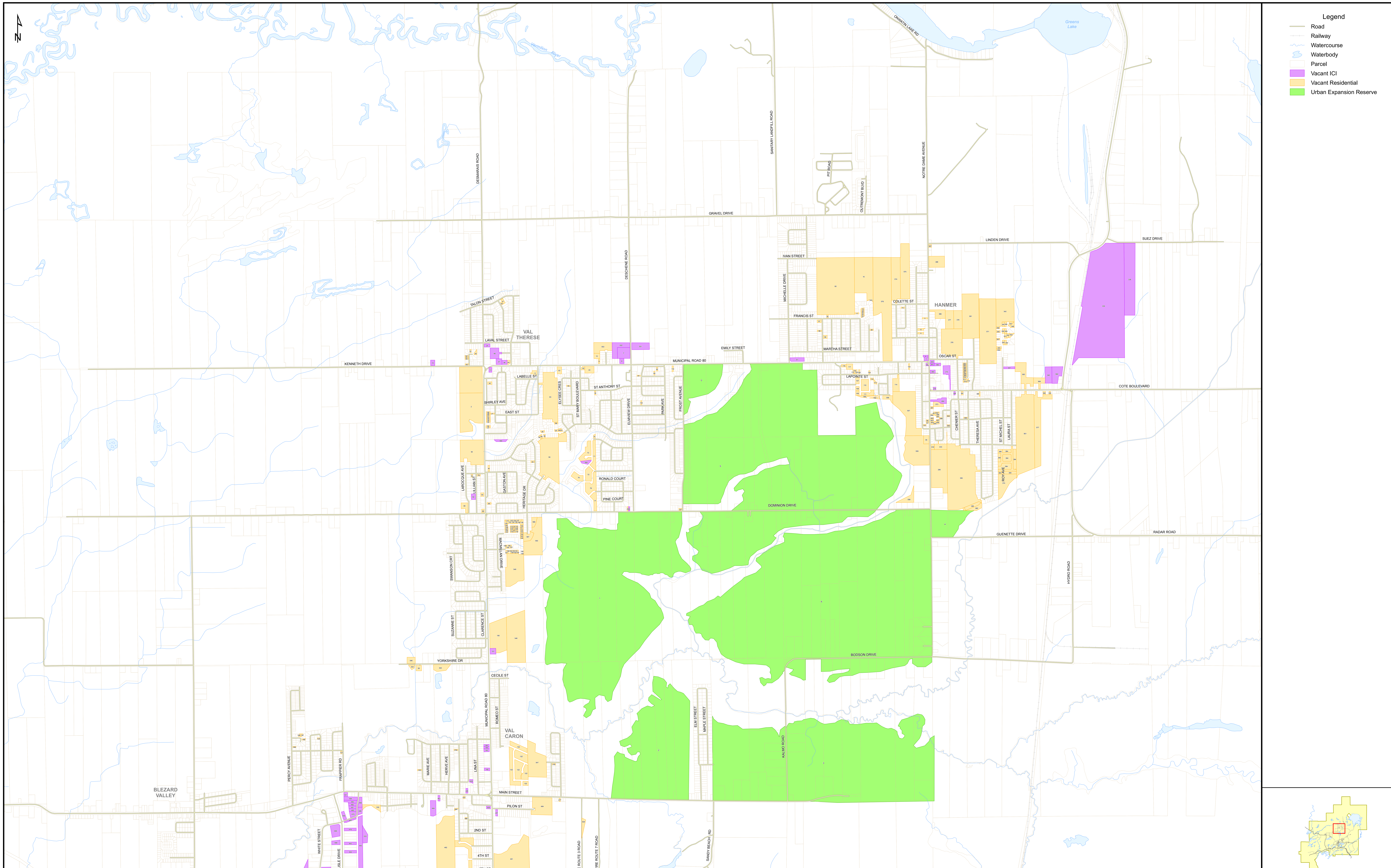
APPENDIX

A RESIDENTIAL AND ICI DEVELOPMENT AREAS



Sudbury
WSP

Project No. 121-23026-00

Sudbury Water and Wastewater Master Plan

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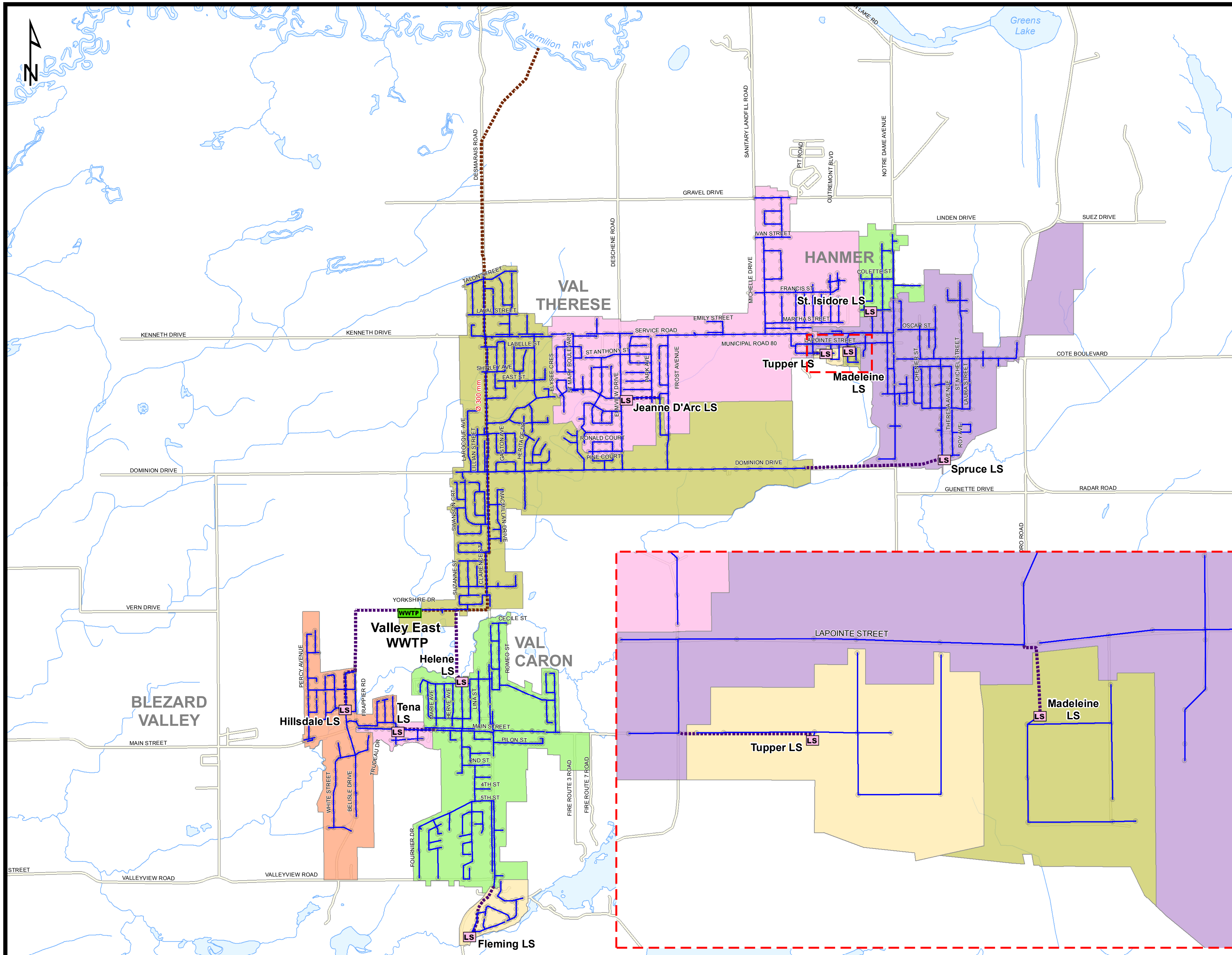
Map 4

Valley East
2011 - Ultimate Buildout
Residential and ICI
Development Areas

APPENDIX

B WASTEWATER MODEL RESULTS



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Sudbury Water and Wastewater Master Plan

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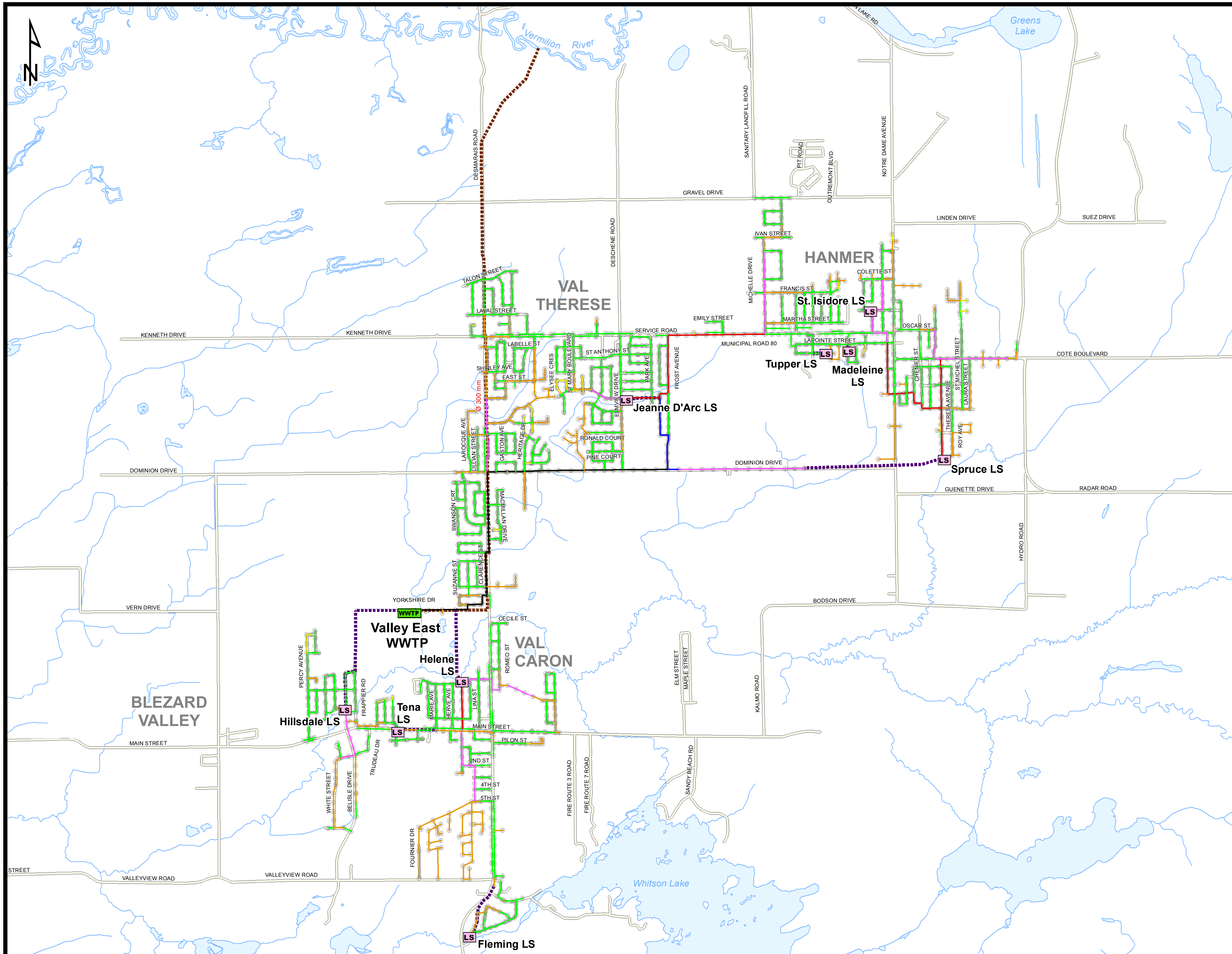
Map 4.1

