Combined Sewer Overflow Reduction Strategy
Eastern Portion of EWWTF Collection System

Draft Report

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Prepared for:

The City of Saint John

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Glossary of Terms

Average Dry Weather Flow (ADWF)	*Average flow that occurs in a sewer during dry weather.*
Baseflow	*Minimum flow that occurs in the night. In a predominantly residential area it is assumed that the baseflow is comprised mostly of infiltration.*
CCME	*Canadian Council of Ministers of the Environment*
CSO	*Combined Sewer Overflow*
Development Boundary	*Limits of a planned service area for a wastewater collection system and/or treatment plant.*
Diurnal Curve	*Characteristic flow pattern including two peaks, in the morning and evening.*
Hydraulic	*Calculations related to the flow of stormwater and wastewater through the system of pipes, pumping stations, flow regulators and outfalls.*
Hydrologic	*Calculations related to the generation of stormwater and wastewater in the sewers.*
Design Flow	*Estimation of Hydraulic Design Flows based on the procedure outlined in the Atlantic Canada Standards & Guidelines for the Collection, Treatment & Disposal of Sanitary Sewage.*
Extraneous Flows	*Infiltration plus inflows.*
Infiltration	*Groundwater that enters the sanitary sewer through cracks and other leaks in the pipes and manholes of the system as well as any foundation drains that are connected to the sanitary sewer laterals. Magnitude changes by season, highest in wettest months of spring and fall.*
Inflows	*Stormwater that enters the sanitary sewer by a direct path.*
Rapid Inflows	*Most direct runoff connections to the sanitary sewer system, start shortly after rain starts and finish shortly after rain stops. Examples include: flows through manhole covers, cross connections to storm sewers or drains and roof leaders connected directly to the sanitary sewer.*
Slow Inflows	*Less direct connection, may continue for several days following end of rain. Generated from indirect connections of runoff such as roof leaders that are connected to the foundation drainage that is connected to the sanitary sewer lateral or surface drains with restricted inlets.*
Real time control	*Operation of a system in response to measured or anticipated conditions in the system. Typically used to achieve improved utilization of infrastructure.*
SSO	*Sanitary Sewer Overflow*
Serviceable Boundary	*Limits of a service area for wastewater infrastructure.*

Sewershed	*Area that contributes wastewater to a particular point of interest in the wastewater collection system. Typically closely associated with the natural topography, additional areas may be added by pumping. In this study there are existing areas already contributing as well as areas that could potentially contribute if development proceeds in these areas.*
Sub-sewershed	*Sub-area of the sewershed, typically delineated due to differences in the collections system, such as year constructed or material of construction or type of development.*

CHAPTER 1 ABOUT THE STUDY

1.1 Introduction

Wastewater collection systems that service the east side of Saint John frequently experience excess flows during wet weather. This results in overflows from the wastewater collection systems into the storm drainage systems, typically into nearby watercourses. Wastewater in natural drainage systems causes concerns for many stakeholders including the City, the public, and the regulators (provincial and federal). New guidelines from the Canadian Council of Ministers of the Environment (CCME) require assessment of environmental risks associated with all wastewater discharges (including overflows from collection systems) and the development of plans to reduce these risks.

The City of Saint John requested proposals for the provision of engineering services associated with the development of a CSO Reduction Strategy for the areas serviced by the new Eastern Wastewater Treatment Facility Collection System. The original Request for Proposals (RFP) for this assignment was received from the City by email on October 1, 2009. The scope was modified in a revised proposal submitted in March 2010 to include only the eastern portion of the original study area. The modified Study Area is shown in Figure 1.1

1.2 Background

Many of the sanitary sewers adjacent the Marsh Creek have been subjected to flooding during periods of heavy rainfall. Inflow and Infiltration studies carried out by Hydro-Com Technologies in 2004 through 2007 indicated that storm flows as high as twenty times the average dry weather flow were recorded in several instances. Serious surcharging and flooding occurred in many areas during rainfall intensities less than those corresponding to a return period of one in two years. These studies suggested that extraneous flows are significant in the sewers that service the study area, this is due in part to the types of sewers present and their current condition.

Central wastewater collection systems in the western part of the study area are combined sewers, carrying both sanitary sewage and stormwater in common conduits. Trunk sewers were generally installed to service local drainage areas and discharged to Courtenay Bay at the nearest convenient location. This practice continued until the late 1960's and resulted in several outfalls along the shore. Since then, there have been several projects in the combined sewersheds involving separation of sewers, typically in conjunction with street reconstruction projects.

As development proceeded east of Marsh Creek and Courtenay Bay, separate sanitary sewers were installed. Storm drainage was generally conveyed in natural systems (Marsh Creek, Little River and Hazen Creek) as well as open ditches and culverts. The practice of the day was to install a single sanitary lateral to each building. Foundation and roof drainage was to be discharged to neighbouring ditches or watercourses.

Increasingly stringent standards have been adopted by the City for the design and construction of sanitary sewers and services in an attempt to reduce the potential for extraneous flow development. Dual storm and sanitary piped systems became the norm. Improved pipe and bedding materials were employed. More conservative hydraulic design procedures were generally adopted for the calculation of carrying capacity of the conduits; current designs are based on the Atlantic Canada Standards. The responsibility for provision of complete subdivision servicing was shifted to the developers and the City ensured that the improved design and construction standards were followed through the approval process.

Like many older North American coastal cities, the sewage collection systems in Saint John were built in accordance with the design practices of the day and followed the logical development progression, generally starting near the shoreline and moving inland. However, as a consequence the newer collection systems tend to be at the upper reaches of the overall system. Sanitary sewage collected in a fully separate upstream system often flows down through areas that are subject to more extraneous flows and/or finally into the old combined systems in some areas.

Recently the City has embarked on projects to reduce the discharge of wastewater from the city centre and the east side into the Harbour, including Courtenay Bay. These projects will comprise the wastewater interceptor and treatment system that delivers flows to the new treatment system for east Saint John and include:

- The new Eastern Wastewater Treatment Facility and outfall and diffuser that will treat wastewater flows generated in the eastern portion of the City and discharge the treated effluent into the Harbour;
- The Thorne Avenue sewage lift station (SLS) and forcemain along Bayside Drive that will deliver most of the flows generated in the central portion of the City as well as the areas tributary to the trunk sewers along Marsh Creek to the treatment plant;
- Gravity sewers, sewage lift stations and forcemains to convey wastewater from all of the outfalls along Red Head Road, Bayside Drive, Marsh Creek, Crown Street, Broad Street, the foot of Chesley Drive and Station Street, including the outfalls near Fort LaTour to the Thorne Avenue SLS and/or the forcemain to the treatment plant.

These facilities are being designed to intercept, as a minimum, the peak dry weather flow from each of the major catchment areas currently discharging to the Harbour and Courtenay Bay. Flows in excess of the regulated amount at each interceptor site will be discharged to the existing outfalls as combined sewer overflows (CSOs) or sanitary sewer overflows (SSOs). However, once flows enter the interceptor system, there should be no opportunity for overflows to occur so all flows will normally be treated at the treatment plant.

To address changing regulatory requirements, the City would like to develop a CSO reduction strategy to reduce or eliminate these overflows over time. The City realizes that there needs to be significant work completed to the existing system in order to meet or maintain the current CCME guidelines (which the Province of New Brunswick has also endorsed). These guidelines include the following national standards:

Combined Sewer Overflows

1. No increase in combined sewer overflow frequency due to development or redevelopment, unless it occurs as part of a combined sewer management plan
2. No combined sewer overflow discharge during dry weather, except during spring thaw or emergencies; and
3. Removal of floatable materials where feasible.

Sanitary Sewer Overflows

1. No increase in sanitary sewer overflow frequency due to development or redevelopment, unless it occurs as part of a combined sewer management plan
2. No sanitary overflow discharge during dry weather, except during spring thaw or emergencies.

Included with the national standards, it is required to have a long term plan in place by December 31, 2016 that will ultimately allow the wastewater collection system to meet and maintain national standards and to allow development to proceed.

Development must be preceded by measures to reduce SSOs and CSOs. Determination of the most cost effective measures in each sub-sewershed in the eastern portion of the Eastern Wastewater Treatment Facility Sewershed and an implementation plan to coincide with planned development is the main focus of this study.

1.3 Project Scope and Objectives

There are two studies identified in the Term of Reference for this project:

- Develop a combined sewer overflow reduction strategy for the Eastern Wastewater Treatment Facility Sewershed;
- Develop a sewer separation and relocation plan for the sewers near the Shamrock Park recreation facilities.

This report addresses the first objective listed. A separate report entitled "Shamrock Park Sewer Separation – Preliminary Design Report" was prepared and submitted under separate cover for the Shamrock Park sewer separation assessment.

1.3.1 Systems Included in the Assessments

Overflows from wastewater collection systems generally result from:

- Interruption of power supply;
- Failure of pumping equipment;

- Excessive flows in the sewers, typically stormwater.

The first two causes of overflows are typically addressed during the design of the stations by providing backup power to supply power during power failure as well as redundancy of pumping equipment in the station. To address overflows due to excessive flows in the sanitary sewer requires a comprehensive assessment of the flows generated in the sanitary and storm sewers as well as detailed hydraulic assessments of both sanitary and stormwater collection systems, including pumping systems.

1.3.2 Areas Included in the Assessments

The Study Area includes all areas serviced by sewers tributary to the Eastern Wastewater Treatment Facility (EWWTF) east of Rothesay Avenue, Russell Street and Bayside Drive. The relevant tributary systems include:

- Watersheds in the study area are shown on Figure 1.3.2 (a).
- Figure 1.3.2 (b) (i) presents the major sewersheds included within the study area as well as information of the existing sewage systems and existing overflow locations. A schematic of the systems is shown in Figure 1.3.2 (b) (ii). It presents the main trunk sanitary sewers in each sewershed and the areas tributary as well as the new interceptor system that connects these sewersheds.

1.3.3 Objectives

There were two main objectives for this study:

1. Create a model of the sanitary, storm and combined sewer systems in the study area;
2. Develop strategies for reduction of overflows in each sewershed in the study area.

1.4 Approach to the Study

The general approach taken in the completion of this study was as follows:

- Develop an understanding of the existing collection systems based on the best available information on the system make-up and on the flows that reach each system and must be conveyed;
- Identify capacity deficiencies within the existing systems;
- Assess available techniques for remedying the capacity deficiencies and/or reducing the impacts of discharging excess flows to the environment as well as assess their applicability to each collection system;
- Recommend the most cost effective measures for the short term as well as the long term.

This report is arranged as follows:

- Chapter 2 describes the existing combined sewers, sanitary sewers, and storm sewers as well as the locations of overflows and the receiving waters where they discharge.
- Chapter 3 describes the assessments completed in the development of computer based hydrologic and hydraulic models of the wastewater and stormwater collection systems and assessment of existing system performance;

- Chapter 4 discusses potential water use objectives for the overflow receiving waters and presents water quality objectives associated with these water use objectives. It finishes by suggesting water use objectives for each of the receiving waters;
- Chapter 5 first presents a detailed plan for sewer separation in the study area, in fulfillment of the City's current policy. It requires the construction of new sanitary sewers (as well as deep storm sewers in some areas) on an on-going basis to eventually achieve and maintain completely separate stormwater and sanitary sewers. These are long term goals. The chapter goes on to summarize the investigations, assessments and comparisons made of various alternative strategies for reducing overflows;
- Chapter 6 summarizes the recommended approaches to be taken to reduce overflows in each sewershed in a cost effect manner, that is consistent with current guidelines and allows development to continue;
- Chapter 7 outlines the development of preliminary designs for key components of the recommended approaches.

Combined Sewer Overflow Reduction Strategy - EWWTF Collection System

Study Area

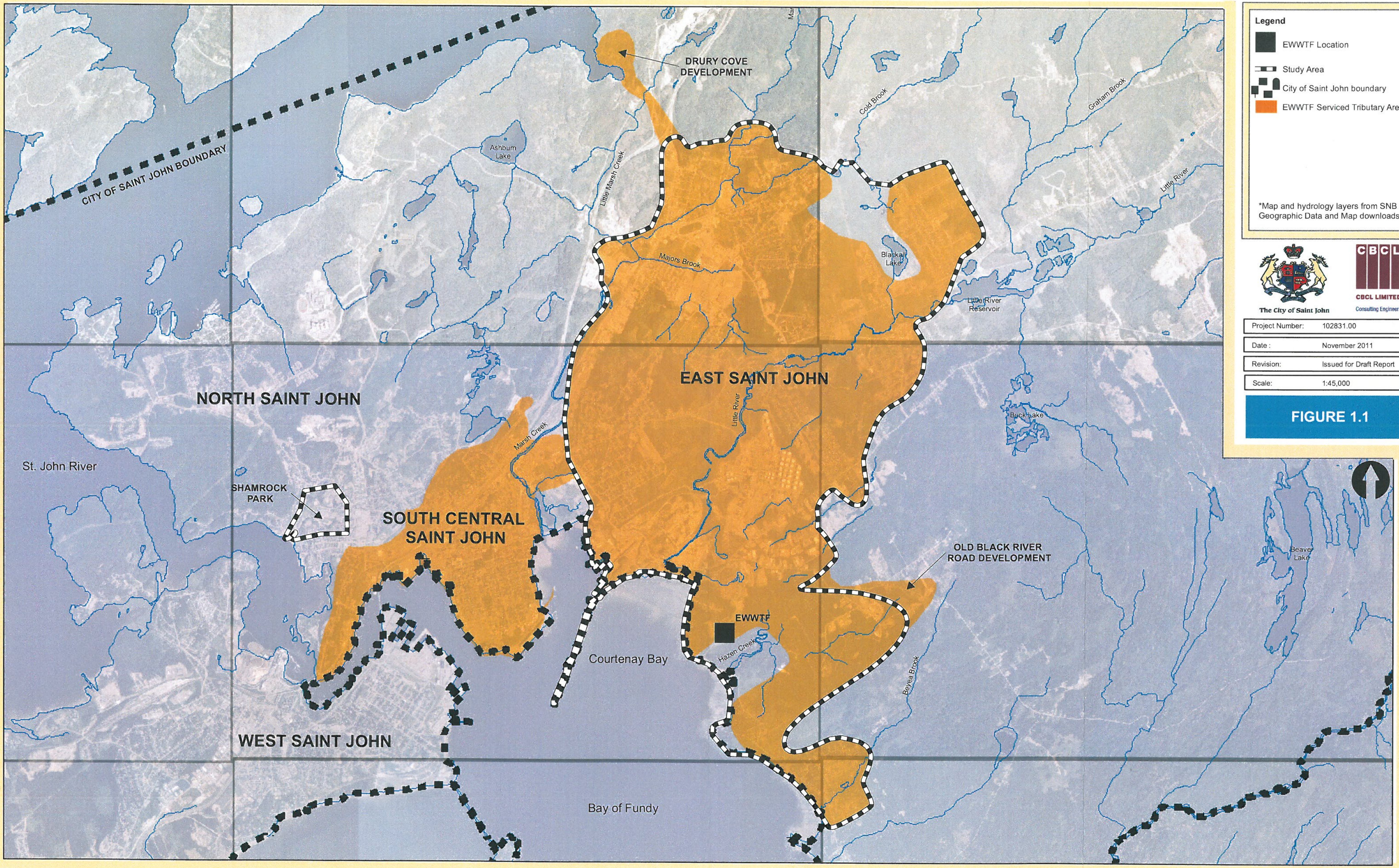

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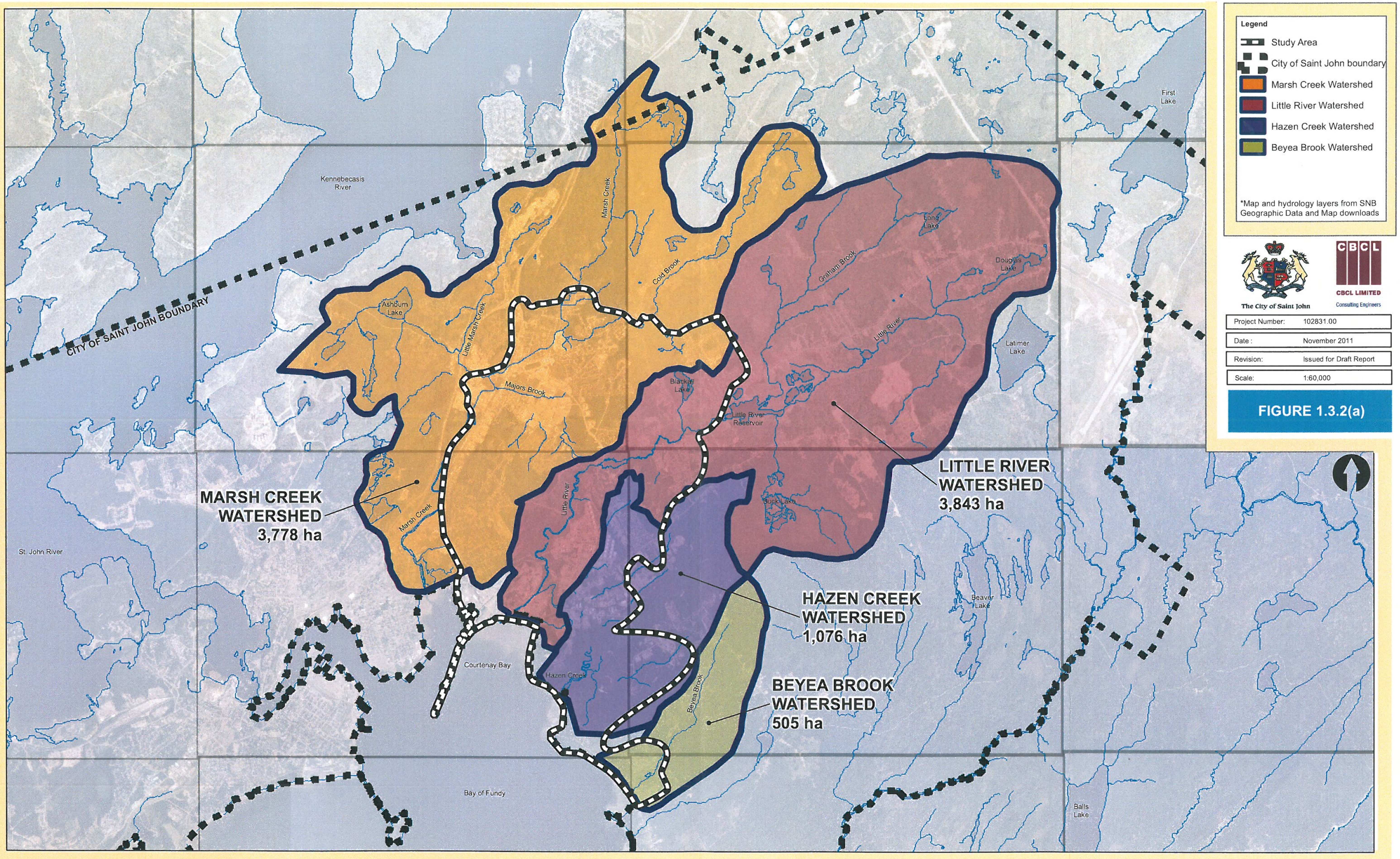
FIGURE 1.1

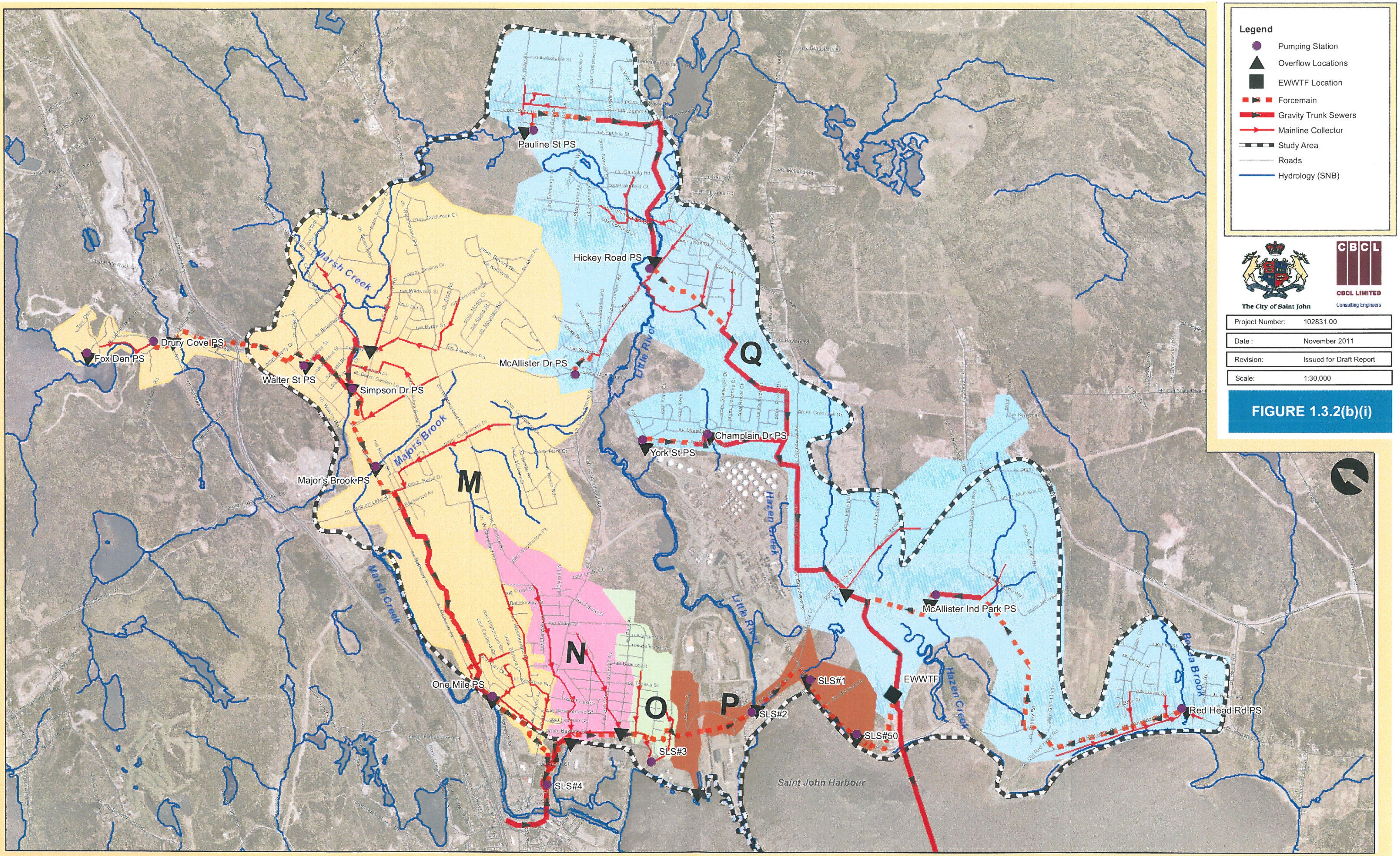
Combined Sewer Overflow Reduction Strategy - EWWTF Collection System

Watersheds Map



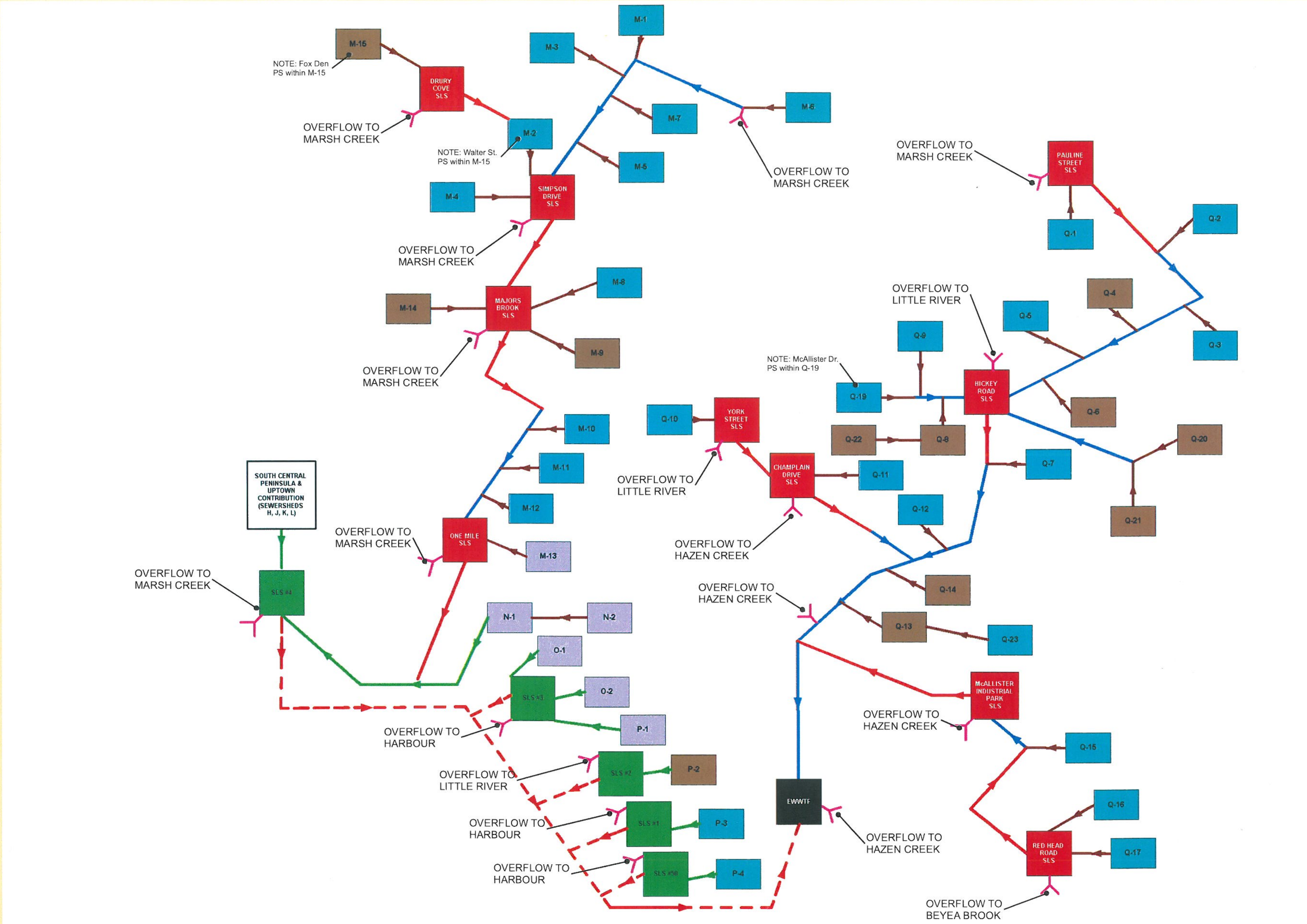
Combined Sewer Overflow Reduction Strategy - EWWTF Collection System

Major Sewersheds Map



Combined Sewer Overflow Reduction Strategy - EWWTF Collection System

Sanitary System Schematic - EAST



Legend

- EXISTING SLS
- HARBOUR CLEANUP SLS
- EASTERN WASTEWATER TREATMENT PLANT
- COMBINED STORM AND SANITARY SEWERS
- SEPARATE STORM AND SANITARY SEWERS
- SUSPECTED WET SANITARY SEWERS
- EXISTING FORCEMAIN
- FORCEMAIN (HARBOUR CLEANUP)
- EXISTING SANITARY TRUNK SEWER
- SANITARY SEWER (HARBOUR CLEANUP)
- EXISTING GRAVITY SEWER
- OVERFLOW LOCATION

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FIGURE 1.3.2(b)(ii)

CHAPTER 2 EXISTING SYSTEMS

2.1 General

In order to effectively assess alternatives to reduce overflows from the sanitary and combined sewers in the study area, the existing systems must be fully understood. This requires an understanding of:

- The make-up of each system;
- The flows that reach each system;
- The system's performance capabilities or ability to accommodate the flows it receives.

Once the system is understood, then modifications to the system or additions to the system to enhance performance may be assessed.

2.2 Tributary Areas

2.2.1 Study Area Watersheds

Four watersheds drain through the study area and ultimately to the Saint John Harbour. During wet weather, drainage that enters the combined and sanitary systems is relieved at designed overflow locations. The location where each overflows discharge is shown on Figure 1.3.2(a). Descriptions of the four watersheds are provided below.

2.2.1.1 MARSH CREEK WATERSHED

The Marsh Creek watershed is a 3778-hectare drainage basin that begins at the upper reach of the Renforth bog and extends to the Saint John Harbour through an outlet at the Courtenay Bay causeway. This distance is approximately 11 km and is made up of various developments including residential, multi-unit apartments, heavy industrial, and commercial. There are various drainage networks included within the basin – smaller brooks and streams like Majors Brook and Cold Brook that flow into Marsh Creek, as well as various storm sewers that outfall to Marsh Creek from the lower developed areas.

There are 7 overflow locations from the sanitary and combined sewers within the watershed. Figure 2.2.1 (a) shows the overflow locations and the adjacent storm systems

2.2.1.2 Little River Watershed

The Little River watershed is a 3843-hectare drainage basin that begins at its upper reaches adjacent the Saint John Airport location and extends to the Saint John Harbour with a total length of approximately 14.2 km. The watershed is made up of various developments including residential, multi-unit apartments, heavy industrial and commercial. There are various drainage networks included within the basin including smaller brooks and streams that flow into Little River, as well as various storm sewer infrastructure that drain to Little River from the developed areas.

There are 4 overflow locations within the watershed. Figure 2.2.1 (b) shows the area of the drainage basin included in the study area as well as the existing tributary storm sewer networks.

2.2.1.3 Hazen Creek Watershed

Hazen Creek Waterhed is a 1076-hectare drainage basin that outfalls to the Saint John Harbour adjacent to the current EWWTF location on Red Head Road. The watershed extends approximately 5.7 km from the upper section of Grandview Avenue to its outlet at the harbour. It includes mostly heavy industrial development with small amounts of institutional (NBCC Saint John) and residential (Champlain Heights) developments. Included with these developments are various storm sewer networks that drain to Hazen Creek.

There are 3 overflow locations within the watershed. Figure 2.2.1(c) shows the area of the drainage basin included in the study area as well as the existing tributary storm sewer network.

2.2.1.4 Beyea Brook Watershed

Beyea Brook Watershed is a 505-hectare drainage basin that outfalls to the Saint John Harbour. The watershed extends approximately 4.6 km from its uppermost point to its outlet at the harbour. Most of the watershed is currently undeveloped. At the lower portion of the area there is residential development including the Ocean Drive Subdivision off of Red Head Road. Included in this development is a storm sewer network that outfalls to Beyea Brook south-east of the development. Figure 2.1.2(d) shows the area of the drainage basin included in the study area as well as the existing tributary storm sewer networks.

2.2.2 Study Area Sewersheds

A description of the various wastewater collection areas encompassing the study area is provided below and are illustrated on Figure 1.3.2(b)(i):

2.2.2.1 Sewershed M – Marsh Creek

Sewershed M is an 885-hectare drainage zone that generally follows the Marsh Creek Valley and is currently serviced by the Thorne Avenue Wastewater Treatment Facility (WWTF). While the topography of the Forest Hills and Silver Falls subdivisions is high, the majority of the catchment is low lying and prone to flooding. Stretching over 5½ km from the catchment boundary to the outfall, this major area includes commercial areas such as the businesses along Rothesay Avenue and McAllister Drive as well as the Forest Hills, Glen Falls, Silver Falls Park subdivisions and Drury Cove subdivision. The existing infrastructure consists of six sewage lift stations. The stations are located at Drury Cove subdivision (2 stations), Walter Street, Simpson Drive, Rothesay Avenue (Majors

Brook), and Russell Street. Wastewater currently is transported through force mains and gravity sewer lines to the Thorne Avenue WWTF adjacent to Bayside Drive and Thorne Avenue. During Harbour Cleanup the Thorne Avenue WWTP will be decommissioned and sewage will be conveyed to the new Marsh Creek Sewage Lift Station (SLS #4), which will pump sewage along the entire length of Bayside Drive to ultimately discharge at the EWWTF. This system currently has seven overflow locations that ultimately discharge to Marsh Creek.

2.2.2.2 Sewershed Q – Hazen Creek

Sewershed Q is a 1173-hectare drainage zone that extends over 7 km from the catchment boundary at the Lakewood Heights subdivision to the Eastern Wastewater Treatment Facility (EWWTF) headworks. Major users within this catchment include the Grandview and McAllister Industrial Parks as well as the Champlain Heights, Red Head and Lakewood Heights subdivisions. The existing infrastructure consists of gravity sewer lines, seven sewage lift stations, and force mains collecting and transporting wastewater to the treatment plant. This system currently has one overflow location that discharges to Marsh Creek, two overflow locations that discharge to Little River ; three overflows that discharge to Hazen Creek and one that discharges to Beyea Brook.

2.2.2.3 Sewershed N – Westmorland

Sewershed N is a relatively small catchment compared to sewersheds M and Q, covering 138-hectares and is located in East Saint John between Westmorland Road and Mt. Pleasant Avenue. While primarily made up of residential users, several major institutional users including Bayside Middle School, the Loch Lomond Villa and the Loch Lomond Mall are located within the catchment. The catchment drains to a single major outfall discharging untreated sewage into Dutchman's Creek through a 1200x1500 wooden sewer, then to Marsh Creek just upstream of the Courtenay Bay causeway.

The flow from Area N will be piped by gravity during Harbour Cleanup to the proposed Marsh Creek SLS #4, a short distance from the Area N outfall. There will be an overflow location for SLS#4 when it is constructed that will discharge to Marsh Creek. Crandall Engineering Ltd., as a part of Harbour Cleanup, has installed a new sanitary sewer along Bayside Drive from Park Street northward to the existing 1200mm diameter outfall across from Bayside Middle School. This sewer eliminates all "dry weather" sanitary sewer flow into the existing storm sewer on the west side of Bayside Drive which currently discharges into Dutchman's Creek. It should be noted that because the sewer flows at the base of Park Street are part of a combined sewer system there will be an overflow from this manhole into the existing storm sewer on Bayside Drive, this is necessary to handle significant rainfall events until this system can be separated, at that time this overflow will be removed.

2.2.2.4 Sewershed O & P – Mount Pleasant Avenue and Bayside Drive

Sewersheds O and P (43 and 71-hectares respectively) are two drainage zones located south of the Courtenay Bay Causeway running along Bayside Drive. Residential, commercial, and large industrial lands are all included in the areas and include such users as the Irving Wallboard Plant, Irving Paper, Irving Oil Refinery, and Bayside Power. Irving Paper, Bayside Power and the Irving Oil Refinery each provide onsite wastewater treatment for ultimate discharge into Saint John Harbour or Little River. The remaining untreated sewage from these areas will be intercepted during Harbour Cleanup by

SLS #1, 2, 3, and 50 which will all pump directly into the 600mm diameter forcemain along Bayside Drive (installed in 2010, Harbour Cleanup) and then ultimately to the EWWTF. There will be four overflow locations once the Harbour Cleanup interceptor components listed above are commissioned. Three of the overflows will be to the Harbour and one to the outlet of Little River.

2.3 Collection Systems

Information about the existing sanitary and storm drainage systems was collected from several sources:

2.3.1 Existing Data

Information about the existing wastewater and stormwater collection systems was gathered from sources including the City's GIS databases as well as Record Drawings where available. Information collected included:

- Location and depth to all pipes entering or leaving each structure;
- Pipe and manhole dimensions and materials;
- Locations and configuration of pumping stations as well as pump data;

This information was used to develop a "Project GIS" for this study. It contains the information collected that is needed for the development of the hydraulic models including information on the size and material of pipes, pipe invert and top elevations of all manholes, as well as information on wastewater generation in each sewershed and the size, slope and permeability of all watersheds.

2.3.2 Structures Survey

Information was collected by topographic survey for critical collection system components that did not have information available from the City GIS or Record Drawings. The survey data is also included in the project GIS.

2.3.3 Summary of Collection Systems Information

A summary of components that comprise the existing sanitary and storm systems in presented by overflow sewershed in Table 2.3.2. The collection systems tributary to the new Eastern Wastewater Treatment Facility are comprised of three general standards of service, namely:

- *The older and largely combined systems on the western portion of the study area:* These used to discharge directly to Courtenay Bay, but a portion will eventually be intercepted and pumped to the new treatment plant. Peak flows in the combined systems can be as much as 100 times the dry weather flows during significant storm events. Excess flows will overflow (CSOs) at the proposed interceptor lift stations through the original outfalls to:
    - Fresh water where the discharges are upstream of the causeway or;
    - To salt water where the discharge is directly to Courtenay Bay;
- *The systems along Rothesay Avenue and through Champlain Heights:* These systems conveyed wastewater to treatment plants on Thorne Avenue and at Hazen Creek. They were originally installed as separate sanitary sewers but with limited piped storm drainage in these areas they have become fraught with inflow and infiltration resulting in sanitary sewer overflows at the pumping stations in the systems. Peak flows in the older sanitary systems can exceed 20 times

dry weather flow. Flows in excess of the system capacity (typically the pumping station capacity) overflows, into freshwater drainage systems;

- *Separate storm and sanitary systems serving newer developments and built to a more rigid design standard:* The sanitary sewer systems in these areas should not overflow on a regular basis. The newer well-constructed sanitary systems are much less influenced by precipitation and peak flows are normally less than 4 to 5 times the dry weather flow. Overflows may still occur due to equipment malfunction at pumping stations but on an infrequent basis.

Relevant information about the existing sanitary and storm water collection systems in each subsewershed is shown on Figure 2.3.3. The various combinations of services (described above) are further differentiated by pipe materials used in construction of these systems as well as the depths of the sewers. Each affects the inflow and infiltration generating potential of the sewage collection system. Combinations of services in the study area are listed in Table 2.3.3.

Table 2.3.3 Combinations of Services in the Study Area

Services Combination	Sanitary	Deep Storm	Shallow Storm	Typical Lateral Services
1	Concrete Combined Sewer			Combined
2(a)	Concrete		Roadside Ditches	Separate
2(b)	Concrete		Shallow Storm Sewer	Combined
2(c)	Concrete	Deep Storm Sewer		Separate
3(a)	PVC		Roadside Ditches	Separate
3(b)	PVC		Shallow Storm Sewer	Combined
3(c)	PVC	Deep Storm Sewer		Separate

Locations in the study areas where these combinations of services exist are shown in Figure 2.3.3.

Although specific information on sewer laterals was not always available, experience with service laterals elsewhere in the City indicates that typically there are separate storm and sanitary sewer laterals when there are roadside ditches or deep storm sewers available when the properties are developed. Otherwise the service lateral is typically a single pipe conveying sanitary as well as storm drainage from the property.

2.4 Flow Monitoring Data

Flow monitoring was completed at several sites in the study area as part of previous studies to determine the extent and magnitude of extraneous flow issues in the sanitary sewers. These reports were reviewed and the findings summarized in the Flow Monitoring Summary in Appendix B. Following is a summary of the additional analysis of the data completed as part of this study.

2.4.1 Locations of Previous Flow Monitoring

Figure B1 (in Appendix B) shows the location of each flow monitoring site, the study in which it was completed and the area tributary to each monitoring site. Assessment of the flows from these sites provided information on stormwater flow generation for approximately 65 percent of the current study area.

2.4.2 Design Flows Compared to Observed

New sanitary sewers are designed to accommodate the design flows generated using the method outlined in the *Atlantic Canada Standards & Guidelines for the Collection, Treatment & Disposal of Sanitary Sewage*. This method uses typical flow generation rates based on the equivalent population serviced. Drawing B3 (in Appendix B) shows a comparison of average flows (based on typical generation rates times equivalent populations) compared to measured average flows. The greater the ratio between the estimated and observed values, the greater the probability that extraneous flows (typically infiltration) may be a concern.

In the method outlined in the Atlantic Canada Standards and Guidelines Manual, the average flow is multiplied by a peaking factor (also based on equivalent population) as well as an allowance for extraneous flows based on the gross area serviced. Comparison of design flows to peak flows observed during the flow monitoring programs provides an indication of the type of extraneous flows that might enter the wastewater collection system. The higher the ratio of measured peak flow to design flow, the larger the number of direct inflow sources there are in the system. Drawing B4 (in Appendix B) shows the findings of this assessment.

