Sewer System Master Plan

Greater Shediac Sewerage Commission (GSSC)
Final Report

March 13, 2023
2200536.000

ENGLOBE

Greater Shediac Sewerage Commision

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Revisions and publications log

REVISION No.	DATE	DESCRIPTION
0A	December 22, 2022	Draft Report
1A	March 13, 2023	Final Report

Distribution

1 PDF copy	Mr. Joey Frenette – Director, Operations

Executive Summary

Englobe Corp. was hired by the Greater Shediac Sewerage Commission to review their sewer collection system and identify hydraulic risks and infrastructure deficiencies. The Commission is frequently asked about the capacity of its system for potential development and has primarily made infrastructure investments reactively.

By creating a hydraulic model of the collection system, the Commission can better respond to development requests and make informed infrastructure investment decisions. This Sewer System Master Plan report details the construction of the hydraulic model and provides findings and recommendations.

Project Objectives

The objectives of this assignment included:

- Construct a comprehensive hydraulic model of the Commission's sewer collection network;
- Estimate peak flow conditions throughout the network;
- Analyze capacity constraints and hydraulic risk areas;
- Estimate flow changes which could result from development within the study area;
- Identify infrastructure which is likely to see an increased hydraulic risk from increased development;
- Make recommendations to address the deficiencies identified.

Methodology

Englobe used available system information from multiple sources, including previous baseplan data, record drawings and operator knowledge to assemble a SewerCAD (CONNECT edition) of the Commission's collection system in the Town.

This model was validated using available observations on flow to represent peak flow conditions to identify capacity constraints during such a scenario. Estimated flows from future development areas were also reviewed to highlight system components anticipated to be at increased hydraulic risk from continued development

System Performance

While the collection system was generally found to have a sufficient level of service, specific sections of the sewer system were identified as being at a heightened risk of surcharging. In cases where capacity constraints were identified, both under existing conditions and following additional development, one or more corresponding recommendations were made.

Recommended Improvements

The following recommendations were made to address the current and projected capacity needs of the Commission's collection system infrastructure in the Town of Shediac.

- **Inflow and Infiltration Reduction**: Flows in Shediac associated with infiltration and inflow were relatively high compared to similar jurisdictions. Reduction of these flow components presents the most important opportunity for the Commission to reduce risk and alleviate capacity for future development
  - **Flow Monitoring and Continued Study:** To provide focus to the Commission's Inflow and Infiltration reduction efforts and to estimate the benefits associated with upgrades, it is recommended the Commission continue its program of flow monitoring. Where possible, this program should be accelerated. **Estimated cost of $100,000 per year.**
- **SCADA Improvements:** While many of the Commission's lift stations are equipped with flow meters, there remain a few which are not, including a couple of key locations. Flow meters at lift stations provide an excellent opportunity to obtain continuous long-term data on flow conditions in the Commision's collection system.
- **Gravity Sewer Improvements:** The following sections were recommended for an upgrade to reduce existing hydraulic risks and provide capacity for future development in contributing areas.
  - Backlot Gravity Sewer near Greenwood Prom. **(Est. cost - $36k)**
  - West Main Street and Dock Street **(Est. cost - $1.9 M)**
  - Main Street East **(Est. cost - $0.7M)**
  - Main St. and Caissie Avenue **(Est. cost - $0.9M)**
- **Lift Stations & Forcemains:** Several lift stations are expected to require additional capacity to serve current and future flow conditions. Those stations include:
  - LS #2 **(Est. cost - $7.8 M)**
  - LS #3 **(Est. Cost - See "Trunk Sewer Bypass")**
  - LS #14 **(Est. cost - $3.1 M)**
  - LS #15 **(Est. cost - $2.3 M)**
  - LS #5 **(Est. cost - $2.8 M)**
- **Trunk Sewer Bypass**: The proposed upgrade includes bypassing both Lift Station 3 and Lift Station 4 to relieve capacity in the trunk sewer and re-routing a new forcemain to the south. This would reduce hydraulic surcharging and provide additional capacity for future development in the Trunk Sewer sewershed. A Preliminary Design study is recommended if the proposal is favourable to the Commission to understand the infrastructure upgrades required and select a preferred forcemain alignment. **(Est. cost - $21.1 M)**
- **Long-Term Servicing:** Large undeveloped areas in the Town were reviewed to identify possible infrastructure requirements to service those areas.
  - **West of Ohio Road:** A proposed servicing concept for a large undeveloped area is to install a new collector sewer along a watercourse parallel to an existing trail system. The area is a high priority for development, and the collector sewer could relieve existing infrastructure and de-commission Lift Station 16. It could also provide servicing to lands south of the highway, including the Southeast Regional Correctional Center. The proposed collector sewer would also play a role in the Trunk Sewer Bypass recommendation. **(Est. Cost - See "Trunk Sewer Bypass")**

  - **East of Ohio Road:** A proposed servicing concept for this area includes a network of collector sewer branches which generally follow existing watercourse in the area. The feasibility of a new trunk sewer crossing Route 133 and running along the south end of the WWTF to the headworks lift station needs to be explored, or a new upgraded lift station near the low point on Route 133 with significant piping upgrades would be required. **(Cost not estimated)**

- **Model Updates:** It is recommended the Commission establish a budget for ongoing maintenance and upgrade of the SewerCAD model and this associated Sewer Master Plan study. These updates would include changes related to ongoing development, new flow data, information collected through field programs, and completed projects among others. (**Budget to be established separately)**

Property and Confidentiality

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1 Introduction

1.1 Background

Englobe Corp. (Englobe) was retained by the Greater Shediac Sewerage Commission (GSSC, the Commission) to complete a detailed review of the Commission's sewer collection system. The overall purpose of this assignment was to identify current and anticipated hydraulic risks and infrastructure deficiencies in the collection system, which includes gravity sewers and lift stations.

The Commission is routinely asked to comment on the available capacity of their collection system to service prospective development in the Town. Furthermore, infrastructure investment has been largely reactionary to known capacity issues or to leverage cost efficiencies when partnering with the Town on infrastructure renewal programs.

By investing in a hydraulic model of the complete collection system, the Commission will be better equipped to respond to development requests and make informed infrastructure investment decisions.

This report aims to document the construction of the hydraulic model and present findings and recommendations that have resulted.

1.2 Project Objectives

The objectives of this assignment included:

- Construct a comprehensive hydraulic model of the Commission’s sewer collection network;
- Estimate peak flow conditions throughout the network;
- Analyze capacity constraints and hydraulic risk areas;
- Estimate flow changes which could result from development within the study area;
- Identify infrastructure which is likely to see an increased hydraulic risk from increased development;
- Make recommendations to address the deficiencies identified.

It is anticipated the model will continue to be improved and updated to reflect new information (such as flow monitoring data), system expansions, capital works projects, and flow changes associated with development. The Commission should continue to leverage this asset for infrastructure planning moving forward.

2 Existing Conditions

2.1 Project Area

The Town of Shediac is located on the southeast shore of New Brunswick in Westmorland County just south of Moncton. The population in 2021 was just above 7,500, a growth of 13.1% from the last census in 2016 (2.62% annual growth). Significant high-density development is expected in the short term where approximately 3200+ units are expected covering about 170.5 hectares (~18.75 units/hectare). In addition, approximately 1,730 hectares have yet to be developed. Depending on the density and the zoning of the development could significantly impact the population and the municipal infrastructure. The Town is mostly residential lots with commercial and seasonal camping sites scattered throughout.

Please refer to **Map No 2-1 – Existing Sanitary Network** in **Appendix A.**

2.2 Sources of Data

2.2.1 GIS Model

The primary source of information used to construct the hydraulic model was the Commission's GIS database. This database was originally developed using the Master Plan (2018) prepared by Englobe (previously Crandall Engineering) using a combination of as-built drawings and field information collected over the years. This database has been periodically updated when new information becomes available.

Where gaps were identified in the database, Englobe attempted to address those gaps through a review of alternative information sources described below.

2.2.2 Record Drawings

Where information gaps or inconsistencies were identified from the Commission's GIS dataset, Englobe assembled and reviewed record drawing information to improve the model. Many of these record drawings were assembled from Englobe's project records as much of the system was designed by Englobe (formerly Crandall Engineering).

2.2.3 SCADA

The GSSC SCADA system provides real-time information for the lift stations and the WWTF for Shediac and Scoudouc. For Shediac, the following is included in the SCADA:

- Each LS operated by GSSC with exception of LS #17 (information not entered yet by client);
- At the WWTP:
  - The submersible pumps
  - The screw pumps;
  - The UV building;
  - The blower building.

Each of the lift stations has minor differences in the information that is provided. In general, the following information is provided:

— Pump run time, hours and settings;

— Flow meter total flow and active flow (if available);

— Level sensor readings;

— Elevations for pipe inverts, bottom of wet well, and ground elevation;

— LS operating set levels;

— Standby generator including the number of starts and hours of operation;

2.2.4 Field Data Collection

Once the available information (record drawings and master plan) was entered into Englobe's GIS model, information gaps were identified for further investigation by topographic and intrusive surveys. The collected information was then processed and entered in the GIS model.

Some manholes which were identified in the GIS model with missing information were not able to be found or opened during Englobe's survey. Some manholes were found with the help of a metal detector which have been paved over. To avoid damaging the road structure, these manholes were left as is and assumptions were made for the hydraulic model. Please see Section 2.2.7 for more information.

2.2.5 Previous Reports

2.2.5.1 I&I Study West Shediac (Englobe Project # 2006201)

Englobe was retained by GSSC to complete a comprehensive study on a portion of the wastewater collection system infrastructure in West Shediac. The intent of the study was to identify sources of rainfall-derived inflow and infiltration (RDII) which increases hydraulic loading on the wastewater collection system during and following periods of rain. The study included six (6) strategically located flow meters for a period of six (6) weeks.

The findings of this report, in conjunction with flow meter data from the SCADA, were used to estimate I&I flows in West Shediac. The values from the report also provided a better understanding of I&I which provided some validation for other I&I estimates for the many sewersheds throughout the Shediac wastewater system.

The final report was submitted to GSSC on June 6th, 2022.

2.2.5.2 Shediac East Long-Term Strategy (Englobe Project # 17250)

In 2014, GSSC retained the services of Englobe (formerly Crandall Engineering) to complete a long-term wastewater management strategy for the Shediac East area (Cap-Brulé). The purpose of this study was to complete a comprehensive review of the entire wastewater treatment facility (WWTF) and to provide conceptual design and review of options to upgrade the facility to meet long term needs.

The findings of this report were used to validate model results for average dry weather flows (ADWF), peak dry weather flows (PDWF) and peak wet weather flows.

2.2.6 Assumptions & Model Validation

The elevations from the GIS model were used in most instances unless elevation conflicts were noticed. GIS Model information gaps were identified, and a topographic and intrusive survey was completed. However, due to some manholes being buried or inaccessible, some assumptions were required.

- Missing top of manhole elevations were assumed with Lidar information as found on GeoNB.SNB.ca. The surrounding known top of manhole elevations were used to validate the Lidar elevations.
- In the instances where the top of cover was provided but no inverts and there was enough information for nearby structures, linear interpolation was used to approximate the pipe inverts. If no nearby infrastructure had reliable information, it was assumed that the pipe grade to not equal less than 0.3%. Although conservative, during the analysis of the sanitary hydraulic model, if a section of pipe was identified as a potential issue, this assumption was further scrutinized.

Following the process of populating all available physical model information and filling any remaining gaps, the entire model was reviewed by viewing pipe profiles to identify any discrepancies with elevations or grades. **Appendix A** includes a **Map 2-2 – Assumptions for Pipe Info** showing areas where assumptions were made and their associated adjustments.

2.3 Existing Infrastructure

2.3.1 Existing Collection System

The sanitary sewer collection system includes just over 91 km of sewer main where just under 75% is 200mm in diameter. In general, the diameter of the arterial roads is 200mm, and larger diameters are found for the trunk sewer main to the WWTP and the minor trunk sewer currently installed on the West main project. Please refer to **Map No 2-1 – Existing Sanitary Network** in **Appendix A**.

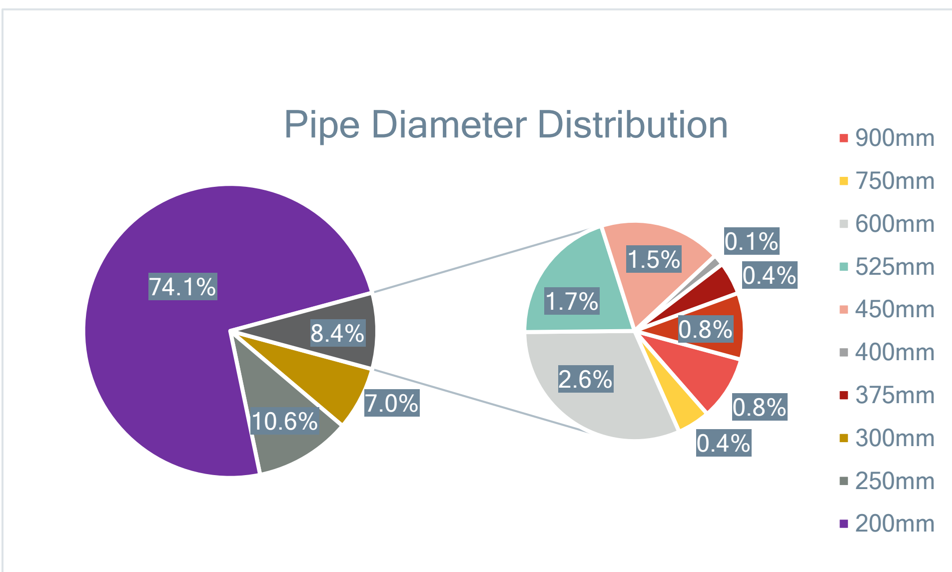


Figure 2-1: Pipe Diameter Distribution

2.3.2 Existing Lift Stations

A total of 24 lift stations contribute flows to the existing Cap Brule WWTF. 21 are operated and maintained by GSSC and three (3) additional private lift stations pump into the GSSC sanitary system. Table 2-1 below provides information for each lift station. Pumping capacities were taken from the flow meter data from Commission's SCADA where available.

For stations without a metering device, GSSC performed drawdown tests which have been identified in **bold text in the table below**. LS #9 and #14 are scheduled to be upgraded in the near future thus drawdown tests were not performed, and the capacity of these stations was provided from a previous report.

Table 2-1: Existing Lift Station Information

Lift Station	Location	Make and Type	HP (Each)	Pumping Capacity (one pump)
Shediac				
LS #1	63 Prom. Evergreen Drive	Hydr-O-Matic Self Primers - 4"	5 H.P.	**8.1 L/s**
LS #2	227 rue Main Street	Gorman Rup Self Priming - 4"	10 H.P.	22.8 L/s
LS #3	38 rue Dock Street	Gorman Rup Self Priming - 6"	20 H.P.	32.9 L/s
LS #4	71 rue Hamilton Street	Gorman Rup Self Priming - 6"	20 H.P.	38.0 L/s
LS #5	150 rue Pleasant Street	Hydr-O-Matic Self Priming - 4"	5 H.P.	**6.6 L/s**
LS #6	476 rue Paturel Street	Flygt Submersible	10 H.P.	22.2 L/s
LS #11	560 rue Wayne Street	Flygt Submersible	5 H.P.	8.5 L/s
LS #13	77 rue Napoleon Street	Gorman Rup Self Priming - 4"	7.5 H.P.	**9.6 L/s**
LS #15	87 ch Cornwall Road	Hydr-O-Matic Self Priming - 4"	5 H.P.	**6.3 L/s**
LS #16	302 Côte Bellevue Hills	Hydr-O-Matic Self Priming - 4"	5 H.P.	**4.4 L/s**
LS #17	30 rue Bordeaux Street	Hydr-O-Matic Submersible	3 H.P.	1.6 L/s
Priv. LS #1	Chandler Shore Road	Unknown	Unknown	1.6 L/s*
Priv. LS #2	Off Riverside Prom.	Unknown	Unknown	0.8 L/s*
Cap Brulé				
Lagoon Bldg.	25 ch Brulé Rd	Flygt Submersible	30 H.P.	66.2 L/s
		Screw Pumps Archimedes Type	20 H.P.	94.6 L/s
LS #12	28 allée Pussyfoot Ln	Hydr-O-Matic Self Priming - 4"	3 H.P.	**5.5 L/s**
Priv. LS#3	Euclide Leger Road	Unknown	Unknown	0.8 L/s*
Point du Chene				
LS #7	48 rue Jarvis Street	Flygt Submersible	5 H.P.	10 L/s
LS #8	135 ch Point du Chene Rd	Hydr-O-Matic Self Priming - 4"	5 H.P.	**8.8 L/s**
LS #9	27 allée Hunters Ln	Hydr-O-Matic Self Priming - 4"	3 H.P.	6.3 L/s

Lift Station	Location	Make and Type	HP (Each)	Pumping Capacity (one pump)
LS #10	63 ave McKenzie Avenue	Flygt Submersible	25 H.P.	34.6 L/s
LS #18	Pointe du Chene Wharf	Flygt Submersible	5 H.P.	**4.6 L/s**
Shediac Cape				
LS #14	28 allée Herron Way	Hydr-O-Matic Self Priming - 4”	7.5 H.P.	12.6 L/s
Boudreau West				
LS #19	910 Route 133	Flygt Submersible	15 H.P.	41.1 L/s
Cap Bimet				
LS #20	63 Cap Bimet Blvd	Flygt Submersible	15 H.P.	29.1 L/s
Scoudouc (outside study area)				
LS #21	3755 Rte 132, Scoudouc	Hydr-O-Matic Submersible	25 H.P.	8.9 L/s**
LS #22	3526 Rte 132, Scoudouc	Hydr-O-Matic Submersible	15 H.P.	12.3 L/s
LS #23	3180 Rte 132, WWTP	Flygt Submersible	10 H.P.	29 - 32 L/s

*Information not available, therefore assumption has been made for pumping capacity.
** VFD is set at 48 Hz; therefore, additional capacity is available.

2.4 Population and Growth

Historical growth rates from the Canadian Census of Population data were analyzed to establish the current population and estimate future population growth.

Table 2-2: Population Statistics

Year	Population	Growth Between Periods	Avg. Annual Growth Between Periods
2011	6,053	-	-
2016	6,664	611	2.02 %
2021	7,535	871	2.62 %

Also based on the Census data, the average household size is estimated at 2.1 in 2016 and in 2021. Based on this value and a count of individual dwellings using high-resolution aerial photography, the following populations are represented in the hydraulic model:

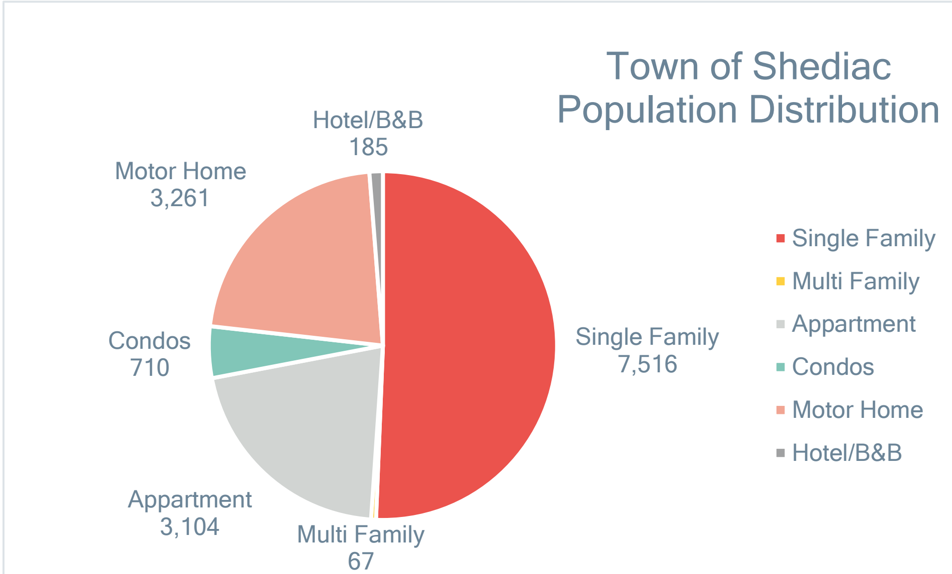


Figure 2-2: Population Distribution

The estimates for individual development types shown above result in an estimated population of 14,843, which is considerably higher than the Census population for the same area.
It is proposed this discrepancy is related, in large part, to the seasonal variations in population in the Shediac area. Many of the dwellings in the Town would be secondary residences and would not be captured in the Census of Population surveys. The hydraulic wastewater model must include the seasonal populations to better understand the impacts on the sanitary system the total population has.

3 Hydraulic Modelling

3.1 Methodology

3.1.1 General

The SewerCAD CONNECT edition model developed for this assignment was prepared to represent the hydraulic conditions of the entire wastewater system. Where available, flow conditions were adjusted to reflect the results from flow monitoring completed by Englobe for past projects and information available on the Commission's SCADA system.

The hydraulic model was used to identify portions of the Commission's collection system with capacity constraints and heightened risks associated with surcharging. Furthermore, the anticipated impacts associated with future development in the Town were reviewed to identify likely bottlenecks.

3.1.2 Model Construction

The shapefiles extracted from Englobe's GIS model of the GSSC wastewater collection system were used directly in the construction of the sewer system model using the ModelBuilder tool-kit available in SewerCAD Connect edition. This tool-kit allows for rapid construction of the physical model by using geometry from the shapefiles provided and information such as pipe material and diameter. Where data was not available, assumptions were made based on adjacent infrastructure. See Section 2.2.6 for more information.

Once imported into the model, additional manholes were added at the dead-end piping. This is because all piping needs to be connected to two structures in the SewerCAD Connect edition.

The hydraulic sanitary model was developed as a "Steady State" configuration, which aims to reflect peak flow conditions. This means that a single time step is used for the analysis, where an extended period simulation (EPS) model evaluates each step over a predetermined time period.

3.1.3 Flows - Existing Conditions

The SewerCAD model was populated with estimated flows according to published values and Englobe's experience with flow monitoring studies of similar areas. Flow within a sanitary sewer system is generally comprised of the following components:

- **Dry Weather flows**: This component of flow represents those flows that are generated by rate payers through domestic, commercial, industrial, and institutional processes. These flows are generally independent of precipitation.
- **Inflow and Infiltration**: This component of flow is derived from precipitation (rainfall or snowmelt) which enters the system through a variety of avenues including pipe and manhole joints, manhole covers, direct connections (catch basins or ditch leads), downspout connections, sump pumps, etc. Inflow and Infiltration can vary substantially from system to system and even area to area within the same system.
- **Pumped flows**: This component of flow aims to capture the impact of pumping facilities on the flow dynamics of a system. Due to the nature of a typical wastewater pumping facility, the flow downstream of the station is driven by the pumping capacity and not necessarily the flow entering that pumping facility. This can result in a potential for higher peak flows than if those same areas were connected with a gravity sewer system.

Please see the following sections for further discussion.

3.1.3.1 Dry weather Flows

3.1.3.1.1 RESIDENTIAL LOADING

The model was populated with dry weather (sanitary) loading by utilizing aerial imagery (2022) to create a node for individual dwellings in GIS. These nodes were then imported directly into the model using SewerCAD's "load builder" function to assign the flows to the nearest manhole.

Initial average loading for residential development was set at 340 l/person/day in accordance with the Atlantic Canada Wastewater Guidelines (2006). However, initial calculations necessitated an increase of this value to more closely represent the flows as provided in the GSSC SCADA. As a result, residential loading was adjusted to be **380L/person/day**, which aligns with the Atlantic Canada Wastewater Guidelines 2022 edition.