Valley East Wastewater System - Sewershed Map



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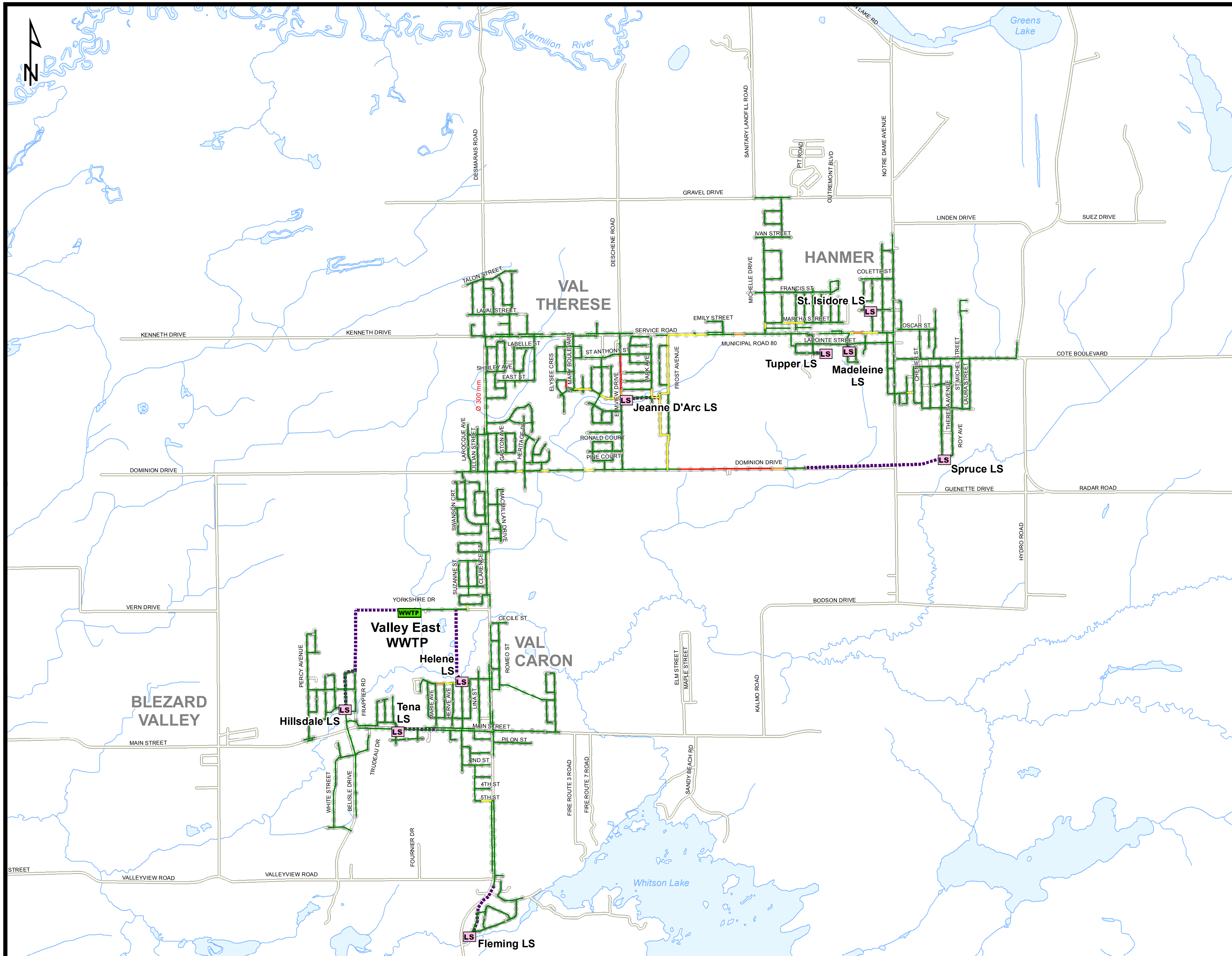
Map 4.2

Valley East Wastewater System - Conduit Diameter



Legend
- LS Lift Station
- WWTP Sanitary Treatment Plant
- Manhole
- Waterbody
- Road
- Watercourse
- Forcemain

Conduit: Flow/Capacity (Design) (%)
- <= 50
- <= 80
- <= 100
- > 100

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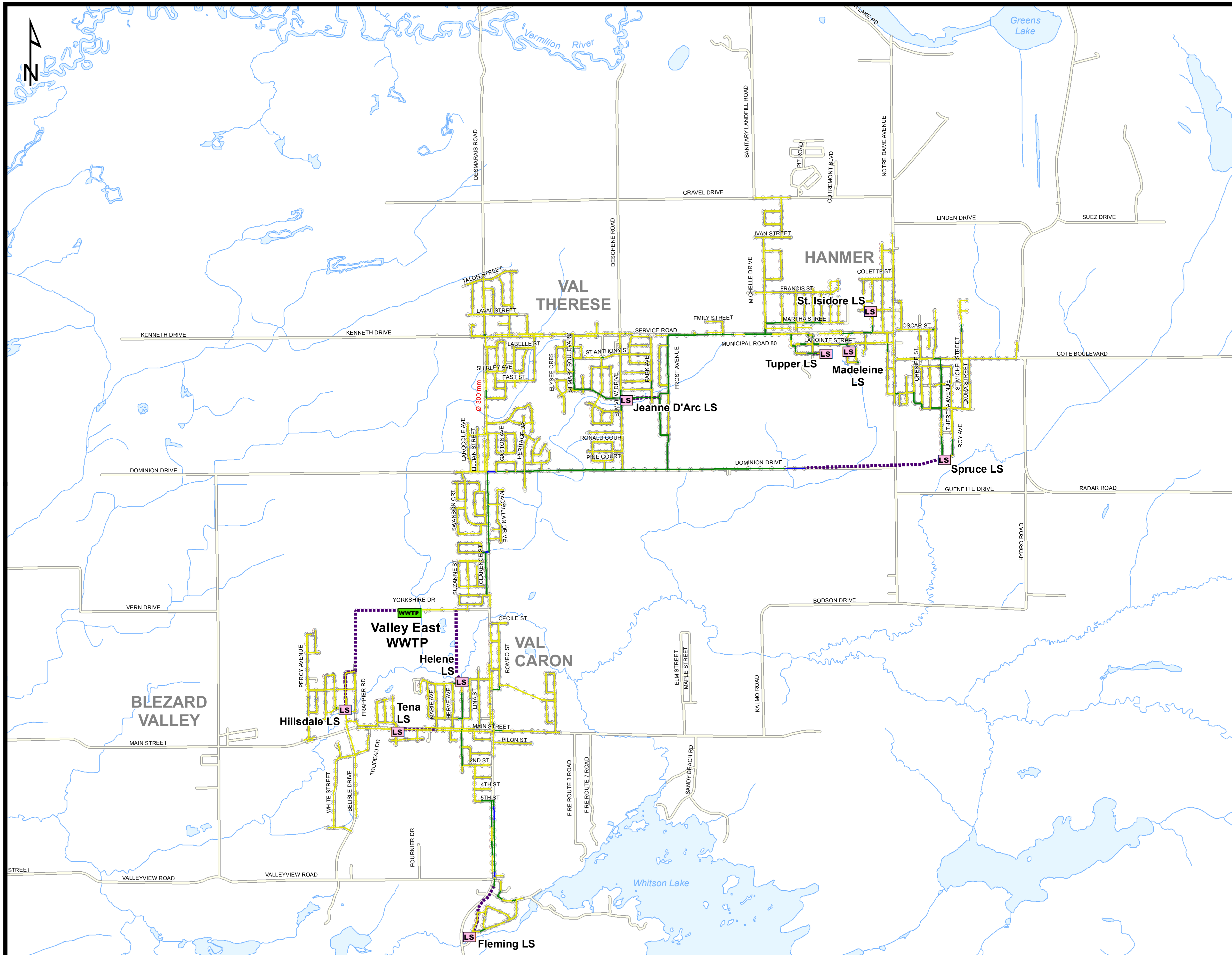
Map 4.3.1

Valley East Wastewater System - Conduit Flow/Capacity (Design)

Scenario: 2011WWF_2yrStorm



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Sudbury Water and Wastewater Master Plan

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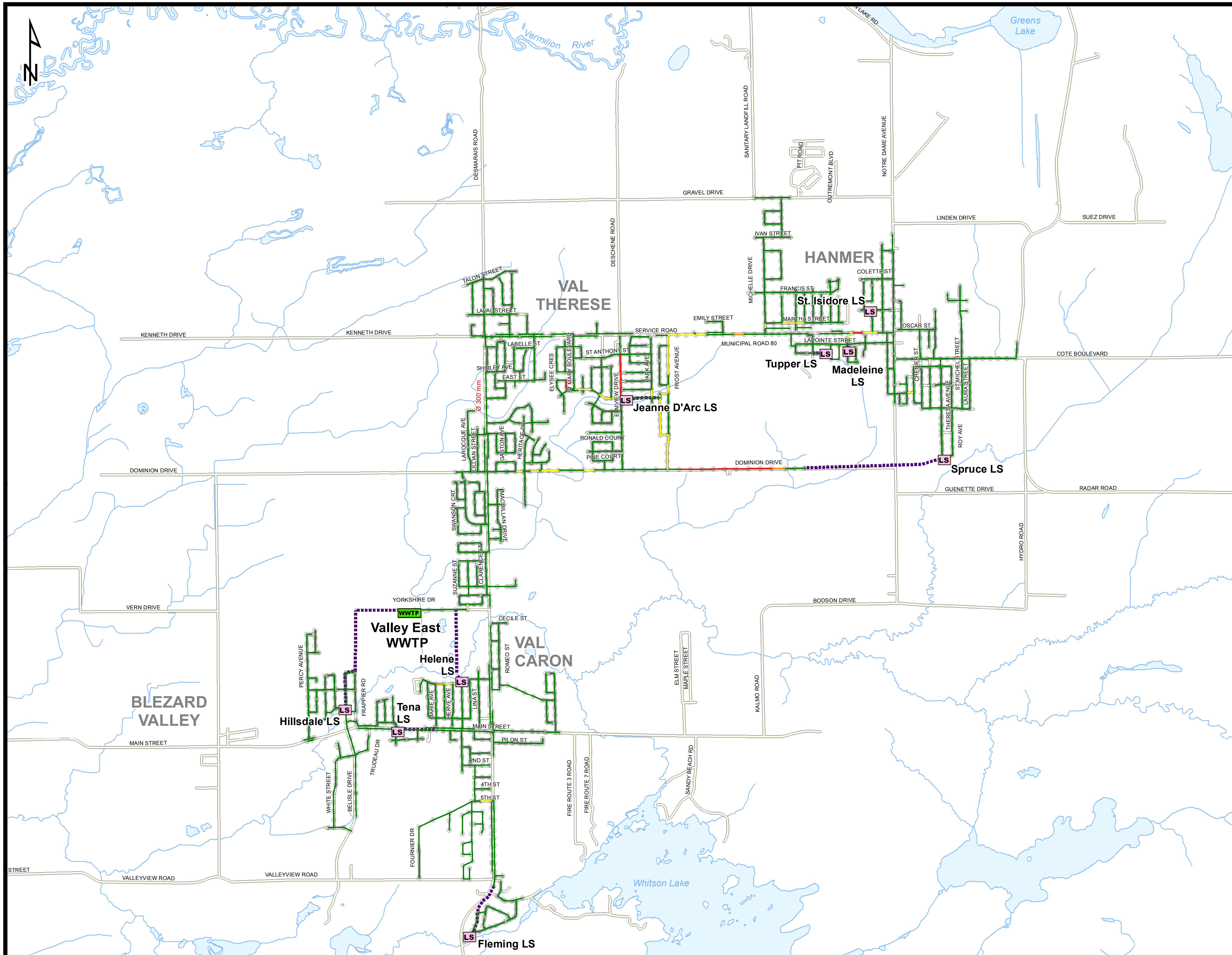
Map 4.3.2

Valley East Wastewater System - Conduit Velocity

Scenario: 2011WWF_2yrStorm



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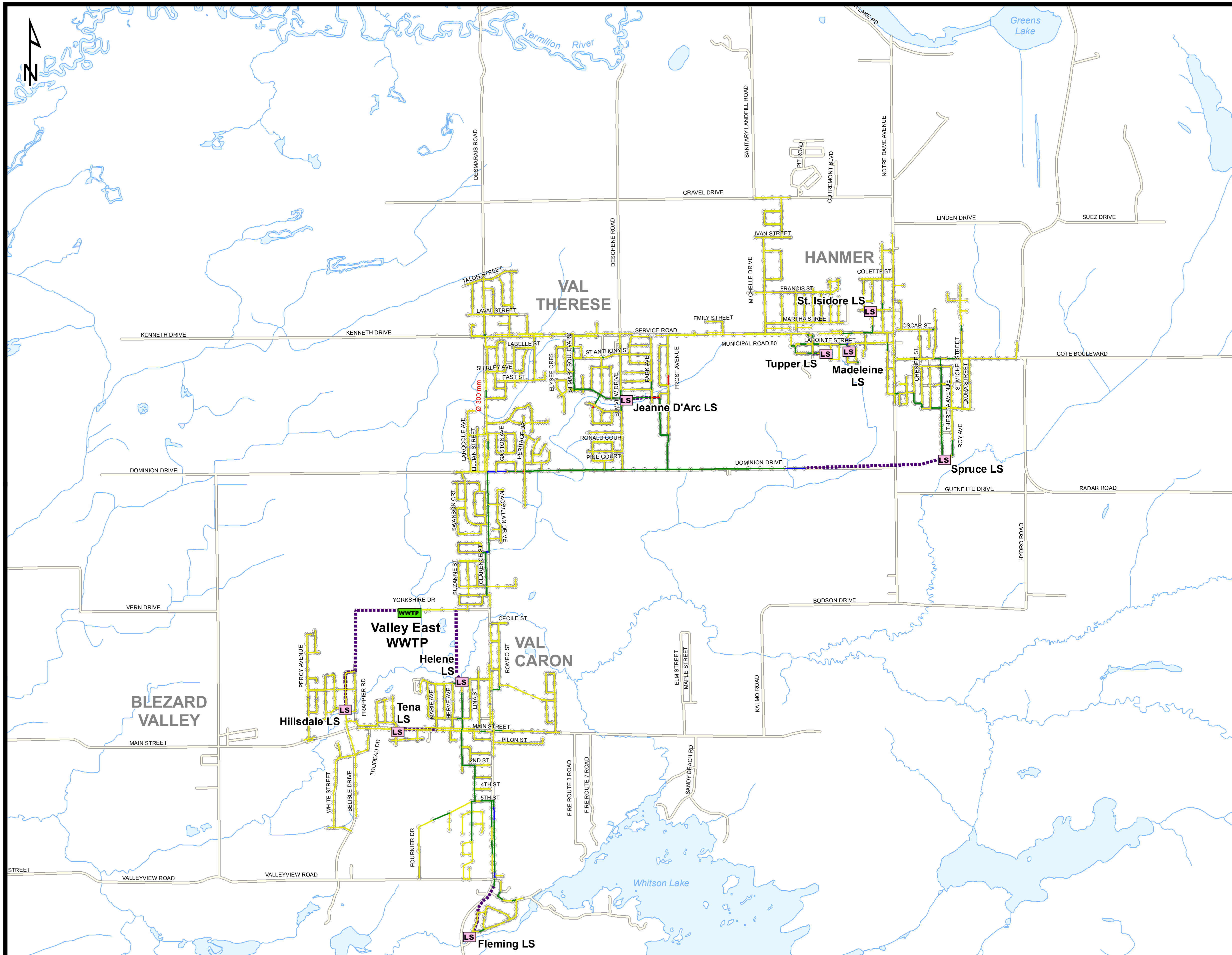
Map 4.4.1