Drawing B2 (in Appendix B) shows the results of an assessment of the portion of rainfall that entered the sanitary or combined sewer during the most significant rainfall event of the flow monitoring period in each gauging area. Comparison of the results in Drawing B2 (in Appendix B) with the servicing configurations shown in Figure 2.3.3 provides insight into their stormwater generating capabilities. The various combinations of sanitary and storm sewers are ranked in this respect in the following table:

Table 2.4.2 Relative Amounts of Stormwater in the Combinations of Services in the Study Area

Services Combination	Sanitary	Deep Storm	Shallow Storm	Rank
1	Concrete Combined Sewer			1
2(a)	Concrete		Roadside Ditches	3
2(b)	Concrete		Shallow Storm Sewer	2
2(c)	Concrete	Deep Storm Sewer		4
3(a)	PVC		Roadside Ditches	6
3(b)	PVC		Shallow Storm Sewer	5
3(c)	PVC	Deep Storm Sewer		7

Notes: 1. A ratio of 4 times the dry weather flow is typically used as a design interception rate from combined sewers so is used in this assessment.

2. A ranking of 1 indicates that the sanitary sewer receives the most stormwater, 7 the lowest amount.

2.5 Overflows

The locations of all known overflows from the sanitary sewers in the study area are shown in Figure 2.5. Where the available capacity is less than the flows generated in the upstream system the sewers will surcharge and water levels rise. Where there is an opening to the receiving environment as there typically is at pumping stations, an overflow will occur if the water level in the receiving environment is below the surcharge level in the sewers.

2.5.1 SSOs

Overflows are typically located at wastewater pumping stations to minimize the risk of surcharging upstream sewers. By default, this is usually where overflows from sanitary sewers occur. In some extreme cases, overflows from the sanitary sewers are created as cross connections to the storm sewers.

2.5.2 CSOs

Combined sewers convey much larger flows than strictly sewage flows during wet seasons when groundwater is high and during rainfall events. Combined sewer overflows (CSOs) are created where an interceptor system is designed to convey only a portion of the peak flows in the combined sewer (it is typically not economically feasible to construct interceptor systems to convey the peak flows in these systems). Typically flows in excess of the interceptor capacity overflow to the closest receiving waters. Overflows from combined sewer systems typically occur more frequently, create larger peak flows and larger overflow volumes than overflows from sanitary sewers.

2.6 Receiving Waters

Receiving waters for the existing overflows from the storm and sanitary sewer are shown on Figure 2.6 and listed as follows:

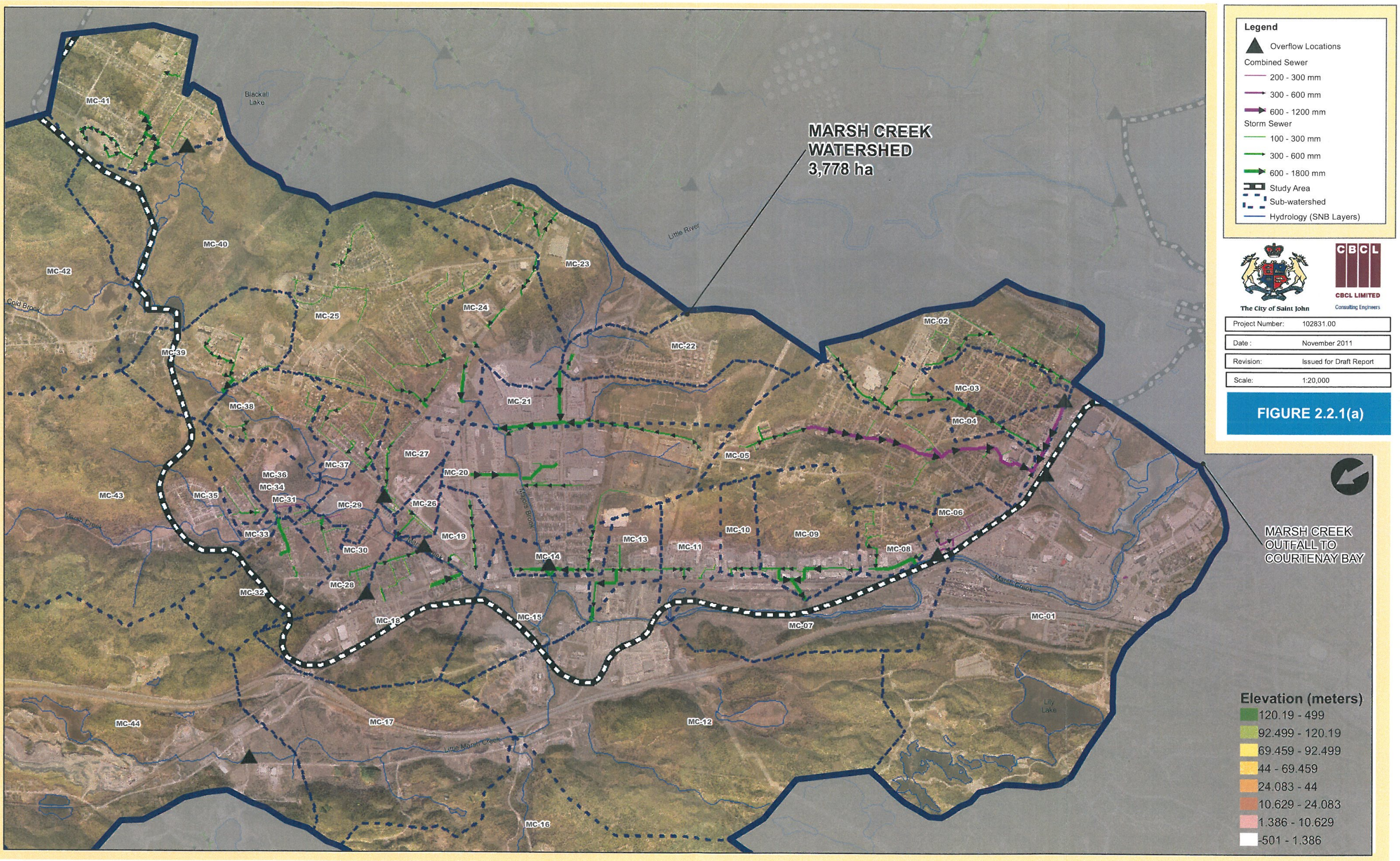
- Courtenay Bay - Saint John Harbour – Courtenay Bay receives flows from Marsh Creek, and the Little River as well as Hazen Creek where the Bay meets the Harbour. In addition, it receives overflows from Sewershed areas N, O, and P shown in Figure 1.3.2 (b) (i);
- Marsh Creek – The creek receives urban runoff, storm sewer discharges and overflows from sewershed M;
- Little River – The creek receives urban runoff, storm sewer discharges and overflows from the upstream areas of sewershed Q;
- Hazen Creek – The creek receives urban runoff, storm sewer discharges and overflows from the downstream areas of sewershed Q;
- Beyea Brook – The creek receives urban runoff, storm sewer discharges and overflows from the Red Head Road Pumping Station in the furthest downstream section.

Table 2.3.2 - Estimates of Existing Infrastructure Tributary to each Overflow

		Estimates of Existing Tributary Infrastructure Between Overflows					
Overflow Location	Overflow Label	Sanitary/ Combined Manholes (#)	Storm Manholes (#)	Catch Basins (#)	Sanitary/ Combined Sewer (m)	Service Laterals	Storm Sewer (m)
Sewershed M		**683**	**449**	**524**	**38483**	**2629**	**25634**
Fox Den PS	OF-M15.1	5	1	6	214	8	279
Drury Cove PS	OF-M15.2	36	18	11	1782	60	923
Parkhill Dr	OF-M06	160	109	133	9324	677	6818
Walter St PS	OF-M02	-	-	-	-	-	-
Simpson Dr PS	OF-M05	219	179	234	12361	821	10098
Major's Brook PS	OF-M14	43	37	43	2051	351	2266
One Mile PS	OF-M13	220	105	97	12751	712	5250
Sewershed N		**202**	**87**	**128**	**10515**	**1033**	**5017**
Park Avenue	OF-N01.1	68	7	46	3313	317	841
Dutchman's Creek	OF-N01.2	134	80	82	7202	716	4176
Sewershed O		**88**	**50**	**55**	**4230**	**269**	**2280**
SLS#3	OF-SLS#3	88	50	55	4230	269	2280
Sewershed P		**32**	**7**	**26**	**1814**	**74**	**797**
SLS#2	OF-P02	10	5	8	496	1	378
SLS#1	OF-P03	14	2	18	800	60	419
SLS#50	OF-P04	8	0	0	518	13	0
Sewershed Q		**781**	**515**	**701**	**50462**	**2658**	**35182**
Pauline St PS	OF-Q01	102	37	79	5929	377	3084
Hickey Rd PS	OF-Q20	281	185	244	17288	1002	12345
York St PS	OF-Q10	34	11	18	1758	144	794
Champlain Dr PS	OF-Q11	35	14	21	1662	93	643
Bayside Drive	OF-Q13	188	132	138	13208	616	8060
McAllister Ind. Park PS	OF-Q15	58	54	91	4656	61	4165
Red Head Rd PS	OF-Q16	83	82	110	5961	365	6091
Study Area Total		**1786**	**1108**	**1434**	**105504**	**6663**	**68910**

Combined Sewer Overflow Reduction Strategy - EWWTF Collection System

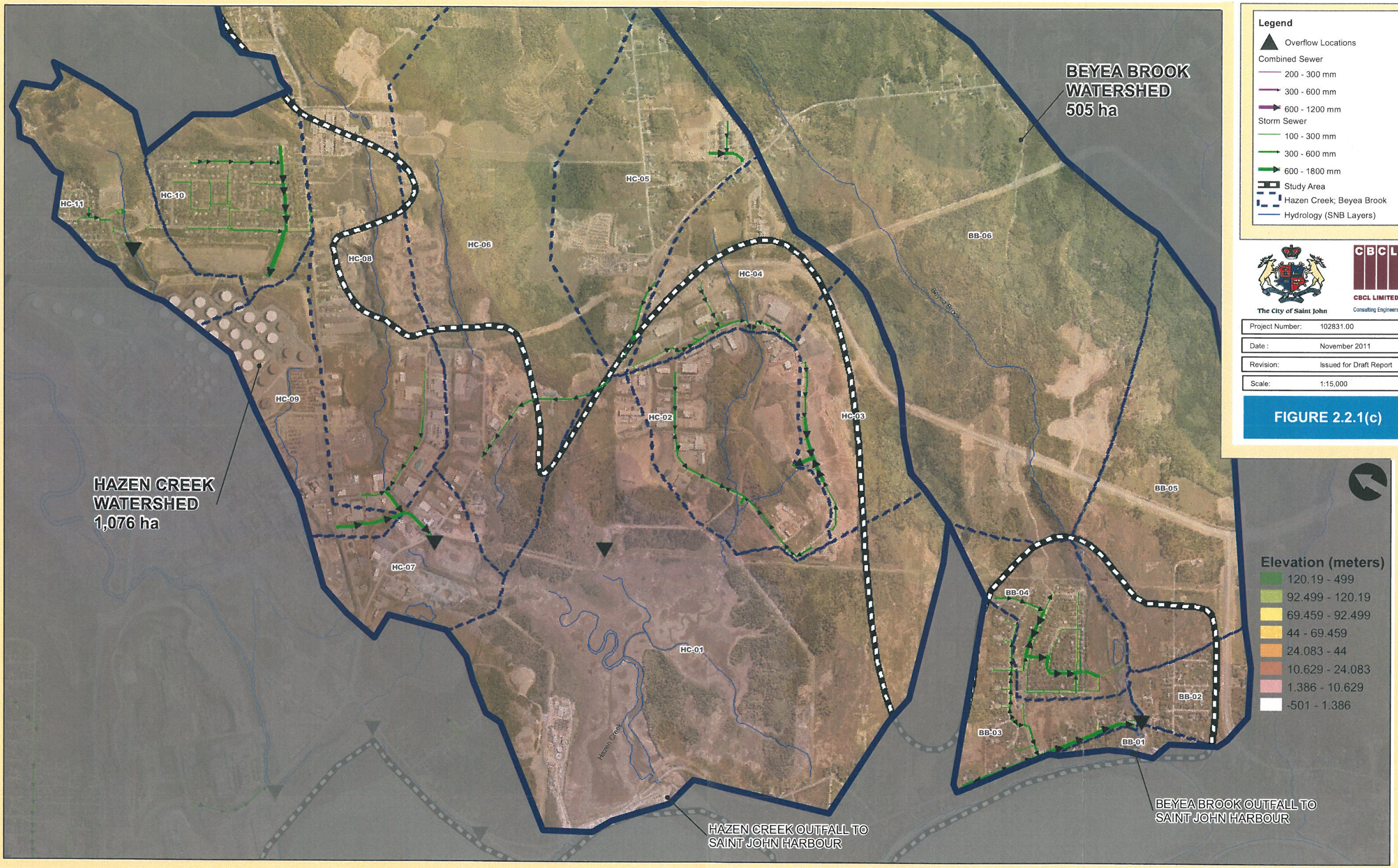
Marsh Creek Watershed & Tributary Storm Sewers



Combined Sewer Overflow Reduction Strategy - EWWTF Collection System | **Little River Watershed & Tributary Storm Sewers**

Combined Sewer Overflow Reduction Strategy - EWWTF Collection System

Hazen Creek & Beyea Brook Watersheds with Tributary Storm Sewers



Combined Sewer Overflow Reduction Strategy - EWWTF Collection System

Combinations of Existing Services in Study Area

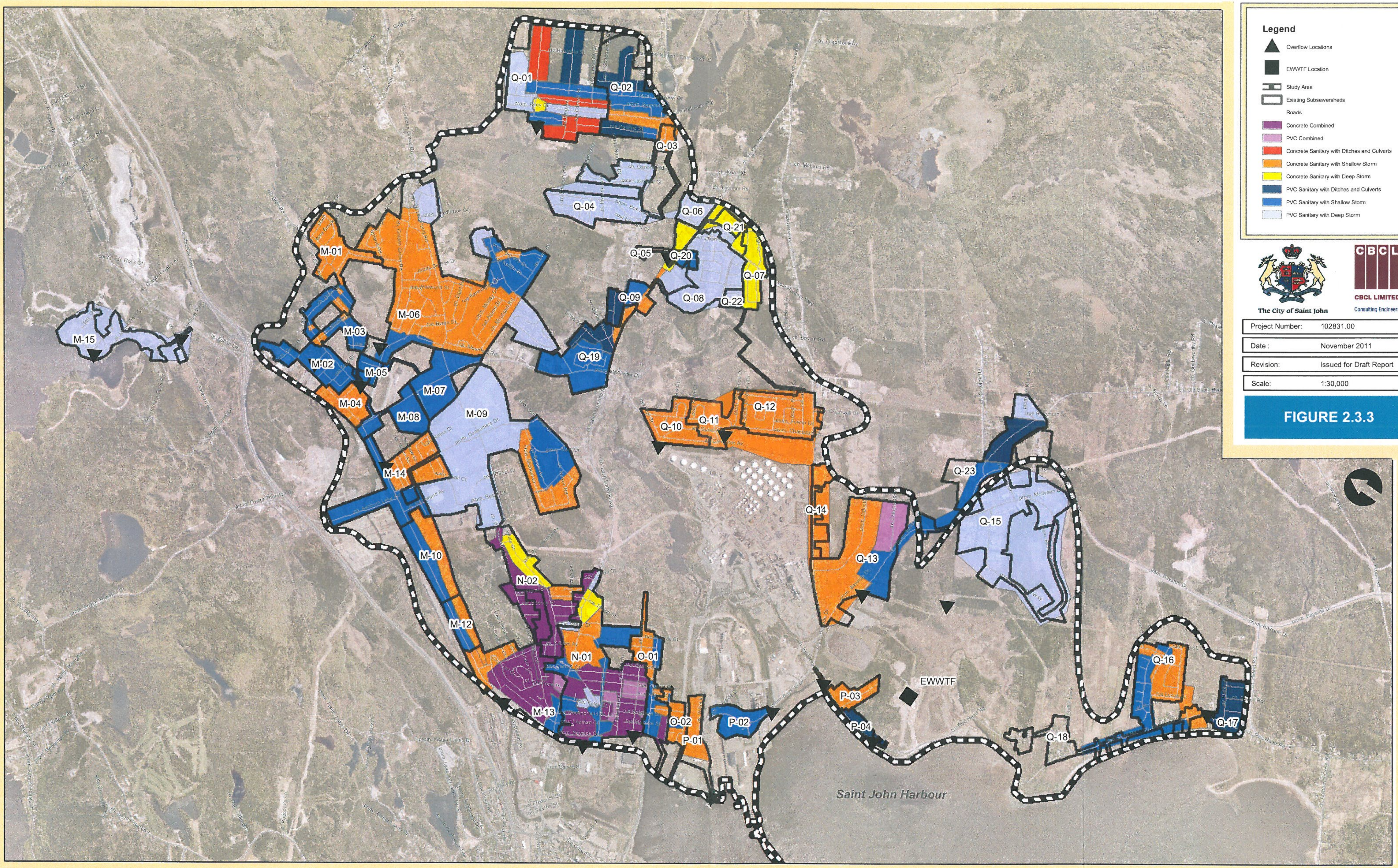


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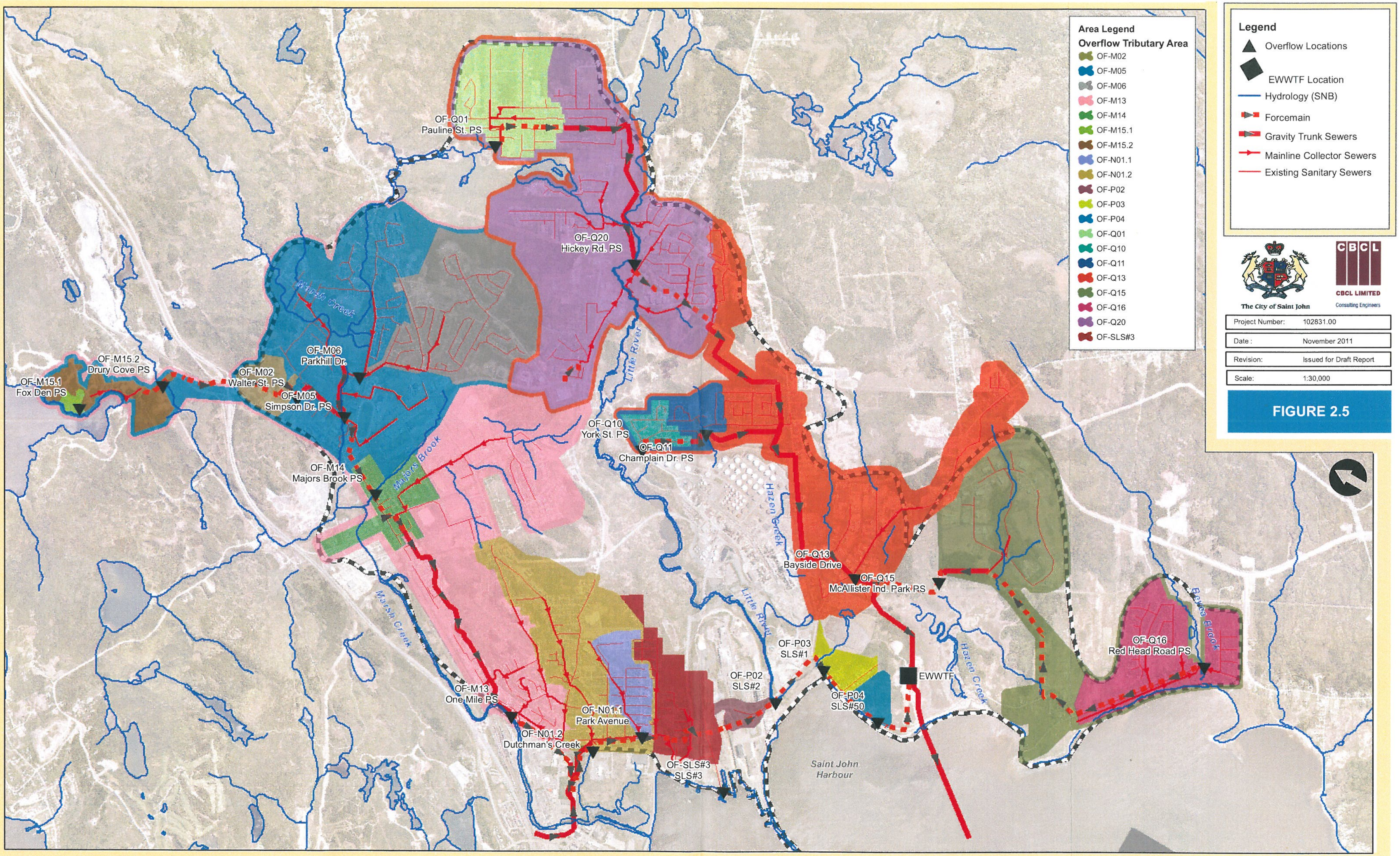
FIGURE 2.3.3

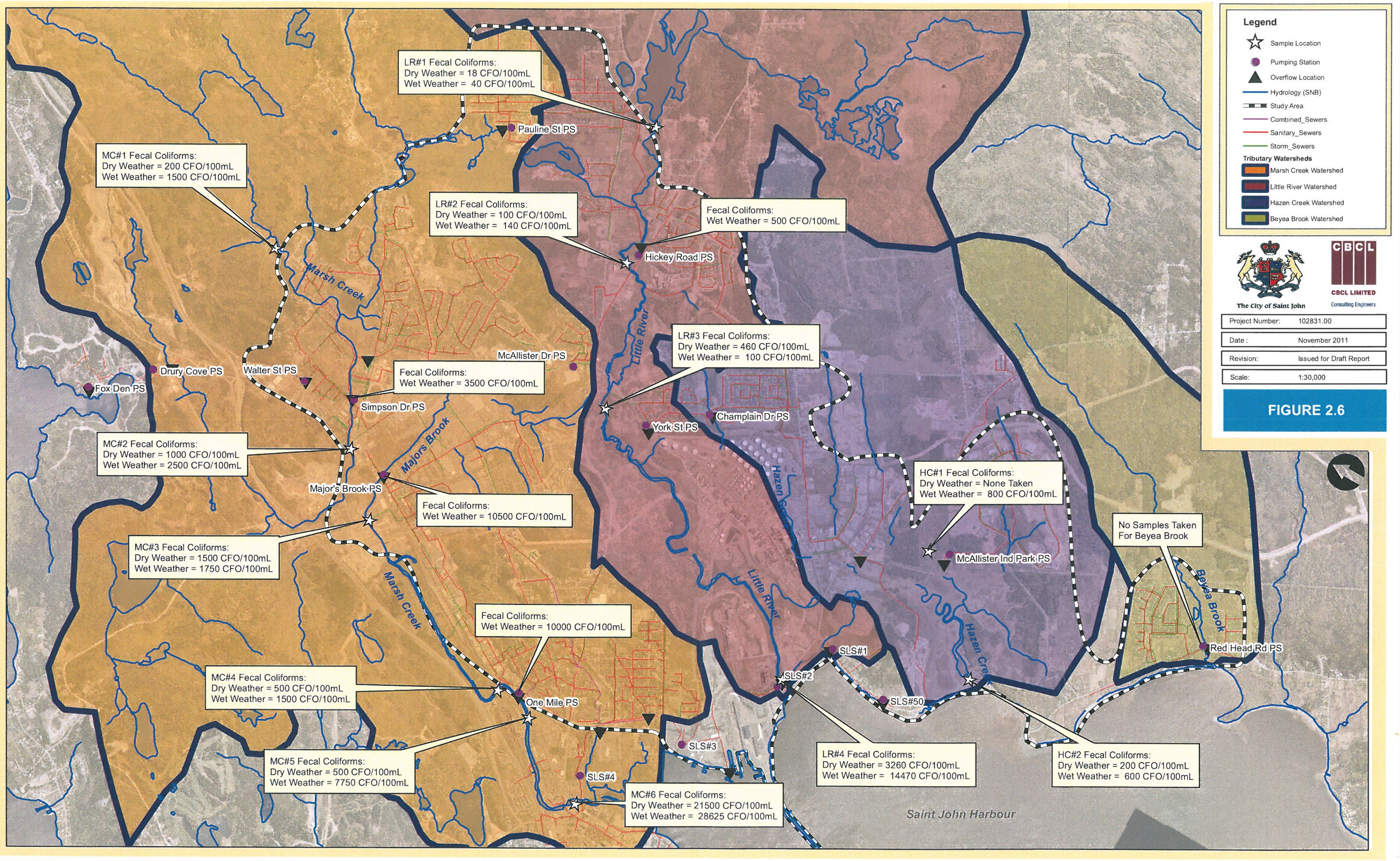
Combined Sewer Overflow Reduction Strategy - EWWTF Collection System

Existing Overflows and Corresponding Tributary Areas



Combined Sewer Overflow Reduction Strategy - EWWTF Collection System

Receiving Waters - Water Quality Sampling Results



CHAPTER 3 ANALYSIS OF THE WASTEWATER AND STORMWATER COLLECTION SYSTEMS

3.1 General

A computer based model of the wastewater and stormwater collection systems as well as the areas that generate flows in these systems was developed. This was used to complete the assessment of existing hydraulic conditions in these systems as well as to evaluate alternative wastewater management strategies. Results are discussed in this section.

3.2 Hydrologic and Hydraulic Model of the System

A hydrologic model of the processes that generate wastewater in the sewers was compiled based on the information presented in Chapter 2. This model was used to assess the possible range of flows that might be generated in the study area for existing development conditions. Generated flow information was then input to a hydraulic model of the collection system to simulate the reaction of the system to wastewater flows currently experienced within the study area. The complete sanitary and storm sewer systems were included in the model, including; sanitary, storm and combined sewer pipes, pumping stations and forcemains, and natural drainage systems.

3.2.1 Model Used and Applicability

The City selected SewerGems to model their wastewater and stormwater systems for its ability to be linked to the City's GIS database, allowing the transfer of information from the database to the models. This proprietary software is a comprehensive computer model for the analysis of quantity and quality problems associated with urban runoff and wastewater. The model solves complete dynamic flow routing equations for accurate simulation of backwater due to inline restrictions, looped connections (multiple flow paths), surcharging, and pressure flow.

Previous models created in the study area were generated using the USEPA Storm Water Management Model (SWMM). SewerGems and SWMM are comprehensive computer models for the analysis of quantity and quality problems associated with urban runoff and wastewater. SewerGems has an option to use the SWMM calculation algorithms so is able to accept the SWMM data files as inputs, provided the SWMM version is compatible with the selected SewerGems version. The models developed in previous assessments were lumped models with skeleton collection systems, typically only the main trunk systems were included. These were converted to SewerGems input files and details of the piped systems and tributary systems were added.

3.2.2 Model Inputs

Inputs to the model included:

- Tributary areas, developed area based on the area serviced by sanitary sewers and where buildings currently exist.
- Flow measurement results for each monitoring site, were used to estimate wastewater generation and assigned to each manhole including:
  - Dry weather flow curves, including base flows; and
  - Rainfall induced inflows;
- Soils information for pervious areas used to assess the potential for infiltration of precipitation into the groundwater regime in the sewershed. This was based on soils information obtained from soils mapping.
- Pipe characteristics obtained from the project GIS, including:
  - Pipe inverts or, if unavailable, estimation of inverts;
  - Diameter; and
  - Material;
- Pumping Station pump and forcemain characteristics.
- Design storm events based on the 37-year precipitation records for the climate station at the Saint John Airport. These are presented with the simulated flow hydrographs where applicable.

A schematic of the model, based on the overall system schematic is presented in Figure 1.3.2 (b) (ii). It is intended to represent the major components in the system, and in the interest of clarity, not all of the detail contained in the actual model is presented in this figure. This information is contained in the project GIS for this study.

3.2.3 Model Calibration

Model calibration is the most critical part of hydraulic modelling, since relevant results will not be obtained without calibration of the model. Calibration of the dry weather flow and stormwater inputs to the model was done in the following manner:

- Development of average dry weather flow for the gauged sewersheds. Diurnal curves representing typical sewage flow from the gauging sites are provided in Appendix B;
- Development of wet weather peak flow and volume characteristics for each of the gauged sewersheds. This assessment was based on the flows measured during the largest rainfall events recorded during the flow monitoring programs in each sub-sewershed;
- Conveyance of these results to the ungauged areas based on the types of services provided (see Figure 2.3.3) as well as the age of the systems where this information was available; and
- Dry weather flow and stormwater response to rainfall were input at each manhole in the model based on unit flow generation rates derived from the measured flow information.

3.3 Existing Conditions

Once calibrated to a satisfactory level, the model was used to assess the response of the collection and trunk sewer system to a range of flows, from dry weather flows to the flows experienced during wet weather.

3.3.1 Dry Weather Flows

A dry weather flow assessment was conducted, based on the dry weather flows measured in the flow monitoring program. Diurnal curves for each gauging areas were input to the model and distributed to each manhole in the system. The model simulates flows and water levels through the wastewater collection system during dry weather. The wastewater flows are smallest at the upper reaches of each sewershed and increase in the downstream direction as the overall tributary area increases.

3.3.2 Peak Flows

The test for collection systems is typically during wet weather events. Wet weather flows in the sanitary sewers include:

- The sanitary flow component, the same flows generated during dry weather, including some infiltration;
- Stormwater flow components including:
  - Rapid inflows generated by direct connections such as catch basins and roof leaders;
  - Slow inflow such as foundation drains connected to sanitary service laterals;
  - Increasing amounts of infiltration into the main sewers as well as the service laterals.

A range of design rainfall events based on the IDF curves for the climate station at the Saint John Airport were generated. The design rainfall events included those with return periods of 1 in 2 years, 1 in 5 years, and 1 in 100 years. These were input to watersheds tributary to the storm sewers and the combined sewers. Hydrologic characteristics that affect the amount of precipitation that becomes runoff in each watershed were included in the model. Runoff hydrographs were generated in the model for each watershed for each design rainfall event.

Peak flows during dry weather as well as the peak flows developed during the design rainfall events for each pipe are compiled in the project GIS database.

3.4 Pipe Capacities

Theoretical capacity of each pipe in the system was assessed using the hydraulic model. The Manning's equation was used to determine the theoretical capacity of each pipe within the model. This was based on:

- The difference in elevation of the pipe at upstream and downstream manholes;
- Length of pipe between the manholes;
- Pipe material from the project GIS; and
- Pipe diameter from the project GIS.

Estimated pipe capacities, based on these hydraulic conditions, are compiled in the project GIS database.

The hydrologic and hydraulic model was used to assess the portion of full capacity used in each pipe in the system relative to the design precipitation events. Under actual conditions, the capacity of pipe sections can be different than the theoretical capacity. Surcharging in the system can result in the capacity being dependant on the difference in water level between the inlet and outlets, rather than on the pipe slope.

3.5 Capacity Deficiencies

The model was used to assess the portion of capacity used during various flow conditions in the existing system. The model produces colour coded estimates for each pipe. It must be noted that these assessments are based on the assumptions that the existing systems are clean and in proper working order.

3.5.1 Dry Weather

As shown on Figure 3.5.1(a), only a few of the pipes used more than 100 percent of available capacity during simulated dry weather flow conditions (SLS#3 and Red Head Road). A second assessment was made using Atlantic Canada Standards & Guidelines - Design Flows, with similar results provided in Figure 3.5.1(b). A few capacity deficiencies were identified in the existing sanitary sewer system during dry weather (near Hickey Road pumping station and Red Head Road).

3.5.2 Wet Weather

Figure 3.5.2 shows the results of the capacity assessment for the sanitary sewers during the 1 in 2 year and 1 in 5 year design rainfall events. Many of the trunk sanitary sewers are surcharged during these events, particularly in sewersheds M and Q.

All of the sewers connected to the main pumping stations surcharge during overflow events as the water level must reach the overflow elevation before an overflow can occur. However, it is typically only the trunk sewers and/or pumping stations that do not have the capacity necessary to convey the peak flows received, adjacent sewers surcharge as a result of being connected to the pumping stations but are not necessarily considered “under capacity”.

3.6 Overflows

Overflows at the sites identified on Figure 2.5 were assessed for the same range of wet weather events. The assessments included estimations of the peak overflow rates as well as the volume of overflow generated by each event. An assessment based on the rainfall record for the station at the Saint John Airport was used to generate design rainfall events with return periods less than 1 in 2 years including 1 in 2 months, 1 in 6 months and 1 in 1 year. These design rainfall events were used to assess overflow frequency for existing conditions.

3.6.1 Peak Overflows Rates

Figure 3.6.1 shows the peak overflow rates simulated at each overflow site for the range over design rainfall events.

3.6.2 *Volume of Overflows*

Figure 3.6.2 shows the simulated volume that overflows at each overflow site for a range of design rainfall events.

3.6.3 *Some Pumping Stations Do Not Overflow*

Model simulations show no overflows generated at the Simpson Drive overflow or the Hickey Road overflow even during the 1 in 100 year design event, although the sewers near these stations surcharge.

3.6.3.1 Simpson Drive Lift Station

It was determined at Simpson Drive that the overflow invert is at geodetic elevation 0.85 metres (the height required to drain to Marsh Creek). The highest surcharge elevation reached was approximately 0.25 m during the 1 in 100 year event. Based on this comparison it was determined that the trunk sewer entering the Simpson Drive lift station would need to be surcharged in the order of 1.7 m above the top of the inlet in order to reach the overflow elevation.

The Parkhill Drive overflow is located upstream of the Simpson Drive overflow and is where the overflows from this system are generated. The Parkhill Drive overflow invert is at elevation 1.73 m and the hydraulic grade here reached approximately elevation 3.1 m during the 1 in 100 year event. The simulated results are supported by the findings of the Marsh Creek Sewage Scheme flow monitoring program in this area, refer to Appendix B for a summary of the flow monitoring findings.

3.6.3.2 Park Hill Drive Lift Station

Looking further at the Hickey Road overflow configuration the invert elevation is at 42.1 m (height required to drain to Little River) and the surcharging during the 1 in 100 year event reached approximately 40.75 m. Additional surcharging (in the order of 2.1 m above the top of the inlet pipe) at Hickey Road lift station would be required to generate overflows at this location.

Simulated flow results and hydraulic assessments indicate that with existing configurations at Simpson Drive and Hickey Road lift stations overflows will occur as a result of extended pump failure or power outages.

3.7 Impacts on Receiving Waters

Impacts of these overflows on the receiving waters were demonstrated by the findings of the receiving water quality assessment, provided in Figure 2.6. Grab samples were collected on two dates in December 2010:

- On a dry weather day in which there was no precipitation;
- On a day with rain.

The samples were analysed for several parameters that typically indicate the presence of wastewater. The results provided on Figure 2.6 are for Fecal coliform, an indicator organism used to assess the possibility of exposure to organisms that may cause sickness. The figure shows several interesting findings:

- Fecal coliform counts are significant in all sections of Marsh Creek, even at the upstream sampling location. The concentrations increase dramatically in the downstream direction, even during dry weather. This may potentially be due to wastewater discharges in the overflows;
- Concentrations are much lower in the Little River system. Upstream levels of Fecal coliform are relatively low, they increase downstream to levels that indicate some impact and by the time the flows reach the Bay, the concentrations are high;
- Concentrations in the Hazen Creek system are lower still, even downstream of the original Hazen Creek treatment facility.