3.1.3.1.2 COMMERCIAL AND INDUSTRIAL LOADING

Commercial areas were assigned loading using an initial load/area allowance of 35 m^3/ha (Atlantic Canada Wastewater Guidelines). This theoretical value was found to significantly over-represent the flows compared to recorded values. Therefore, to adjust the model to recorded values, the commercial and industrial loading rates were adjusted to 17 m^3/ha which is consistent with Englobe's experience with previously completed flow monitoring assignments in similar areas

Included in commercial areas are the following:

— Restaurants, cafés, bars;

— Shops, stores and boutiques;

— Auto repair;

— Office buildings and city hall ;

— Hairdressers, nail salons, etc.

The commercial loading rate might be higher for some of these types of loads, but it is important to consider that the property can be purchased and replaced by higher load types of businesses.

For all other types of loading including churches, institutional flows, day-cares, gyms, etc. Please see the full table below.

Table 3-1: Additional Loading Rates

Type	Loading	Source	Comment
Public Washroom and or Shower	Between 25,000 to 80,0000 L/day	Assumption	Assumptions made based on experience, number of toilets and showers and location and likelihood to be used.
School	115 L/pers/day	ACWWG	Schools were contacted to estimate the student population.
Church	4 * 380 L/pers/day	Assumption	ACWWG provides a flow of 25 or 35 (if kitchen) L/day per seat. However, this was considered to be over estimated based on the number of people who use the facilities and who attend churches.
Daycare	115 L/day/child	ACWWG	Daycares were contacted to estimate the sanitary flows.
Car Wash	43,600 L/day	Assumption	Based on max flow through water service and number of wash bays.
Dental Clinic	760 L/seat + 75 per staff	NBTG[1]	Based on number of seats and staff.

Type	Loading	Source	Comment
Correctional Facilities	136 L/inmate/day + 23 L/staff/day	NBTG[1]	Estimate is based on maximum number of inmates and an estimated quantity for the staff.
Splash Pad	272,400 L/day	Supplier Technical	Based on previous projects similar in size.

[1]New Brunswick Technical Guidelines for On-Site Sewage Disposal

3.1.3.1.3 PEAK FLOW ESTIMATION

Referring to the Atlantic Canada Wastewater Guidelines Manual, the peak flow and the peak factor are based on the Harmon formula as follows:

$$\mathrm{Q(d)} = \frac{\mathrm{pqM}}{86{,}4} + \mathrm{Infiltration} \qquad M = 1 + \frac{14}{4 + p^{0,5}} > 2.0$$

Q(d) : Peak Domestic Flow (L/s)
p : Maximum Population in Thousands
q : Average Domestic Flow (L/person)
M : Peak Factor (calculated with Harman Formula)

With the help of the Harmon formula, a table was developed to correlate total flow with a peaking factor. This table was then added to a tool called “Extreme Flows” as part of the SewerCAD CONNECT edition. The tool allows for peaking factor to be applied to each manhole based on the total upstream loading. Therefore, the peaking factor is variable and is adjusted based on the applicable loading.

3.1.3.2 Inflow and Infiltration

Inflow and Infiltration were initially assumed to be 0.36 m^3/cm of pipe diameter/km of pipe/day. The Atlantic Canada Wastewater Guidelines recommend infiltration to be between 0.24-0.48 m^3/cm of pipe diameter/km of pipe/day. However, based on previous flow monitoring studies completed in the Town of Shediac and other similar municipalities, this value can vary substantially from one community to another and even between sewersheds in the same community.

3.1.3.3 Impact of Lift Stations

Lift station pump capacities (single pump) were provided from GSSC for each of the stations that do not have a flow meter. These were approximated by performing drawdown tests (1 pump) for each station with the exception of LS #9, #14, #17 and the private lift stations. Both lift stations #9 and #14 will be upgraded in the coming months and #17 is not yet online. The one pump capacity for #9 was taken from a previous report for GSSC and LS #14 was approximated using the existing pump curve and other related available data.

For the two (2) pump capacities, the stations with flow meters were approximated by reviewing the highest recorded value by the flow meter over the past 3-5 years. These values were then compared to the one (1) pump capacity to calculate the pumping capacity ratio. An average of the ratios calculated from metered lift stations was taken to estimate the two (2) pump capacity of non-metered stations. This resulted in a value for two (2) pump capacity of 1.53 times the one (1) pump capacity.

It is worth noting the relationship between the capacity of a single pump running and two pumps running is not linear and involves multiple variables such as forcemain interior diameter, forcemain material, pump design capacities and much more. However, for the purpose of this study, this approach was deemed appropriate.

Table 3-2: Lift Station Pumping Capacities

Lift Station	Pumping Capacity (1 pump)	Pumping Capacity (2 pumps)	Receiving Sewershed
LS #1	8.1 L/s	12.4 L/s	LS #2
LS #2	22.8 L/s	32.9 L/s	LS #3
LS #3	32.9 L/s	47.5 L/s	Trunk Sewer
LS #4	38.0 L/s	71.5 L/s	Trunk Sewer
LS #5	6.6 L/s	10.1 L/s	LS #4
LS #6	22.2 L/s	28.8 L/s	Trunk Sewer
LS #7	12.4 L/s	18.9 L/s	LS #10
LS #8	8.8 L/s	13.4 L/s	LS #10
LS #9	6.3 L/s	9.7 L/s	Trunk Sewer
LS #10	34.6 L/s	49.7 L/s	Trunk Sewer
LS #11	8.5 L/s	16.5 L/s	Trunk Sewer
LS #12	5.5 L/s	8.4 L/s	Trunk Sewer
LS #13	9.6 L/s	14.7 L/s	LS #3
LS #14	12.6 L/s	19.2 L/s	LS #2
LS #15	6.3 L/s	9.7 L/s	LS #2
LS #16	4.4 L/s	6.8 L/s	Trunk Sewer
LS #17	1.6 L/s	2.4 L/s	LS #3
LS #18	4.6 L/s	7.1 L/s	LS #10
LS #19	41.1 L/s	62.8 L/s	WWTP
LS #20	29.1 L/s	44.5 L/s	LS #19
Priv. LS #1	1.6 L/s*	2.4 L/s	LS #14
Priv. LS #2		(unknown) 19 homes	LS #13
Priv. LS#3	0.8 L/s*	1.2 L/s	Trunk Sewer

*Information not available, therefore assumption has been made for pumping capacity.

Typically for similar wastewater system studies, if the hydraulic model is configured as steady state, lift stations are assumed to be pumping their max pumping capacities. This is a conservative approach as it provides a “worst-case scenario”. However, with all the available information on the GSSC SCADA, actual peak events were reviewed. Based on a discussion with GSSC, peak events are typically accompanied by the elevation of water in the WWTF wet well of about 4.5m (15 feet). Such recent events have occurred as follows:

1. Aug 10, 2019 - 5.07m (16.65 feet)
2. Sep 7, 2019 - 5.04m (16.55 feet)
3. Mar 26, 2021 - 4.50m (14.78 feet)
4. Feb 18, 2022 - 4.81m (15.78 feet)

A total of 4 significant flow events have occurred over 3 years. Based on the data, a significant event occurs approximately ± once per year. Therefore, the existing conditions scenarios is based on the February 18th, 2022 peak event as recommended by GSSC. This event is the most recent on record and resulted in overflows at multiple locations.

The pumps running and suspected overflow information in the table below is based on SCADA information for the pumps and wet well elevation. **Table 3-3** below provides a snapshot for each lift station during the significant flow event on February 18th, 2022.

Table 3-3: Storm Impacts on Lift Stations

Lift Station	Number of Pumps Running During Event	Pumping Capacity	Suspected Overflow
LS #1	2	12.4 L/s	No
LS #2	2	**32.9 L/s**	Yes
LS #3	2	**47.5 L/s**	Yes
LS #4	2	**71.5 L/s**	Yes
LS #5	2	**10.1 L/s**	Yes
LS #6	1	22.2 L/s	No
LS #7	1	18.9 L/s	No
LS #8	2	13.4 L/s	No
LS #9	2	9.7 L/s	No
LS #10	2	49.7 L/s	No
LS #11	1	16.5 L/s	No
LS #12	1	8.4 L/s	No
LS #13	1	14.7 L/s	No
LS #14	2	19.2 L/s	No
LS #15	1	6.3 L/s	No
LS #16	2	6.8 L/s	No[1]
LS #18	1	7.1 L/s	No
LS #19	1	62.8 L/s	No
LS #20	1	44.5 L/s	No

[1]While SCADA data appears to show an overflow during this event, the ditch downstream of the LS was blocked, resulting in water entering the wet well. This LS is not known to overflow.
[2]No SCADA Information for LS #17 and the private lift stations.

Typically, lift station pumping capacity is equivalent to the capacity of one (1) pump (for duplex configuration). However, to estimate peak flows in the model, individual pumping stations were reviewed to highlight stations with a history of two (2) pumps running during a significant flow event. Specifically, the February 18th, 2022 event was reviewed to highlight the window of time where several lift stations were operating with both pumps running simultaneously. This scenario was reviewed with the Commission and reflected in the model as the peak flow condition.

3.1.4 Model Calibration

Following the assembly of the theoretical flow estimates and adjustments described in the previous section, these flows were validated against available data representing observed system flows. While theoretical flow estimates are based on industry best-practices, there is significant variability between systems and it is prudent to adjust flow estimates when data is available.

The first step to calibrate the hydraulic sanitary model was to retrieve available information from the GSSC SCADA for each lift station such as:

- Metered Flows (where available);
- Pump operations (on, off, hours, etc.);
- Wet-well water elevations;
- Station information including:
  - Elevations for inlet and outlet piping;
  - Operational elevations (start, stops, alarms, etc.).Once the data was assembled, the following process was followed:

3.1.4.1 Dry-Weather Flows

While our review of hydraulic capacity was based entirely on peak flows throughout the collection system, dry-weather flows are generally better understood and typically have less variability from one system to the next when compared to wet-weather flows. By calibrating dry-weather flows, the modeller gains improved confidence in the model and a better understanding of the influence of inflow and infiltration on each sewershed. The process below was generally followed:

1. $Calibrated\ ADWF = Calibrated\ Residential\ Loading + Calibrated\ Non\ Residential\ Loading + Incoming\ LS\ Average\ Loading$
2. Calibrated residential loading was determined with the review of individual sewersheds to compare average dry-weather flow amounts vs. observed values. Data (SCADA flow meters, pump runtimes) was selected for apparent dry-weather periods to limit the influence of inflow and infiltration.
   a. Sewersheds with predominately single-family dwellings were reviewed first to calibrate the per-capita estimate.
   b. Any adjustments required to the residential loading rate are applied globally. Although it can result in some sewershed loadings being higher than the measured averages from the SCADA, the selected value should be representative of most of the sewersheds in the sanitary system.
3. Once the per capita estimate is established, sewersheds with significant non-residential developments can be evaluated to repeat step two (2) above.

4. Where a sewershed had a pumped contribution from an adjoining sewershed, the incoming LS Average Loading was assumed to be equivalent to the average loading of the contributing sewershed.

Average dry weather loads were then multiplied by the peaking factor as discussed in section 3.1.3.1.3 to calculate peak dry weather flows.

3.1.4.2 Wet-Weather Flows

Once the peak dry weather loading was finalised, the peak wet weather flows were evaluated. As mentioned above, the wet weather flows are highly variable from one sanitary system to the next. Where little information is available, theoretical I&I rates from the ACWWG are typically used to estimate peak wet weather flows.

However, considering that multiple I&I studies have been conducted in Shediac over the past few years, it is understood that the rates as presented in the ACWWG are too low and not representative of the observed conditions in Shediac.

Peak wet weather flows were approximated by using the following formulas:

$$Modeled\ PWWF = PDWF + Incoming\ LS\ Flows + I\&I$$

And

$$Modeled\ PWWF = Observed\ PWWF$$

Where PDWF and incoming LS flows were calibrated in the first step (see previous section), the calibrated I&I was adjusted until the Modelled PWWF was similar to the Observed PWWF.

A combination of LS SCADA data, flow monitoring studies (West Shediac), overflow records and flow data from the WWTF were used to establish the “Observed PWWF” for comparison with modelled PWWF. Please see Section 3.2.1 for results of the calibration exercise.

3.1.4.3 Lift Station Overflows

To account for flow that is “lost” through overflow events, an analysis was completed at three (3) lift stations which are known to have the most significant overflow frequency and rates.

The analysis reviewed wet well levels (from SCADA) during the known overflow on February 18th, 2022 compared with the elevation of the overflow pipe. The tide level data was downloaded from the Shediac Bay tide gauge to account for the tailwater effects of a high tide or storm surge during this event. The data was reviewed to identify the period with the most significant difference between wet-well and tide levels, resulting in the greatest flow out through the overflow.

Flow through the overflow pipe was then estimated by considering the pipe to act as a culvert, with the wet well level equalling the headwater depth and the tide governing the tailwater depth.

This process provided an estimate of peak overflow rates, which were then accounted for in the model by distributing that flow component as I&I throughout the contributing sewershed. As a result, the total flow in the model more accurately reflects observed conditions at each affected pumping station.

3.1.5 Future Development

Following the establishment of existing flow conditions through the model calibration process, scenarios were further developed to reflect anticipated flow conditions for future development.

While calibrated flow values were adjusted to reflect observed flow conditions, flows for future development were assigned in accordance with theoretical estimates from published sources. This approach is meant to reflect the uncertainties associated with development types and improvements to construction practices/standards (impacting I&I rates in new development areas).

The following sections provide additional discussion on the assumptions employed for future development areas.

3.1.5.1 Overall Approach

When evaluating the impacts of future development on GSSC’s collection system infrastructure, two (2) planning horizons were considered.

The following sections provide additional detail on those scenarios and how they were established.

3.1.5.1.1 PLANNED DEVELOPMENT

This scenario is meant to represent known prospective developments in the Town of Shediac, meaning they have the potential to occur in a short-term planning horizon.

To populate these prospective developments, a meeting was held between the Town of Shediac, The Commission, Englobe (sewer system consultant for GSSC), and EXP (water system consultant for the Town). A shared map was developed whereby individual properties were highlighted and assigned a development type and density. In the case of residential-type developments, the number of units was also established.

Please see Section 3.3 for more information.

3.1.5.1.2 LONG TERM PLANNING

This scenario was developed to review growth within a total planning horizon of 50 years (2072). Reflecting the uncertainty associated with Long Term Planning, no infrastructure recommendations were made to address any deficiencies that were identified through hydraulic modelling. However, those deficiencies are highlighted in this report and accompanying drawings to provide the Commission with a sense of where capacity could be constrained in the future.

Long Term Planning development was assumed to occur in undeveloped areas that were not specifically identified in the Planned Development scenario. An annual growth rate was applied to estimate the growth that could occur between the end of the Planned Development scenario and the end of the 50-year planning horizon (2072).

The following additional assumptions were made when estimating Long Term Planning growth:

- Non-residential development (commercial/industrial) growth is equal to residential growth;
- Growth is based on the estimated seasonal population (# of dwellings at 2.1 people per dwelling)

Please refer to Section 3.4 for more information.

3.1.5.2 Dry-Weather Loads

Future dry-weather flows follow the same methodology as the calibrated existing loading as detailed in section 3.2.1:

- Planned developments (Residential)
  - Average dry weather load = 380 L/pers/day;
  - Average household size = 2.1 pers/unit;
- Residential Density (Long Term Planning) = 45 person/hectare;
- Average commercial/industrial load = 17 m^3/hec/day

The peaking factor will also follow the Harmon factor as seen in section 3.1.3.1.3.

3.1.5.3 Inflow and Infiltration

The inflow and infiltration of the existing loading is based on applying an I&I rate to the length of pipe in km. Since future street configurations are unknown, pipe lengths/sizes are also unknown. Therefore, an area loading factor is applied to the development area to account for potential I&I.

As found in the Atlantic Canada Wastewater Guidelines, the area allowance ranges from 0.14 to 0.28 L/sec per gross hectare. For the purpose of the future loading analysis, a factor of **0.21 L/sec** per gross hectare was used as it is the average value of the range.

3.1.5.4 Lift Station Upgrades

For the purposes of this scenario, it was assumed that the lift station would be upgraded to accommodate the current pumping capacity, plus flows for planned development including I&I. This approach is meant to respect the Canada-wide Strategy for the Management of Municipal Wastewater Effluent (CCME) for Combined and Sanitary Sewer Overflows whereby:

- No increase in combined sewer/sanitary sewer overflow frequency <u>due to development or redevelopment</u>, unless it occurs as part of an approved combined sewer overflow management plan;
- No combined sewer/sanitary sewer overflow discharge during dry weather, except during spring thaw and emergencies; and
- Removal of floatable materials where feasible.

The pumped flow component in a receiving sewershed was increased according to the approach described above. This means no additional capacity was assumed at lift stations to accommodate existing I&I conditions. Therefore, to reduce overflows at these lift stations, a continued focus on I&I reduction would be required.

If lift station capacity upgrades are considered which go beyond the assumed flows described in Section 3.3.3, downstream impacts should be reviewed in detail.

3.1.5.5 WWTF Upgrades

When evaluating the system performance during Future Development scenarios, it was assumed the planned upgrades to the Cap Brule WWTF will have been completed. This is expected to significantly impact the hydraulic grade line (HGL) conditions in the trunk sewer, as a significant increase in pumping capacity is planned.

Therefore, for the Future Development scenarios, the assumed wet-well level (tailwater conditions) was updated as follows:

- Existing tailwater elevation: 1.85 m
- Upgraded tailwater elevation: - 1.55 m

3.1.6 Hydraulic System Analysis

The computed hydraulic grade-line in a model such as SewerCAD is a representation of the water level in a piped system. An HGL which is calculated to be above the top of a pipe is indicative of a pipe which has insufficient hydraulic capacity and is in a surcharged condition. A surcharged pipe segment can be a risk factor for flooding, as water is more likely to back-up into basements.

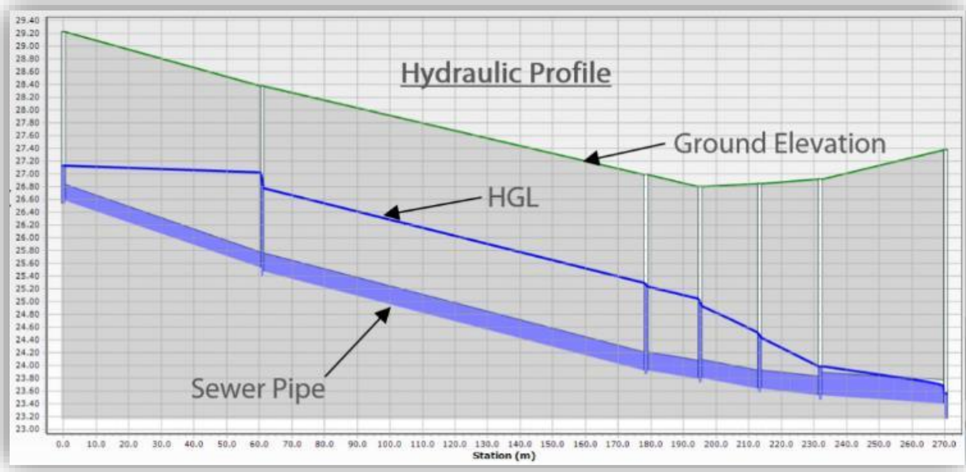


Figure 3-1: Hydraulic Profile Example

To quickly identify inadequate pipe segments, colour coding was applied in the SewerCAD model to identify the risk level posed by the calculated hydraulic grade-line. The colour coding was prepared as follows:

— HGL ≤ 100% of pipe dia.	Green
— HGL ≤ 100% of pipe dia. & less than 2.2m from ground level	Yellow
— HGL > 100% ≤ 130% of pipe dia.	Orange
— HGL > 130% of pipe dia. & HGL > 2.2m from ground level	Blue
— HGL > 130% of pipe dia. & HGL < 2.2m from ground level	Red

Ideally, the piping would be green throughout the system. Yellow indicates that the HGL is likely not resulting in sewer backups, but there is a potential that issues will arise if conditions worsen. Black, blue and red indicate increased risk factors associated with hydraulic surcharging.

For the purposes of this study, pipe segments which were found to fall in the "**Red**" category were reviewed in detail and were listed as deficiencies.

3.1.7 Cost Estimates

This section presents a summary of the improvement cost estimates for the entire study area. To assist the City in the prioritization and budgeting of capital works, preliminary cost estimates were completed for each upgrade presented in this report. A summary of the cost estimates for the required upgrades to the various sewer systems is presented in Section 4 of this report.

The preliminary cost estimates are based on preliminary designs for recommended sewer upgrades presented in subsequent Sections of this report. The estimates are based on anticipated 2023 construction costs and Englobe's experience on 2022 projects in the Town of Shediac; as a result, these cost estimates will need to be updated to reflect construction cost based on the year the work is to be completed in. Each estimate includes construction costs, engineering services (15%), construction contingencies (15%), and the current 2022 HST rate of 15%.

For this exercise, it was assumed that any pipe upgrades will be sized assuming a 0.5% pipe slope. Also, pipes are designed at approximately 80% full capacity as per typical design standards.

3.2 Existing Conditions

A scenario was established in the SewerCAD model to represent existing conditions, calibrate existing flows and understand current capacity constraints.

3.2.1 Model Calibration Results

The hydraulic sewer model was developed and calibrated by comparing two different values:

— **Calibrated values** were determined by applying the assumptions as explained in section 3.1.4 above.

— **Observed values** were determined based on information taken from the SCADA either from the lift station flow meter data (where available) or through an analysis of pumping hours and wet well elevation values.

3.2.1.1 Average Dry Weather Flows

Calibrating average dry weather flow values aims to validate assumptions associated with system connectivity and overall wastewater flow generation rates. Due to variations in non-residential flow rates, lack of metered flow data in all sewersheds, and assumptions associated with pumped flows, achieving alignment throughout the system can be challenging.

Where this study aims to highlight deficiencies associated with peak flow conditions, the relative importance of ADWF calibration is reduced. However, results were generally found to be conservative (higher) by between 0 - 40% on average.

Table 3-4: Average Flow Calibration Values

Sewershed	Calibrated Avg (L/s)	Recorded Avg (L/s)	% Difference
LS #1	0.81	1.10	30%
LS #2	9.63	6.31	42%
LS #3	18.12	11.99	41%
LS #4	8.99	8.83	2%
LS #5	3.72	2.44	42%
LS #6	1.94	2.20	13%
LS #7	0.81	0.67	19%
LS #8	0.46	0.42	9%
LS #9	0.30	0.19	45%
LS #10	6.62	6.18	7%
LS #11	0.68	0.69	1%
LS #12	0.27	0.15	57%
LS #13	0.45	2.66	142%

Sewershed	Calibrated Avg (L/s)	Recorded Avg (L/s)	% Difference
LS #14	2.08	2.02	3%
LS #15	0.11	0.12	9%
LS #16	0.08	0.14	55%
LS #18	0.12	0.63	136%
LS #19	1.75	1.23	35%
LS #20	2.35	0.98	82%

While several locations are noted to have a significant discrepancy between recorded and calibrated average dry weather flow (ADWF), these values were deemed acceptable for the purposes of this study for the following reasons:

- Calibrated values are derived from the model, which considers pumped flows from upstream sewersheds. Where this study used a steady-state model, the impact of variable flows over time is not accurately reflected.
- Many of the recorded values used for calibration were based on pump run times where no flow meter data was available. Deriving average flows from pump runtime data is an imprecise method, particularly for the analysis of average daily flows.
- The focus of this study is an evaluation of system capacity with respect to peak flows. Therefore, more emphasis was placed on validating peak flows to individual sewersheds.

Please see the following sections for more information on the results of peak flow calibration.

