COLLINGWOOD

Town of Collingwood

FINAL MASTER PLAN

Collingwood Water and Sanitary Sewer Systems

COLE

DECEMBER 2019

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December 19, 2019
Reference No.: 2017-1013

Town of Collingwood
P.O Box 157, 545 Tenth Line North
Collingwood, ON L9Y 3Z5

Attention: John Velick, P.Eng.
Manager, Engineering Services

Final Master Plan for Collingwood Water and Sanitary Sewer Systems

We are pleased to submit our Final Report for the Collingwood Water and Sanitary Sewer Systems Master Servicing Plan. It has been a pleasure to work with you on this project and we look forward to supporting the Town of Collingwood on future assignments.

Best Regards,
COLE ENGINEERING GROUP LTD.

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Issues and Revisions Registry

Identification	Date	Description of issued and/or revision
Draft Report	May 3, 2019	For internal review
Draft Report	May 3, 2019	For client review
Revised Draft Report	September 17, 2019	For internal review
Revised Draft Report	September 20, 2019	For client review
Final Draft Report	December 6, 2019	For client review
Final Report	December 19, 2019	Final Report

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Executive Summary

The Town of Collingwood has completed a Master Servicing Plan for Water and Sanitary Sewer Systems to identify water and sanitary servicing projects that will be required to accommodate growth over the planning horizon, including residential and employment growth. The planning horizon included planned growth, which is anticipated to be completed by the year 2032, potential growth, which is anticipated to be completed by the year 2044, and built boundary growth. The Master Servicing Plan also considered servicing of neighbouring communities and servicing of currently unserviced development areas within the Town. By considering water and sanitary requirements together, an optimal design and delivery of services can be planned for.

Alternative water and sanitary servicing solutions have been developed and evaluated based on their natural, physical, social/cultural and financial impacts. In accordance with Approach #1 under the Municipal Class Environmental Assessment document (MCEA, October 2000 as amended in 2007, 2011 and 2015), this Master Plan report documents the completion of Phases 1 and 2 to satisfy the requirements of the MCEA Master Plans. Most importantly, this report identifies the necessary projects that should be completed to achieve the objectives of this Master Plan over the planning horizon. This Master Plan identifies the methodology and rationale for identifying required Schedule “A”, “A+”, “B” and “C” projects to accommodate and facilitate growth within the Town to the year 2044.

Public and Stakeholder consultation was conducted throughout this study. A Notice of Study Commencement was advertised in November 2017, a Notice of Public Information Centre was advertised in March 2019 and a Public Information Centre was held in March 2019. Comments were received from review agencies and the general public and these have been identified and addressed throughout this Master Plan. The Study Area for this Master Plan includes the entire municipal jurisdiction of the Town of Collingwood. The Master Plan also considered relevant regulatory requirements and policies including the Safe Drinking Water Act, Provincial Policy Statement, Growth Plan for the Greater Golden Horseshoe, the Town’s Official Plan and Simcoe County’s Official Plan. The Growth Plan for the Greater Golden Horseshoe identified growth targets for the year 2031 for the Town. The Growth Plan identified a target residential population of 33,400 residents and a target employment population of 13,500 jobs in 2031.

To determine the timing and location of specific developments, the Town’s Planning Department provided detailed information which was used to develop two growth horizons. A total of 41 planned developments were identified throughout the Town. These developments included both residential and non-residential developments. It was estimated that planned developments would increase the residential population by 12,366 persons for a total population of 34,159 persons. It was estimated that these developments would be completed by the year 2032. A total of 45 potential developments were identified throughout the Town. These residential and non-residential developments would occur between the years 2033 and 2044 and would increase the residential population by 9,631 persons. In 2044, the projected population was estimated to be 43,790 persons with the completion of all planned and potential developments. Consideration was also given to the Town’s built boundary. In total, 484ha of lands have been identified as developable in the period beyond 2044. Based on a density of 50 persons or jobs per hectare, it was estimated that the built boundary lands could accommodate an additional residential population of 16,104 persons. With the completion of development in the built boundary lands, the future residential population of Collingwood is anticipated to be 59,894 persons. Assuming a constant growth rate over this period, completion of development in the built boundary lands is anticipated to occur in 2064.

A detailed review of the study area was completed based on available information sources as well as analysis and data collection. Information on the natural and social environment was consolidated and detailed analysis of the Town's water and sanitary systems was completed. The performance of these systems was analyzed using modelling tools developed through this study. To assess performance, water and sanitary system performance criteria were developed. The water system assessment identified that the demands on the Raymond Baker Water Filtration Plant (WTP) are stabilizing but are reaching 85% of the WTP's available capacity, available storage and pumping capacity is sufficient for existing conditions and fire flow capacity was identified as generally adequate but three areas of concerns were identified, two of which will be addressed through the construction of the Stewart Road Pumping Station. The sanitary system assessment identified that the Collingwood Wastewater Treatment Plant (WWTP) currently has sufficient rated capacity for existing flows, all pumping stations and forcemains have sufficient capacity, a small number of local sewers were identified as having capacity issues, and the capacity of the trunk sewer system is limited by the peak flow capacity of the Collingwood WWTP.

Future growth needs were identified for the water and sanitary systems for planned growth, potential growth, built boundary growth, servicing of neighbouring communities and servicing of unserviced areas. Based on the servicing requirements identified, a series of water system and sanitary system alternatives were developed and evaluated. Evaluation criteria considered the natural, social, technical and economic environments and impacts. For the water system, alternatives were developed for supply, storage and pumping. Preferred alternatives included water efficient measures for water supply, a new Zone 1 storage tank and a new Zone 3 booster pumping station. Additional pumping was also identified at Bob Davey to address built boundary growth. The preferred alternatives also included upgrades to major watermains, local watermains and system valves.

For the sanitary system, alternatives were developed for treatment, local sewer capacity issues and trunk sewer capacity issues. Following evaluation of alternatives, the preferred alternatives included expansion of the Collingwood WWTP, construction of a new forcemain from the Black Ash Sewage Pumping Station (SPS) to the Collingwood WWTP, local sewer improvements on Minnesota Street, Hurontario Street, Mountain Road and Huron Street, improvements to the flow diversion chamber at Hurontario and Second Street and replacement of siphons at Hickory Street and Spruce Street with new pumping stations and forcemains. Implementation of an inflow and infiltration reduction program was identified as an implementation measure which could defer local sewer improvements. A strategy was developed and included in this Master Plan to achieve reductions in inflow and infiltration.

Servicing of neighbouring communities was also considered. For the water system, additional upgrades were identified to provide additional water demand for New Tecumseth, Clearview Township and the Town of Blue Mountains (ToBM). For each of these municipalities, supply, storage, pumping and watermain capacity needs were used to identify and evaluate options. For the sanitary system, upgrades required to provide servicing to Nottawa were considered. Two options were developed. The preferred option includes connection of Nottawa to the existing sewer on Sixth Line and the installation of a third pump at the St. Clair SPS.

Sanitary servicing to five areas that currently have private septic systems for sanitary servicing was also evaluated. These areas included Oliver Crescent, Princeton Shores, West Highway 26, Mountain Road West and Beachwood. For each area, needs were considered and options for servicing were developed and evaluated. Preferred options include the installation of grinder pumps and low pressure sewers in Oliver Crescent, Princeton Shores, consideration of servicing of West Highway 26 and Mountain Road West through ToBM, and a combination of conventional sewer and low pressure sewers in the Beachwood area.

Figure ES-1-1 presents the recommended water system projects to service planned and potential development. **Figure ES-1-2** presents the recommended sanitary projects to service planned and potential development.

Implementation plans were developed for water and sanitary recommendations. The implementation plan for water system improvements includes projects to upgrade water supply, implement water efficiency measures, storage improvements in Zones 1 and 2, upgrades to two pumping stations, decommissioning of one pumping station, upgrades to six major watermains, 14 local watermain projects, nine system valve projects and three studies. For each project, project costs, rationale, funding, duration, completion timeline and EA schedule have been identified.

The implementation plan for sanitary system improvements includes an expansion to the Collingwood WWTP, a new forcemain from Black Ash SPS to the Collingwood WWTP, modifications to the flow diversion chamber at Hurontario and Second, sewer upgrades on Mountain Road, Minnesota and Huron. For each project, project costs, rationale, funding, trigger date, and EA schedule have been identified.