Combined Sewer Overflow Reduction Strategy - EWWTF Collection System

Measured Dry Weather Capacity & Atlantic Canada Design Standards Capacity

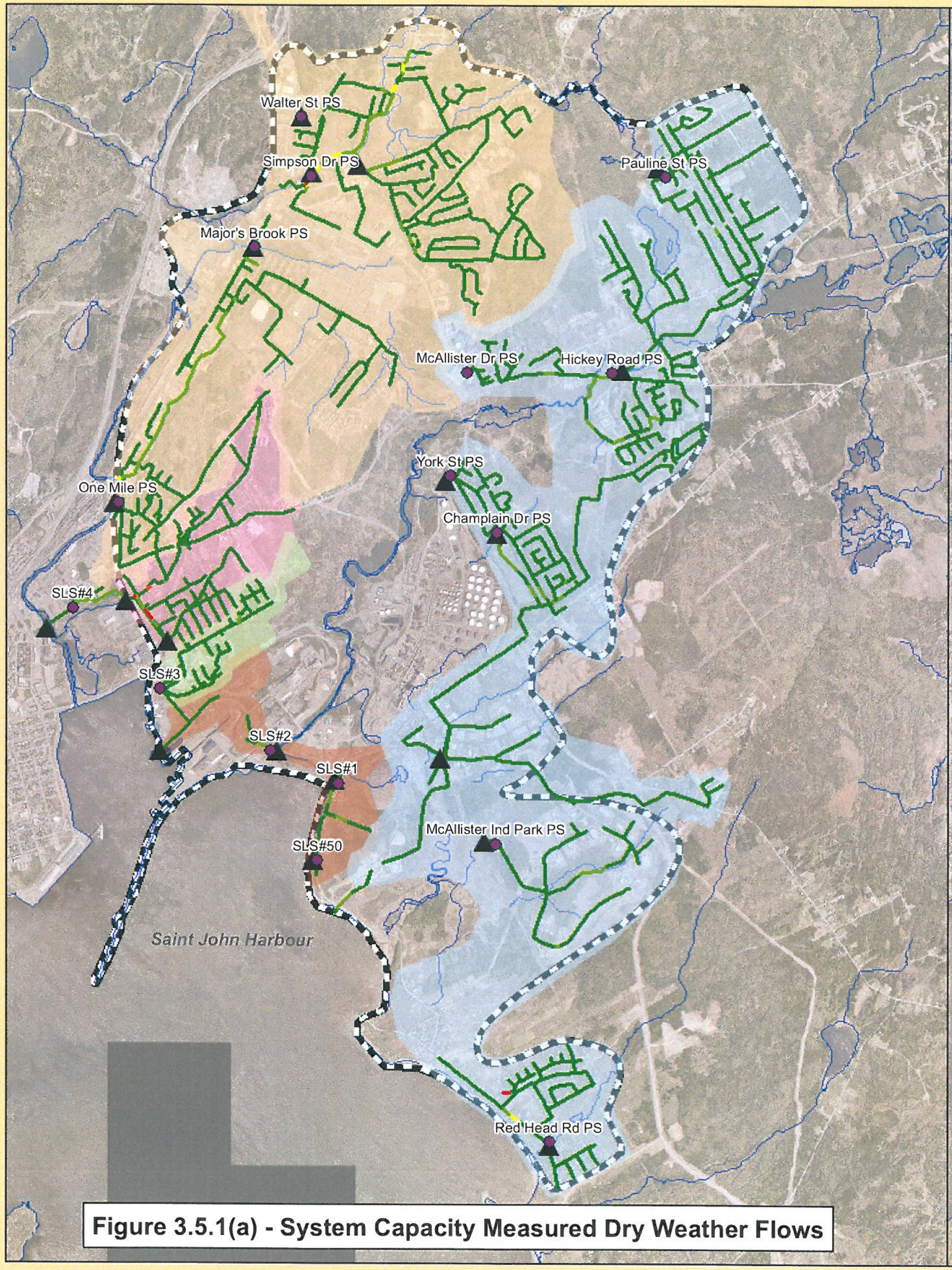


Figure 3.5.1(a) - System Capacity Measured Dry Weather Flows

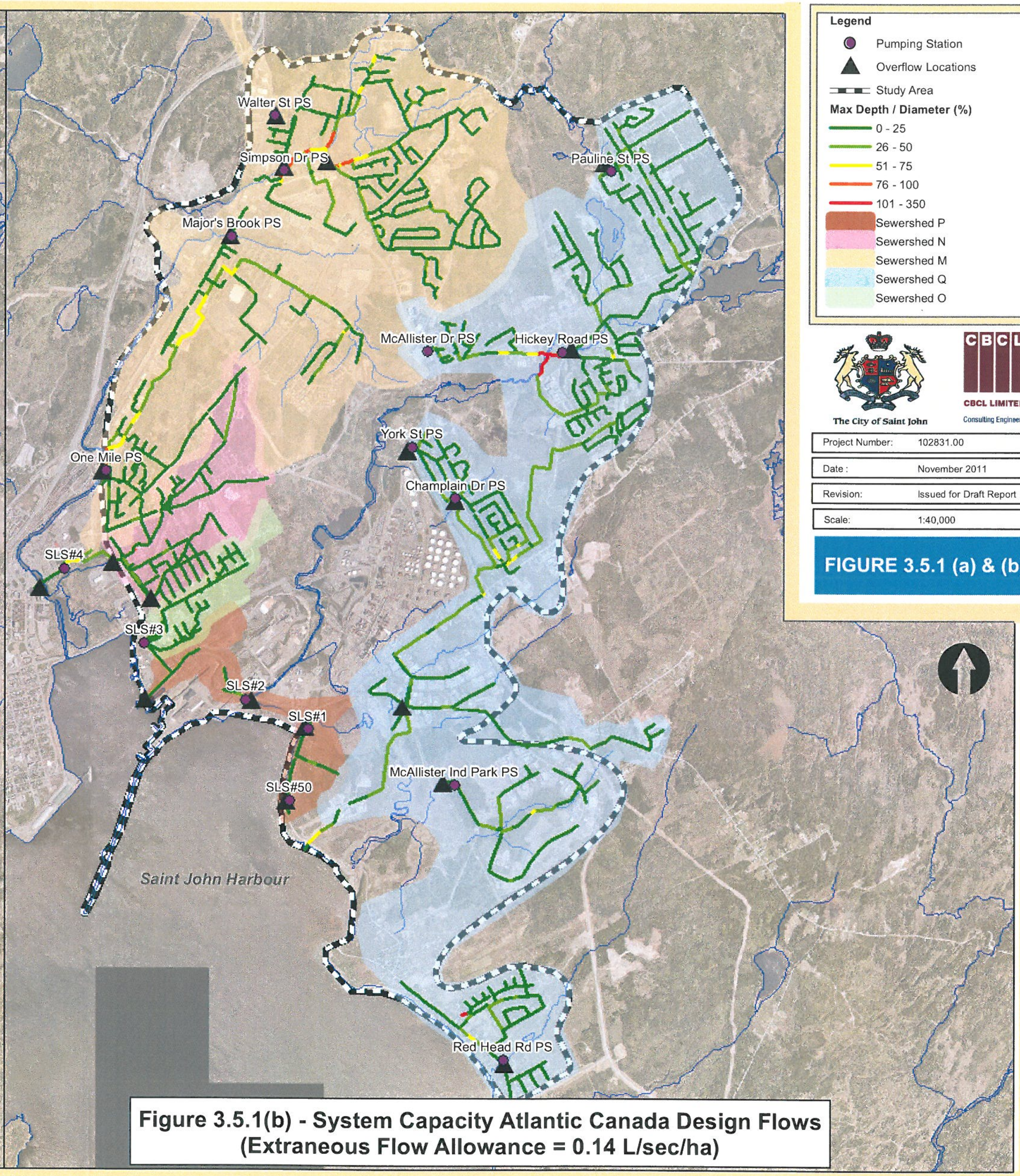


Figure 3.5.1(b) - System Capacity Atlantic Canada Design Flows (Extraneous Flow Allowance = 0.14 L/sec/ha)

Legend

- Pumping Station
- Overflow Locations
- Study Area

Max Depth / Diameter (%)

- 0 - 25
- 26 - 50
- 51 - 75
- 76 - 100
- 101 - 350
- Sewershed P
- Sewershed N
- Sewershed M
- Sewershed Q
- Sewershed O

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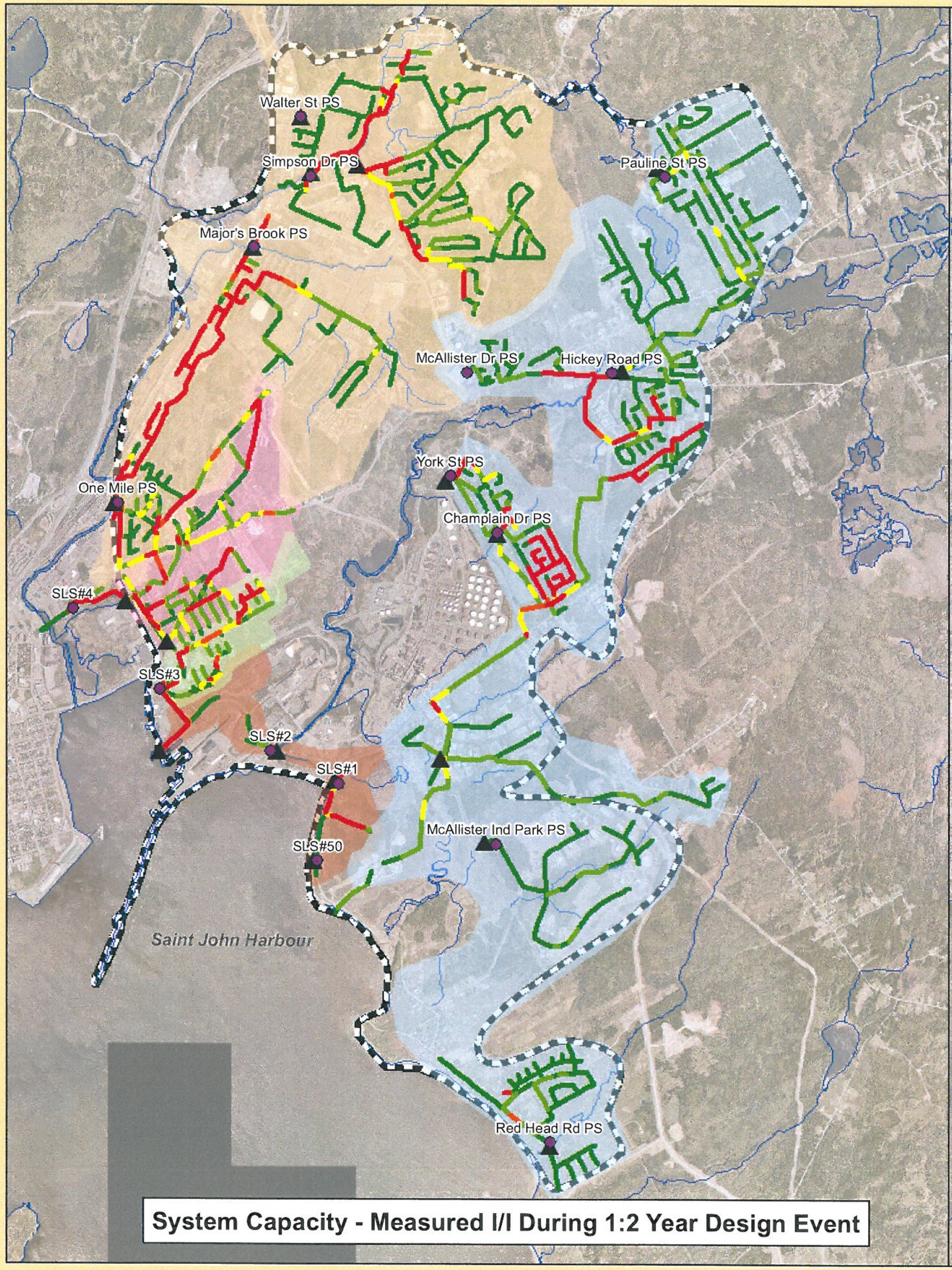
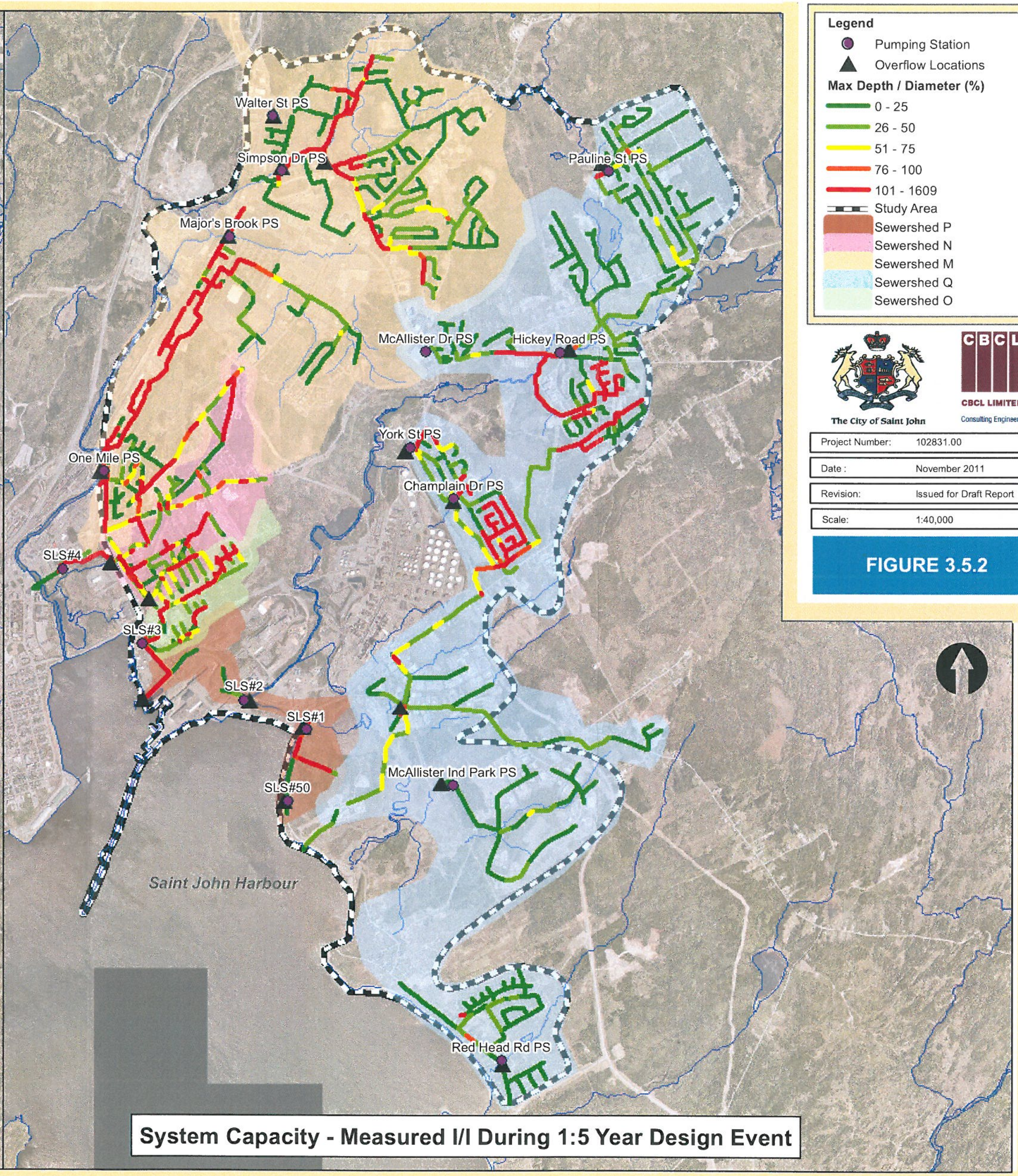
FIGURE 3.5.1 (a) & (b)

Combined Sewer Overflow Reduction Strategy - EWWTF Collection System

Wet Weather Flow Capacities



System Capacity - Measured I/I During 1:2 Year Design Event



System Capacity - Measured I/I During 1:5 Year Design Event

Legend

- Pumping Station
- Overflow Locations

Max Depth / Diameter (%)

- 0 - 25
- 26 - 50
- 51 - 75
- 76 - 100
- 101 - 1609
- Study Area
- Sewershed P
- Sewershed N
- Sewershed M
- Sewershed Q
- Sewershed O

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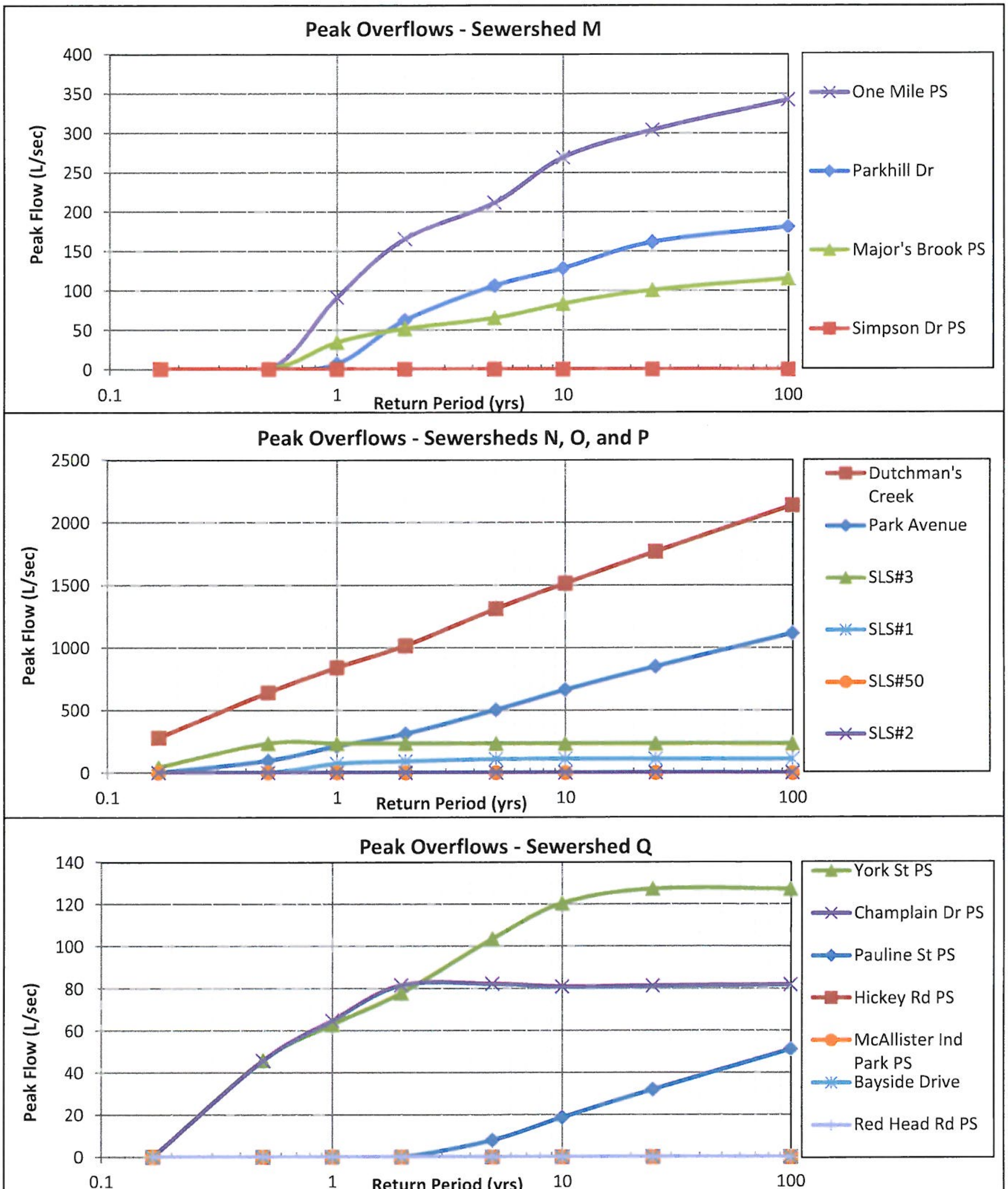
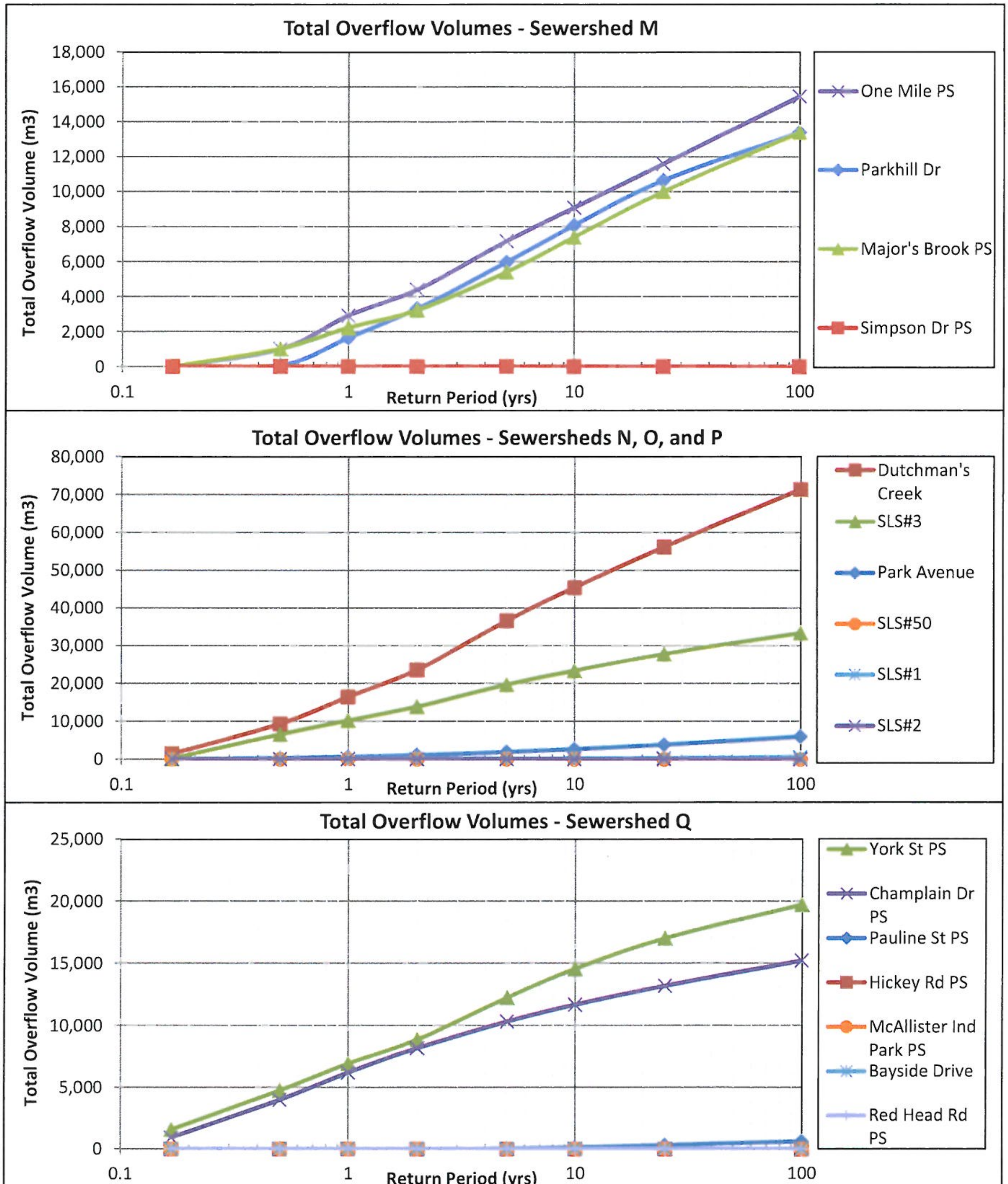
FIGURE 3.5.2



CBCL LIMITED Consulting Engineers	Combined Sewer Overflow Reduction Strategy EWWTF Collection System	November 2011
	Overflow Rates by Overflow	**Figure 3.6.1**



CBCL LIMITED Consulting Engineers	Combined Sewer Overflow Reduction Strategy EWWTF Collection System	November 2011
	Overflow Volumes by Overflow	**Figure 3.6.2**

CHAPTER 4 RECEIVING WATER OBJECTIVES

4.1 Water Use Objectives

Results of the receiving water quality assessment presented in Section 3.7 indicate that the overflows from the sanitary sewers have significant impacts on water quality in the receiving waters, particularly in Marsh Creek and the Little River systems. An overflow reduction plan needs objectives and typically these are water quality objectives for the receiving waters. In order to set water quality objectives desired water uses must be considered. Typical water uses for the receiving waters in the Study Area include:

- Fishing and consumption of fish from the waterbody;
- Primary contact by humans, including swimming;
- Secondary contact by humans such as boating;
- Habitat for a range of fish and other organisms;
- Aesthetics.

4.2 Water Quality Objectives

There are published requirements for water quality associated with these potential water uses. The following table summarizes potential water uses and required water quality to sustain these uses.

Table 4.2 Typical Receiving Water Quality Objectives

Use	Water Quality Required to Sustain the Water Use
Health	*Concentration of Fecal coliform (Fc) and/or E coli*
Fishing	To preserve habitat, CCME Guidelines should be met. To allow consumption, a Fc count of <14 per 100 mL sample is suggested
Shellfish Gathering	To preserve habitat, CCME Guidelines should be met. To allow consumption, a long term median Fc count of <14 per 100 mL sample is required by Environment Canada
Swimming	Fc count < 200 per 100 mL sample is required
Boating	Fc count < 1000 per 100 mL sample is required
Habitat and Aesthetics	*Trophic Status based on concentrations of available nutrients and active growth*
Maximum - Oligotrophic	(micrograms/L) Total phosphorous (TP) < 20; Total nitrogen (TN)<35;Disolved Oxygen (DO)>7.2, Chlorophyll A <3.5;(mg/L)

4.3 Summary of Objectives

Following is a summary of suggested water use objectives and associated water quality objectives for the receiving waters in the study area. It is expected that these objectives would be attained gradually, as measures to reduce overflows and the negative effects are implemented.

Table 4.3 Receiving Water Quality Objectives

Receiving Water	Water Use Objective	Water Quality Objective
Courtenay Bay	Secondary Contact	Fc count < 1000 counts per 100 mL sample is required
Marsh Creek	Habitat Secondary contact	TP < 20(micrograms/L); TN<35(micrograms/L); DO>7.2, Chlorophyll A <3.5;(mg/L), Fc<1000 counts per 100 mL sample is suggested
Little River	Habitat Secondary contact	TP < 20(micrograms/L); TN<35(micrograms/L); DO>7.2, Chlorophyll A <3.5;(mg/L), Fc<1000 counts per 100 mL sample is suggested
Hazen Creek	Habitat Primary contact	TP < 20(micrograms/L); TN<35(micrograms/L); DO>7.2, Chlorophyll A <3.5;(mg/L), Fc<200 counts per 100 mL sample is suggested
Beyea Brook	Habitat Primary contact	TP < 20(micrograms/L); TN<35(micrograms/L); DO>7.2, Chlorophyll A<3.5;(mg/L), Fc<200 counts per 100 mL sample is suggested

4.4 Overflow Reduction Priority

It is likely that not all of the work that will be required to reduce and eventually eliminate the overflows in the study area will be able to occur at the same time. Priorities should be established as to which receiving water should be upgraded first.

4.4.1 Freshwater Before Saltwater

Typically, freshwaters are more sensitive to intermittent discharges of wastewater than saltwater, given:

- Greater dilution potential;
- Faster die off of pathogens in saltwater.

4.4.2 Most Impacted before Least Impacted

Overflows to receiving waters with the worst water quality should be reduced before the others as current health risks are greatest in these systems. At the same time, all efforts should be made to minimize the potential for generating additional overflows to receiving waters currently minimally impacted by overflows. Overflows to freshwater receiving waters should be reduced in the following order:

1. Marsh Creek;
2. Little River;
3. Hazen Creek.

Overflows that discharge directly to Courtenay Bay would be the last to be removed.

CHAPTER 5 MANAGEMENT OF OVERFLOWS

Most of the trunk sewers do not have capacity to accommodate peak flows generated by the collection systems during rainfall events (see Figure 3.5.2). Peak wet weather flows from rainfall events result in surcharging in these systems and overflows at overflow sites shown in Figure 2.5. As shown in Figure 2.6, the negative impacts of these overflows on the receiving waters are significant.

These overflows have been tolerated in the past but resistance to continued overflows is increasing. Regulatory agencies including the provincial Department of Environment and the Federal Department of Fisheries and Oceans as well as the public require reduction of the frequency and duration of overflows. Although it may be some time in the future before elimination of overflows might be an expectation of these groups, eventually any overflows of wastewater to the environment will simply not be tolerated.

5.1 Regulatory Requirements

CCME guidelines published in 2010 for wastewater systems include the following requirements:

- No new combined sewers;
- No overflows during dry weather;
- Overflows are permitted during emergency conditions or in wet weather;
- Screening of overflows or the use of baffles is required.

These guidelines are being regulated through the provincial Department of Environment.

Other applicable federal regulations include the Fisheries Act. This Act prohibits the discharge of deleterious materials into potential fish habitat. Discharges of raw sewage from sanitary sewers or dilute sewage from combined sewers may be considered deleterious and are therefore not acceptable.

5.2 Objectives

The objective of this study is to identify alternative strategies for reducing overflows. The feasibility of implementing these strategies either on their own or in combination are to be investigated and those that best fit the systems and limitations of the study area are to be recommended as part of an overall program(s). Recommended strategies will best accomplish the following:

- Control overflows to a level that will satisfy all receiving water quality objectives;
- Reduce surcharging and flooding within the system;
- Minimize dilution of sanitary sewage by stormwater intrusion into the system to minimize the flows (particularly peak flows) that the conveyance and treatment systems must accommodate.

5.3 Separation of Stormwater

Combined sewers are distributed through most of the older collection systems in sewersheds N, O and P. Sewersheds M and Q have sanitary sewers and various systems for collection of stormwater. Most of the wastewater collection systems showed significant flow increases from wet weather (see Drawings B2, B3 and B4 in Appendix B). This occurs even though there have been significant storm sewer systems constructed in some areas.

The City has adopted a policy of separating sewers. Sewer separation is to be implemented as a specific policy wherever there is reconstruction or replacement of the combined system or where there is major street reconstruction which would permit access to the sewers.

Typically, the most feasible method of sewer separation follows steps including:

- Construction of a new sanitary sewer and controlled sanitary connections. Careful separation requires testing during the construction of the new sewer and connection of services. This has the added benefit of eliminating most of the extraneous flows typically associated with older sewer infrastructure. Each service should be checked prior to connection to the new sanitary sewer.
- Any existing laterals found to have stormwater connections or to be in poor condition should be replaced with a new sanitary service lateral connected directly to the building sanitary plumbing.
- Once all of the sanitary connections have been removed, the existing sewer and laterals may remain as a deep storm sewer system provided it is in reasonable condition. If it is not in reasonable condition it should be replaced when the new sanitary system is constructed.

Some systems that are considered separate do not always function as effectively as anticipated. Poor construction practices, unanticipated cross-connections, and deteriorating infrastructure can result in I&I problems, which seriously jeopardize the intent of the system. Surcharging and flooding is often the result and sanitary sewage overflows (SSO's) are the ultimate consequence. Such has been the case in parts of the sanitary sewage network, particularly in the older systems.

As presented in section 2.3.3 there are several types of sewers collecting wastewater in the study area. To completely separate storm water from sanitary sewers requires the following:

- A well constructed sanitary sewer with minimal risk of groundwater leaking into the pipes or laterals and no direct stormwater connections;
- Completely separate sanitary and storm service laterals to each property;

- A storm sewer that is deep enough so that typical drainage needs on private properties (foundation drains are the main requirement) can be serviced.

Table 5.3 (a) lists the components necessary to provide a complete system for each of the existing service combinations listed in Table 2.3.3.

Table 5.3 (a) New Services Required to Achieve Complete Sewer Separation in the Study Area

Existing Services Combination	Sanitary	Deep Storm (Minimum Capacity - Foundation Drainage)	Shallow Storm (Surface Drainage)
1	New Sanitary Sewer and Laterals	Existing Sewer and Laterals if their Condition Permits (1)	Street Gutters(2) or Existing Roadside Ditches
2(a)	New Sanitary Sewer and Laterals	Existing Sewer and Laterals if their Condition Permits (1)	Existing Roadside Ditches
2(b)	New Sanitary Sewer and Laterals	Existing Sewer and Laterals if their Condition Permits (1)	Existing Shallow Storm Sewer
2(c)	New Sanitary Sewer and Laterals	Existing Deep Storm Sewer and Laterals	Street Gutters(2) or Existing Roadside Ditches
3(a)	Existing Sanitary Sewer and Laterals	New Deep Storm Sewer and Laterals	Roadside Ditches
3(b)	Existing Sanitary Sewer and Laterals	New Deep Storm Sewer and Laterals	Shallow Storm Sewer
3(c)	Existing Sanitary Sewer and Laterals	Existing Deep Storm Sewer and Laterals	Street Gutters(2) or Existing Roadside Ditches

Notes:

(1) Deep storm sewers are only required where there are basements associated with existing development; this is typical of residential but not commercial development. This scheme works in most areas by diverting the existing sanitary sewers to the closest surface drainage system. This simplified scheme does not work in areas that are subject to wide spread surface flooding such as the low areas of Glen Falls. In these areas, the complete sanitary sewer as it currently exists (including existing lift stations) is required to provide a drainage system for foundations. Depending on the condition and capacity of the existing sewage lift stations, the existing station would be converted to lift flows from the converted sanitary sewers (the deep storm sewers) into the closest surface drainage system. Where the condition or capacity of the existing station warrants, a new wet weather station would be constructed. If there are no direct connections of the surface flows to the foundation drains, such as roof leaders, the converted deep storm sewer should provide relief for properties currently subjected to basement flooding in these low areas. The measures listed in the City's Information Bulletin "Measures to Reduce the Risk of Basement Flooding from Rainfall Runoff" should be implemented by all property owners, those in the flood prone area in particular.

(2) *A well defined major drainage corridor from the gutters to local drainage systems is required. While this corridor may be planned in new developments, it is not always the case in areas that are already developed.*

City design guidelines require that new developments be serviced with separate sanitary and deep storm sewers with capacity to convey foundation drainage as well as runoff generated by a design rainfall event with a return period of 1 in 5 years. New sewers required to achieve full separation of sanitary and storm flows in areas that are already serviced are shown on Figure 5.3. This figure is based on a sewer separation GIS database that contains design criteria (including design flows) and recommended specifications for each pipe section in each required sewer in the Study Area. A summary of the pipes required is presented in Table 5.3 (b). The costs shown are in 2011 dollars. They do not include H.S.T. but do include an allowance of 25 percent for contingency and engineering.

5.3.1 Long Term

Widespread separation plans are not typically the lowest cost solution to accommodate peak flows of the magnitude developed in most of the tributary systems. Long term plans for separation when the pipes are at the end of their serviceable life or whenever sewer replacement occurs is cost effective. This is recommended as a long term strategy for all of the tributary collection systems no matter which overflow reduction strategy is selected for the short term. This plan is compliant with the City's policy of sewer separation.

Separation of combined sewers into storm and sanitary sewers significantly reduces the magnitude and volume of flows conveyed to the treatment plant. While storm flows typically have much lower concentrations of pollutants than the flows in sanitary sewers, they may still carry significant concentrations of pollutants during wet weather following prolonged dry weather periods.

Concern remains for the contaminants discharged to the receiving water from urban stormwater, especially during the first flush at the beginning of rainfall events. If this stormwater is directly discharged into the receiving waters, it may produce instances where pollutant concentrations exceed water quality objectives (see Table 4.3) unless additional measures are taken to reduce the concentration of pollutants in stormwater as well.

5.3.2 Short Term

Several potential separation projects that should be considered for the short term include provision of:

- New trunk sanitary sewers to connect isolated upstream sanitary sewer systems to the trunk sewers and interceptor sewers;
- New sanitary sewers in areas subject to surface flooding and conversion of the existing sanitary system to a pumped deep storm sewer system to provide relief for properties affected by basement flooding.

These measures are identified on Figure 5.3.2 and in the sewer separation GIS and are summarized in Table 5.3.2.

5.4 Alternative Overflow Reduction Strategies Considered

Several alternative overflow reduction strategies were also considered in this Study Area. Rather than proceed with the separation of sanitary and storm sewers, overflows could be reduced by one or more of the following:

- Alternate control of stormwater inflows at source including:
  - Inlet control and surface storage of stormwater before it enters the combined sewers. A well-defined major drainage corridor is required for this concept to be employed without causing additional surface flooding issues;
  - Inflow and infiltration reduction programs for sanitary sewers.
- Increased trunk sewer capacity to convey larger flows to the treatment plant and less to the overflows;
- Storage of flows in excess of trunk sewer and interceptor capacity for conveyance and treatment when capacity is available; and
- Treatment of excess flows can be provided, either at an expanded treatment plant, or in one or more direct overflow treatment systems. The level treatment required depends on the source of the overflow (SSOs typically discharge pollutants at higher concentrations than CSOs) as well as the sensitivity of the receiving waters (salt waters typically tolerate more pollutants than fresh water due to typically higher dilution potential).

Each of the strategies listed is discussed in greater detail in this section.

5.4.1 Control of Stormwater at Source

There are alternative methods to constructing new sanitary sewers (and deep storm sewers) to limit the amount of stormwater in the combined sewers and sanitary sewers, including:

1. Reduce the stormwater component in combined sewers by reducing the inlet capacity for stormwater connections;
2. Reduce the stormwater component in sanitary sewers by reducing the number and magnitude of existing stormwater connections.

5.4.1.1 Reduced Inlet Capacity

An approach taken in some jurisdictions is to limit the inflows to the combined sewers using inlet controls at the major sources of inflows (catch basins). Limiting the inflows forces more stormwater to remain on the surface and not enter the piped system. Typically, the capacity of combined sewers is limited to the flows generated by relatively minor rainfall events. Note the amount of surcharging generated by a 1 in 2 year rainfall event in the combined sewer areas as shown on Figure 3.5.2. When this happens, excess flows are forced to remain on the surface. Limiting the inflows causes this to occur during less frequent rainfall events. If there is opportunity for the flows to be conveyed as gutter flow or if there are areas that can flood without causing hazards or damage then this is a feasible alternative. This must be confirmed prior to considering this alternative.

On a sewershed by sewershed basis, detailed investigations such as smoke testing are used to identify all stormwater connections. These are then used to create a detailed model of the sewershed that includes these connections. The detailed model is then able to simulate conditions in the sewers and at the street level for various levels of inlet restriction that can be achieved without creating new flooding issues.

This type of system can be used as an interim solution or short term measure to reduce overflows. However, additional flows in street gutters as well as on-street ponding are typically not in line with the level of service required in the City's Stormwater Management Guidelines so it is not considered a long term solution.

5.4.1.2 Sewer Rehabilitation

Some sewers that were designed as sanitary sewers tend to have relatively high peak flows during wet weather from inflows and infiltration due to several factors including:

- Age of the system, older systems typically have more deficiencies;
- Pipe materials used, rigid or inflexible materials are easier to break;
- Number of joints and type of joints and gaskets, more joints mean greater potential for leaks; and
- Availability of storm drainage systems.

Excessive extraneous flows are typical in areas where the sanitary sewers are constructed of concrete pipe (see Figure 2.3.3). Flow monitoring results for these areas show higher extraneous flow generation rates than areas with newer sewer construction. This may also be due in part to the lack of storm sewer systems in some of these areas. Typically without storm sewer systems, the sanitary sewers are used for drainage as well to some degree.

To remove these extraneous flows and reduce overflows from these systems requires detailed investigations to locate each source, determine the feasibility of removing it and develop plans to accomplish the removals. An overflow reduction strategy that relies on reduction of extraneous flows is beneficial in that it resolves several potential concerns with the existing system, including:

1. Sewer backups and surcharging by reducing flows in the system;
2. Overflows from collection systems and pumping stations by reducing the overall peak flow to the capacity of the sewer system;
3. Operation and performance of the downstream systems including the trunk sewers and the new interceptor sewers, as well as the treatment plant and outfall.

There are numerous examples of programs for reduction of stormwater in sanitary sewer systems. The City of Toronto has developed a comprehensive program for management of combined sewers and stormwater within the watershed tributary to its interceptor and treatment systems. It recognizes the negative impacts on the receiving waters of stormwater as well as combined sewer flows and has adapted programs to reduce both. Key components of the program include:

- Surface storage to attenuate peak stormwater flows;
- Underground storage to reduce peak wastewater flows; and
- A downspout disconnection program to reduce stormwater discharges from private properties.

Past experience with I/I reduction programs involving removal of stormwater connections on private property and repair of leaking sewer components indicate that initial reductions may not be realised in the long term. This is due to reconnections and continued deterioration of the sanitary sewer components. An ongoing program of identification of problems and development and implementation of remedial measures to address these problems must be implemented in order to maintain the reductions at the planned levels.

Estimates of the capital and operating costs for extraneous flow reduction in the study area were recently developed based on detailed assessments in similar sewersheds in other areas of the City. Costs were developed for the areas where extraneous flow reduction was considered a feasible strategy from costs generated in previous detailed studies and prorated by sewershed areas and weighted based on the relative peak extraneous flow generation rates developed for each sub-sewershed.

If it is determined by the assessment that I/I reduction is a feasible concept for reduction of overflows, detailed studies to identify the I/I sources (other than combined sewer inputs) and the most cost effective means of reducing the magnitude of each source would be required to produce a list of specific remedial measures for each area. Recommended measures might include:

- Reducing infiltration and inflows to existing manholes by;
    - Raising manhole covers to reduce potential inflows through the covers;
    - Plugging holes or replacing existing covers with covers with one hole;
    - Pre-cast manhole section joint repairs;
    - Replacing manhole covers with multiple holes with gasketed covers; and
    - Installing chimney seals in the manholes;
- Repairing cracked pipes and leaking joints by grouting or other spot repairs; and
- Redirecting sources of inflow from private properties.

Again, these are interim measures to prolong the useful life of existing sewers so are considered short term measures.

5.4.2 *Increased System Capacity*

Overflows may be decreased by increasing the trunk sewer, interceptor and treatment capacity. The new sewers and treatment plant would be sized to convey and treat all of the flows from the tributary systems. This is straightforward for intercepting sanitary sewer systems but requires relatively large systems to accommodate flows from sanitary sewers with significant extraneous flows and is not practical for most combined systems. Capacity increases can be achieved as follows:

5.4.2.1 Real Time Control

Traditionally the collection system has been passively operated; the pumping stations are operated in response to changes in water level in the station and for the most part are not influenced by conditions in the remainder of the system. Real time control measures can be implemented to maximize the effectiveness of existing facilities. The system can be operated in response to changing conditions all over the system. For instance, rather than come on and overload a downstream trunk sewer, a station may stay off until all possible storage is used, and then only to the capacity that is available in the downstream system as determined by flow monitoring. This type of system requires more instrumentation and control systems that respond to inputs from the instrumentation. As well, detailed operating rules must be set out and priorities set so that all conditions are met with predetermined system reactions.

5.4.2.2 Interceptor Capacity Upgrades

To provide additional capacity, trunk sewers and interceptor sewers would be twinned where possible or upsized so that excess flows are relocated from the areas where they currently overflow to suitable discharge locations such as at the treatment plant. Here they could receive treatment to reduce their impacts on the Harbour. This would be a massive undertaking to implement everywhere as the interceptors were designed to convey a relatively small portion of the flows experienced in combined sewers.

5.4.2.3 Treatment of Wet Weather Flows

It is not typically feasible to consider the concept of conveying peak wet weather flows from combined sewers that are generated by extreme rainfall events to the treatment plant for full treatment. The range in hydraulic loads that the treatment plant would need to handle would be excessive. Typically a wet weather treatment plant would be required at the treatment plant site that could be started up quickly and operated intermittently. See section 5.4.4 for a discussion of wet weather flow treatment plants. A wet weather treatment plant would require significantly greater capacity than the new treatment plant to be able to treat full wet weather flows, potentially equivalent to the sum of the peak wet weather flows from all areas serviced by the treatment plant. It would not require the ancillary components such as sludge handling equipment. Additional hydraulic capacity requires additional space and this is limited on the existing site.

5.4.2.4 Assessment

To assess this concept, all trunk and interceptor sewers that surcharged during a range of events from the 1 in 2 month to the 1 in 100 year events were identified and the size required to accommodate the peak flows was assessed. The costs to upsize these pipes to accommodate the peak flows without surcharging were estimated using unit costs from similar construction projects in the City for a range of feasible events. Estimates of probable costs to upgrade the trunk sewers and interceptor systems are provided in Table 5.4.2. In addition to the costs of the trunk sewers and interceptors, the cost of a wet weather treatment plant must also be included.

Treatment plant capacity increases needed to accommodate the increased interceptor capacity were estimated based on providing high rate primary clarifiers and disinfection using ultraviolet radiation as described in section 5.4.4.