Valley East Wastewater System - Conduit Flow/Capacity (Design)

Scenario: 2041WWF_2yrStorm



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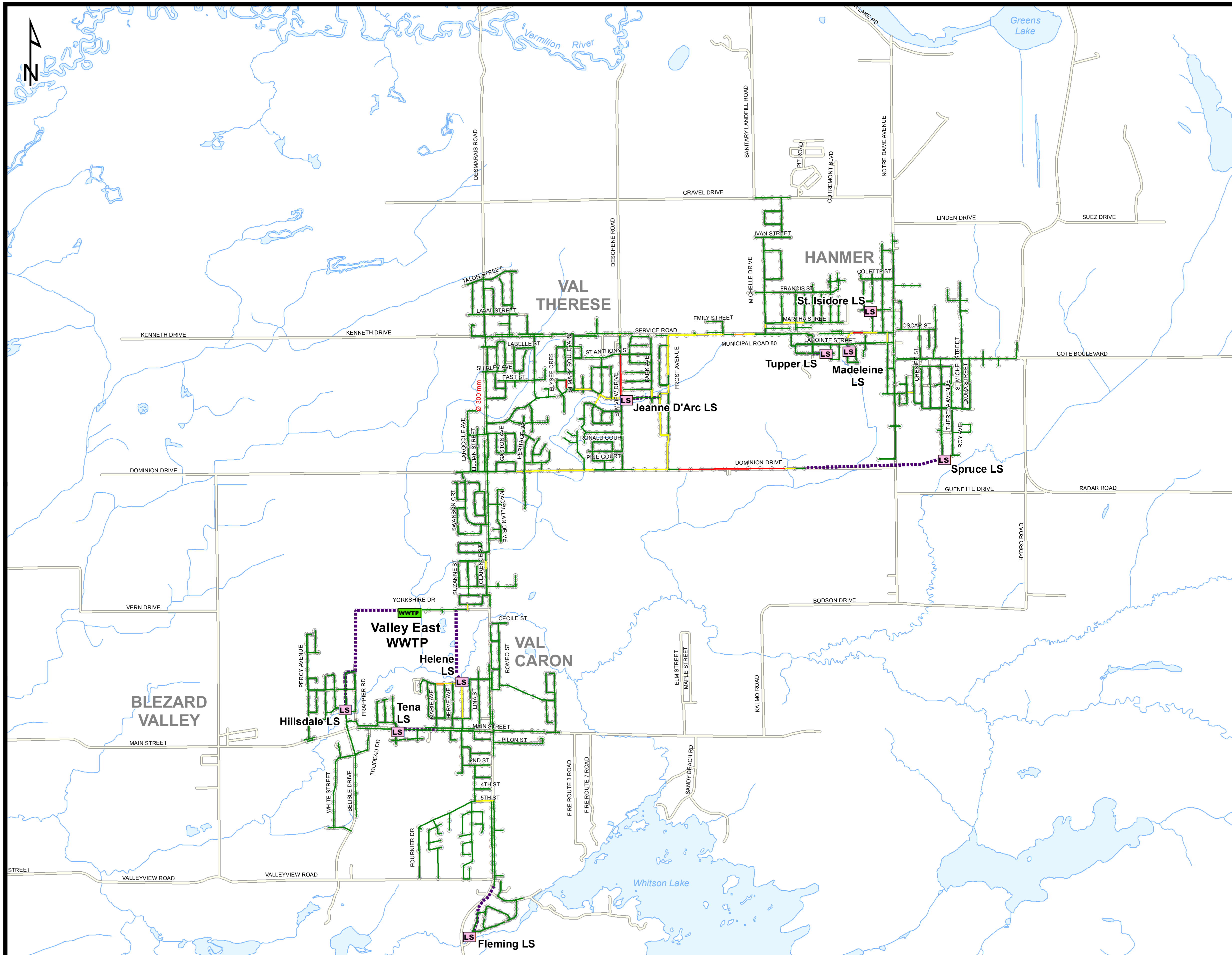
Map 4.4.2

Valley East Wastewater System - Conduit Velocity

Scenario: 2041WWF_2yrStorm



Legend

- LS Lift Station
- WWTP Sanitary Treatment Plant
- Manhole
- Waterbody
- Road
- Watercourse
- Forcemain

Conduit: Flow/Capacity (Design) (%)

- <= 50
- <= 80
- <= 100
- \> 100

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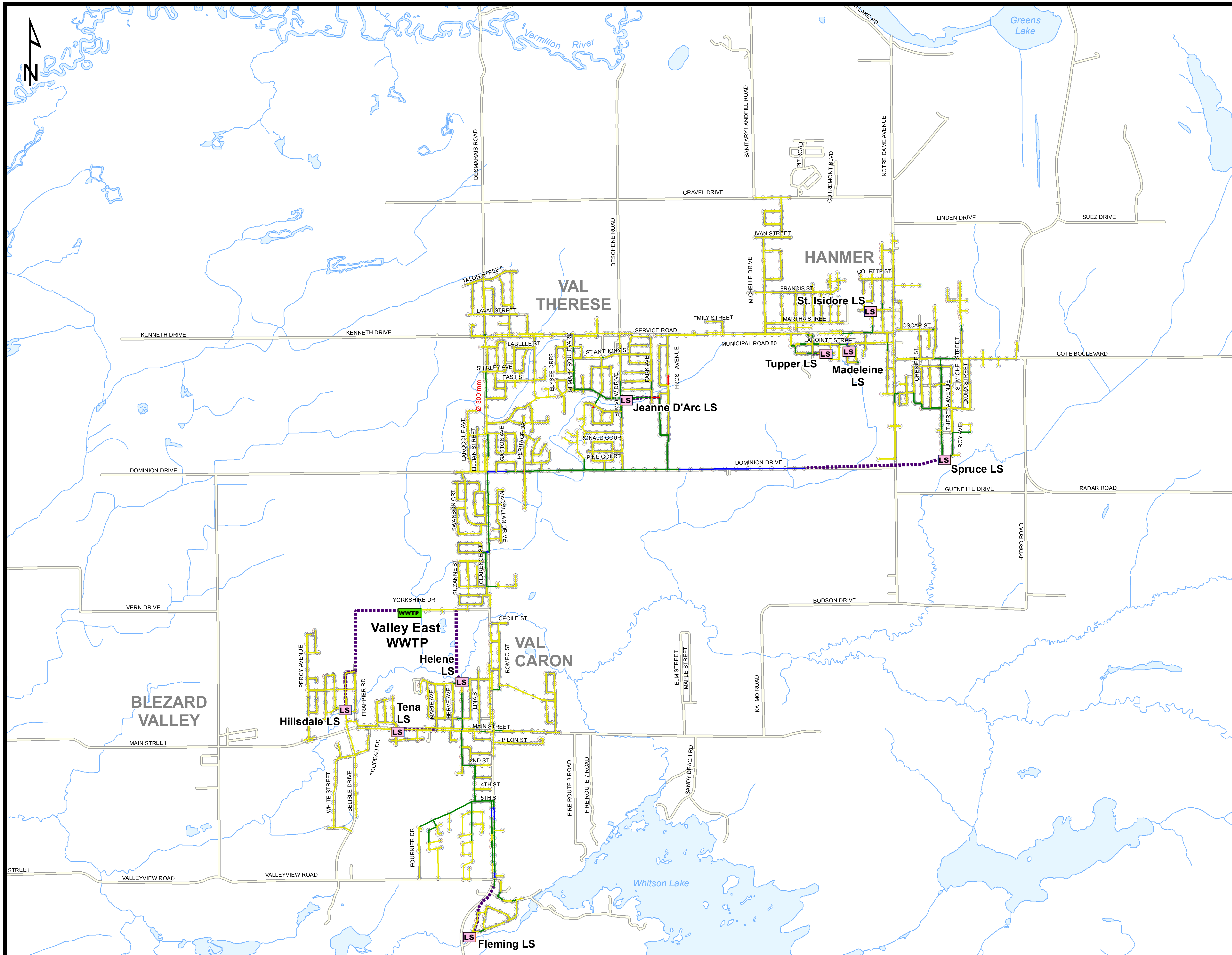
Map 4.5.1

Valley East Wastewater System - Conduit Flow/Capacity (Design)

Scenario: ULTWWF_2yrStorm



Legend

- LS Lift Station
- WWTP Sanitary Treatment Plant
- Manhole
- Waterbody
- Road
- Watercourse
- Forcemain

Conduit: Velocity (m/s)

- <= 0.6
- <= 1.5
- <= 3.0
- > 3.0

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Sudbury Water and Wastewater Master Plan

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Map 4.5.2

Valley East Wastewater System - Conduit Velocity

Scenario: ULTWWF_2yrStorm

APPENDIX

C WASTEWATER GENERATION AND CAPACITY ASSESSMENTS

Valley East - Wastewater Flow Forecasts

	Units	2009	2010	2011	2012	2013	Summary	Design Criterion	Comments
Average Day Flow	(m^3/d)	6,893	4,958	4,928	4,566	5,724	5,414	5,414	From Annual Reports
Max Day Flow	(m^3/d)	23,060	6,370	14,450	8,580	22,844	15,061	15,061	From Annual Reports
Max Day Factor		3.35	1.28	2.93	1.88	3.99	2.69	2.69	Calculated - Max Day Flow divided by Average Day Flow
Peak Flow	(m^3/d)	Not Available						0	Peak hour flows were not available
Peak Flow	(L/s)							0	
Peak Flow Factor									

	2009	2010	2011	2012	2013	Summary	Design Criterion
Population (Existing Areas)	19,119	19,119	19,119	19,119	19,119	19,119	19,119
Ratio of Residential to Total Customers	0.86	0.86	0.86	0.86	0.86	0.86	
Residential Share of Average Day Demand (m^3/d)	5951	4281	4255	3942	4942	4674	
Residential Flow Unit Rate $(m^3/cap/d)$	0.311	0.224	0.223	0.206	0.258	0.244	0.250
Average Institutional Flow Unit Rate $(m^3/ha/d)$							28.0
Average Commercial Flow Unit Rate $(m^3/ha/d)$							28.0
Average Industrial Flow Unit Rate $(m^3/ha/d)$							35.0
Average Extraneous Flow Unit Rate $(m^3/ha/d)$							11.23

	2016	2021	2026	2031	2036	2041	Ultimate Buildout
Population (Existing Areas)	19,119	19,119	19,119	19,119	19,119	19,119	19,119
Population (Growth Areas)	526	1,100	1,609	1,910	2,086	2,113	12,350
Total Population	19644	20219	20728	21028	21205	21231	31469
Residential (ha)	3.00	9.51	11.00	13.72	13.72	13.72	271.72
Institutional (ha)			4.64	4.64	4.64	4.64	4.64
Commercial (ha)			5.25	5.25	13.43	13.43	13.43
Industrial (ha)			23.02	23.02	102.14	102.14	102.14
ICI (ha)	0.00	0.00	32.91	32.91	120.21	120.21	120.21
Total (ha)	3.00	9.51	43.91	46.63	133.93	133.93	391.93

	2016	2021	2026	2031	2036	2041	Ultimate Buildout
Average Residential Flows (m^3/d) - Existing	5,414	5,414	5,414	5,414	5,414	5,414	5,414
Average Residential Flows (m^3/d)	131	275	402	477	522	528	3,088
Average Institutional Flow (m^3/d)	0	0	130	130	130	130	130
Average Commercial Flow (m^3/d)	0	0	147	147	376	376	376
Average Industrial Flow (m^3/d)	0	0	806	806	3,575	3,575	3,575
Average Extraneous Flow (m^3/day)	34	107	493	524	1,504	1,504	4,403
Average Day Flow (m^3/d)	5,579	5,796	7,392	7,498	11,521	11,527	16,985
Max Day Flow (m^3/d)	14,988	15,570	19,858	20,142	30,950	30,968	45,630

Comments

Total Population (Hemson)

ICI development areas were assigned to planning years based on the stage of the application. Draft Approved were assigned to 2016, Legal Lots of Record to 2026, and Designated Developable to 2036.

Areas are cumulative and carry from the development year, all the way through to Ultimate Buildout

This ratio is based on Water Billing Records for the area and is an approximation of the residential portion of demand.

CGS design standards do not suggest average rates for institutional and commercial, but require that rates be developed based on usage. In this case, water demands rates were used and assume a conservative 1:1 ratio of water to sewage.

CGS design standards do not suggest average rates for institutional and commercial, but require that rates be developed based on usage. In this case, water demands rates were used and assume a conservative 1:1 ratio of water to sewage.

CGS design standards average rate for light industrial of 35 $m^3/ha/d$.

From CGS Design Standards, peak rate for new developments divided by an assumed peaking factor of 3. This factor would be applied only to new developments, which are assumed to be leak-tight, and have minimal extraneous flow.

This includes all contribution from existing ICI and infiltration. The base flow was assumed to be the average day flow to the plant for the 2011-2013 period.

Obtained by multiplying the projected population growth by the unit rate.

Institutional growth area multiplied by unit flow rate.

Commercial growth area multiplied by unit flow rate.

Industrial growth area multiplied by unit flow rate.