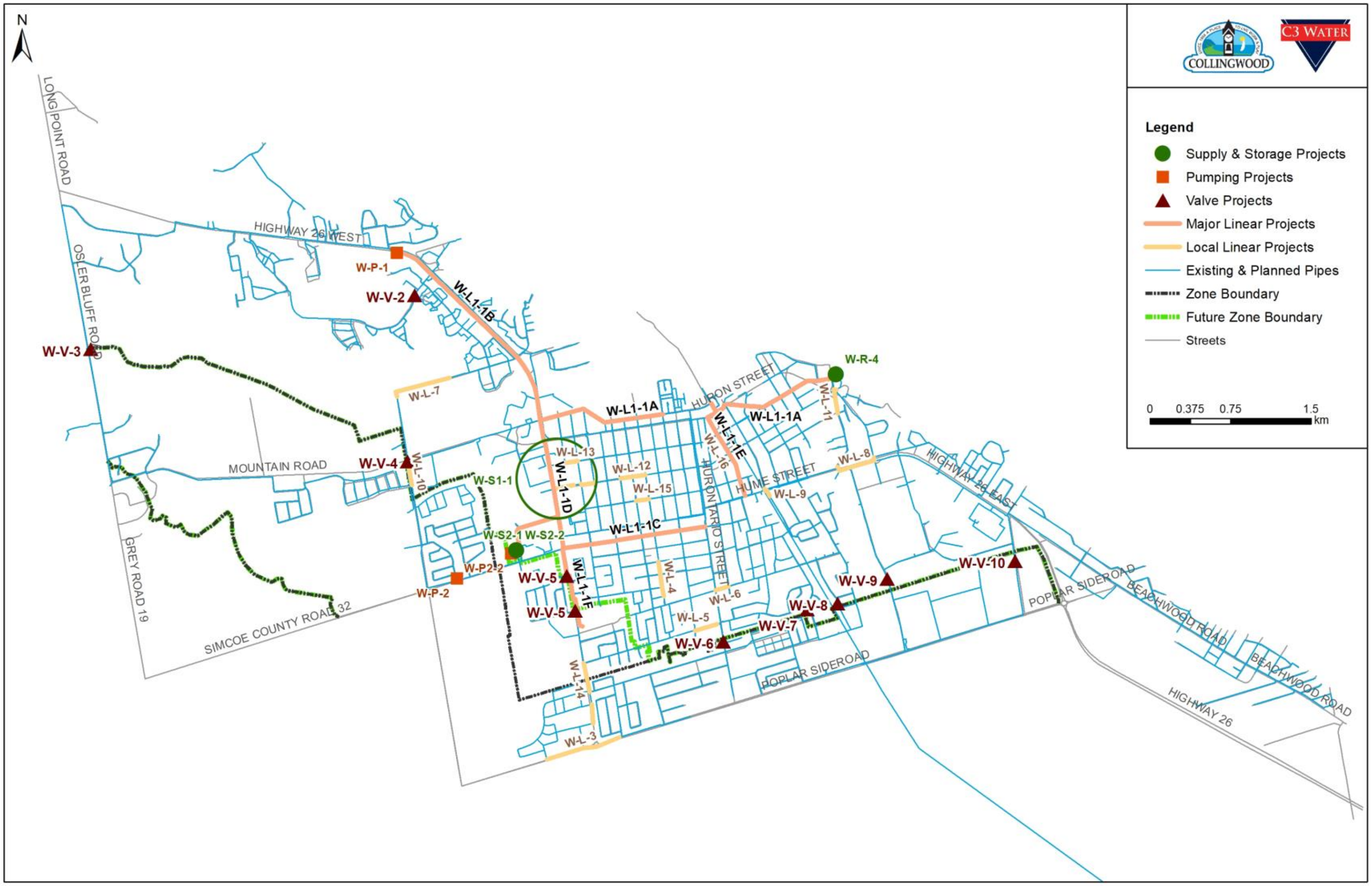


Figure ES-1-1 Recommended Water Projects

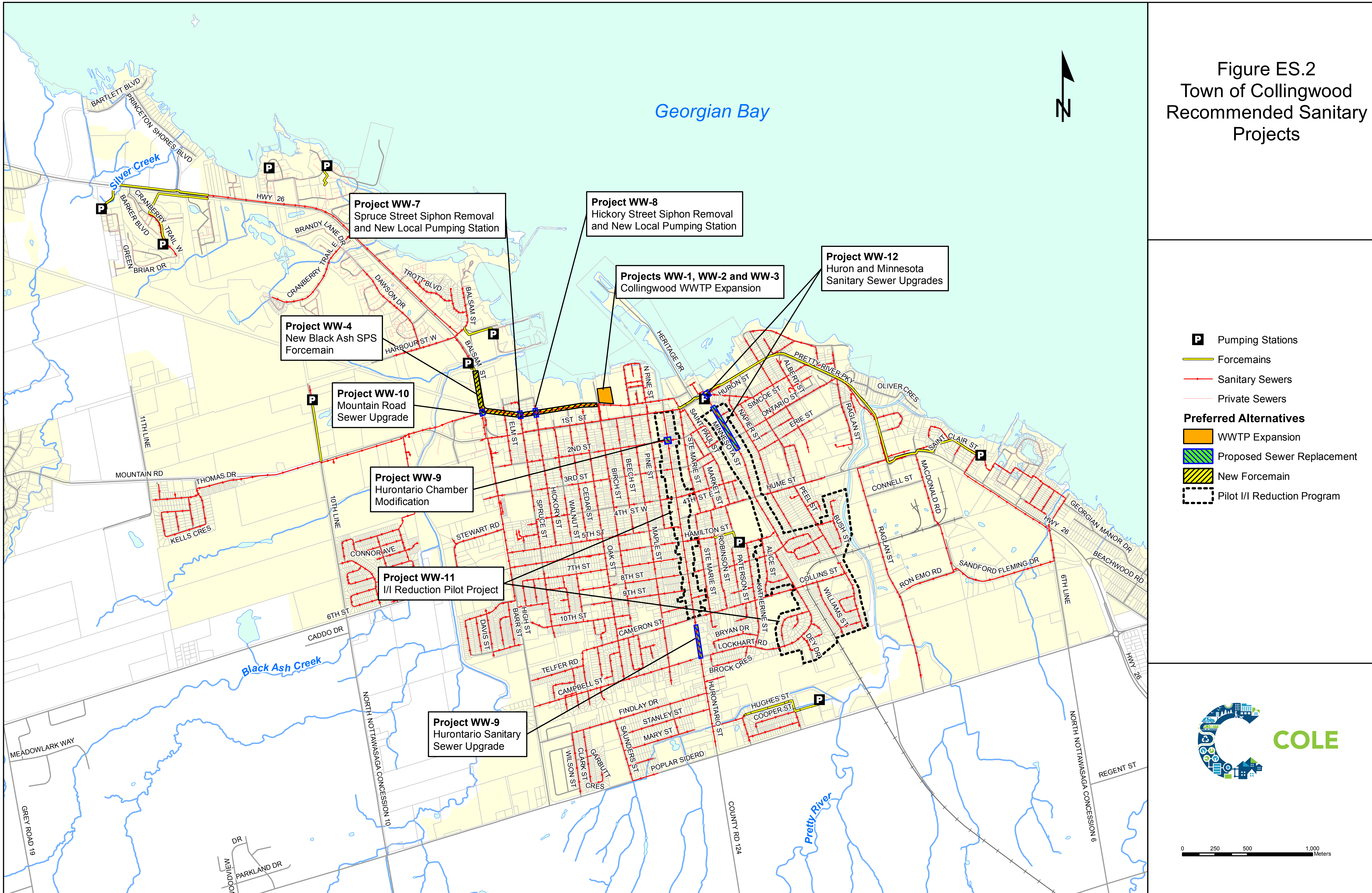


Figure ES.2
Town of Collingwood
Recommended Sanitary Projects

1 Introduction

The Town of Collingwood has completed a Master Servicing Plan for Water and Sanitary Sewer Systems to identify water and sanitary servicing projects that will be required to accommodate growth over the planning horizon, including residential and employment growth to accommodate planned and potential developments. The Master Servicing Plan also considers servicing of neighbouring municipalities as well as providing servicing to unserviced areas in the Town. By considering water and wastewater requirements, an optimal design and delivery of services can be planned for. This Master Plan has been developed to facilitate Collingwood's current and projected growth to ensure that sufficient water and sanitary servicing can be provided to support growth to the year 2044.

Alternative water and sanitary servicing solutions have been developed and evaluated based on their natural, physical, social/cultural and financial impacts. Solutions have been developed to provide servicing for planned and potential growth in the Town. Longer term built boundary growth, servicing of neighbouring communities and servicing of currently unserviced areas of the Town have also been considered. In accordance with Approach #1 under the Municipal Class Environmental Assessment document (MCEA, October 2000, as amended in 2007, 2011, and 2015), this Master Plan Report documents the completion of Phases 1 and 2 to satisfy the requirements of an MCEA Master Plan. Most importantly, this report identifies recommended projects to achieve the objectives of the Master Plan over the planning horizon. For Schedule 'B' Class EA projects identified in this report to be constructed within the next 10 years, the related public consultation, technical studies and detailed assessment of alternative solutions relating to these projects are completed under this assignment. If Schedule "C" Class EA projects are identified and prioritized within the next 10 years, then the Town will need to complete a detailed evaluation of alternatives to satisfy Phases 3 and 4 of the MCEA prior to the public review of an Environmental Study Report.

1.1 Municipal Class EA Process

As required under the Ontario *Environmental Assessment Act (EAA)*, this study followed the Municipal Class Environmental Assessment (MCEA) (October 2000, as amended in 2007, 2011, and 2015) planning process. The MCEA establishes a framework by which broad environmental outcomes of public sector infrastructure projects are reviewed and evaluated. The stated purpose of the EAA is to provide *the betterment of the people of the whole or any part of Ontario by providing for the protection, conservation and wise management in Ontario of the environment*. The EAA interprets environmental outcomes to be those associated with the natural, social, cultural, built, and economic environments.

The EAA requires that municipalities complete a MCEA for public works and infrastructure projects, including those for roads, transit ventures, and water and wastewater projects. Key principles of the MCEA process include:

- Consultation with stakeholders and affected parties upon study commencement, and throughout the process of the project;
- Consideration of all reasonable alternatives, including "alternatives to" and "alternative methods" of implementing a preferred solution;
- Identification and consideration of broad environmental effects, as identified previously, for each alternative under evaluation;

- The systematic evaluation of all alternative solutions and/or methods to determine the net environmental effects, based on available information; and,
- The provision of clear and comprehensive documentation that demonstrates how the MCEA planning process was followed, and to ensure transparency and traceability of the decision-making process for the project.

Under the MCEA, the Master Plan process allows a proponent, such as the Town of Collingwood, to prepare the planning, design, and construction of a group of related municipal works, rather than individually on a project-by-project basis. The benefits of the Master Plan approach include:

- The rationale for each individual project is more clearly articulated;
- The range of alternatives are more broadly addressed;
- The extent of potential environmental outcomes is better understood;
- There is an enhanced ability to assess cumulative outcomes; and,
- The process allows for the integration of land use planning.

The Master Planning process differs from project specific undertakings in several aspects and facilitates long range planning that enables the municipality to identify opportunities and proactively develop strategies for addressing any associated issues. This approach generally yields a series of individual activities, projects, and programs, together with a phased implementation plan that covers over an extended time period. Accordingly, the works may be implemented separately as individual projects but, collectively, they form part of the overall management system embodied in the Master Plan.

The Study is being undertaken in accordance with Approach #1, as described in Appendix 4 of the MCEA document. An overview of the Municipal Class Environmental Assessment process is provided in **Figure 1-1.** This approach involves the preparation of a Master Plan document upon the completion of Phases 1 and 2 of the process. The Master Plan document is then made available for public comment prior to being approved by the municipality.

The objective of the Master Plan is to identify required projects and their MCEA schedule. Further study and the completion of Phases 3 and 4 are also required to fulfill the requirements for any specific “C” projects identified within the Master Plan itself.

The Master Plan would therefore become the basis for, and be used in support of, any future Schedule “B” and Schedule “C” projects identified within it. Schedule B projects require project-specific investigations and analyses and the filing of the Project File for public review, while Schedule C projects would have to fulfill Phases 3 and 4 of the MCEA process prior to filing an Environmental Study Report (ESR) for public review.

The Town of Collingwood Master Servicing Plan for Water and Sanitary Sewer Systems demonstrates the methodology and rationale for identifying the required Schedule “A”, “A+”, “B” and “C” projects to accommodate and facilitate growth within the Town of Collingwood to the year 2044.