The benefit of this scheme compared to the scheme of providing the treatment at every overflow site, or groups of outfalls is that all of the treatment would be consolidated in a single location, close to the main treatment plant and this would facilitate operation. The disadvantage is that land is scarce at the treatment plant location, as it is where the overflows are located. Acquisition of additional land in close proximity to the treatment plant would be required to implement this strategy.

5.4.3 *Storage of Excess Flows*

Incorporating storage in the collection and trunk sewer systems contains the excess flows that would typically overflow and reduces peak flows downstream in the systems. These flows are later returned to the trunk sewer system and flow to the treatment plant for treatment during periods when capacity is available in the collection system, interceptor system and the treatment plant.

Figure 3.6.2 shows the volume of overflow generated at each overflow site by a range of design rainfall events. These were used to establish preliminary storage requirements in each sewershed. Orthophotos of each sewershed were used to identify currently undeveloped property in each sewershed. Potential sites for storage are typically located in the lower reaches of the sewersheds but upstream of sewers that were identified as surcharging in Figure 3.5.2. The available sites are shown on Figures 5.4.3 (a)(i, ii, iii).

5.4.3.1 Off-line Storage Tanks

An assessment of the feasibility of constructing traditional buried wastewater storage tanks was completed based on constructing them on the undeveloped properties. The volume of storage that could be located on each site was estimated based on the offsets from the property boundaries. This information was used with a conceptual layout for these types of tanks (see Figures 5.4.3 (a)(i, ii, iii)) to develop estimates of probable costs for each storage unit and these are provided in Table 5.4.3.1.

The storage tanks would include the following recommended components, considered in the cost estimates:

- Mixers to maintain material in suspension prior to pumping; and
- Automated flushing systems to limit operational requirements for cleaning out any material that settles during the storage period.

A schematic representation of the features of the tanks is included in Figure 5.4.3(a)(i).

5.4.3.2 In-line Storage

Storage in the system can be created in the conduits that make up the collection and trunk systems as follows:

- In the existing sewer system itself. In most collection systems there are some large sewers that run only partially full even at the height of large rainfall events. To use this unused capacity requires addition of flow control within the collection and or trunk sewer system and development of a control strategy to cause these systems to fill when required. This is usually a practical low cost measure where it is feasible, but is unlikely to eliminate all overflows;
- Similarly, inline storage may be created by providing oversized pipes. The extreme case is where large tunnels are constructed to perform the function of conveyance as well as storage. This is most cost effective where the pipe must be replaced for capacity purposes as it is simply sized larger than required for conveyance. Storage tunnels are an example of this technique.

Prior to constructing any new storage facilities, a detailed evaluation of the potential to generate additional storage within each existing collection, trunk sewer and interceptor sewer system should be considered. Overflows from the existing systems can occur without all of the pipes being surcharged or even full. Installation of control structures within the sewer system itself may be used to utilize this storage to reduce overflows. Flow control devices can be used to flood un-surcharged, upstream systems during smaller, more frequent rainfall events. These control systems would be designed to fail open to minimize the risk of uncontrolled flooding during power failures.

Preliminary assessments of the potential for developing additional storage were carried out in this study. The hydraulic model of the collection system was used to assess the potential storage available within the system during significant rainfall events. All sewers with a diameter of 450 mm or greater were assessed for volume available during various rainfall events. The available storage volumes are summarized in Figure 5.4.3 (b) and Table 5.4.3.2 by sewershed and compared to the total volume available in the same pipes. The sewers highlighted in green and yellow on Figure 5.4.3 (b) indicate larger diameter pipes with capacity available to potentially be used as storage during a 1:2 year rainfall event. Sections highlighted in red indicate sewers that are already full or surcharging during a 1:2 year rainfall event.

Table 5.4.3.2 Pipe Volume Available in Large Diameter Pipes for In-line Storage during Rainfall Events

Sewershed	Total Volume of Large Diameter Pipes (m^3)	1:2 Year (m^3)	1:5 Year (m^3)	1:10 Year (m^3)
Sewershed M	1,346	139	109	97
Sewershed N	2,033	1,326	1,222	1,168
Sewershed O	831	538	526	520
Sewershed P	2	0	0	0
Sewershed Q	4,126	2,521	2,350	2,261
Total	**8,337**	**4,523**	**4,207**	**4,046**

These investigations indicate opportunities to generate significant storage volumes during more frequent rainfall events and minimal storage during the largest design rainfall event analysed.

The costs of utilizing this potential storage are the costs of constructing control stations and flow/water level monitoring sites as well as real time control systems. These control systems include software required to monitor and respond to the changing conditions in the sewers as well as anticipated flows from changing weather conditions. It is estimated that these costs might be in the order of 10 to 20 percent of the cost of building separate underground storage structures, as described previously (Table 5.4.3.1).

5.4.3.3 Summary of Storage Requirements

A total storage volume in the order of 200,000 cubic metres located at the overflow sites would virtually eliminate the overflows in the study area. There is undeveloped land to locate the necessary volumes at each overflow except those in sewershed Q. Undeveloped land will only allow half of the required volume in sewershed Q to be located where it is required. The additional storage must be created as in-line storage.

5.4.4 Treatment of Overflows

Flows in excess of the interceptor capacity may be treated to an acceptable level so that the treated effluent meets the water quality objectives of the receiving waters. This treatment would be done at:

- A wet weather treatment train at the East Saint John Wastewater Treatment Facility as described in section 5.4.2; or
- One or more satellite treatment systems. Small treatment plants that would be active only in wet weather would be located to treat overflows from one or more overflows. Potential treatment sites are the same as those identified for storage.

Pollutants that are typically targeted by overflow treatment include:

- Floatables and gross solids that pose aesthetic concerns; and
- Pathogens that may pose a health risk. Their concentration is measured by measuring the concentration of representative micro - organisms including Fecal coliform and E coli.

Removal of floatables and gross solids can be achieved by a range of techniques including screening and sedimentation by techniques such as the following:

- Baffles – typically located immediately upstream of the overflow;
- Screening – typically located at the overflow weir. This is the most efficient method available to ensure that all material of a given size is removed, regardless of its settling ability;
- Sedimentation by:
  - Retention Treatment Basins – the tank acts as a storage tank for small events. When the flows exceed the available storage, the tank overflows and acts as a clarifier to remove settleable or floatable material. This material must be removed after each overflow event;
  - High Rate Sedimentation – uses aids to enhance coagulation of particles and settling, there are two proprietary systems available that use this technique; and
  - Vortex Separation is enhanced sedimentation using a tangential inlet to generate centrifugal forces in a circular chamber. These forces increase settling velocities of settleable material to reduce the retention time required to remove particles of a certain mass so are much smaller than a settling tank.

Water quality objectives for Fecal coliform were listed in Table 4.3 and include:

- 200counts per 100 mL sample for Hazen Creek and Beyea Brook; and
- 1000counts per 100 mL sample for the Harbour, Courtenay Bay, Mash Creek and Little River.

Currently, the preferred technology for disinfection of overflows is with ultraviolet radiation. This requires a relatively high level of treatment to remove solids that can block the ultraviolet radiation. Advanced primary treatment is currently the minimum level of treatment used to remove this material and allow for effective disinfection of overflows using ultraviolet radiation.

On a small site that might be available near the overflows, high rate sedimentation systems based on ballasted floc such as the Densadeg or Actiflo systems are applicable. These treatment systems remove solids from the overflows and concentrate them. Solids are discharged to the trunk sewers or interceptor system where they are conveyed to the main treatment plant for removal and disposal. By this process they can reduce the solids content of the wastewater during storm events to treatable levels so that the UV disinfection process of the overflows is effective. Similar treatment plants in other jurisdictions are able to produce end of pipe pollutant concentrations in the order of 30 mg/L BOD and 10 mg/L suspended solids and less than 1000 counts per 100 mL sample for Fecal coliform.

The capital and operating costs of a suitable system vary depending on the peak flow to be treated as well as the frequency and duration of overflow events. Most of the operating cost is for operators to maintain the plant during and between overflow events and for the chemicals required by the process to remove material from the flow. Published capital costs and operating cost data for similar installations were used in this study to generate order of magnitude costs at each overflow site for a range of peak flows resulting from the design rainfall events at these sites. The estimated peak flows and associated treatment costs for the design rainfall events are summarized in Table 5.4.4.

Each wet weather treatment plant would treat all of the flows from the design rainfall events up to the rated treatment plant capacity and overflow the remainder untreated. Although it would produce a marked improvement over existing conditions, it may still not be enough to meet water quality objectives for all of the receiving waters. A higher level of treatment might be required for upstream overflows into fresh waters. This would need to be assessed through detailed receiving water assessments.

Consolidation of several overflow outfalls will be beneficial and in fact required, particularly where land for a small treatment plant is not available. Although there is undeveloped land at some of the overflow sites, it may be difficult to site treatment plants at these locations along the waterfront, generally in the locations shown for the storage tank alternative.

5.5 Evaluation of Alternative Strategies

Each alternative strategy was developed for the main sub-sewersheds (M, N, O, P, and Q) to the point where it could be evaluated and compared to other strategies based on a set of evaluation criteria. Evaluation criteria included:

- Life cycle cost, typically evaluated using the Present Worth of all capital and operating costs as well as replacement costs during the life of the project;
- Reduction in the frequency and duration of overflows as well as surcharging in the collection system;
- Reduction of pollutants introduced into the receiving waters;
- Land requirements;
- Reliability; and
- Compatibility with the capacity of existing trunk sewers as well as the new interceptor system (from SLS# 4 to the treatment plant) and treatment system at the Eastern Wastewater Treatment Facility.

The study area was divided into sub-sewersheds comprised of the tributary system to each overflow. These are shown in Figure 1.3.2 (b) (i). Alternative strategies for reducing overflows were considered in isolation to facilitate the evaluation process. Results are presented for a range of design rainfall events with return periods from 1 in 2 months to 1 in 100 years (having a 1 percent chance of being the largest rainfall event each year on average). Relative benefits, measured as a reduction in the annual number of overflows which results in improved receiving water quality, are compared to the capital costs of achieving these benefits.

An assessment of flows was provided for each sewershed in Table5.5 (a). Measured dry weather flows were compared to design flows. The sewersheds were then categorized based on the ratio of the measured dry weather flows to design flows for each sewershed. This assessment indicates very high ratios for the combined sewer areas in sewersheds N, O, and P as expected from combined sewers.

Results of this assessment were used in a screening process to determine the feasibility of the various alternative wastewater management strategies considered. Those alternative strategies considered most feasible in each sewershed are presented in Table 5.5(b) and were considered in detail.

Capital costs of the various alternative strategies were developed for comparison of the alternatives. These were based on the sizes and capacities necessary to accommodate the flows from a range of design rainfall events. Costs are summarized in Figures 5.5 (a, b, and c) for all feasible alternatives in each overflow sewershed. Operation and Maintenance Costs are summarized as well for those alternatives with significant annual costs. Life Cycle Costs of the alternatives considered, developed from these contributing costs, are summarized in Table 5.5 (c) based on the facilities required to ultimately accommodate flows from a 1 in 100 year rainfall event.

Advantages and disadvantages for the most feasible alternatives were considered by comparison of the alternative schemes to the evaluation criteria listed previously, presented in a matrix format. Evaluation criteria were weighted based on the relative importance of each criterion and each alternative was rated on its performance with respect to the criterion. The evaluation matrix is presented in Table 5.5 (d). Although some of the alternatives produce better results than others in any sewershed, only sewer separation addresses all of the objectives. The others are unable to manage overflows on their own. They typically require additional measures, particularly sewer capacity increases.

Costs used for comparison of alternative overflow reduction strategies refer to the systems able to accommodate flows from a 1 in 100 year design rainfall event. If a more frequent rainfall event is selected as the design objective and more frequent overflows are accepted, the level of effort on overflow reduction can be reduced. The relative magnitude of selecting a different overflow object ive can be seen by comparison of the capital cost curves generated for each overflow in Figures 5.5 (a, b, and c).

5.6 Conclusions and Recommendations

Conclusions reached and recommendations resulting from this assessment of overflow reduction strategies are presented in this section. Assessment of appropriate approaches to be taken in each overflow sewershed is presented in Chapter 6.

Feasible overflow reduction strategies for each sewershed are summarized and presented in Table 5.5 (d).

5.6.1 Conclusions

A number of conclusions were reached including the following:

- System capacity increases is the alternative that best suits the evaluation criteria in most of the sub-sewersheds, however it is not a strategy that can be implemented on a sewershed by sewershed basis, Every section must be completed for the system to function as suggested. It is the most comprehensive means of reducing pollutant loads to the Harbour as all flows are collected and receive treatment at the level set at the treatment plant. However it is the highest capital cost and life cycle cost alternative so may not be feasible to implement;
- Because of the magnitude of combined flows, the storage alternative is very costly in sewersheds serviced by combined sewers. Much smaller storage volumes are required to contain the flows generated by sanitary sewers subjected to larger extraneous flows. This alternative also provides a comprehensive means of reducing pollutant loads to the receiving waters as all flows would eventually be delivered to the Eastern Wastewater Treatment Facility for full treatment;
- Satellite treatment may be cost effective but will only achieve some solids reduction and reduction of pathogens. This may be a reasonable approach where the overflow is discharged to salt water (Courtenay Bay) but may only be an interim solution for discharges to fresh waters.

5.6.2 Recommendations for Short Term Overflow Reduction Strategies in Each Sewershed

Sewer separation is the preferred long term strategy for reduction of overflows in all sewersheds. With limited budgets available for the construction of new sewers, alternative strategies may be considered where overflow reduction is required in the short term.

Combinations of alternative strategies were considered to best meet all of the objectives for reduction of overflows resulting from the 1 in 100 year design rainfall event.

- Reduction of extraneous flows is typically the lowest cost alternative in areas that are currently serviced by separate storm and sanitary sewers and overflows occur from the sanitary sewers. It is the recommended strategy for overflow reduction from these areas;
- In the remaining sewersheds serviced by combined sewers, the preferred alternative is capacity upgrades and satellite treatment so is recommended as a short term alternative to sewer separation. Due to its ability to be implemented in a relatively short amount of time compared to separation of sewers, it is recommended as a short term measure to address overflow reductions if regulations require that they be addressed in the short term.

Table 5.3 (b) - Recommended Sewer Separation - Long Term Strategy

Overflow Location	Overflow Label	Probable Upstream Separation Cost ($)
Sewershed M		
Fox Den PS	OF-M15.1	$ -
Drury Cove PS	OF-M15.2	$ -
Parkhill Dr	OF-M06	$ 17,190,000
Walter St PS	OF-M02	$ -
Simpson Dr PS	OF-M05	$ 12,592,000
Major's Brook PS	OF-M14	$ 91,000
One Mile PS	OF-M13	$ 13,991,000
Sewershed N		
Park Avenue	OF-N01.1	$ 16,499,000
Dutchman's Creek	OF-N01.2	$ 8,949,000
Sewershed O		
SLS#3	OF-SLS#3	$ 7,376,000
Sewershed P		
SLS#2	OF-P02	$ -
SLS#1	OF-P03	$ 994,000
SLS#50	OF-P04	$ -
Sewershed Q		
Pauline St PS	OF-Q01	$ 5,834,000
Hickey Rd PS	OF-Q20	$ 10,526,000
York St PS	OF-Q10	$ 4,138,000
Champlain Dr PS	OF-Q11	$ 1,029,000
McAllister Ind Park PS	OF-Q15	$ -
Bayside Drive	OF-Q13	$ 14,885,000
Red Head Rd PS	OF-Q16	$ 8,361,000
Total for Study Area		**$ 122,455,000**

1. *Costs are in 2011 dollars and include 25% for engineering and contingency, they do not include HST.*

Table 5.3.2 - New Trunk Sanitary Sewers from Isolated Separate Upstream Systems

Overflow Location	Overflow Label	Total New Trunk Sewer Length (m)	Probable Cost of New Trunk Sewers from Isolated Separate Upstream Systems ($)
Sewershed M			
Fox Den PS	OF-M15.1	0	$ -
Drury Cove PS	OF-M15.2	0	$ -
Parkhill Dr	OF-M06	3872	$ 5,881,000
Walter St PS	OF-M02	0	$ -
Simpson Dr PS	OF-M05	934	$ 1,845,000
Major's Brook PS	OF-M14	0	$ -
One Mile PS	OF-M13	854	$ 1,396,000
Sewershed N			
Park Avenue	OF-N01.1	0	$ -
Dutchman's Creek	OF-N01.2	1694	$ 2,606,000
Sewershed O			
SLS#3	OF-SLS#3	0	$ -
Sewershed P			
SLS#2	OF-P02	0	$ -
SLS#1	OF-P03	0	$ -
SLS#50	OF-P04	0	$ -
Sewershed Q			
Pauline St PS	OF-Q01	0	$ -
Hickey Rd PS	OF-Q20	0	$ -
York St PS	OF-Q10	0	$ -
Champlain Dr PS	OF-Q11	0	$ -
McAllister Ind Park PS	OF-Q15	0	$ -
Bayside Drive	OF-Q13	1022	$ 1,468,000
Red Head Rd PS	OF-Q16	0	$ -
Total for Study Area		8376	**$ 13,196,000**

1. *Costs are in 2011 dollars and include 25% for engineering and contingency, they do not include HST.*

Table 5.4.2 - Probable Costs of Capacity Upgrades for Extraneous Flows

Overflow Location	Overflow Label	1:2 Month						
		Gravity Sewer	Pumping Station		Forcemain			Total Cost ($)
		Cost ($)	Design Flow (L/sec)	Cost ($)	Length (m)	Upgrade to Size (mm)	Cost ($)	
Sewershed M		$ -		$ -			$ -	$ -
Fox Den PS	OF-M15.1							$ -
Drury Cove PS	OF-M15.2							$ -
Parkhill Dr	OF-M06							$ -
Walter St PS	OF-M02							$ -
Simpson Dr PS	OF-M05							$ -
Major's Brook PS	OF-M14							$ -
One Mile PS	OF-M13							$ -
Sewershed N		$ -		$ -			$ -	$ -
Park Avenue	OF-N01.1							$ -
Dutchman's Creek	OF-N01.2							$ -
Sewershed O		$ -		$ -			$ -	$ -
SLS#3	OF-SLS#3							$ -
Sewershed P		$ -		$ -			$ -	$ -
SLS#2	OF-P02							$ -
SLS#1	OF-P03							$ -
SLS#50	OF-P04							$ -
Sewershed Q		$ -		$ -			$ -	$ -
Pauline St PS	OF-Q01							$ -
Hickey Rd PS	OF-Q20							$ -
York St PS	OF-Q10							$ -
Champlain Dr PS	OF-Q11							$ -
Bayside Drive	OF-Q13							$ -
McAllister Ind Park PS	OF-Q15							$ -
Red Head Rd PS	OF-Q16							$ -
Total for Study Area		$ -		$ -			$ -	$ -

Overflow Location	Overflow Label	1:6 Month						
		Gravity Sewer	Pumping Station		Forcemain			Total Cost ($)
		Cost ($)	Design Flow (L/sec)	Cost ($)	Length (m)	Size (mm)	Cost ($)	
Sewershed M		$ 3,879,000		$ 1,207,000			$ 7,579,100	**$ 12,665,100**
Fox Den PS	OF-M15.1	$ -						$ -
Drury Cove PS	OF-M15.2	$ -						$ -
Parkhill Dr	OF-M06	$ 1,202,000						**$ 1,202,000**
Walter St PS	OF-M02	$ -						$ -
Simpson Dr PS	OF-M05	$ 2,519,000	241	$ 470,000	1284	450	$ 2,965,641	**$ 5,954,641**
Major's Brook PS	OF-M14	$ 21,000	40	$ 156,000	10	100	$ 659,357	**$ 836,357**
One Mile PS	OF-M13	$ 137,000	618	$ 581,000	765	600	$ 3,954,102	**$ 4,672,102**
Sewershed N		$ 2,230,000		$ -			$ -	**$ 2,230,000**
Park Avenue	OF-N01.1	$ 1,036,000						**$ 1,036,000**
Dutchman's Creek	OF-N01.2	$ 1,194,000						**$ 1,194,000**
Sewershed O		$ 465,000		$ -			$ -	**$ 465,000**
SLS#3	OF-SLS#3	$ 465,000						**$ 465,000**
Sewershed P		$ -		$ -			$ -	$ -
SLS#2	OF-P02	$ -						$ -
SLS#1	OF-P03	$ -						$ -
SLS#50	OF-P04	$ -						$ -
Sewershed Q		$ 2,327,000		$ 1,078,000			$ 3,295,490	**$ 6,700,490**
Pauline St PS	OF-Q01	$ -						$ -
Hickey Rd PS	OF-Q20	$ 381,000	242	$ 470,000	817	450		**$ 851,000**
York St PS	OF-Q10	$ -	59	$ 259,000	455	200	$ 1,318,278	**$ 1,577,278**
Champlain Dr PS	OF-Q11	$ 1,111,000	126	$ 349,000	7	300	$ 1,977,213	**$ 3,437,213**
Bayside Drive	OF-Q13	$ 835,000						**$ 835,000**
McAllister Ind Park PS	OF-Q15	$ -						$ -
Red Head Rd PS	OF-Q16	$ -						$ -
Total for Study Area		$ 8,901,000		$ 2,285,000			$ 10,874,590	**$ 22,060,590**

Overflow Location	Overflow Label	1:1 Year						
		Gravity Sewer	Pumping Station		Forcemain			Total Cost ($)
		Cost ($)	Design Flow (L/sec)	Cost ($)	Length (m)	Size (mm)	Cost ($)	
Sewershed M		$ 5,983,000		$ 1,310,000			$ 8,238,021	**$ 15,531,021**
Fox Den PS	OF-M15.1	$ -						$ -
Drury Cove PS	OF-M15.2	$ -						$ -
Parkhill Dr	OF-M06	$ 1,368,000						**$ 1,368,000**
Walter St PS	OF-M02	$ -						$ -
Simpson Dr PS	OF-M05	$ 3,157,000	364	$ 470,000	1284	450	$ 2,965,641	**$ 6,592,641**
Major's Brook PS	OF-M14	$ 21,000	59	$ 259,000	10	200	$ 1,318,278	**$ 1,598,278**
One Mile PS	OF-M13	$ 1,437,000	792	$ 581,000	765	600	$ 3,954,102	**$ 5,972,102**
Sewershed N		$ 2,830,000		$ -			$ -	**$ 2,830,000**
Park Avenue	OF-N01.1	$ 1,628,000						**$ 1,628,000**
Dutchman's Creek	OF-N01.2	$ 1,202,000						**$ 1,202,000**
Sewershed O		$ 465,000		$ -			$ -	**$ 465,000**
SLS#3	OF-SLS#3	$ 465,000						**$ 465,000**
Sewershed P		$ 1,023,000		$ -			$ -	**$ 1,023,000**
SLS#2	OF-P02	$ -						$ -
SLS#1	OF-P03	$ 1,023,000						**$ 1,023,000**
SLS#50	OF-P04	$ -						$ -
Sewershed Q		$ 6,349,000		$ 1,166,000			$ 3,954,429	**$ 11,469,429**
Pauline St PS	OF-Q01	$ -						$ -
Hickey Rd PS	OF-Q20	$ 1,436,000	289	$ 470,000	817	450		**$ 1,906,000**
York St PS	OF-Q10	$ 672,000	76	$ 305,000	455	250	$ 1,647,743	**$ 2,624,743**
Champlain Dr PS	OF-Q11	$ 1,462,000	151	$ 391,000	7	350	$ 2,306,685	**$ 4,159,685**
Bayside Drive	OF-Q13	$ 2,779,000						**$ 2,779,000**
McAllister Ind Park PS	OF-Q15	$ -						$ -
Red Head Rd PS	OF-Q16	$ -						$ -
Total for Study Area		$ 16,650,000		$ 2,476,000			$ 12,192,450	**$ 31,318,450**

Max vel 2.4 m/sec

Overflow Location	Overflow Label	1:2 Year						
		Gravity Sewer	Pumping Station		Forcemain			Total
		Cost ($)	Design Flow (L/sec)	Cost ($)	Length (m)	Size (mm)	Cost ($)	Cost ($)
Sewershed M		$ 14,491,000		$ 1,512,000			$ 10,215,036	**$ 26,218,036**
Fox Den PS	OF-M15.1	$ -						$ -
Drury Cove PS	OF-M15.2	$ -						$ -
Parkhill Dr	OF-M06	$ 1,418,000						**$ 1,418,000**
Walter St PS	OF-M02	$ -						$ -
Simpson Dr PS	OF-M05	$ 11,255,000	462	$ 470,000	1284	450	$ 2,965,641	**$ 14,690,641**
Major's Brook PS	OF-M14	$ 122,000	61	$ 259,000	10	200	$ 1,318,278	**$ 1,699,278**
One Mile PS	OF-M13	$ 1,696,000	966	$ 783,000	765	900	$ 5,931,117	**$ 8,410,117**
Sewershed N		$ 7,114,000		$ -			$ -	**$ 7,114,000**
Park Avenue	OF-N01.1	$ 2,502,000						**$ 2,502,000**
Dutchman's Creek	OF-N01.2	$ 4,612,000						**$ 4,612,000**
Sewershed O		$ 2,050,000		$ -			$ -	**$ 2,050,000**
SLS#3	OF-SLS#3	$ 2,050,000						**$ 2,050,000**
Sewershed P		$ 1,131,000		$ -			$ -	**$ 1,131,000**
SLS#2	OF-P02	$ -						$ -
SLS#1	OF-P03	$ 1,131,000						**$ 1,131,000**
SLS#50	OF-P04	$ -						$ -
Sewershed Q		$ 7,952,000		$ 1,186,000			$ 4,119,167	**$ 13,257,167**
Pauline St PS	OF-Q01	$ -						$ -
Hickey Rd PS	OF-Q20	$ 2,719,000	327	$ 470,000	817	450		**$ 3,189,000**
York St PS	OF-Q10	$ 697,000	83	$ 305,000	455	250	$ 1,647,743	**$ 2,649,743**
Champlain Dr PS	OF-Q11	$ 1,464,000	172	$ 411,000	7	375	$ 2,471,423	**$ 4,346,423**
Bayside Drive	OF-Q13	$ 3,072,000						**$ 3,072,000**
McAllister Ind Park PS	OF-Q15	$ -						$ -
Red Head Rd PS	OF-Q16	$ -						$ -
Total for Study Area		$ 32,738,000		$ 2,698,000			$ 14,334,202	**$ 49,770,202**

Overflow Location	Overflow Label	1:5 Year						
		Gravity Sewer	Pumping Station		Forcemain			Total
		Cost ($)	Design Flow (L/sec)	Cost ($)	Length (m)	Size (mm)	Cost ($)	Cost ($)
Sewershed M		$ 16,348,000		$ 1,663,000			$ 11,697,817	**$ 29,708,817**
Fox Den PS	OF-M15.1	$ -						$ -
Drury Cove PS	OF-M15.2	$ -						$ -
Parkhill Dr	OF-M06	$ 1,621,000						**$ 1,621,000**
Walter St PS	OF-M02	$ -						$ -
Simpson Dr PS	OF-M05	$ 12,909,000	603	$ 526,900	1284	525	$ 3,459,868	**$ 16,895,768**
Major's Brook PS	OF-M14	$ 122,000	78	$ 259,100	10	200	$ 1,318,278	**$ 1,699,378**
One Mile PS	OF-M13	$ 1,696,000	1391	$ 877,000	765	1050	$ 6,919,671	**$ 9,492,671**
Sewershed N		$ 7,194,000		$ -			$ -	**$ 7,194,000**
Park Avenue	OF-N01.1	$ 2,572,000						**$ 2,572,000**
Dutchman's Creek	OF-N01.2	$ 4,622,000						**$ 4,622,000**
Sewershed O		$ 2,372,000		$ -			$ -	**$ 2,372,000**
SLS#3	OF-SLS#3	$ 2,372,000						**$ 2,372,000**
Sewershed P		$ 1,131,000		$ -			$ -	**$ 1,131,000**
SLS#2	OF-P02	$ -						$ -
SLS#1	OF-P03	$ 1,131,000						**$ 1,131,000**
SLS#50	OF-P04	$ -						$ -
Sewershed Q		$ 9,308,000		$ 1,289,000			$ 4,942,854	**$ 15,539,854**
Pauline St PS	OF-Q01	$ -						$ -
Hickey Rd PS	OF-Q20	$ 3,101,000	400	$ 470,000	817	450		**$ 3,571,000**
York St PS	OF-Q10	$ 712,000	107	$ 349,000	455	300	$ 1,977,213	**$ 3,038,213**
Champlain Dr PS	OF-Q11	$ 1,654,000	244	$ 470,000	7	450	$ 2,965,641	**$ 5,089,641**
Bayside Drive	OF-Q13	$ 3,841,000						**$ 3,841,000**
McAllister Ind Park PS	OF-Q15	$ -						$ -
Red Head Rd PS	OF-Q16	$ -						$ -
Total for Study Area		$ 36,353,000		$ 2,952,000			$ 16,640,671	**$ 55,945,671**

1. Costs are in 2011 dollars and include 25% for engineering and contingency, they do not include HST.

Table 5.4.3.1 - Summary of Overflow Volumes and Potential Storage Tank Costs

		Total Overflow Volume (m^3)									Proposed Storage Tanks								
Overflow Location	Overflow Label	Dry Weather	1:2 Month Event	1:6 Month Event	1:1 Year Event	1:2 Year Event	1:5 Year Event	1:10 Year Event	1:25 Year Event	1:100 Year Event	Tank ID	Subsewershed Location	Estimated Width (m)	Estimated Length (m)	Available Area (sq.m)	Depth (m)	Available Volume (cu.m)	Rain Event Stored With Available Volume	Estimated Storage Cost
Sewershed M		**0**	**0**	**2,067**	**6,758**	**10,909**	**18,511**	**24,538**	**32,223**	**42,270**							**43,900**		**$ 31,767,000**
Fox Den PS	OF-M15.1	0	0	0	0	0	0	0	0	0	-	-	-	-	0	0.0	0		$ -
Drury Cove PS	OF-M15.2	0	0	0	0	0	0	0	0	0	-	-	-	-	0	0.0	0		$ -
Parkhill Dr	OF-M06	0	0	64	1,638	3,305	5,956	8,062	10,634	13,397	TANK - M-06	M-06	25.2	132.3	3,334	4.2	14,000	1:100 year	$ 9,323,000
Walter St PS	OF-M02	0	0	0	0	0	0	0	0	0	-	-	-	-	0	0.0	0		$ -
Simpson Dr PS	OF-M05	0	0	0	0	0	0	0	0	0	-	-	-	-	0	0.0	0		$ -
Major's Brook PS	OF-M14	0	0	1,006	2,213	3,215	5,395	7,397	9,999	13,406	TANK - M-14	M-14	37.8	114.7	4,336	3.2	13,900	1:100 year	$ 9,272,000
One Mile PS	OF-M13	0	0	998	2,907	4,389	7,159	9,079	11,589	15,467	TANK - M-10.1	M-10	18.9	113	2,136	3.5	7,500	1:5 year	$ 5,755,000
											TANK - M-10.2	M-10	18.9	68.5	1,295	3.5	4,500	1:1 year	$ 3,877,000
											TANK - M-13	M-13	18.9	64	1,210	3.3	4,000	1:1 year	$ 3,540,000
											TOTAL ONE MILE PS TANKS						16,000	1:100 year	$ 13,172,000
Sewershed N		**0**	**1,410**	**9,506**	**17,037**	**24,648**	**38,457**	**47,989**	**59,889**	**77,243**							**77,600**		**$ 45,573,000**
Park Avenue	OF-N01.1	0	0	189	580	1,032	1,839	2,568	3,760	5,927	TANK - N-01.1	N-01	12.6	160	2,016	3.0	6,000	1:100 year	$ 4,843,000
Dutchman's Creek	OF-N01.2	0	1,410	9,317	16,456	23,615	36,619	45,421	56,129	71,316	TANK - N-01.2.1	N-01	18.9	273	5,160	3.0	15,500	1:6 month	$ 10,086,000
											TANK - N-01.2.2	N-01	18.9	180	3,402	3.0	10,200	1:6 month	$ 7,299,000
											TANK - N-01.2.3	N-01	113.4	135	15,309	3.0	45,900	1 in 10 year	$ 23,345,000
											TOTAL DUTCHMAN'S CREEK TANKS						71,600	1:100 year	$ 40,730,000
Sewershed O		**0**	**82**	**6,487**	**10,174**	**13,844**	**19,682**	**23,367**	**27,804**	**33,411**							**35,000**		**$ 18,931,000**
SLS#3	OF-SLS#3	0	82	6,487	10,174	13,844	19,682	23,367	27,804	33,411	TANK - SLS#3	O-01	75.6	154.3	11,665	3.0	35,000	1:100 year	$ 18,931,000
Sewershed P		**0**	**0**	**0**	**32**	**46**	**67**	**87**	**197**	**446**							**500**		**$ 709,000**
SLS#2	OF-P02	0	0	0	0	0	0	0	0	0	-	-	-	-	0	0.0	0	-	$ -
SLS#1	OF-P03	0	0	0	32	46	67	87	197	446	TANK - P-03	P-03	12.6	13.3	168	3.0	500	1:100 year	$ 709,000
SLS#50	OF-P04	0	0	0	0	0	0	0	0	0	-	-	-	-	0	0.0	0	-	$ -
Sewershed Q		**0**	**2,470**	**8,714**	**13,079**	**16,994**	**22,539**	**26,304**	**30,459**	**35,511**							**16,300**		**$ 14,292,000**
Pauline St PS	OF-Q01	0	0	0	0	0	23	124	297	609	TANK - Q-01	Q-01	12.6	17.2	217	3.0	700	1:100 year	$ 920,000
Hickey Rd PS	OF-Q20	0	0	0	0	0	0	0	0	0	-	-	-	-	0	0.0	0	-	$ -
York St PS	OF-Q10	0	1,556	4,743	6,908	8,846	12,224	14,535	16,993	19,704	TANK - Q-10.1	Q-10	12.6	35	441	3.0	1,300	NA	$ 1,485,000
											TANK - Q-10.2	Q-10	6.3	235	1,481	3.3	4,900	1:6 month	$ 4,141,000
											TOTAL YORK ST TANKS						6,200	1:6 month	$ 5,626,000
Champlain Dr PS	OF-Q11	0	913	3,972	6,171	8,147	10,292	11,645	13,169	15,199	TANK - Q-11.1	Q-11	44.1	56.7	2,500	3.0	7,500	1:1 year	$ 5,755,000
											TANK - Q-11.2	Q-11	25.2	25.2	635	3.0	1,900	1:2 month	$ 1,991,000
											TOTAL CHAMPLAIN DR TANKS						9,400	1:2 year	$ 7,746,000
Bayside Drive	OF-Q13	0	0	0	0	0	0	0	0	0	-	-	-	-	0	0.0	0	-	$ -
McAllister Ind Park PS	OF-Q15	0	0	0	0	0	0	0	0	0	-	-	-	-	0	0.0	0	-	$ -
Red Head Rd PS	OF-Q16	0	0	0	0	0	0	0	0	0	-	-	-	-	0	0.0	0	-	$ -
Total for Study Area		**0**	**3,961**	**26,775**	**47,080**	**66,439**	**99,256**	**122,286**	**150,571**	**188,882**							**173,300**		**$ 111,272,000**

1. *Costs are in 2011 dollars and include 25% for engineering and contingency, they do not include HST.*

Table 5.4.4 - Probable Cost of Satellite Treatment

		Peak Overflow (L/sec)									Cost of Overflow Treatment ($)								
Overflow Location	Overflow Label	Dry Weather	1:2 Month Event	1:6 Month Event	1:1 Year Event	1:2 Year Event	1:5 Year Event	1:10 Year Event	1:25 Year Event	1:100 Year Event	Dry Weather	1:2 Month Event	1:6 Month Event	1:1 Year Event	1:2 Year Event	1:5 Year Event	1:10 Year Event	1:25 Year Event	1:100 Year Event
Sewershed M											$ -	$ -	$ 5,215,000	$ 7,299,000	$ 8,242,000	$ 8,953,000	$ 9,547,000	$ 9,986,000	$ 10,397,000
Fox Den PS	OF-M15.1	0	0	0	0	0	0	0	0	0	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Drury Cove PS	OF-M15.2	0	0	0	0	0	0	0	0	0	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Parkhill Dr	OF-M06	0	0	7	62	106	128	161	181	189	$ -	$ -	$ 978,000	$ 2,169,000	$ 2,638,000	$ 2,827,000	$ 3,077,000	$ 3,210,000	$ 3,263,000
Walter St PS	OF-M02	0	0	0	0	0	0	0	0	0	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Simpson Dr PS	OF-M05	0	0	0	0	0	0	0	0	0	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Major's Brook PS	OF-M14	0	0	34	52	65	83	101	115	134	$ -	$ -	$ 1,743,000	$ 2,024,000	$ 2,206,000	$ 2,414,000	$ 2,588,000	$ 2,720,000	$ 2,875,000
One Mile PS	OF-M13	0	0	91	166	212	269	304	343	392	$ -	$ -	$ 2,494,000	$ 3,106,000	$ 3,398,000	$ 3,712,000	$ 3,882,000	$ 4,056,000	$ 4,259,000
Sewershed N											$ -	$ 3,770,000	$ 7,631,000	$ 9,043,000	$ 9,959,000	$ 11,305,000	$ 12,164,000	$ 13,061,000	$ 14,193,000
Park Avenue	OF-N01.1	0	0	94	212	311	503	664	850	1117	$ -	$ -	$ 2,527,000	$ 3,402,000	$ 3,913,000	$ 4,668,000	$ 5,169,000	$ 5,659,000	$ 6,256,000
Dutchman's Creek	OF-N01.2	0	281	641	842	1017	1312	1514	1766	2136	$ -	$ 3,770,000	$ 5,104,000	$ 5,641,000	$ 6,046,000	$ 6,637,000	$ 6,995,000	$ 7,402,000	$ 7,937,000
Sewershed O											$ -	$ 1,882,000	$ 3,527,000	$ 3,527,000	$ 3,527,000	$ 3,527,000	$ 3,527,000	$ 3,527,000	$ 3,527,000
SLS#3	OF-SLS#3	0	42	234	234	234	234	234	234	234	$ -	$ 1,882,000	$ 3,527,000	$ 3,527,000	$ 3,527,000	$ 3,527,000	$ 3,527,000	$ 3,527,000	$ 3,527,000
Sewershed P											$ -	$ -	$ -	$ 2,280,000	$ 2,479,000	$ 2,654,000	$ 2,663,000	$ 2,663,000	$ 2,663,000
SLS#2	OF-P02	0	0	0	0	0	0	0	0	0	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
SLS#1	OF-P03	0	0	0	71	90	108	109	109	109	$ -	$ -	$ -	$ 2,280,000	$ 2,479,000	$ 2,654,000	$ 2,663,000	$ 2,663,000	$ 2,663,000
SLS#50	OF-P04	0	0	0	0	0	0	0	0	0	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Sewershed Q											$ -	$ -	$ 3,872,000	$ 4,375,000	$ 4,747,000	$ 6,035,000	$ 6,549,000	$ 6,910,000	$ 7,235,000
Pauline St PS	OF-Q01	0	0	0	0	0	8	19	32	51	$ -	$ -	$ -	$ -	$ -	$ 1,021,000	$ 1,399,000	$ 1,699,000	$ 2,018,000
Hickey Rd PS	OF-Q20	0	0	0	0	0	0	0	0	0	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
York St PS	OF-Q10	0	0	46	63	78	103	120	127	127	$ -	$ -	$ 1,937,000	$ 2,176,000	$ 2,354,000	$ 2,613,000	$ 2,763,000	$ 2,820,000	$ 2,820,000
Champlain Dr PS	OF-Q11	0	0	46	65	81	82	81	81	82	$ -	$ -	$ 1,935,000	$ 2,199,000	$ 2,393,000	$ 2,401,000	$ 2,387,000	$ 2,391,000	$ 2,397,000
Bayside Drive	OF-Q13	0	0	0	0	0	0	0	0	0	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
McAllister Ind Park PS	OF-Q15	0	0	0	0	0	0	0	0	0	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Red Head Rd PS	OF-Q16	0	0	0	0	0	0	0	0	0	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Total for Study Area											$ -	$ 5,652,000	$ 20,245,000	$ 26,524,000	$ 28,954,000	$ 32,474,000	$ 34,450,000	$ 36,147,000	$ 38,015,000

