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APPENDIX

Appendix 3-A WASTEWATER SYSTEM GAP ANALYSIS & STATUS QUO REPORTS

3 VOLUME 3: EXISTING WASTEWATER SYSTEMS

The CGS owns and operates thirteen (13) independent wastewater collection systems that service the various communities in the City. The names of the systems are listed below:

1. Azilda Wastewater System – 5 Lift Stations
2. Capreol Wastewater System – 2 Lift Stations
3. Chelmsford Wastewater System – 8 Lift Stations
4. Coniston Wastewater System – 2 Lift Stations
5. Copper Cliff Wastewater System – 2 Lift Stations
6. Dowling Wastewater System – 1 Lift Station
7. Falconbridge Wastewater System – 0 Lift Stations
8. Garson Wastewater System – 3 Lift Stations
9. Onaping-Levack Wastewater System – 1 Lift Station
10. Lively/Walden Wastewater System – 7 Lift Stations
11. Sudbury Wastewater System – 27 Lift Stations
12. Valley East Wastewater System – 9 Lift Stations
13. Wahnapitae Wastewater System – 1 Lift Station

Each system, with the exception of the Falconbridge which does not contain any lift stations, includes a wastewater treatment plant or lagoon and, at least one (1) lift station. This results in a total of ten (10) wastewater treatment plants, four (4) lagoons and sixty-eight (68) lift stations within the CGS.

The Wastewater Systems Baseline Review Report (WSP, 2014) documents the compiled information on the City's existing wastewater infrastructure. This document establishes the baseline or starting point in the assessment of the wastewater systems to service the existing and projected development. The report includes an overview of the regulatory requirements relevant to the planning and design of wastewater systems. The report also includes a description of the various independent wastewater treatment systems.

Additionally, a capacity review of each wastewater system was conducted as a gap analysis in order to determine future system requirements. The following sections, Volume 3, of this report will summarize the information in the Wastewater Systems Baseline Review Report (WSP, 2014) and the Wastewater System Gap Analysis and Status Quo Reports (WSP, 2015-2016) for each of the individual systems.

3.1 AZILDA WASTEWATER SYSTEM

The area known as the Valley, comprised of the communities of Valley East, Capreol, Azilda and Chelmsford, is located in the north end of the CGS and is the second most populated area, following the community of Sudbury. The communities of Valley East, Capreol, Azilda and Chelmsford are serviced by four independent wastewater systems. Figure 3-1 illustrates the existing wastewater infrastructure in the Azilda Wastewater System.

Figure 3-1 Azilda Wastewater System: Existing Infrastructure

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3.1.1 EXISTING SYSTEM

Wastewater collected in the Azilda Wasetwater System is treated at the Azilda WWTP. Raw wastewater entering the Azilda WWTP is primarily of domestic origin. The plant is a circular extended aeration plant, and has a rated capacity of 3,300 m^3/d, and a maximum day capacity of 6,680 m^3/d.

The Azilda Wastewater System also includes five (5) lift stations within its catchment area. Information regarding their capacity and other details can be seen in Table 3-1.

Table 3-1 Azilda Wastewater System Lift Station Summary

	CURRENT FIRM CAPACITY (L/S)	EXISITNG PEAK FLOW (L/S)
Landry	41.30	106.10
Laurier	90.1	296.10
Maple	17.8	2.01
Marier	10.8	14.7
Principale	32.9	12.1

3.1.2 EXISTING AND FUTURE REQUIREMENTS

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

Table 3-2 Azilda Wastewater System Flow Criteria

CRITERIA	VALUE	REFERENCE
Average Day Residential Flow	400 L/cap/d	Average of historical values, rounded up to nearest 50 L/cap/d
Average Day Commercial and Institutional Generation	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	7.5 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

Table 3-3 Azilda Wastewater 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

3.1.3 GAP ANALYSIS CONCLUSIONS

As previously mentioned, a gap analysis was conducted in order to determine existing and future wastewater system deficiencies for each system. The following information is a summary of the Azilda Wastewater Gap Analysis and Status Quo Report (WSP, 2015), contained in Appendix 3-A. The report can be referenced for more details regarding the analysis of the Azilda Wastewater System.

An important note regarding the Azilda Wastewater System analysis is that R.V. Anderson Associates Limited are currently completing an Azilda Wastewater Treatment Plant and Collection System Class Environmental Assessment for the CGS. Draft versions of this report have been reviewed regularly and recommendations made, regarding solutions for wet weather storage issues, have been included in our analysis for consistency. Cost information from the report has been encompassed in our recommendations, outlined in Volume 7 of this report.

BYPASSES AND WET WEATHER FLOW

Historical bypass events were reviewed as part of the gap analysis for each wastewater system in the CGS. It was discovered that from 2011 to 2014, there have been six bypasses at the Azilda WWTP, and one each at Landry, Laurier, and Main Lift Stations. Additionally, from January 2014 to November of 2016, twelve bypasses occurred at the Azilda WWTP. As mentioned, wet weather storage issues noted at the plant are being addressed through the Azilda Wastewater Treatment Plant and Collection System Class Environmental Assessment (R.V. Anderson Associated Ltd., 2017)

It was also noted during system review that City staff have noticed, 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. Additionally, the Azilda Wastewater Treatment Plant and Collection System Class Environmental Assessment (R.V. Anderson Associated Ltd., 2017) identifies I&I issues in the system. Further discussion regarding I&I issues will follow in Volume 5 and in Volume 7.

TREATMENT

Analysis of the Azilda WWTP concluded that there would be sufficient capacity to service the population growth to the year 2041. This can be seen in Figure 3-2 where the projected flows are plotted against the capacity of the plant.

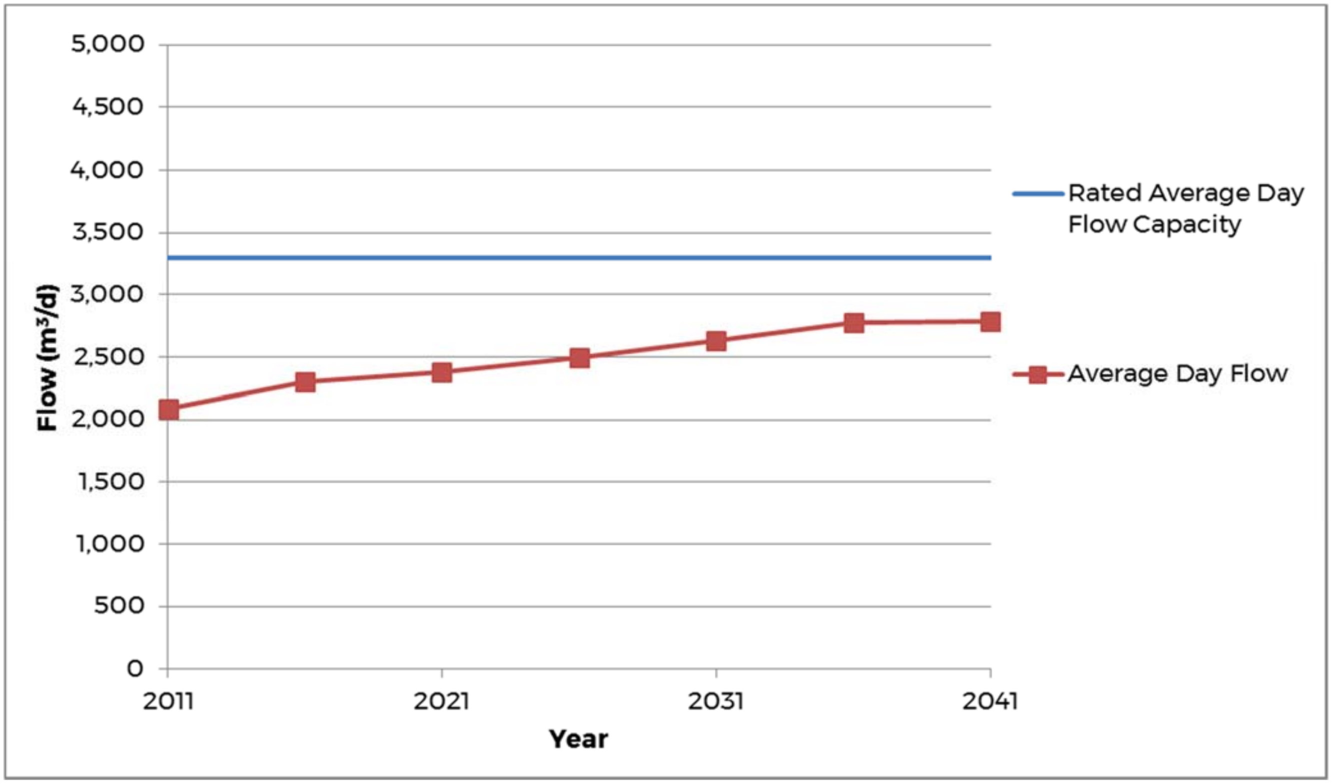

Figure 3-2 Azilda Wastewater Flow Projections vs. Rated Capacity

LIFT STATIONS

Analysis of the Azilda lift stations concluded that the Landry, Laurier and Marier lift stations all need to be expanded to meet the existing peak flow requirements.

SEWERS

During assessment of the sewers within the Azilda Wastewater System, hydraulic computer modeling identified that the majority of the sewers flow at less than 50% of the available capacity through to 2041 under the wet weather flow condition. It was also determined that 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 2041 under the wet weather flow condition. Refer to Azilda Wastewater System Gap Analysis and Status Quo Report (WSP, 2015) in Appendix 3-A, which outlines areas identified to have sewer capacity deficiencies within the Azilda Wastewater System.

3.2 CAPREOL WASTEWATER SYSTEM

Capreol is a community located along Old Highway 69, North of Valley East. Figure 3-3 illustrates the existing wastewater infrastructure in the Capreol Wastewater System.

Figure 3-3 Capreol Wastewater System: Existing Infrastructure

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3.2.1 EXISTING SYSTEM

Wastewater in the Capreol Wastewater System is treated at the Capreol Lagoon which is owned and operated by the CGS. The lagoon is a two-cell waste stabilization lagoon, operated as a continuous discharge exfiltration system, discharging to the Vermilion River (MOECC, Certificate of Approval Number 8214-4UVPUZ, 2001). The rated capacity of the Lagoon is 5,000 m^3/d.

The Capreol Wastewater System also comprises two lift stations. Information can be seen in Table 3-4.

Table 3-4 Capreol Wastewater System Lift Station Summary

STATION	CURRENT FIRM CAPACITY (L/S)	EXISITING PEAK FLOW (L/S)
Lloyd LS	11.42	6.23
Vermilion LS	100	75.8

3.2.2 EXISTING AND FUTURE REQUIREMENTS

Using the population projections and wastewater rate development process described in Section 1.4 of Volume 1 of this report, Capreol's future wastewater projections were calculated. Table 3-5 summarizes the Capreol wastewater flow criteria and the reference used to determine the criteria, and Table 3-6 summarizes the calculated projections.

Table 3-5 Capreol Wastewater System Flow Criteria

CRITERIA	VALUE	REFERENCE
Average Day Residential Flow	650 L/cap/d	Average of historical values, rounded up to nearest 50 L/cap/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
Max Day Peaking Factor	2.69	Historical data was not available; matched with nearby Valley East

Table 3-6 Capreol Wastewater 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

3.2.3 GAP ANALYSIS CONCLUSIONS

As previously mentioned, a gap analysis was conducted in order to determine existing and future wastewater system deficiencies for each system. The following information is a summary of the Capreol Wastewater Gap Analysis and Status Quo Report (WSP, 2015) contained in Appendix 3-A. The report can be referenced for more details regarding the analysis of the Capreol Wastewater System.

BYPASSES AND WET WEATHER FLOW

There was no data available to determine if there have been any overflow events at the Capreol Lagoons. Discussions with the City have indicated that there are currently no issues with regards to storage concerns at the wells for existing wastewater flows collected in the system.