ALTERNATIVE CALCULATION METHOD

Per Capita Flow (m3/cap/day)	0.361	0.259	0.258	0.239	0.299	0.283	0.283

	2016	2021	2026	2031	2036	2041	Ultimate Buildout	Comments
	5563	5725	5869	5955	6005	6012	8911	
Average Day Flow (m^3/d)								Multiplying the total population by the consolidated per capita flow factor.
Max Day Flow (m^3/d)	14,944	15,381	15,768	15,997	16,131	16,151	23,939	

SENSITIVITY ANALYSIS

Analyze sensitivity of forecasted flows to unit rate

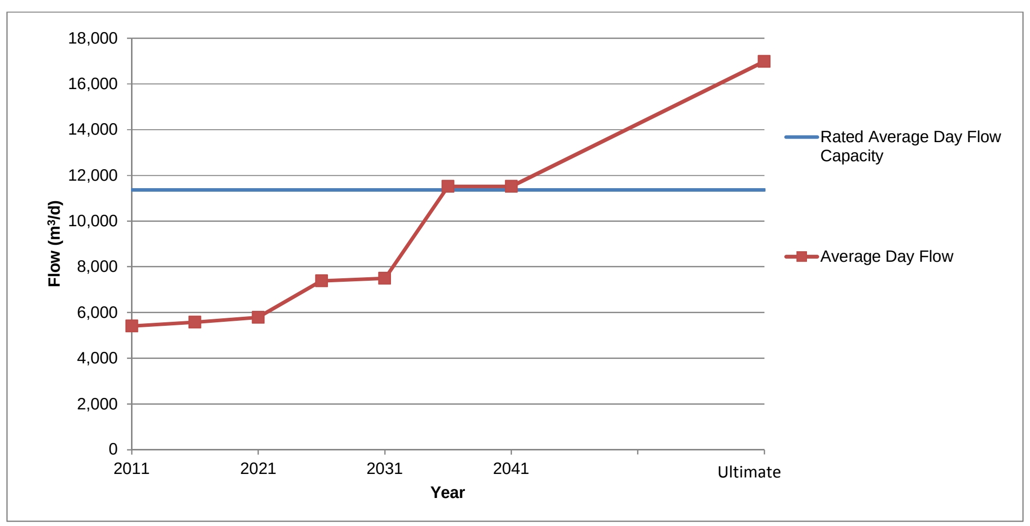
	Average Day Flow (m^3/d)							Ultimate Buildout
	Unit Rate (m^3/cap/d)	2016	2021	2026	2031	2036	2041	2061
Base Scenario - Residential Historical Maximum	0.244	5,579	5,796	7,392	7,498	11,521	11,527	16,985
Combined Historical Maximum	0.283	5,596	5,832	7,445	7,561	11,590	11,597	17,394
City Standards	0.360	5,637	5,917	7,569	7,708	11,750	11,760	18,343

Analyze sensitivity of forecasted flows to max day factor

	Max Day Flow (m^3/d)							Ultimate Buildout
	Max Day Peaking Factor	2016	2021	2026	2031	2036	2041	2061
Base Scenario - Historical Max	2.52	14,069	14,615	18,641	18,907	29,053	29,069	42,832
Historical Average	3.99	14,988	15,570	19,858	20,142	30,950	30,968	45,630

CAPACITY CHECK

								Ultimate Buildout
	2011	2016	2021	2026	2031	2036	2041	2061
Rated WPCP ADF Capacity (m^3/d)	11,365	11,365	11,365	11,365	11,365	11,365	11,365	11,365
Average Day Flow (m^3/d)	5,414	5,579	5,796	7,392	7,498	11,521	11,527	16,985
Maximum Day Flow (m^3/d)	15,061	14,988	15,570	19,858	20,142	30,950	30,968	45,630



CITY OF GREATER SUDBURY WATER AND WASTEWATER MASTER PLAN

WAHNAPITAE WASTEWATER SYSTEM GAP ANAYSIS AND STATUS QUO REPORT

CITY OF GREATER SUDBURY

DRAFT

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

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 existing Wahnapitae Wastewater System. Based on population growth projections and design criteria discussed in the *Population Projections and Development of Unit Rates Technical Memorandum* (WSP, 2014), 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 Wahnapitae Wastewater System would continue to be a stand-alone system. Any potential interconnections between Wahnapitae 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

Wahnapitae is a small community located in the east end of the City of Greater Sudbury, east of both Coniston and Sudbury proper. The wastewater from Wahnapitae is treated at the Wahnapitae sewage lagoons. The lagoons also receive backwash water from the Wanapitei Water Treatment Plant (WTP). Discharge from the lagoons occurs seasonally. Additional information regarding the plant is provided in Section 3.2.

Mapping in Appendix A shows the Wahnapitae 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 *Population Projections and Development of Unit Rates Technical Memorandum* (WSP, 2014).

Existing development in the study area is mixed, and includes residential as well as industrial land uses. Based on the City's planning data, little growth is expected for Wahnapitae. The area population is expected to increase from 1,397 in 2011 to 1,418 by 2041 and 1,479 by Ultimate Buildout. No ICI growth is expected. Growth is discussed further in Section 6.1.

3 OVERVIEW OF EXISTING SYSTEM

All wastewater generated in Wahnapitae is collected and treated at the Wahnapitae Wastewater Lagoon. There is a single lift station in Wahnapitae and a sewer collection system.

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

The Wahnapitae Wastewater System is shown in Appendix B.

3.1 LIFT STATIONS

Table 3-1 below provides a summary of the main features of the lift station.

Table 3-1 Wahnapitae Sewage Lift Stations

LIFT STATION	YEAR CONSTRUCTED[1]	LAST UPGRADED AND/OR RETROFITTED[2]	WET WELL VOLUME (M3)[1]	LIFT STAITON CAPACITY AND FORCEMAIN INFORMATION[2]
Riverside	1979	1980	8.0 (7.3 sq.m)	Two wet well pumps with a firm capacity of 52 L/s nominal 1332 m long, 300 mm diameter forcemain of unknown material

[1] Obtained or estimated from dimensions found in as-built and record, assuming water level does not exceed the High Water Alarm Level or, in absence of this alarm level, the inlet sewer invert.
[2] Obtained from the City's Wastewater Lift Stations Operations Manual and station as-built drawings.

3.2 WAHNAPITAE LAGOON

The Wahnapitae Lagoons, comprised of three cells, are located in the Town of Nickel Centre and has a rated capacity of 1,246 m^3/d. Treated wastewater is seasonally discharged to Wahnapitae River. The discharge periods are limited to between March 15 and April 30, and between November 1 and December 15.

The treatment process is illustrated schematically in Figure 3-1 below

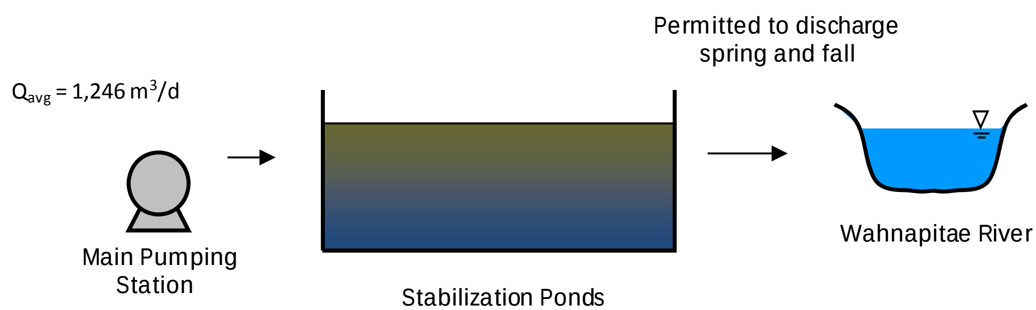

Figure 3-1 Wahnapitae Lagoon System

3.3 KNOWN CHALLENGES

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

- The City is aware of reports that homes in the area have experienced basement flooding events.
- Ground conditions in Wahnapitae pose a challenge for maintaining buried infrastructure. In general, several man holes require resetting annually due to ground conditions.
- Unlike other communities inside the City of Greater Sudbury, Wahnapitae was built around the logging industry not the mining industry. The community hugs the Wahnapitae River which may be a cause of flooding during wet weather events.
- The City's records indicate that connections are typically flushed or otherwise restored or even reconstructed to overcome blockages or other causes of sewer backup and/or basement flooding.

4 HISTORICAL FLOWS AND REVIEW OF OPERATIONAL DATA

Data reported in the 2009 to 2013 *Annual Reports* for the Wahnapitae WWTP was reviewed and analyzed to determine average day and maximum day flows as well as review effluent parameters.

4.1 FLOW DATA

Lagoon flow data from 2009 to 2013 was reviewed. Historical maximum and peak flow data were not available.

The recorded average day and maximum day flows are summarized in Table 4-1 and plotted in Figure 4-1 below.

Table 4-1 Historical Wastewater Flow Data

YEAR	AVERAGE DAY FLOW (M3/D)[1]
2009	1,321
2010	884
2011	565
2012	556
2013	689

[1]*Annual Reports (2009 - 2013).*

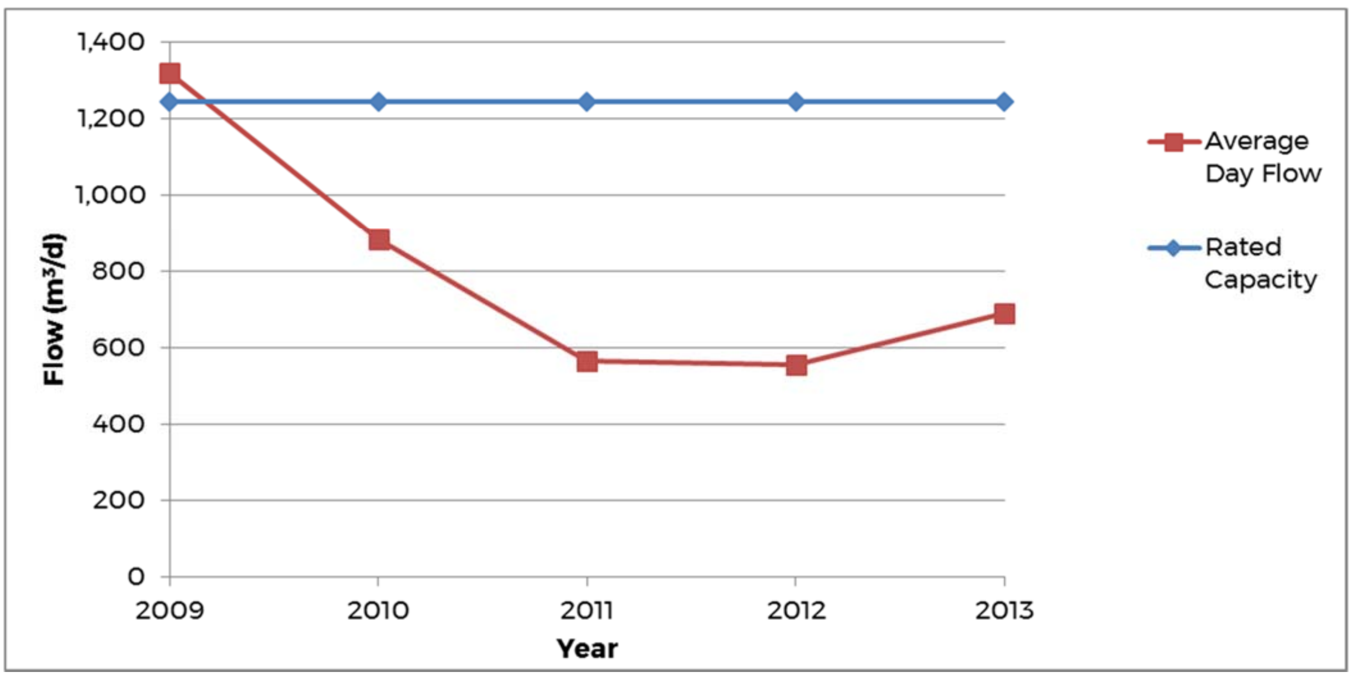

Figure 4-1 Historical Wastewater Flows at the Wahnapitae Lagoons

The trend in the average day flows to the Lagoons from 2009 to 2013 have shown significant variability over the 2009 to 2013 period; however, the average of the flows in the latter three years in this period have been steadier, averaging 603 m^3/day. The historical maximum and peak day flow variations were unable to discern since there was no available data.

Upon comparison with historical rainfall data for the Sudbury station, one finds there is a slight correlation. For example, 2009 had the most precipitation (986.4 mm) for Sudbury station as well as for Lagoon (1,321mm). In 2011 and 2012 the precipitation levels were quite constant for both Sudbury and Wahnapitae. Although in 2010 the precipitation recorded was at its lowest (659.8 mm), flows to the plant were higher than the previous, wetter year. This may indicate that maximum day wastewater generation is weather-dependent and that the system may have substantial levels of inflow and infiltration.

There is no historical available data for the maximum day peaking factor.

4.2 RAW WASTEWATER CHARACTERISTICS

The average raw wastewater characteristics from 2009 to 2012 are summarized in Table 4-2 below. Raw wastewater temperatures were not reported.

Table 4-2 Average Raw Wastewater Characteristics at the Wahnapitae Lagoons (2009-2012)

PARAMETER	AVERAGE VALUE
$CBOD_5$	92.0 mg/L
Suspended Solids	1110 mg/L
Total Phosphorus	12.6
TKN	19.7 mg/L
pH	N/A

Wastewater flows to the Wahnapitae Lagoons are generated predominantly by residential users

4.3 EFFLUENT CRITERIA

The Wahnapitae Lagoons are operated in accordance with MOECC Certificate of Approval (C of A) No. 7439-8BBJYJ dated April 1, 2011.The C of A for the Wahnapitae Lagoons stipulates effluent concentration objectives of $CBOD_5$ and Suspended Solids of 25 mg/L and 30 mg/L, respectively, and concentration limits of 30 mg/L and 40 mg/L, respectively.