3.2.1.2 Peak Wet Weather Flows

Based on the methodology as explained in section 3.1.4, PWWF is calibrated by adjusting I&I values until the calibrated PWWF is equivalent to the recorded PWWF. Calibrating through the I&I flows allows the additional loading to be applied to the entire sewershed instead of a point load. However, if the additional I&I is coming from a single source, the methodology may be inconsistent with actual conditions. Based on the calibrated values of I&I rates and previous I&I studies conducted in the Shediac wastewater system, it is clear that the Town of Shediac currently has a significant I&I wastewater component. The following formula was used to evaluate the I&I impacts to each of the lift stations.

$$\%\ I\&I\ Flow = \frac{I\&I\ (L)}{I\&I\ (L) + PDWF\ (L)}$$

The graph below summarizes the comparison between total PWWF and the calibrated I&I component of the flow to each sewershed. This analysis is not meant to be used to highlight problematic sewersheds for the purpose of I&I reduction. However, it provides an overall indication of the prevalence of I&I in the system and how substantial those flows are to the hydraulic conditions in the modelled results.

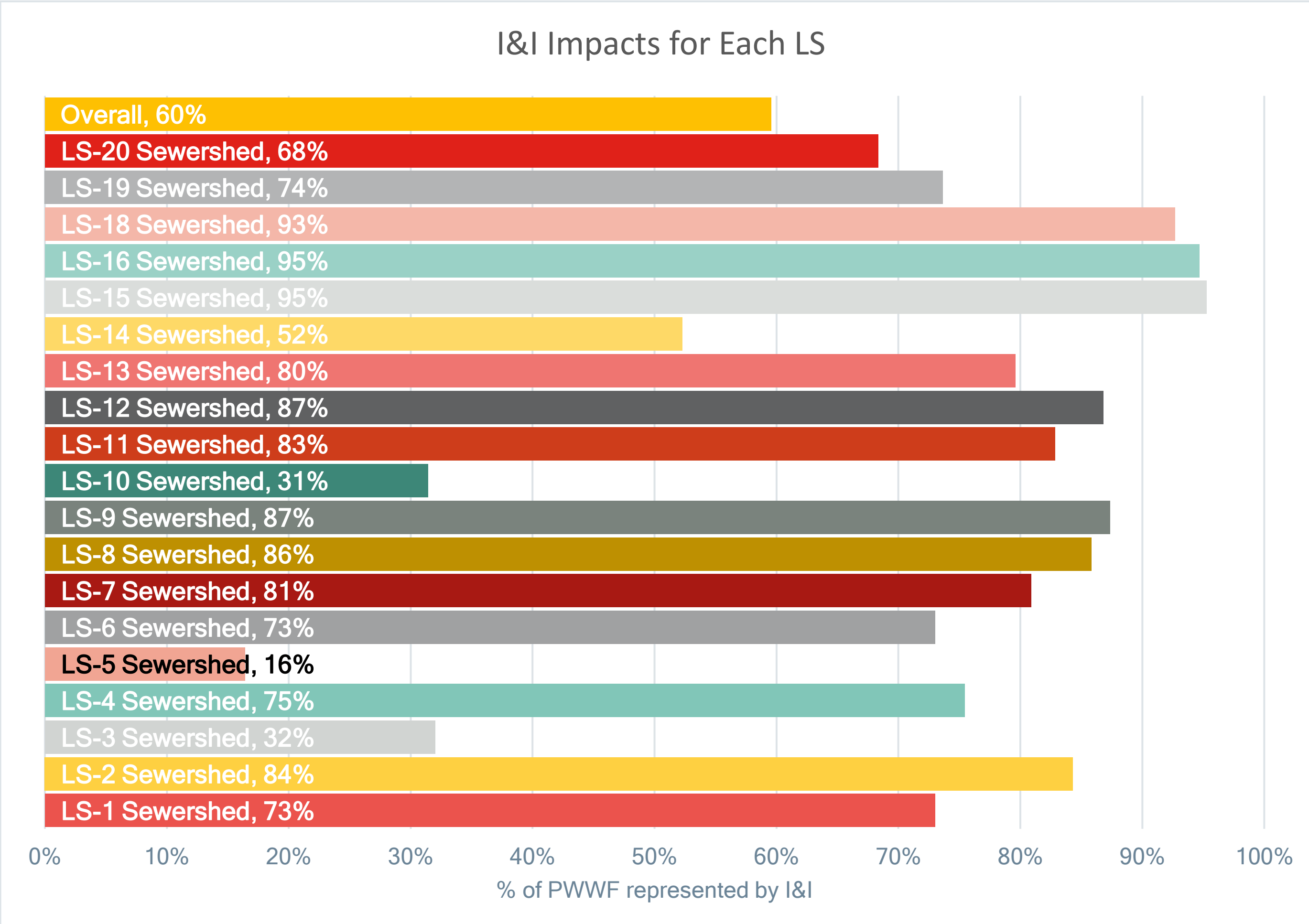


Figure 3-2:I&I Impacts for Each LS

Overall, infiltration and inflow represent approximately 60% of the total peak flow for the entire system. This value is based on an assessment of available data from individual sewersheds and not necessarily metered data. There are indications from available data at the WWTF this proportion could be even higher during peak wet-weather events.

To illustrate this, Figure 3-3 below provides a snapshot view of the wet well elevation and the pumping capacity at the WWTF screw pumps From February to the end of March 2020. The graph in the back is the screw pump wet well elevation and in orange is the screw pump flow meter.

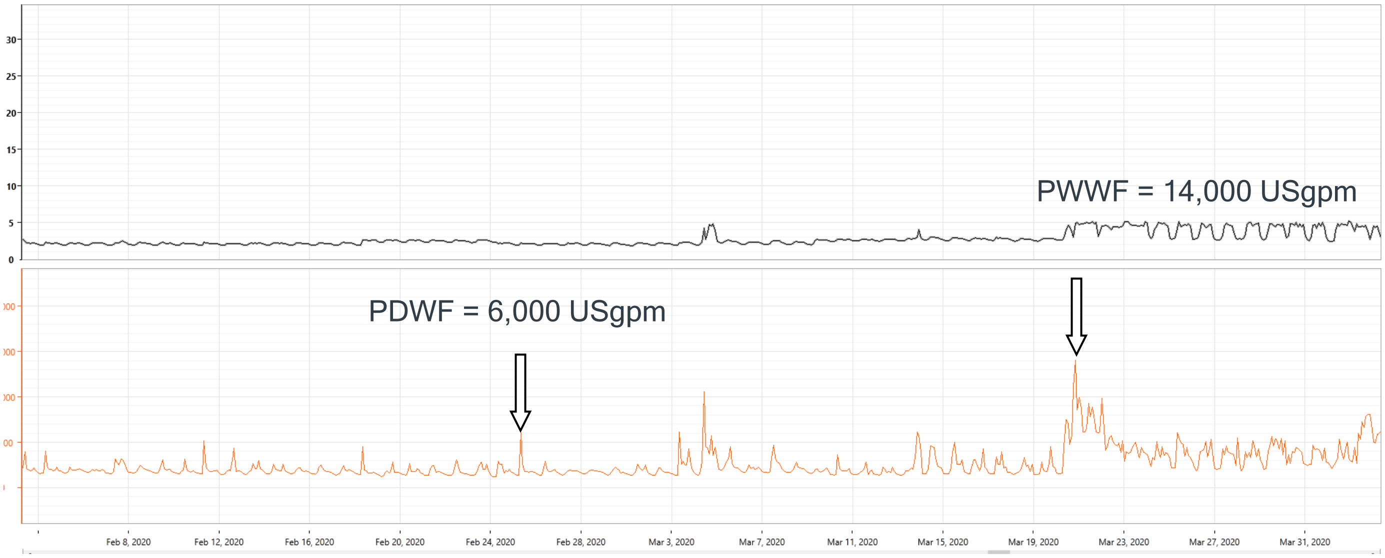


Figure 3-3: Recorded I&I Impact

Looking at the flow meter data, the flow follows a typical dry period diurnal curve with a few localized peaks. The average flow is roughly 2,500 USgpm and the highest peak is at approximately 14,000 USgpm. The peak value takes place on March 20th, 2020 which would line up with the spring snow melt. The peak event recorded flow is about 5.5 times larger than the average dry weather flow and 2.25 larger than the PDWF. This also translates to the I&I portion of the flows being around 57% of the total flow which provides some validation for the modelled scenario.

A separate analysis based on Cap-Brule Wastewater Treatment Facility preliminary design report (project # 18411) was also reviewed. In this report, the received peak flow at the WWTF in 2018 was estimated to be 383 L/s. The calibrated flows for the existing conditions scenario are estimated at 474 L/s. This represents a 21% increase over the past four (4) years. Considering the development rate that Shediac has seen in recent years, we consider the percent difference is within an acceptable range for this type of study.

Please note that the flow meter data is not considered as accurate by GSSC and Englobe. 14,000 USgpm is equivalent to 883 L/s which is almost 2.5 times larger than the estimated flow in 2018. Although the values appear to be inaccurate, the error is assumed to be relative meaning that when we compare the PDWF with the PWWF values, the difference between them is assumed to be accurate.

Table 3-5 below provides an analysis which compares the calibrated flows with the theoretical PWWF (before calibration) using values from the Atlantic Canada Wastewater Guidelines. If a new collection system were being designed, it is likely it would be designed using these theoretical flow values. Therefore, the table below indicates how the flow conditions in Shediac compare with standard values for similar development.

Table 3-5: Theoretical PWWF vs Actual PWWF

Sewershed	ACWWG Theoretical PWWF (L/s)	Calibrated PWWF (L/s)	% Difference
LS #1	3.59	12.34	344%
LS #2	70.17	185.38	264%
LS #3	82.48	92.52	112%
LS #4	38.46	108.28	282%

Sewershed	ACWWG Theoretical PWWF (L/s)	Calibrated PWWF (L/s)	% Difference
LS #5	15.48	17.13	111%
LS #6	8.06	22.31	277%
LS #7	3.47	17.41	502%
LS #8	2.89	13.42	464%
LS #9	1.29	9.65	748%
LS #10	52.72	59.71	113%
LS #11	2.94	16.38	557%
LS #12	1.2	8.38	698%
LS #13	5.97	10.15	170%
LS #14	10.31	19.25	187%
LS #15	0.52	9.63	1852%
LS #16	0.4	1.13	283%
LS #18	1.09	7.03	645%
LS #19	33.6	41.05	122%
LS #20	9.99	29.02	290%

3.2.2 System Performance

The SewerCAD model was run in an existing conditions scenario, to identify current capacity constraints and areas at risk due to surcharged conditions.

Due to the preliminary nature of the findings presented herein, the results should be confirmed through a more detailed analysis of individual deficiencies as part of preliminary design activities for an associated project.

3.2.2.1 Gravity Sewer

The gravity sanitary sewer generally functions adequately during current flow conditions except for the sections below. **Map No.3-1** in **Appendix A** provides an overview of the gravity system during the existing conditions scenario.

3.2.2.1.1 TRUNK SEWER

Much of the sanitary trunk sewer is presenting an elevated surcharging risk during a peak flow scenario (existing conditions. This indicates a higher likelihood of issues such as surcharging manholes and piping and backing into the many lateral system branches. Although private lateral connections to the trunk sewer are limited, the hydraulic grade line in the trunk sewer impacts the hydraulics of connected system branches.

The apparent reasons for these issues are as follows:

1. The relatively low slope along the entire length;
2. During peak events, the WWTF creates a high tailwater effect which in turn surcharges a significant portion of the trunk sewer;
3. During peak events, the trunk sewer is above capacity due to the substantial I&I volume being discharged from the many sewersheds and lift stations.

Figure 3-4: Existing Conditions -Trunk sewer piping from Weldon Street to the WWTF

The following figures present the hydraulic grade line (HGL) profiles from the top of the trunk sewer near Weldon St., down to the WWTF at Cap-Brule. As seen in these profiles, the predicted HGL is near or above the ground surface along much of its length, indicating the potential for the system to surcharge onto the surface during peak flow events. This was validated with observations from the Commission during peak wet-weather events.

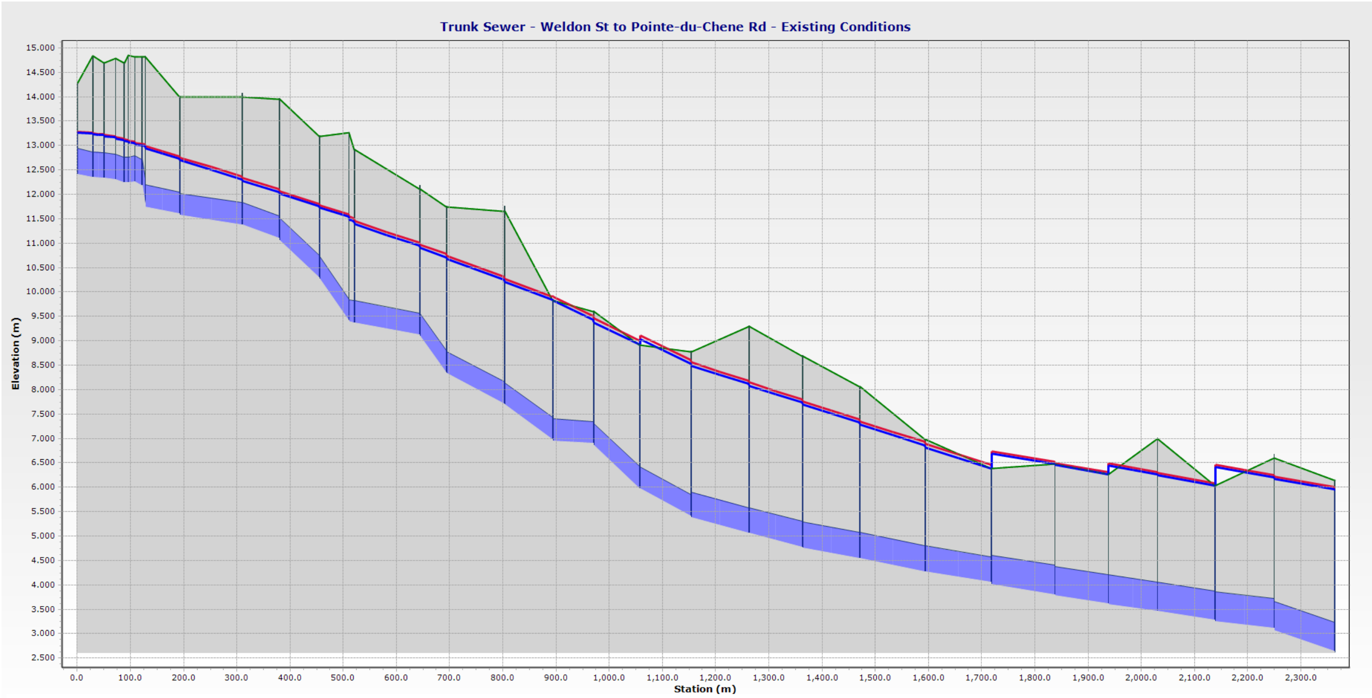


Figure 3-5: Existing Conditions -Trunk Sewer HGL Weldon St to Pointe-du-Chene Rd

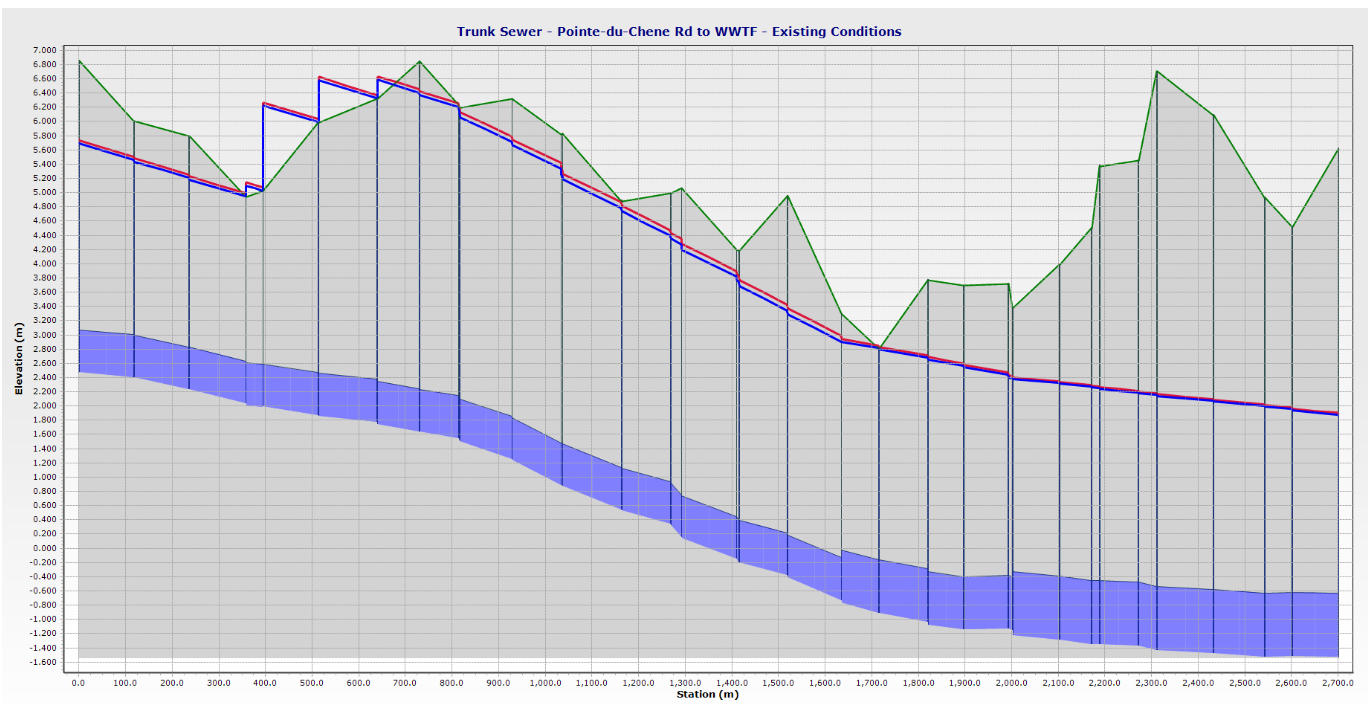


Figure 3-6: Existing conditions - Trunk Sewer HGL Pointe-du-Chene Rd to WWTF

In reviewing the design documentation for the trunk sewer, it was clear there was an understanding at the time that I&I reduction was an integral and critical element in the overall system planning. There was a recognition in the design documentation that it was not practical to size the trunk sewer to accommodate peak wet weather events and the selected design flow (8.0 MGD) assumed a certain reduction to I&I flow during the design life of the infrastructure. Based on calibrated model flows in the trunk sewer, the modelled flows (9.8 MGD) are significantly higher than those used in the design of this trunk sewer section. The intended reduction to I&I has not yet been achieved.

3.2.2.1.2 BACKLOT GRAVITY SEWER NEAR GREENWOOD PROMENADE

Although not identified as having HGL issues, the sanitary piping identified in the figure below is known as problematic by GSSC and has even overflowed in the past into a nearby resident's backyard.

Figure 3-8:Deficient Manhole

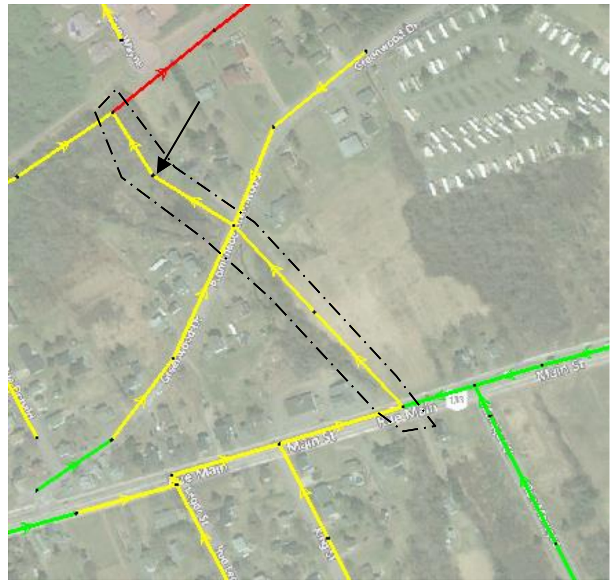


Figure 3-7:MH 0137 Location

Sanitary manhole MH0137 located between Greenwood Promenade and the sanitary trunk sewer is relatively short where the top of pipes are near the manhole frame and cover. This manhole is also in poor condition as seen in **Figure 3-8** where roots are growing through the structure and there are large gaps open to the environment. This manhole has been identified as surcharging in the hydraulic model which if true, could result in a significant issue where sanitary flow is discharged on the nearby private lots. It is recommended to immediately repair this structure and install a lockdown watertight frame and cover.

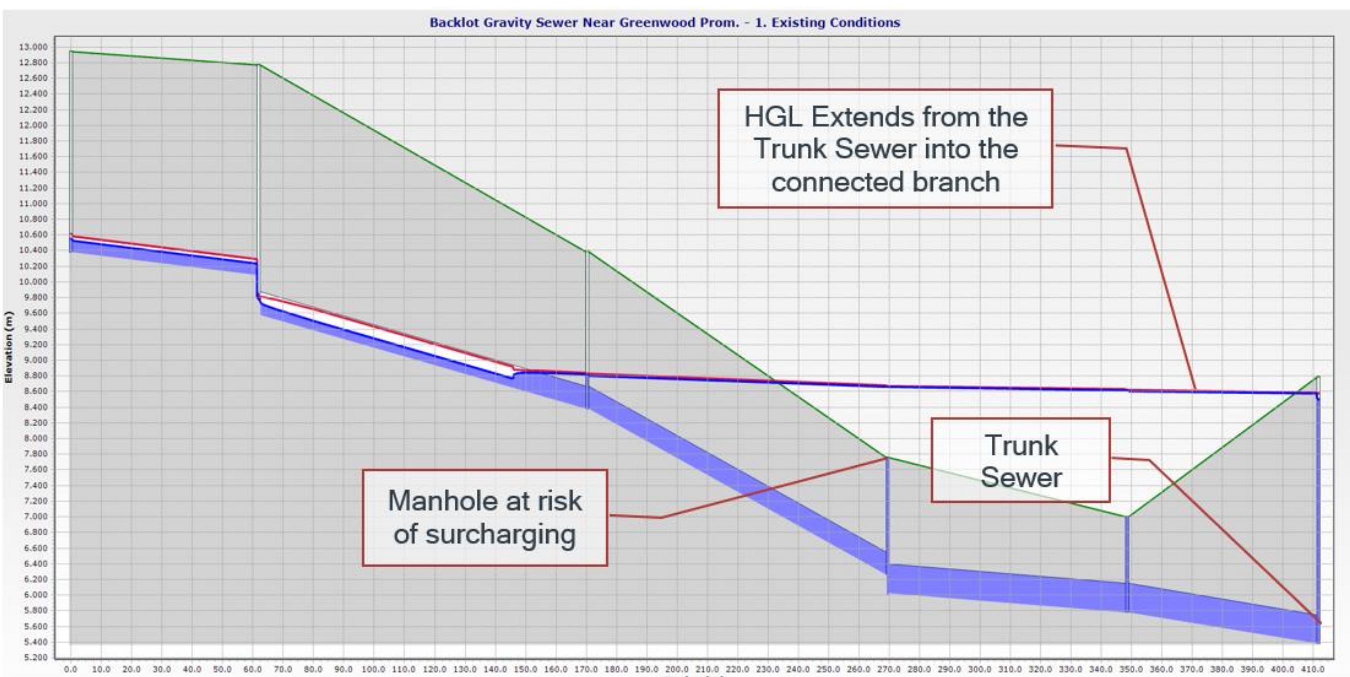


Figure 3-9: Backlot Gravity Sewer - Profile

There appears to be a surcharging risk in this section associated with the hydraulic conditions of the trunk sewer immediately downstream. The manhole immediately upstream of the trunk sewer appears to be significantly lower (rim elevation) than the modelled hydraulic grade line in the trunk sewer. In this case, while the pipe itself appears to have sufficient capacity, there is the potential for this manhole to overflow.