Table 5.5 (a) - Assessment of Flows by Overflow Location

Assessed Extraneous Flow Impact	
Low	Estimated peak flow is equal to or less than 4 times the average dry weather flow
Medium	Estimated peak flow is greater than 4 times the average dry weather flow but less than the Atlantic Canada Standards Design Flow
High	Estimated peak flow is greater than the Atlantic Canada Design Flow but less than 4 times the Atlantic Canada Standards Design Flow
Extreme	Estimated peak flow is greater than 4 times the Atlantic Canada Standards Design Flow
	Indicates low extraneous flow impact and zero overflows per year

Overflow Location	Overflow Label	Extraneous Flow Model Based On Flow Monitoring Sites:	Estimated Tributary Equivalent Population (capita)	Gross Tributary Area (ha)	4 X Average Dry Weather Flow at 340 L/cap/day (L/s)	Design Flow (L/s)	Estimated Tributary 1:100 Year Peak Flow (SewerGems Model) (L/s)	Estimated Number of Overflows Per Year	Assessed Extraneous Flow Impact
Sewershed M									
Fox Den PS	OF-M15.1	Phase 1-Site 9	8	3.5	0.1	0.6	0	0	Low
Drury Cove PS	OF-M15.2	Phase 1-Site 9	80	31.7	1.3	5.8	1	0	Low
Overflow on Parkhill Drive	OF-M06	Phase 5-Sites 3,5,6	1971	154.2	31.0	49.4	329	3	Extreme
Walter Street PS	OF-M02	Phase 1-Site 9	305	20.4	4.8	7.7	3	0	Low
Simpson Drive PS	OF-M05	Phase 1-Site 9	6989	465.4	110.0	150.6	211	0	High
Major's Brook PS	OF-M14	MB-Site 1,4,5	1231	43.0	19.4	24.1	151	2	Extreme
One Mile PS	OF-M13	Phase 1-Sites 1,5 & Phase 4-Site 5	14304	788.0	225.2	267.9	726	2	High
Sewershed N									
Overflow at Park Avenue	OF-N01.1	Phase 4-Site 2C	1149	29.3	18.1	21.1	971	3	Extreme
Overflow at Dutchman's Creek	OF-N01.2	Phase-Sites 1,2B	5168	142.0	81.3	85.6	2077	7	Extreme
Sewershed O									
SLS#3	OF-SLS#3	Phase 4-Site 3,4	1163	63.0	18.3	26.0	1226	6	Extreme
Sewershed P									
SLS#2	OF-P02	Typical DWF	1618	10.4	25.5	24.7	8	0	Low
SLS#1	OF-P03	SJ Harbour Cleanup-Site 4	183	16.5	2.9	5.3	89	1	Extreme
SLS#50	OF-P04	Typical DWF	25	13.0	0.4	2.2	0	0	Low
Sewershed Q									
Pauline Street PS	OF-Q01	Phase 2-Site 1	1023	79.0	16.1	26.3	110	0.5	Extreme
Hickey Road PS	OF-Q20	Phase 2-Site 1,3 & Phase 3-Site 1	5357	388.6	84.3	122.2	163	0	High
York Street PS	OF-Q10	Phase 3-Site 2	360	14.9	5.7	7.8	128	6	Extreme
Champlain Drive PS	OF-Q11	Phase 3-Site 2	697	31.3	11.0	15.1	153	5	Extreme
Overflow on Bayside Drive	OF-Q13	Phase 3-Site 2	8910	630.0	140.3	193.5	740	0	High
McAllister Industrial Park PS	OF-Q15	Phase 3-Site 3	2421	269.3	38.1	71.2	70	0	Medium
Red Head Road PS	OF-Q16	Phase 3-Site 3	913	94.6	14.4	27.0	44	0	High

1. *Estimated peak flows represent actual inflows to the interceptor system - upstream overflows and manhole flooding not included.*
2. *Assessments are based on reducing I/I to the level required to prevent an overflow during the 1 in 100 year rainfall event.*
3. *I/I Reduction in combined sewersheds will consist of sewer separation to some degree followed by other inflow reduction measures.*
4. *Atlantic Canada Standards guideline for ADWF = 340 L/cap/day*
5. *Atlantic Canada Standards formula for Design Flow used in the design of sanitary sewers: peak sanitary flow Q = (P*q*M / 86.4 + I*A)*

Factor of Safety	1
ADWF=	340 L/cap/day
I/I Allowance	0.14 L/sec/ha

Table 5.5 (b) - Applicable Overflow Reduction Strategies

Overflow Location	Overflow Label	Associated Waterbody	Assessed Extraneous Flow Characteristic	Most Applicable Overflow Reduction Strategies				
				Sewer Separation	I/I Reduction	Interceptor Capacity Increases	Storage	Overflow Treatment
Sewershed M								
Fox Den PS	OF-M15.1	Kennebecasis River	Low					
Drury Cove PS	OF-M15.2	Kennebecasis River	Low					
Overflow on Parkhill Drive	OF-M06	Marsh Creek	Extreme	Applicable	Applicable	Applicable	Applicable	
Walter Street PS	OF-M02	Marsh Creek	Low					
Simpson Drive PS	OF-M05	Marsh Creek	High	Applicable	Applicable	Applicable	Applicable	
Major's Brook PS	OF-M14	Marsh Creek	Extreme	Applicable	Applicable	Applicable	Applicable	
One Mile PS	OF-M13	Marsh Creek	High	Applicable	Applicable	Applicable	Applicable	Applicable
Sewershed N								
Overflow at Park Avenue	OF-N01.1	Marsh Creek	Extreme	Applicable		Applicable	Applicable	Applicable
Overflow at Dutchman's Creek	OF-N01.2	Marsh Creek	Extreme	Applicable		Applicable	Applicable	Applicable
Sewershed O								
SLS#3	OF-SLS#3	Saint John Harbour	Extreme	Applicable		Applicable	Applicable	Applicable
Sewershed P								
SLS#2	OF-P2	Saint John Harbour	Low					
SLS#1	OF-P3	Saint John Harbour	Extreme	Applicable	Applicable	Applicable	Applicable	Applicable
SLS#50	OF-P4	Saint John Harbour	Low					
Sewershed Q								
Pauline Street PS	OF-Q01	Marsh Creek	Extreme	Applicable	Applicable		Applicable	
Hickey Road PS	OF-Q20	Little River	High	Applicable	Applicable	Applicable	Applicable	
York Street PS	OF-Q10	Little River	Extreme	Applicable	Applicable	Applicable	Applicable	
Champlain Drive PS	OF-Q11	Hazen Creek	Extreme	Applicable	Applicable	Applicable	Applicable	
Overflow on Bayside Drive	OF-Q13	Hazen Creek	High	Applicable	Applicable	Applicable	Applicable	Applicable
McAllister Industrial Park PS	OF-Q15	Hazen Creek	Medium		Applicable			
Red Head Road PS	OF-Q16	Beyea Brook	High	Applicable	Applicable			

Assessed Extraneous Flow Impact	
Low	Estimated peak flow is equal to or less than 4 times the average dry weather flow
Medium	Estimated peak flow is greater than 4 times the average dry weather flow but less than the Atlantic Canada Standards Design Flow
High	Estimated peak flow is greater than the Atlantic Canada Design Flow but less than 4 times the Atlantic Canada Standards Design Flow
Extreme	Estimated peak flow is greater than 4 times the Atlantic Canada Standards Design Flow

1. *Strategies that are applicable have been considered further. Some of these strategies require combinations to completely mitigate the impacts of overflows.*

Table 5.5 (c) - Probable Costs of Alternative Overflow Reduction Strategies for 1 in 100 Year Event

Notes:

1. Life Cycle Cost based on the following factors:

    100 years
    5% per annum inflation
    19.48 Present Worth Factor for Annual Operating & Maintenance Costs

2. Assessment is based on Capacity to Accommodate Peak Flows from 1 in 100 Year Rainfall Event
3. I/I Reduction Assessment is based on Estimates of Probable Costs for I/I Reduction in the Beach Cresent Study: $ 37,000.00 per hectare

Overflow Location	Sewer Separation			I/I Reduction			Interceptor and Treatment Capacity Increases			Storage			Satellite Treatment		
	Capital Cost	O & M Cost	Life Cycle Cost	Capital Cost	O & M Cost	Life Cycle Cost	Capital Cost	O & M Cost	Life Cycle Cost	Capital Cost	O & M Cost	Life Cycle Cost	Capital Cost	O & M Cost	Life Cycle Cost
Sewershed M	**$ 43,864,000**	**$ 48,308**	**$ 44,805,240**	**$ 29,292,000**	**$ 119,630**	**$ 43,574,015**	**$ 29,708,817**	**$ 9,656**	**$ 33,891,108**	**$ 28,097,000**	**$ 14,475**	**$ 29,525,390**	**$ 10,397,000**	**$ 97,208**	**$ 15,472,476**
Fox Den PS	-	-	-	-	-	-	$ -	-	-	-	-	-	-	-	-
Drury Cove PS	-	-	-	-	-	-	$ -	-	-	-	-	-	-	-	-
Parkhill Dr	$ 17,190,000	$ 19,287	$ 17,565,783	$ 5,739,500	$ 31,367	$ 8,692,375	$ 1,621,000	$ 1,000	$ 1,640,481	$ 9,011,000	$ 6,093	$ 9,497,370	$ 3,263,000	$ 36,022	$ 4,963,339
Walter St PS	-	-	-	-	-	-	$ -	-	-	-	-	-	-	-	-
Simpson Dr PS	$ 12,592,000	$ 13,239	$ 12,849,943	$ 11,548,500	$ 41,474	$ 17,068,375	$ 16,895,768	$ 7,027	$ 18,039,535	-	-	-	-	-	-
Major's Brook PS	$ 91,000	$ 101	$ 92,962	$ 1,624,500	$ 12,447	$ 2,529,817	$ 1,699,378	$ 109	$ 2,029,049	$ 9,016,000	$ 4,063	$ 9,463,025	$ 2,875,000	$ 29,726	$ 4,333,928
One Mile PS	$ 13,991,000	$ 15,682	$ 14,296,553	$ 10,379,500	$ 34,342	$ 15,283,448	$ 9,492,671	$ 1,520	$ 12,182,042	$ 10,070,000	$ 4,318	$ 10,564,995	$ 4,259,000	$ 31,459	$ 6,175,209
Sewershed N	**$ 25,448,000**	**$ 27,060**	**$ 25,975,228**	**$ 5,322,000**	**$ 35,931**	**$ 8,193,460**	**$ 7,194,000**	**$ 4,255**	**$ 8,726,502**	**$ 37,616,000**	**$ 33,273**	**$ 39,799,021**	**$ 14,193,000**	**$ 150,992**	**$ 21,477,995**
Park Avenue	$ 16,499,000	$ 17,047	$ 16,831,139	$ 1,118,500	$ 12,186	$ 1,812,288	$ 2,572,000	$ 1,758	$ 3,124,512	$ 4,797,000	$ 4,306	$ 5,076,624	$ 6,256,000	$ 45,389	$ 9,054,705
Dutchman's Creek	$ 8,949,000	$ 10,013	$ 9,144,090	$ 4,203,500	$ 23,745	$ 6,381,172	$ 4,622,000	$ 2,497	$ 5,601,990	$ 32,819,000	$ 28,966	$ 34,722,397	$ 7,937,000	$ 105,603	$ 12,423,290
Sewershed O	**$ 7,376,000**	**$ 7,946**	**$ 7,530,823**	**$ 2,364,500**	**$ 15,956**	**$ 3,640,103**	**$ 2,372,000**	**$ 1,791**	**$ 2,884,858**	**$ 18,263,000**	**$ 17,980**	**$ 19,358,458**	**$ 3,527,000**	**$ 54,698**	**$ 5,671,993**
SLS#3	$ 7,376,000	$ 7,946	$ 7,530,823	$ 2,364,500	$ 15,956	$ 3,640,103	$ 2,372,000	$ 1,791	$ 2,884,858	$ 18,263,000	$ 17,980	$ 19,358,458	$ 3,527,000	$ 54,698	$ 5,671,993
Sewershed P	**$ 994,000**	**$ 1,187**	**$ 1,017,131**	**$ 644,500**	**$ 7,490**	**$ 1,053,399**	**$ 1,131,000**	**$ 1,186**	**$ 1,382,006**	**$ 650,000**	**$ 477**	**$ 685,823**	**$ 2,663,000**	**$ 23,903**	**$ 3,943,600**
SLS#2	-	-	-	-	-	-	$ -	-	-	-	-	-	-	-	-
SLS#1	$ 994,000	$ 1,187	$ 1,017,131	$ 644,500	$ 7,490	$ 1,053,399	$ 1,131,000	$ 1,186	$ 1,382,006	$ 650,000	$ 477	$ 685,823	$ 2,663,000	$ 23,903	$ 3,943,600
SLS#50	-	-	-	-	-	-	$ -	-	-	-	-	-	-	-	-
Sewershed Q	**$ 44,773,000**	**$ 51,447**	**$ 45,775,393**	**$ 33,511,500**	**$ 175,810**	**$ 50,609,682**	**$ 15,539,854**	**$ 7,590**	**$ 18,819,010**	**$ 22,902,000**	**$ 25,350**	**$ 24,330,313**	**$ 7,235,000**	**$ 90,846**	**$ 11,218,962**
Pauline St PS	$ 5,834,000	$ 6,412	$ 5,958,935	$ 2,956,500	$ 19,961	$ 4,551,668	$ -	-	-	$ 826,000	$ 272	$ 865,009	$ 2,018,000	$ 20,979	$ 3,044,265
Hickey Rd PS	$ 10,526,000	$ 13,963	$ 10,798,062	$ 11,489,500	$ 52,953	$ 17,208,948	$ 3,571,000	$ 2,806	$ 4,345,233	-	-	-	-	-	-
York St PS	$ 4,138,000	$ 4,895	$ 4,233,369	$ 585,500	$ 9,623	$ 1,011,882	$ 3,038,213	$ 735	$ 3,664,731	$ 12,142,000	$ 14,361	$ 12,917,210	$ 2,820,000	$ 18,250	$ 4,038,503
Champlain Dr PS	$ 1,029,000	$ 1,176	$ 1,051,913	$ 640,500	$ 9,228	$ 1,081,622	$ 5,089,641	$ 1,308	$ 6,140,689	$ 9,934,000	$ 10,716	$ 10,548,094	$ 2,397,000	$ 51,617	$ 4,136,194
Bayside Drive	$ 14,885,000	$ 15,882	$ 15,194,453	$ 7,807,500	$ 39,569	$ 11,763,918	$ 3,841,000	$ 2,740	$ 4,668,357	-	-	-	-	-	-
McAllister Ind Park PS	-	-	-	$ 6,497,500	$ 19,654	$ 9,531,411	$ -	-	-	-	-	-	-	-	-
Red Head Rd PS	$ 8,361,000	$ 9,118	$ 8,538,661	$ 3,534,500	$ 24,823	$ 5,460,231	$ -	-	-	-	-	-	-	-	-
Total	**$ 122,455,000**	**$ 135,948**	**$ 125,103,816**	**$ 71,134,500**	**$ 354,818**	**$ 107,070,658**	**$ 55,945,671**	**$ 24,478**	**$ 65,703,485**	**$ 107,528,000**	**$ 91,555**	**$ 113,699,004**	**$ 38,015,000**	**$ 417,647**	**$ 57,785,026**

1. Storage volumes specified will reduce the number of overflows to 1 in 100 years. No additional measures are required to meet this objective.
2. I/I Reduction will require Sewer Separation in truly combined sewer areas to accomplish inflow reductions.
3. Capacity increases will eliminate surcharging in the system for the flows resulting from a 1 in 100 year rainfall event. Untreated overflows will result without additional measures. The most suitable additional measure is storage of overflows upstream of overflow locations.
4. The most suitable location for overflow treatment is just upstream of the overflow locations. The level of treatment would need to be more than advanced primary, at higher costs, to treat overflows into freshwater.
5. Sewer Separation cost includes costs of complete separation including separate deep storm sewers (new or re-use existing if possible) and sanitary sewers, including separate laterals to properties.

Table 5.5 (d) Evaluation Matrix of Alternatives by Overflow Location

Evaluation Criteria Weighting Methodology	
Criteria	*Evaluation of Strategies*
Life Cycle Cost	1=highest cost, 5=lowest cost
Reduction in Overflows	1=lowest reduction, 5=highest reduction
Improved Receiving Water Quality	1=least improved, 5=most improved
Public Disruption	1=highest impact, 5=lowest impact
Land Requirements	1=land required, 5=no land required
Reliability	1=least reliable, 5=most reliable
Compatibility with Existing System	1=least compatible, 5=most compatible

Overflow Location	Estimated Number of Overflows Per Year	Extraneous Flow Impact	Evaluation Criteria	Weighting Factor 1 to 7 (1=minimal impact at this site; 7=large impact at this site)	Overflow Reduction Strategies Strategies are rated 1 to 5 for each evaluation criteria as described above					LONG-TERM Overflow Reduction Strategy	SHORT-TERM Overflow Reduction Strategy	Comments
					Sewer Separation	I & I Reduction	Capacity Increases	Storage	Satellite Treatment			
Overflow on Parkhill Drive	3	Extreme	Life Cycle Cost	7	1	3	4	2		Sewer Separation	I & I Reduction	Sewer separation is the preferred option for reduction in overflows. I/I reduction would be the second best alternative which could be used in the short term for the tributary area.
			Reduction in Overflows	6	4	2	1	3				
			Improved Receiving Water Quality	5	4	2	1	3				
			Public Disruption	4	1	3	2	4				
			Land Requirements	3	4	3	2	1				
			Reliability	2	4	1	2	3				
			Compatibility with Existing System	1	3	4	2	1				
				Weighted Total	78	70	59	73	0			
Simpson Drive PS	0	High	Life Cycle Cost	7	3	2	1			Sewer Separation	I & I Reduction	Sewer separation is the preferred option for reduction in overflows. I/I reduction would be the second best alternative which could be used in the short term for the tributary area. Storage and treatment of overflows were not an option here because there were no overflows produced at this location. There was significant surcharging in tributary sewers, indicating high storm water influence, leaving the other options viable for evaluation.
			Reduction in Overflows	6	3	2	1					
			Improved Receiving Water Quality	5	3	2	1					
			Public Disruption	4	1	3	2					
			Land Requirements	3	3	2	1					
			Reliability	2	3	1	2					
			Compatibility with Existing System	1	2	3	1					
				Weighted Total	75	59	34	0	0			
Major's Brook PS	2	Extreme	Life Cycle Cost	7	4	2	3	1		Sewer Separation	See Long-term Strategy (Sewer Separation)	Sewer separation is the preferred option for reduction in overflows. I/I reduction would be the second best alternative which could be used in the short term for the tributary area. Storage could also be a good short term option having a rating very close to I/I reduction at this location.
			Reduction in Overflows	6	4	2	1	3				
			Improved Receiving Water Quality	5	4	2	1	3				
			Public Disruption	4	1	3	2	4				
			Land Requirements	3	4	3	2	1				
			Reliability	2	4	1	3	2				
			Compatibility with Existing System	1	4	3	2	1				
				Weighted Total	100	62	54	64	0			
One Mile PS	2	High	Life Cycle Cost	7	3	1	2	4	5	Sewer Separation	I & I Reduction	Sewer separation is the preferred option for reduction in overflows. I/I reduction would be the second best alternative which could be used in the short term for the tributary area.
			Reduction in Overflows	6	5	4	2	3	1			
			Improved Receiving Water Quality	5	5	3	1	4	2			
			Public Disruption	4	1	5	2	4	3			
			Land Requirements	3	5	4	3	1	2			
			Reliability	2	5	2	1	4	3			
			Compatibility with Existing System	1	5	4	3	1	2			
				Weighted Total	110	86	53	94	77			
Overflow at Park Avenue	3	Extreme	Life Cycle Cost	7	1		2	3	4	Storage	Storage	Sewer separation is the preferred option for reduction of overflows. I/I reduction would reduce the volume of overflows in the short term but would likely not eliminate overflows. Storage would be a good short term option at this location.
			Reduction in Overflows	6	4		2	3	1			
			Improved Receiving Water Quality	5	4		1	3	2			
			Public Disruption	4	1		2	4	3			
			Land Requirements	3	4		3	1	2			
			Reliability	2	4		2	3	1			
			Compatibility with Existing System	1	4		3	2	1			
				Weighted Total	79	0	55	81	65			
Overflow at Dutchman's Creek	7	Extreme	Life Cycle Cost	7	3		4	1	2	Sewer Separation	See Long-term Strategy (Sewer Separation)	Sewer separation is the preferred option for reduction of overflows. I/I reduction would reduce the volume of overflows in the short term but would likely not eliminate overflows. Capacity increases plus overflow treatment would be a good short term solution.
			Reduction in Overflows	6	4		2	3	1			
			Improved Receiving Water Quality	5	4		1	3	2			
			Public Disruption	4	1		2	3	4			
			Land Requirements	3	4		3	1	2			
			Reliability	2	4		2	3	1			
			Compatibility with Existing System	1	4		3	1	2			
				Weighted Total	93	0	69	62	56			
SLS#3	6	Extreme	Life Cycle Cost	7	2		4	1	3	Sewer Separation	Capacity Increases	Sewer separation is the preferred option for reduction of overflows. I/I reduction would reduce the volume of overflows in the short term but would likely not eliminate overflows. Capacity increases plus overflow treatment would be a good short term solution.
			Reduction in Overflows	6	4		2	3	1			
			Improved Receiving Water Quality	5	4		1	3	2			
			Public Disruption	4	1		2	3	4			
			Land Requirements	3	4		3	1	2			
			Reliability	2	4		3	2	1			
			Compatibility with Existing System	1	4		3	1	2			
				Weighted Total	86	0	71	60	63			

Table 5.5 (d) Evaluation Matrix of Alternatives by Overflow Location

Evaluation Criteria Weighting Methodology	
Criteria	*Evaluation of Strategies*
Life Cycle Cost	1=highest cost, 5=lowest cost
Reduction in Overflows	1=lowest reduction, 5=highest reduction
Improved Receiving Water Quality	1=least improved, 5=most improved
Public Disruption	1=highest impact, 5=lowest impact
Land Requirements	1=land required, 5=no land required
Reliability	1=least reliable, 5=most reliable
Compatibility with Existing System	1=least compatible, 5=most compatible

Overflow Location	Estimated Number of Overflows Per Year	Extraneous Flow Impact	Evaluation Criteria	Weighting Factor 1 to 7 (1=minimal impact at this site; 7=large impact at this site)	Overflow Reduction Strategies Strategies are rated 1 to 5 for each evaluation criteria as described above					LONG-TERM Overflow Reduction Strategy	SHORT-TERM Overflow Reduction Strategy	Comments
					Sewer Separation	I & I Reduction	Capacity Increases	Storage	Satellite Treatment			
SLS#1	1	Extreme	Life Cycle Cost	7	4	3	2	5	1	**Sewer Separation**	**Storage**	Sewer separation is the preferred option for reduction in overflows. Storage would be the second best alternative which could be used in the short term for the tributary area if land is available.
			Reduction in Overflows	6	5	3	2	4	1			
			Improved Receiving Water Quality	5	5	2	1	4	3			
			Public Disruption	4	1	3	2	5	4			
			Land Requirements	3	5	4	3	1	2			
			Reliability	2	5	1	3	4	2			
			Compatibility with Existing System	1	5	4	1	3	2			
			Weighted Total		117	79	55	113	56			
Pauline Street PS	0.5	Extreme	Life Cycle Cost	7	1	2		3		**Storage**	**Storage**	Sewer separation is the preferred option for reduction in overflows. Storage would be the second best alternative which could be used in the short term to reduce overflows if land is available.
			Reduction in Overflows	6	3	1		2				
			Improved Receiving Water Quality	5	3	1		2				
			Public Disruption	4	1	2		3				
			Land Requirements	3	3	2		1				
			Reliability	2	3	1		2				
			Compatibility with Existing System	1	3	2		1				
			Weighted Total		62	43	0	63	0			
Hickey Road PS	0	High	Life Cycle Cost	7	2	1	3			**Sewer Separation**	**See Long-term Strategy (Sewer Separation)**	Sewer separation is the preferred option for reduction in overflows. Storage is an option here to reduce the significant surcharging in the majority of tributary sewers but it would need to be distributed in the sewershed.
			Reduction in Overflows	6	3	2	1					
			Improved Receiving Water Quality	5	3	2	1					
			Public Disruption	4	1	3	2					
			Land Requirements	3	3	2	1					
			Reliability	2	3	1	2					
			Compatibility with Existing System	1	3	2	1					
			Weighted Total		69	51	48	0	0			
York Street PS	6	Extreme	Life Cycle Cost	7	2	3	4	1		**Sewer Separation**	**I & I Reduction**	Sewer separation is the preferred option for reduction in overflows. I/I reduction has an equal rating which could be used in the short term for the tributary area.
			Reduction in Overflows	6	4	3	1	2				
			Improved Receiving Water Quality	5	4	3	1	2				
			Public Disruption	4	1	4	2	3				
			Land Requirements	3	4	2	3	1				
			Reliability	2	4	3	2	1				
			Compatibility with Existing System	1	4	3	2	1				
			Weighted Total		86	85	62	47	0			
Champlain Drive PS	5	Extreme	Life Cycle Cost	7	4	3	2	1		**Sewer Separation**	**I & I Reduction**	Sewer separation is the preferred option for reduction in overflows. I/I reduction would be the second best option which could be used in the short term for the tributary area.
			Reduction in Overflows	6	4	3	1	2				
			Improved Receiving Water Quality	5	4	3	1	2				
			Public Disruption	4	1	4	2	3				
			Land Requirements	3	4	3	2	1				
			Reliability	2	4	3	2	1				
			Compatibility with Existing System	1	4	3	2	1				
			Weighted Total		100	88	45	47	0			
Overflow on Bayside Drive	0	High	Life Cycle Cost	7	3	2	1			**Sewer Separation**	**I & I Reduction**	Sewer separation is the preferred option for reduction in overflows. Storage is an option here to reduce the significant surcharging in the majority of tributary sewers but it would need to be distributed in the sewershed.
			Reduction in Overflows	6	3	2	1					
			Improved Receiving Water Quality	5	3	2	1					
			Public Disruption	4	1	3	2					
			Land Requirements	3	3	1	2					
			Reliability	2	3	2	1					
			Compatibility with Existing System	1	3	2	1					
			Weighted Total		76	57	35	0	0			
McAllister Industrial Park PS	0	Medium	Life Cycle Cost	7		1				**I & I Reduction**	**See Long-term Strategy (I & I Reduction)**	There were no overflows generated at this location and no capacity issues in the upstream pipes. The existing sanitary sewer system were considered to be constructed to City Specifications, there are deep storm and PVC sanitary sewers. Sewer separation is not required. Flow data from this area still showed signs of I/I influence so I/I reduction should still be considered, but this areas would be considered low priority compared to areas with overflows.
			Reduction in Overflows	6		1						
			Improved Receiving Water Quality	5		1						
			Public Disruption	4		1						
			Land Requirements	3		1						
			Reliability	2		1						
			Compatibility with Existing System	1		1						
			Weighted Total		0	28	0	0	0			

Table 5.5 (d) Evaluation Matrix of Alternatives by Overflow Location

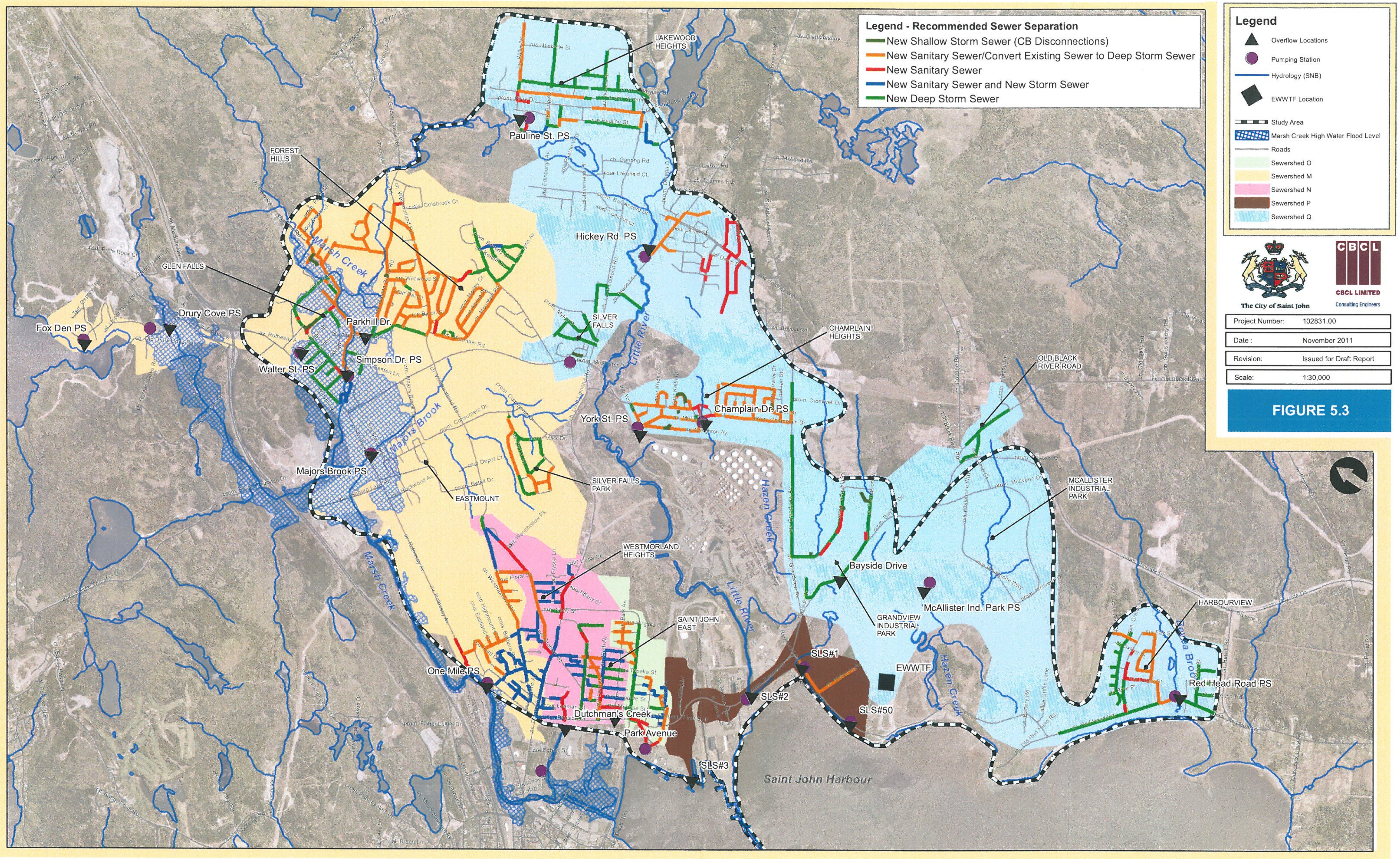
Evaluation Criteria Weighting Methodology	
Criteria	*Evaluation of Strategies*
Life Cycle Cost	1=highest cost, 5=lowest cost
Reduction in Overflows	1=lowest reduction, 5=highest reduction
Improved Receiving Water Quality	1=least improved, 5=most improved
Public Disruption	1=highest impact, 5=lowest impact
Land Requirements	1=land required, 5=no land required
Reliability	1=least reliable, 5=most reliable
Compatibility with Existing System	1=least compatible, 5=most compatible

Overflow Location	Estimated Number of Overflows Per Year	Extraneous Flow Impact	Evaluation Criteria	Weighting Factor 1 to 7 (1=minimal impact at this site; 7=large impact at this site)	Overflow Reduction Strategies Strategies are rated 1 to 5 for each evaluation criteria as described above					LONG-TERM Overflow Reduction Strategy	SHORT-TERM Overflow Reduction Strategy	Comments
					Sewer Separation	I & I Reduction	Capacity Increases	Storage	Satellite Treatment			
Red Head Road PS	0	High	Life Cycle Cost	7	1	2				Sewer Separation	I & I Reduction	There were no overflows generated at this location and no capacity issues in the upstream pipes. Evidence of cross connected sewers and conversations during field survey with residents about possible service lateral cross connections, sewer separation and I/I reduction should still be considered in this area.
			Reduction in Overflows	6	2	1						
			Improved Receiving Water Quality	5	2	1						
			Public Disruption	4	1	2						
			Land Requirements	3	2	1						
			Reliability	2	2	1						
			Compatibility with Existing System	1	2	1						
			Weighted Total		45	39	0	0	0			

1. *Evaluation and weighting are based on available information and may significantly change the matrix outcome as new information develops.*

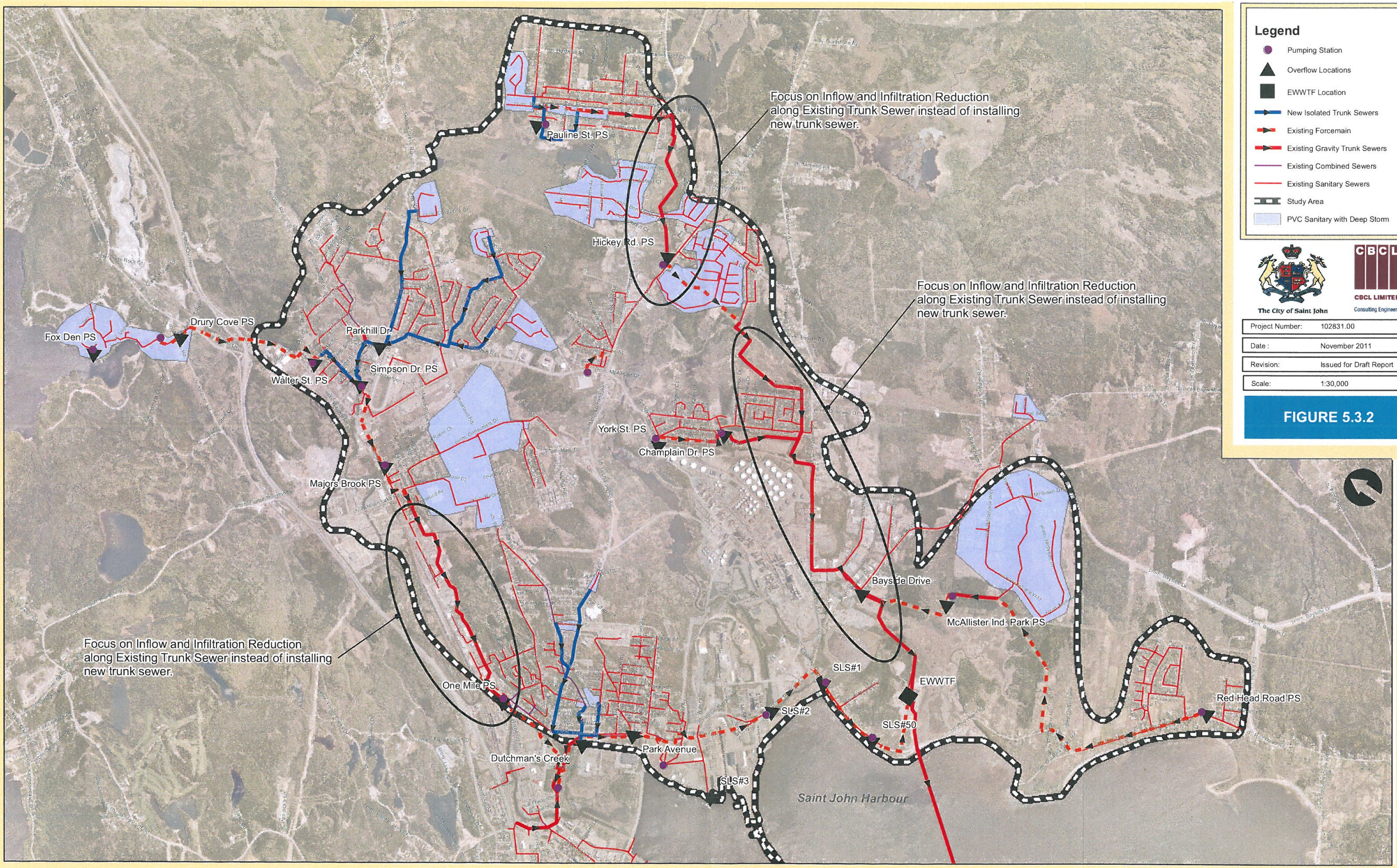
Combined Sewer Overflow Reduction Strategy - EWWTF Collection System

Recommended Sewer Separation - Long Term Strategy



Combined Sewer Overflow Reduction Strategy - EWWTF Collection System

New Trunk Sanitary Sewers from Isolated Separate Upstream Systems



Legend

- Pumping Station
- Overflow Locations
- EWWTF Location
- New Isolated Trunk Sewers
- Existing Forcemain
- Existing Gravity Trunk Sewers
- Existing Combined Sewers
- Existing Sanitary Sewers
- Study Area
- PVC Sanitary with Deep Storm

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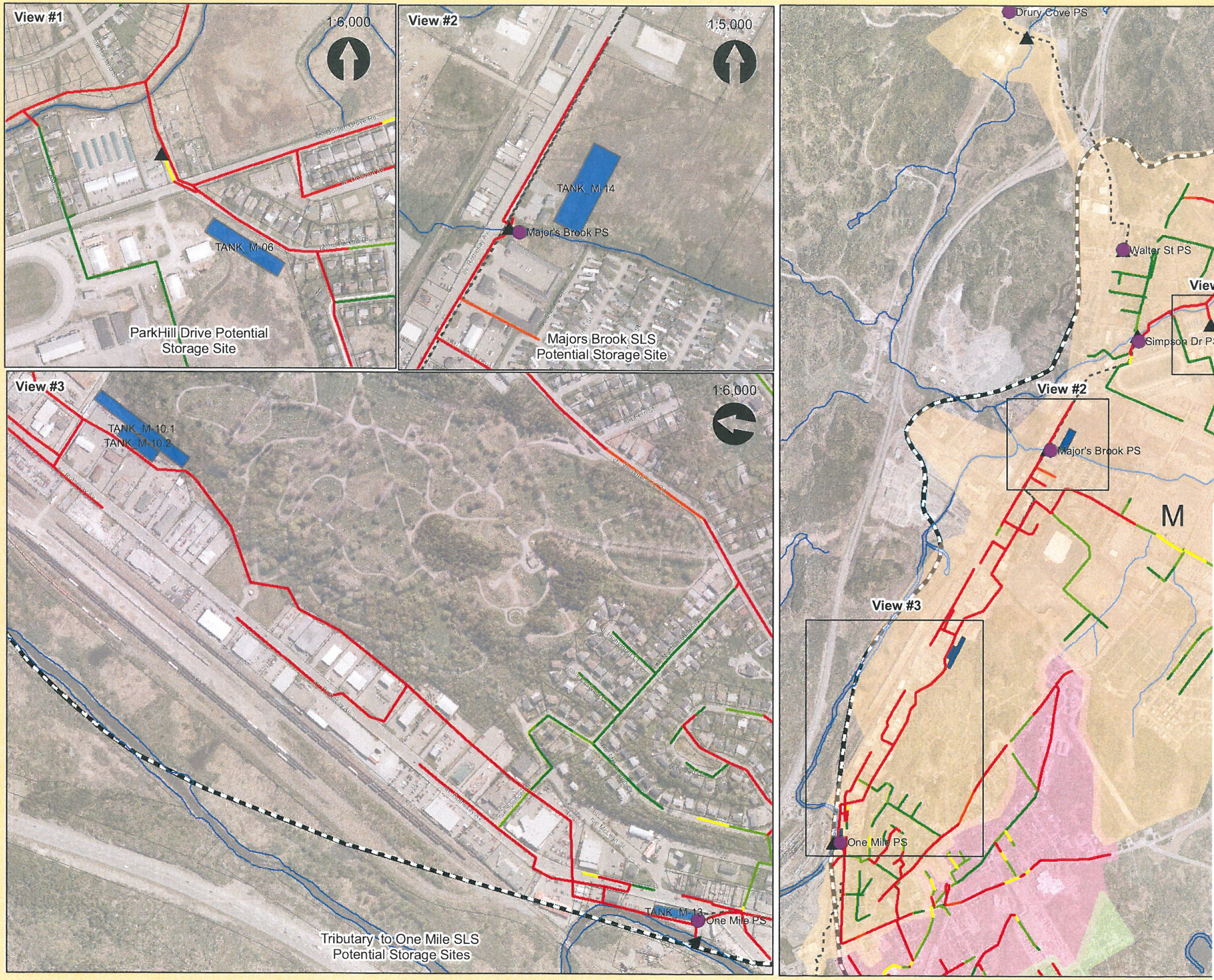
FIGURE 5.3.2