TREATMENT

Analysis of the Capreol Lagoon concluded that there would be sufficient capacity to service the population growth to the year 2041. This can be seen in Figure 3-4 where the projected flows are plotted against the capacity of the Lagoon.

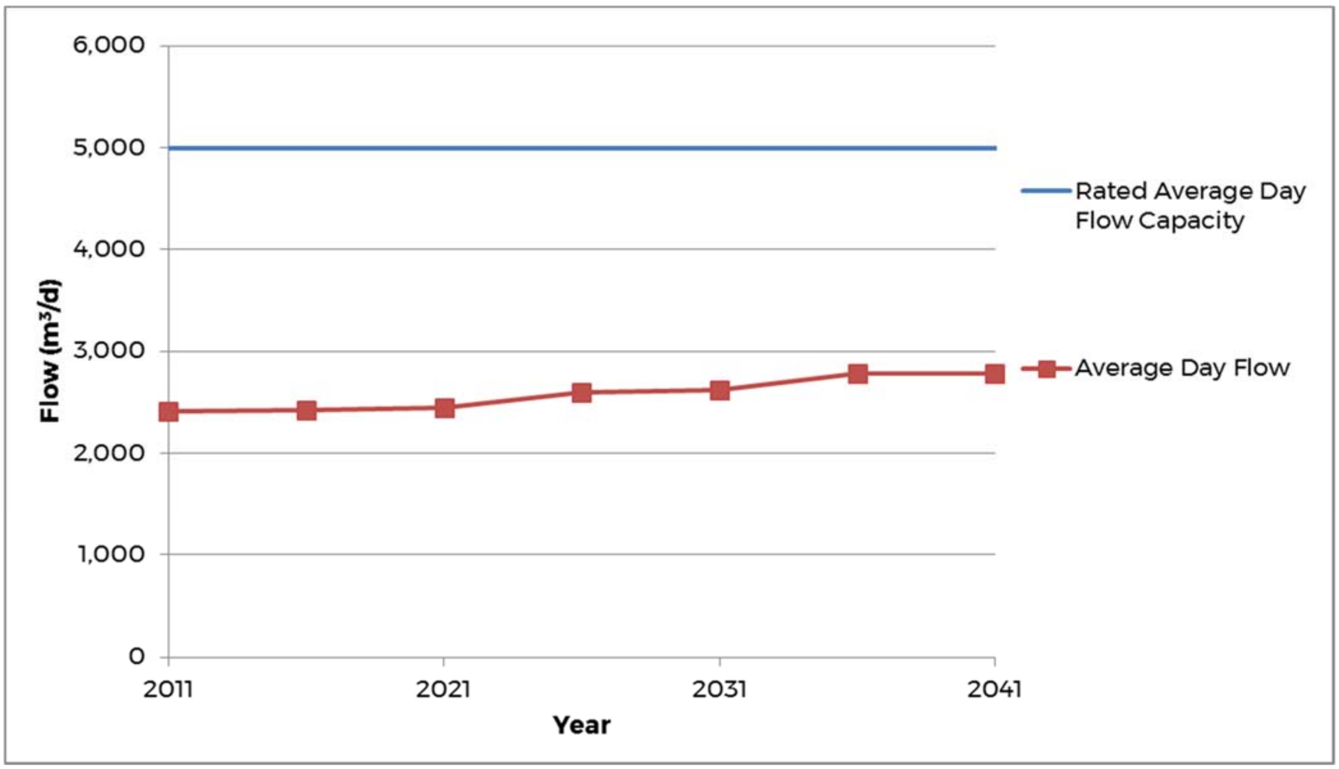

Figure 3-4 Capreol Wastewater Flow Projections vs. Rated Capacity

LIFT STATIONS

Analysis of the Capreol lift stations concluded that existing capacities are sufficient to service the population growth to the year 2041.

SEWERS

During assessment of the sewers within the Capreol Wastewater System, hydraulic computer modeling identified that that the majority of the sewers in the Capreol system flow at less than 50% of the available capacity through to 2041 under wet weather flow conditions. Flow velocities through most of the Capreol sewers are also generally below the City's standard of 0.6 m/s. It was concluded that this is consistent through to 2041 under the wet weather flow condition. Refer to the Capreol Wastewater System Gap Analysis and Status Quo Report (WSP, 2015) in Appendix 3-A which outlines areas identified to have sewer capacity deficiencies within the Capreol Wastewater System.

3.3 CHELMSFORD WASTEWATER SYSTEM

Chelmsford is located along Old Highway 69, north of Valley East. Figure 3-5 illustrates the existing wastewater infrastructure in the Chelmsford Wastewater System.

3.3.1 EXISTING SYSTEM

Wastewater treatment in the Chelmsford Wastewater System occurs at the Chelmsford WWTP and the Chelmsford Lagoon. All wastewater generated in the system is ultimately treated at the WWTP; however, the lagoon is occasionally used for storage in cases of wet weather events (i.e. heavy rain and/or snow melt). The WWTP is owned and operated by the CGS and it consists of three (3) aeration plants with common preliminary treatment and disinfection. The average day rated capacity is 7,100 m^3/d, maximum day capacity is 18,200 m^3/d, and peak instantaneous capacity is 24,000 m^3/d (MOECC, 2009). The Chelmsford Lagoon has a capacity of approximately 222,170 m^3.

In addition to the treatment facilities in the system, Chelmsford comprises eight (8) lift stations, for which information can be found in Table 3-7.

Table 3-7 Chelmsford Wastewater System Lift Station Summary

STATION	CURRENT FIRM CAPACITY (L/S)	EXISTING PEAK FLOW
Belanger	6.25	8.8
Brookside	13.5	6.1
Charette	14	2.3
Hazel	51.7	16.5
Keith	45.2	4.2
Main	40.1	32.8
Radisson	6.5	1.1
Whitson	22.5	4.3

Figure 3-5 Chelmsford Wastewater System: Existing Infrastructure

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3.3.2 EXISTING AND FUTURE REQUIREMENTS

Using the population projections and wastewater rate development process described in Section 1.4 of Volume 1 of this report, Chelmsford's future wastewater projections were calculated. Table 3-8 summarizes the Chelmsford wastewater flow criteria and the reference used to determine the criteria, and Table 3-9 summarizes the calculated projections.

Table 3-8 Chelmsford Wastewater System Flow Criteria

CRITERIA	VALUE	REFERENCE
Average Day Residential Flow	450 L/cap/d	Average of historical values, rounded up to nearest 50 L/cap/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
Max Day Peaking Factor	3.95	Average of historical values

Table 3-9 Chelmsford Wastewater 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

3.3.3 GAP CONCLUSIONS

As previously mentioned, a gap analysis was conducted in order to determine existing and future wastewater system deficiencies for each system. The following information is a summary of the Chelmsford Wastewater Gap Analysis and Status Quo Report (WSP, 2015), contained in Appendix 3-A. The report can be referenced for more details regarding the analysis of the Chelmsford Wastewater System.

BYPASSES AND WET WEATHER FLOW

Historical bypass events were reviewed as part of the gap analysis for each wastewater system in the CGS. It was discovered that from 2011 to 2014, there were two bypasses at the Chelmsford WWTP, one at the Chelmsford Lagoon, and one each at the Belanger and Main Lift Stations. From January 2014 to November of 2016, twelve bypass events were reported at the WWTP.

I&I has also been recognized as an issue within the Chelmsford Wastewater System. Further discussion will follow in Volume 5 and in Volume 7.

TREATMENT

Analysis of the Chelmsford WWTP concluded that there would be sufficient capacity to service the population growth to the year 2031, but that an additional 356 m^3/d would be required to accommodate flows to 2041. This can be seen in Figure 3-6 where the projected flows are plotted against the capacity of the treatment plant.

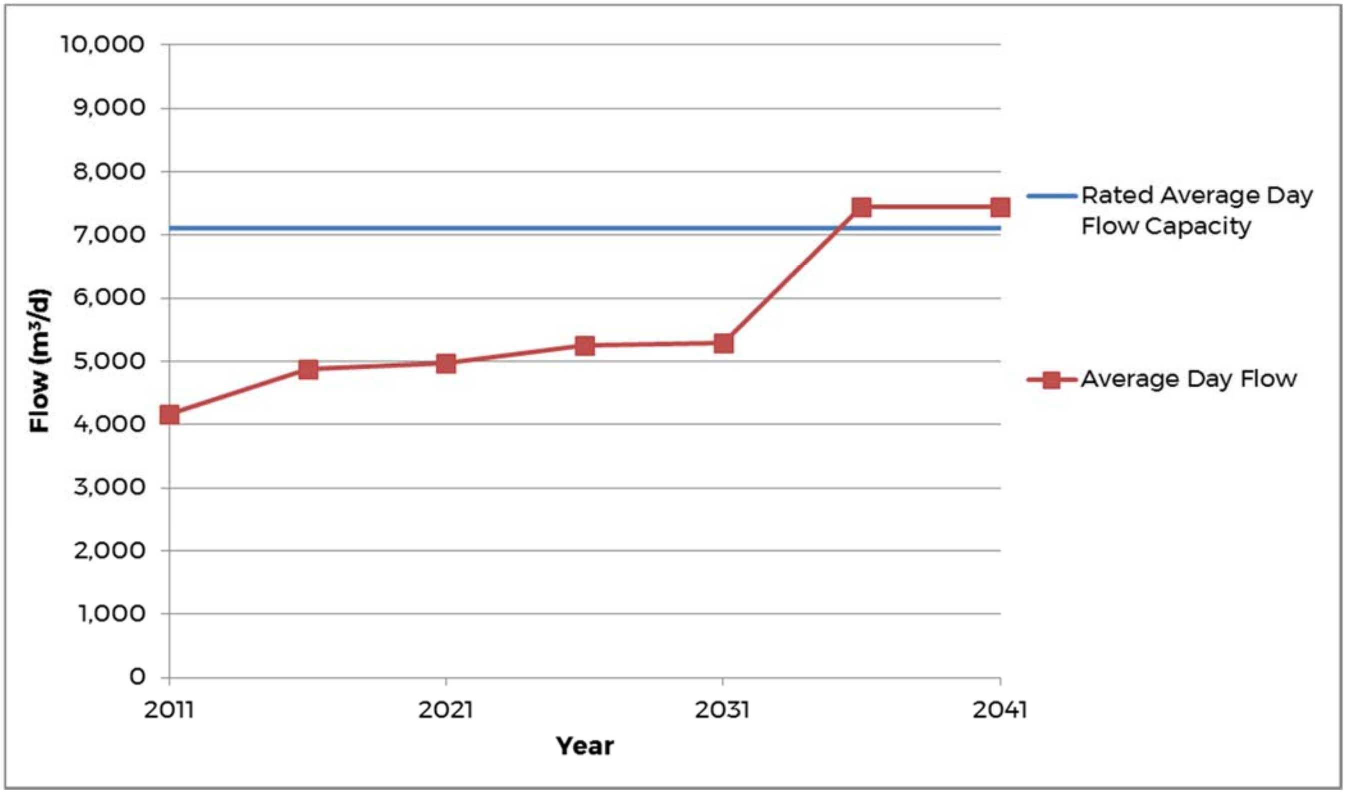

Figure 3-6 Chelmsford Wastewater Flow Projections vs. Rated Capacity

LIFT STATIONS

Analysis of the Chelmsford lift stations concluded that the Belanger lift station needs to be expanded immediately to meet the existing peak flow requirements. Radisson lift station also requires expansion to service the population growth to the year 2041.

SEWERS

During assessment of the sewers within the Chelmsford Wastewater System, hydraulic computer modeling identified that the majority of the sewers in the Chelmsford system flow at less than 50% of the available capacity through to 2041 under wet weather flow conditions, with the exception of the sewer upstream of Charette LS, and the sewer extending west from Pilon and Edna to the Chelmsford WWTP. Flow velocities through most of the Chelmsford sewers are also generally below the City's standard of 0.6 m/s. It was concluded that this is consistent through to 2041 under the wet weather flow condition. Refer to the Chelmsford Wastewater System Gap Analysis and Status Quo Report (WSP, 2015) in Appendix 3-A, which outlines areas identified to have sewer capacity deficiencies within the Chelmsford Wastewater System.