3.2.2.1.3 MAIN STREET AND DOCK STREET

The highlighted sections along Main St. and Dock St. are undersized for peak flow conditions and pose an elevated risk of sewer surcharging in these sections and connected systems.

The potential reasons for these issues are as follows:

1. Over 50% of the length in red does not meet the minimum slope as recommended by ACWWA;
2. High I&I rates combined with pumped flows from LS 2 result in peak flows in excess of the pipe capacity;

Figure 3-10: Existing Conditions – West Main and Dock Street

3.2.2.1.4 MAIN STREET EAST

The highlighted red sections are undersized and could lead to increased surcharging risk during a peak flow event.

The potential reasons for these issues are as follows:

1. The diameter of the sanitary piping in this area decreases on Main street (see circled sections) which can lead to capacity issues;
2. The existing piping on Main street is undersized based on the calibrated flows;

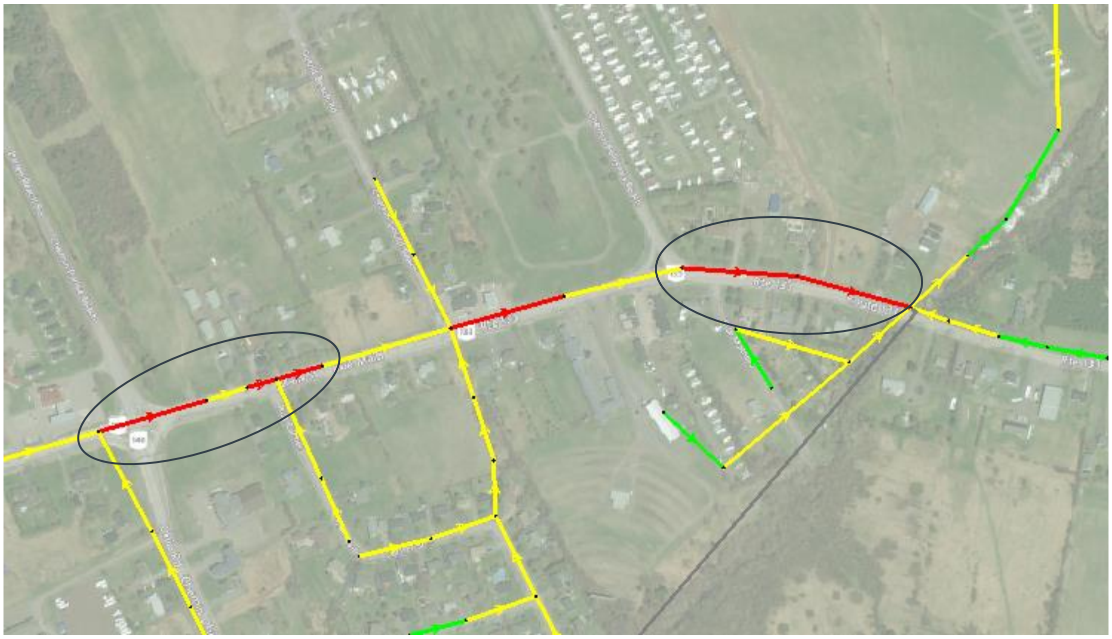


Figure 3-11: Existing Conditions - Main Street East

3.2.2.1.5 MAIN STREET AND CASSIE AVENUE

The highlighted sections along Main St. near Caissie Ave are undersized for peak flow conditions and pose an elevated risk of sewer surcharging in these sections and connected systems

The potential reasons for these issues are as follows:

1. The piping sections meet the recommended minimum slope based on a pipe diameter of 200mm (0.4%)
2. The existing piping is undersized based on the calibrated flows;

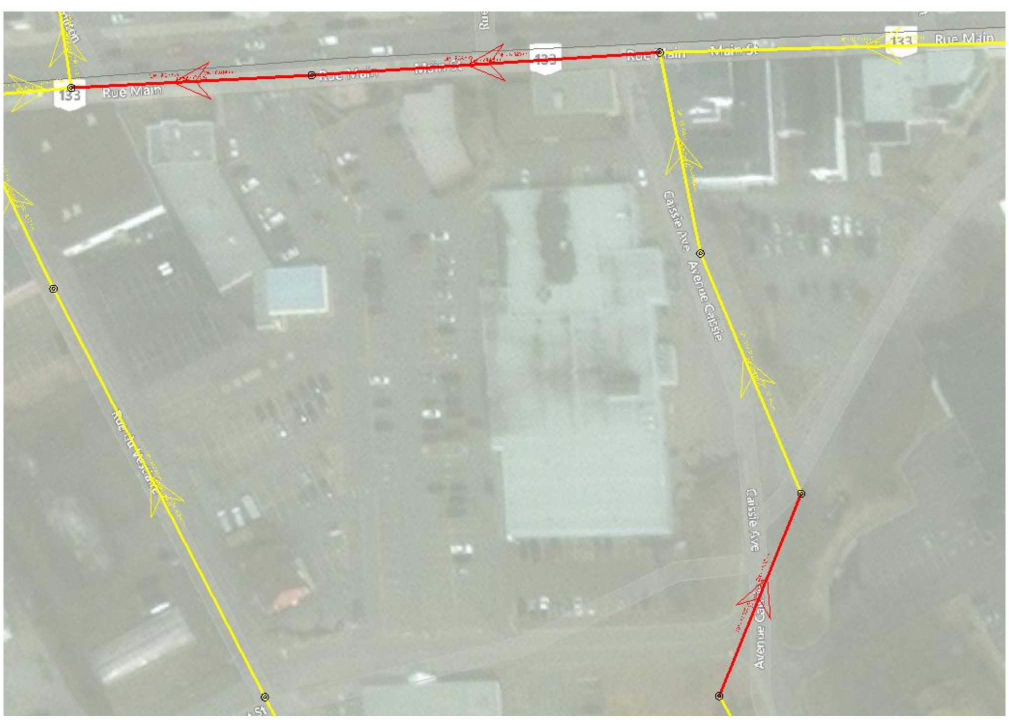


Figure 3-12: Existing Conditions - Main and Cassie Avenue

Pipe diameter upgrades are recommended for this issue.

3.2.2.1.6 OTHER AREAS

Rue Saint John

A single pipe section is suspected to be over capacity due to received flows from LS 18 combined with the gravity flow and the pipe slope not meeting the recommended minimum slope of 0.4% for 200mm piping (0.23%). Further investigation is recommended to assess the severity of this issue during peak events. No upgrades are recommended at this time.

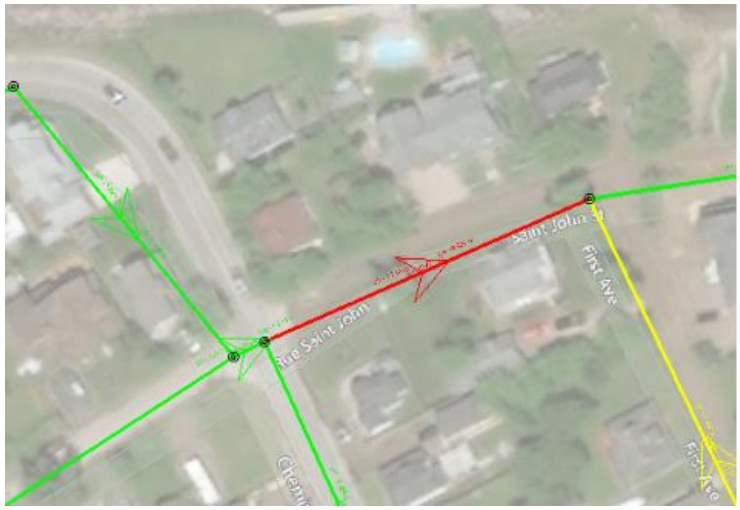


Figure 3-13: Existing Conditions - Saint-John and Pointe du Chene Intersection

Shore Drive

A single pipe section is over capacity due to the flows received from LS #5. Considering that Pleasant Street is scheduled for upgrades in the next couple of years, where LS #5 will now be discharged directly into the trunk sewer main, based on the hydraulic model, this issue will be resolved.

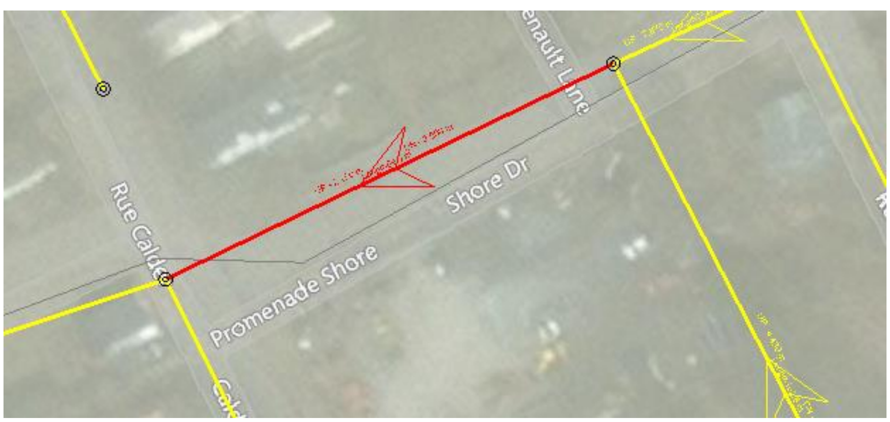


Figure 3-14: Existing Conditions - Calder and Promenade Intersection

Evergreen Road

The model highlights a single pipe section on Evergreen Road as a surcharging risk. However, as seen on the profile view below (Figure 3-16), the pipe is not surcharged. It is highlighted because the pipe is less than 2.2m deep and is over capacity due to its flat slope. Based on the piping network layout, it is unlikely that any sanitary laterals would be connected to this pipe section. Therefore, this section was not considered for improvement.

Figure 3-15: Existing Conditions - Evergreen Road

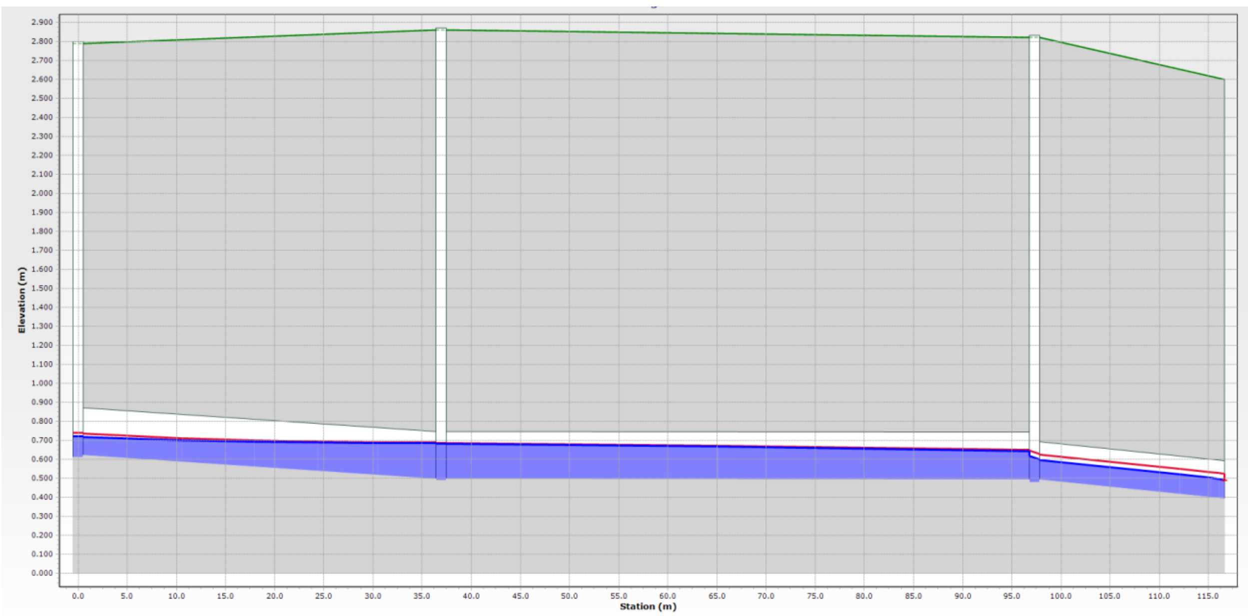


Figure 3-16: Existing Conditions - Evergreen Rd Deficient Pipe HGL

3.2.2.2 Lift Stations

Typically, a duplex lift station is designed for each pump to be sized to pump the total expected flow received at the lift station. Assuming this methodology was used for the design of each lift station in Shediac, as found in Table 3-3, the following stations could be considered deficient:

- LS #1
- LS #2
- LS #3
- LS #4
- LS #5
- LS #8
- LS #9
- LS #10
- LS #14
- LS #16

However, looking at the pumping hours for each lift station listed above with exception to Lift Stations 2 through 5, it appears that the stations only have two (2) pumps running for relatively short periods and only for larger peak events. Further analysis of lift stations #2 to #5 can be found below.

3.2.2.2.1 OVERFLOWS

GSSC provides overflow reports for each overflow event since 2018 on their website. A total of 24 such reports have been submitted as of October 2022. As noted in those reports, not all overflow events are due to surcharging in the sanitary system and may be related to operational interruptions such as municipal construction project issues (watermain and forcemain breaks, open sewer main during rain event) and others related to power outages as indicated in these reports.

With the help of the information available on the GSSC SCADA, validated with the GSSC overflow reports available online, it was determined that the following stations have overflowed at least once during or following a rain event in recent years:

- LS #1 - 2 Reports
- **LS #2 - 13 Reports**
- **LS #3 - 9 Reports**
- LS #4 - 3 Reports
- **LS #5 - 6 Reports**
- LS #7 - 1 Report
- LS #12 - 2 Reports
- LS #14 - 1 Report
- WWTP - 4 Reports
- UV Building - 2 Reports

Lift stations #2, #3 and #5 account for approximately 65% of the overflow reports. This count is only based on the overflow reports, and it is suspected there have been additional instances where the stations would have overflowed if the tide had not been high. Based on previous discussions with the Commission, it appears that some of the coastal lift stations sometimes have the wet well water elevation above the overflow pipe invert but do not overflow due to high tides. Depending on the ocean tide water elevation and the incoming flows to these stations, the combination of these events can lead to surcharging in the gravity system.

Figure 3-17 below was developed based on the February 18th, 2022 peak event as previously discussed.

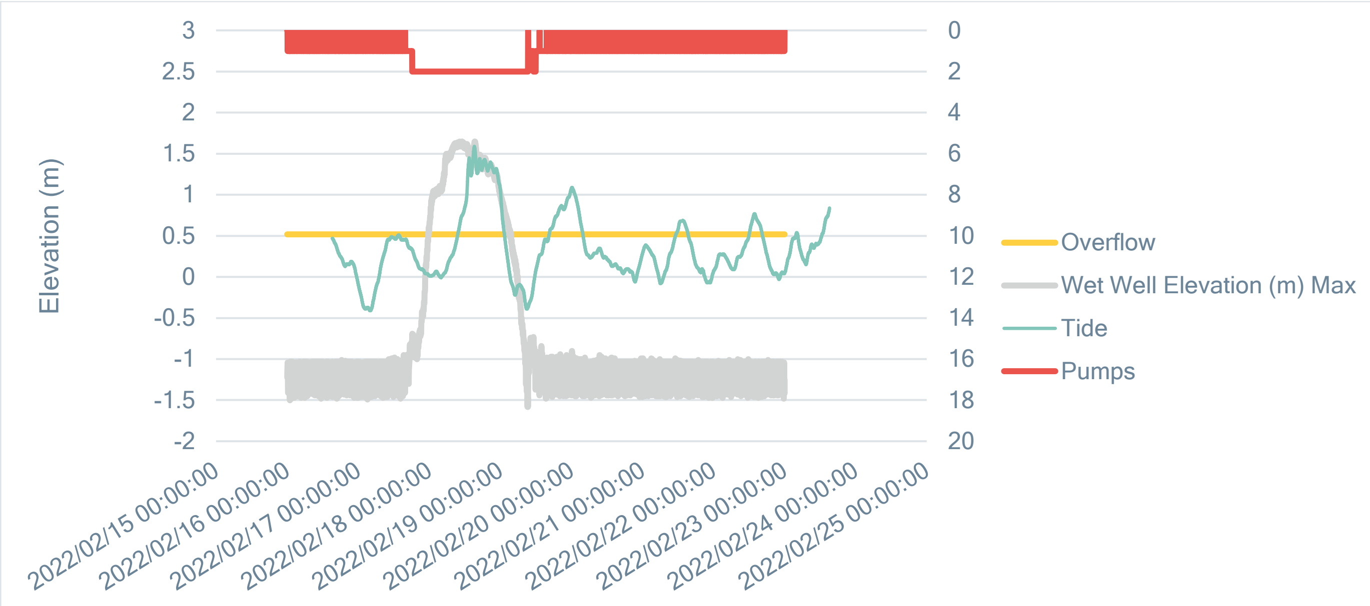


Figure 3-17: LS #2 Overflow Data

As seen in the graph above, during the February 18th, 2022 event the wet well level increased to an approximate elevation of 1.6m, resulting in a suspected overflow. However, the tide elevation also increased during the storm event, and you can see a direct correlation between the tide level and the wet-well level in the later part of the event. This suggests the tide partially blocked the overflow and the elevated hydraulic grade line duration was extended as a result.

The following table summarizes the information used to estimate the overflow rates at each lift station. The rates were estimated using the assumptions and methodology described in Section 3.1.4.3. These estimates are based on a review of a single overflow event (February 18, 2022), therefore they may not reflect worst-case conditions. The timing of each overflow event varied slightly between locations, which is reflected in the differing Tide Elevations used in the estimate.

Table 3-6: Overflow Conditions During February 18, 2022 event

Overflow Location	Wet Well Water Elev. (m)	Overflow Invert (m)	Overflow Size (mm)	Length (m) / Slope (%)	Tide Elev. (m)
LS #2 Wet Well	1.65	0.52	250	75 / 0.05	0.9
LS #2 Manhole	1.65	0.92	250	20 / 2.1	0.9
LS #3	1.24	0.8	200	30 / 0.33	0.8
LS #4	0.9	0.42	250	70 / 0.32	0.7
LS #5	0.72	0.61	200	8.5 / 3.3	0.5

The analysis provided an approximate flow that was discharged through the overflow pipe. Based on the SCADA information, each of these stations were pumping with both pumps during the suspected overflow events.

According to the analysis conducted for this study, it is believed that each lift station (except LS #2) current pumping capacity can pump the sanitary (dry weather) flow received with one pump only.

When reviewing SCADA data during a dry-weather period, there are cases where LS 2 requires two pumps running to keep up with the flows. Based on discussions with the Commission, the issue at LS 2 has since been resolved through upgrades completed in 2022. New data suggests LS 3 (downstream of LS 2) now requires two pumps more often during dry-weather periods.

3.3 Planned Development

A scenario was established in the SewerCAD model representing the Planned Development planning horizon. This scenario was used to identify infrastructure upgrades that may be required in the short term to service upcoming development.

3.3.1 Growth Areas

This map provides locations and development types for the planned development forecasted by The Town of Shediac and GSSC. The number of units provided was based on conversations and requests that have taken place between prospective developers and either the Town or the Commission. It represents the current understanding of where and what type of developments are likely to occur within the planning horizon.

Please refer to **Map 3-2 – Future Loading** in **Appendix A**.

The following table summarizes the planned developments by sewershed.

Table 3-7: Summary of Planned Development Areas

Sewershed	Type of Loading	Dev. Area (Hectares)	Equivalent Population	# of Units
LS #2	Res - Low	0.87	40	19
	Res - High	7.19	958	456
LS #3	Res - Med	0.27	97	46
	Res - High	4.26	956	455
LS #4	Res - High	1.03	229	109
LS #14	Res - High	0.64	32	15
LS #15	Res - High	2.50	408	194
Trunk	Commercial	39.20	1,754	-
	Institutional	5.32	714	-
	Res - Med	15.90	404	192
	Res - High	111.74	4723	2,249
Sub-Total - Residential		144.40	7,847	3,735
Sub-Total : Non-Residential		44.52	2,468	-
Total :		**188.92**	**10,315**	**3,735**

3.3.1.1 Planning Horizon

To gain a sense of the timelines associated with planned developments, growth rate scenarios were reviewed by Englobe. Growth in the Town has been very dynamic in recent years, so these timelines are meant to indicate how quickly these areas could be established for infrastructure investment planning.

As shown in the table above, the planned development areas represent an approximate population growth of 7,850 in the Town.

The following table summarizes the estimated development timelines associated with varying growth rate scenarios.

Table 3-8: Growth Rate Scenarios - Planned Developments

Growth Rate	Number of Years until Full
2%	36
2.62% (2021 census)	28
4%	19
6%	13
8%	10
10%	8
12%	7
14%	6
16%	5

To establish the Long-Term Planning horizon, a growth rate of 4% was assumed. This would result in Planned Development areas being fully developed in 19 years.

3.3.2 Flows

The values in the table below represent the estimated increase to peak flows for each sewershed due to planned development. Each development loading was assumed to be added to the nearest or most logical existing gravity sections.

Table 3-9: Additional Peak Flow - Planned Development

Sewershed	Additional PWWF (L/S)	Cumulative PWWF (L/s)
LS #2	18.36	26.77
LS #3	18.44	45.21
LS #4	4.33	4.33
LS #14	0.69	0.69
LS #15	7.72	7.72

Sewershed	Additional PWWF (L/S)	Cumulative PWWF (L/s)
Trunk	138.77	188.31
Total (WWTF)	**-**	**188.31**

Please refer to **Map 3-2 – Future Loading** in **Appendix A** for more information.

3.3.3 Lift Station Upgrades

As mentioned in section 3.1.5.4, the assumed lift station upgrades only account for additional flows associated with development (not necessarily the full estimated future PWWF for that sewershed). Table 3-10 below provides a summary of the assumed pumping rates for each Lift Station that were considered in the Planned Development scenario.

Table 3-10: Future LS Capacity - Planned Development

Sewershed	Existing Pumping Capacity (2 pumps) (L/s)	Contributing Sewersheds Additional Pumping Capacity (L/s)	Future Additional - PWWF (L/s)	Future Pumping Rate (1 Pump) (L/s)
LS #2	32.81	LS #14 - 3.23 LS #15 - 7.72	18.36	**62.12**
LS #3	47.32	LS #2 - 29.31	18.44	**95.07**
LS #4	71.50	-	4.33	**71.50**[3]
LS #14	19.24	-	0.65	**15.82**[2]
LS #15	9.7	-	7.72	**17.42**
LS #5	<u>10.09</u>	-	7.37[1]	**17.46**

[1]Although no planned development is expected for LS #5, Pleasant Street is expected to be upgraded within the next year. These upgrades redirect gravity sewer flow from a short section of Avenue Belliveau and Pleasant Street from the LS #4 sewershed to the LS #5 sewershed. Beginning in 2023, LS #5 will be redirected away from the LS #4 sewershed and towards the trunk sewer on Weldon St.

[2]The pumps for LS #14 are scheduled to be upgraded in the coming months as part of an ongoing project. The values shown in the table above reflect the planned pumping capacities.

[3]As discussed in Table 3-6 above, the total flow received during the peak event is estimated at 108.28 L/s. Where LS #5 will no longer discharge into the LS #4 sewershed and the gravity section of Pleasant Street is being redirected to LS #5, an estimated 20.41 L/s is removed from LS #4. Additionally, there is an estimated 4.33 L/s expected for LS #4 as part of the planned development. Therefore, 108.28 - 20.41 + 4.33 = 92.20 L/s. Since the projected net flow is less than the existing flows, based on CCME guidelines, it is not recommended to upgrade LS #4.

3.3.4 System Performance

The additional flows from Planned Development as presented in **Table 3-9,** were entered into the hydraulic model for this scenario. Hydraulic performance was reviewed considering the following:

- Lift stations were assumed to be upgraded to the required capacities listed in Table 2-1**Table 3-10**

- Gravity sewer components match existing conditions (deficiencies highlighted in Section 3.2.2 were not addressed)

Therefore, any sections which were highlighted as deficiencies in the Existing Conditions scenario are also highlighted for Planned Development.