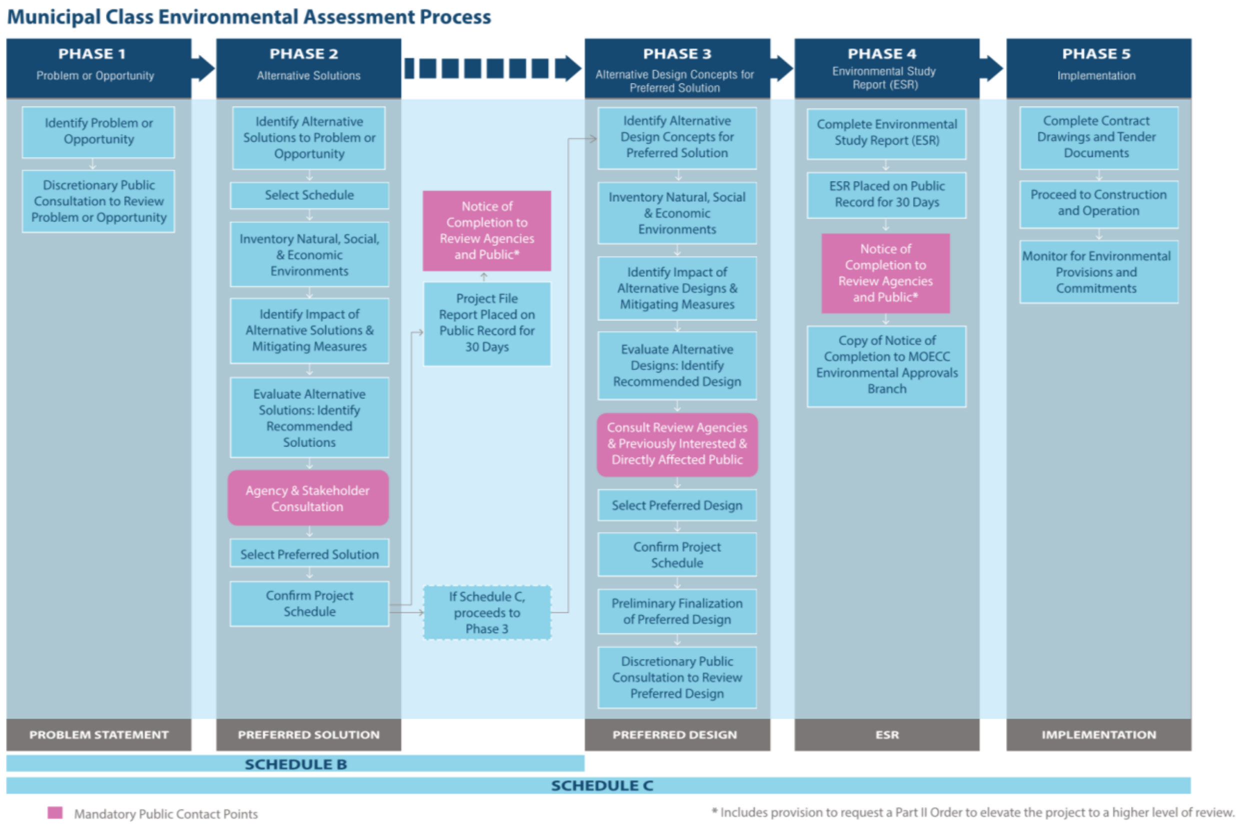


Figure 1-1 Municipal Class Environmental Assessment Process

1.2 Public and Stakeholder Consultation

This Master Plan falls under the requirement of a Schedule B project requiring Phase 1 and Phase 2. One requirement of Phase 2 is the need to consult with review agencies and the public once alternative solutions have been identified. Typically, consultation involves presenting the problem or opportunity that will be addressed, the environmental considerations and potential impacts of each alternative, and the approach used for evaluating the alternatives. The comments and the input from the public and other stakeholders are taken into consideration in the identification of the preferred alternative.

Consultation early and throughout the process is a key feature of environmental assessment planning. The purpose of the consultation process is to notify stakeholders of the project details and provide an opportunity for interested parties to review and submit comments related to the study. The following public and stakeholder consultation activities were completed throughout the Master Planning process. Refer to **Appendix A** for copies of all Notices, stakeholder contact lists and Public Information Centre material. This information is provided in accordance with the standards prescribed by the Class EA document, which outlines the guidelines for establishing contact with appropriate review agencies in relation to the nature of the project.

1.2.1 Notice of Study Commencement

A Notice of Study Commencement was issued to both the Town of Collingwood website, as well as in the local newspaper. Notification was provided through the following means:

- By advertisement in the Collingwood Connection on November 3, 2017 and November 10, 2017;
- By posting to the Town's website on October 10, 2017 and,
- Via e-mail to all agency contacts provided in the project contact list (**Appendix A**).

As a result, all relevant review agencies, indigenous communities, and the public were notified of the project being initiated, the problem and opportunity being addressed, and given the opportunity to provide comments.

1.2.2 Notice of Public Information Centre

A Notice of Public Information Centre (PIC) was issued through both the Town of Collingwood website, as well as in the newspaper. Notification was provided through the following mediums:

- By advertisement in the Collingwood Connection on March 15, 2019 and March 22, 2019;
- By posting to the Town's website on March 11, 2019; and,
- Via e-mail to all agency contacts provided in the project contact list (**Appendix A**).

As a result, all relevant review agencies and the public were notified of the Public Information Centre being held, the problem and opportunity being addressed, alternatives considered and given the opportunity to provide comments and feedback. A record of the Notice of Public Information Centre is located in **Appendix A**.

1.2.3 Public Information Centre

A Public Information Centre was held on March 27, 2019, at the Town of Collingwood Public Library (3rd Floor) at 55 Ste. Marie Street in the Town of Collingwood.

The Public Information Centre was a drop-in, open house format, beginning at 4:00PM and lasting 3-hours. It included a series of display boards describing the project and the Master Plan process. During this time, Town staff, as well as members of the consultant team, were in attendance to discuss the Master Plan and address any questions from community residents.

The general purpose of the Public Information Centre was to present the findings of the Master Plan by providing the following information:

- Scope of the Master Plan process;
- Class EA Master Plan process;
- Growth projections;
- Review of the Water System and alternatives for servicing future growth;
- Review of the Sanitary System and alternatives for servicing future growth;
- Recommendations from the Master Plans;
- An overview of the Master Plan process time line and next steps; and,
- An opportunity for residents to provide comments and feedback on the Master Plan updates.

A total of 15 community members attended the Public Information Centre. In total, three comment sheets were received and one email communication was received following the meeting. A record of these documents is provided in **Appendix A.**

1.2.4 Agency and Public Comments and Responses

Table 1.1 presents a summary of comments received through consultation with regulatory agencies and members of the public during the Master Plan process. A copy of all notices and comments received during the study are provided in **Appendix A**.

Table 1.1 Summary of Comments

Comment Source	Comment	Response
Member of the Public	I am concerned that there is no disclosure of costs. Water efficiency measures will directly impact citizens but no detail was provided.	Additional detail is provided in the Master Plan document. (Section 8.2)
Member of the Public	The defunct MacDonald Road ethanol treatment plant lagoons were significant sources of odours affecting the enjoyment of property for nearby residents. I do not want to experience this again. The OMB decision regarding the disposition of the MacDonald property warrants review. Nearby residential areas are perilously close to MECP restrictions on this odour causing industry.	The MacDonald Road ethanol treatment plant was considered as an alternative to providing wastewater treatment for the Town. This alternative was not selected as the recommended preferred alternative in this Master Plan. Review of the OMB decision is outside of the scope of this Master Plan.
Member of the Public	I am strongly opposed to any use of the old ethanol property use for sewage treatment as it will produce foul odours.	The MacDonald Road ethanol treatment plant was considered as an alternative to providing wastewater treatment for the Town. This alternative was not selected as the recommended preferred alternative in this Master Plan. Review of the OMB decision is outside of the scope of this Master Plan.
Member of the Public	How was the Fire Underwriter Survey (FUS) method accounted for when updating/analyzing the Town's water system model?	The Town's Standards were used to develop fire flow criteria in the Master Plan analysis and water modelling. The maximum fire flow for the purposes of this study was considered to be 189L/s as per the Town's current standards. The FUS method is used to calculate fire flow requirements for specific buildings when detailed information is available through development applications, but is not a feasible method for system wide planning. The current standards will be updated through a separate assignment to clarify the role of FUS when determining design flow.

Table 1.1 Summary of Comments

Comment Source	Comment	Response
Member of the Public	Nottawa needs servicing and the study should ensure that there is adequate capacity for Clearview Nottawa and Batteaux.	The Master Plan presents the development and evaluation of servicing options for servicing of the Nottawa community through Collingwood and identifies a preferred alternative. The Master Plan recommends that servicing of Nottawa be considered in an EA addendum for expansion of the Collingwood WWTP.
Member of the Public	Concerned that Master Plan does not include unserviced area wastewater allocations and servicing strategies.	This Master Plan presents servicing strategies for five areas which currently have water servicing and private wastewater servicing. For each area, options have been developed and cost estimates prepared. This Master Plan has identified a preferred option and/or strategy for servicing each area.
Town of Blue Mountains	Town of Blue Mountains is currently undertaking a Water Distribution Master Plan EA and requests that the Town of Collingwood consider the water supply and distribution implication of providing water servicing capacity of 16,400 m^3/d to the Town of Blue Mountains.	This Master Plan presents the water supply and water distribution implications of provide a water supply of 16,400m^3/d to the Town of Blue Mountains.
Member of the Public	Request that the Town consider servicing of the Princeton Shores area in the Master Plan.	This Master Plan presents servicing strategies for five areas which currently have water servicing and private wastewater servicing. Princeton Shores is one of these areas. For each area, options have been developed and cost estimates prepared. This Master Plan has identified a preferred option and/or strategy for servicing each area.

1.2.5 Notice of Study Completion

A Notice of Study Completion will be prepared and issued following approval of this Master Plan by Town Council.

1.3 Master Plan Study Timeline

A review of key milestones and timelines to the Collingwood Master Plan Study for Water and Sanitary Sewer Systems is shown in **Table 1.2**.