3.4 CONISTON WASTEWATER SYSTEM

Coniston is a community located in the southeast end of the CGS, just east of Sudbury proper. Figure 3-7 illustrates the existing wastewater infrastructure in the Coniston Wastewater System.

Figure 3-7 Coniston Wastewater System: Existing Infrastructure

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3.4.1 EXISTING SYSTEM

All wastewater generated in Coniston is collected and treated at the Coniston WWTP. The plant is an oxidation ditch system with a rated capacity of 3,000 m^3/d.

Additionally, there are two (2) lift stations in the Coniston Wastewater System that discharge to the Coniston WWTP. Information is summarized in Table 3-10.

Table 3-10 Coniston Wastewater System Lift Station Summary

STATION	CURRENT FIRM CAPACITY (L/S)	EXISTING PEAK FLOW (L/S)
Edward LS	89.4	106.9
Government Road LS	18.1	125.5

3.4.2 EXISTING AND FUTURE REQUIREMENTS

Using the population projections and wastewater rate development process described in Section 1.4 of Volume 1 of this report, Coniston's future wastewater projections were calculated. Table 3-11 summarizes the Coniston wastewater flow criteria and the reference used to determine the criteria, and Table 3-12 summarizes the calculated projections.

Table 3-11 Coniston Wastewater System Flow Criteria

CRITERIA	VALUE	REFERENCE
Average Day Residential Flow	400 L/cap/d	Average of historical values, rounded up to nearest 50 L/cap/d
Average Day Institutional & Commercial Flow	28 m^3/ha/d	Water unit rates, assuming a 1:1 correspondence
Average Industrial Flow	35 m^3/ha/d	Water unit rates, assuming a 1:1 correspondence
Max Day Peaking Factor	3.67	Average of historical values

Table 3-12 Coniston Wastewater 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

3.4.3 GAP ANALYSIS CONCLUSIONS

As previously mentioned, a gap analysis was conducted in order to determine existing and future wastewater system deficiencies for each system. The following information is a summary of the Coniston Wastewater Gap Analysis and Status Quo Report (WSP, 2015), contained in Appendix 3-A. The report can be referenced for more details regarding the analysis of the Coniston Wastewater System.

BYPASSES AND WET WEATHER FLOW

Historical bypass events were reviewed as part of the gap analysis for each wastewater system in the CGS. Fifteen plant bypasses were reported at the Coniston WWTP between 2004 and 2012. An additional nineteen events were recorded between 2014 and November of 2016 at the plant.

Most of the bypass events occurred at the Coniston WWTP as a result of heavy precipitation and snow melt. Further discussion regarding I&I issues will follow in Volume 5 and in Volume 7.

TREATMENT

Analysis of the Coniston WWTP concluded that there would be sufficient capacity to service the population growth to the year 2041. This can be seen in Table 3-8 where the projected flows are plotted against the capacity of the plant.

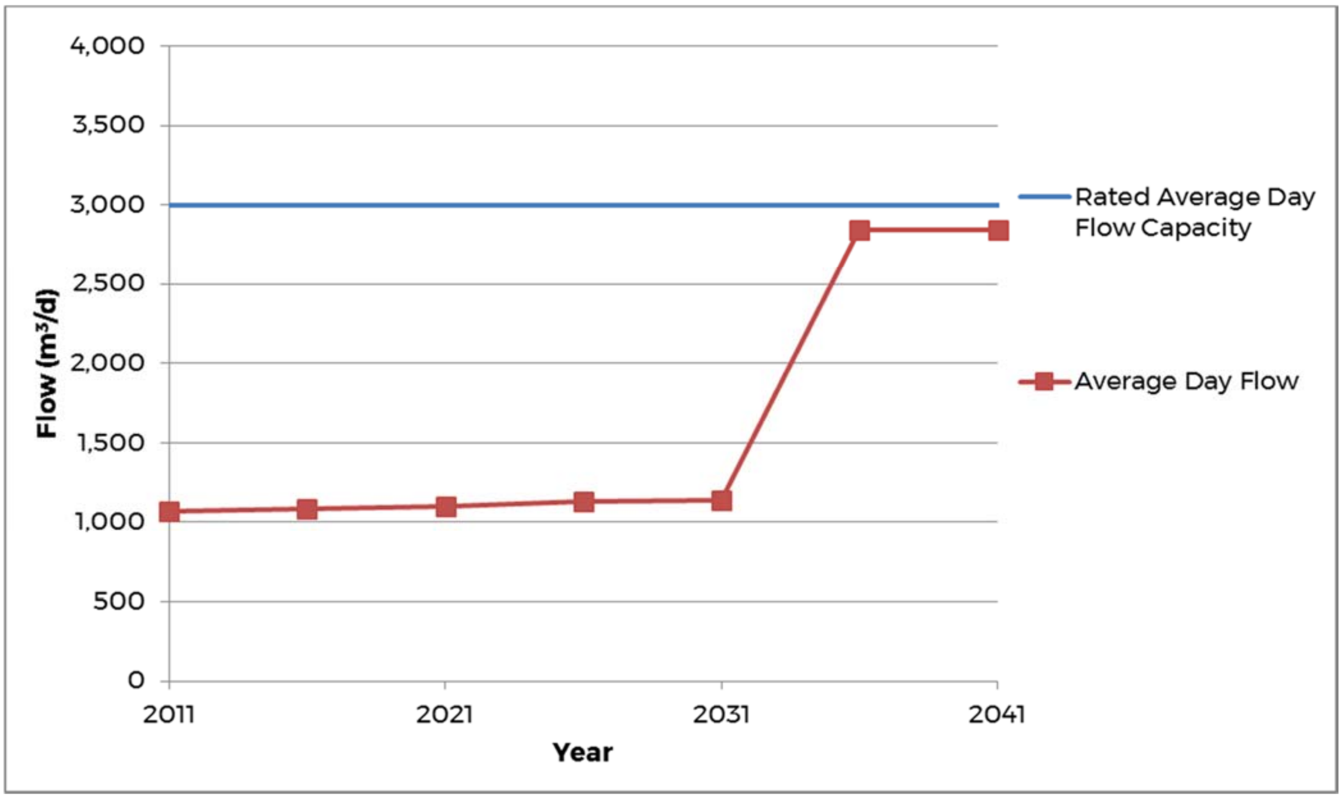

Figure 3-8 Coniston Wastewater Flow Projections vs. Rated Capacity

LIFT STATIONS

Analysis of the Coniston lift stations concluded that the both Edward and Government Road lift stations need to be expanded immediately to meet the existing peak flow requirements.

SEWERS

During assessment of the sewers within the Coniston Wastewater System, hydraulic computer modeling identified that that many of the sewers in the Coniston system operate at less than 50% capacity up to the 2041 growth scenario, but that a number of the sewers reach 100% capacity and above. Additionally, the flow velocity in some of the sewers is less than 0.6 m/s up to the 2041 scenario. The Coniston Wastewater System Gap Analysis and Status Quo Report (WSP, 2015), provided in Appendix 3-A, outlines areas identified to have sewer capacity deficiencies within the Coniston Wastewater System.

3.5 COPPER CLIFF WASTEWATER SYSTEM

Copper Cliff is a large community located in the center of the CGS, north-east of the communities of Lively and Walden. Figure 3-9 illustrates the existing wastewater infrastructure in the Copper Cliff Wastewater System.

3.5.1 EXISTING SYSTEM

The Copper Cliff Wastewater System is partially owned and operated by the CGS, but a third party (Vale) owns and operates the system's treatment facility; the Copper Cliff WWTP, which has a rated capacity of 231,360 m^3/d. For informative purposes, Vale also owns a second WWTP which does not treat municipal wastewater. An important note regarding the system is that the CGS is currently working towards the implementation of infrastructure to redirect all Copper Cliff wastewater flows from the most downstream LS (Nickel LS, described further below) to the Sudbury WWTP. This was taken into account when assessing the system during the Master Plan.

The Copper Cliff Wastewater System also comprises two (2) lift stations, for which information can be found in Table 3-13.

Table 3-13 Copper Cliff Wastewater System Lift Station Summary

STATION	CURRENT FIRM CAPACITY (L/S)	EXISTING PEAK FLOW (L/S)
Nickel LS	Currently under expansion	Currently under expansion
Orford LS	18.9	12.4

3.5.2 EXISTING AND FUTURE REQUIREMENTS

Using the population projections and wastewater rate development process described in Section 1.4 of Volume 1 of this report, Copper Cliff's future wastewater projections were calculated. Table 3-14 summarizes the Copper Cliff wastewater flow criteria and the reference used to determine the criteria, and Table 3-15 summarizes the calculated projections.

Table 3-14 Copper Cliff Wastewater System Flow Criteria

CRITERIA	VALUE	REFERENCE
Average Day Residential Flow	500 L/cap/d	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 Industrial Flow	35 m^3/ha/d	Water unit rates, assuming a 1:1 correspondence
Max Day Peaking Factor	4.05	Estimated by using the same factor as Lively

Figure 3-9 Copper Cliff Wastewater System: Existing Infrastructure

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Table 3-15 Copper Cliff Wastewater 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

3.5.3 GAP ANALYSIS CONCLUSIONS

As previously mentioned, a gap analysis was conducted in order to determine existing and future wastewater system deficiencies for each system. The following information is a summary of the Copper Cliff Wastewater Gap Analysis and Status Quo Report (WSP, 2016) contained in Appendix 3-A. The report can be referenced for more details regarding the analysis of the Copper Cliff Wastewater System.

BYPASSES AND WET WEATHER FLOW

Historical bypass events were reviewed as part of the gap analysis for each wastewater system in the CGS. No reported bypass events were identified for the Copper Cliff Wastewater System; however, I&I is a major concern in the Copper Cliff Wastewater System. Specifically, the City has noted high levels of I&I in the system near the private development located at Power Street and Highway 55. The parking lot in this development is noted to be often flooded. Smoke testing has been undertaken in this area.

There are several contributors to the high levels of I&I in the system. Firstly, 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 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. Further discussion regarding I&I issues will follow in Volume 5 and in Volume 7.

TREATMENT

As mentioned, the City is in the process of implementing redirection infrastructure, and it has therefore been determined that there will be sufficient capacity to service future flows generated in the Copper Cliff Wastewater System at the Sudbury WWTP. This has been based on the next planned expansion to the Sudbury plant per the rated capacity planned for in the Wastewater Treatment Options for the City of Sudbury and the Settlement of Garson in the Town of Nickel Centre Environmental Study Report Addendum (Dennis Consultants, 2009).

LIFT STATIONS

Analysis of the Copper Cliff lift stations concluded that the Orford LS has sufficient capacity to 2041. 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.

SEWERS

During assessment of the sewers within the Copper Cliff Wastewater System, hydraulic computer modeling identified that the majority of sewers in the system flow at less than 50% of the available capacity from 2011 to 2041 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 condition. Refer to the Copper Cliff Wastewater System Gap Analysis and Status Quo Report (WSP, 2015) in Appendix 3-A, which outlines areas identified to have sewer capacity deficiencies within the Coniston Wastewater System.

3.6 DOWLING WASTEWATER SYSTEM

The community of Dowling is located in the northwest end of Greater Sudbury along Route 144, between the communities of Onaping and Chelmsford. Figure 3-10 illustrates the existing wastewater infrastructure in the Dowling Wastewater System.

3.6.1 EXISTING SYSTEM

Wastewater from the Dowling Wastewater System is treated at the Dowling WWTP, which is owned and operated be the CGS. The plant is an extended aeration activated sludge facility with an average day rated capacity of 3,200 m^3/d and maximum day capacity of 6,400 m^3/d (MOECC, 1998).

The Dowling Wastewater System comprises only one lift station, for which information can be found in Table 3-16.