3.3.4.1 Gravity Sewer

Generally, the gravity sewer network performed adequately under the Planned Development scenario, except for select areas which were found to have elevated surcharging risks.

Please refer to **Map 3-3 – Planned Development – HGL condition of Sanitary Network** for an overview of the system performance for this scenario.

3.3.4.1.1 TRUNK SEWER

Additional sections of the trunk sewer are identified as potential hydraulic issues. Over 85% of the length of the trunk sewer is highlighted as a surcharging risk (red or blue).

When comparing the hydraulic grade line of the existing conditions vs planned developments, it can be seen that the hydraulic grade line of the sections closest to the WWTF has been reduced significantly. This improvement is directly related to the modified tailwater conditions based on the planned upgrades to the WWTF, which is expected to substantially reduce the wet-well water level. Please see Section 3.1.5.5 for more information.

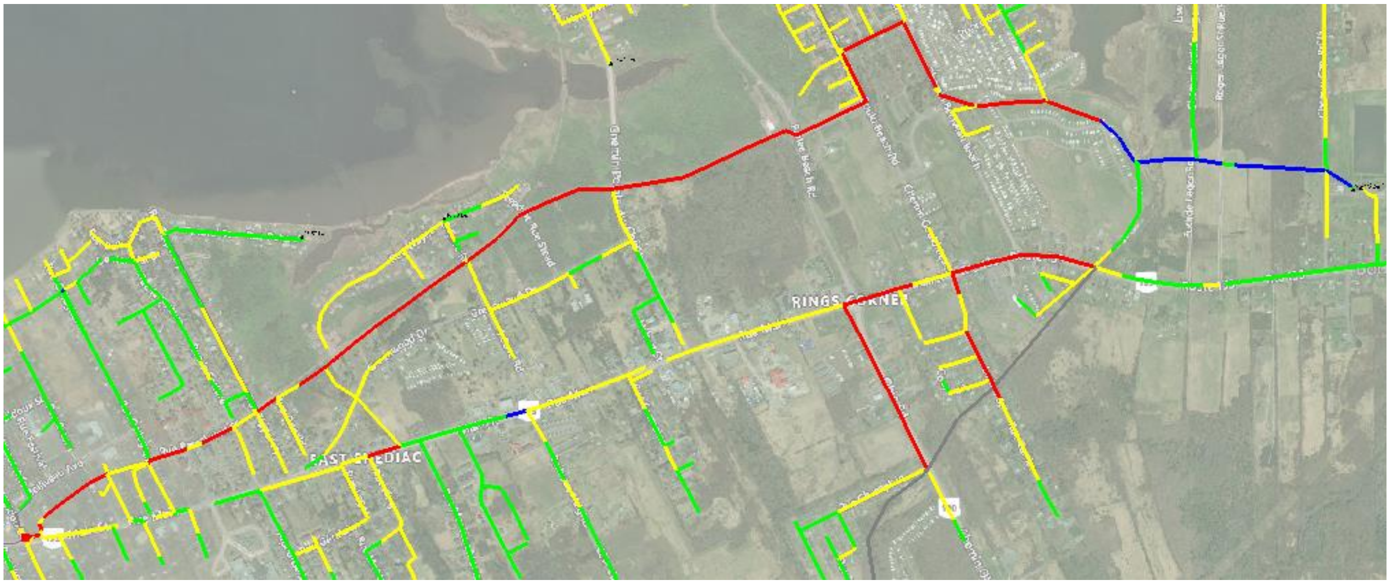


Figure 3-18: Planned Development -Trunk sewer piping from Weldon Street to the WWTF

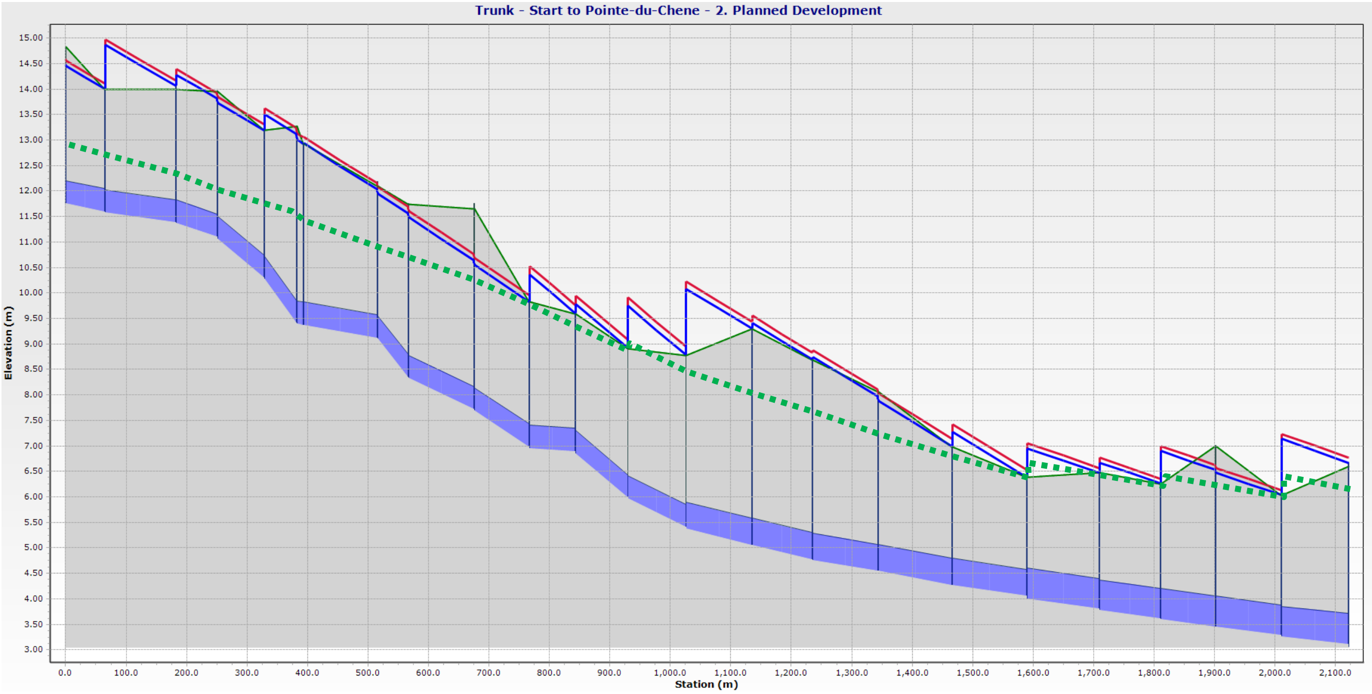


Figure 3-19: Planned Development -Trunk Sewer HGL Weldon St to Pointe-du-Chene Rd

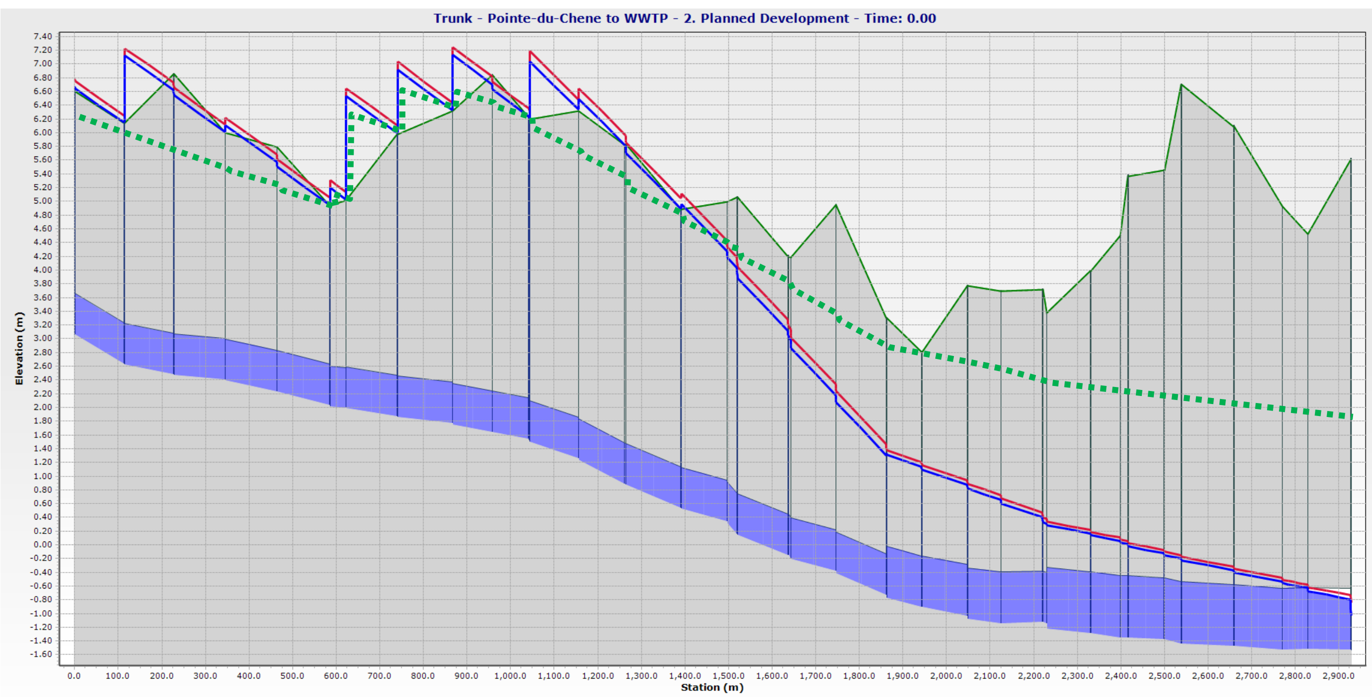


Figure 3-20: Planned Development - Trunk Sewer HGL Pointe-du-Chene Rd to WWTF

Figure 3-19 and Figure 3-20 provide a visual representation of the hydraulic grade line (blue line) vs the existing scenario HGL (green dashed line) for the entire length of the sanitary trunk sewer. Multiple additional manholes are shown as "surcharged" when compared to the existing conditions scenario.

3.3.4.1.2 MAIN STREET AND DOCK STREET

When compared to the evaluation of the existing conditions, there are an additional two (2) gravity sections highlighted in red which extend along Main Street to past Chesley St. Beyond the additional pipe sections, the hydraulic grade line from Chesley to LS #3 also increases when compared to the Existing Conditions scenario (green dashed line) which could lead to a higher likelihood of issues.

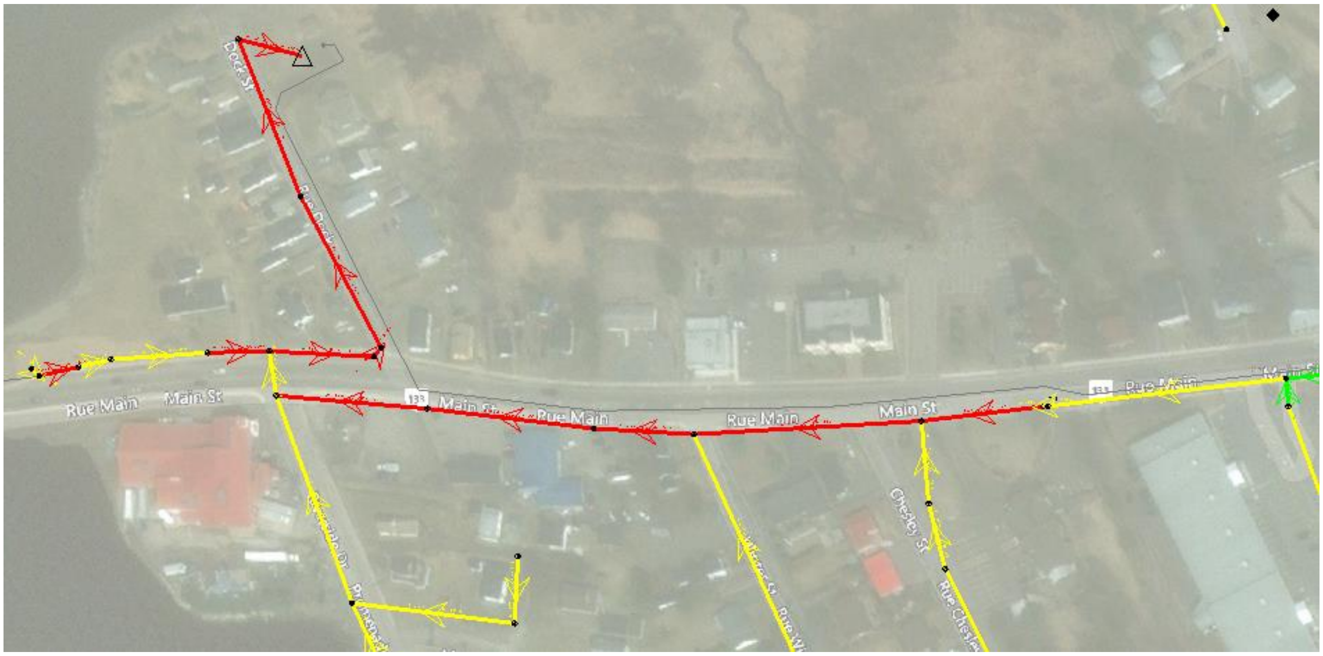


Figure 3-21: Planned Development - West Main and Dock Street

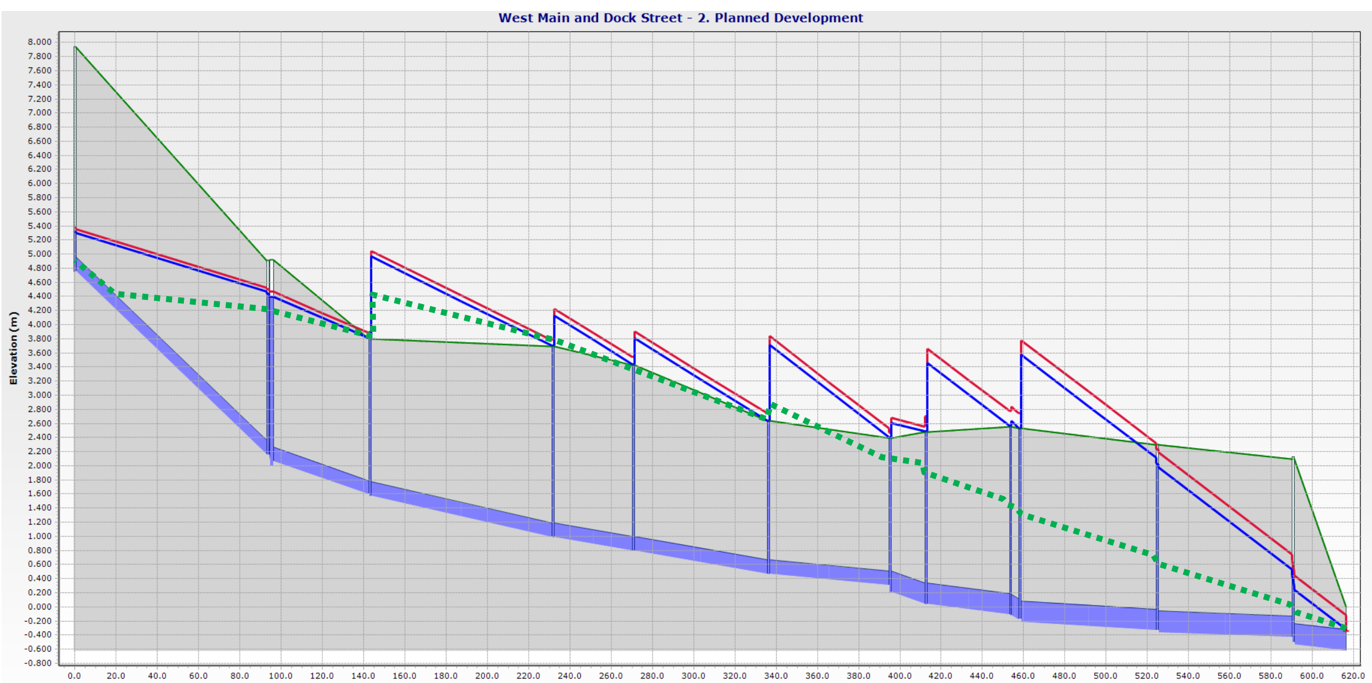


Figure 3-22: Planned Development - West Main and Dock Street Profile

3.3.4.1.3 MAIN STREET EAST

Additional sections along Ohio Road were highlighted as being at increased risk of surcharging during a Planned Development scenario. There are also sections on Cartier Street which appear to be impacted.

The section between Route 133 to the trunk sewer saw an improvement compared to the existing conditions scenario (shown in a green dashed line). This is due to the tailwater effects improving with the proposed new WWTF.

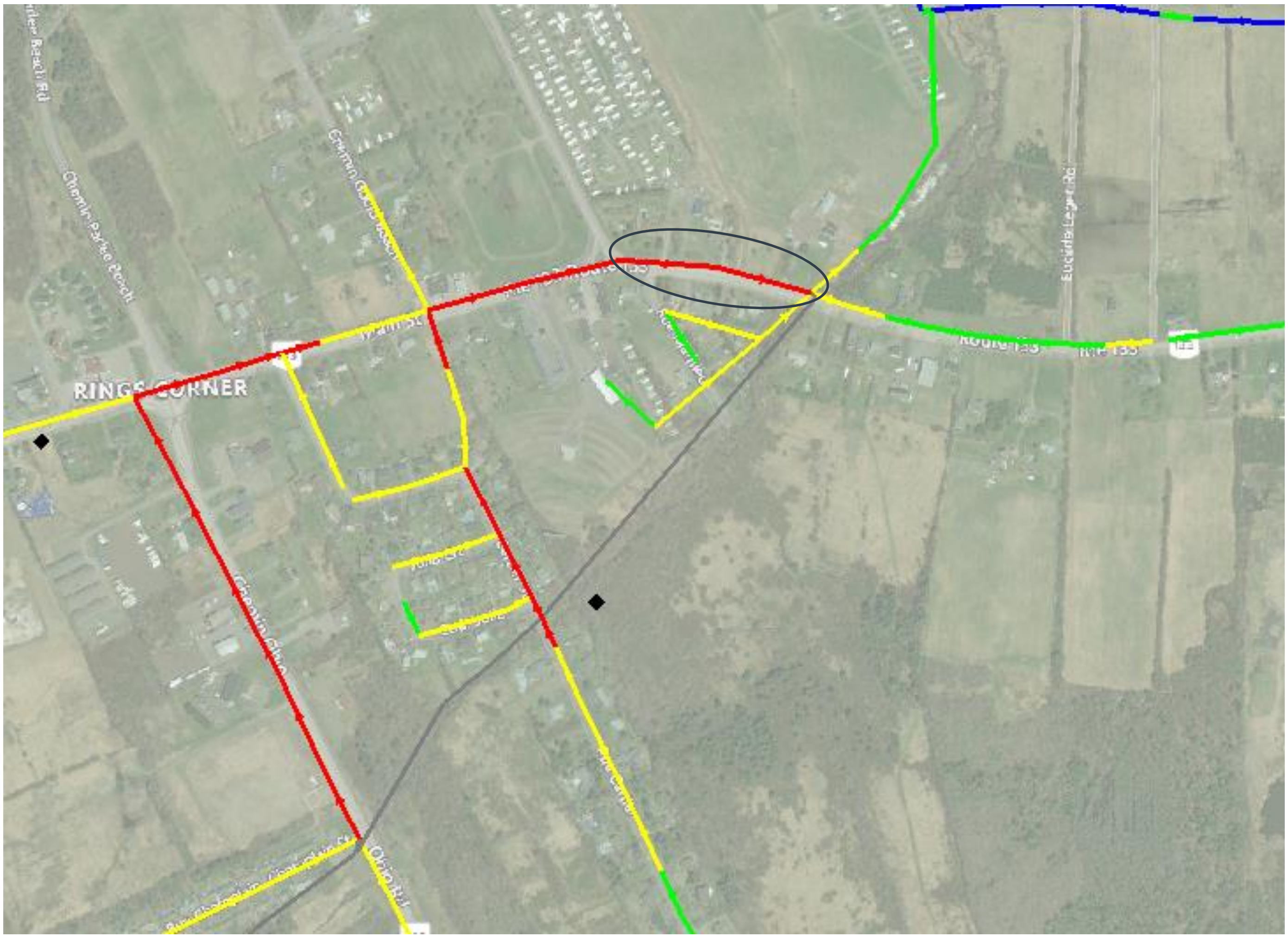


Figure 3-23: Planned Development - Main Street East

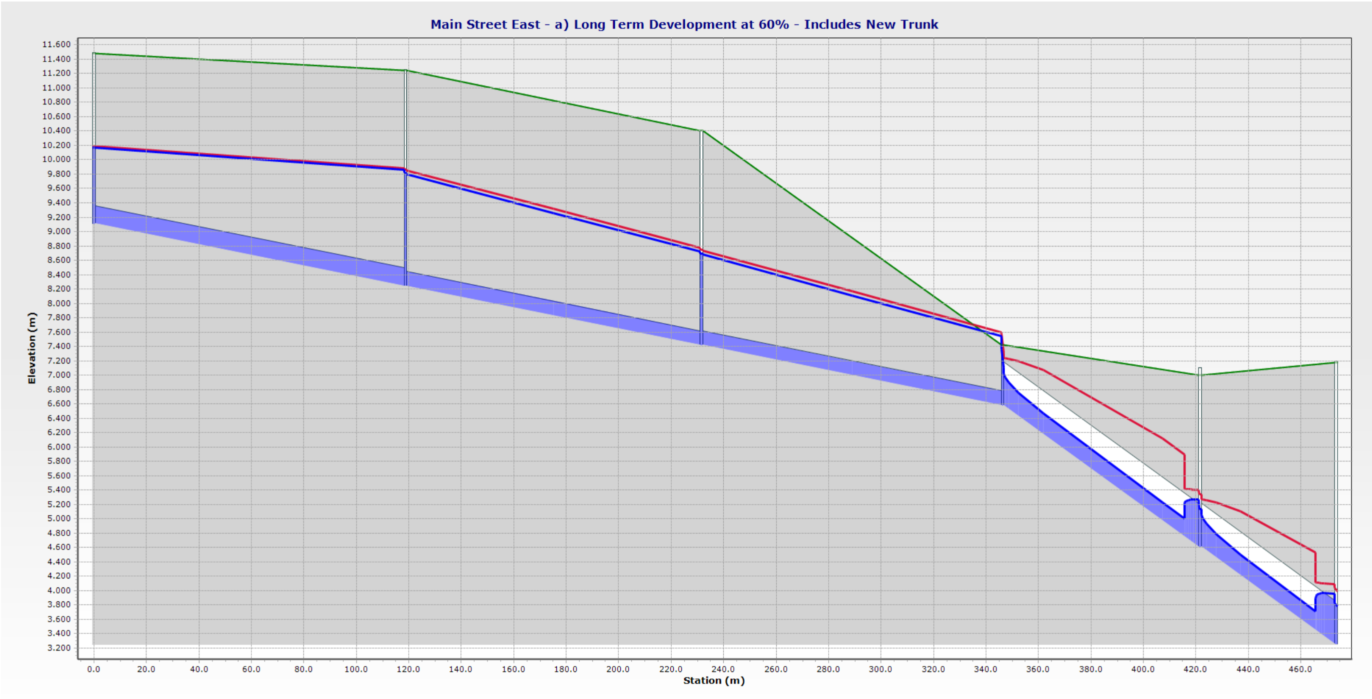


Figure 3-24: Long Term - Main Street East Profile

Most of the deficiencies would be eliminated if the proposed collector sewer is installed as described in Section 4.5.1. However, the two (2) final sections (circled in black) would be recommended for an upgrade in diameter.