4.4 OPERATIONAL DATA

The general lagoon operation was reviewed against the Wahnapitae Lagoon C of A requirements and historical data provided in the Annual Reports from 2009-2012. Historical data is summarized in the table below.

Table 4-3 Historical Effluent Concentrations

EFFLUENT PARAMETER	ANNUAL AVERAGE			
	2009	2010	2011	2012
CBOD5 (mg/L)	12.0	4.8	15.0	4.3
TSS (mg/L)	23.9	28.3	11.7	13.6
TP (mg/L)	0.09	0.05	0.05	0.06

EFFLUENT PARAMETER	ANNUAL AVERAGE			
	2009	2010	2011	2012
pH	6.84	7.92	7.92	7.20
TAN (mg/L)	5.48	2.17	0.79	2.69
E. coli (organisms/100 mL)	N/A	N/A	N/A	N/A

The Wahnapitae Lagoons met effluent limits for $CBOD_5$ and TSS in all years, except for TSS in 2010.

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 Development of 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 WASTEWATER DESIGN CRITERIA

The unit flow criteria for growth adopted for this assessment are shown in Table 5-1 below. These values were recommended in the *Population Projections and Development of Unit Rates Technical Memorandum* (WSP, 2014).

Note that the term "extraneous flows" is used interchangeably with "I&I flows".

Table 5-1 Wastewater System Design Criteria

CRITERIA	VALUE	REFERENCE
Average Day Residential Flow	500 L/cap/day	City's Engineering Design Manual, rounded down from 471 L/ca/d
Average Day Commercial and Institutional Flow	28 m^3/ha/d	Water unit rates, assuming a 1:1 correspondence
Average Day Industrial Flow	35 m^3/ha/d	Water unit rates, assuming a 1:1 correspondence
Average Extraneous Flow	11.2 m^3/ha/d	Peak from City's Engineering Design Manual and assuming a peaking factor of three
Peak Extraneous Flow	33.7 m^3/ha/d	Estimated by assuming the same flow as Coniston community.
Max Day Peaking Factor	3.67	Estimated by assuming the same factor as Coniston community.

Residential average day flows were obtained by multiplying the residential unit rate by the service population.

Maximum day flows to the Lagoons are obtained by multiplying the average day flow by the maximum day peaking factor.

5.2 DESIGN CRITERIA FOR WASTEWATER SYSTEM COMPONENTS AND OPERATION

5.2.1 WASTEWATER TREATMENT

Wastewater treatment facilities are rated for average day flows. Plant effluent limits and objectives are established in the C of A or ECA for each facility. In the case of the Wahnapitae Lagoons, effluent limits and objectives are only available for $CBOD_5$ and TSS.

5.2.2 LIFT STATION PUMPING CAPACITY

The firm capacity of the lift station (with the largest pump out of service) must allow pumping of peak wet weather flows corresponding to its catchment area (MOECC, 2008).

Starting limitations on pump motors generally dictate the minimum size of a wet well. The wet well should be large enough to prevent pump motors from overheating due to frequent starting and stopping, but small enough to avoid long retention times leading to septicity and odor problems (Lin & Lee, 2001).

The station wet well shall be sized such that the number of pump starts per hour does not exceed the maximum value recommended by the pump manufacturer. In other words, the time between pump starts and stops (i.e. the pump cycle time) should not exceed that which results in a pump start frequency greater than that recommended by the pump manufacturer. Typically, submersible pumps can cycle four to 10 times per hour with a maximum cycle time not exceeding 30 minutes (Lin & Lee, 2001). A maximum value of four pump starts per hour was assumed to evaluate wet well sizing requirements.

5.2.3 SEWERS

The sewer system is typically sized to convey peak instantaneous (peak wet weather) flows. Sewage flows are made up of wastewater discharges from residential, commercial, institutional and industrial establishments, plus extraneous flow components from such sources as groundwater and surface runoff.

In addition to being able to convey peak flows, sufficient flow velocity should be maintained to transport the sewage solids to avoid deposition and the development of nuisance conditions under lower flow conditions. The minimum acceptable flow velocity in sewers is 0.6 m/s (City of Greater Sudbury, 2012).

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 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 *Population Projections and Development of 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 Wahnapitae population with wastewater servicing is projected to increase by 21 people by 2041 and 82 people by Ultimate Buildout.

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

Table 6-1 Wahnapitae Population Projections

SYSTEM	2011	2016	2021	2026	2031	2036	2041	ULTIMATE BUILDOUT
Wahnapitae	1,397	1,402	1,408	1,413	1,416	1,418	1,418	1,479

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.

Wahnapitae does not have any expected ICI growth.

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 Wahnapitae 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 *Population Projections and Development of Unit Rates Technical Memorandum* (WSP, 2014).

6.3 FUTURE WASTEWATER FLOW PROJECTIONS AND INFRASTRUCTURE NEEDS

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

$$\textbf{Projected Average Day Generation} = \textbf{Base Generation} + \textbf{Additional Residential Generation} + \textbf{Additional ICI Generation} + \textbf{Average Extraneous Flow}$$

The flows corresponding to the population growth forecasts to Ultimate Buildout are presented in Table 6-2 below.

Table 6-2 Flow Projections

YEAR	POPULATION	AVERAGE DAY FLOW (M^3/D)	MAXIMUM DAY FLOW (M^3/D)
Base	1,397	832	N/A
2016	1,402	839	3,078
2021	1,408	847	3,110
2026	1,413	855	3,138
2031	1,416	861	3,159
2036	1,418	864	3,170
2041	1,418	864	3,170

Ultimate	1,479	896	3,288

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 demand was calculated by applying the respective peaking factor to the average day demand.

A desktop analysis of historical wastewater flows and future flow projections is included in Appendix C.

6.3.1 WAHNAPITAE LAGOONS CAPACITY

The rated average day capacity of the Wahnapitae Lagoons is 1,246 m^3/d, and is compared to the current and future flow projections on the figure below.

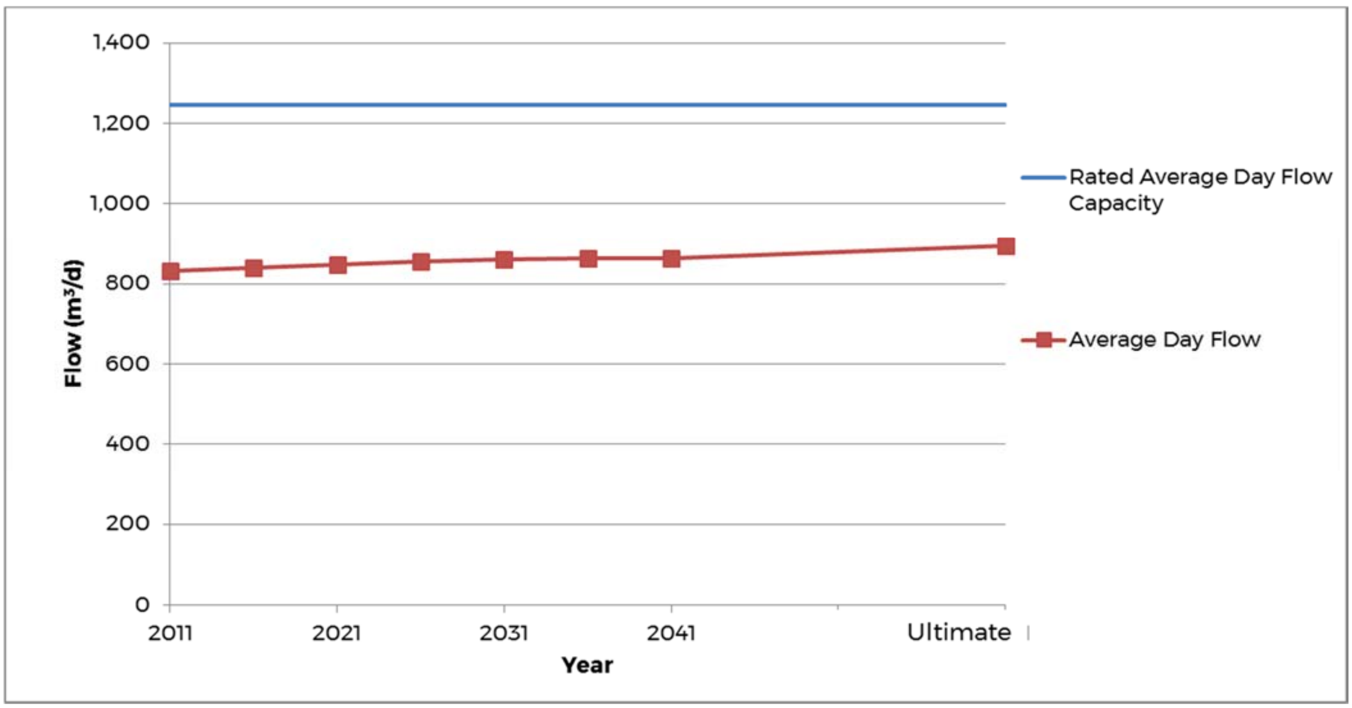

Figure 6-1 Wastewater Flow Projections Compared to Wahnapitae Lagoons Rated Capacity

As indicated in the above analysis, the Wahnapitae Lagoons can continue operating under the current capacity until beyond 2041 and Ultimate Buildout.

6.3.2 SEWER NETWORK AND LIFT STATION

For each of the scenarios modeled, the system was checked for surcharging of sewers and capacity exceedance at the lift stations. The peak flows into each of the lift stations was determined from the computer simulations for the various planning scenarios and is presented in Table 6-3 below. The table also shows the design/rated flow for the pumps, their capacity based on drawdown tests and the computer simulated flow for comparison.

Table 6-3 Lift Station Peak Influent Flow Rates

	CURRENT FIRM CAPACITY	EXISTING PEAK FLOW	2041 PEAK FLOW	ULTIMATE BUILDOUT
Riverside	52	141.7	141.9	141.9

Therefore, the inflows to the Riverside LS exceed the drawdown capacity in the 2011 to Ultimate Buildout scenarios.

7 HYDRAULIC MODELLING

7.1 APPROACH

A basic sanitary model for the City of Greater Sudbury was received from the City. The model was created in Bentley's SewerGEMS by City staff. The model is an all pipe model of the sanitary network in these systems, but some critical information such as pipe data, invert elevations and lift station characteristics were missing. The model now includes this information as well as key vertical infrastructure in each system, including lift stations and treatment facilities.

The model was loaded with wet weather flow data. A water balance was completed to determine I&I rates for both dry and wet weather flow. The results from the water balance were compared against I&I rates developed through flow monitoring, and the greater of the two values, for each system, was used to load the model.

Current (2011) and future (2016-Ultimate Buildout, in 5 year increments) population data was added to the model using the City's planning data, summarized in previous sections of this report.

Future dry and wet weather flow scenarios were developed for each of the horizon years: 2016, 2021, 2026, 2031, 2036, 2041, and Ultimate Buildout. However, model results did not vary from 2016 to 2041; therefore, this report discusses findings for 2041 and Ultimate Buildout, compared against existing (2011).

7.2 MODELLING FINDINGS

The model was used to check sewer capacity and flow velocity. Maps in Appendix B illustrate the modeling results for the 2011, 2041, and Ultimate wet weather flow scenarios based on a theoretical 2-year storm.

Many of the sewers in the Wahnapitae system operate at less than 50% capacity. One sewer segment near the Riverside LS is above capacity, detailed in the appended maps. These findings are generally consistent from 2011 through to Ultimate Buildout.

From 2011 through to Ultimate Buildout, flow velocities in some sewers in Wahnapitae are below the City's standard of 0.6 m/s. However, remaining sewers are between 0.6 and 1.5 m/s, in accordance with the standards.

8 CONCLUSIONS

An assessment of the Wahnapitae Wastewater System was completed to identify infrastructure requirements to service forecasted growth in the community.

The conclusions of the assessment are summarized below.

— The Lagoons are deemed to have sufficient average day capacity to service growth to 2041 and Ultimate Buildout.

— The Riverside LS does not have capacity to convey peak flows from 2011 through to Ultimate Buildout.

— Many of the sewers in the Wahnapitae system operate at less than 50% capacity, except for one segment near the Riverside LS. This is apparent from 2011 through to Ultimate Buildout.

— From 2011 through to Ultimate Buildout, flow velocities in some sewers in Wahnapitae are below the City's standard of 0.6 m/s. However, remaining sewers are between 0.6 and 1.5 m/s, in accordance with the standards.

BIBLIOGRAPHY

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Lin, S. D., & Lee, C. C. (2001). *Water and Wastewater Calculations Manual.* McGraw-Hill.

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WSP. (2014). Baseline Review Report.