Table 3-16 Dowling Wastewater System Lift Station Summary

STATION	CURRENT FIRM CAPACITY (L/S)	EXISTING PEAK FLOW (L/S)
Lionel LS	18.61	9.3

3.6.2 EXISTING AND FUTURE REQUIREMENTS

Using the population projections and wastewater rate development process described in Section 1.4 of Volume 1 of this report, Dowling's future wastewater projections were calculated. Table 3-17 summarizes the Dowling wastewater flow criteria and the reference used to determine the criteria, and Table 3-18 summarizes the calculated projections.

Table 3-17 Dowling Wastewater System Flow Criteria

CRITERIA	VALUE	REFERENCE
Average Day Residential Flow	900 L/cap/d	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
Max Day Peaking Factor	1.79	Average of historical values

Figure 3-10 Dowling Wastewater System: Existing Infrastructure

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Table 3-18 Dowling Wastewater 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

3.6.3 GAP ANALYSIS CONCLUSIONS

As previously mentioned, a gap analysis was conducted in order to determine existing and future wastewater system deficiencies for each system. The following information is a summary of the Dowling Wastewater Gap Analysis and Status Quo Report (WSP, 2015), contained in Appendix 3-A. The report can be referenced for more details regarding the analysis of the Dowling Wastewater System.

BYPASSES AND WET WEATHER FLOW

Historical bypass events were reviewed as part of the gap analysis for each wastewater system in the CGS. There have been no reported bypass events between 2011 and November of 2016 in the Dowling Wastewater System.

TREATMENT

Analysis of the Dowling WWTP concluded that there would be sufficient capacity to service the population growth to the year 2041. This can be seen in Figure 3-11 where the projected flows are plotted against the capacity of the plant.

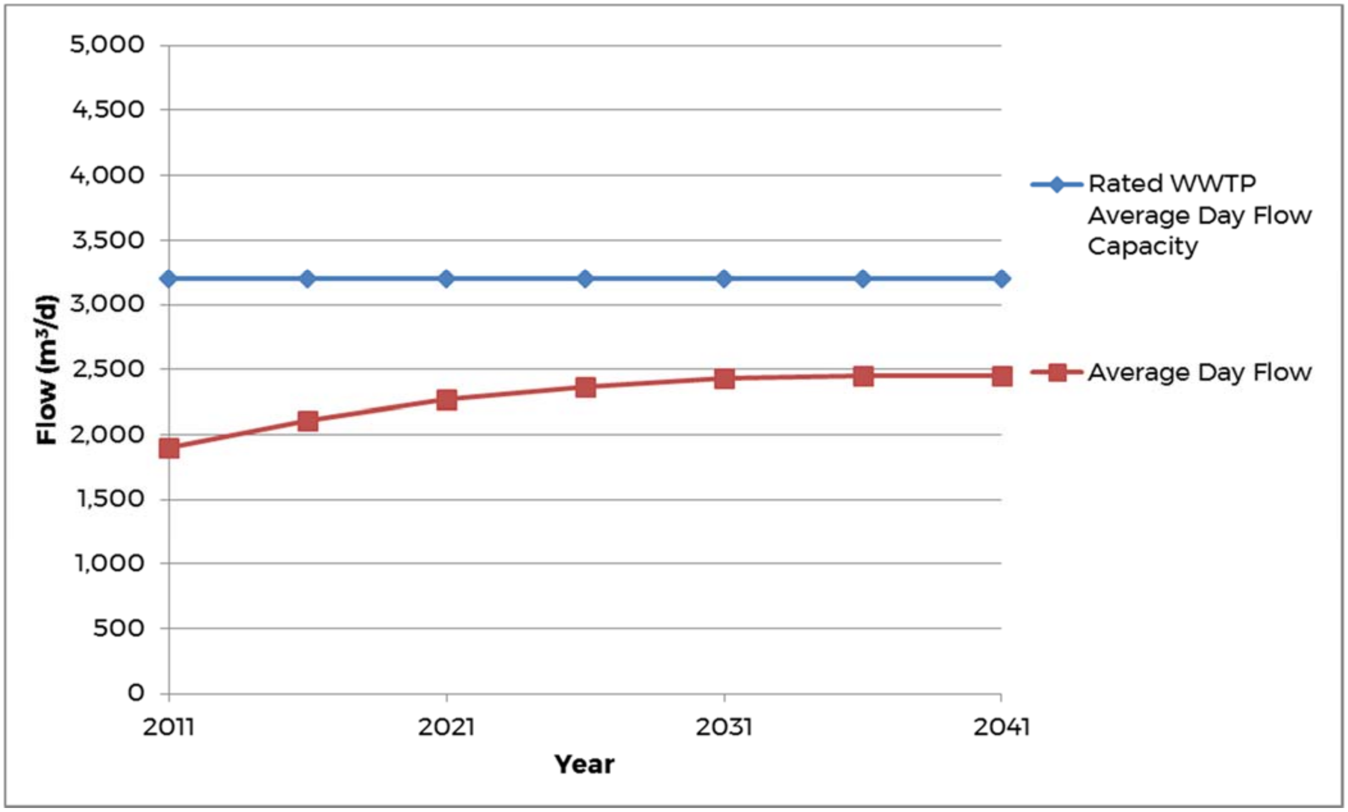

Figure 3-11 Dowling Wastewater Flow Projections vs. Rated Capacity

LIFT STATIONS

Analysis of Lionel lift station concluded that existing capacities are sufficient to service the population growth to the year 2041.

SEWERS

During assessment of the sewers within the Dowling Wastewater System, hydraulic computer modeling identified that no capacity issues were identified in the system, as most of the sewers flow at less than 50% capacity; however, it was noted that flow velocities through most of the Dowling sewers are generally below 0.6 m/s. This is consistent through to 2041 under the wet weather flow condition. Refer to the Dowling Wastewater System Gap Analysis and Status Quo Report (WSP, 2015), in Appendix 3-A, which outlines areas identified to have sewer capacity deficiencies within the Dowling Wastewater System.

3.7 FALCONBRIDGE WASTEWATER SYSTEM

Falconbridge is a small community located in the east end of the CGS. Figure 3-12 illustrates the existing wastewater infrastructure in the Faconbridge Wastewater System.

Figure 3-12 Falconbridge Wastewater System: Existing Infrastructure

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3.7.1 EXISTING SYSTEM

All wastewater generated in Falconbridge is collected and treated at the Falconbridge WWTP. The plant 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.

It should be noted that all wastewater within the Falconbridge Wastewater System flows by gravity sewers, as there are no lift stations in the system.

3.7.2 EXISTING AND FUTURE REQUIREMENTS

Using the population projections and wastewater rate development process described in Section 1.4 of Volume 1 of this report, Falconbridge's future wastewater projections were calculated. Table 3-19 summarizes the Falconbridge wastewater flow criteria and the reference used to determine the criteria, and Table 3-20 summarizes the calculated projections.

Table 3-19 Falconbridge Wastewater System Flow Criteria

CRITERIA	VALUE	REFERENCE
Average Day Residential Flow	400 L/cap/d	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
Max Day Peaking Factor	2.25	Average of historical values

Table 3-20 Falconbridge Wastewater 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

3.7.3 GAP ANALYSIS CONCLUSIONS

As previously mentioned, a gap analysis was conducted in order to determine existing and future wastewater system deficiencies for each system. The following information is a summary of the Falconbridge Wastewater Gap Analysis and Status Quo Report (WSP, 2015) contained in Appendix 3-A. The report can be referenced for more details regarding the analysis of the Falconbridge Wastewater System.

BYPASSES AND WET WEATHER FLOW

Historical bypass events were reviewed as part of the gap analysis for each wastewater system in the CGS. No bypass events were recorded in the Falconbridge Wastewater System between 2011 and 2014, but one event was reported at the plant in 2015, due to a blocked sewer.

TREATMENT

Analysis of the Falconbridge WWTP concluded that there would be sufficient capacity to service the population growth to the year 2041. This can be seen in Figure 3-13 where the projected flows are plotted against the capacity of the plant.

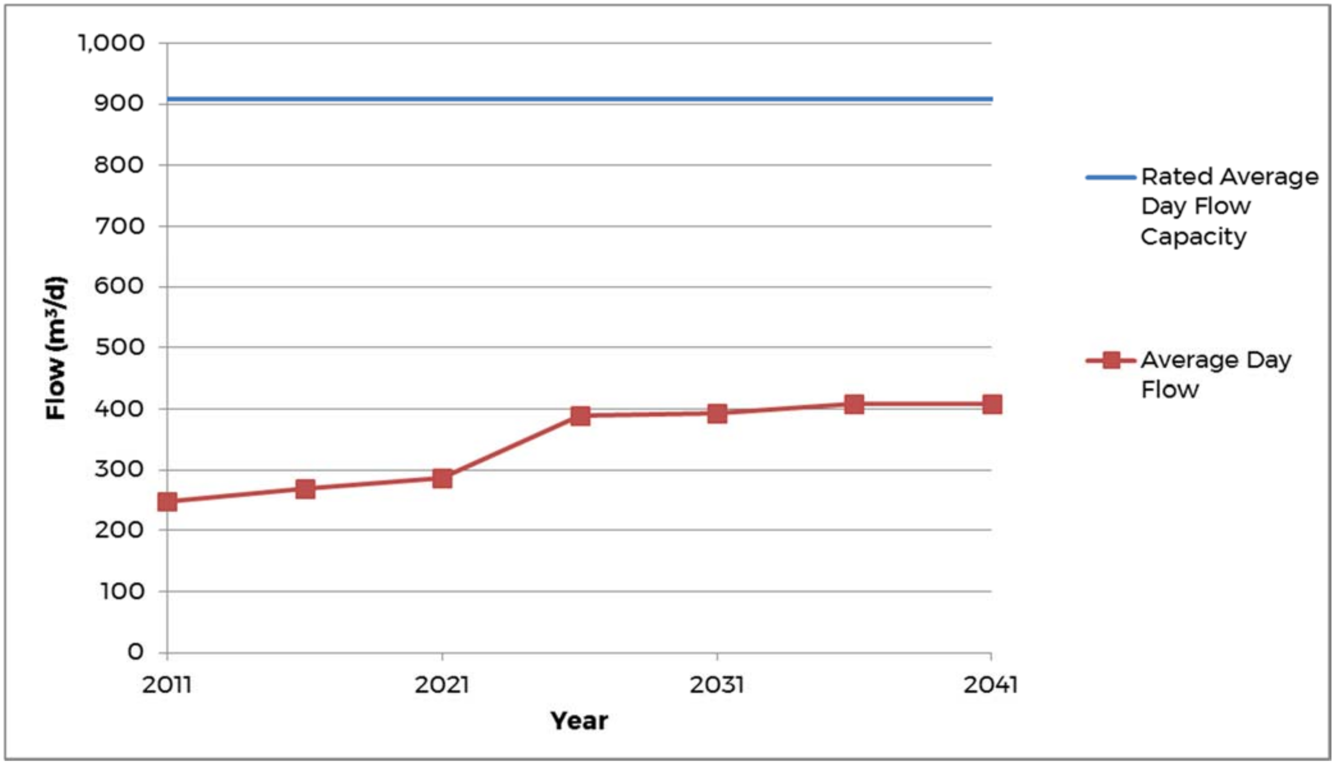

Figure 3-13 Falconbridge Wastewater Flow Projections vs. Rated Capacity

SEWERS

During assessment of the sewers within the Falconbridge Wastewater System, hydraulic computer modeling identified that no capacity issues were identified in the system, as most of the sewers flow at less than 50% capacity; however, it was noted that flow velocities through most of the Falconbridge sewers are generally below 0.6 m/s. This is consistent through to 2041 under the wet weather flow condition. Refer to the Falconbridge Wastewater System Gap Analysis and Status Quo Report (WSP, 2015), in Appendix 3-A, which outlines areas identified to have sewer capacity deficiencies within the Falconbridge Wastewater System.

3.8 GARSON WASTEWATER SYSTEM

Garson is a small community within Sudbury proper, however it has its own small wastewater system. Figure 3-14 illustrates the existing wastewater infrastructure in the Garson Wastewater System.