3.3.4.1.4 MAIN STREET AND CASSIE AVENUE

No additional sections were identified as being surcharged for the planned development scenario. However, the hydraulic grade line worsens for the highlighted sections when compared to the existing conditions scenario (shown in a green dashed line).

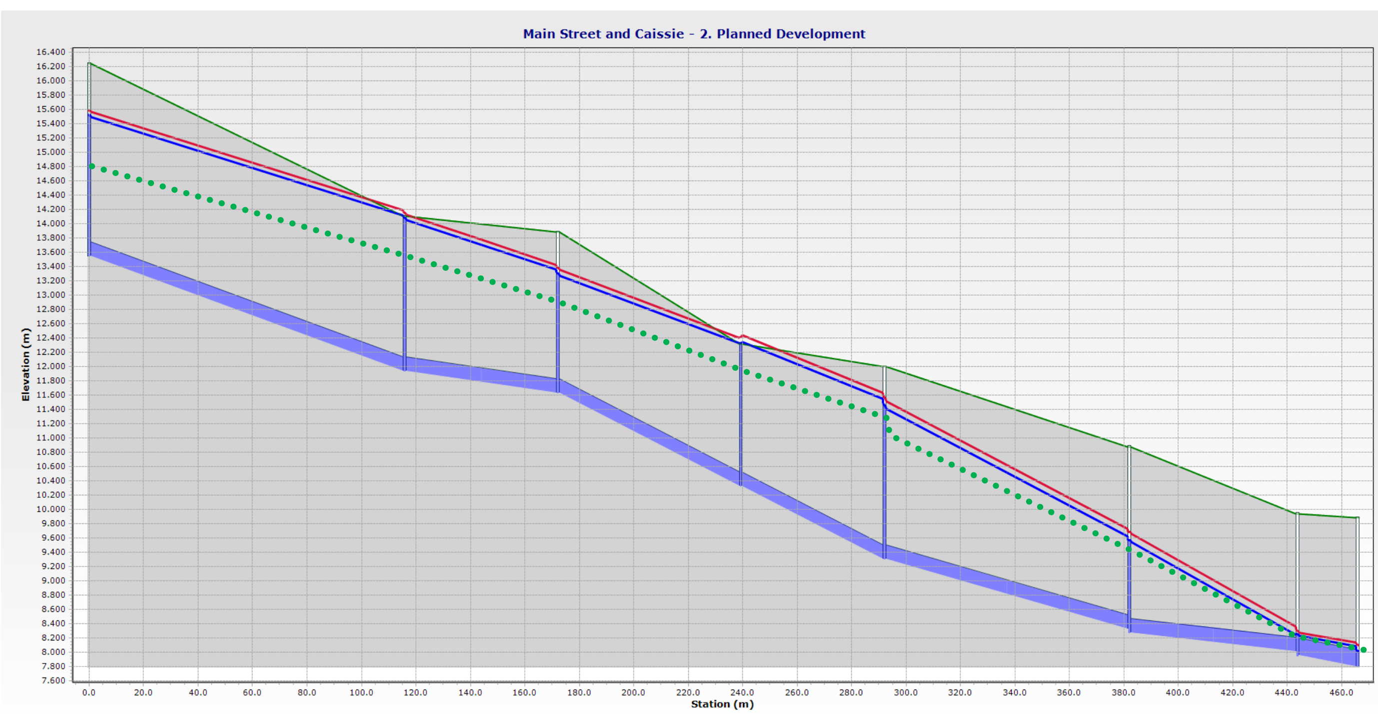


Figure 3-25: Planned Development - Main Street and Caissie Avenue

3.3.4.1.5 [NEW] ACADIE STREET, MANON AND CHATELLERAULT STREET

The highlighted sections along Main St. and Dock St. are undersized for peak flow conditions and pose an elevated risk of sewer surcharging in these sections and connected systems.

The potential reasons for these issues are as follows:

1. Over 50% of the length in red does not meet the minimum slope as recommended by ACWWA;
2. High I&I rates combined with pumped flows from LS 2 result in peak flows in excess of the pipe capacity;

The at-risk sections are a direct result from a large high-density development upstream which was added as part of the planned development scenario. The highlighted sections would be undersized for peak flow conditions and pose an elevated risk of sewer surcharging in these sections and connected systems

The existing piping has pipe slopes ranging from 0.350 to 0.755%. If existing pipe slopes are to remain, an increase in diameter would be required to improve hydraulic capacity.

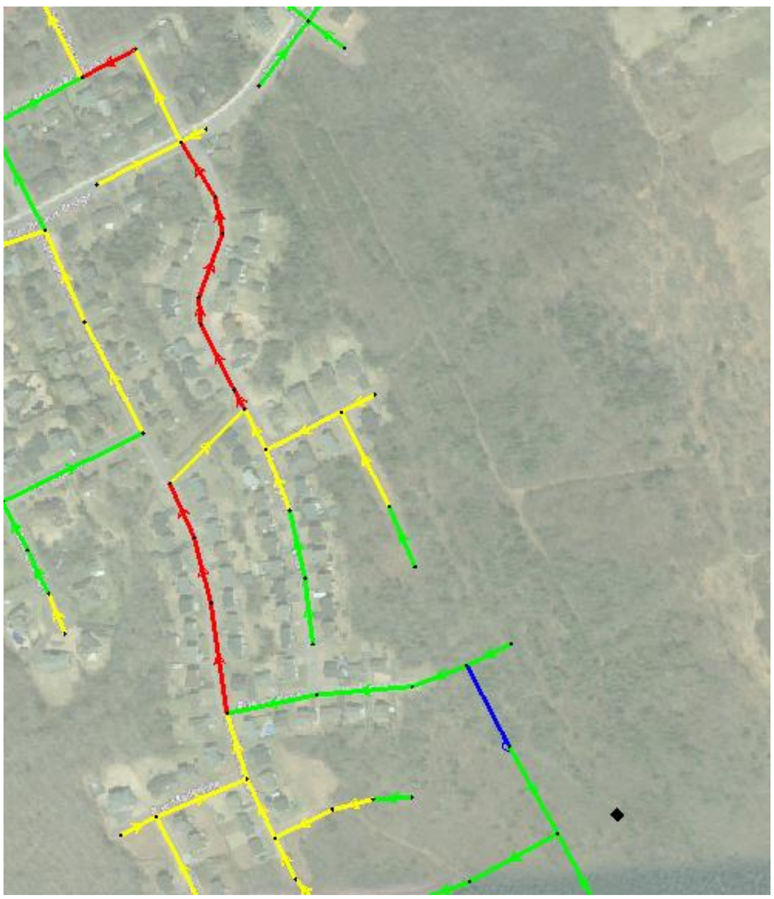


Figure 3-26: Planned Development - Acadie St., Manon and Chatellerault St.

This deficiency would be eliminated if the proposed collector sewer is installed as described in Section 4.5.1. In this case, The Planned Developments would be connected to the collector sewer and would not rely on existing infrastructure.

3.3.4.1.6 [NEW] OTHERS

Riverside Drive

Two (2) pipe sections along Riverside Drive are shown to have an increased risk of surcharging with the new planned development loading. This is largely due to the surcharging of the downstream infrastructure. Based on conceptual sizing for the West Main and Dock Street recommended upgrades, the issues for this piping are expected to be reduced and no pipe upgrades are recommended.

Figure 3-27: Planned Development - Riverside Drive

Main Street Near King Street

A single pipe section from along Main Street headed east from the intersection with King Street was highlighted as a result of Planned Development upstream. The existing diameter is 200mm Ø with a pipe slope of about 0.6%. This deficiency would be eliminated if the proposed collector sewer is installed as described in Section 4.5.1.

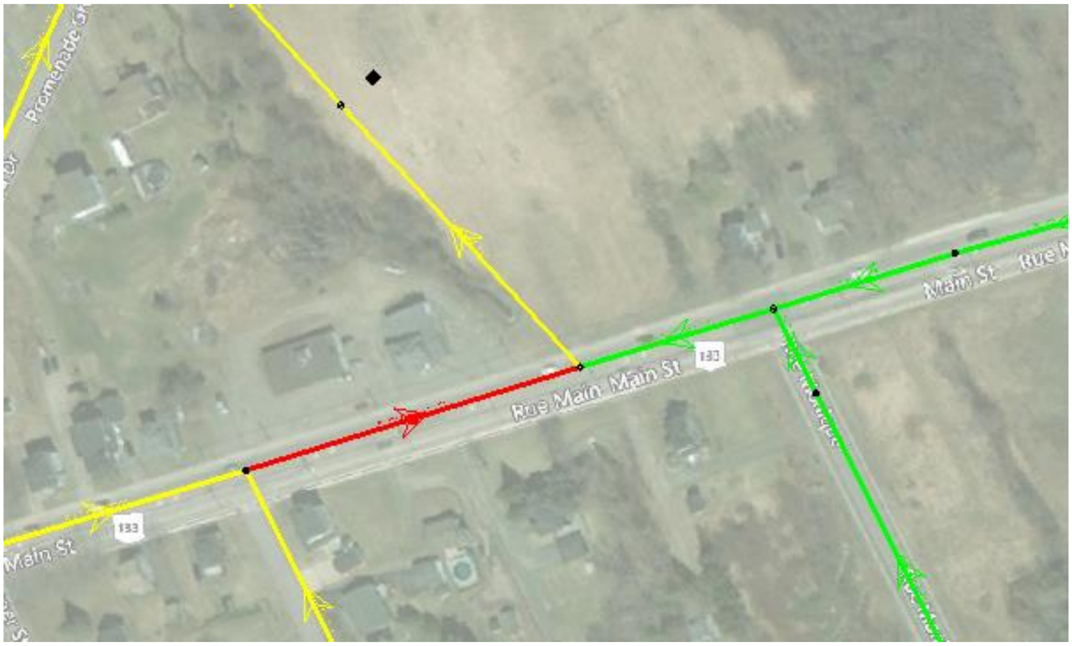


Figure 3-28: Planned Development - Main Street Near King Street

Main Street West Shediac

Two (2) pipe sections along Main Street headed west from the intersection with Inglis Road were highlighted as a result of Planned Development upstream. Although highlighted as a potential surcharging risk following Planned Development, flow conditions are proposed to be monitored in the LS 2 sewershed to confirm if the predicted flows are achieved. Much of the anticipated growth in the flows on this section of Main Street is associated with potential upgrades of LS 15, which may be rerouted in the future.

Figure 3-29: Planned Development - Main Street West Shediac

Heron Way

Two (2) pipe sections along Heron Way are identified as being at an increased risk of surcharging during the Planned Development scenario, due to the limited slope of each pipe (0.081% and 0.244%).

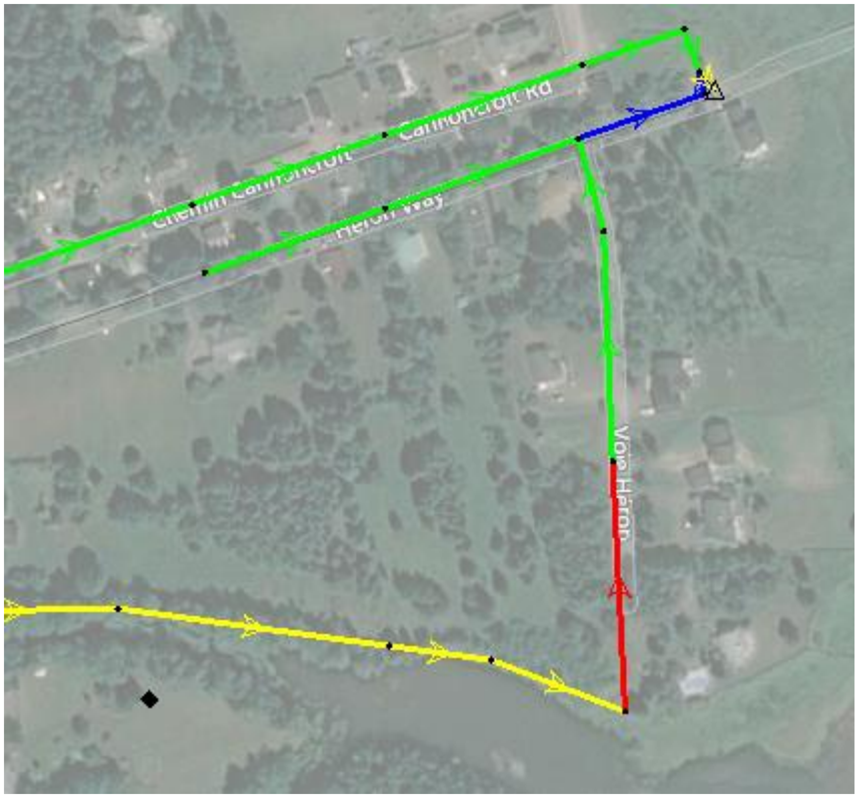


Figure 3-30: Planned Development - Heron Way

However, as shown in the HGL profile below, the surcharging is relatively minor and therefore was not recommended for an upgrade.

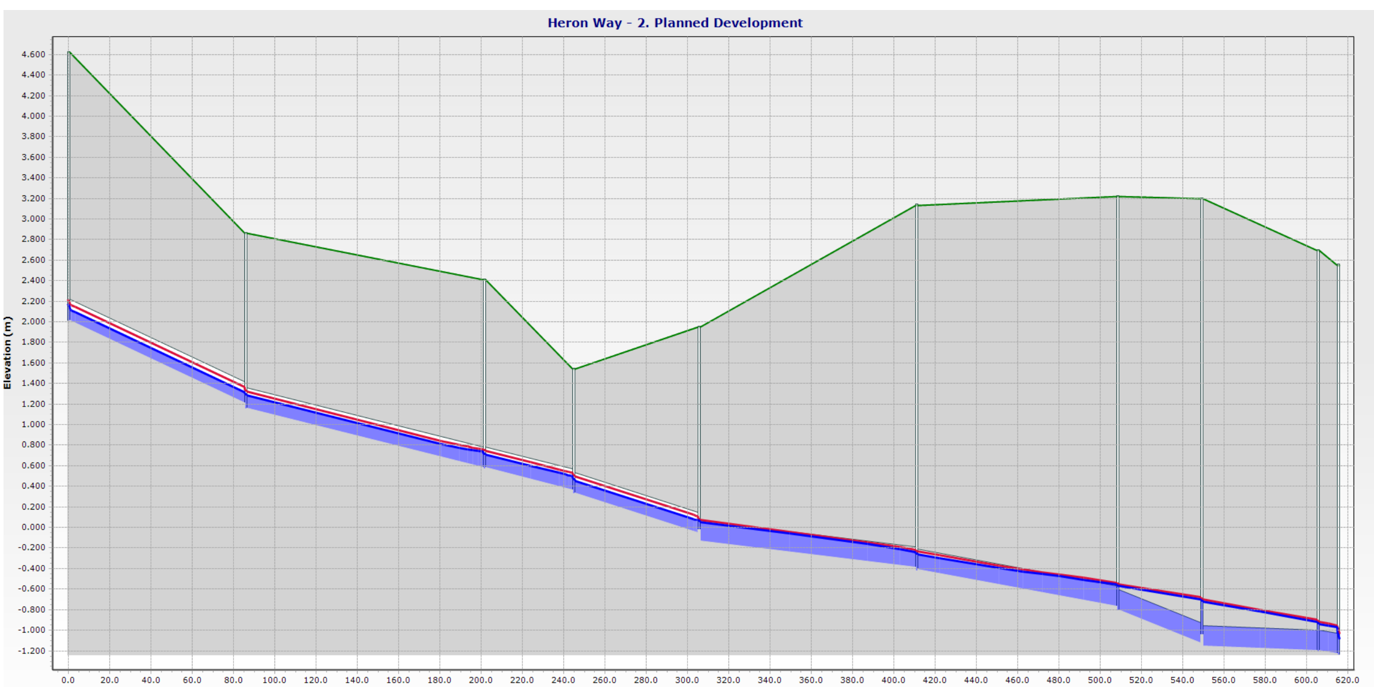


Figure 3-31: HGL Profile - Heron Way (Planned Development)

3.3.4.2 Lift Stations

The following table summarizes the projected design flow to accommodate Planned Development per CCME guidelines (See Section 3.1.5.4 for more information). Where there is Planned Development in the sewershed and the projected design flow is greater than the Pumping Capacity that station has been highlighted for an upgrade.

Lift Station	Pumping Capacity (1 pump)	Pumping Capacity (2 pumps)	Projected Design Flow	Requires Upgrade?
LS #1	8.1 L/s	12.4 L/s	12.4 L/s	No[1]
LS #2	22.8 L/s	32.9 L/s	**62.1 L/s**	**Yes**
LS #3	32.9 L/s	47.5 L/s	**95.1 L/s**	**Yes**
LS #4	38.0 L/s	71.5 L/s	71.5 L/s	No[1]
LS #5	6.6 L/s	10.1 L/s	**17.5 L/s**	**Yes**
LS #6	22.2 L/s	28.8 L/s	22.2 L/s	No[1]
LS #7	12.4 L/s	18.9 L/s	12.4 L/s	No[1]
LS #8	8.8 L/s	13.4 L/s	13.4 L/s	No[1]
LS #9	6.3 L/s	9.7 L/s	9.7 L/s	No[1]
LS #10	34.6 L/s	49.7 L/s	49.7 L/s	No[1]
LS #11	8.5 L/s	16.5 L/s	16.5 L/s	No[1]
LS #12	5.5 L/s	8.4 L/s	8.4 L/s	No[1]
LS #13	9.6 L/s	14.7 L/s	10.2 L/s	No[1]
LS #14	12.6 L/s	19.2 L/s	**15.8 L/s**	**Yes**
LS #15	6.3 L/s	9.7 L/s	**17.4 L/s**	**Yes**
LS #16	4.4 L/s	6.8 L/s	4.4 L/s	No[1]
LS #17	1.6 L/s	2.4 L/s	1.6 L/s	No[1]
LS #18	4.6 L/s	7.1 L/s	7.1 L/s	No[1]
LS #19	41.1 L/s	62.8 L/s	41.1 L/s	No[1]
LS #20	29.1 L/s	44.5 L/s	29.1 L/s	No[1]

1. No planned development in this sewershed, therefore no flow increase is anticipated. If development is proposed, a more detailed review should be completed. The Commission should continue to monitor flow conditions at this station and highlight cases where 1 pump is insufficient during dry-weather flow conditions.

3.4 Long Term Planning

Additional development beyond the Planned Development scenario has been considered as identified in **Map No 3-2 – Future Loading** in **Appendix A.**

For these Long Term Planning areas, the development type (residential vs. non-residential) is based on the zoning associated with each property and the assumptions listed in Section 3.1.5. The long-term development is essentially the remaining areas either not developed or identified in the planned development.

As highlighted in Section 3.3.1.1, it was assumed for this study that all Planned Development areas would be completed in 19 years. Therefore, to achieve a 50-year planning horizon (2072), an additional 31 years of growth was included in the Long Term Planning scenario.

Therefore, the following additional growth was assumed to occur for the Long Term Planning horizon:

- In the period following Planned Development, it was assumed growth rates would align with historical growth rates in the Town. For the purposes of this study, a growth rate of 2.5% was assumed.
- This growth rate results in assumed population growth of 215% (2.15x) over the 31-year planning horizon. This would see the population grow from 15,385 at the end of the Planned Development scenario, to 33,220 at the end of the Long Term Planning scenario.
- This includes approximately 210 hectares of industrial/commercial development over the planning horizon.
- To achieve the projected population of 32,400 in 2072, it is estimated that 60% of developable residential areas within the current Town boundary would need to be developed to a density of 45 p/hectare.

3.4.1 Flows

Please see **Section 3.1.5** for information on the assumptions used for estimating flow increases for the Long Term Planning scenario.

The following table summarizes the projected increase in flow to individual sewersheds as a result of development within the Long Term Planning scenario.

Please see **Map 3-2 – Future Loading** for locations where the development flows were assumed to be added in the model.

Table 3-11: Additional Peak Flow - Long-Term Estimated Flows

Sewershed	Type of Loading	Development Area (Hectares)	Equivalent Population	# of Units	Additional PWWF* (L/s)	Cumulative PWWF (L/s)
LS #1	Residential	2.30	104	50	2.39	2.39
LS #2	Residential	2.25	102	49	1.96	120.24
	Commercial	103.01	4,608	-	88.83	
LS #3	Residential	3.36	152	73	3.49	131.54
	Commercial	2.27	102	-	2.34	
LS #4	Residential	0.17	8	0.04	0.18	0.18
LS #5	Residential	0.65	30	0.13	0.69	0.69
LS #6	Residential	2.06	93	45	2.14	2.14
LS #8	Residential	2.59	117	56	2.58	2.58
LS #10	Residential	11.69	527	251	11.68	14.26

Sewershed	Type of Loading	Development Area (Hectares)	Equivalent Population	# of Units	Additional PWWF* (L/s)	Cumulative PWWF (L/s)
LS #11	Residential	1.66	75	36	1.70	1.70
LS #13	Residential	5.74	259	124	5.47	5.47
LS #14	Residential	26.19	1,179	562	25.09	25.09
LS #15	Residential	2.29	104	50	1.97	1.97
LS #19	Residential	118.20	5,319	2,533	97.27	153.14
LS #20	Residential	61.23	2,756	1,313	55.57	55.57
Trunk	Residential	129.82	7,011	2,782	125.94	520.29
	Commercial	87.83	4,715	-	90.97	
Sub-Total - Residential		396.15	17,836	8,501	338.14	-
Sub-Total - Non Residential		210.68	9,425	-	182.15	-
Total		**606.83**	**27261.20**	**8,501**	**520.29**	**520.29**

*The peaking factor has been estimated based on new hectares to be developed only meaning it does not consider the existing population in the calculation. This leads to higher peaking values which is deemed acceptable during this high-level conceptual phase.

3.4.2 System Improvements

When analyzing the performance of existing system components, it was assumed that some additional infrastructure and improvements would be implemented to reduce the reliance on existing system components.

Those improvements include:

- New collector sewer starting near the south end of Bellevue Heights and running to the north-east along an existing trail, which will service development along this corridor;
- Reconnecting forcemains from both LS 3 and LS 4 from the existing trunk sewer to the top end of the new collector sewer described above.

For more information on these improvements, please see Section 4.5.

3.4.3 System Performance

The long-term additional flows as presented in Table 3-11 were entered into the hydraulic model for the long-term scenario. The only upgrades that have been included in the model for this scenario are as presented in section 3.4.2 above. This means that most of the deficiencies already identified in the existing conditions and planned development scenarios are still deficient for the Long Term Planning scenario.

Since this scenario is highly theoretical and includes multiple assumptions over a 50-year period, the identified " issues " will not be recommended for upgrades. The analysis is meant to highlight sewer components which could be at risk of future surcharging issues because of future development.

This evaluation assumes that I&I flows for existing areas are not reduced significantly.

Please refer to map 3-4 - Long Term Development - HGL condition of Sanitary Network for an overview of the system performance for this scenario and proposed upgrades.

3.4.3.1 Gravity Sewer

3.4.3.1.1 TRUNK SEWER

Considering the proposed system improvements described in the previous section, the trunk sewer is expected to resume surcharging conditions by the end of the planning horizon (2072). That being said, it was noted the hydraulic conditions in the trunk sewer are expected to be better (less surcharging) than existing conditions/development.

While pipe capacity was found to be deficient in the Long-Term Planning scenario in the trunk sewer, upgrades to pipe size are not currently recommended. It is recommended that the hydraulic and flow conditions in the trunk sewer continue to be monitored to confirm if flows increase at the projected rate and to see if I&I reduction efforts successfully offset growth to flow associated with development.

3.4.3.1.2 TRUNK SEWER - FROM DISCHARGE OF NEW COLLECTOR SEWER TO WWTF

As mentioned in Section 3.4.2, the piping from the new collector sewer discharge (west of Euclide Leger Rd.) to the WWTF would need to be upgraded to accommodate the new sanitary flows from Long Term Planning development. This upgrade would provide additional benefits to the hydraulic conditions in upstream areas.