Table 1.2 Master Plan Study Milestones and Timelines

Milestone	Timeline	Description
Notice of Study Commencement	October 2017	A Notice of Study Commencement was issued on October 10, 2017. The Notice reviewed the purpose of the study and the study process. Contact information for the Town of Collingwood Project Manager was provided.
Notice of PIC	March 2019	A Notice of Public Information Centre was issued on March 11, 2019. The Notice identified the location, time, and purpose of the PIC. Contact information for the Town of Collingwood Project Manager was provided.
Public Information Centre (PIC)	March 27, 2019	A Public Information Centre was held on March 27, 2019 at the Collingwood Public Library (55 Ste. Marie Street). The PIC was 3-hours in length, from 4:00PM to 7:00PM.
Notice of Study Completion	December 19, 2019	A Notice of Study Completion will be issued on December 2019.
Master Plan Endorsement	January 27,2020	Subject to endorsement by Collingwood Town Council.

2 Background and Context

2.1 Study Area

The Study Area for this Master Plan includes the entire municipal jurisdiction as shown in **Figure 2-1.**

2.1.1 Town of Collingwood

The Town of Collingwood is situated along the shoreline of Nottawasaga Bay (Georgian Bay) in the extreme northwest corner of the County of Simcoe. In 1994, the Town's boundaries were expanded as a result of municipal restructuring initiated by the County of Simcoe, raising the municipality's total area to approximately 3,300ha. The 2016 census estimated the Town's population as 21,793 persons.

Collingwood is situated approximately midway between the Cities of Barrie and Owen Sound on Highway 26, which provides access to Grey and Bruce Counties in the west and to Toronto, via Highway 400, in the southeast. County Road 124 (formerly Highway 24) originates in Collingwood and provides access to the heavily populated areas of the Greater Golden Horseshoe (GGH) to the south.

This Master Plan has been developed to facilitate Collingwood's current and projected growth to ensure that sufficient water and wastewater servicing can be provided to support this growth to the year 2044.

2.1.2 Town of Collingwood Demographic Statistics

According the 2016 Census Profile (Statistics Canada, 2017), the Town boundary encompasses nearly 33.78 square kilometers and as of the 2016 census, has a population of 21,793. This represents a growth rate of 13.3% from the 2011 census. The median age of residents in 2016 was 49.2 years, with those aged 15 to 64 years representing approximately 59% of the total population. The median total income of households in 2015 was $62,671, with a total labour force aged 15 years and older of 10,055 employees, representing an employment rate of 55.8%. English is the predominant mother tongue, spoken by 99% of the population (Source – Statistics Canada. 2016 Census)

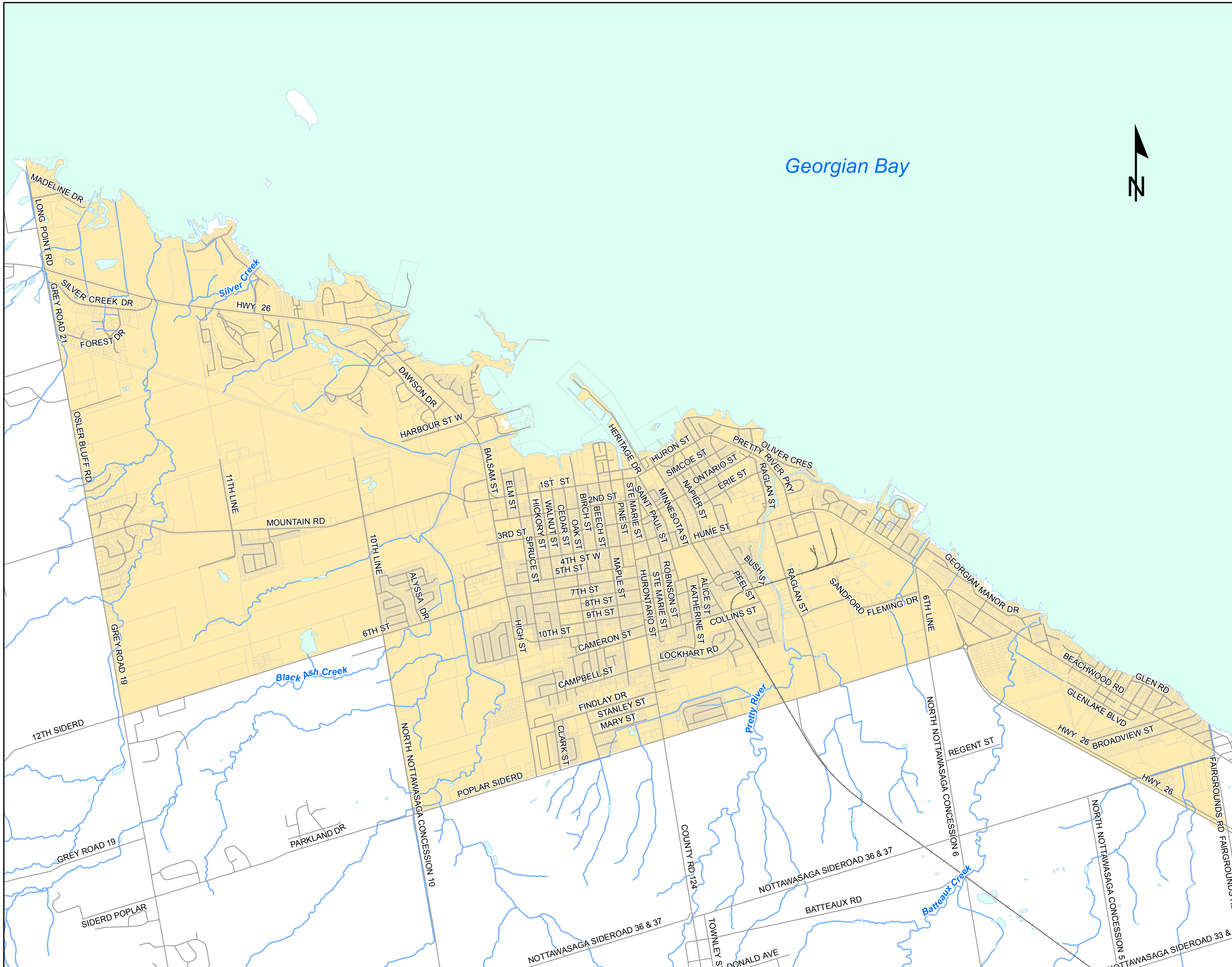
2.2 Regulatory Framework

A fundamental purpose of updating the Master Plan is to comply with and meet regulatory requirements. These include various acts, regulations, guidelines and policies that govern water and wastewater supply, collection and treatment, as well as the pattern of development for which these systems will be expanded to service. Several of the key regulatory requirements impacting the Master Plan update are reviewed in the following sections.

2.2.1 Safe Drinking Water Act, 2002

The *Safe Drinking Water Act, 2002* provides the legislative framework for municipal drinking water systems. It establishes a set of province-wide standards, rules and regulations to ensure the population has access to safe and reliable drinking water. The Act specifies requirements for drinking water systems, testing services and the certification of system operators and water quality analysts and includes regulatory water quality standards and mechanisms for compliance.

Figure 2.1
Town of Collingwood
Study Area



2.2.2 Provincial Policy Statement, 2014

The Provincial Policy Statement (PPS), 2014, is issued by the Province from time to time under the authority of Section 3 of the *Planning Act*. The PPS contains provides policy direction on matters relating to land use planning and development and applies to any land use planning decisions made under the *Planning Act* by municipal councils, local boards, planning boards, provincial ministers, provincial government and agency officials, including the Ontario Municipal Board. Municipal planning decisions are to be consistent with the policies of the PPS.

The PPS includes policies relevant to water and wastewater infrastructure planning including the requirement that infrastructure be provided in a coordinated, efficient and cost-effective manner. Additional requirements under the 2014 PPS include:

- Water and wastewater systems are to be sustainable, feasible, financially viable and comply with all regulatory requirements, as well as protect human health and the natural environment (Section 1.6.6.1.b); and,
- That water and wastewater infrastructure will be integrated at all stages of land use planning and implementation processes (Section 1.6.6.1.d).

The 2014 PPS also states that settlement areas will be serviced by municipal water and wastewater systems, with intensification and redevelopment within these areas provided by municipal water services wherever feasible (Section 1.6.6.2).

2.2.3 Growth Plan for Greater Golden Horseshoe, 2017

The Growth Plan for the Greater Golden Horseshoe (the "Growth Plan") 2017, developed pursuant to the *Places to Grow Act, 2005*, and as an update to the Growth Plan for the Greater Golden Horseshoe, 2006, is a framework for implementing the Province's vision for building stronger, prosperous communities by better managing growth. The Master Plan includes policies for the provision of well-planned infrastructure and strategic investment decisions to support forecasted population and economic growth.

The Growth Plan establishes that municipal water and wastewater systems will be planned, designed, constructed or expanded through a comprehensive water or wastewater master plan, informed by watershed planning that takes into consideration the following:

- That effluent discharge will not negatively impact the quality and quantity of water (Section 3.2.6.3.c.i);
- That the preferred option for servicing growth and development will not exceed the assimilative capacity of the effluent receivers and sustainable water supply for servicing, ecological, and other needs (Section 3.2.6.3.c.ii); and,
- That the full life cycle costs of the system can be sustained over the long-term (Section 3.2.6.3.c.ii).

The Growth Plan further requires that municipalities that share an inland water source or receiving water body will co-ordinate their planning for potable water, stormwater, and wastewater systems based on watershed planning to ensure the quality and quantity of water is protected, improved, or restored (Section 3.2.4.6).

2.2.4 Town of Collingwood Official Plan (January 2018 Office Consolidation)

The Town of Collingwood Official Plan (the "Official Plan") provides direction for managing growth and change within the Town. The Town's Official Plan recognizes the role that the Province of Ontario and Simcoe County play in the local planning process. This includes the consideration of land use change, the provision of public works, and the responsibilities of local boards, the municipality, and the actions of private enterprises. Official Plan policies have been developed to be in accordance with Provincial Long Range Land Use interests, PPS principles, the Growth Plan for the Greater Golden Horseshoe policies and the goals of the County of Simcoe Official Plan. These interests, principles, policies and goals are identified below:

- Province of Ontario Long Range Land Use interests:
    - The protection of ecological systems, including natural areas, features and functions;
    - The conservation of features of significant architectural, cultural, historical, archaeological, or scientific interest;
    - The adequate provision and efficient use of sewage and water services and waste management systems;
    - The orderly development of safe and healthy communities;
    - Accessibility for persons with disabilities to all facilities and services;
    - Adequate provision and distribution of educational, health, social, cultural and recreational facilities;
    - The adequate provision of a full range of housing and employment opportunities;
    - The protection of the economic well-being of the Province and its municipalities;
    - The appropriate location of development; and
    - The promotion of development that is designed to be sustainable to support public transit and to be oriented to pedestrians.
- PPS principles:
    - Promote efficient development and land use patterns which sustain the financial wellbeing of the municipality over the long term;
    - Accommodate an appropriate range and mix of residential, employment (industrial, commercial and institutional land uses), recreation and open space uses to meet long term needs;
    - Avoid development and land use patterns which may cause environmental, public health and/or safety concerns.
- Growth Plan for Greater Golden Horseshoe Policies:
    - Build compact, vibrant and complete communities;
    - Plan and manage growth to support a strong and competitive economy;
    - Protect, conserve, enhance and wisely use the valuable natural resources of land, air and water for current and future generations;
    - Optimize the use of existing and new infrastructure to support growth in a compact efficient form;
    - Provide for different approaches to managing growth that recognize the diversity of communities in the Greater Golden Horseshoe;
    - Promote collaboration among all sectors – government, private and non-profit and residents to achieve the vision.