Figure 3-14 Garson Wastewater System: Existing Infrastructure

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3.8.1 EXISTING SYSTEM

The community of Garson is serviced by the Sudbury Wastewater System, which includes the Sudbury WWTP and the Garson Lagoons. Wastewater generated in the Garson Wastewater System is treated at the Sudbury WWTP; however, the lagoons are used occasionally in cases of wet weather events (i.e. heavy rainfall and/or snowmelt).

The Garson Wastewater System comprises three (3) lift stations, for which information can be found in Table 3-21.

Table 3-21 Garson Wastewater System Lift Station Summary

STATION	CURRENT FIRM CAPACITY (L/S)	EXISITING PEAK FLOW (L/S)
Gar-Con LS	24.3	18.52
O'Neil LS	98.6	N/A
Penman LS	8.3	6.5

Within the Garson Wastewater System, all flows generated are conveyed to the O'Neil LS. The O'Neil LS normally operates by allowing the wet well to overflow, and the overflow pipe conveys flows by gravity to the Sudbury WWTP. During wet weather events, 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.

3.8.2 EXISTING AND FUTURE REQUIREMENTS

Using the population projections and wastewater rate development process described in Section 1.4 of Volume 1 of this report, Garson's future wastewater projections were calculated. Table 3-22 summarizes the Garson wastewater flow criteria and the reference used to determine the criteria. Since the Garson Wastewater System is contained within the Sudbury Wastewater System, population and wastewater flow projections used for the Sudbury system were also used as the Garson criteria. Table 3-31 in Section 3.11.2 summarizes these criteria.

Table 3-22 Garson Wastewater System Flow Criteria

CRITERIA	VALUE	REFERENCE
Average Day Residential Flow	500 L/cap/d	Average of historical values, rounded up to nearest 50 L/cap/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
Max Day Peaking Factor	3.86	Average of historical values

3.8.3 GAP ANALYSIS CONCLUSIONS

As previously mentioned, a gap analysis was conducted in order to determine existing and future wastewater system deficiencies for each system. The following information is a summary of the Sudbury Wastewater Gap Analysis and Status Quo Report (WSP, 2015) contained in Appendix 3-A. The report can be referenced for more details regarding the analysis of the Garson Wastewater System.

BYPASSES AND WET WEATHER FLOW

Historical bypass events were reviewed as part of the gap analysis for each wastewater system in the CGS. No spills have been reported at the Garson LSs or the lagoons; however, the City has indicated that the area near the Gar-Con LS has experienced flooding and sewer backups, as a result of high I&I in this part of the system. Further discussion regarding I&I issues will follow in Volume 5 and in Volume 7.

TREATMENT

During analysis of the Garson Lagoons, it was identified that the process for filling and draining the lagoons during wet weather events is a manual undertaking, which has been noted to cause a strain on operations.

Capacity analysis of the Sudbury Wastewater System treatment (which includes the Garson wastewater flows) is detailed in Section 3.11.3.

LIFT STATIONS

Analysis of the Garson lift stations concluded that the Penman LS requires upgrading.

SEWERS

Refer to the Sudbury Wastewater System Gap Analysis and Status Quo Report (WSP, 2015) in Appendix 3-A, which outlines areas identified to have sewer capacity deficiencies within the Garson Wastewater System.

3.9 ONAPING-LEVACK WASTEWATER SYSTEM

Levack and Onaping are small communities located in the north-west end of the City of Greater Sudbury. The two communities are supplied by a single wastewater system and therefore are considered as one for the purposes of wastewater system analysis. Figure 3-15 illustrates the existing wastewater infrastructure in the Onaping-Levack Wastewater System.

3.9.1 EXISTING SYSTEM

All wastewater flows generated in the Onaping-Levack Wastewater System are treated at the Levack WWTP. The plant is a twin-celled extended aeration plant, which has a rated capacity of 2,270 m^3/d, and a maximum day capacity of 5,675 m^3/d.

The Onaping-Levack Wastewater system comprises also one (1) lift station, for which information can be found in Table 3-23.

Table 3-23 Onaping-Levack Wastewater System Lift Station Summary

STATION	CURRENT FIRM CAPACITY (L/S)	EXISTING PEAK FLOW
Fraser LS	27	36.8

Figure 3-15 Onaping-Levack Wastewater System: Existing Infrastructure

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3.9.2 EXISTING AND FUTURE REQUIREMENTS

Using the population projections and wastewater rate development process described in Section 1.4 of Volume 1 of this report, Onaping-Levack's future wastewater projections were calculated. Table 3-24 summarizes the Onaping-Levack wastewater flow criteria and the reference used to determine the criteria, and Table 3-25 summarizes the calculated projections.

Table 3-24 Onaping-Levack Wastewater System Flow Criteria

CRITERIA	VALUE	REFERENCE
Average Day Residential Flow	200 L/cap/d	Rounded up from average of historical values (156 L/cap/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
Max Day Peaking Factor	3.20	Average of historical values

Table 3-25 Onaping-Levack Wastewater Flow Projections

YEAR	POPULATION	AVERAGE DAY FLOW (M3/D)	MAXIMUM DAY FLOW (M3/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

3.9.3 GAP ANALYSIS CONCLUSIONS

As previously mentioned, a gap analysis was conducted in order to determine existing and future wastewater system deficiencies for each system. The following information is a summary of the Onaping-Levack Wastewater Gap Analysis and Status Quo Report (WSP, 2015), contained in Appendix 3-A. The report can be referenced for more details regarding the analysis of the Onaping-Levack Wastewater System.

BYPASSES AND WET WEATHER FLOW

Historical bypass events were reviewed as part of the gap analysis for each wastewater system in the CGS. One bypass event was noted in the Onaping-Levack Wastewater System in 2015, which was due to system maintenance.

TREATMENT

Analysis of the Levack WWTP concluded that there would be sufficient capacity to service the population growth to the year 2041. This can be seen in Figure 3-16 where the projected flows are plotted against the capacity of the plant.

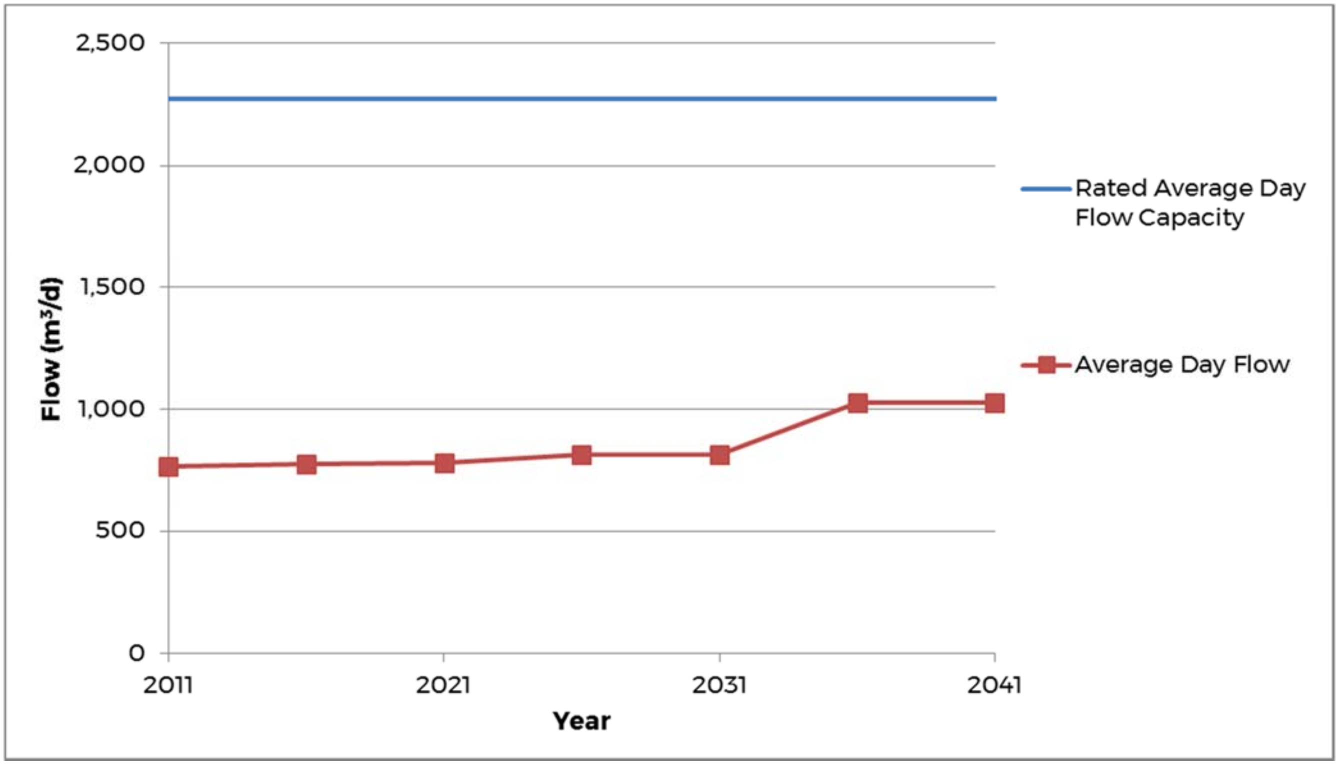

Figure 3-16 Onaping-Levack Wastewater Flow Projections vs. Rated Capacity

LIFT STATIONS

Analysis of the Onaping-Levack lift stations concluded that the Fraser LS requires upgrades to meet existing peak capacity requirements.

SEWERS

During assessment of the sewers within the Onaping-Levack Wastewater System, hydraulic computer modeling identified that the majority of sewers in the system flow at less than 50% of the available capacity from 2011 to 2041 under the wet weather flow conditions. Flow velocities through many sewers however, are generally below the City's standard of 0.6 m/s. This is consistent through to 2041 under the wet weather flow condition. Refer to the Onaping-Levack Wastewater System Gap Analysis and Status Quo Report (WSP, 2015), in Appendix 3-A, which outlines areas identified to have sewer capacity deficiencies within the Onaping-Levack Wastewater System.

3.10 LIVELY-WALDEN WASTEWATER SYSTEM

The Lively-Walden wastewater servicing area includes the areas of Lively, Walden, Naughton, and Mikkola. Figure 3-17 illustrates the existing wastewater infrastructure in the Lively-Walden Wastewater System.

Figure 3-17 Lively-Walden Wastewater System: Existing Infrastructure

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3.10.1 EXISTING SYSTEM

The Lively-Walden Wastewater System includes two (2) WWTPs; the Lively WWTP and the Walden WWTP. The Lively plant uses a conventional wastewater treatment process and services the community of Lively. The plant has an average rated capacity of 1,600 m^3/d, and a maximum day capacity of 3,000 m^3/d. The Walden WWTP is an extended aeration plant that services the community of Walden and its surrounding area. It has an average rated capacity of 4,500 m^3/d, and a maximum day capacity of 8,000 m^3/d.

The Lively-Walden Wastewater System comprises also seven (7) lift stations, for which information can found in Table 3-26.

Table 3-26 Lively-Walden Wastewater System Lift Station Summary

	CURRENT FIRM CAPACITY (L/S)	EXISITING PEAK FLOW (L/S)
Anderson	97.8	173.2
Jacob (includes the flows from Lively)	138.9	622.5
Magill	20.1	0.4
Oja	15.39	5.26
Simon Lake East	39.4	34.1
Simon Lake West	37.85	13.5
Vagnini	32.50	2.4

3.10.2 EXISTING AND FUTURE REQUIREMENTS

Using the population projections and wastewater rate development process described in Section 1.4 of Volume 1 of this report, Lively-Walden's future wastewater projections were calculated. Table 3-27 summarizes the Lively-Walden wastewater flow criteria and the reference used to determine the criteria, and Table 3-28 summarizes the calculated projections.