3.4.3.1.3 EAST SHEDIAC MAIN STREET AND LS 19

The piping along Route 133 east of the existing WWTF would need to be upgraded to accommodate future development areas and associated flows. Please see **Section 4.5.2** for more information.

3.4.3.1.4 LS 14 SEWERSHED

The Long-Term Planning scenario would see a significant increase in flow when compared to existing conditions, which would necessitate significant upgrades to the collection system infrastructure.

Just over 30% of the existing piping in LS #14 sewershed is identified as potentially at risk. Either pipe upgrades or I&I reduction may be required for the ultimate buildout.

Other isolated sections are expected to be impacted by the Long-Term Planning scenario. These sections can be seen on **Map 3-4** in **Appendix A**.

3.4.3.2 Lift Stations

The table below summarizes the additional flows anticipated from development in the Long Term Planning scenario, including additional pumped flows from contributing lift stations.

Table 3-12: Long Term - Additional PWWF

Lift Station	Planned Development Pumping Capacity (L/s)	Long-Term Planning - Additional PWWF (L/s)	Total Additional Pumped Flows (L/s)	Projected Pumping Capacity (L/s)
LS #1	12.40	2.39	-	14.79
LS #2	62.10	90.79	LS #1 - 2.39 LS #14 - 25.09 LS #15 - 1.97	182.34
LS #3	95.10	5.83	LS #2 - 120.24	226.64

Lift Station	Planned Development Pumping Capacity (L/s)	Long-Term Planning - Additional PWWF (L/s)	Total Additional Pumped Flows (L/s)	Projected Pumping Capacity (L/s)
			LS #13 - 5.47	
LS #4	71.50	0.18	-	71.68
LS #5	17.46	0.69	-	18.15
LS #6	22.20	2.14	-	24.34
LS #7	17.40	0.00	-	17.4
LS #8	13.40	2.58	-	15.98
LS #9	9.70	0.00	-	9.70
LS #10	49.70	11.68	LS #8 - 2.58	63.96
LS #11	16.50	1.70	-	18.20
LS #12	8.40	0.00	-	8.40
LS #13	14.70	5.47	-	20.17
LS #14	15.80	25.09	-	40.89
LS #15	17.42	1.97	-	19.39
LS #16	6.80	0.00	-	6.80
LS #18	7.10	0.00	-	7.10
LS #19	62.80	97.27	LS #20 - 55.57	215.64
LS #20	44.50	55.57	-	100.07

The table above provides a high-level forecast of additional PWWF expected related to long-term development. For any upcoming LS upgrades, it is recommended to base the design on the Planned Development as provided in Section 3.3.2, but to also consider including the projected flows for the long-term scenario if it is feasible and cost-efficient.

4 Recommended Improvements

The following sections present recommended measures and improvements to address deficiencies that were identified through this study. Where cost estimates are provided for individual improvements, more detailed cost estimate tables can be found in **Appendix B.**

4.1 Inflow and Infiltration Reduction

Inflow and infiltration are relatively high in the Town of Shediac when compared to similar municipalities. In many cases, I&I is the dominant component of flow in areas where hydraulic deficiencies or capacity constraints were identified as part of this study. This further accents the importance of understanding and eliminating sources of I&I in Shediac.

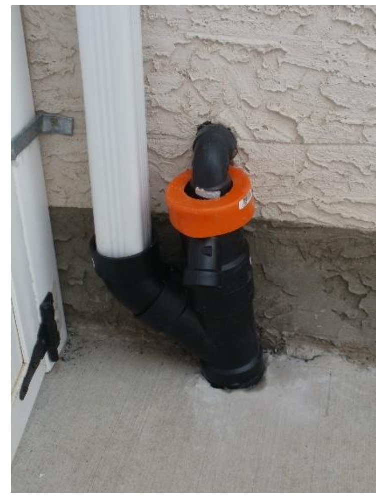


Figure 4-1: Direct connection of Roof drain and sump pump to sewer

Inflow and Infiltration reduction should remain a key area of focus for the Commission. Beyond flow monitoring, additional investigations should be scheduled as priority areas are identified.

This additional investigation could include:

- Dye testing of storm inlets suspected of being cross-connected with the sewer;
- Visual assessment of manhole cover locations to highlight structures with a high inflow potential;
- Visual assessment of private properties (from the roadway) to highlight suspected cross connections such as downspouts and sump pumps (see image on the right);
- Smoke testing of sewers to identify inflow sources;
- Closed Circuit Television (CCTV) inspection of sewers during periods of high groundwater to identify leaking joints, lateral connections, etc.;
- Visual inspection of manholes during high groundwater conditions to identify leaking structures.

4.1.1 Flow Monitoring and Continued Study

To better characterize this flow component, the Commission is engaged in an ongoing flow monitoring program. This program has been designed to strategically move up through the collection system to identify priority areas for I&I reduction efforts.

The Commission's 5-year plan currently includes $100,000/year for flow monitoring and I&I analysis. The Commission should continue to budget as much as can be afforded for this program to accelerate a more complete understanding of the priority areas.

4.1.2 SCADA Improvements

Several of the Commission’s lift station facilities do not currently have flow meters (or meters are not functional) and therefore rely on pump runtime records as a proxy for flow. For a few of these stations, it would be valuable to install flow meters for the purpose of long-term monitoring and to establish a baseline that can be used to track the success of I&I reduction efforts. While there’s value in installing flow meters at all lift stations, the following stations should be prioritized:

- Lift Station No.1
- Lift Station No.2
- Lift Station No.4
- Lift Station No.5
- Lift Station No. 14 (flow meter scheduled for installation in early 2023)

4.1.3 Reduction Methods – General Approach

Inflow and Infiltration reduction methods are as varied as their sources and are therefore highly specific to individual sewersheds. However, some common improvements have been found to be effective in reducing I&I.

- **Private Lateral Separation:** Once identified, moving the connection of basement sump pumps and storm drain connections to the storm collection system can significantly reduce the total I&I in the sanitary collection system. A single 2x3" downspout can carry 186 USgpm (11.7 l/s) whereas a 1/2hp sump pump can discharge 53 USgpm (3.3 l/s). It is easy to see how a few illicit connections can overwhelm a sewer pipe. For example, a 200mm PVC sewer pipe at 0.6% grade has a capacity of 523 USgpm (33 l/s).
- **Disconnection of Storm Inlets:** If direct connections are discovered such as ditch inlets or catch basins, disconnection of these can provide significant relief to a sewer system. A single catch basin for instance can collect over 1500 USgpm (95 l/s). Reconnection of storm inlets may require the installation of a storm sewer on the street if drainage cannot be achieved through ditches alone.
- **Replacement/Rehabilitation of Sewer Systems:** Replacement or Rehabilitation of sewer pipes is a viable option to reduce overall I&I, particularly the Infiltration component. CCTV videoing of the sewer systems during periods of high groundwater (spring thaw) can identify pipe sections with leaky joints and connections.
- **Manhole Replacement/Rehabilitation:** Where manholes are identified as being infiltration and/or inflow sources through visual inspection, these issues can be addressed through several methods including grout injection, epoxy, lining, frame/cover replacement or sealing, etc.

4.1.4 Impact of Inflow and Infiltration Reduction

To illustrate the impact of reduction to inflow and infiltration on overall system performance, multiple scenarios were analyzed. These included:

- Existing Conditions
  - 25% reduction to I&I
  - 50% reduction to I&I
- Planned Developments
  - 25% reduction to I&I (in existing areas)
  - 50% reduction to I&I (in existing areas)

The following table summarizes the reduction in flow to each sewershed associated with each reduction scenario and the additional development that could be accommodated through I&I reduction alone. These values are based on reductions to the calibrated I&I flow component for individual sewersheds.

Table 4-1: Existing Conditions - Flow Impacts of I&I Reduction Scenarios

Sewershed	25% I&I Reduction		50% I&I Reduction	
	Flow Red. (l/s)	Capacity Gained (population)	Flow Red. (l/s)	Capacity Gained (population)
LS #1	2.25	512	4.50	1,024
LS #2	30.12	6,849	60.24	13,697
LS #3	4.04	919	8.08	1,837
LS #4	18.52[1]	4,211	37.03	8,421
LS #5	0.7[1]	160	1.40	320
LS #6	3.64	828	7.28	1,656
LS #7	3.52	801	7.04	1,602
LS #8	2.88	655	5.76	1,310
LS #9	2.11	480	4.21	959
LS #10	2.11	480	4.22	960
LS #11	3.39	772	6.78	1,543
LS #12	1.82	414	3.64	828
LS #13	1.08	246	2.16	491
LS #14	2.44	554	4.87	1,108
LS #15	2.29	522	4.59	1,044
LS #16	0.20	45	0.39	90
LS #18	1.63	371	3.26	741
LS #19	2.22	504	4.43	1,008
LS #20	4.96	1,128	9.91	2,255
Overall [2]	89.91	20,451	179.81	40,894

[1]Values differ slightly between existing conditions and planned development scenarios. This is due to the proposed upgrades on Pleasant Street diverting flows from LS 4 to LS 5 in the planned development scenario.

[2] Represents the sum of projected I&I reduction in individual sewersheds. Where some Lift Stations currently overflow during peak wet weather events, not all of this I&I flow currently reaches the WWTF. Therefore, the reduction in peak flow at the WWTF would be somewhat less than the values shown.

Please refer to **Map 4-1** and **4-2** in **Appendix A** which show the impacts of the 50% I&I reduction on the hydraulic performance of the sewer infrastructure for the existing conditions and planned development scenarios.

The I&I reduction scenarios were not found to be as impactful as anticipated. Possible reasons for this include:

— The hydraulic model assumes that all lift stations are not equipped with variable frequency drives (VFDs). Even though the I&I reduction lowers the PWWF, unless the reduction in flows is enough to go from 2 pump capacity to 1 pump capacity, the flow impacts downstream are not changed in the model;

— Reduction in I&I in a pumped sewershed would result in a reduction of pumping duration/frequency, which in reality would translate to a reduced impact on flows in the receiving system. The steady state model does not account for this effect.

— Inflow and Infiltration rates in the trunk sewer were calibrated based on achieving the observed flow at the WWTF. Because of the way pumped flows were considered in the model, it is possible the I&I rates impacting the trunk sewer gravity sewershed were underestimated (because the pumped flow contribution may be overestimated);

Based on this analysis, the following observations were made

<u>**Existing Conditions**</u>

- 25% Reduction – Existing Conditions
    - Minor reduction in highlighted in red pipe sections along the trunk sewer main;
    - Minor HGL improvements along Main street near Caissie Avenue;
- 50% Reduction – Existing Conditions (additional to 25%)
    - Moderate HGL improvement along Main Street near Caissie Avenue;

4.2 Gravity Sewer Improvements

In most cases, the existing sanitary sewer collection system appears to be adequately sized for the estimated flow rates. The following sections have been identified for upgrades based on existing conditions only.

4.2.1.1 Backlot Gravity Sewer near Greenwood Prom.

As described in Section 3.2.2.1, a portion of the off-road sewer runs between Greenwood Promenade and the trunk sewer, which has a history of surcharging through manhole covers. Our modelling has suggested this could be due to the tailwater conditions in the trunk sewer backing up through the system.

To reduce the risk of surface overflow, it is recommended that both impacted manholes be rehabilitated and the frame and covers replaced with sealed/lockdown units.

This improvement is expected to cost approximately $35,770.

4.2.1.2 West Main Street and Dock Street

The gravity section on Main Street from one (1) pipe section east of Chesley to LS #3 on Dock Street including the piping from the LS #2 discharge location is recommended to be upsized. The pipe diameters on Main Street are recommended to be 250mm Ø and the piping on Dock Street is proposed as 400mm Ø. A total of just over 620m of piping and road is proposed for upgrades.

This improvement is expected to cost approximately $1,885,330.

4.2.1.3 Main Street East

While the hydraulic performance of this section was found to benefit from the proposed collector sewer (see Section 4.5), there are two (2) sections of sewer which remain at an increased risk of surcharging in the Planned Development scenario.

While the overall risk appears relatively low, the Commission should take advantage of any planned projects along this roadway to upgrade this pipe section.

This improvement is expected to cost approximately $695,130.

4.2.1.4 Main Street and Caissie Ave.

As shown in Section 3.2, a section of sewer along Main St. between Hamilton Road and Caissie Ave. is undersized. Furthermore, a portion of Caissie Ave. should be upgraded as well.

Therefore, an upgrade is recommended for this run of pipes (five pipe sections in total), including upsizing the pipes on Main Street to 300mm.

This improvement is expected to cost approximately $918,940.

4.2.1.5 Trunk Sewer

As previously mentioned, the sanitary trunk sewer has been identified as being surcharged and has HGL issues. However, an alternative approach is recommended due to the significant cost to upsize the trunk sewer from start to finish. This approach involves the reduction of flows through re-routing Lift Stations 3 and 4. Please see Section 4.5 for more information.

4.3 Lift Stations & Forcemains

As presented in Section 3.3.4.2, five (5) lift stations were identified as having insufficient capacity in either Existing Conditions or Planned Development scenarios.

The following table summarizes the current capacity, current forcemain size, projected capacity, required forcemain size and estimated cost associated with each Lift Station requiring an upgrade.

Table 4-2: Recommended Lift Station Upgrades

Station	Existing Pumping Capacity (L/s)	Existing Forcemain Size (mm)	Projected Design Capacity (L/s)	Required Forcemain Size (mm)	Estimated Cost of Upgrade
LS #2	32.81	200	182.34	**400**	$7,844,080
LS #3	47.32	200	226.64	**400 and 350 (Twin)**	See Section 4.4
LS #14	19.24	150	40.89	**200**	$ 3,094,650
LS #15	9.7	100	19.39	**150**	$ 2,268,560
LS #5	10.09	150	18.15	150	$ 2,763,700

Recommended upgrades of lift stations are based on expected Planned Development and the capacity of the station will need to be evaluated during preliminary design.

4.4 Trunk Sewer Bypass

To relieve capacity in the trunk sewer, it is proposed that both Lift Station 3 and Lift Station 4 could be re-routed to bypass the trunk sewer. This would remove all flow from the west end of Shediac from the trunk sewer.

To synergize with investments being made to service growth areas, it is proposed the new forcemain could be directed to the south toward the Highway along Sackville Road and discharged into the new collector sewer described in Section 4.5.1.

One concept for this bypass is to intercept the existing forcemains from both Lift Station 3 and Lift Station 4 near Weldon St. and combine them into a shared forcemain along Sackville Road.

By removing flows from both Lift Stations, modelling shows the existing hydraulic surcharging in the trunk sewer and adjacent branches would be greatly reduced. This would further provide additional capacity to service development within the Trunk Sewer sewershed.

If this concept is favourable to the Commission, a Preliminary Design study is recommended to better understand the infrastructure upgrades required and to select the preferred forcemain alignment.

Please see **Drawing 3-4** in **Appendix A** for more information on this concept and how it connects to the proposed collector sewer to the south.

The estimated cost for the proposed concept is $ 21,066,830 (cost for both the trunk sewer bypass and the new collector sewer)

4.5 Long Term Servicing

As discussed in Section 3.4, there are large unserviced areas that are within the service boundary of the Commission. Those areas were reviewed to highlight future infrastructure requirements to identify possible alignments for future collection system infrastructure based on topography.

The following service areas were reviewed.

4.5.1 West of Ohio Road

There is a large section of undeveloped land that is generally tributary to a watercourse that runs from south of the Highway to the north-east towards Belliveau Beach.

The proposed servicing concept for this area includes a new collector sewer which would run along the watercourse, parallel to which there is an existing trail system. The collector sewer would be installed so that manholes could be accessed along the trail system, with stubs crossing the watercourse at key locations to service the land on the opposite side.

This area is a high priority for development, with several large-scale Planned Developments which could be collected by this new collector sewer, which would relieve capacity on existing infrastructure. Furthermore, this collector sewer could provide servicing to the lands south of the highway including the Southeast Regional Correctional Center. This would further relieve existing infrastructure, where these flows are currently discharged.

Furthermore, this new collector sewer could be leveraged to de-commission Lift Station 16 at the south end of Bellevue Heights.

For more information on the proposed alignment, please see **Drawing 3-4** in **Appendix A**.

4.5.2 East of Ohio Road

There is a large undeveloped and unserviced area to the east of Ohio Road which could see an increased demand for development once other areas within the Town's core are exhausted. This area would require a network of collector sewer branches, which could follow alignments similar to those shown on **Drawing 3-4** in **Appendix A.** Currently, these areas would naturally drain to Route 133, which would necessitate a significant upgrade to gravity sewer and Lift Stations which currently service developments along this corridor.

There may be a potential for these areas to be collected by a new trunk sewer which would cross the Route 133 to the west of Lac des Boudreau. If the grades allow, this trunk sewer could then run west along the south end of the WWTF to the headworks lift station. The feasibility of this option would need to be explored in further detail as part of a more focused servicing study of this area.

Alternatively, a new upgraded lift station would be required near the low point on Route 133, with significant piping upgrades required.

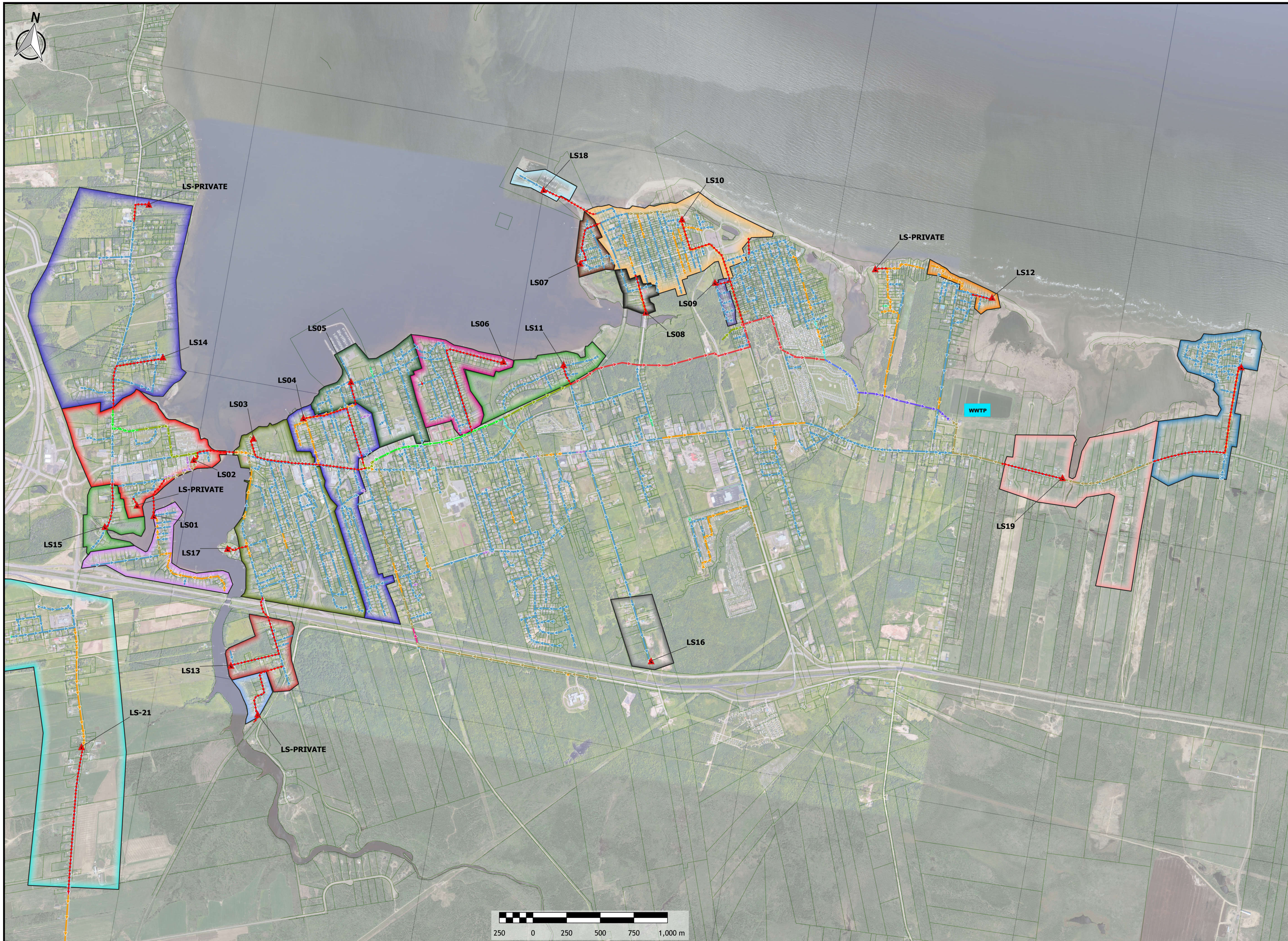
4.6 Model Updates

To continue relying on the SewerCAD model and associated recommendations, it is recommended the Commission make an allowance for these costs in its annual budget. These updates could include flow changes associated with completed developments, infrastructure changes from capital works projects, system extensions/changes completed by developers, adjustments to flow calibration from flow monitoring and/or SCADA data, and field investigations to populate current data gaps.

The Commission may choose to budget these items separately or to budget additional costs within the projects that produce the data (for example, budgeting additional costs for record drawing preparation on Capital works projects to transfer those infrastructure changes into the model).

Appendix A
Maps

ENGLOBE



Legend / Notes:

LEGEND

Zones
- LS #1 Sewershed
- LS #14 Sewershed
- LS #15 Sewershed
- LS #2 Sewershed
- LS #4 Sewershed
- LS #3 Sewershed
- LS #10 Sewershed
- LS #11 Sewershed
- LS #12 Sewershed
- LS #13 Sewershed
- LS #16 Sewershed
- LS #18 Sewershed
- LS #19 Sewershed
- LS #20 Sewershed
- LS #25 Sewershed
- LS #5 Sewershed
- LS #6 Sewershed
- LS #7 Sewershed
- LS #8 Sewershed
- LS #9 Sewershed
- Scoudouc LS #21

Network

SANITARY GRAVITY MAINS COPY - DIAMETERS
- 100
- 150
- 200
- 250
- 300
- 375
- 400
- 450
- 525
- 600
- 750
- 900

FORCE MAINS
- Active FM

SANITARY STRUCTURES copy
- Sanitary Manhole
- Wet Well
- Lift Station

WWTP WWTP

Project Location:

SHEDIAC, N.B.

Project Title:

SEWER SYSTEM MASTER PLAN

Map Title:

EXISTING SANITARY NETWORK

Map ID:

MAP No: **2-1**
PAGE No: **1 of 1**
SCALE: 1:13000

Revision:

DATE: 2022-12-22 BY: SAL
PROJ No: 2200536 APPR: SEB

GREATER SHEDIAC SEWERAGE COMMISSION DES ÉGOUTS SHEDIAC ET BANLIEUES

Legend / Notes:

LEGEND

Network

SANITARY STRUCTURES

- Sanitary Manhole
- Wet Well
- Lift Station

WWTP WWTP

SANITARY GRAVITY MAIN

- Both Inverts Assumed
- One Invert Assumed
- No Assumptions

Project Location:

SHEDIAC, N.B.

Project Title:

SEWER SYSTEM MASTER PLAN

Map Title:

ASSUMPTIONS FOR PIPE INFO

Map ID:

MAP No: **2-2**

PAGE No: **1 of 1**

SCALE: 1:13000

Revision:

DATE: 2022-12-22 BY: SAL

PROJ No: 2200536 APPR: SEB

GREATER SHEDIAC SEWERAGE COMMISSION DES ÉGOUTS SHEDIAC ET BANLIEUES

Legend / Notes:

LEGEND

Non Plot

SewerCAD Legend

- HGL ≤ 100% of pipe dia
- HGL ≤ 100% of pipe dia. & less than 2.2m from ground level
- HGL > 100% ≤ 130% of pipe dia.
- HGL > 130% of pipe dia. & HGL > 2.2m from ground level
- HGL > 130% of pipe dia. & HGL < 2.2m from ground level

Project Location:

SHEDIAC, N.B.