- Simcoe County Official Plan Goals:
  - To protect, conserve and enhance the County's natural and cultural heritage;
  - To achieve wise management and use of the County's resources;
  - To implement growth management to achieve lifestyle quality and efficient and cost effective municipal servicing, development and land use;
  - To achieve coordinated land use planning among the County's local municipalities and with neighbouring counties and First Nations lands;
  - To further community economic development which promotes economic sustainability in Simcoe County Communities, providing enjoyment and business opportunities; and
  - To promote, protect and enhance public health and safety.

Municipal servicing policies are identified based on the goal of providing adequate and sufficient systems of water supply, sanitary sewerage disposal and storm drainage to all areas of development in the municipality in accordance with the staging program established by the Official Plan and sound financial planning. Specific servicing objectives are as follows:

- To optimize the opportunity for provision of full municipal water and sewage services in new development areas;
- To encourage progressive, staged development from existing built-up areas in order to minimize the need for major servicing extensions;
- To encourage the substantial completion (more than 50%) in one neighbourhood or development area prior to initiating developments in an adjacent neighbourhood and thus minimize leapfrogging and scattered development;
- To develop new municipal services and undertake improvements to existing servicing infrastructure bearing in mind the ultimate servicing requirements of the existing municipality, and the municipality's ability to finance such projections; and
- To develop a system of storm drainage sympathetic to areas of environmental sensitivity including the Town's natural heritage features and hazard lands.

Furthermore, the Town has identified that the expansion of existing municipal services should only be considered when:

- Strategies for water conservation and other water demand management initiatives are being implemented in the existing service area; and
- Plans for expansions are to serve growth in a manner that supports achievement of the intensification target and density targets.

The Town's Official Plan has designated a number of service areas. Service Area 1 includes lands that are fully serviced and within the built boundary with some minor adjacent pre-designated lands outside of the built boundary. Service Area 2 are vacant lands adjacent to Service Area 1 lands and represent the areas beyond the built boundary to which municipal services can be easily and efficiently be extended. Service Area 3 lands includes portions of the Mountain Road West corridor where new development is constrained by the availability of partial or private services and significant improvements are needed. Service Area 4 is the Highway 26 East Corridor where future servicing options are problematic due to complexity and cost.

3 Future Growth

The Greater Golden Horseshoe Growth Plan defined residential and employment population targets for the Town of Collingwood to the year 2031. **Table 3.1** presents these target values. If we assume uniform growth in each year between 2016 and 2031, the population target represents residential growth of 774 persons per year.

Table 3.1 Town of Collingwood Growth Targets

Year	Residential Population	Employment Population
2016	21,793	10,055
2031	33,400	13,500

To better define where growth will occur, the Town provided information on developments that are currently planned, identified as Planned Development, as well as lands that have been identified as ready for development, identified as Potential Development. A third category of growth was equated to the Town's built boundary. The following sections provide additional information.

3.1 Planned Development

The Town has identified a number of planned developments located within the Town's built boundary.

Figure 3-1 presents the location of Planned Developments as well as the location of Potential Developments. **Table 3.2** presents the name, land use, area, anticipated units, area of any non-residential or industrial, commercial and institutional (I/C/I) development and the estimated growth populations. A total of 41 Planned Developments have been identified. Land use for these developments ranges from community services to residential to ICI. To estimate population, persons per unit values of 1.9, 2.4 and 2.9 have been used for apartment / condo units, semi-detached units and single family detached units. Each planned development has been assigned an ID consisting of a number followed by the designation PLANNED. For sanitary modelling purposes, each development location was assigned a discharge sewer/ location in the sanitary sewer system. **Appendix B** contains additional information on the downstream sanitary sewers which would receive flows from planned developments.

Completion of all of the above planned developments would result in a new residential growth population of 12,366 or a total estimated residential population of 34,159 persons. Based on the residential growth rate of 774 persons per year, calculated based on the Places to Grow 2031 Population target, it is estimated that planned developments could be completed by 2032.

Figure 3.1
Town of Collingwood
Location of Planned and Potential Developments

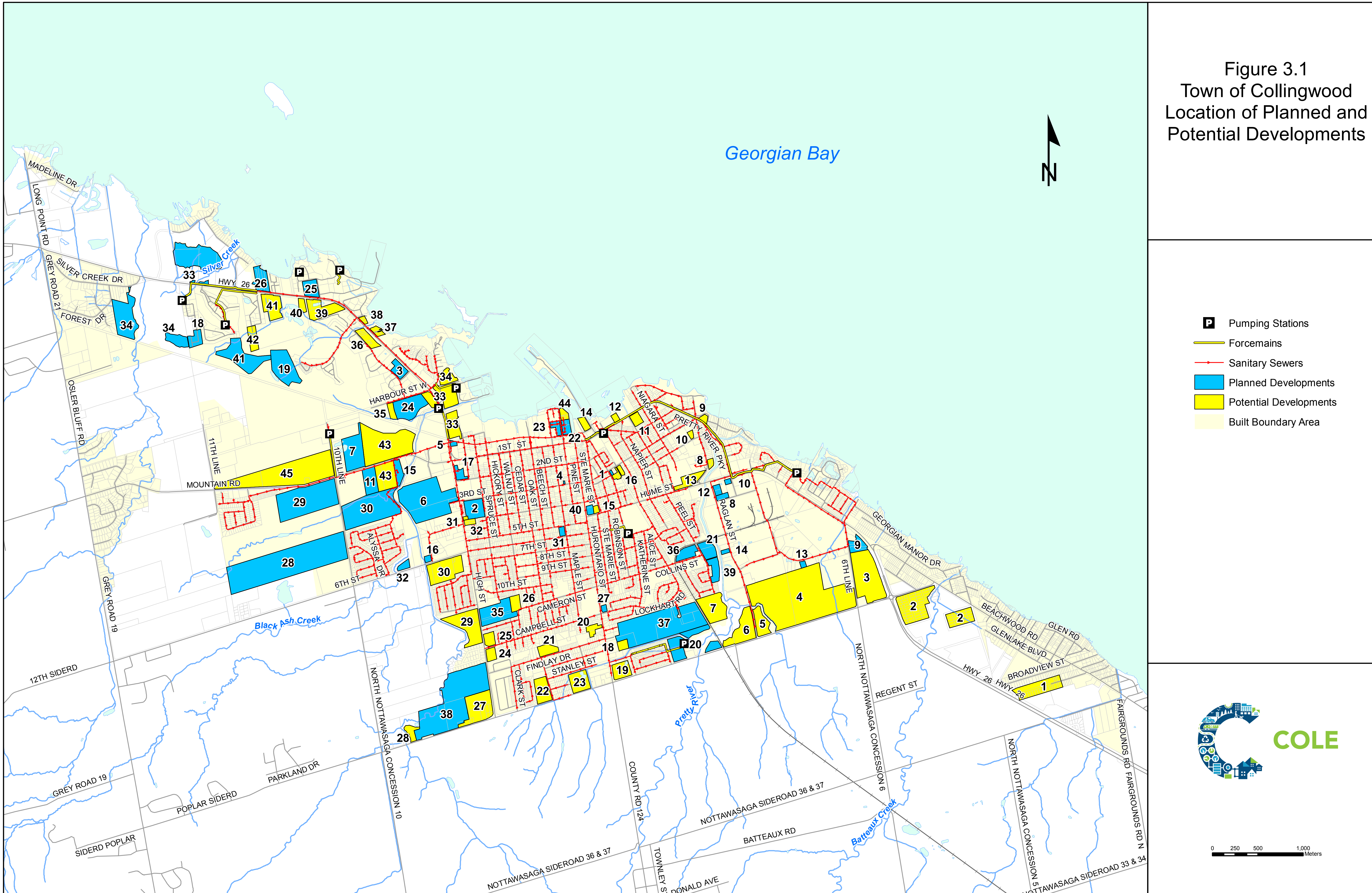


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Table 3.2 Planned Developments

ID (Status)	Name	Land Use	Area (Ha)	Number of Residential Units	ICI Development (m^2)	Estimated Residential Population or Equivalent Residential Population
1-PLANNED	Ambulance Station Expansion	Community Services	0.15			-
2-PLANNED	Mountainview Public School Expansion	Community Services	4.11			-
3-PLANNED	Cranberry Inn Extension	Commercial	2.20			-
4-PLANNED	75 Third Street	Commercial	0.06			-
5-PLANNED	10 Balsam Commercial Plaza	Commercial	0.40			-
6-PLANNED	Regional commercial district	Commercial	21.07			-
7-PLANNED	Van Dolder's Subdivision	Industrial	8.09			-
8-PLANNED	Ace Cabs.	Industrial	0.78			-
9-PLANNED	BMC Automotive	Industrial	2.50			-
10-PLANNED	Collingwood Service Station	Industrial	0.38			-
11-PLANNED	Georgian Bay Biomed	Industrial	4.00			-
12-PLANNED	Dunn Hotel	Commercial	0.88			-
13-PLANNED	Isowater	Industrial	0.41			-
14-PLANNED	360 Raglan	Industrial	0.40			-
15-PLANNED	100 Mountain Road	Industrial	2.12			-
16-PLANNED	Stewart Road Reservoir	Other	0.50			-
17-PLANNED	Affordable Housing Project	Residential	1.32	147 apartments		279
18-PLANNED	Silver Glen	Residential	2.27	50 Towns		120
19-PLANNED	Blue Fairways	Residential	8.49	262 Towns		629
20-PLANNED	Pretty River Estates Phase 2	Residential	7.19	21 Singles and Semis 152 Towns		426