Table 3-27 Lively-Walden Wastewater System Flow Criteria

CRITERIA	VALUE	REFERENCE
Average Day Residential Flow	450 L/cap/d	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 (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)
Max Day Peaking Factor	4.05 for Lively 3.36 for Walden	Average of historical values

Table 3-28 Lively-Walden Wastewater Flow Projections

YEAR	POPULATION	AVERAGE DAY FLOW (M3/D)	MAXIMUM DAY FLOW (M3/D)
LIVELY WASTEWATER SYSTEM			
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
WALDEN WASTEWATER SYSTEM			
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
2036	6,299	7,515	25,282
2041	6,324	7,273	24,466

3.10.3 GAP ANALYSIS CONCLUSIONS

As previously mentioned, a gap analysis was conducted in order to determine existing and future wastewater system deficiencies for each system. The following information is a summary of the Lively-Walden Wastewater Gap Analysis and Status Quo Report (WSP, 2015) contained in Appendix 3-A. The report can be referenced for more details regarding the analysis of the Lively-Walden Wastewater System.

BYPASSES AND WET WEATHER FLOW

Historical bypass events were reviewed as part of the gap analysis for each wastewater system in the CGS. There have been numerous bypasses and spills in the Lively/Walden system since 2004, many of which occurred at the Lively WWTP. Between 2012 and November of 2016, eighteen (18) events were recorded at the Lively WWTP.

Additionally, there have been four (4) bypasses at the Walden WWTP between 2004 and 2012; all bypasses were primary bypasses, and three (3) were due to heavy precipitation. Between 2012 and November of 2016, fifteen (15) events were identified at the plant.

Bypasses at the Lively WWTP were mainly primary bypasses, due to heavy precipitation and/or snow melt, indicating I&I issues in the Lively-Walden Wastewater System. Additionally, 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 (1) of the manholes in this part of the system. Further discussion regarding I&I issues will follow in Volume 5 and in Volume 7.

TREATMENT

Analysis of the Lively and Walden WWTPs concluded that there would not be sufficient capacity to service the population growth to the year 2041. In the Lively/Walden Environmental Summary Report (J.L. Richards & Associates Ltd., 2013) this 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.

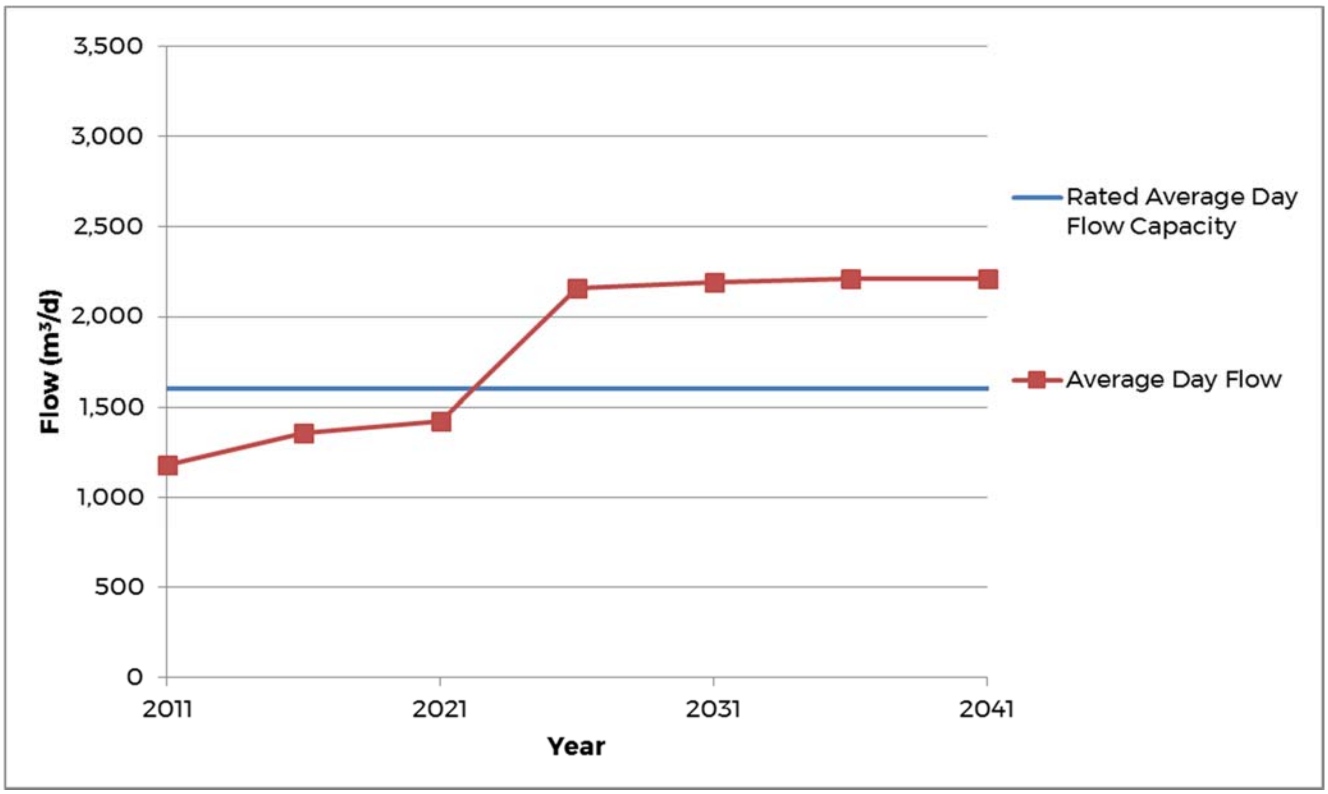

Figure 3-18 Lively Wastewater Flow Projections vs. Rated Capacity

Figure 3-19 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 is illustrated to have already reached about 85% of its capacity in 2013, based on the projections. An additional 1,240 m^3/d would be required by 2021, and an additional 4,986 m^3/d by 2041.

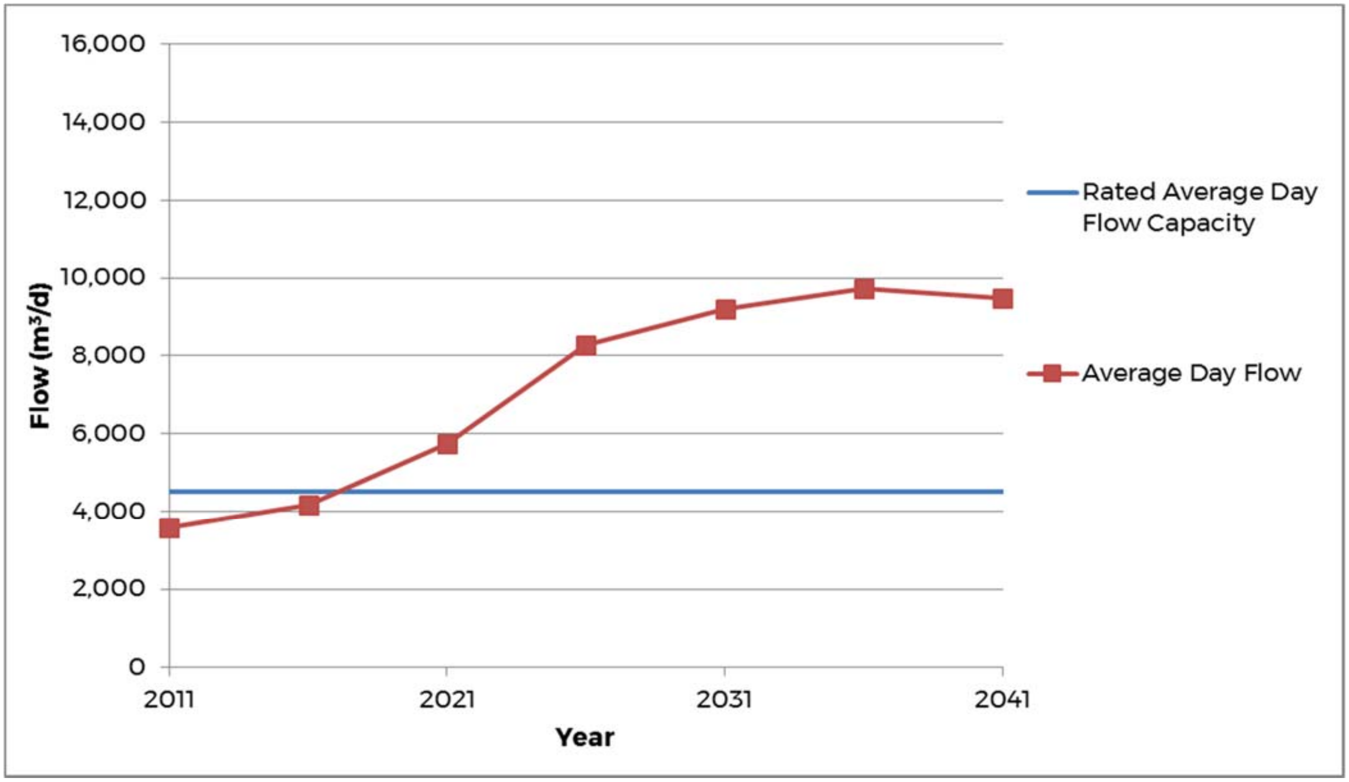

Figure 3-19 Walden Wastewater Flow Projections vs. Rated Capacity

LIFT STATIONS

Analysis of the Lively-Walden lift stations concluded that 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.

SEWERS

During assessment of the sewers within the Lively-Walden Wastewater System, hydraulic computer modeling identified that no capacity issues were identified in the system, as most of the sewers flow at less than 50% capacity; however, the sewer along MR24 from Lively to Walden does not have sufficient capacity to convey 2041 peak flows. Additionally, the sewer along 3rd Avenue does not have sufficient capacity to convey flows from 2011 through 2041. It was also noted that flow velocities through many sewers in the Lively-Walden system are generally below the City's standard of 0.6 m/s. This is consistent through to 2041 under the wet weather flow condition. Refer to Appendix 3-A of the Lively-Walden Wastewater System Gap Analysis and Status Quo Report (WSP, 2015) which outlines areas identified to have sewer capacity deficiencies within the Lively-Walden Wastewater System.

3.11 SUDBURY WASTEWATER SYSTEM

Sudbury is located in the central portion of the City of Greater Sudbury and it is the most populated community. Figure 3-20 illustrates the existing wastewater infrastructure in the Sudbury Wastewater System.

Figure 3-20 Sudbury Wastewater System: Existing Infrastructure

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3.11.1 EXISTING SYSTEM

The Sudbury Wastewater System services the community of Sudbury and Garson (See Section 3.8), for which wastewater is treated at the Sudbury WWTP and the Garson Lagoons. As previously described, wastewater generated in Garson is typically treated at the Sudbury WWTP; however, the lagoons are used occasionally in cases of wet weather emergencies. The WWTP is a conventional activated sludge plant with an average rated capacity of 79,625 m^3/d and a maximum day capacity 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.

The Sudbury Wastewater System comprises also twenty seven (27) lift stations. Table 3-29 contains relevant information regarding the Sudbury lift stations.

Table 3-29 Sudbury Wastewater System Lift Station Summary

NAME	CURRENT FIRM CAPACITY (L/S)	EXISITING PEAK FLOW (L/S)
Bell Park	N/A	N/A
Beverly	28.8	36.62
Brenda	13.3	7.28
Cerilli	14	2.33
Countryside	7.6	3.79
Don Lita	30.3	52.06
Dufferin	6.4	4.8
Ester	28.4	13.98
Fourth	15.2	31.24
Helen's Point	7.6	5.99
Kincora	8.7	2.91
Lagace	14	56.95
Lakeview	20.9	0.64
Levesque	167.6	176.83
Loach's Road	12.1	5.44
Marcel – Bouchard	303.3	N/A
Mark	41.7	17.22
Moonlight	16.3	19.73
Moonlight Beach	N/A	N/A
Northshore	11.4	4.23
Ramsey	32.2	46.43

Selkirk	38.7	31.65
Sherwood	30	24.68
Southview	58.8	108.13
St. Charles	383	254.44
Walford East	127	77.98
York	13.2	25

Another important note is that 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. Currently, there are four (4) 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 the Sudbury tunnels is shown in Figure 3-21.