Combined Sewer Overflow Reduction Strategy - EWWTF Collection System

Potential Storage Sites - Sewershed M



Legend

- Pumping Station
- Potential_Storage_Location
- Overflow Locations

Max Depth / Diameter (%)

1 in 100yr

- 0-25%
- 25-50%
- 50-75%
- 75-100%
- >100%
- Forcemain
- Study Area
- Hydrology (SNB)

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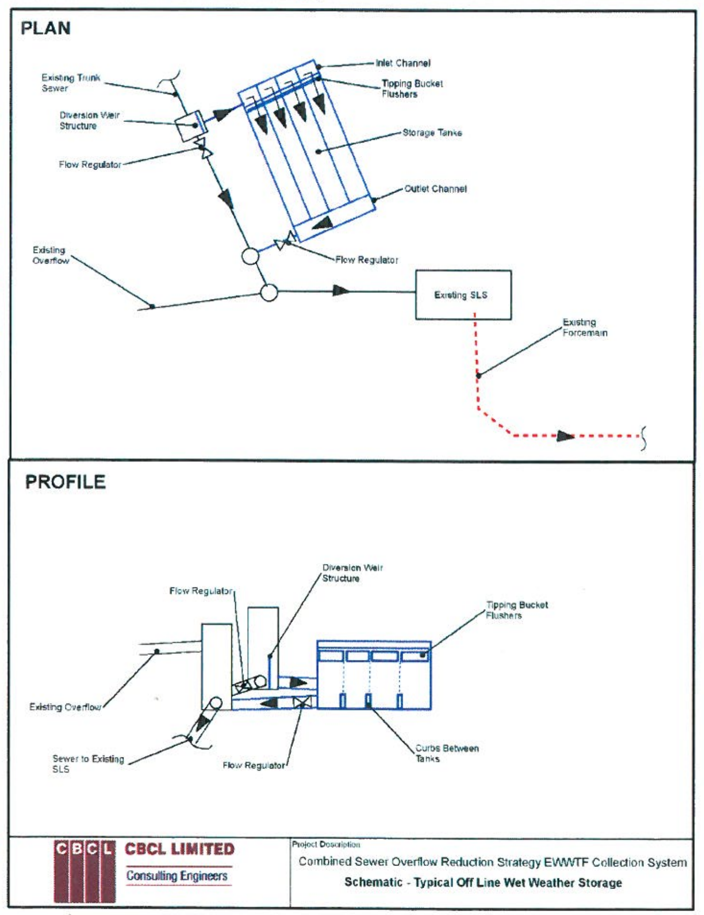
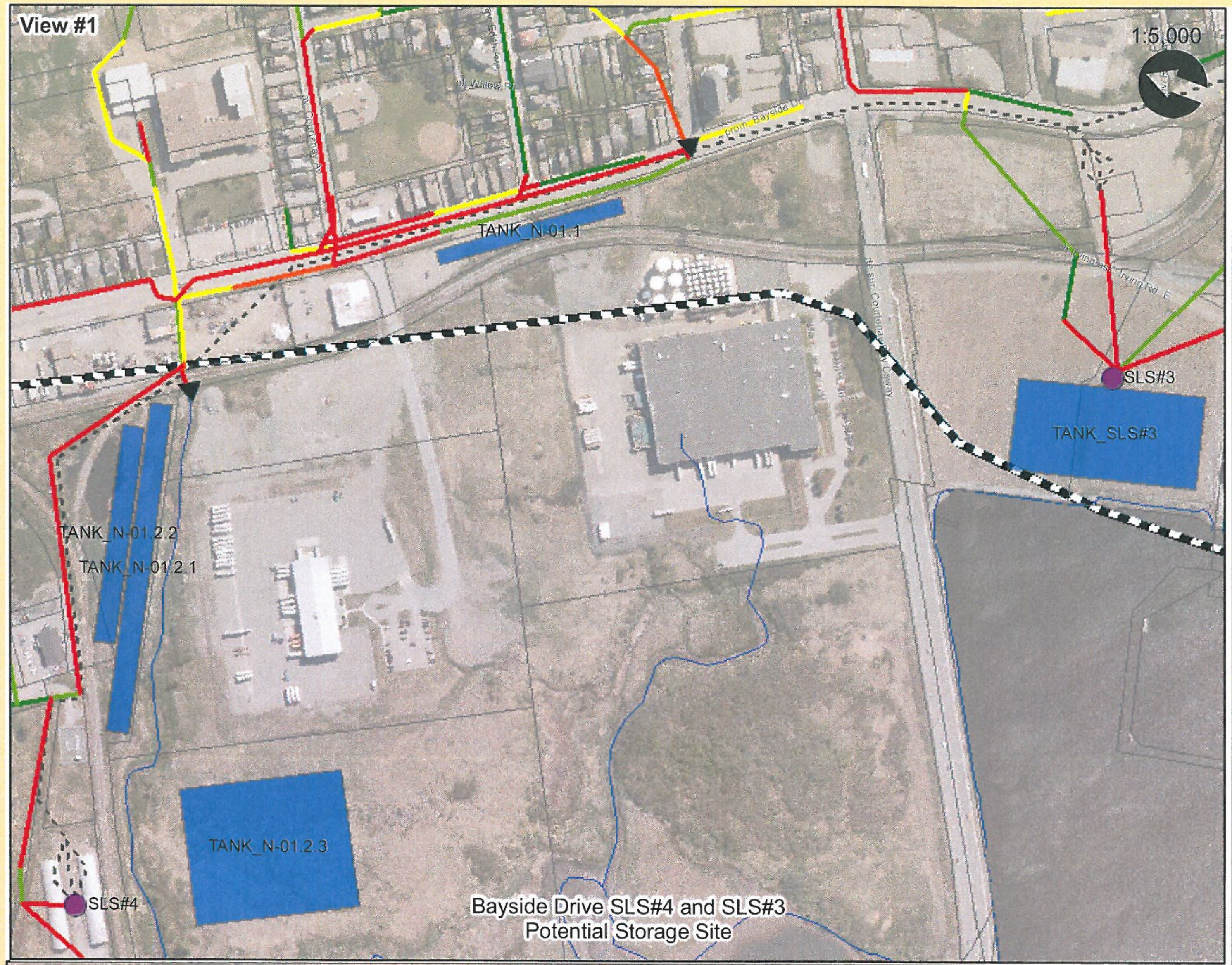
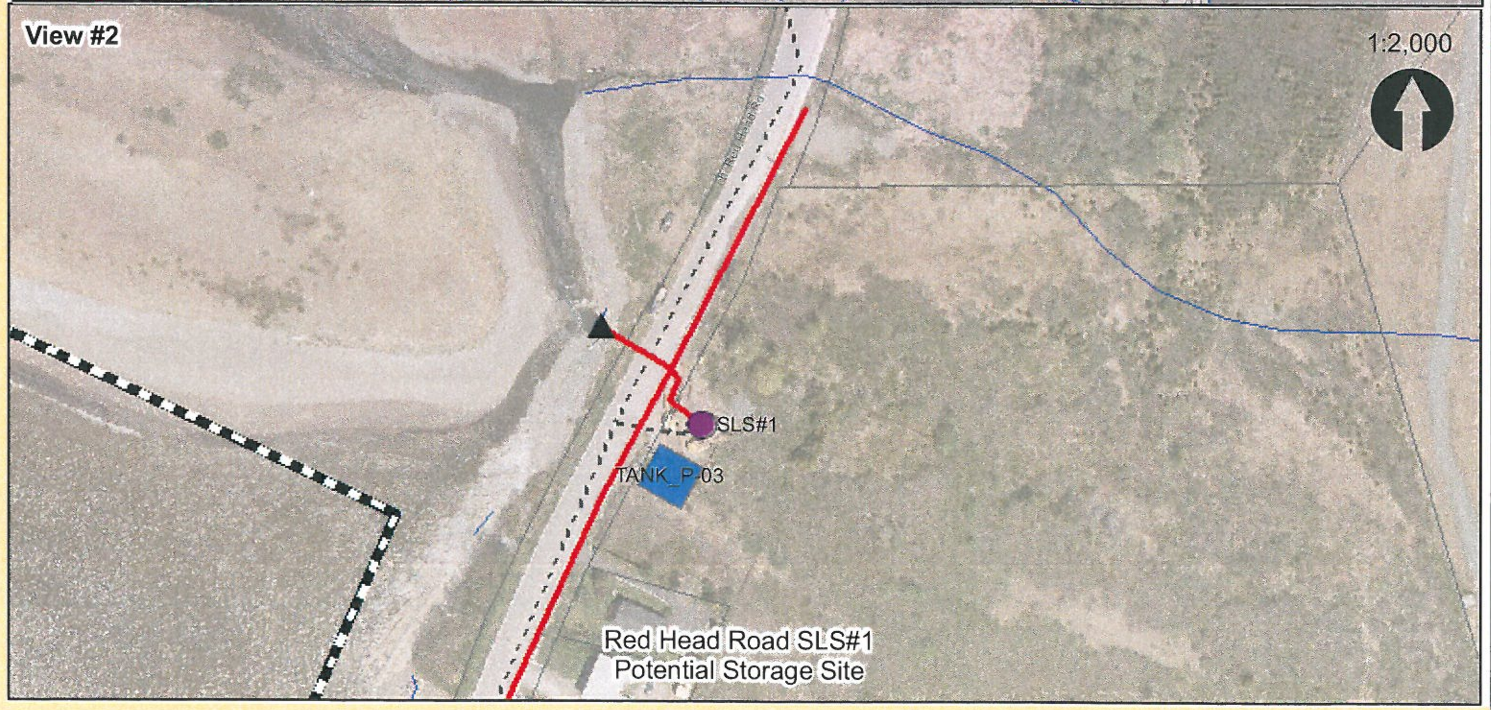
FIGURE 5.4.3(a)(i)

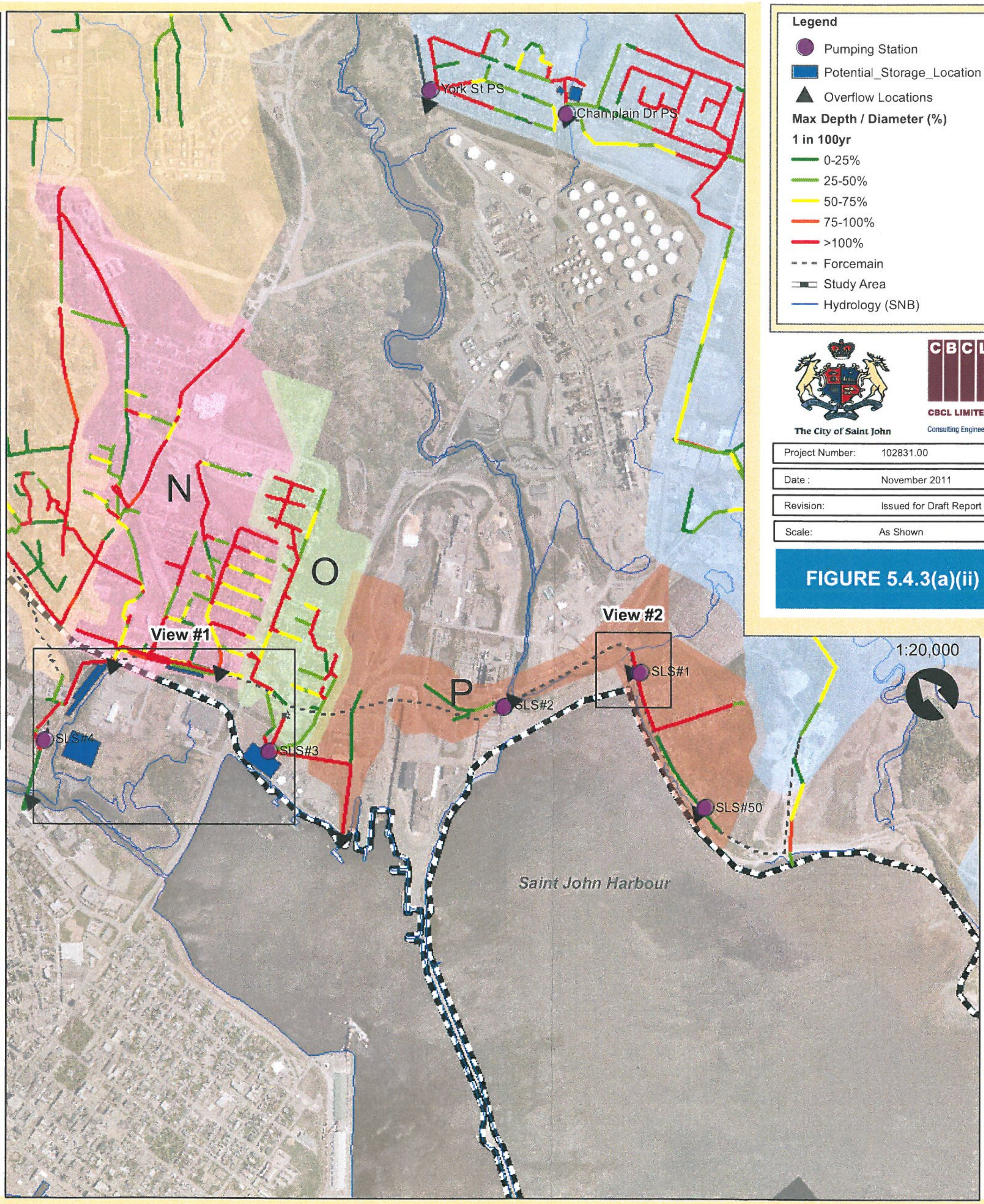


Combined Sewer Overflow Reduction Strategy - EWWTF Collection System

Potential Storage Sites - Sewersheds N, O, P







Legend

- Pumping Station
- Potential_Storage_Location
- Overflow Locations

Max Depth / Diameter (%)

1 in 100yr

- 0-25%
- 25-50%
- 50-75%
- 75-100%
- >100%
- Forcemain
- Study Area
- Hydrology (SNB)

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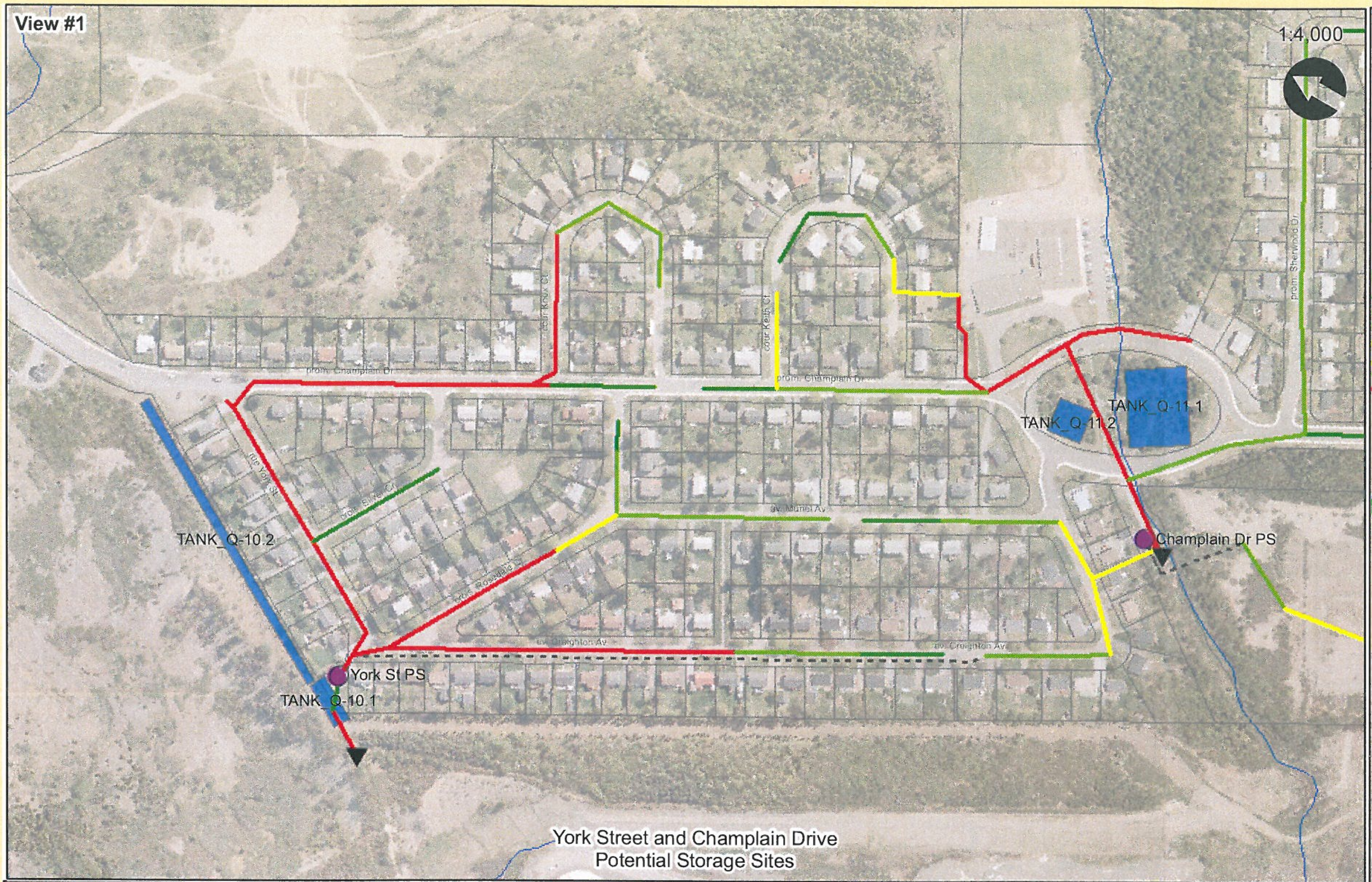
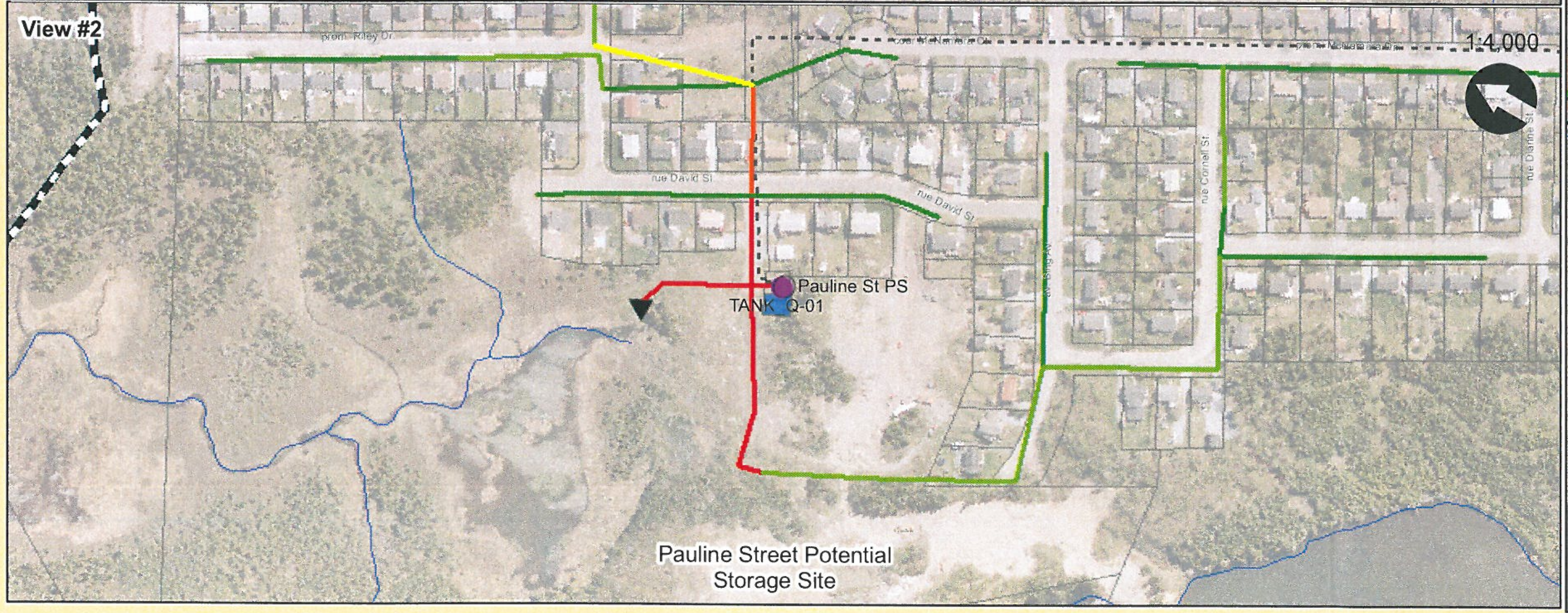
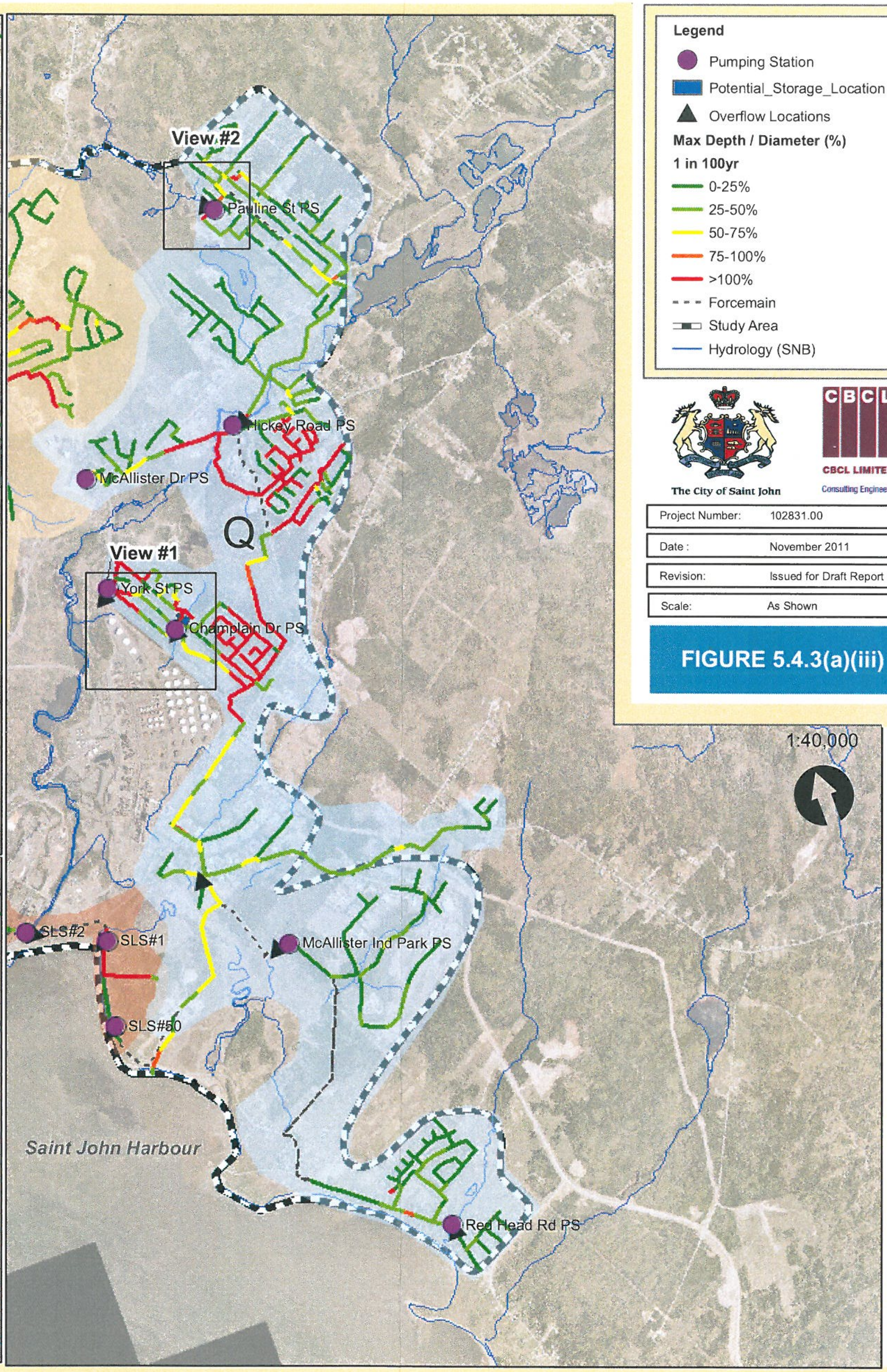
FIGURE 5.4.3(a)(ii)

Combined Sewer Overflow Reduction Strategy - EWWTF Collection System

Potential Storage Sites - Sewersheds Q







Legend

- Pumping Station
- Potential_Storage_Location
- Overflow Locations

Max Depth / Diameter (%)

1 in 100yr

- 0-25%
- 25-50%
- 50-75%
- 75-100%
- >100%
- Forcemain
- Study Area
- Hydrology (SNB)

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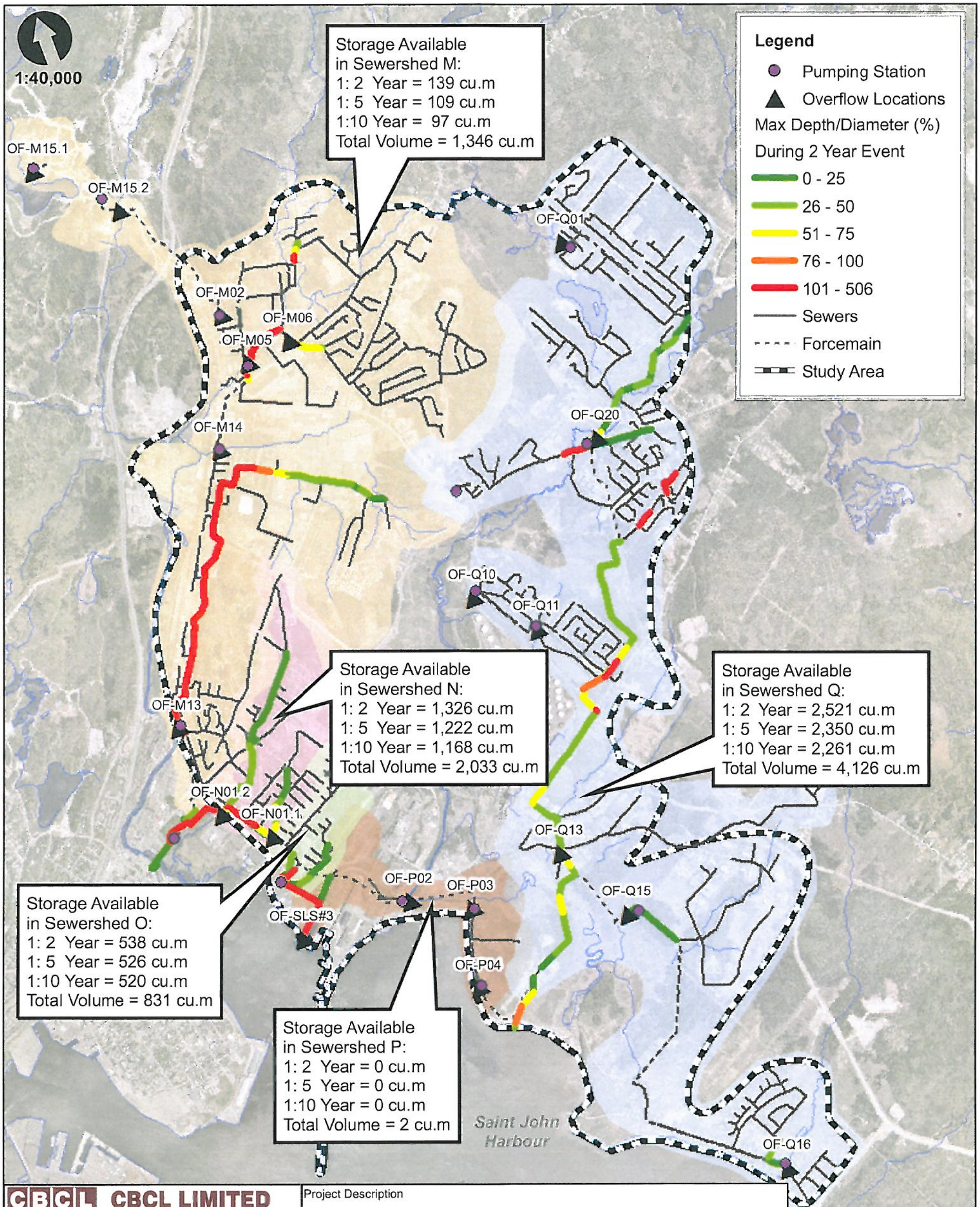
FIGURE 5.4.3(a)(iii)



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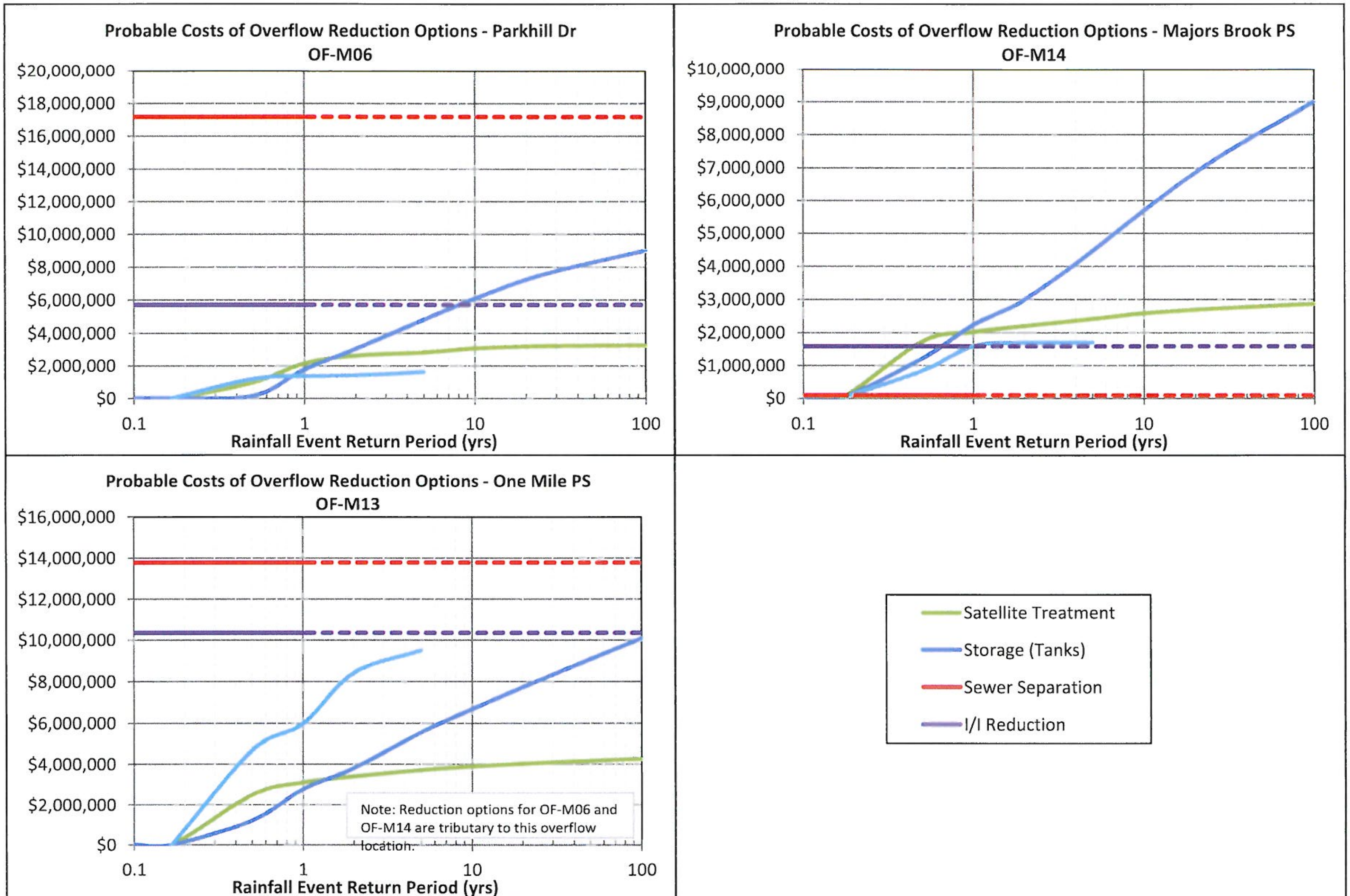
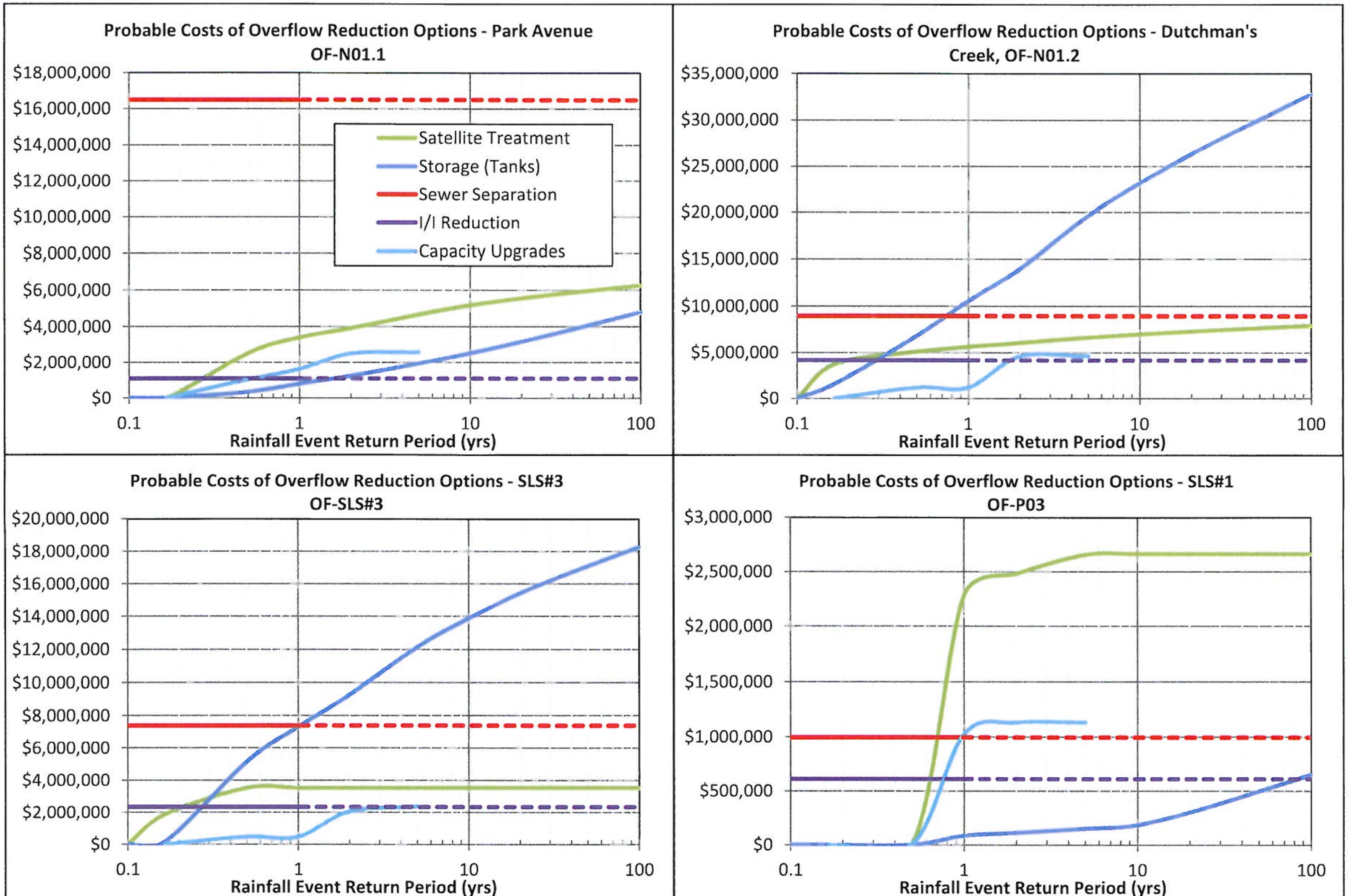
Project Description

Combined Sewer Overflow Reduction Strategy
EWWTF Collection System
Storage Available in Large Diameter Pipes (>=450 mm Dia.)

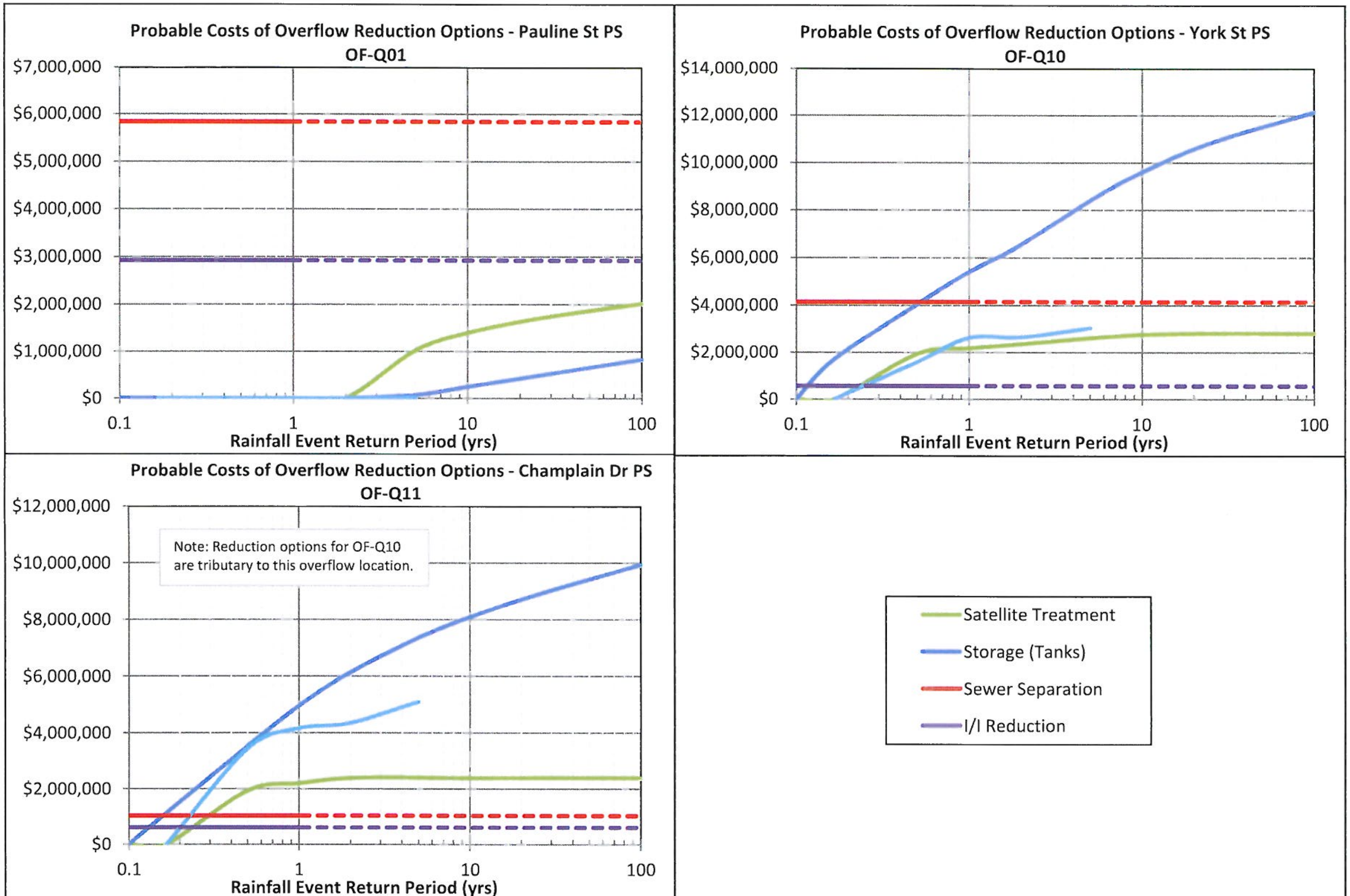
FIGURE 5.4.3(b)



CBCL LIMITED Consulting Engineers	Combined Sewer Overflow Reduction Strategy EWWTF Collection System	November 2011
	Probable Costs Relative to Design Rainfall Events – Sewershed M	**Figure 5.5 (a)**



CBCL LIMITED Consulting Engineers	Combined Sewer Overflow Reduction Strategy EWWTF Collection System	November 2011
	Probable Costs Relative to Design Rainfall Events – Sewersheds N, O & P	**Figure 5.5 (b)**



CBCL LIMITED Consulting Engineers	Combined Sewer Overflow Reduction Strategy EWWTF Collection System	November 2011
	Probable Costs Relative to Design Rainfall Events – Sewershed Q	**Figure 5.5 (c)**

CHAPTER 6 RECOMMENDED APPROACH TO REDUCE OVERFLOWS

6.1 Overall Approach

The City's policy of sewer separation should be implemented. Given the magnitude of the effort required to build new sanitary sewers in most of the study area that is currently serviced as well as construct new storm sewers in a large portion of the study area, it will not be feasible to implement the plan in the short term. It is a long term objective.