Table 3.2 Planned Developments

ID (Status)	Name	Land Use	Area (Ha)	Number of Residential Units	ICI Development (m^2)	Estimated Residential Population or Equivalent Residential Population
21-PLANNED	Riverside Midrise	Residential	2.85	156 Towns		374
22-PLANNED	Shipyards Condo E	Residential	1.48	28 Towns		67
23-PLANNED	Mackinaw Village	Residential	1.21	28 Towns		67
24-PLANNED	Balmoral	Residential	6.95	54 Semis, 199 towns	2,800m^2	733
25-PLANNED	Harhay	Residential	2.81	154 Towns		370
26-PLANNED	Wyldewood Cove	Residential	3.60	177 Towns		425
27-PLANNED	655 Hurontario Street Apartments	Residential	0.42	32 Apartments		77
28-PLANNED	Linksview	Residential	40.68	439 single family, 8 towns, 190 apartments	School	1653
29-PLANNED	Mair Mill Villages	Residential	19.70	192 apartments and 127 single family		733
30-PLANNED	Red Maple	Residential	17.89	131 Singles and Semis 147 Towns		733
31-PLANNED	Victoria Annex	Residential	0.60	19 Towns		46
32-PLANNED	Georgian Meadows	Residential	1.01	25 Towns		60
33-PLANNED	The Preserve at Georgian Bay	Residential	12.26	75 Singles and Semis 249 Towns		815
34-PLANNED	Huntingwood –	Residential	11.82	92 Singles and Semis 62 Towns		416
35-PLANNED	Helen Court Homes	Residential	7.56	66 Singles and Semis 189 Towns		645
36-PLANNED	Riverside Townhomes	Residential	2.54	57 Towns		137

Table 3.2 Planned Developments

ID (Status)	Name	Land Use	Area (Ha)	Number of Residential Units	ICI Development (m^2)	Estimated Residential Population or Equivalent Residential Population
37-PLANNED	Eden Oak McNabb	Residential	27.00	256 Singles and Semis 120 Towns		1,030
38-PLANNED	Summitview Phases 1 and 2	Residential	31.58	233 Singles and Semis 173 Towns		1,091
39-PLANNED	Harmony Living	Residential	2.45	80 Towns		192
40-PLANNED	Monaco	Residential	0.76	260 condo units (apart.)	2600m^2 commercial	494
41-PLANNED	Cranberry	Residential	9.14	314 Towns		754

3.2 Potential Developments

Figure 3-1 also identifies a total 45 potential developments that could develop in the years beyond 2032. It is important to note that water and sanitary servicing will not be provided to all of these developments. The Braeside Development (1-POTENTIAL) and Batteaux Creek Subdivision (2-POTENTIAL) are anticipated to receive municipal water servicing only from the Town but will receive sanitary servicing through private systems. **Table 3.3** presents the name, land use, area, anticipated units, area of any non-residential or I/C/I development and the estimated growth populations. To estimate population, persons per unit values of 1.9, 2.4 and 2.9 have been used for apartment / condo units, semi-detached units and single family detached units. Each potential development has been assigned an ID consisting of a number followed by the designation POTENTIAL. **Appendix B** contains additional information on the downstream sanitary sewers which would receive flows from potential developments.

Table 3.3 Potential Developments

ID (Status)	Name	Land Use	Area (Ha)	Number of Residential Units	ICI Development	Estimated Residential Population
1-POTENTIAL	Braeside	Residential	7.26	15 – singles		44
2-POTENTIAL	Batteaux Creek Subdivision (Beachwood Estates)	Residential	15.28	20 – singles		58
3-POTENTIAL	2906 Sixth Street and 7026 Poplar Sideroad	Industrial	14.99	-	-	-
4-POTENTIAL	Eden Oaks Industrial	Industrial	50.73	-	-	-
6-POTENTIAL	Poplar and Raglan	Industrial	7.29	-	-	-

Table 3.3 Potential Developments

ID (Status)	Name	Land Use	Area (Ha)	Number of Residential Units	ICI Development	Estimated Residential Population
7-POTENTIAL	King (452 Raglan)	Residential	7.44	57 Singles 205 townhomes (Includes 148 stacked towns)		657
8-POTENTIAL	Memory Care Facility	Hospital	0.61			72
9-POTENTIAL	500 Ontario Street	Residential	0.64	60 Towns		144
10-POTENTIAL	Legion Redevelopment	Residential	0.44			70
11-POTENTIAL	Parkridge	Office	1.40		40,000sqft commercial	-
12-POTENTIAL	Courthouse	Residential	0.57	68 Towns		163
13-POTENTIAL	Hospital	Hospital	3.00			-
14-POTENTIAL	Duncap Waterfront hotel	Hotel and Commercial	1.15	80 hotel units (apartments)	2,280sqm commercial	152
15-POTENTIAL	Admirals Village	Residential and Commercial	0.48	70 Towns	1,100sqm commercial	168
16-POTENTIAL	Reinhart Warehouse	Residential	1.19	23 Singles and Semis		68
18-POTENTIAL	Church Severance	Residential	1.16	44 Singles and Semis		128
19-POTENTIAL	Poplar and Hurontario	Highway Commercial	3.26			-
20-POTENTIAL	Blackmoor Gate property	Residential	1.35	34 Singles and Semis		99
21-POTENTIAL	Findlay property	Residential	2.20	22 Singles and Semis		64
22-POTENTIAL	50 Saunders Drive	Residential	4.17	74 Singles and Semis		215
23-POTENTIAL	Old Organic Farm	Residential	4.32	76 Singles and Semis		220
24-POTENTIAL	Collingwood Nursing Home	Residential	1.41	47 Singles and Semis		136
25-POTENTIAL	197 Campbell Street "Saunders"	Residential	1.62	32 Singles and Semis		93
26-POTENTIAL	Property adjacent to Helen Court Homes	Residential	1.84	59 Singles and Semis		171

Table 3.3 Potential Developments

ID (Status)	Name	Land Use	Area (Ha)	Number of Residential Units	ICI Development	Estimated Residential Population
27-POTENTIAL	Summitview Phase 3	Residential	6.89	36 Singles, 52 Semis and 68 Towns		392
28-POTENTIAL	8070 Poplar Sideroad	Residential	1.56	30 Singles and Semis		87
29-POTENTIAL	Fumo Development	Residential	8.86	300 Singles and Semis		870
30-POTENTIAL	580 Sixth Street and adjacent property	Residential	8.42	308 Singles and Semis		893
31-POTENTIAL	115 High Street	Residential	0.21	15 Towns		44
32-POTENTIAL	121 High Street	Residential	0.75	6 Towns		17
33-POTENTIAL	Hotel Development	Commercial	9.63			-
34-POTENTIAL	Living waters	Hotel	2.34	253 Towns		481
35-POTENTIAL	16 Harbour Street or Law property	Residential	1.18	23 Singles and Semis		68
36-POTENTIAL	Dawson Drive East property	Residential	2.46	48 Singles and Semis		141
37-POTENTIAL	White Street property	Residential	1.02	20 Singles and Semis		58
38-POTENTIAL	#38F – Gunn Club Road	Residential	0.49	10 Singles and Semis		28
39-POTENTIAL	Rollings property	Residential	5.57	200 Singles and Semis		580
40-POTENTIAL	Griffith's property	Residential	1.02	30 Singles and Semis		87
41-POTENTIAL	Greentree property	Residential	4.93	88 Singles and Semis		281
42-POTENTIAL	Georgian Manor Resorts	Residential	2.49	150 apartments		285
43-POTENTIAL	Mountain Road Industrial property	Industrial	24.16			-
44-POTENTIAL	Huronic Village	Residential	1.0	13 Townhomes		31
45-POTENTIAL	Mair Mills North	Residential	26.6	128 singles, 265 towns, 508 apartments		1,972

Development of all 45 potential developments would increase the residential service population by 9,631 persons, increase ICI development areas by 119ha and increase residential development areas by 126ha. It is important to note that a total residential population of 102 persons located within the Batteaux Creek and Braeside Developments (total area of 22ha) will receive water servicing only. These two subdivisions will receive sanitary servicing through private systems.

Completion of all of the above potential developments would result in a new residential growth population of 9,631 or a total estimated residential population following completion of all planned and potential development 43,790 persons. Based on the residential growth rate of 774 persons per year, calculated based on the Places to Grow 2031 Population target, it is estimated that the potential developments could be completed by 2044.

3.3 Built Boundary

Growth beyond 2044 was considered to be growth up to the Town's built boundary into lands currently designated as rural and not designated as environmental protections areas. The locations of these lands are shown in **Figure 3-2**. These lands have been separated into Areas A, B, F, G1, G2, G3 and G4. To estimate future population within the built boundary lands, a population density of 50 persons per hectare (residents or jobs) was used. **Table 3.4** presents the breakdown of areas and population for each Built Boundary Sub-area.

Table 3.4 Built Boundary Lands

Built Boundary Sub-Area	Estimated Developable Area (ha)	Estimated Future Residential and Employment Population
A	193	9,650
B	97	4,850
F	51	2,550
G1	56	2,800
G2	41	2,050
G3	35	1,750
G4	11	550
Total Built Boundary Area	484	24,200

In total, 484ha of lands have been identified as developable in the period beyond 2044 and up to the built boundary. The total residential and employment population is estimated to be 24,200 persons. It is assumed that 66% of these lands would be developed as residential lands while the remaining 33% would be developed as ICI lands. Based on the Town's residential development density target of 50 persons or jobs/ hectare, buildout of the built boundary would add 16,120 residents and 8,080 employees to the Town's population. With completion of development within the built boundary, the Town of Collingwood's residential population is estimated to reach 59,910 persons. Assuming the same growth rate of 774 persons per year used to estimate the time required for the planned and potential growth horizons to be reached, completion of development within the built boundary could occur by 2064.

3.4 Summary of Growth Projections

Based on the information contained in the previous sections, **Table 3.5** presents the summary of growth projections for the planned, potential and beyond 2044.

Table 3.5 Summary of Growth Projections

Scenario Name	Number of Residential Units	Anticipated Residential Growth Population	Residential Growth Area (ha)	ICI Development Area (ha)
Planned Developments (2032)	4,909	12,366	223.6	48.1
Potential Developments (2044)	3,721	9,631	126.4	119.0
Built Boundary (2064)		16,120	323	161.3

Based on the information shown in **Table 3.5,** completion of all planned developments will increase the Town's serviced residential population by 12,366 persons, completion of all planned and potential developments will increase the Town's residential population by 21,997 persons. Completion of all developments up to the built boundary will increase the Town's residential population by 38,117 persons.