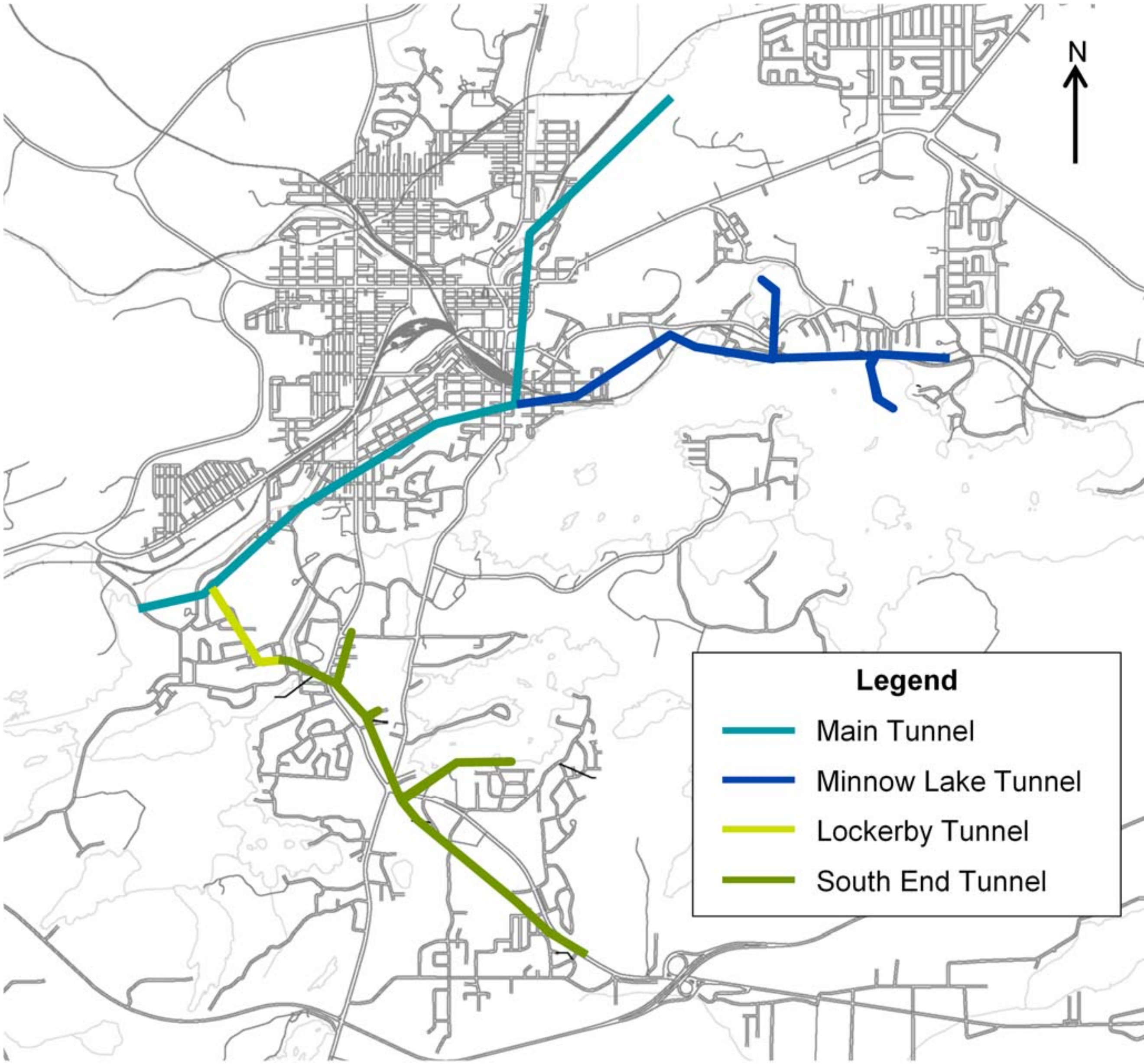

Figure 3-21 Sudbury Rock Tunnel Locations

3.11.2 EXISTING AND FUTURE REQUIREMENTS

Using the population projections and wastewater rate development process described in Section 1.4 of Volume 1 of this report, Copper Cliff's future wastewater projections were calculated. Table 3-31 summarizes the Sudbury wastewater flow criteria and the reference used to determine the criteria, and Table 3-32 summarizes the calculated projections for wastewater flow generated in Sudbury, Garson and Copper Cliff. Wastewater flows generated in Copper Cliff have been used for the analysis of the Sudbury WWTP since the City has in its plans to pump all wastewater flows generated in Copper Cliff into the Sudbury wastewater network via the Nickel LS.

Table 3-30 Sudbury Wastewater System Flow Criteria

CRITERIA	VALUE	REFERENCE
Average Day Residential Flow	500 L/cap/d	Average of historical values, rounded up to nearest 50 L/cap/d
Average Day Commercial and Industrial Flow	28 m^3/ha/d	Water unit rates, assuming a 1:1 correspondence
Average Day Institutional Flow	35 m^3/ha/d	Water unit rates, assuming a 1:1 correspondence
Max Day Peaking Factor	3.86	Average of historical values

Table 3-31 Sudbury, Garson and Copper Cliff Wastewater Flow Projections

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.

3.11.3 GAP ANALYSIS CONCLUSIONS

As previously mentioned, a gap analysis was conducted in order to determine existing and future wastewater system deficiencies for each system. The following information is a summary of the Sudbury Wastewater Gap Analysis and Status Quo Report (WSP, 2015) contained in Appendix 3-A. The report can be referenced for more details regarding the analysis of the Sudbury Wastewater System.

BYPASSES AND WET WEATHER FLOW

Historical bypass events were reviewed as part of the gap analysis for each wastewater system in the CGS. From 2004 to 2011, forty-one (41) spills were reported at the Sudbury WWTP, and from 2009 to 2011 four (4) overflow/sewer bypass events were reported at both the Stewart LS and Moonlight LS, as well as one (1) bypass at the Green LS. It should be noted, however, that the Stewart and Green LS's have since been decommissioned. Between 2012 and November of 2014, twenty-three (23) events were recorded at the Sudbury WWTP.

It was also noted that there are recognized I&I issues in the Sudbury Wastewater System. Further discussion regarding I&I issues will follow in Volume 5 and in Volume 7.

TREATMENT

Analysis of the Sudbury WWTP concluded that the Phase 2 upgrades mentioned in Section 3.11.1 will be required for sufficient capacity to service the population growth to the year 2041. This can be seen in Figure 3-22 where the projected flows are plotted against the capacity of the plant.

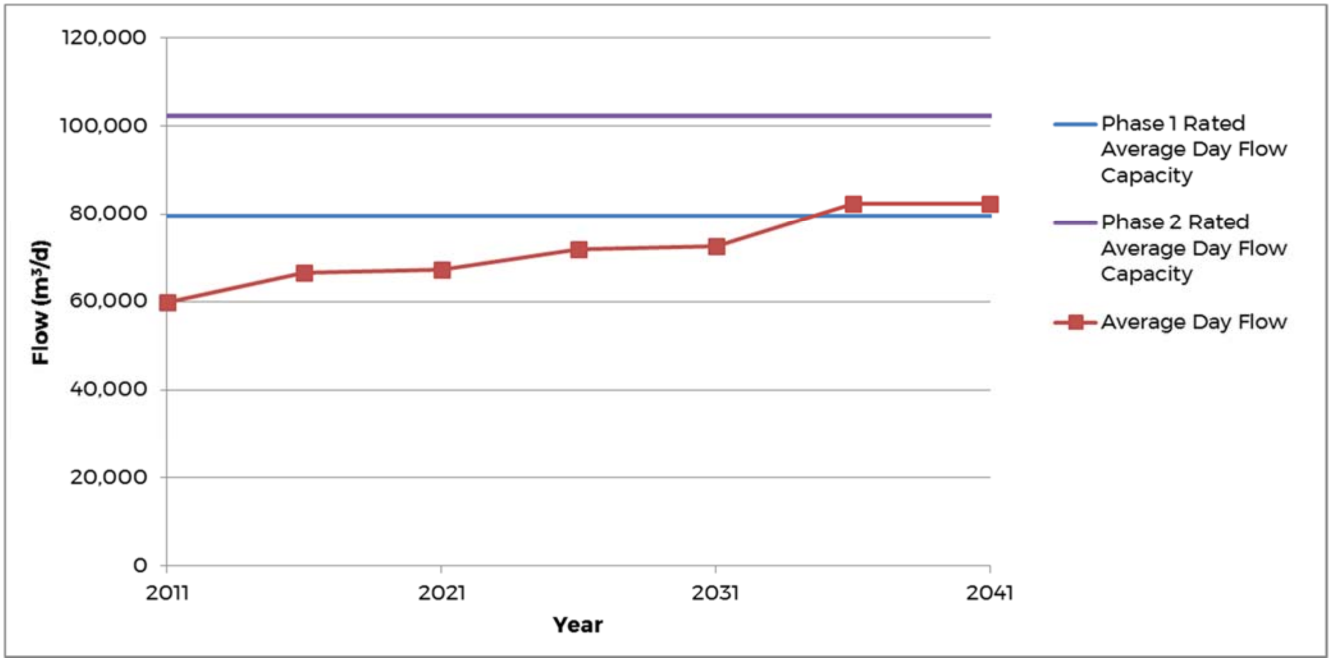

Figure 3-22 Sudbury Wastewater Flow Projections vs. Rated Capacity

LIFT STATIONS

Analysis of the Sudbury lift stations concluded that there are capacity concerns at the following lift stations: Levesque, Lagace, Moonlight, Beverley, Don Lita, Fourth, Ramsey, Sherwood, Southview, St. Charles, and York. These lift station do not have sufficient capacity to service the population growth to the year 2041.

SEWERS

During assessment of the sewers within the Sudbury Wastewater System, hydraulic computer modeling identified that no capacity issues were identified in the system, as most of the sewers flow at less than 50% of the available capacity through to 2041 under the wet weather flow condition, with some exceptions. Of note, the trunk sewer that generally parallels Junction Creek would exceed 50% capacity at 2041. Additionally, many areas also flow at less than the city's current standard of 0.6 m/s.

Appendix 3-A of the Sudbury Wastewater System Gap Analysis and Status Quo Report (WSP, 2015) outlines areas identified to have sewer capacity deficiencies within the Sudbury Wastewater System.

3.12 VALLEY EAST WASTEWATER SYSTEM

The Valley East Wastewater System services the communities of Hanmer, Vale Caron, and Val Therese. Figure 3-23 illustrates the existing wastewater infrastructure in the Valley East Wastewater System.

Figure 3-23 Valley East Wastewater System: Existing Infrastructure

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3.12.1 EXISTING SYSTEM

The Valley East Wastewater System is serviced by the Valley East WWTP, which is owned and operated by the CGS. It is a conventional activated sludge treatment plant with an average day rated capacity of 11,365 m^3/d.

The Valley East Wastewater System also comprises nine (9) lift stations, information for which can be found in Table 3-32.

Table 3-32 Valley East Wastewater System Lift Station Summary

	CURRENT FIRM CAPACITY	EXISITNG PEAK FLOW
Fleming	25.1	6.6
Helene	40.3	92.4
Hillsdale	52.2	9.1
Jeanne D'Arc	110	170.1
Madeleine	15.2	3.0
Spruce	74	119.3
St. Isidore	27.9	18
Tena	22	1.75
Tupper	9.4	0.94

3.12.2 EXISTING AND FUTURE REQUIREMENTS

Using the population projections and wastewater rate development process described in Section 1.4 of Volume 1 of this report, Valley East's future wastewater projections were calculated. Table 3-33 summarizes the Valley East wastewater flow criteria and the reference used to determine the criteria, and Table 3-34 summarizes the calculated projections.