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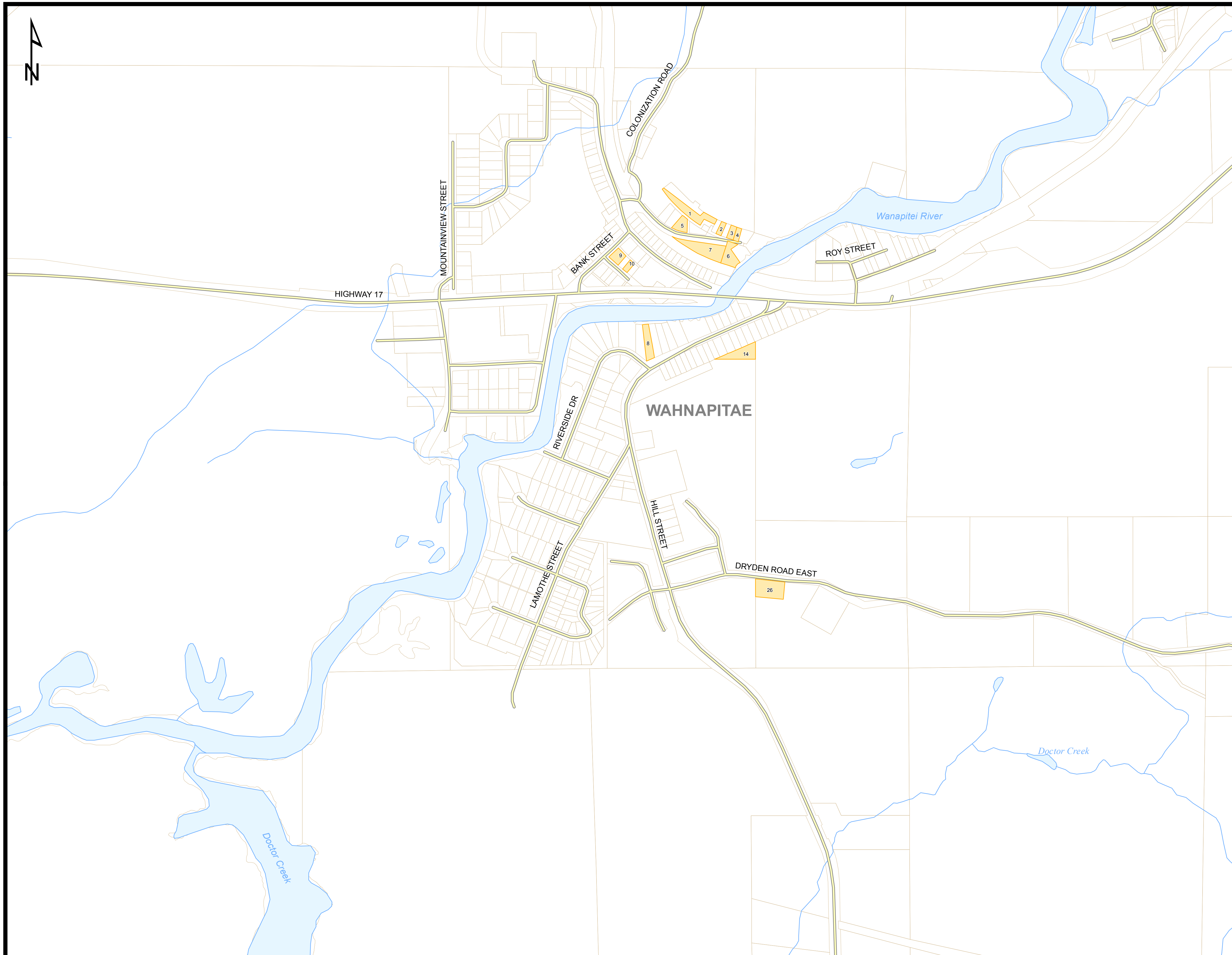
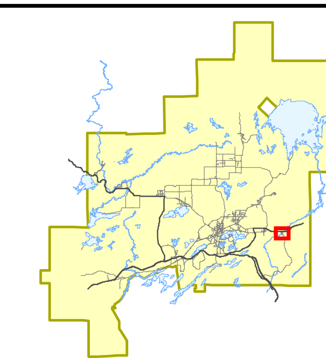
APPENDIX

A RESIDENTIAL AND ICI DEVELOPMENT AREAS



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



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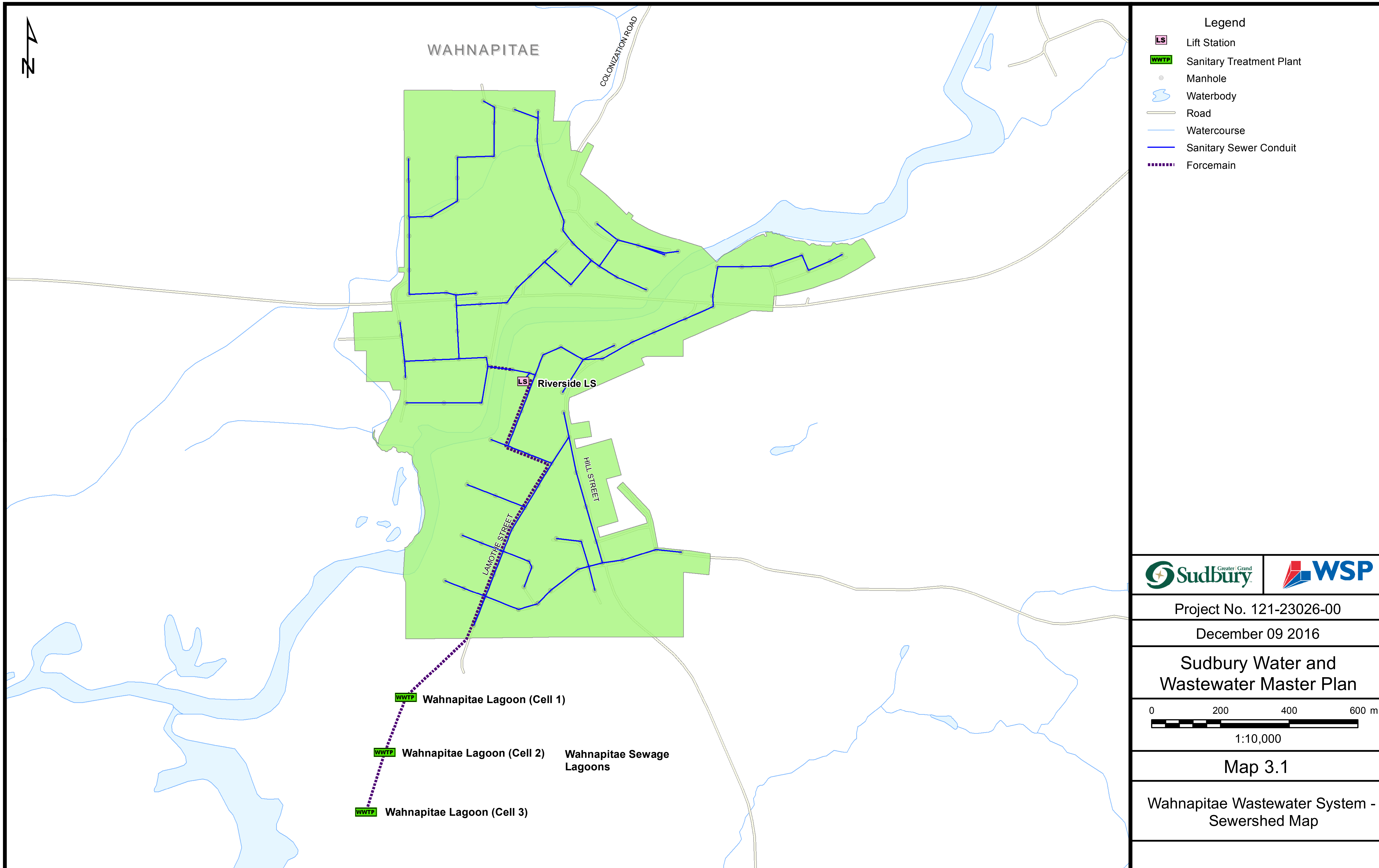
Map 14

Wahnapitae
2011 - Ultimate Buildout
Residential and ICI
Development Areas

APPENDIX

B WASTEWATER MODEL RESULTS



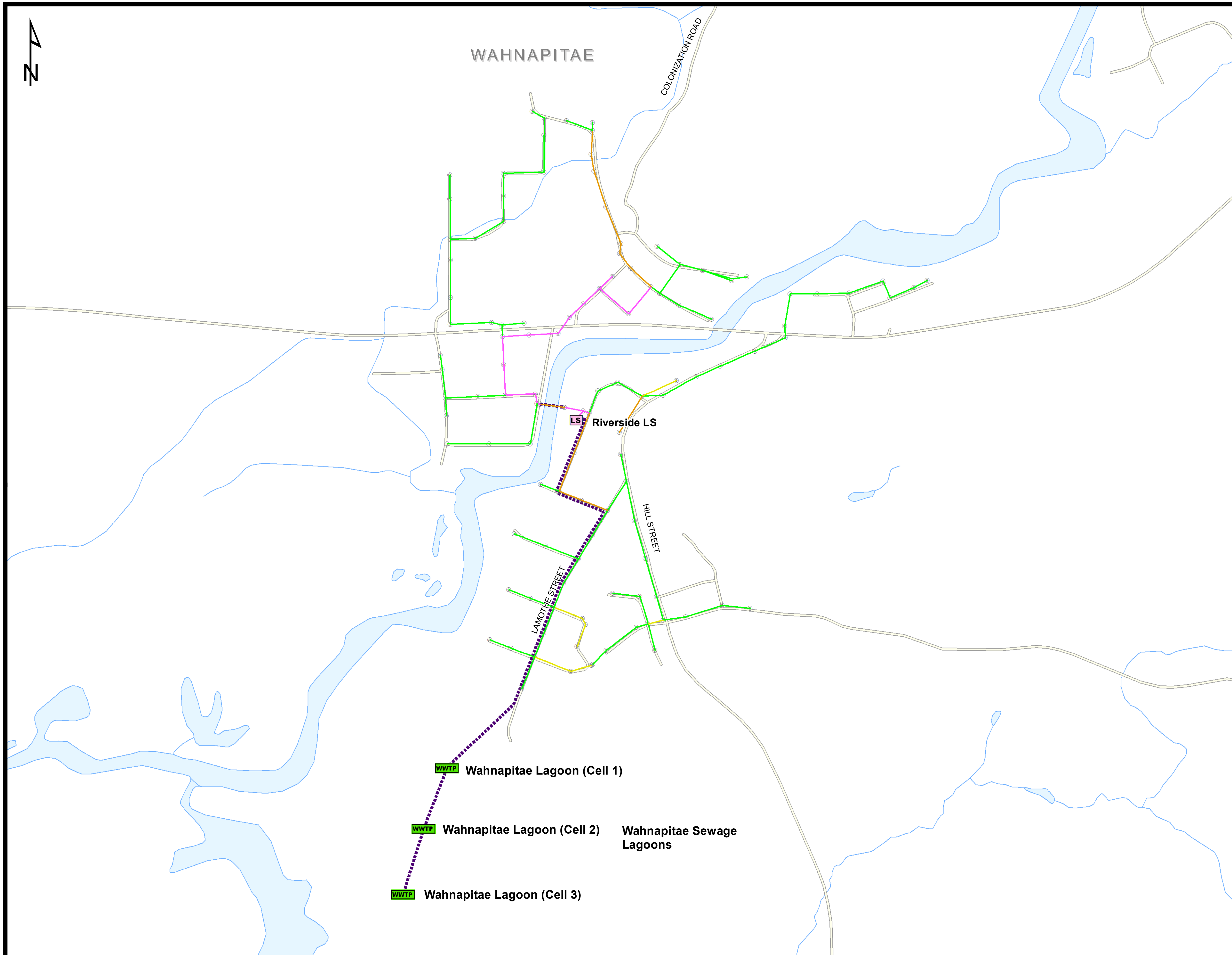
Project No. 121-23026-00

December 09 2016

Sudbury Water and Wastewater Master Plan

Map 3.1

Wahnapitae Wastewater System - Sewershed Map



Legend

- Lift Station
- Sanitary Treatment Plant
- Manhole
- Waterbody
- Road
- Watercourse
- Forcemain

Conduit: Diameter (mm)

- <= 150
- <= 200
- <= 300
- <= 450
- <= 600
- <= 750
- > 750

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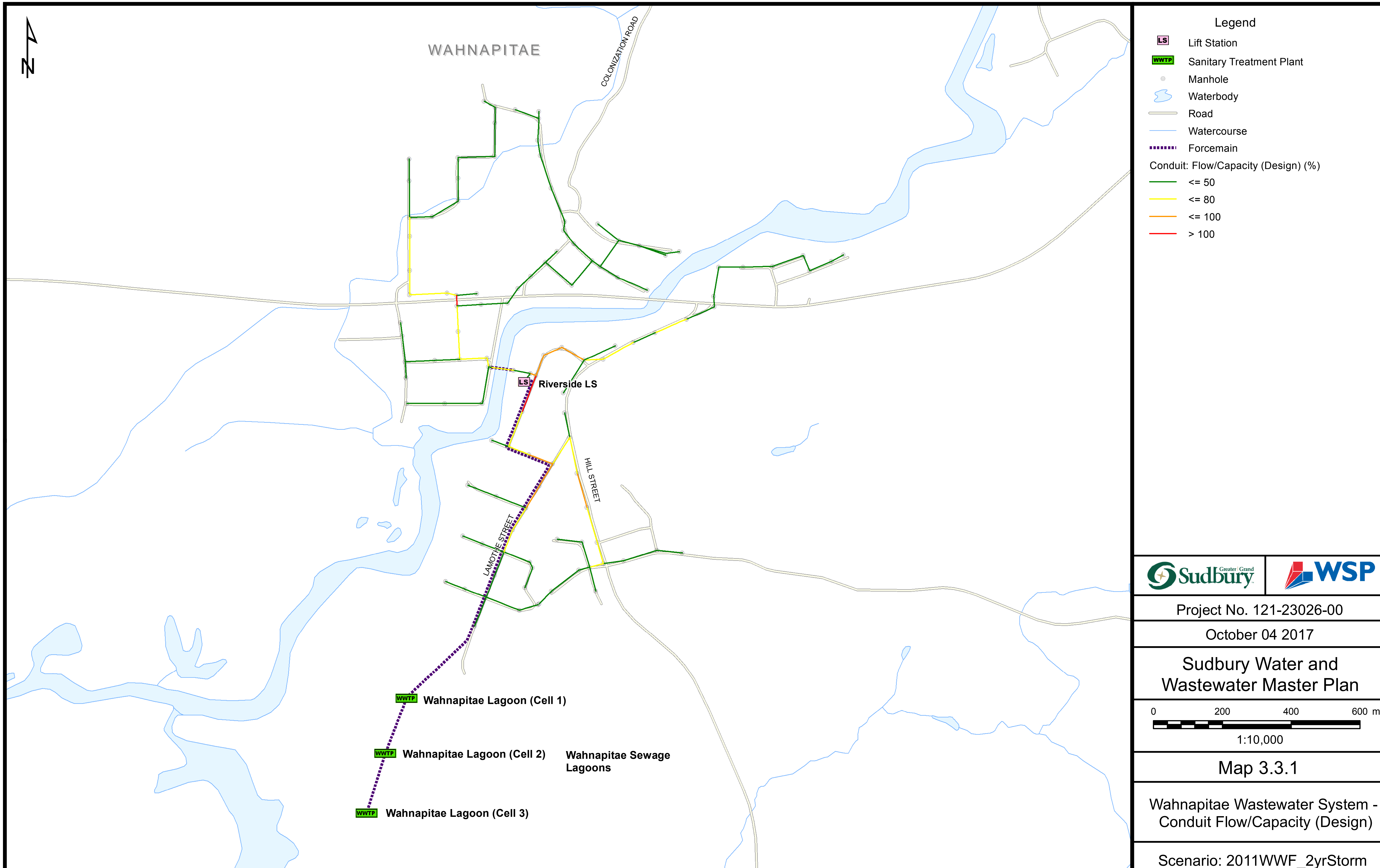
Map 3.2