For planning purposes, sewers are typically considered to have a useful life of up to 100 years, provided there is ongoing maintenance. A large portion of the study area is serviced by sewers that are least 50 years old (see Figure 6.1). If the assumption of useful life is correct, a large portion of the sewers will need to be replaced or significantly rehabilitated. Every 100 years they must be replaced again and so on. If they are not, the extraneous flows will only grow, reducing capacity and increasing the associated problems discussed previously. To achieve a 100 year replacement cycle, at least 1 percent of the sewers on average must be replaced each year. This will ensure that no pipes are older than 100 years at any time.

A sewer separation program to reduce overflows should be implemented as part of this ongoing infrastructure renewal program. Table 5.3 (b) presented an overall cost to construct the new sewers necessary to achieve sewer separation. It is a large number. However, if only 1 percent of the costs must be expended each year, it is achievable. If a 100 year program is unacceptable and this initial project must be completed in 30 years, then 3.3 percent of the overall project must be implemented each year (on average) to meet this objective.

One hundred years is a long time to wait for improvements in water quality in Marsh Creek and Little River. Rather than spread the work in all sewersheds, new sewers should be constructed in the tributary sewersheds for these receiving waters first. Conversely, new sewers in the tributary sewersheds for the less impacted receiving waters should be constructed later.

In the short term, efforts should be made to optimize the performance of the existing wastewater and stormwater systems. For sanitary sewer systems this should involve:

- Installation of permanent monitoring systems (water level and flows) in all lift stations, particularly those on the trunk systems;
- Identification and removal of inflow from sanitary services;
- Repairs and upgrades of existing sanitary sewers to maintain or increase conveyance capacity and to reduce inflows;
- Generation of inline storage in sanitary sewers.

For combined sewer systems this should involve:
- Inlet control of inflow from surface stormwater;
- Treatment of overflows to minimize their impacts.

These plans are described in the following sections.

6.2 Long Term

The recommended strategy for managing overflows is to proceed with the stated City policy of sewer separation in all areas of Saint John. Separation of stormwater from sanitary flows by construction of new sanitary and storm sewers should eventually eliminate wet weather overflows.

6.2.1 Construction Process

Construction of the sanitary trunk sewers should begin at the downstream end of the sewershed and proceed in an upstream direction. Following construction of the new trunk sewers, local collection systems and laterals can be constructed. Both the new sanitary sewers and the existing sanitary sewers will stay in service until all of the new sanitary sewers and services are completed in each sub-sewershed.

Once the construction of new sanitary sewers and laterals are completed, the existing sanitary sewers will be converted to deep storm sewers by diverting them to the closest surface water drainage system. In areas where the deep storm sewer is below the delineated flood elevation such as in the Glen Falls area, the connection to the surface drainage system will include a lift station. A schematic of this connection is presented in Figure 6.2.1. Under normal conditions, all of the deep storm sewers will drain by gravity to the surface drainage systems. Under flood conditions, when the lowest of the deep storm sewers cannot drain by gravity, the pumps will be engaged to lift drainage from the deep storm sewers to the surface drainage system. To function as planned, all direct inflows will need to be identified by detailed investigations and then removed.

This system will provide all properties with a discharge point for foundation drainage. If property owners connect their foundation drains <u>and</u> follow the recommendations in the City's Information Bulletin "Measures to Reduce the Risk of Basement Flooding from Rainfall Runoff" they should significantly reduce the risk of basement flooding. It should provide a level of service similar to level provided to the properties serviced by the project recently completed in Millidgeville on Brentwood Crescent and Woodward Avenue that experienced basement flooding during Tropical Storms Hanna in 2008 and Dan in 2009.

6.2.2 Implementation

Implementation of the new sewer construction program should be concentrated in one sewershed at a time to maximize benefits. Recommended priorities were listed in section 4.4.2, beginning with sewershed M, with overflows to Marsh Creek. Within sewershed M the recommended implementation plan follows the implementation priorities presented in Figure 6.2.2. These are aimed at reducing basement flooding risks. The implementation process is described as follows:

- Construction of new sanitary sewer in the floodplain areas of the Simpson Drive sewershed, conversion of the existing sanitary sewer to a deep storm sewer;
- Construction of a stormwater lift station or conversion of the existing station and construction of a new sanitary lift station at Simpson Drive;
- Construction of diversion and overflow facilities to isolate excess flows from the trunk sewer system at the following sewage lift stations:
    - Simpson Drive;
    - One Mile;
- Construction of new sanitary sewer in the remaining areas of the Simpson Drive sewershed, conversion of the existing sanitary sewer to a deep storm sewer. Every effort should be made to construct new trunk sewer from areas currently serviced by separate sanitary and storm sewers to the trunk sewer system first, then construct the other local sanitary sewers;
- Construction of new sanitary sewers in the Majors Brook sub-sewershed, conversion of the existing sanitary sewer to a deep storm sewer;
- Construction of new sanitary sewers in the One Mile sub-sewershed, conversion of the existing sanitary sewer to a deep storm sewer.

Construction in sewersheds Q, O and P and then N should follow a similar pattern.

6.2.3 Construction Duration

The actual duration of the sewer construction program will depend on factors including:

- Regulatory mandates;
- Receiving water remediation and basement flooding risk reduction priorities;
- Availability of funding for the projects;
- Availability of City Staff; and
- Development pressures.

At a construction rate of 1 percent of the required piping system completed per year, it would take approximately 30 years to complete all of the recommended new sewers in Sewershed M.

6.2.4 Costs of Construction

Estimates of probable cost of the long term plan are provided in Table 6.2.4. These estimates of probable capital cost were developed based on historical costs of similar pipe replacement projects. They include allowances for rock excavation, pipe, manholes, reinstatement of asphalt, sidewalks etc. as provided in the 2011 version of estimation spreadsheet for Division 4.5 of the Standard Specification for typical conditions. They include a combined allowance for contingency and engineering of 25 percent of the estimated cost of construction.

6.3 Short Term

In the short term, overflows and surcharging can be reduced by making efforts to optimize the performance of the existing wastewater and stormwater systems. For sanitary sewer systems this should involve:

6.3.1 Flow Monitoring

Monitoring of the flows and water levels in the sanitary sewer systems will facilitate optimum operation of the system. Permanent flow-monitoring stations are recommended that would allow the continuous assessment of long-term trends in wastewater generation for various sub-sewersheds in the overall system.

Flow monitoring and assessment of the results plays an equally important role in monitoring the results of programs to reduce inflows.

6.3.2 Sewer Inspections and I/I Reduction Programs

Smoke testing is an effective and relatively inexpensive way to identify potential inflow sources in sanitary sewers. This should be completed in all areas on a regular basis, particularly in areas where there are no storm sewers or where the storm sewers are shallow.

Video inspection of the sewers is recommended at a frequency of once in 5 to 10 years to monitor changes in the physical condition of the system.

Extraneous flow reduction measures discussed in section 5.4.1 should be implemented to the extent required to limit inflows and thus overflows from these systems to an acceptable level. The level of implementation will depend on:

- Sources and magnitudes of extraneous flows identified in the detailed investigations; and
- Effectiveness of the recommended measures at reducing inflows.

6.3.3 Sewer Maintenance

On-going upkeep of the system is required to:

- Ensure that the design capacities of existing facilities are available;
- Inflows are as low as economically possible.

This requires regular maintenance of the collection system including:

- Removal of sediment deposits with regular sewer flushing is recommended
- Regular inspections of manholes and pipes
- Testing and replacement of worn pumping equipment.

6.3.4 Utilization of Inline Storage

Preliminary assessments were completed as part of this study and they indicated that there is potential to utilize inline storage in the existing collection and trunk sewer systems, particularly in Sewershed Q (see Table 5.4.3.2). This should be further investigated at a detailed level.

6.3.5 *Inlet Control on Catch Basins*

Inlet control for catch basins connected to combined sewers or inadvertently connected to sanitary sewers should be considered in greater detail to reduce overflows at the interceptor sites in the short term. As discussed in section 5.4.1, care must be taken not to generate additional flooding problems. A detailed study is required on a sub-sewershed basis where this is considered. The most suitable locations are the areas with steep slopes in sewershed N. These slopes provide the best opportunity of conveying excess flows overland and in the street gutters.

6.3.6 *Provide Overflow Treatment*

CCME guidelines require the screening of overflows where feasible. Screening of overflows is a preliminary level of satellite treatment. If more stringent discharge requirements, such as disinfection, are placed on discharges to Courtenay Bay and the Harbour, then more advanced levels of treatment will be required. These were discussed in section 5.4.4.

A typical overflow objective for many jurisdictions is one or less than 1 per year that doesn't meet receiving water quality objectives. It is recommended that this be used as a basis for discussion with the regulators.

6.3.7 *Additional Measures*

In addition to some of the short term overflow reduction / mitigation measures, over the next ten (10) years it is recommended that land acquisition and easements are obtained for the following sewer separation projects:

- New lift station in the Simpson Drive sewershed, preferably adjacent the existing station;
- Overflow treatment at the following sites:
  - SLS#4;
  - SLS#3.

6.3.8 *Costs of the Short Term Plan*

Costs of recommended measures are summarized in Table 6.3.8. These estimates of probable capital cost were developed based on historical costs of similar projects and include a combined allowance for contingency and engineering of 25 percent of the estimated cost of construction. Where significant operating and maintenance costs are expected, these are listed as well as the Life Cycle costs.

In some sewersheds, the overall costs of the alternatives are comparable to the costs of the long term plans. In these areas, the recommended plan is to proceed with the long term plan even when overflows must be reduced or impacts mitigated in the short term.

6.4 Future Development

Much of the available capacity in the trunk sewers and interceptor sewers is currently being used by stormwater. Removal of the stormwater will immediately provide capacity for additional development in all of the existing systems.

The long term sewer replacement program has accounted for some additional land development in the sewersheds. The areas currently undeveloped but considered for future development are identified in Figure 6.4.

Table 6.2.4 - Long Term Plan and Probable Cost

Overflow Location	Overflow Label	LONG-TERM Overflow Reduction Strategy	Probable Cost of Long Term Plan		
			Capital Cost	O & M Cost	Life Cycle Cost
Sewershed M			**$ 43,864,000**	**$ 48,308**	**$ 44,805,240**
Parkhill Dr	OF-M06	Sewer Separation	$ 17,190,000	$ 19,287	$ 17,565,783
Simpson Dr PS	OF-M05	Sewer Separation	$ 12,592,000	$ 13,239	$ 12,849,943
Major's Brook PS	OF-M14	Sewer Separation	$ 91,000	$ 101	$ 92,962
One Mile PS	OF-M13	Sewer Separation	$ 13,991,000	$ 15,682	$ 14,296,553
Sewershed N			**$ 25,448,000**	**$ 27,060**	**$ 25,975,228**
Park Avenue	OF-N01.1	Storage	$ 16,499,000	$ 17,047	$ 16,831,139
Dutchman's Creek	OF-N01.2	Sewer Separation	$ 8,949,000	$ 10,013	$ 9,144,090
Sewershed O			**$ 7,376,000**	**$ 7,946**	**$ 7,530,823**
SLS#3	OF-SLS#3	Sewer Separation	$ 7,376,000	$ 7,946	$ 7,530,823
Sewershed P			**$ 994,000**	**$ 1,187**	**$ 1,017,131**
SLS#1	OF-P03	Sewer Separation	$ 994,000	$ 1,187	$ 1,017,131
Sewershed Q			**$ 51,270,500**	**$ 71,101**	**$ 55,306,804**
Pauline St PS	OF-Q01	Storage	$ 5,834,000	$ 6,412	$ 5,958,935
Hickey Rd PS	OF-Q20	Sewer Separation	$ 10,526,000	$ 13,963	$ 10,798,062
York St PS	OF-Q10	Sewer Separation	$ 4,138,000	$ 4,895	$ 4,233,369
Champlain Dr PS	OF-Q11	Sewer Separation	$ 1,029,000	$ 1,176	$ 1,051,913
Bayside Drive	OF-Q13	Sewer Separation	$ 14,885,000	$ 15,882	$ 15,194,453
McAllister Ind Park PS	OF-Q15	I & I Reduction	$ 6,497,500	$ 19,654	$ 9,531,411
Red Head Rd PS	OF-Q16	Sewer Separation	$ 8,361,000	$ 9,118	$ 8,538,661
Total Probable Cost			**$ 128,952,500**	**$ 155,602**	**$ 134,635,227**

1. *Costs are in 2011 dollars and include 25% for engineering and contingency, they do not include HST.*

Table 6.3.8 - Short Term Plan and Probable Cost

Overflow Location	Overflow Label	SHORT-TERM Overflow Reduction Strategy	Probable Cost of Short Term Plan		
			Capital Cost	O & M Cost	Life Cycle Cost
Sewershed M			**$ 27,758,500**	**$ 107,284**	**$ 41,137,159**
Parkhill Dr	OF-M06	I & I Reduction	$ 5,739,500	$ 31,367	$ 8,692,375
Simpson Dr PS	OF-M05	I & I Reduction	$ 11,548,500	$ 41,474	$ 17,068,375
Major's Brook PS	OF-M14	See Long-term Strategy (Sewer Separation)	$ 91,000	$ 101	$ 92,962
One Mile PS	OF-M13	I & I Reduction	$ 10,379,500	$ 34,342	$ 15,283,448
Sewershed N			**$ 13,746,000**	**$ 14,319**	**$ 14,220,713**
Park Avenue	OF-N01.1	Storage	$ 4,797,000	$ 4,306	$ 5,076,624
Dutchman's Creek	OF-N01.2	See Long-term Strategy (Sewer Separation)	$ 8,949,000	$ 10,013	$ 9,144,090
Sewershed O			**$ 5,899,000**	**$ 56,489**	**$ 8,556,851**
SLS#3	OF-SLS#3	Capacity Increases and Treatment	$ 5,899,000	$ 56,489	$ 8,556,851
Sewershed P			**$ 650,000**	**$ 477**	**$ 685,823**
SLS#1	OF-P03	Storage	$ 650,000	$ 477	$ 685,823
Sewershed Q			**$ 30,417,500**	**$ 117,133**	**$ 40,512,136**
Pauline St PS	OF-Q01	Storage	$ 826,000	$ 272	$ 865,009
Hickey Rd PS	OF-Q20	See Long-term Strategy (Sewer Separation)	$ 10,526,000	$ 13,963	$ 10,798,062
York St PS	OF-Q10	I & I Reduction	$ 585,500	$ 9,623	$ 1,011,882
Champlain Dr PS	OF-Q11	I & I Reduction	$ 640,500	$ 9,228	$ 1,081,622
Bayside Drive	OF-Q13	I & I Reduction	$ 7,807,500	$ 39,569	$ 11,763,918
McAllister Ind Park PS	OF-Q15	See Long-term Strategy (I & I Reduction)	$ 6,497,500	$ 19,654	$ 9,531,411
Red Head Rd PS	OF-Q16	I & I Reduction	$ 3,534,500	$ 24,823	$ 5,460,231
Total Probable Cost			**$ 78,471,000**	**$ 295,702**	**$ 105,112,682**

1. Costs are in 2011 dollars and include 25% for engineering and contingency, they do not include HST.

2. Costs do not include cost for generation of inline storage and inlet conrtol on catch basins. These would be determined after detailed assessments of the opportunities to apply these techniques.

Combined Sewer Overflow Reduction Strategy - EWWTF Collection System

Age of Sanitary and Combined Infrastructure

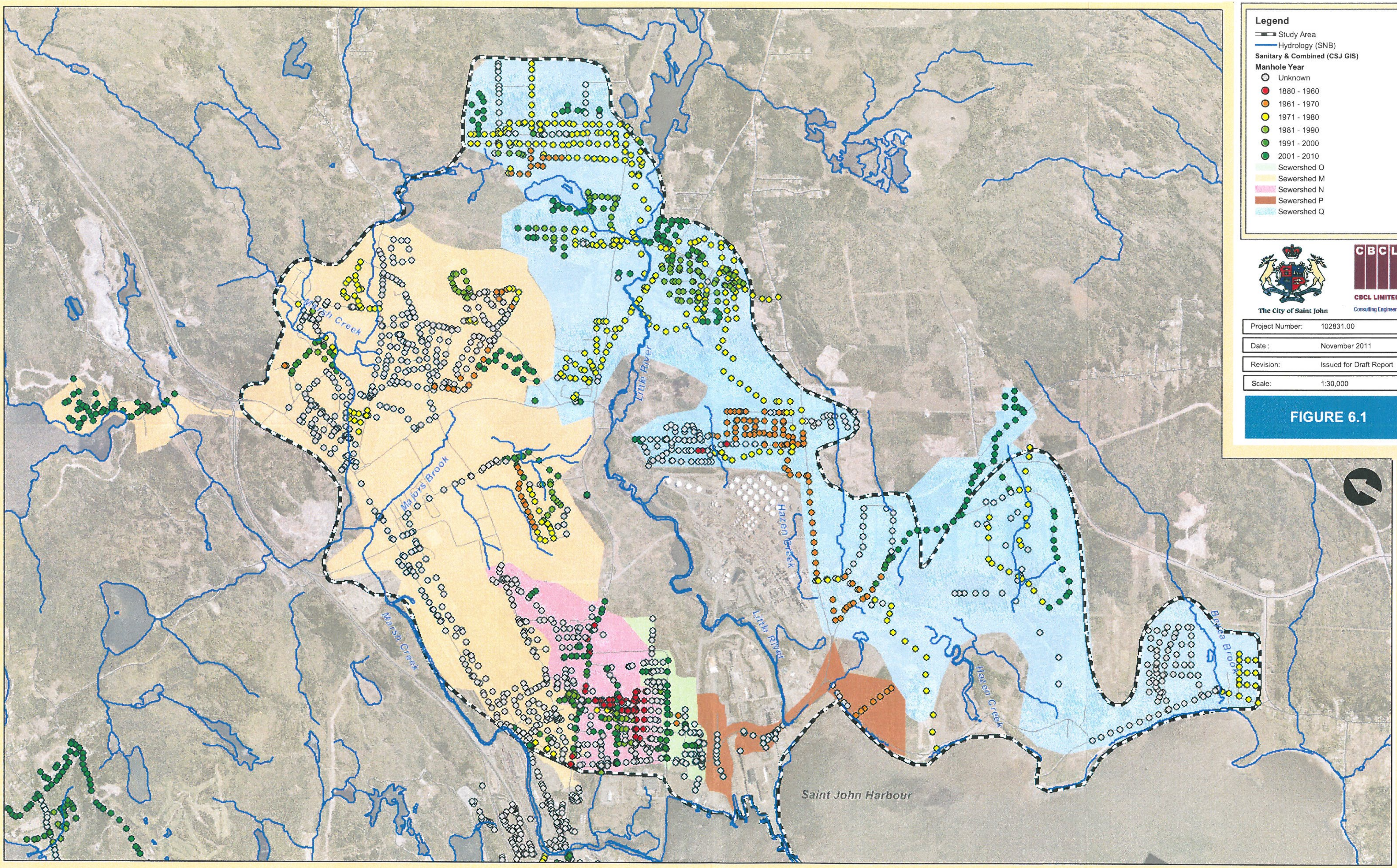


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FIGURE 6.1

Combined Sewer Overflow Reduction Strategy - EWWTF Collection System

Deep Storm Sewers Influenced During Flood Conditons

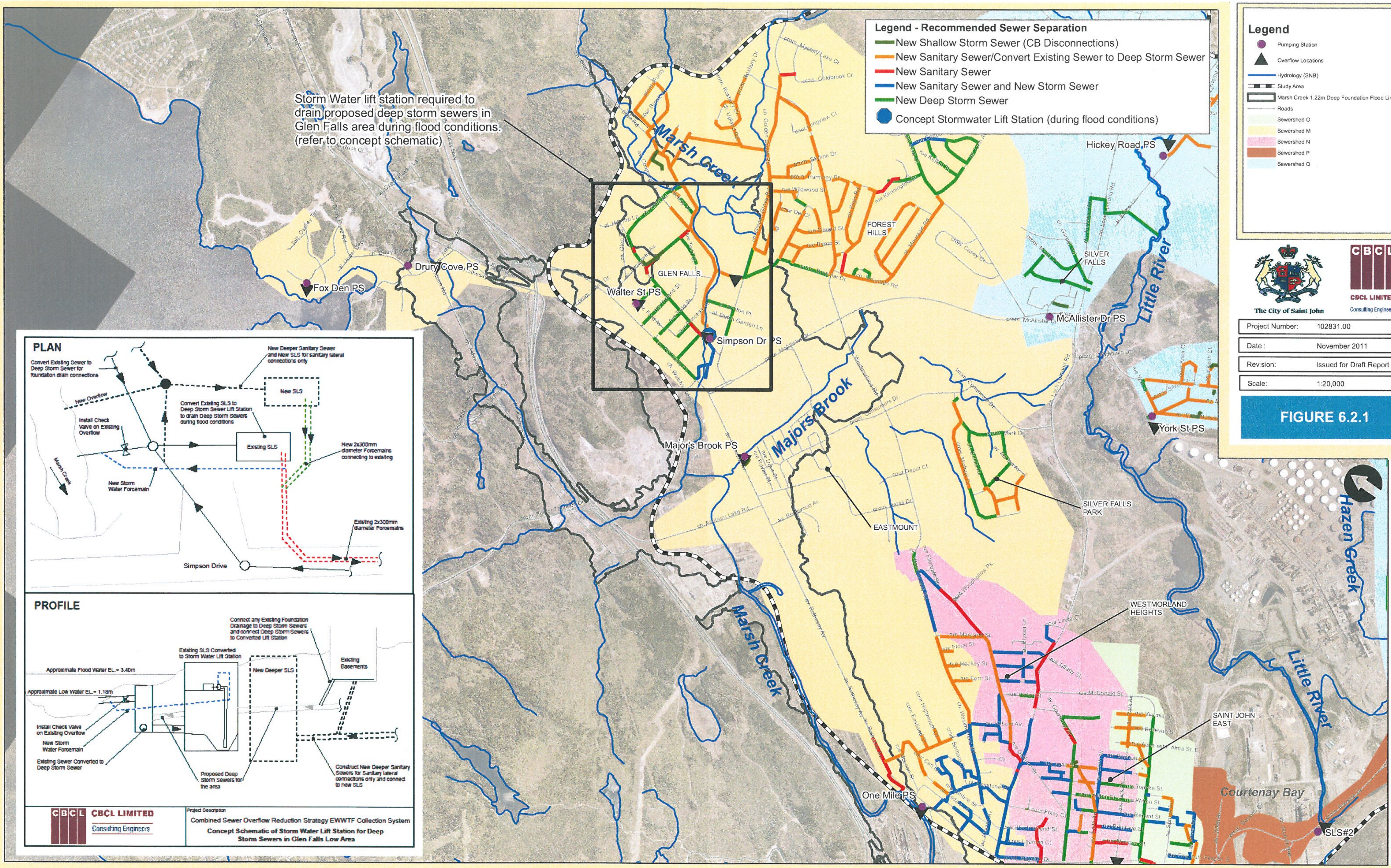

FIGURE 6.2.1

Combined Sewer Overflow Reduction Strategy - EWWTF Collection System

Sewer Separation Priority Areas

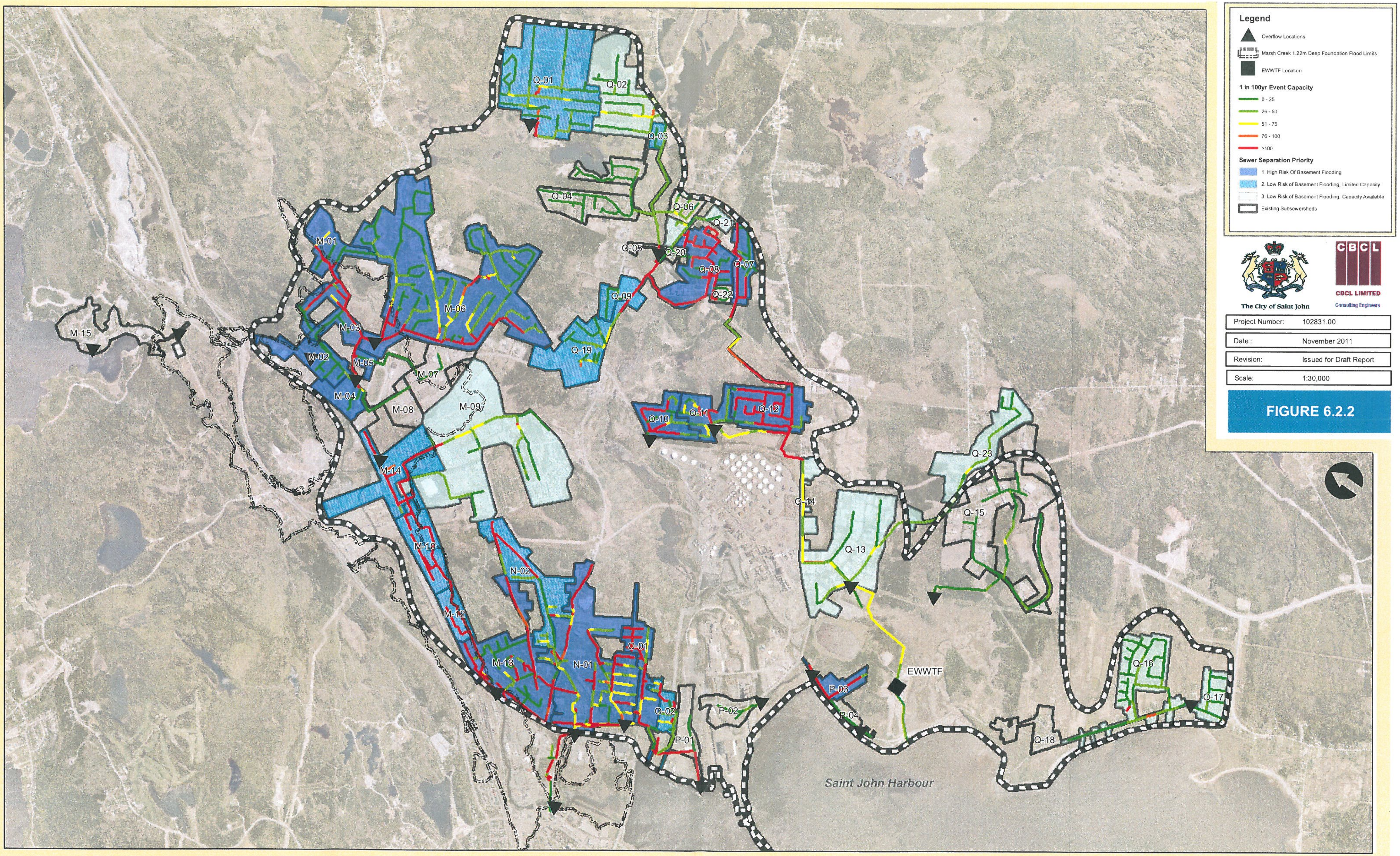

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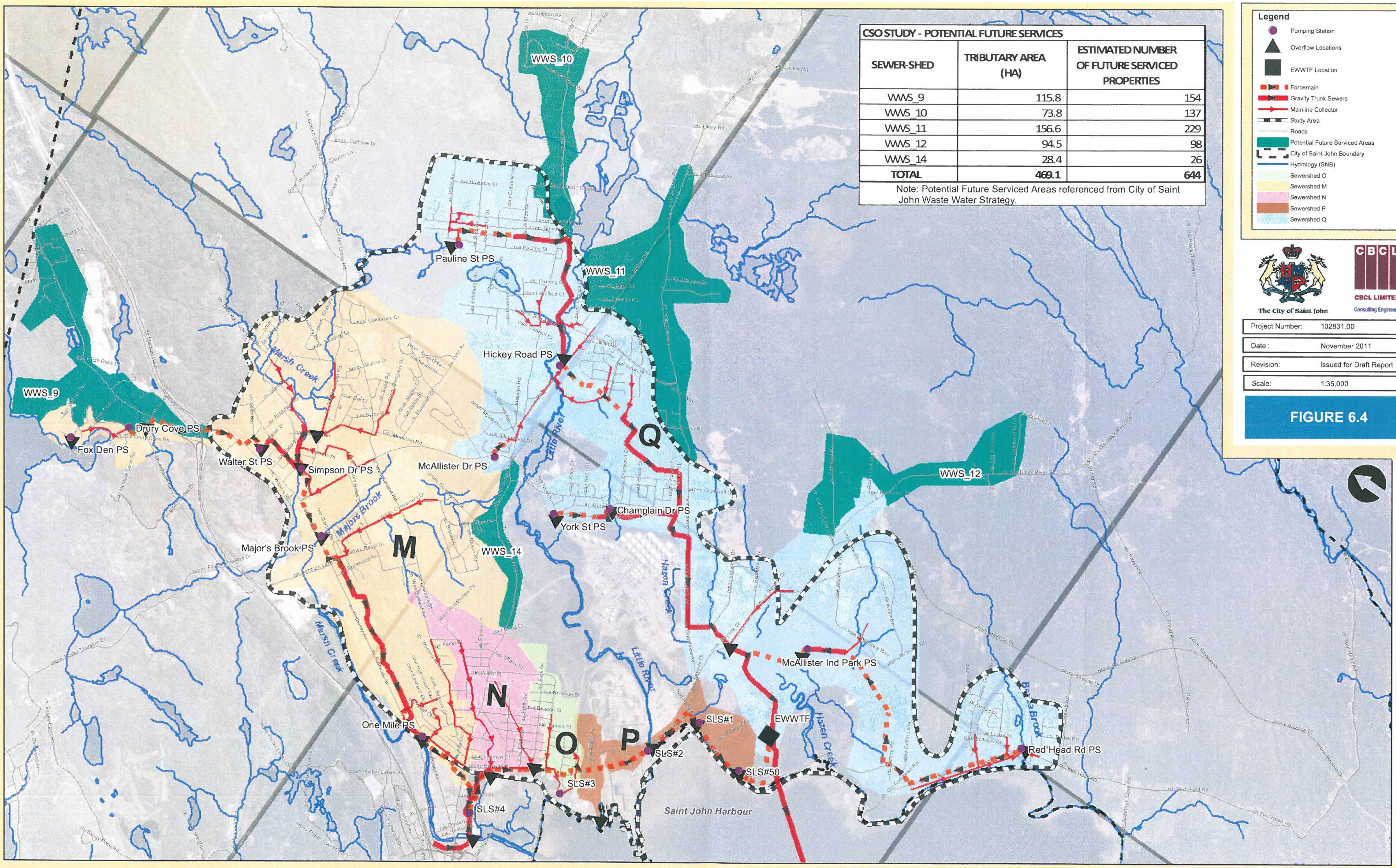
FIGURE 6.2.2

Combined Sewer Overflow Reduction Strategy - EWWTF Collection System

Potential Land Development Tributary to Study Area



CSO STUDY - POTENTIAL FUTURE SERVICES

SEWER-SHED	TRIBUTARY AREA (HA)	ESTIMATED NUMBER OF FUTURE SERVICED PROPERTIES
WWS_9	115.8	154
WWS_10	73.8	137
WWS_11	156.6	229
WWS_12	94.5	98
WWS_14	28.4	26
TOTAL	**469.1**	**644**

Note: Potential Future Serviced Areas referenced from City of Saint John Waste Water Strategy.

Legend

- Pumping Station
- Overflow Locations
- EWWTF Location
- Forcemain
- Gravity Trunk Sewers
- Mainline Collector
- Study Area
- Roads
- Potential Future Serviced Areas
- City of Saint John Boundary
- Hydrology (SNB)
- Sewershed O
- Sewershed M
- Sewershed N
- Sewershed P
- Sewershed Q

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FIGURE 6.4

CHAPTER 7 PRELIMINARY DESIGNS

Preliminary designs were developed for the sewers required for the long term overflow reduction plan involving separation of stormwater from sanitary flows.

7.1 Design Standards

A summary of the design standards used for preliminary design of the main components recommended in this study are summarized below in the following sections.

7.1.1 Atlantic Canada Wastewater Standards &Guidelines

The Environment Canada, Atlantic Canada Standards and Guidelines Manual for the Collection, Treatment, and Disposal of Sanitary Sewage was used to generate estimated average domestic design flows tributary to each of the sub-sewersheds in the study area. These flow estimates were generated using estimated equivalent domestic populations assuming the following Design Manual's suggested sewage generation rates:

- Residential – 2.5 persons/residence using 340 L/person/day;
- Commercial – 6 L/m^2/day (area of building); and
- Industrial – water usage data (where available) or 6 L/m^2/day (area of building).

As suggested by the Manual, extraneous flows range from 0.14 to 0.28 L/sec/ha which include peak infiltration and peak inflows. The peak flow per gross hectare varies depending upon the density of the area as well as the development level. For all the sub-sewershed areas 0.14 L/sec/ha was used to account for extraneous flows. These estimated flows were used to check capacities of the existing system as well as for design of any new pipes required for CSO reduction strategies. Design criteria for new sanitary pipes included the following:

- Minimum pipe diameter of 200 mm or matching size of larger upstream pipes;
- Available slope on each street or surface where pipe is to be located;
- A manning's coefficient of 0.01.

7.1.2 City of Saint John Storm Drainage Design Manual

Peak runoff flows tributary to the existing drainage system in the study area were estimated using a hydrologic – hydraulic model of the tributary areas and the existing conditions storm sewer networks. Peak flows from 1 in 5 year and 1 in 100 year design storms were estimated for each of the sub-watershed areas using the existing conditions hydrologic and hydraulic model. As described in the City's current stormwater design guidelines, all new piped storm sewers (minor drainage systems) should have capacity to convey the peak flows generated for the 1 in 5 year design rainfall event. The major drainage system must convey the difference between the capacity of the minor drainage system and the peak flows from a 1 in 100 year design rainfall event. These guidelines were used as a base line for checking capacities of the existing system as well as the design of any new storm sewers required for CSO reduction strategies.

Design criteria for new pipes in the minor drainage systems were estimated using the following criteria:

- Minimum pipe size of 300mm diameter or matching the size of larger upstream pipes;
- Available slopes on each street or surface where the pipe is to be located;
- A manning's coefficient of 0.013 to accommodate the use of existing storm sewers.

7.1.3 Canadian Council of Ministers of the Environment (CCME) Guidelines

As stated in the Background section of Chapter 1 these guidelines include the following national standards:

7.1.3.1 Combined Sewer Overflows

1. No increase in combined sewer overflow frequency due to development or redevelopment, unless it occurs as part of a combined sewer management plan;
2. No combined sewer overflow discharge during dry weather, except during spring thaw or emergencies; and
3. Removal of floatable materials where feasible.

7.1.3.2 Sanitary Sewer Overflows

1. No increase in sanitary sewer overflow frequency due to development or redevelopment, unless it occurs as part of a combined sewer management plan;
2. No sanitary overflow discharge during dry weather, except during spring thaw or emergencies.

These national standards were used as a baseline during the combined sewer overflow analysis of the existing sewage system to determine if the current system meets these requirements. These were also used to determine what should be completed to meet these regulations.

7.2 Presentation of Designs

Preliminary designs for every pipe required in the sewer separation program for sanitary and storm sewers are presented in the project GIS for Sewer Separation. These designs may be included as part of overall infrastructure renewal plans on a street by street basis.

A description of the project GIS is provided in Appendix A with the description of the SewerGems data files.

APPENDIX A

SewerGEMS Data & Project Databases

Appendix A - SewerGEMS Data & Project Databases

1.1 Collection System Characteristics

The SewerGEMS model of the collection system was constructed using information from a variety of different sources, including survey data, record drawings, City GIS information, orthophotos, and LIDAR digital elevation maps. This information was used to model each manhole and pipe section as accurately as possible.

1.1.1 Topographic Survey

Topographic survey is typically the most accurate way to gather information on the collection system as it currently exists. The City GIS was used to map out serviced areas where topographic survey was to be carried out for sewer manholes and catch basins. Topographic survey was completed for various service areas between September 2010 and August 2011. A total of 3044 structures (approximately 53 % of the study area) were surveyed. The following information was collected for each structure that was surveyed:

- Location – Northing and Easting;
- Cover elevation;
- ID label – typically the structure label from the City GIS database;
- Structure type – CB, SMH, STMH;
- Date of survey;
- Inspector;
- Data quality – measure of the accuracy, typically +/- 0.020m or less;
- Number of pipes entering the manhole;
- Manhole depth;
- Size, material, angle, and invert of outlet pipe;
- Size, material, angle, and invert of each inlet pipe; and
- Notes – indicating additional relevant information (e.g. if the cover was stuck or covered and all of the information could not be surveyed).

1.1.2 Survey Gaps

In areas where topographic survey was not carried out due to safety concerns and/or time and budget constraints, a combination of other information sources were used to fill in the gaps as follows:

- Location – the City GIS database was used to determine structure locations (accuracy varies depending on the source);
- Cover elevation – the LIDAR digital elevation map obtained for the study area was used to calculate the elevation, typically +/-0.10 m; and
- Size, material, and invert of pipes entering – the City GIS database and available record drawings were used to fill in pipe information, where records were not available inverts were estimated.

1.2 Existing Flow Model

1.2.1 Measured Flows

The City of Saint John provided flow monitoring data collected from previous studies completed in the study area. A flow monitoring report was completed to summarize all of the data from previous studies and is included in Appendix B.

It should be noted that, although nearly all parts of the study area were tributary to at least one of the flow monitoring sites, the average tributary area of the flow monitoring sites is large (greater than 100 ha). This is suitable for big picture assessments like this study but does not provide enough detail for comprehensive subsewershed evaluations. For subsewershed assessments, flow meters should be located so that tributary areas are small enough to produce accurate results for each section of pipe (i.e. on a street by street basis).