Table 3-33 Valley East Wastewater System Flow Criteria

CRITERIA	VALUE	REFERENCE
Average Day Residential Flow	250 L/cap/d	Average of historical values, rounded up to nearest 50 L/cap/d
Average Day Commercial and Industrial Flow	28 m^3/ha/d	Water unit rates, assuming a 1:1 correspondence
Average Day Institutional Flow	35 m^3/ha/d	Water unit rates, assuming a 1:1 correspondence
Max Day Peaking Factor	2.69	Average of historical values

Table 3-34 Valley East Wastewater 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

YEAR	POPULATION	AVERAGE DAY FLOW (M^3/D)	MAXIMUM DAY FLOW (M^3/D)
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

3.12.3 GAP ANALYSIS CONCLUSIONS

As previously mentioned, a gap analysis was conducted in order to determine existing and future wastewater system deficiencies for each system. The following information is a summary of the Valley East Wastewater Gap Analysis and Status Quo Report (WSP, 2015), contained in Appendix 3-A. The report can be referenced for more details regarding the analysis of the Valley East Wastewater System.

BYPASSES AND WET WEATHER FLOW

Historical bypass events were reviewed as part of the gap analysis for each wastewater system in the CGS. From 2009-2013, only one (1) event was recorded at the Valley East WWTP, due to heavy precipitation or snow melt. Additionally, between 2013 and November of 2016, nine (9) overflow events were reported at the Valley East WWTP.

I&I has also been recognized as an issue within the Valley East Wastewater System. Further discussion will follow in Volume 5 and in Volume 7.

TREATMENT

Analysis of the Valley East WWTP concluded that there would not be sufficient capacity to service the population growth to the year 2041. An additional 162 m^3/d would be required by 2041. This can be seen in Figure 3-24 where the projected flows are plotted against the capacity of the plant.

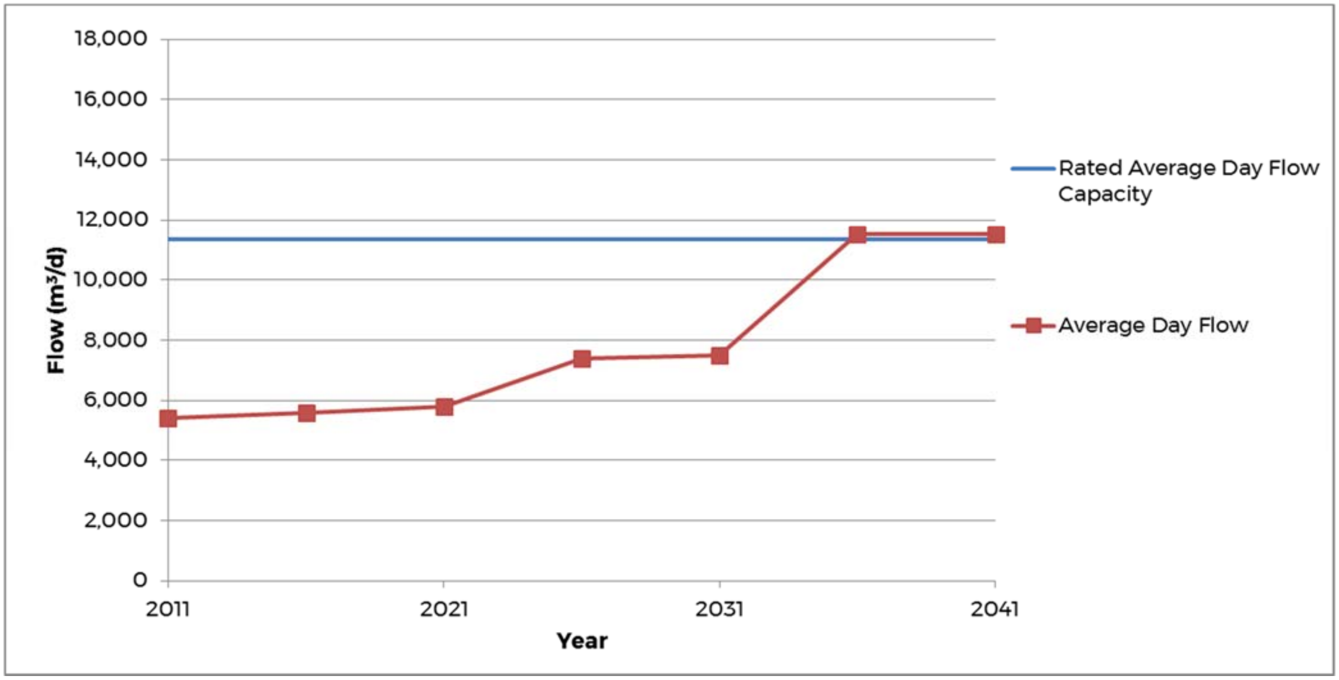

Figure 3-24 Valley East Wastewater Flow Projections vs. Rated Capacity

LIFT STATIONS

Analysis of the Valley East lift stations concluded that existing capacities are not sufficient to service existing peak inflow at the Helene LS, Jeanne D'Arc LS, and Spruce LS.

SEWERS

During assessment of the sewers within the Valley East Wastewater System, hydraulic computer modeling identified that that many of the sewers in the system flow at less than 50% of the available capacity through to 2041 under the wet weather flow condition. Additionally, flow velocities in Valley East are less than 0.6 m/s in most areas through to 2041 under the wet weather flow condition. Appendix 3-A of the Valley East Wastewater System Gap Analysis and Status Quo Report (WSP, 2015) outlines areas identified to have sewer capacity deficiencies within the Valley East Wastewater System.

3.13 WAHNAPITAE WASTEWATER SYSTEM

Wahnapitae is a small community located in the east end of the CGS, east of both Coniston and Sudbury proper. Figure 3-25 illustrates the existing wastewater infrastructure in the Wahnapitae Wastewater System.

3.13.1 EXISTING SYSTEM

All wastewater generated in the Wahnapitae Wastewater System is collected and treated at the Wahnapitae Wastewater Lagoon. The lagoons are comprised of three (3) cells, are located in the Town of Nickel Centre, and have a rated capacity of 1,246 m^3/d. It should be noted that treated water is seasonally discharged to the Wahnapitae River.

The Wahnapitae Wastewater System comprises also one (1) lift station, for which information can be found in Table 3-35.

Table 3-35 Wahnapitae Wastewater System Lift Station Summary

	CURRENT FIRM CAPACITY (L/S)	EXISTING PEAK FLOW (L/S)
Riverside	52	141.7

3.13.2 EXISTING AND FUTURE REQUIREMENTS

Using the population projections and wastewater rate development process described in Section 1.4 of Volume 1 of this report, Wahnapitae's future wastewater projections were calculated. Table 3-36summarizes the Wahnapitae wastewater flow criteria and the reference used to determine the criteria, and Table 3-37 summarizes the calculated projections.

Table 3-36 Wahnapiae Wastewater System Flow Criteria

CRITERIA	VALUE	REFERENCE
Average Day Residential Flow	500 L/cap/d	City's Engineering Design Manual, rounded down from 471 L/ca/d
Average Day Institutional & Commercial Flow	28 m^3/ha/d	Water unit rates, assuming a 1:1 correspondence
Average Industrial Flow	35 m^3/ha/d	Water unit rates, assuming a 1:1 correspondence
Max Day Peaking Factor	3.67	Estimated by assuming the same factor as Coniston community.

Figure 3-25 Wahnapitae Wastewater System: Existing Infrastructure

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Table 3-37 Wahnapitae Wastewater 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

3.13.3 GAP ANALYSIS CONCLUSIONS

As previously mentioned, a gap analysis was conducted in order to determine existing and future wastewater system deficiencies for each system. The following information is a summary of the Wahnapitae Wastewater Gap Analysis and Status Quo Report (WSP, 2015), contained in Appendix 3-A. The report can be referenced for more details regarding the analysis of the Wahnapitae Wastewater System.

BYPASSES AND WET WEATHER FLOW

Historical bypass events were reviewed as part of the gap analysis for each wastewater system in the CGS. No bypass events were recorded in the Wahnapitae Wastewater System between 2011 and November of 2016; however, I&I has been identified as a concern in the system. Further discussion will follow in Volume 5 and in Volume 7.

TREATMENT

Analysis of the Wahnapitae Lagoons concluded that there would be sufficient capacity to service the population growth to the year 2041. This can be seen in Figure 3-26 where the projected flows are plotted against the capacity of the Lagoon.

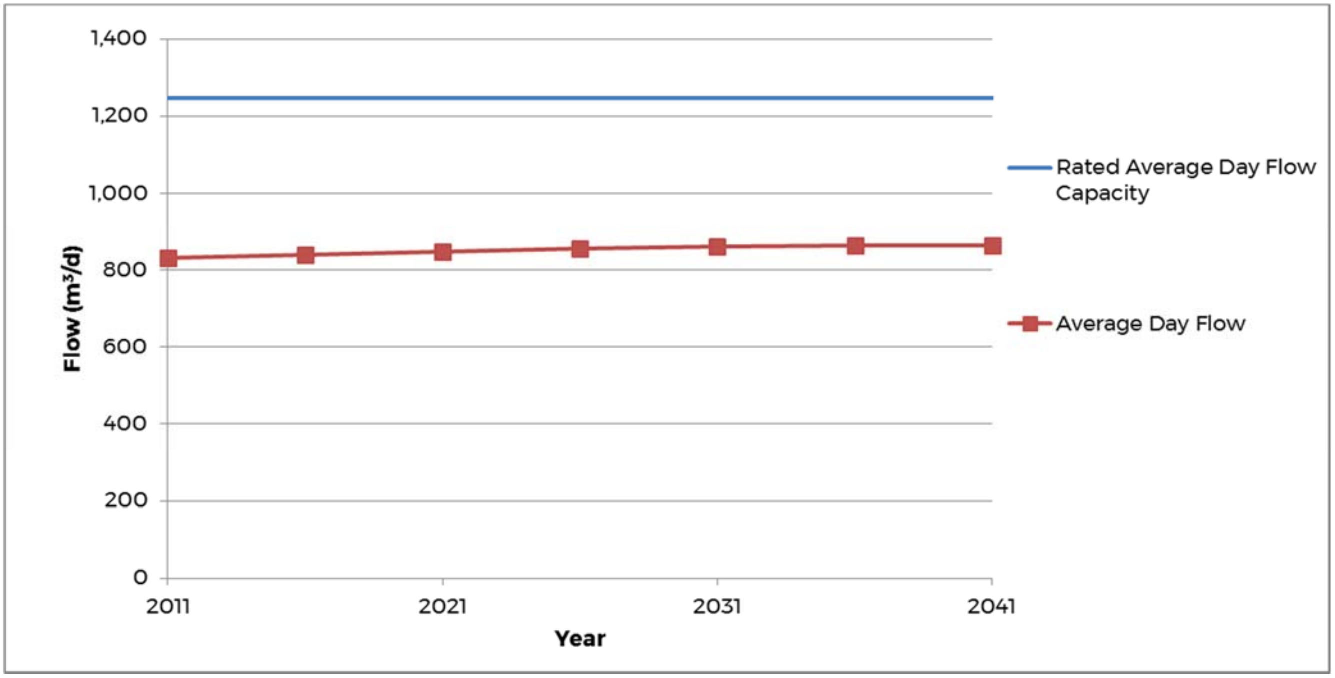

Figure 3-26 Wahnapitae Wastewater Flow Projections vs. Rated Capacity

LIFT STATIONS

Analysis of the Riverside lift station concluded that it does not have capacity to convey peak flows from 2011 through to Ultimate Buildout.

SEWERS

During assessment of the sewers within the Wahnapitae Wastewater System, hydraulic computer modeling identified that many of the sewers in the Wahnapitae system operate at less than 50% capacity, except for one (1) segment near the Riverside LS. This is apparent from 2011 through to 2041. Additionally, from 2011 through to 2041, flow velocities in some sewers in Wahnapitae are below the City's standard of 0.6 m/s. Appendix 3-A of the Wahnapitae Wastewater System Gap Analysis and Status Quo Report (WSP, 2015) outlines areas identified to have sewer capacity deficiencies within the Wahnapitae Wastewater System.

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