Wahnapitae Wastewater System - Conduit Diameter



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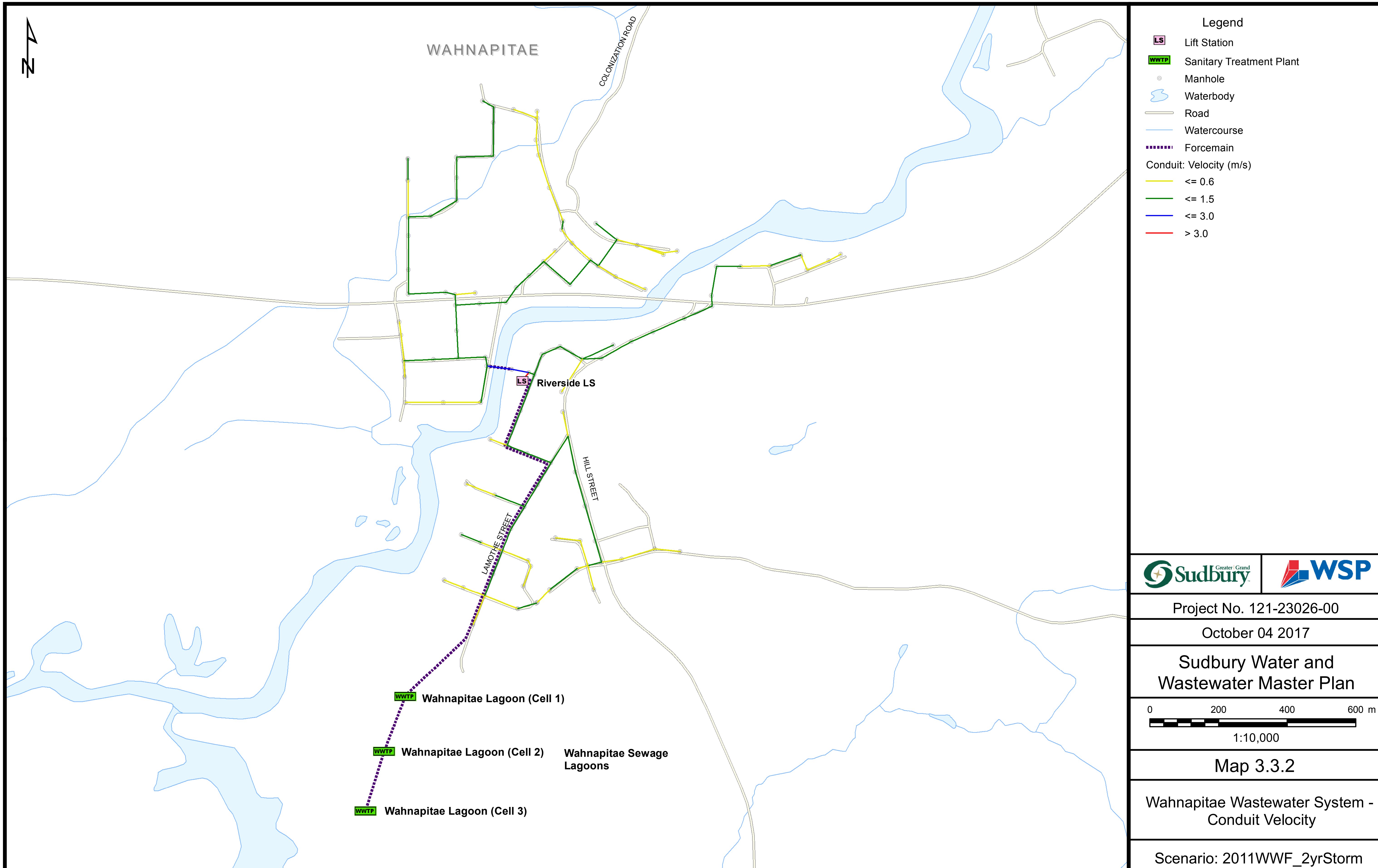
Map 3.3.1

Wahnapitae Wastewater System - Conduit Flow/Capacity (Design)

Scenario: 2011WWF_2yrStorm



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Sudbury Water and Wastewater Master Plan

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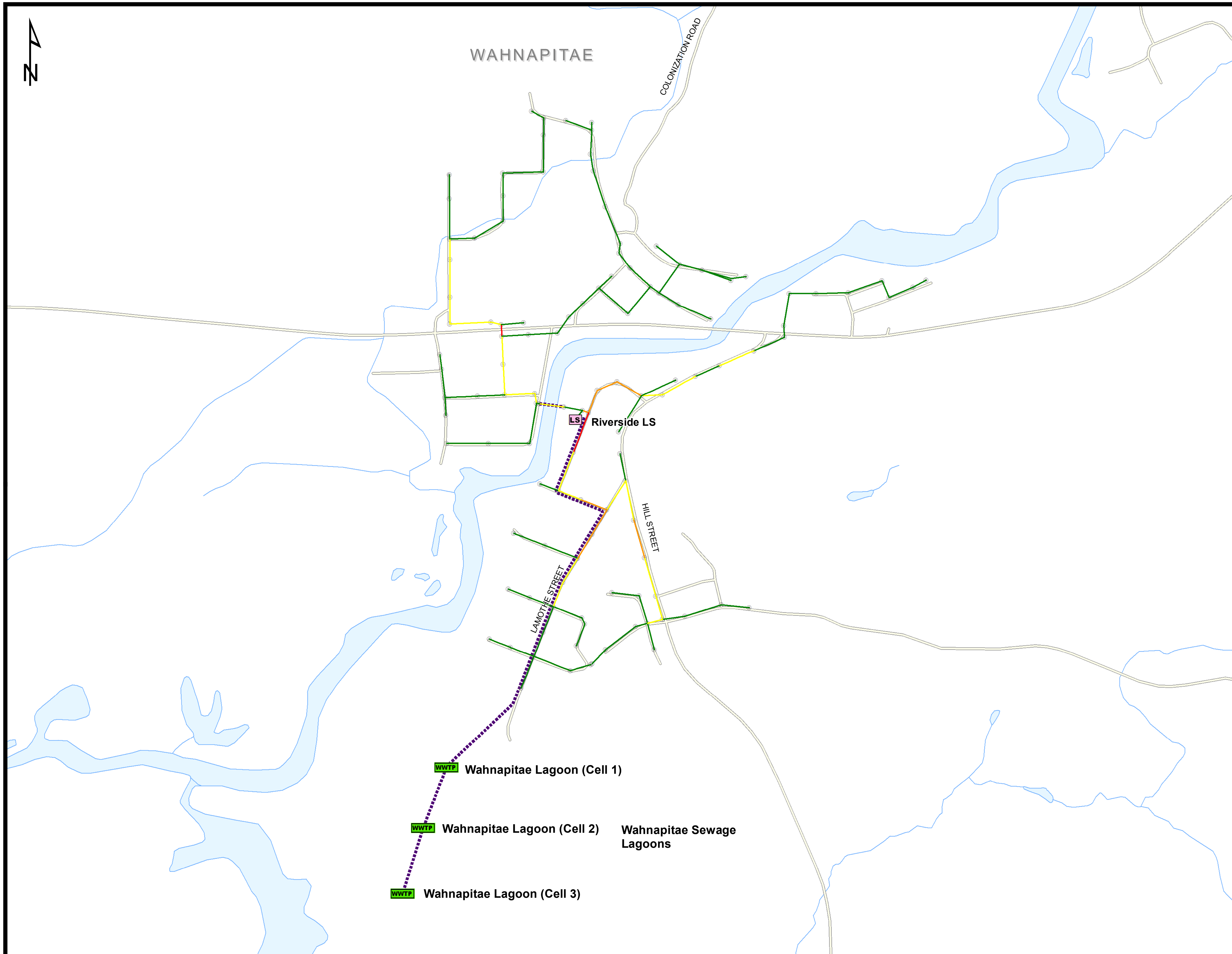
Map 3.3.2

Wahnapitae Wastewater System - Conduit Velocity

Scenario: 2011WWF_2yrStorm



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Sudbury Water and Wastewater Master Plan

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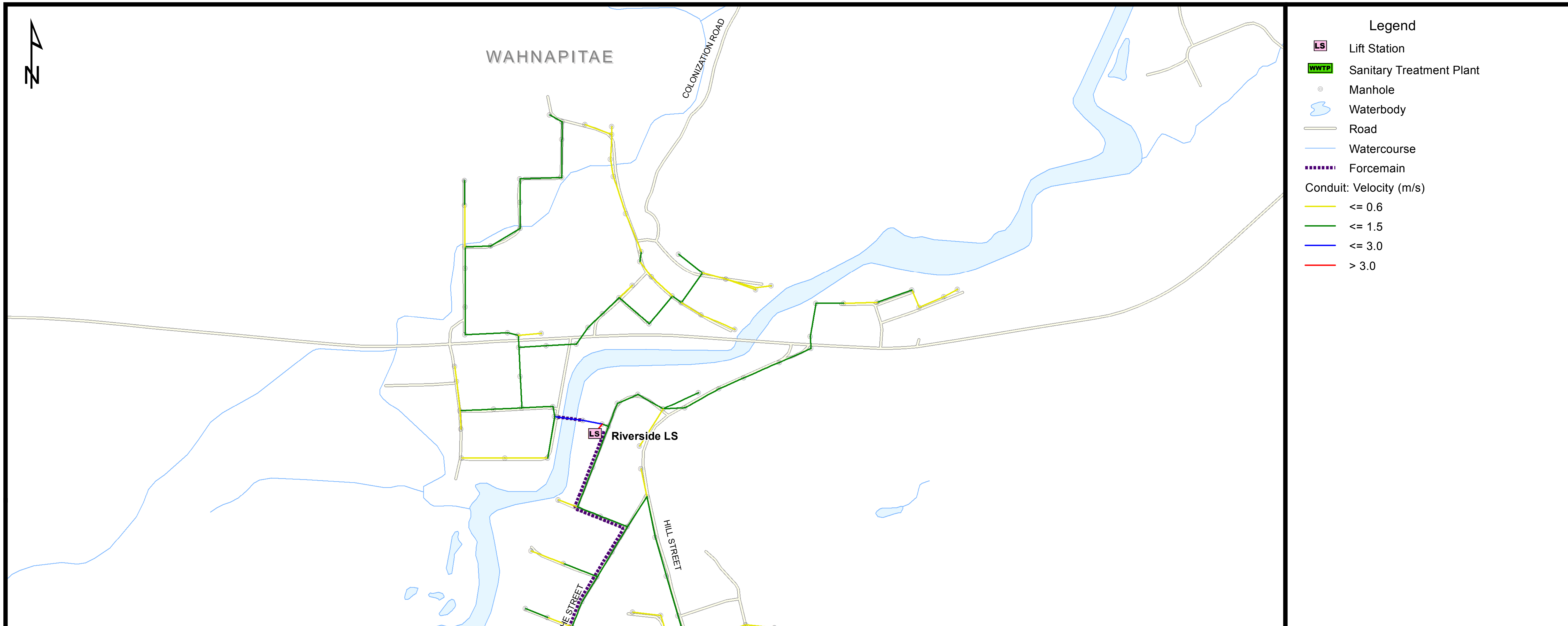
Map 3.4.1

Wahnapitae Wastewater System - Conduit Flow/Capacity (Design)

Scenario: 2041WWF_2yrStorm



Project No. 121-23026-00

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Sudbury Water and Wastewater Master Plan