1.2.1.1 Measured Sanitary Flows

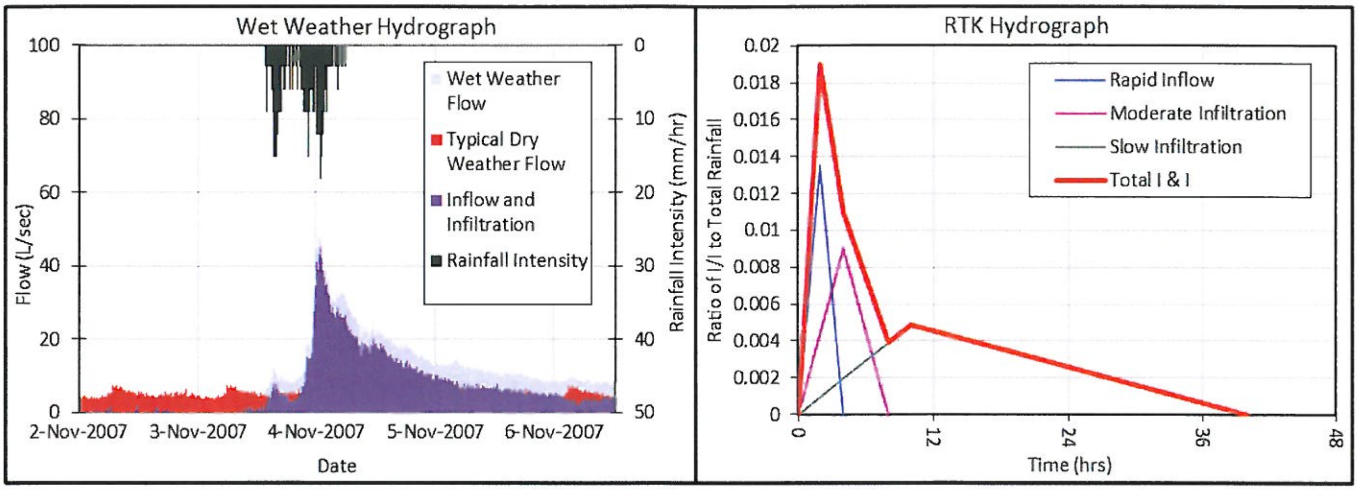
Average dry weather flows were estimated for each gauging site by extracting wet weather flows (days with rain and the two days following rain) and taking the average flow on the remaining days. An average flow and typical dry weather flow pattern were determined for each flow monitoring site.

The sanitary flow was divided equally among the manholes tributary to each site and the relevant flow pattern was applied to each manhole.

1.2.1.2 Measured Extraneous Flows

The extent of inflow and infiltration was determined for each of the relative meter locations in the study area by comparing wet weather flows to the typical dry weather flows at each site. The wet weather hydrograph for each site was used to generate an RTK hydrograph by combining triangular hydrographs from three components of flow: rapid inflow, moderate infiltration, and slow infiltration. Three parameters are determined for each flow component: R is a measure of the ratio of rainfall that enters the system (R should be zero in a sanitary system); T is a measure of the time to peak and depends on the size of the catchment; and K depends on the relative length of the recession curve. An example of a wet weather hydrograph and estimated RTK hydrograph for the same site are shown in Figure A1.

Figure A1 – Wet Weather Hydrograph and associated RTK Hydrograph for Example Site



The RTK parameters for rapid, moderate, and slow were input to the model at each manhole and the model was calibrated to ensure these parameters produced results similar to the measured wet weather data.

1.3 Design Flow Model

1.3.1 Theoretical Flows

1.3.1.1 Theoretical Sanitary Flows

Sanitary flows were calculated based on the Atlantic Canada Wastewater Guidelines Manual. The following sewage generation rates were used:

- Residential – 2.5 persons/residence using 340 L/person/day;
- Commercial – 6 L/m^2/day (area of building); and
- Industrial – water usage data (where available) or 6 L/m^2/day (area of building).

For multi-unit residential an estimate of the number of units was made using the orthophoto and Google Street View to approximate the building footprint and number of stories.

Sanitary loads were assigned to a loading node on each property. These loads were then input to the model using the SewerGEMS LoadBuilder Wizard which automatically distributes loads in a number of ways. The method that was used allocates loads to the nearest pipe, sums all of the loads allocated to that pipe and applies them at the manhole that is closest to the loading node.

1.3.1.2 Theoretical Extraneous Flows

Extraneous flows were calculated based on the Atlantic Canada Wastewater Guidelines Manual. A general inflow and infiltration (I/I) allowance was used based on area. The area allowance ranges from 0.14 to 0.28 L/sec/gross hectare. The total area of a sewershed was used to calculate the I/I allowance and then applied equally to each manhole located within the sewershed. The resulting allowance per manhole ranged from 0.10 to 0.18 L/sec.

1.4 Wastewater Pumping Stations

Information on the wastewater pumping stations in the study area was input to the model from two main sources. An energy feasibility study was carried out by Fundy Engineering & Consulting Ltd. and Terrain group Inc. in 2008 for Saint John's water and wastewater facilities. There was a detailed investigation and report completed for selected stations and a site visit and general report completed for smaller stations. The energy reports were used to obtain information on pumps and operating conditions at each of the stations. The information collected at each station varied due to site conditions and the two levels of reports that were prepared (i.e. less detail was available at the smaller stations).

Record drawings were available for most stations and were used to input physical details, including the following:

- Wet well dimensions and base elevation;
- Inlet size, invert elevation, and material;
- Forcemain size and material (City GIS was used if this was not included);
- Pump schedule (on/off elevations for each pump); and
- Overflow size, invert elevation, and material.

1.5 Database Expansion

The SewerGEMS database contains all of the information contained in the previous sections – physical location and characteristics of each structure and pipe, as well as operation data, sanitary loads, and I/I rates.

The City's current GIS database provides structure location but has no elevation information. It provides pipe diameter and material but each pipe in the GIS does not necessarily start and stop at a structure. In order to expand the database to include other areas of the City, the data contained in the following Tables 1.1 and 1.2 would be necessary, as a minimum, to establish structure and pipe location and connectivity.

Table 1.1 – Minimum Manhole Data

Manholes				
Label	X (m)	Y (m)	Elevation (Ground) (m)	Elevation (Invert) (m)
1973STM37281	2,540,335.65	7,366,250.85	58.27	55.76
1988SAM37455	2,540,273.55	7,366,242.75	56.84	53.58
1988STM37461	2,540,283.25	7,366,211.00	57.82	56.15
1973STM37306	2,540,328.64	7,366,123.23	62.51	60.51
1973STM37546	2,540,326.51	7,366,080.48	63.96	61.96
1973SAM37520	2,540,267.13	7,366,136.68	59.94	56.64
1973SAM37307	2,540,331.55	7,366,133.08	62.12	58.82
1973SAM37549	2,540,327.42	7,366,077.51	64.10	60.80
1967SAM41537	2,538,464.96	7,363,788.14	8.46	6.20

Table 1.2 – Minimum Pipe Data

Pipes				
Label	Start Node	Stop Node	Diameter (mm)	Material
STM-7001	PRSTMH-2099	OF-0000ST147323	300.0	Concrete
1973ST143795	1973STM37281	EMH-211	600.0	Concrete
1973ST143814	1973STM37289	1973STM37281	600.0	Concrete
1973SA144223	1988SAM37455	1973SAM37454	200.0	PVC
1988SA144247	1988SAM37459	1988SAM37455	200.0	PVC
1988ST144245	1988STM37461	1988CBT37457	200.0	PVC
1988ST144267	1988CBT37462	1988STM37461	200.0	PVC
1973ST143838	1973STM37306	1973STM37289	300.0	Concrete
1973ST144426	1973STM37546	1973STM37306	300.0	Concrete

APPENDIX B

Flow Monitoring Data Analysis

Appendix B - Flow Monitoring Data Analysis

1.1 Background

The City of Saint John provided flow monitoring data collected from previous studies for use in the "Harbour Clean-up" project (*Wastewater System Upgrades – Conceptual Design Report*, December 2008, CBCL Limited and Crandall Engineering). Additional flow monitoring was also carried out by CBCL at key locations for use in the Harbour Clean-up report. Data relative to the Combined Sewer Overflow study area was compiled from these previous studies and analysed. This data was used to define existing flows for a number of tributary areas in East Saint John for use in the overall existing conditions hydraulic model of the study area. Areas tributary to each flow monitoring site were delineated based on existing sewer information and are identified on Drawing B1.

Previously conducted flow monitoring programs relative to the study area included the following:

- Majors Brook Pumping Station Sewershed, 2006, Hydro-Com Technologies Ltd.;
- I/I Reduction Program Phase 1 – Evaluation of the Marsh Creek Sewerage Scheme, 2005, Hydro-Com Technologies Ltd.;
- I/I Reduction Program Phase 2 – Hickey Road Pumping Station Sewershed, 2005, Hydro-Com Technologies Ltd.;
- I/I Reduction Program Phase 3 – Hazen Creek Trunk Sewer Capacity Assessment, 2006, Hydro-Com Technologies Ltd. for Touchie Engineering;
- I/I Reduction Program Phase 4 – Bayside Drive, East Saint John, 2006, Hydro-Com Technologies Ltd.;
- I/I Reduction Program Phase 5 – East Saint John, 2007, Hydro-Com Technologies Ltd.; and
- Saint Harbour Cleanup – East Saint John, 2008, CBCL Limited.

A total of approximately 1100 hectares of East Saint John was gauged in these studies, representing approximately 100 percent of the total serviced study area. It should be noted that, due to on-going renewal and installation of infrastructure in East Saint John, some of the tributary areas and system characteristics have changed since the metering was performed. Therefore, some of the data may not completely represent the current conditions.

Typically, flow monitoring was conducted for a period of about 30 days during the months of November and April, traditionally the wetter times of the year. As a result, estimates of dry weather flow are considered conservative, on the higher side of average dry weather flows.

1.2 Review of Previous Studies

A review of previous flow monitoring reports was performed to obtain relevant information relating to the Combined Sewer Overflow Study. All of the previous monitoring studies used the same assessment criteria of infrastructure. The assessment criteria are shown below.

- *Good state of hydraulic repair = measured average flow/design average flow < 1.0*
- *Fair state of hydraulic repair = measured average flow/design average flow 1.3-2.0*
- *Poor state of hydraulic repair = measured average flow/design average flow >2.0*

The following Table B1 shows a summary of the results from all of the relevant studies relating to areas located in the Combined Sewer Overflow Study Area.

Table B1 Previous Flow Monitoring Summary

Study Report	Area	Avg. Measured Flow (L/s)	Avg. Design Flow (L/s)	Ratio Avg. Measured to Avg. Design Flows	Condition Assessment
Majors Brook	Moorland Trailor Park	5.74	1.28	4.48	POOR
	Rothesay Ave. Commercial	1.32	0.37	3.56	POOR
Phase 1 – Marsh Creek Sewage Scheme	Forest Hills Residential	36.23	6.67	5.43	POOR
	Forest Hills Commercial	2.35	1.39	1.69	FAIR
	Glen Falls	11.19	4.49	2.49	POOR
	Majors Brook	7.77	3.99	1.95	FAIR
	Silver Falls Park	8.77	3.63	2.42	POOR
	Rothesay Ave. Commercial	1.67	4.84	0.35	GOOD
	Westmorland Road Area	0.73	0.42	1.74	FAIR
Phase 2 – Hickey Road Pumping Station Tributary Area	Hickey Road	2.58	4.14	0.62	GOOD
	Silver Falls	6.71	1.03	6.51	POOR
	Bon Accord	1.91	2.08	0.92	GOOD
	Lakewood Heights	8.91	2.43	3.67	POOR
Phase 3 – Hazen Creek Treatment Plant	Champlain Heights	38.54	4.74	8.13	POOR
	Grandview Industrial Park	14.83	2.68	5.53	POOR
	McAllister Industrial	18.35	2.92	6.28	POOR

Study Report	Area	Avg. Measured Flow (L/s)	Avg. Design Flow (L/s)	Ratio Avg. Measured to Avg. Design Flows	Condition Assessment
Majors Brook	Moorland Trailor Park	5.74	1.28	4.48	POOR
	Rothesay Ave. Commercial	1.32	0.37	3.56	POOR
Phase 1 – Marsh Creek Sewage Scheme	Forest Hills Residential	36.23	6.67	5.43	POOR
	Forest Hills Commercial	2.35	1.39	1.69	FAIR
	Glen Falls	11.19	4.49	2.49	POOR
	Majors Brook	7.77	3.99	1.95	FAIR
	Silver Falls Park	8.77	3.63	2.42	POOR
	Rothesay Ave. Commercial	1.67	4.84	0.35	GOOD
	Westmorland Road Area	0.73	0.42	1.74	FAIR
Phase 2 – Hickey Road Pumping Station Tributary Area	Hickey Road	2.58	4.14	0.62	GOOD
	Silver Falls	6.71	1.03	6.51	POOR
	Bon Accord	1.91	2.08	0.92	GOOD
	Lakewood Heights	8.91	2.43	3.67	POOR
Tributary Area	Park				
	Hickey Road Pumping Station Trib. Area	33.27	9.68	3.44	VERY POOR
Phase 4 – East Saint John, Bayside Drive Area	Bayside Drive	79.0	8.16	9.66	POOR*
	Mount Pleasant Avenue	27.0	1.15	23.5	POOR*
	River Avenue	41.9	0.43	97.3	POOR*
	Westmorland Road/Russell Street	11.6	1.14	10.2	POOR*

**Condition assessments may not be representative of actual condition due to sewers in these areas being combined. Higher ratios of measured average to design average flows are expected due to storm water influence.*

***Note:** Average Design Flows in the above table were estimated in previous reports using an estimated population and an average sewage generation rate of 150 L/person/day (excluding extraneous flows).*

1.2.1 Additional Findings From Previous Flow Monitoring

The Majors Brook Pumping Station I/I Study Report in 2006 noted peak flows from this area during wet weather were slightly delayed which indicated that ground water infiltration could be influencing the sanitary sewers more than direct connections like catch basins which would produce more rapid response. One interesting conclusion in the report was that there were no over flows measured during the study at the Majors Brook Pumping station. This indicates that even though influenced by extraneous flows the pump station appeared to handle the flows entering it during wet weather.

Phase 1 Flow Monitoring Study of the Marsh Creek Sewerage Scheme concluded from depth readings that there was surcharging of the system at all the meter sites during significant rain events. This indicates that there is high potential for overflows at the various overflow locations along the Marsh Creek Sewage Scheme. It is interesting to note, however, that during surcharging of the system at Simpson Drive Pump Station, depths did not reach the elevation of the overflow according to the study report. This indicates that there was no overflow to Marsh Creek at this location and that the station appears to have capacity to handle tributary flows. This being said, there is an overflow to the storm sewer at the intersection of Parkhill Drive and Golden Grove Road which is upstream of the Simpson Drive Pump Station. The study did conclude that there was surcharging of the system at this location and suspected overflows to the storm sewer. If the system did overflow here, this may be why there were no overflows at Simpson Drive.

Phase 2 Flow Monitoring Study was of the Hickey Road Pump Station Tributary area. No information was obtained from the study report about overflows occurring at the Hickey Road Pump Station. The report did indicate that existing pump capacities at this station were found to be well below theoretical capacities however.

Phase 3 Flow Monitoring Study of the Hazen Creek Treatment Plant Sewershed concluded (from flow patterns measured at the Hickey Road Pump Station force main outlet) that the Hickey Road station was surcharged during peak rain events. The basis for this conclusion was measured peak wet weather flows at the forcemain outlet matching the estimated two pumps running capacity of the station determined in the Phase 2 study. This also indicates that there is high potential for overflows into Little River at the Hickey Road Pump Station overflow.

Phase 4 Flow Monitoring Study concluded that there was surcharging during rain events in the sanitary sewer on Russell Street just upstream of One Mile Pump Station. This was indicated by negative flows during significant rain events at this meter site. This observation indicated that the One Mile Pump Station capacity may have been exceeded during rain events that occurred over the study period. The study report indicated that there was an overflow located just upstream of the flow meter location on Russell Street and overflows were observed here during rain events as a result of surcharging at the One Mile pumping station. It should be noted that since this metering study there has been work performed on this section of sewer related to the One Mile Interchange project which may have changed the hydraulics of the system.

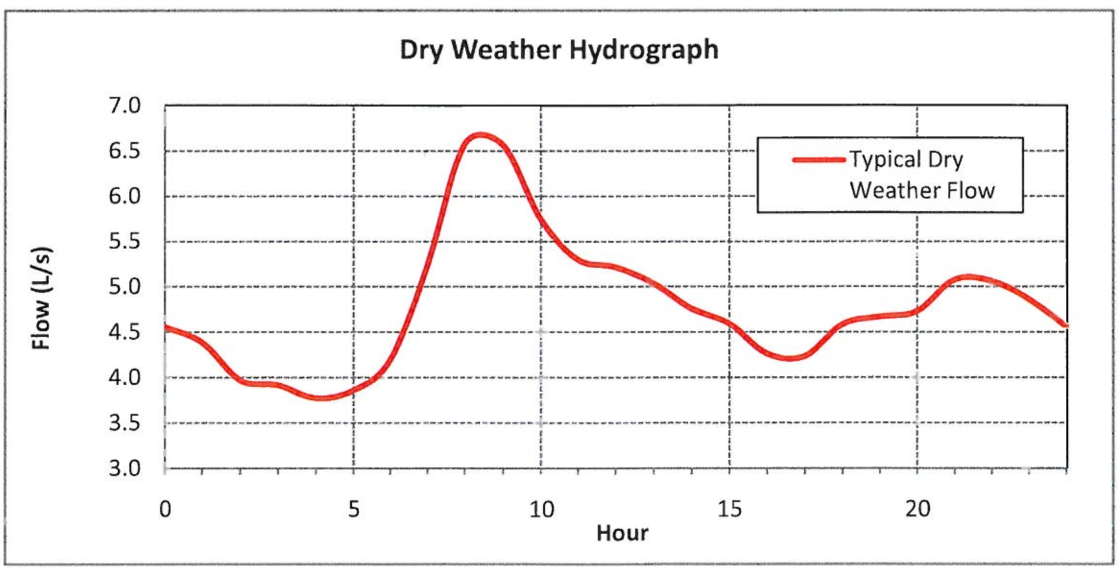
The Phase 5 Flow Monitoring Study of the Forest Hills area only concluded that there was significant surcharging in the main 300 mm sanitary sewer that runs from the bottom of Mountain Road, down Braemer Drive and then Down Parkhill Drive to the trunk sewer that ultimately flows to Simpson Drive pump station. Since then, this sewer has been upgraded to a 375 mm diameter PVC line (in 2008) by the City as per a recommendation from this report. This upgrade may have solved the surcharge problem in this particular area however based on the studies flow data there are major inflow/infiltration that influence the Upper Glengarry Drive and Mountain Road sanitary sewers. This data indicates that these upstream sewers are in need of repair or separation as well, to truly correct the problem.

1.3 Overview of Analysis Performed for CSO Study

The flow monitoring data was used to determine typical dry weather flow patterns for the sub-sewer sheds in the study area. It was also used to determine the extent of inflow and infiltration (I/I) into the system at each of the gauged locations during wet weather.

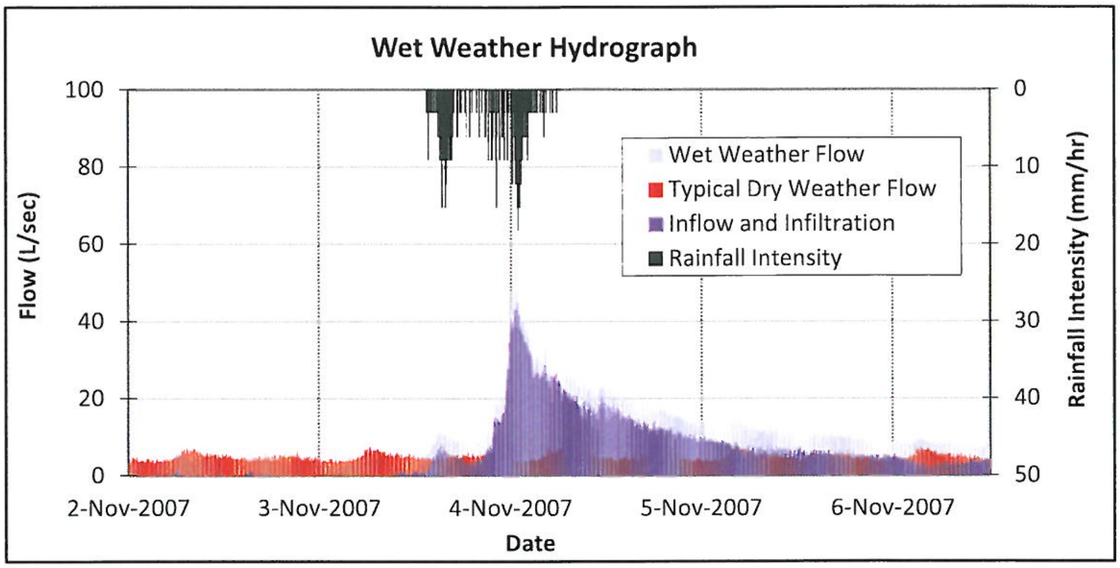
Average dry weather flows were estimated for each gauging site by extracting wet weather flows (days with rain and the two days following rain) and taking the average flow on the remaining days. A typical flow pattern was selected for each site by estimating the hourly flows and comparing these to the average flow for a representative dry weather flow day. Figure B1 shows a typical dry weather flow hydrograph.

Figure B1: Representative Dry Weather Hydrograph



The extent of inflow and infiltration was determined for each of the relative meter locations in the study area by comparing wet weather flows to the typical dry weather flows at each site. Typical dry weather flows for that site were subtracted from the wet weather flows – during a storm event until the increased flows were back down to typical dry weather flows. Figure B2 shows the dry weather flow (red), the wet weather flow (blue), and the difference between the two which represents inflow and infiltration (purple). The rain intensity over the duration of the storm event is also shown on Figure B2 to show how much time it takes for water to enter the system.

Figure B2: Representative Wet Weather Hydrograph



The total volume of inflow and infiltration was then calculated and compared to the total volume of rain that occurred during the wet weather event. The percentage of rainfall that enters the tributary sanitary system was determined for each site.

The following general characterizations can be made based on the percentage of rainfall entering the sanitary systems:

- ***0-4 %*** of rain entering indicates that the sanitary system is relatively tight with minimal inflow and infiltration influence;
- ***4-20 %*** of rain entering indicates a wet sanitary system; connections like foundation drains may be tied directly to the sanitary system, and infiltration could be entering due to poor condition of older pipes and connections;
- ***>20 %*** of rain entering indicates that the system is combined and most likely has direct connections like catch basins, sump pump discharges, roof leaders, and foundation drains.

Drawing B2 shows the estimated percentage of rainfall entering the sanitary system during significant rain events. Table B2 shows a summary of the analyzed data for each flow monitoring site.

The Environment Canada, Atlantic Canada Standards and Guidelines Manual for the Collection, Treatment, and Disposal of Sanitary Sewage was used to generate estimated average domestic design flows tributary to each of the meter sites. These flow estimates were generated using estimated equivalent domestic populations and the following formula:

$$Q(d) = \frac{PqM}{86.4} + IA$$

Q(d) – Peak Domestic Flow (including extraneous flow) in (L/s)
P – Design Population, in Thousands
q – Average Daily per Capita Domestic Flow in L/cap.d (exclusive of extraneous flows)
M – Peaking Factor (as derived from the Harmon Formula)
I – Unit of Extraneous flow, in L/s/ha
A – Tributary Area in Gross Hectares

The design population (P) for the calculations assumed 2.5 persons/ per single family home and a sewage generation rate of 340L/person/day (exclusive of extraneous flows) as suggested in the Design Manual. A peaking factor was then applied to this average domestic flow.

The peaking factor as derived from the Harmon Formula is calculated as follows:

$$M = 1 + \frac{14}{4 + p^{0.5}}$$

M – Peaking Factor as Derived from the Harmon Formula
P – Design Population, in Thousands

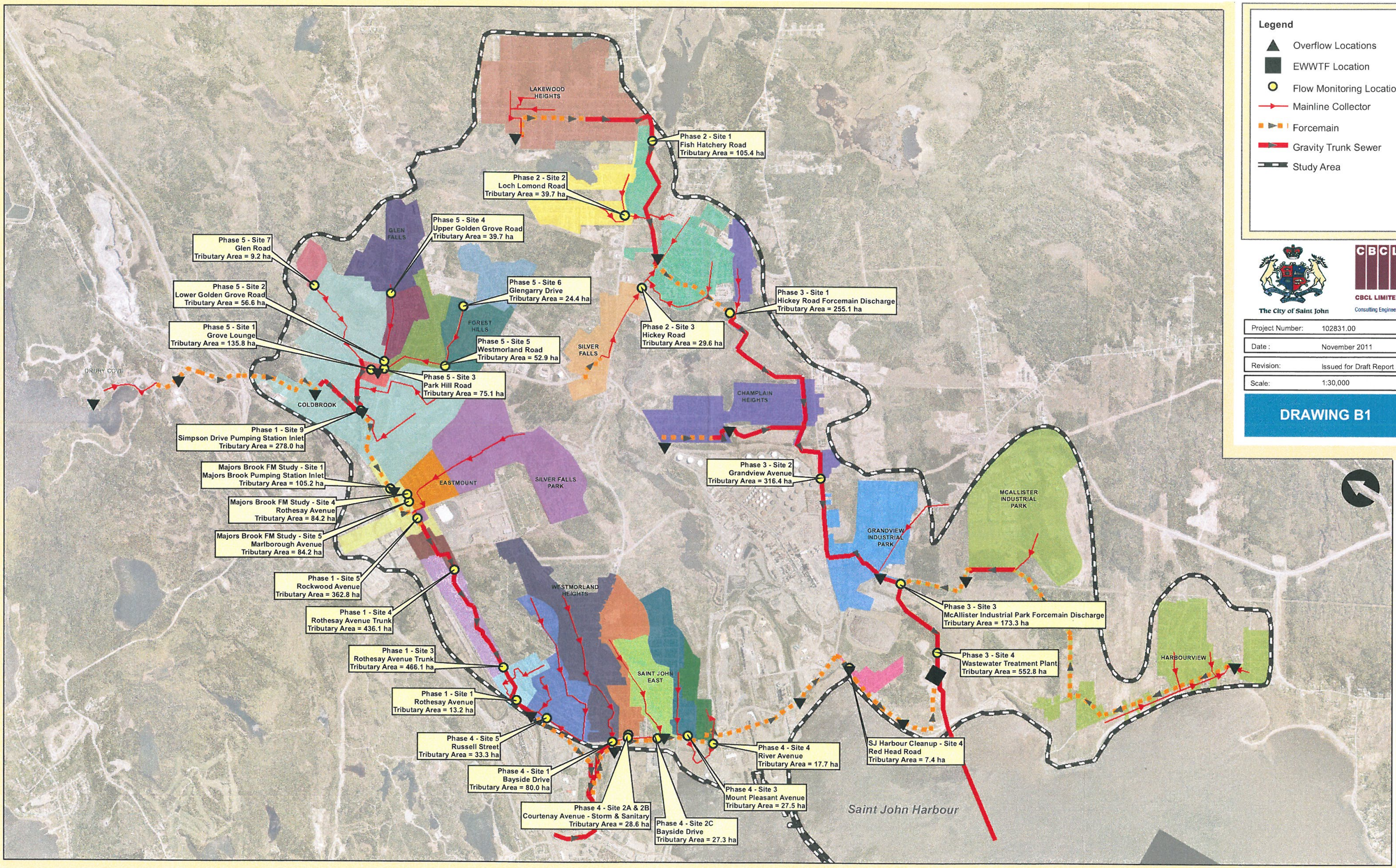
The final variable in the sewage calculation formula is I, the measurement for extraneous flows. The range in values for extraneous flows (including peak infiltration and peak inflow) is between 0.14 to 0.28 L/s per gross hectare. The peak flow per gross hectare varies depending upon the density of the area as well as the development level. For all the flow monitoring areas the allowance of 0.14 L/s/ha was used. These flow estimates were compared to measured flows to further analyze inflow and infiltration. These estimates and comparisons to measured flows are also shown on Drawings B3 and B4 and summarized in Table B2.

1.4 Conclusion

Overall the flow metering data from all relative sites in the study area showed some signs of inflow and infiltration (I/I) during significant rainfall events. As expected, areas with newer infrastructure showed less signs of I/I than areas with older infrastructure. Some areas are truly combined systems with only one sewer conveying both sanitary and storm flows. These areas produced extreme I/I influence which is expected. It should be noted that, although nearly all parts of the study area were tributary to at least one of the flow monitoring sites, the average tributary area of the flow monitoring sites is large (greater than 100 ha). This being said, the data from the previous Flow Monitoring Studies is suitable for big picture assessments like this study, but does not provide enough detail for comprehensive subsewershed evaluations.

Combined Sewer Overflow Reduction Strategy - EWWTF Collection System

Flow Monitoring Locations & Tributary Areas



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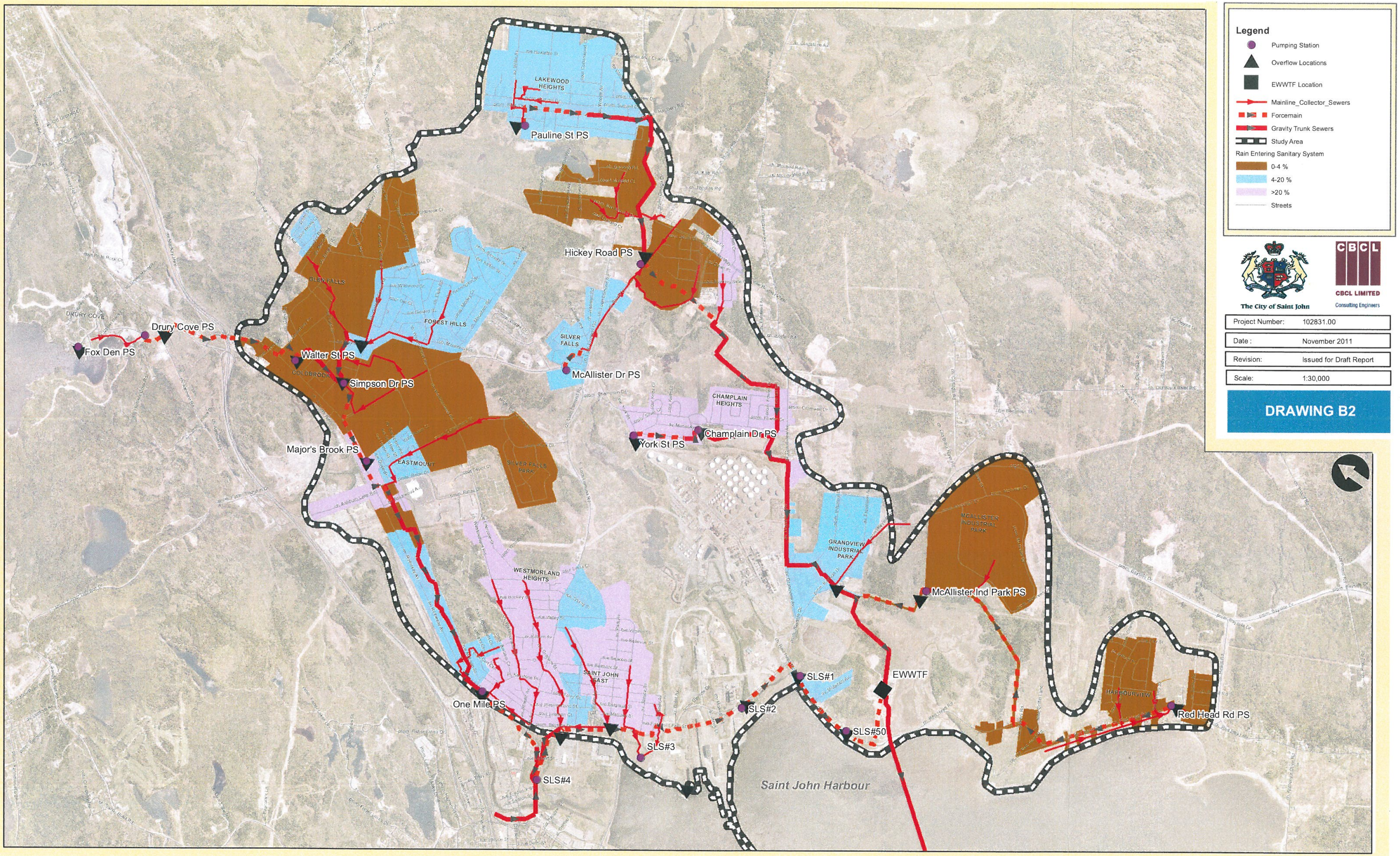
DRAWING B1

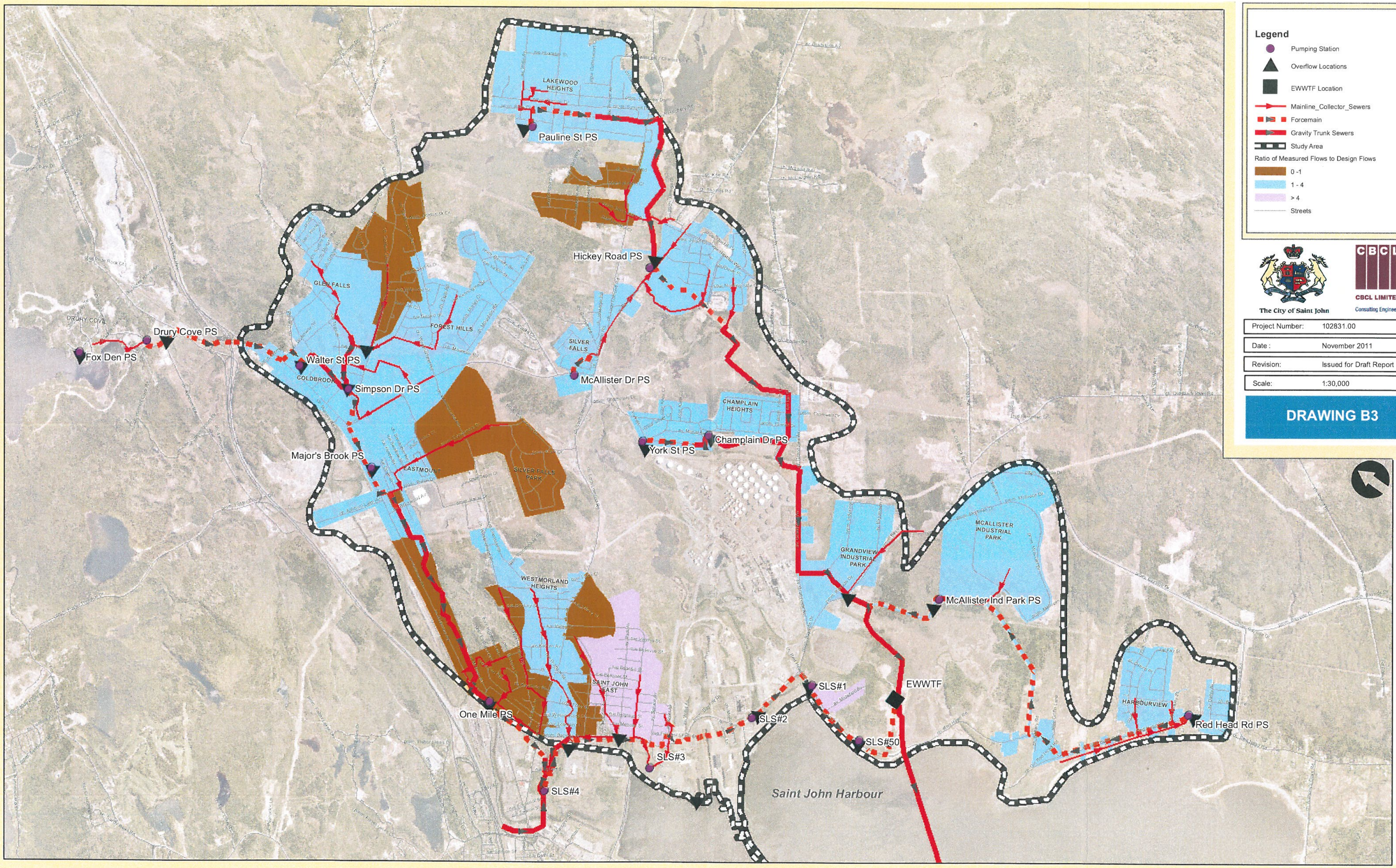
Combined Sewer Overflow Reduction Strategy - EWWTF Collection System

Portion of Rainfall Entering the Sanitary/Combined Sewers During Significant Rain Events



Combined Sewer Overflow Reduction Strategy - EWWTF Collection System

Ratio of Average Measured Flows to Estimated Atlantic Canada Standards Average Flows



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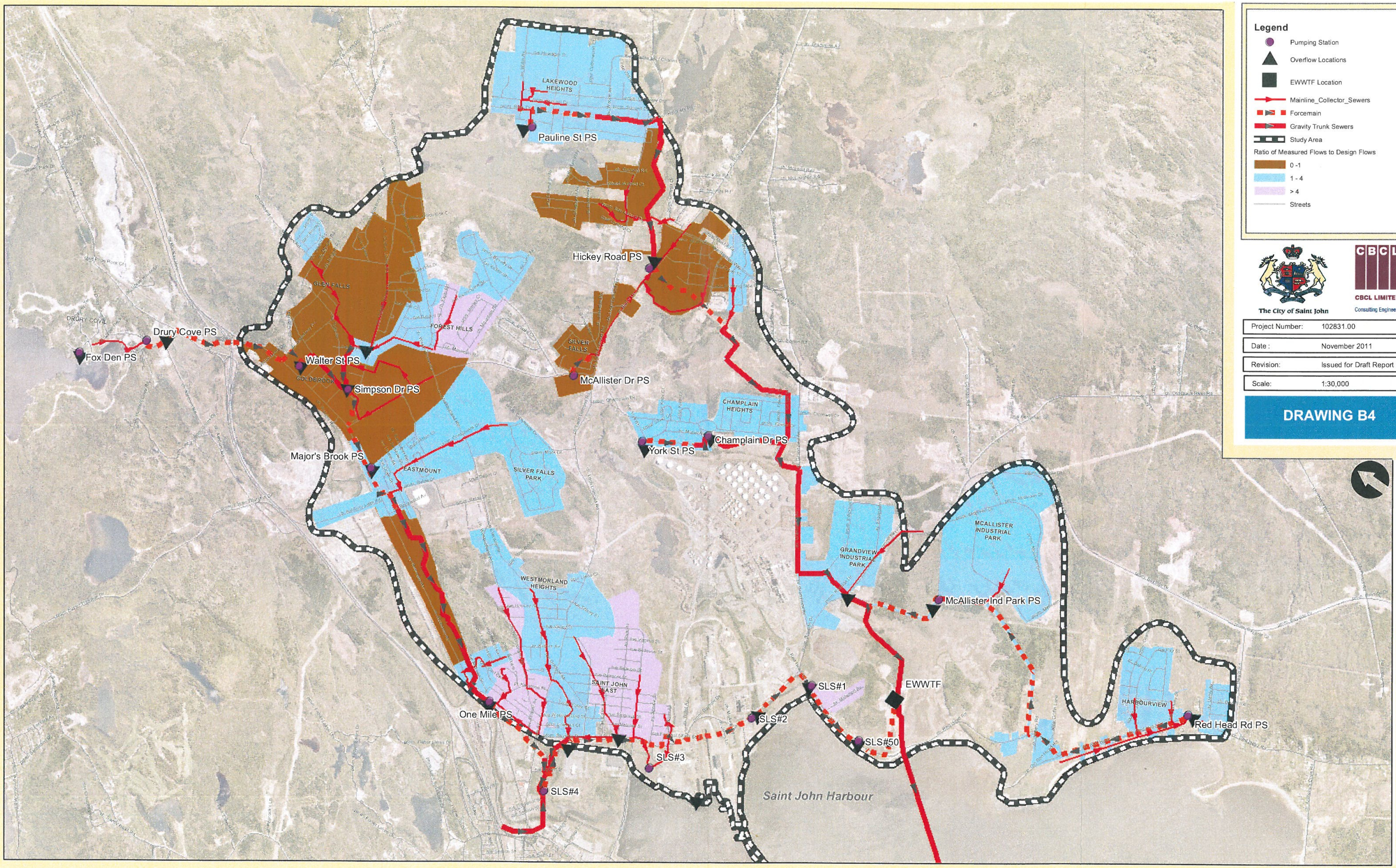
DRAWING B3

Combined Sewer Overflow Reduction Strategy - EWWTF Collection System

Ratio of Peak Measured Flows to Estimated Atlantic Canada Standards Design Flows



Legend

- Pumping Station
- Overflow Locations
- EWWTF Location
- Mainline_Collector_Sewers
- Forcemain
- Gravity Trunk Sewers
- Study Area

Ratio of Measured Flows to Design Flows

- 0 -1
- 1 - 4
- > 4
- Streets

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DRAWING B4