Project Title:

SEWER SYSTEM MASTER PLAN

Map Title:

EXISTING CONDITIONS - HGL CONDITION OF SANITARY NETWORK

Map ID:

MAP No: **3-1**

PAGE No: **1 of 1**

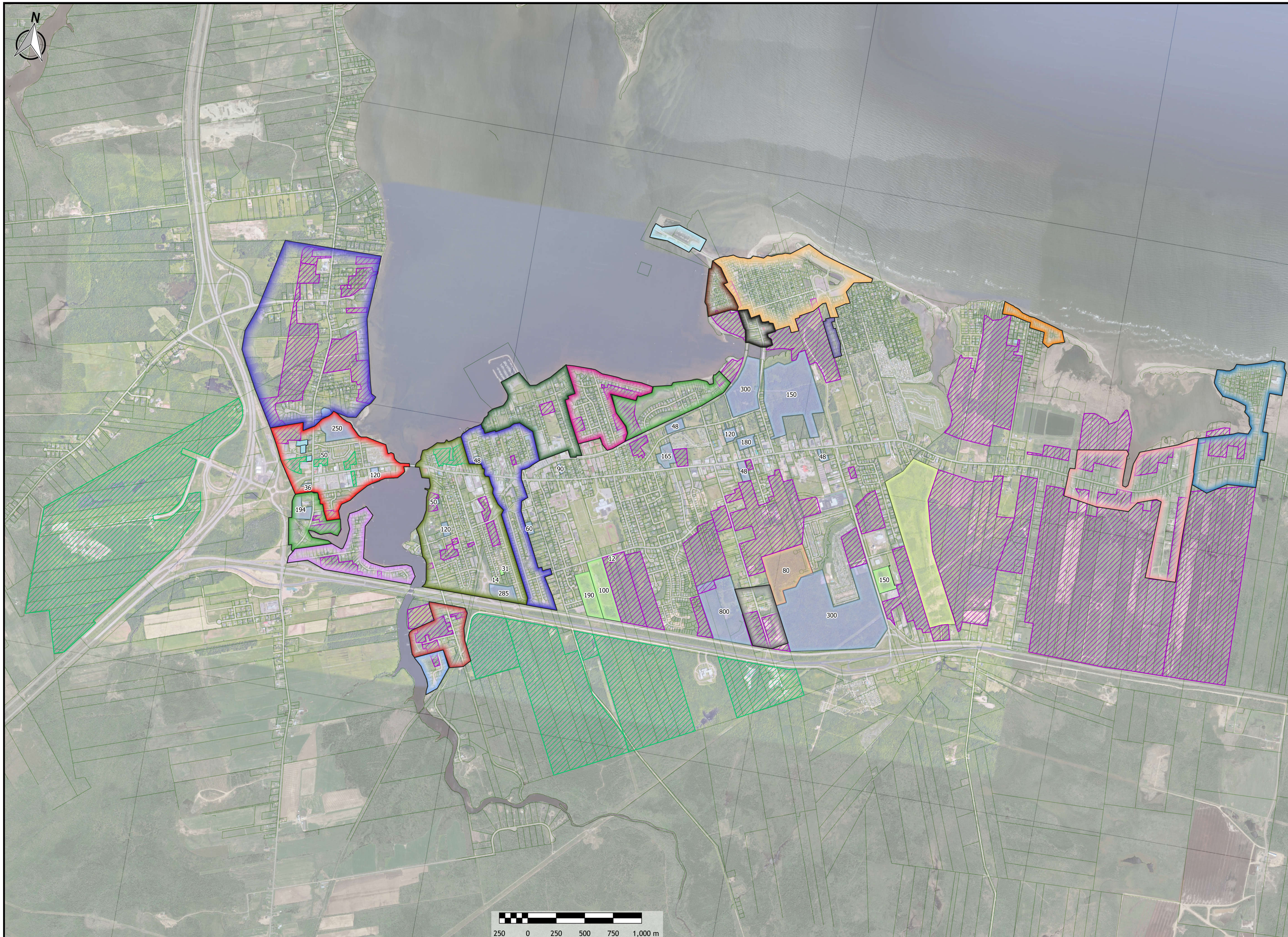
SCALE: 1:13000

Revision:

DATE: 2022-12-22 BY: SAL

PROJ No: 2200536 APPR: SEB

GREATER SHEDIAC SEWERAGE COMMISSION
DES ÉGOUTS SHEDIAC ET BANLIEUES



Legend / Notes:

LEGEND

Network

Zones

- LS #1 Sewershed
- LS #14 Sewershed
- LS #15 Sewershed
- LS #2 Sewershed
- LS #4 Sewershed
- LS #3 Sewershed
- LS #10 Sewershed
- LS #11 Sewershed
- LS #12 Sewershed
- LS #13 Sewershed
- LS #16 Sewershed
- LS #18 Sewershed
- LS #19 Sewershed
- LS #20 Sewershed
- LS #25 Sewershed
- LS #5 Sewershed
- LS #6 Sewershed
- LS #7 Sewershed
- LS #8 Sewershed
- LS #9 Sewershed
- Scoudouc LS #21

Future Loading - Areas copy

- Commercial
- Residential - High Density
- Residential - Low Density
- Residential - Medium
- Mini Home
- Institutional

Large Scale Future Development

- Residential - Not Included
- Commercial - Not Included
- Residential
- Commercial

Project Location:

SHEDIAC, N.B.

Project Title:

SEWER SYSTEM MASTER PLAN

Map Title:

FUTURE SANITARY LOADING

Map ID:

MAP No: **3-2**
PAGE No: **1 of 1**
SCALE: 1:15365

Revision:

DATE: 2022-12-22 BY: SAL
PROJ No: 2200536 APPR: SEB

GREATER SHEDIAC SEWERAGE COMMISSION
DES ÉGOUTS SHEDIAC ET BANLIEUES

Legend / Notes:

LEGEND

Non Plot

SewerCAD Legend

- HGL ≤ 100% of pipe dia
- HGL ≤ 100% of pipe dia. & less than 2.2m from ground level
- HGL > 100% ≤ 130% of pipe dia.
- HGL > 130% of pipe dia. & HGL > 2.2m from ground level
- HGL > 130% of pipe dia. & HGL < 2.2m from ground level

Project Location:

SHEDIAC, N.B.

Project Title:

SEWER SYSTEM MASTER PLAN

Map Title:

PLANNED DEVELOPMENT - HGL CONDITION OF SANITARY NETWORK

Map ID:

MAP No: **3-3**

PAGE No: **1 of 1**

SCALE: 1:13000

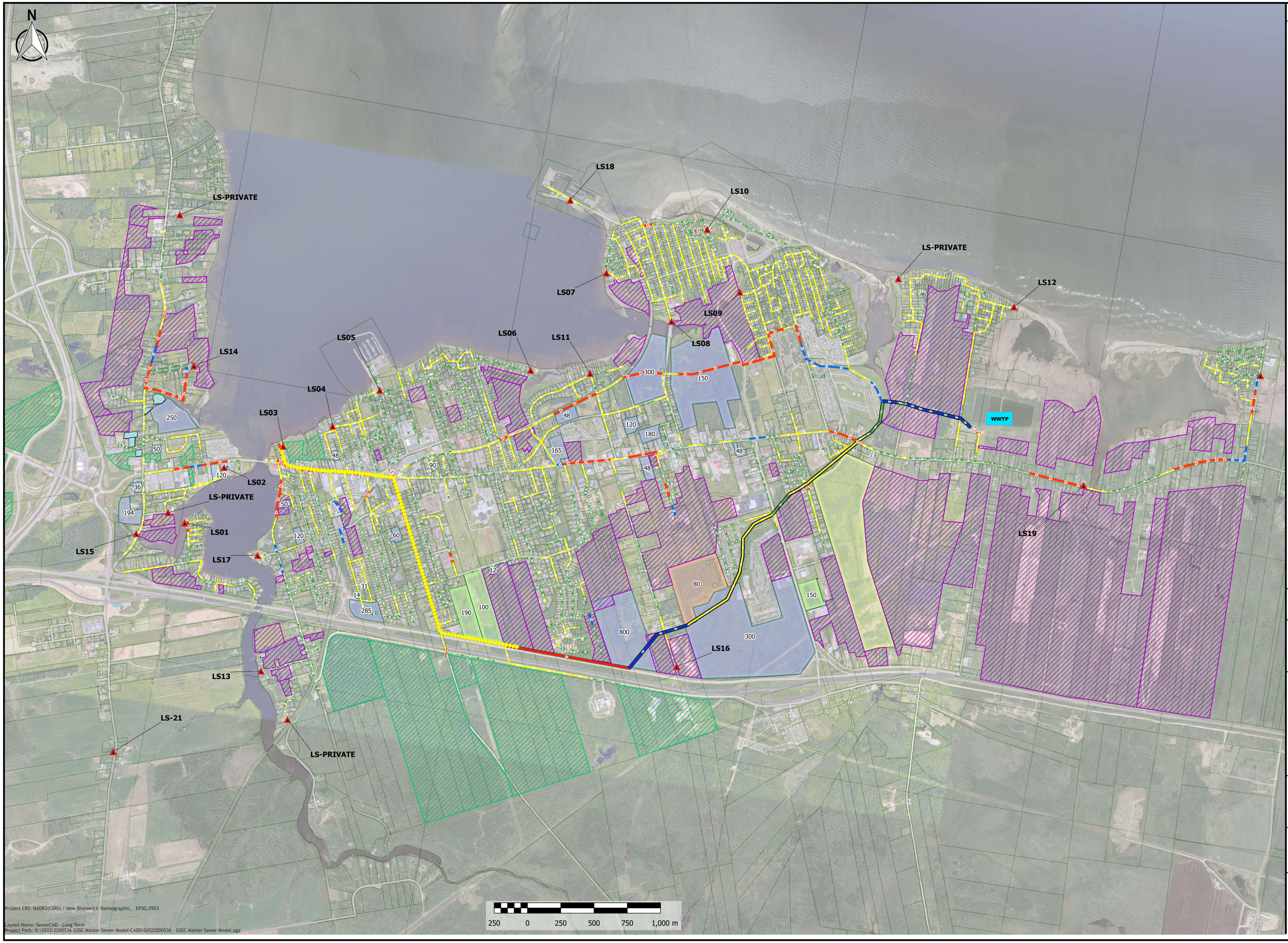
Revision:

DATE: 2022-12-22 BY: SAL

PROJ No: 2200536 APPR: SEB

GREATER SHEDIAC SEWERAGE COMMISSION

DES ÉGOUTS SHEDIAC ET BANLIEUES



Legend / Notes:

LEGEND

Non Plot

SewerCAD Legend

- HGL ≤ 100% of pipe dia
- HGL ≤ 100% of pipe dia. & less than 2.2m from ground level
- HGL > 100% ≤ 130% of pipe dia.
- HGL > 130% of pipe dia. & HGL > 2.2m from ground level
- HGL > 130% of pipe dia. & HGL < 2.2m from ground level

Project Location:

SHEDIAC, N.B.

Project Title:

SEWER SYSTEM MASTER PLAN

Map Title:

LONG TERM DEVELOPMENT-HGL CONDITION OF SANITARY NETWORK

Map ID:

MAP No: 3-4

PAGE No: 1 of 1

SCALE:

Revision:

DATE: 2022-12-22 BY: SAL

PROJ No: 2200536 APPR: SEB

GREATER SHEDIAC SEWERAGE COMMISSION DES ÉGOUTS SHEDIAC ET BANLIEUES

Legend / Notes:

LEGEND

Non Plot

SewerCAD Legend

- HGL ≤ 100% of pipe dia
- HGL ≤ 100% of pipe dia. & less than 2.2m from ground level
- HGL > 100% ≤ 130% of pipe dia.
- HGL > 130% of pipe dia. & HGL > 2.2m from ground level
- HGL > 130% of pipe dia. & HGL < 2.2m from ground level

Project Location:

SHEDIAC, N.B.

Project Title:

SEWER SYSTEM MASTER PLAN

Map Title:

EXISTING CONDITIONS 50% I&I REDUCTION - HGL CONDITION OF SANITARY NETWORK

Map ID:

MAP No: **4-1**

PAGE No: **1 of 1**

SCALE:

Revision:

DATE: 2022-12-22 BY: SAL

PROJ No: 2200536 APPR: SEB

GREATER SHEDIAC SEWERAGE COMMISSION
DES ÉGOUTS SHEDIAC ET BANLIEUES

Appendix B
Cost Estimates

Backlot Gravity Sewer Near Greenwood Promenade					
Item	Description	Unit	Estimated Quantity	Unit Price	Total Cost
1200 mm manhole dia.	New manhole with adjustable frame and cover inlcudes all connections and height adjustments	unit	2	$ 9,000.00	$ 18,000.00
Road Structure	Full Restoration of 4 m wide asphalt including 300mm thick of 0-75mm, 150mm thick of 0-31.5 , 100mm thick of type B Base Asphalt and 40mm thick of type D seal.	lin. M	4	$ 400.00	$ 1,600.00
Landscaping	Full Restoration, 6m wide of landscaping with 100 mm thick top soil and sod.	lin. M	10	$ 120.00	$ 1,200.00
				Sub-Total	$ 20,800.00
				Construction Contingencies (30%)	$ 6,240.00
				Engineering Services (15%)	$ 4,056.00
				HST (15%)	$ 4,664.40
				Total	**$ 35,770.00**

West Main and Dock Street					
Item	Description	Unit	Estimated Quantity	Unit Price	Total Cost
Gravity 250 mm dia.	Pipe Installation in a 3.0 m deep trench includes pipe, fittings, compaction, reinstallation of in-situ material	lin. M	400	$ 575.00	$ 230,000.00
Gravity 375 mm dia.	Pipe Installation in a 3.0 m deep trench includes pipe, fittings, compaction, reinstallation of in-situ material	lin. M	20	$ 675.00	$ 13,500.00
Gravity 450 mm dia.	Pipe Installation in a 3.0 m deep trench includes pipe, fittings, compaction, reinstallation of in-situ material	lin. M	210	$ 850.00	$ 178,500.00
1200 mm manhole dia.	New manhole with adjustable frame and cover inlcudes all connections and height adjustments	unit	15	$ 8,000.00	$ 120,000.00
100 mm dia. Lateral	Assumes 10m long and includes fittings, landscaping, road structure and asphalt driveway.	ea.	22	$ 7,000.00	$ 154,000.00
150 mm dia. Lateral	Assumes 10m long and includes fittings, landscaping, road structure and asphalt driveway.	ea.	5	$ 9,000.00	$ 45,000.00
Curb	Full Restoration of Curb on one side of the street.	lin. M	630	$ 120.00	$ 75,600.00
Road Structure	Full Restoration of 4 m wide asphalt including 300mm thick of 0-75mm, 150mm thick of 0-31.5 , 100mm thick of type B Base Asphalt and 40mm thick of type D seal.	lin. M	700	$ 400.00	$ 280,000.00
				Sub-Total	$ 1,096,600.00
				Construction Contingencies (30%)	$ 328,980.00
				Engineering Services (15%)	$ 213,837.00
				HST (15%)	$ 245,912.55
				Total	**$ 1,885,330.00**

2200536 - Sewer System Master Plan
Appendix B - Cost Estimates

Main Street and Caissie Avenue					
Item	**Description**	**Unit**	**Estimated Quantity**	**Unit Price**	**Total Cost**
Gravity 250 mm dia.	Pipe Installation in a 3.0 m deep trench includes pipe, fittings, compaction, reinstallation of in-situ material	lin. M	180	$ 575.00	$ 103,500.00
Gravity 300 mm dia.	Pipe Installation in a 3.0 m deep trench includes pipe, fittings, compaction, reinstallation of in-situ material	lin. M	155	$ 600.00	$ 93,000.00
1200 mm manhole dia.	New manhole with adjustable frame and cover inlcudes all connections and height adjustments	unit	6	$ 8,000.00	$ 48,000.00
150 mm dia. Lateral	Assumes 10m long and includes fittings, landscaping, road structure and asphalt driveway.	ea.	12	$ 9,000.00	$ 108,000.00
Curb	Full Restoration of Curb on one side of the street.	lin. M	350	$ 120.00	$ 42,000.00
Road Structure	Full Restoration of 4 m wide asphalt including 300mm thick of 0-75mm, 150mm thick of 0-31.5 , 100mm thick of type B Base Asphalt and 40mm thick of type D seal.	lin. M	350	$ 400.00	$ 140,000.00
				Sub-Total	$ 534,500.00
				Construction Contingencies (30%)	$ 160,350.00
				Engineering Services (15%)	$ 104,227.50
				HST (15%)	$ 119,861.63
				Total	**$ 918,940.00**

2200536 - Sewer System Master Plan
Appendix B - Cost Estimates

Main Street East					
Item	**Description**	**Unit**	**Estimated Quantity**	**Unit Price**	**Total Cost**
Gravity 450 mm dia.	Pipe Installation in a 3.0 m deep trench includes pipe, fittings, compaction, reinstallation of in-situ material	lin. M	245	$ 850.00	$ 208,250.00
1200 mm manhole dia.	New manhole with adjustable frame and cover inlcudes all connections and height adjustments	unit	4	$ 8,000.00	$ 32,000.00
100 mm dia. Lateral	Assumes 10m long and includes fittings, landscaping, road structure and asphalt driveway.	ea.	6	$ 6,650.40	$ 39,902.40
Curb	Full Restoration of Curb on one side of the street.	lin. M	245	$ 120.00	$ 29,400.00
Road Structure	Full Restoration of 4 m wide asphalt including 300mm thick of 0-75mm, 150mm thick of 0-31.5 , 100mm thick of type B Base Asphalt and 40mm thick of type D seal.	lin. M	245	$ 386.80	$ 94,766.00
				Sub-Total	$ 404,318.40
				Construction Contingencies (30%)	$ 121,295.52
				Engineering Services (15%)	$ 78,842.09
				HST (15%)	$ 90,668.40
				Total	**$ 695,130.00**

LS #2				
Item	**Description**	**Unit**	**Estimated Quantity**	**Total Cost**
Lift Station Upgrade	Full upgrade including new pumps, generator, building, wet well, piping, etc.	LS	1	$ 4,000,000.00
Forcemain 400mm dia.	Pipe Installation in a 3.0 m deep trench includes pipe, fittings,	lin. M	330	$ 280,500.00
Forcemain 400mm dia. HDD	Pipe installation with HDD, HDPE DR-11 includes all fittings, transition couplings, all costs related to HDD.	lin. M	50	$ 150,000.00
Road Structure	Full Restoration of 4 m wide asphalt including 300mm thick of 0-75mm, 150mm thick of 0-31.5 , 100mm thick of type B Base Asphalt and 40mm thick of type D seal.	lin. M	330	$ 132,000.00
			Sub-Total	$ 4,562,500.00
			Construction Contingencies (30%)	$ 1,368,750.00
			Engineering Services (15%)	$ 889,687.50
			HST (15%)	$ 1,023,140.63
			Total	**$ 7,844,080.00**

LS #5				
Item	Description	lin. M	Estimated Quantity	Total Cost
Lift Station Upgrade	Full upgrade including new pumps, generator, building, wet well, piping, etc.	LS	1	$ 1,000,000.00
Forcemain 150mm dia.	Pipe Installation in a 3.0 m deep trench includes pipe, fittings, compaction, reinstallation of in-situ material, thrust blocks	lin. M	675	$ 337,500.00
Road Structure	Full Restoration of 4 m wide asphalt including 300mm thick of 0-75mm, 150mm thick of 0-31.5 , 100mm thick of type B Base Asphalt and 40mm thick of type D seal.	lin. M	675	$ 270,000.00
			Sub-Total	$ 1,607,500.00
			Construction Contingencies (30%)	$ 482,250.00
			Engineering Services (15%)	$ 313,462.50
			HST (15%)	$ 360,481.88
			Total	**$ 2,763,700.00**

LS #14				
Item	Description	Unit	Estimated Quantity	Total Cost
Lift Station Upgrade	Full upgrade including new pumps, generator, building, wet well, piping, etc.	LS	1	$ 1,100,000.00
Forcemain 200mm dia.	Pipe Installation in a 3.0 m deep trench includes pipe, fittings, compaction, reinstallation of in-situ material, thrust blocks	lin. M	700	$ 420,000.00
Road Structure	Full Restoration of 4 m wide asphalt including 300mm thick of 0-75mm, 150mm thick of 0-31.5 , 100mm thick of type B Base Asphalt and 40mm thick of type D seal.	lin. M	700	$ 280,000.00
			Sub-Total	$ 1,800,000.00
			Construction Contingencies (30%)	$ 540,000.00
			Engineering Services (15%)	$ 351,000.00
			HST (15%)	$ 403,650.00
			Total	**$ 3,094,650.00**

LS #15				
Item	Description	Unit	Estimated Quantity	Total Cost
Lift Station Upgrade	Full upgrade including new pumps, generator, building, wet well, piping, etc.	LS	1	$ 1,000,000.00
Forcemain 150mm dia.	Pipe Installation in a 3.0 m deep trench includes pipe, fittings, compaction, reinstallation of in-situ material, thrust blocks	lin. M	355	$ 177,500.00
Road Structure	Full Restoration of 4 m wide asphalt including 300mm thick of 0-75mm, 150mm thick of 0-31.5 , 100mm thick of type B Base Asphalt and 40mm thick of type D seal.	lin. M	355	$ 142,000.00
			Sub-Total	$ 1,319,500.00
			Construction Contingencies (30%)	$ 395,850.00
			Engineering Services (15%)	$ 257,302.50
			HST (15%)	$ 295,897.88
			Total	**$ 2,268,560.00**

New Trunk Sewer				
Item	Description	Unit	Estimated Quantity	Total Cost
Forcemain 400mm dia.	Pipe Installation in a 3.0 m deep trench includes pipe, fittings, compaction, reinstallation of in-situ material, thrust blocks	lin. M	2800	$ 2,380,000.00
Gravity 600 mm dia.	Pipe Installation in a 3.0 m deep trench includes pipe, fittings, compaction, reinstallation of in-situ material	lin. M	3890	$ 4,084,500.00
Gravity 1050 mm dia.	Pipe Installation in a 3.0 m deep trench includes pipe, fittings, compaction, reinstallation of in-situ material	lin. M	700	$ 1,190,000.00
1500 mm manhole dia.	New manhole with adjustable frame and cover inlcudes all connections and height adjustments	unit	36	$ 360,000.00
1800 mm manhole dia.	New manhole with adjustable frame and cover inlcudes all connections and height adjustments	unit	5	$ 75,000.00
Curb	Full Restoration of Curb on one side of the street.	lin. M	20	$ 2,400.00
Landscaping	Full Restoration, 6m wide of landscaping with 100 mm thick top soil and sod.	lin. M	1080	$ 129,600.00
Trail	3m wide asphalt trail includes 150mm of 0-31.5mm crushed rock base, 50mm type D asphalt, 2m of landscaping.	lin. M	3950	$ 632,000.00
Road Structure	Full Restoration of 4 m wide asphalt including 300mm thick of 0-75mm, 150mm thick of 0-31.5 , 100mm thick of type B Base Asphalt and 40mm thick of type D seal.	lin. M	2250	$ 900,000.00
LS 3 & LS 4 Upgrades	Includes upgrades to existing piping, valves, fittings and pumps	LS	1	$ 2,500,000.00
			Sub-Total	$ 12,253,500.00
			Construction Contingencies (30%)	$ 3,676,050.00
			Engineering Services (15%)	$ 2,389,432.50
			HST (15%)	$ 2,747,847.38
			Total	**$ 21,066,830.00**