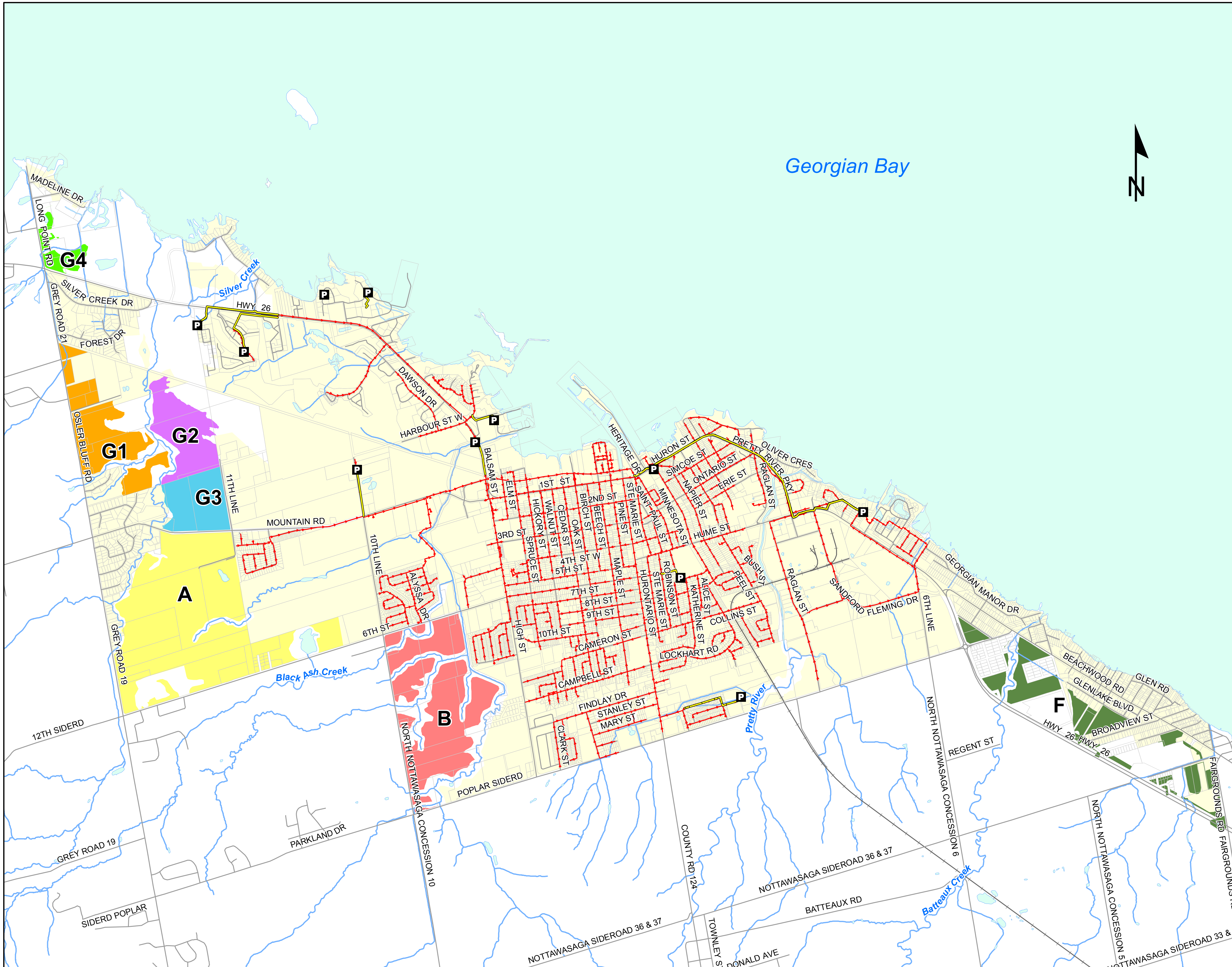


Figure 3.2
Town of Collingwood
Location of Built
Boundary Development

P Pumping Stations
Forcemains
Sanitary Sewers

Development Areas

- A
- B
- F
- G1
- G2
- G3
- G4

COLE

0 250 500 1,000 Meters

4 Study Area Profile

The following section provides additional information on the natural and social environment as well as a description of the existing water and sanitary sewer system.

4.1 Natural and Social Environment Description

The natural and social environments include natural heritage, water resources and land use. The following sections provide information on key features.

4.1.1 Natural Heritage

The Nottawasaga Valley Conservation Authority completed the Town of Collingwood Natural Heritage System Study in 2011 and a peer review was completed in 2012. The results were used to develop natural heritage policies in the Town's Official Plan. **Figure 4-1** presents the location of lands designated for environmental protection – natural heritage resources.

Lands designated for protection due to natural heritage resources include valleylands, wetlands, woodlots and fish spawning and nursery habitat areas. The Town's Natural Heritage system includes the following classifications:

- Category 1 lands are lands where development is prohibited. Category 1 lands are included within the Environmental Protection Areas designation to provide a heightened level of protection to Collingwood's most sensitive natural resources. Category 1 lands are those considered to make the greatest contribution to the natural heritage system of the Town and include Provincially significant wetlands, major river valleys, fish habitat located within significant valley lands and primary woodlands encompassing in excess of 4ha that are more than 75 years old.
- Category 2 lands encompass locally significant wetlands, younger woodland encompassing an area in excess of 10ha and/or fish habitat located outside significant valley lands. Category 2 lands are where limited forms of development, in accordance with the land use designations may be possible subject to the findings of an Environmental Impact Statement.

4.1.2 Soils

The soils within the Town of Collingwood are an important consideration for the construction of new infrastructure. Based on the Soils Map of Simcoe County, Soil Survey Report No. 29, the Town's boundary predominantly contains the following soils classifications: Kemble clay loam shallow (downtown area of Collingwood), Sargent gravelly sandy loam (south east portion of Town), Alliston sandy loam (south), Smithfield silty clay loam (south west part of Town) and Kemble clay loam (west part of Town). These soil types have good drainage characteristics.

Conventional construction techniques may be used for the majority of soils identified within the Town. For areas of where deep sewers are to be constructed and where groundwater percolation is evident, dewatering may be required. In addition, it is noted that the downtown area of Collingwood has shallow soils indicating that deep excavations are likely to be in bedrock.

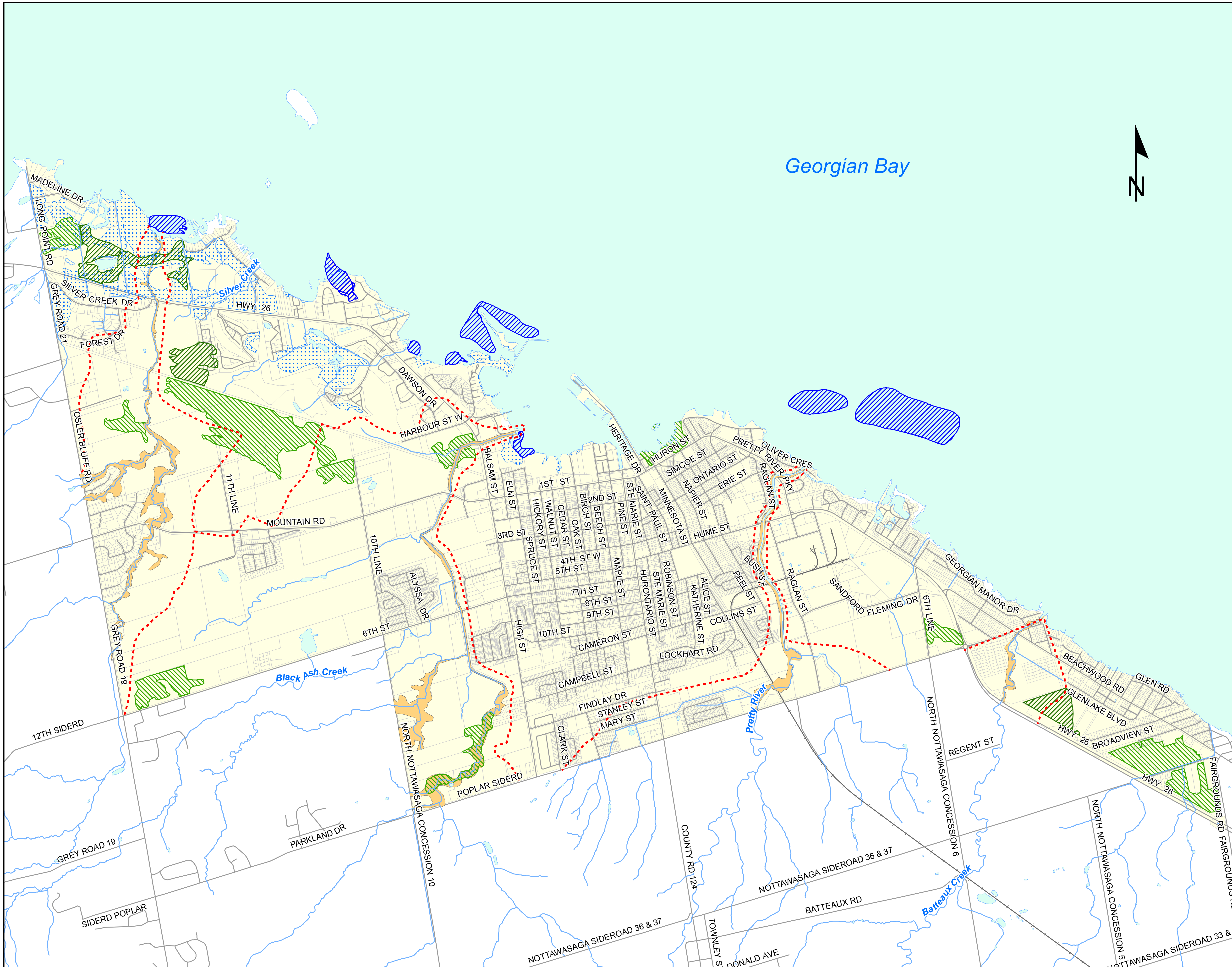


Figure 4.1
Town of Collingwood
Natural Heritage Areas

Schedule B - Natural Features

- Category 1 Valleylands
- Category 1 Wetlands
- Category 1 Woodland
- Category 2 Woodland
- Fish Spawning Nursery Habitat
- Watershed Boundary

COLE

0 250 500 1,000 Meters

4.1.3 Watercourses and Water Features

Watercourses within the Town include the Pretty River, Black Ash Creek, Silver Creek and Batteaux Creek. All of these watercourses are part of the Blue Mountain Subwatershed. These creeks and rivers are characterized by headwaters in the Niagara Escarpment in the Blue Mountains with mountain streams that transition into rolling hills with meadows to very flat plains and outlet to Georgian Bay. The Pretty River main branch passes through the east part of the Town. The River is confined by a series of dikes within Collingwood. Batteaux Creek passes through the eastern part of the Town. Black Ash Creek passes through the western part of the Town. Within Collingwood, Black Ash Creek flows through a flood control channel and discharges into Georgian Bay. Silver Creek is outside the urban area in the western part of Town. **Figure 4-1** presents the location of watercourses and water features.