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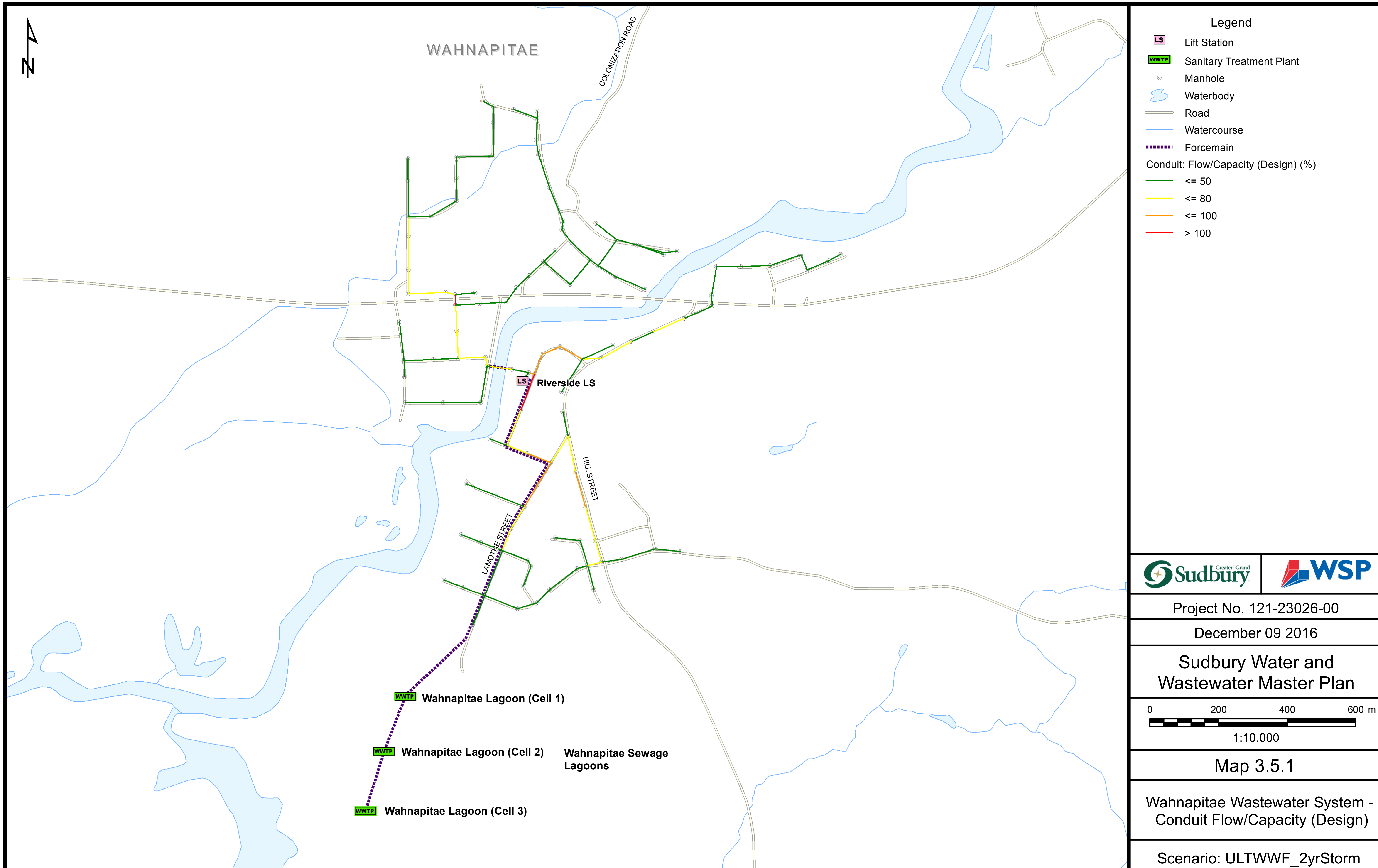
Map 3.4.2

Wahnapitae Wastewater System - Conduit Velocity

Scenario: 2041WWF_2yrStorm



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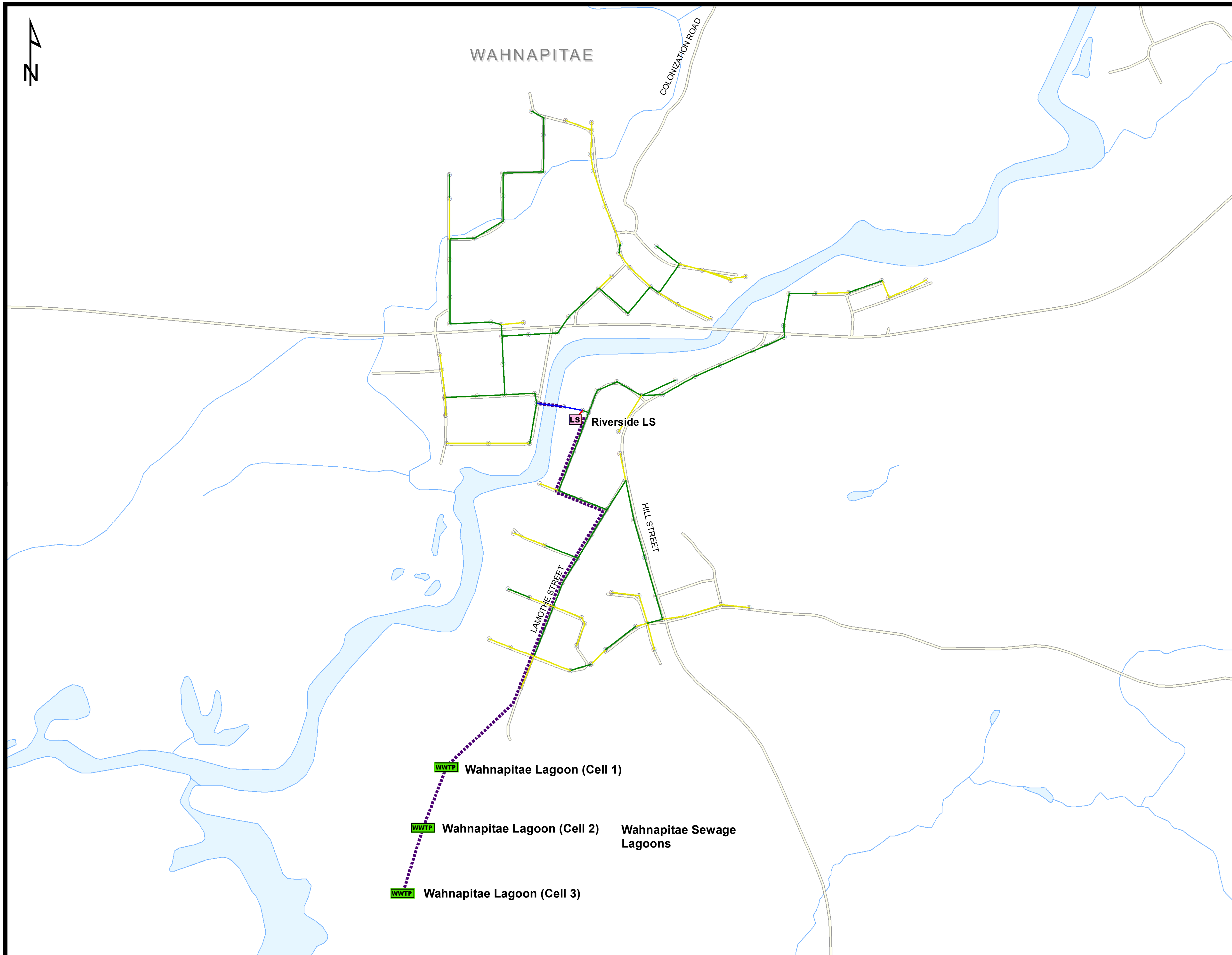
Sudbury Water and Wastewater Master Plan

Map 3.5.1

Wahnapitae Wastewater System - Conduit Flow/Capacity (Design)

Scenario: ULTWWF_2yrStorm



Legend

- LS Lift Station
- WWTP Sanitary Treatment Plant
- Manhole
- Waterbody
- Road
- Watercourse
- Forcemain

Conduit: Velocity (m/s)

- <= 0.6
- <= 1.5
- <= 3.0
- > 3.0

Greater Grand Sudbury

WSP

Project No. 121-23026-00

December 09 2016

Sudbury Water and Wastewater Master Plan

0 200 400 600 m

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Map 3.5.2

Wahnapitae Wastewater System - Conduit Velocity

Scenario: ULTWWF_2yrStorm

APPENDIX

C WASTEWATER GENERATION AND CAPACITY ASSESSMENTS

Wahnapitae - Wastewater Flow Forecasts

	Units	2009	2010	2011	2012	2013	Summary	Design Criterion	Comments
Average Day Flow	(m^3/d)	1,321	884	565	556	689	832	832	From Annual Reports
Max Day Flow	(m^3/d)	Not Available						3,051	This was estimated by multiplying the average day demand by the estimated maximum day factor.
Max Day Factor								3.67	This was estimated using the factor for nearby Coniston
Peak Flow	(m^3/d)							0	Peak hour flows were not available
Peak Flow	(L/s)							0	
Peak Flow Factor									

	2009	2010	2011	2012	2013	Summary	Design Criterion	2016	2021	2026	2031	2036	2041	Ultimate Buildout	Comments
Population (Existing Areas)	1,397	1,397	1,397	1,397	1,397	1,397	1,397	1,397	1,397	1,397	1,397	1,397	1,397	1,397	
Population (Growth Areas)								5	11	16	19	21	21	82	
Total Population								1402	1408	1413	1416	1418	1418	1479	Total Population (Hemson)
Residential (ha)								0.41	0.93	1.38	1.76	1.94	1.94	2.09	ICI development areas were assigned to planning years based on the stage of the application. Draft Approved were assigned to 2016, Legal Lots of Record to 2026, and Designated Developable to 2036.
Institutional (ha)															
Commercial (ha)															
Industrial (ha)															Areas are cumulative and carry from the development year, all the way through to Ultimate Buildout
ICI (ha)								0.00	0.00	0.00	0.00	0.00	0.00	0.00	
Total (ha)								0.41	0.93	1.38	1.76	1.94	1.94	2.09	
Ratio of Residential to Total Customers	0.82	0.82	0.82	0.82	0.82	0.82									This ratio is based on Water Billing Records for the area and is an approximation of the residential portion of demand.
Residential Share of Average Day Demand (m^3/d)	1082	724	463	455	564	681									
Residential Flow Unit Rate (m^3/cap/d)	0.775	0.518	0.331	0.326	0.404	0.471	0.500								
Average Institutional Flow Unit Rate (m^3/ha/d)							28.0								CGS design standards do not suggest average rates for institutional and commercial, but require that rates be developed based on usage. In this case, water demands rates were used and assume a conservative 1:1 ratio of water to sewage.
Average Commercial Flow Unit Rate (m^3/ha/d)							28.0								CGS design standards do not suggest average rates for institutional and commercial, but require that rates be developed based on usage. In this case, water demands rates were used and assume a conservative 1:1 ratio of water to sewage.
Average Industrial Flow Unit Rate (m^3/ha/d)							35.0								CGS design standards average rate for light industrial of 35 m^3/ha/d.
Average Extraneous Flow Unit Rate (m^3/ha/d)							11.23								The CGS design standards do not have an extraneous flow value for this community. The value for nearby Coniston was used instead. From CGS Design Standards, peak rate for new developments divided by an assumed peaking factor of 3. This factor would be applied only to new developments, which are assumed to be leak-tight, and have minimal extraneous flow.

	2016	2021	2026	2031	2036	2041	Ultimate Buildout	Comments
Average Residential Flows (m^3/d) - Existing	832	832	832	832	832	832	832	This includes all contribution from existing ICI and infiltration. The base flow was assumed to be the average day flow to the plant for the 2011-2013 period.
Average Residential Flows (m^3/d)	3	5	8	10	10	11	41	Obtained by multiplying the projected population growth by the unit rate.
Average Institutional Flow (m^3/d)	0	0	0	0	0	0	0	Institutional growth area multiplied by unit flow rate.
Average Commercial Flow (m^3/d)	0	0	0	0	0	0	0	Commercial growth area multiplied by unit flow rate.
Average Industrial Flow (m^3/d)	0	0	0	0	0	0	0	Industrial growth area multiplied by unit flow rate.
Average Extraneous Flow (m^3/day)	5	10	16	20	22	22	23	
Average Day Flow (m^3/d)	839	847	855	861	864	864	896	
Max Day Flow (m^3/d)	3,078	3,110	3,138	3,159	3,170	3,170	3,288	

ALTERNATIVE CALCULATION METHOD

	2016	2021	2026	2031	2036	2041	Ultimate Buildout	Comments
Per Capita Flow (m3/cap/day)	0.946	0.633	0.405	0.398	0.493	0.575	0.575	
Average Day Flow (m^3/d)	806	809	812	814	815	815	850	Multiplying the total population by the consolidated per capita flow factor.
Max Day Flow (m^3/d)	2,958	2,970	2,981	2,987	2,991	2,991	3,119	

SENSITIVITY ANALYSIS

Analyze sensitivity of forecasted flows to unit rate

	Unit Rate (m^3/cap/d)	Average Day Flow (m^3/d) 2016	2021	2026	2031	2036	2041	Ultimate Buildout 2061
Base Scenario - MOE Guidelines Lower Limit of Typical Values	0.225	839	847	855	861	864	864	896
Combined Historical Maximum	0.575	839	848	856	862	865	865	902
Residential Hsitorical Maximum	0.471	839	847	855	860	863	863	893
City Standards	Not Provided							

Analyze sensitivity of forecasted flows to max day factor

	Max Day Peaking Factor	Max Day Flow (m^3/d) 2016	2021	2026	2031	2036	2041	Ultimate Buildout 2061
Base Scenario - Historical Max	Data Not Available							
Historical Average	Data Not Available							

CAPACITY CHECK

	2011	2016	2021	2026	2031	2036	2041	Ultimate Buildout 2061	Comments
Rated Average Day Flow Capacity (m^3/d)	1,246	1,246	1,246	1,246	1,246	1,246	1,246	1,246	From C of A (2011)
Average Day Flow (m^3/d)	832	839	847	855	861	864	864	896	
Maximum Day Flow (m^3/d)	3,051	3,078	3,110	3,138	3,159	3,170	3,170	3,288