4.1.4 Topography

The topography of the Town of Collingwood is generally rolling hills with the land generally sloping from an elevation of approximately 225m at the southwest boundary of the Town to elevations of 175m at the Georgian Bay shoreline in the northeastern part of the Town.

4.1.5 Archaeological and Cultural Heritage

One of the Town of Collingwood's goals is the conservation of Collingwood's cultural heritage. To meet this goal, the Town has developed policies and guidelines governing the preservation of significant archaeological and built cultural heritage landscapes. The Town requires that an archaeological assessment be completed as part of major new public work or private development projects and that where resources warrant conservation, mitigation techniques be developed and incorporated into the projects. For cultural heritage, the Town is working towards developing an inventory of built cultural heritage resources. For public works projects, the Town's policy is to ensure that the design of the works provides for mitigation of negative impacts.

4.2 Existing Water System

The Town of Collingwood takes water from Nottawasaga Bay in Lake Huron where it is treated at the Raymond A. Baker Water Ultra-filtration Treatment Plant (WTP) by membrane filtration and chlorine disinfection. The WTP has a design capacity of 31,140m^3/day as per the Municipal Drinking Water Licence 100-101. In 2017, the maximum demand day was 21,143m^3 and the average daily demand was 17,658m^3. Treated water is pumped from the WTP's clearwell to Collingwood water distribution system (Zone 1) by high lift pumps, and to the Town of New Tecumseth (ToNT) by dedicated pumps at the WTP through a dedicated Regional Pipeline. The Town operates under Ontario Drinking Water Works Permit 100-201 and Permit to Take Water (PTTW) 3451-8CZMJC issued in 2011. The existing PTTW allocates 68,250m^3/day of water taking from the Nottawasaga Bay.

The Collingwood Elevated Water Tower (ET) near the centre of the Town is fed by the Zone 1 high lift pumps and supplies storage for Zone 1. On the West side of Zone 1, the A.R. (Ted) Carmichael West End in-ground Reservoir and Booster Pumping Station (Carmichael BPS) supplies Zone 1 to the east towards the elevated tank and Zone 1 to the west towards Osler Bluff Road Booster Station (Osler Bluff BPS). The reservoir is fed or drained by a single watermain and as such cannot currently be operated as an in-and-out reservoir. However, designs are underway to upgrade the reservoir for this capability. The reservoir is currently filled at night from Zone 1 via a hydraulic flow control valve, and during the day it is operated as an "out" reservoir via its three pumps. The pumps are operated based on Zone 1 ET water levels. When not in operation, water is allowed to bypass the pump station to supply the west side of Zone 1.

The pressure zone referred to as Zone 2 in the Master Plan is currently divided into three separate areas; Osler Bluff, Georgian Meadows, and Davey service areas. In the future, these areas are expected to be connected to create a single Zone 2. The Osler Bluff service area is located at the western boundary of the system and is maintained by a small in-ground Osler Bluff BPS on Osler Bluff Road. The Georgian Meadows BPS currently supplies the Georgian Meadows development from the south-western portion of Zone 1, and is considered to be a temporary BPS. The service area fed by the Bob Davey (South Collingwood) Reservoir and Booster Pumping Station (Davey BPS) is currently the largest portion of Zone 2 and includes properties between Campbell Street and Poplar Sideroad. The Davey BPS is supplied directly from the WTP via a connection and flow control valve from the Regional Pipeline. The reservoir can be bypassed so that Zone 2 is fed directly from the Regional Pipeline, however; this operation is not typical. The Davey service area of Zone 2 can also be fed directly from Zone 1 through an in-field PRV; this operation is not typical.

The Town of Collingwood supplies two neighbouring municipalities with drinking water; the Town of Blue Mountains (ToBM) and ToNT. At the western boundary of the Osler Bluff service area of Zone 2, the ToBM owns and operates the Mountain Road Booster Pumping Station, which meters and pumps water to the ToBM.

The Town's water distribution system consists of approximately 160 km of watermains with diameters ranging from 50mm to 600mm. **Figure 4-2** presents the location of the water system and **Figure 5-3** shows the location in relation to each pressure zone. **Table 4.1** presents a summary of the four pumping stations. **Table 4.2** presents information on water storage.

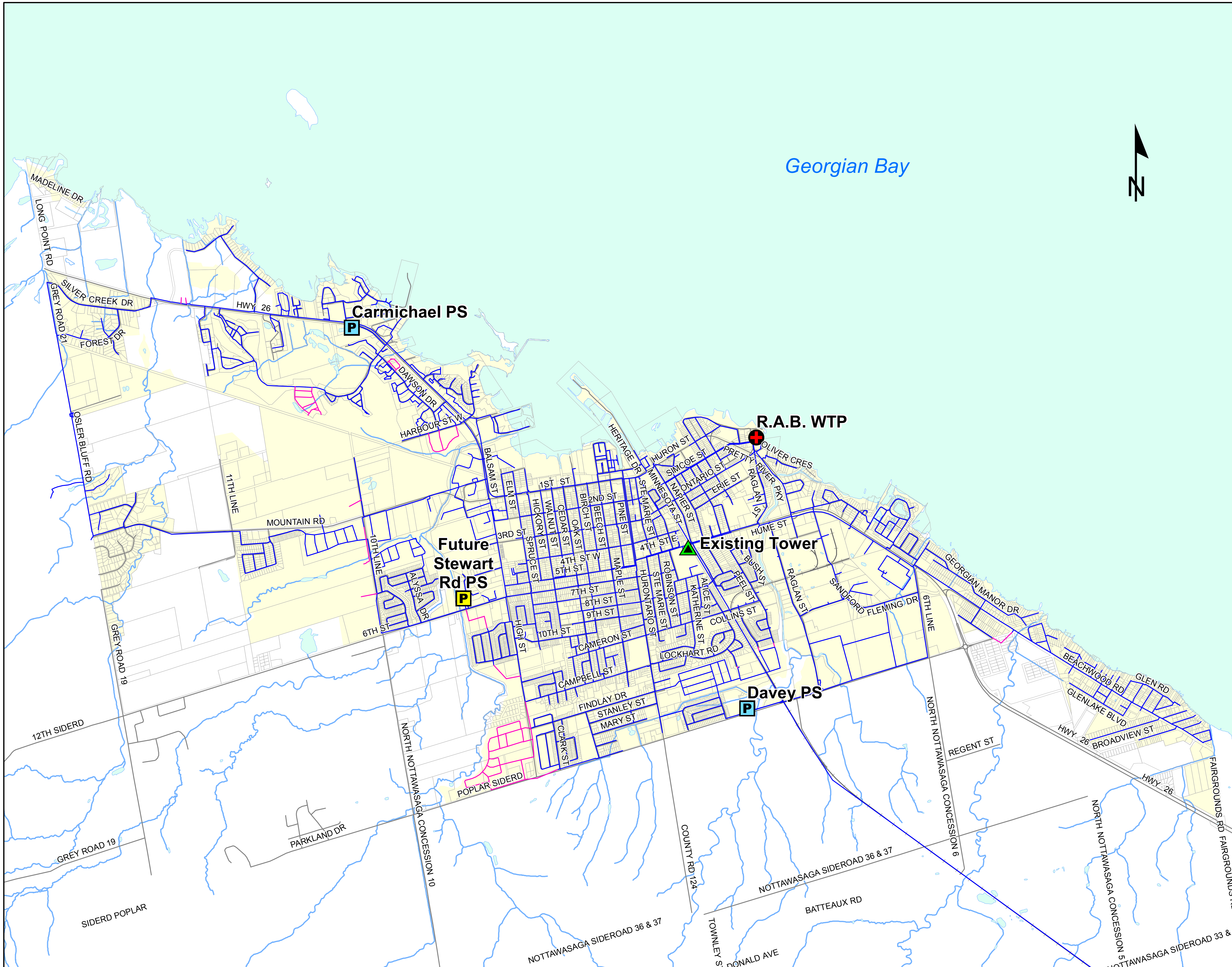
Table 4.1 Water Pumping Stations

Pumping Station	Pumping Capacity (L/s)	Firm Capacity (L/s)	Standby Power	Zone	Re-chlorination
Carmichael BPS	500	300	Yes	Zone 1	Yes
Davey BPS	264	172	Yes	Zone 2	Yes
Osler Bluff BPS	131.7	87.8	Yes	Zone 2	No
Georgian Meadows BPS	21.3	11.8	No	Zone 2	No

Table 4.2 Water Storage

Storage	Capacity (m^3)	Type	Zone
Collingwood Elevated Tower	2,273	Elevated Tower	1
Carmichael Reservoir	6,800	In Ground	1
Davey Reservoir	2,565	In Ground	2
Total	11,638		

Figure 4.2
Town of Collingwood
Existing and Planned
Water Service Locations



Existing Water Pipes
Planned Water Pipes

Water Services

Existing Tower
Future Pumping Station
Pumping Station
Water Treatment Plant
Built Boundary Area

COLE
C3 WATER

0 250 500 1,000 1,500 2,000 Meters

An important component of the Master Plan development was the use of hydraulic models to understand the behaviour of the existing water distribution system and how it will behave under future development scenarios. The Town's existing water distribution model was reviewed in detail as part of this study and recommendations and updates were made to the model so that it better met the requirements of this project. **Appendix C** contains further information on the review of the existing Town of Collingwood Water Model.

4.3 Existing Sanitary System

The following sections provide detailed information on the Collingwood Wastewater Treatment Plant (WWTP) as well as the sanitary sewers, maintenance holes, pumping stations and forcemains, which are collectively discussed.

The Collingwood WWTP is located at 3 Birch Street and has a rated capacity of 24,548m^3/d and a peak flow capacity of 60,900m^3/d (peak factor of 2.5). The facility operates under existing MECP ECA 5807-B8GM4G, dated January 31, 2019.

Table 4.3 presents key information on liquid unit processes and their current ECA capacities. **Figure 4-3** presents the location of the sanitary system.

The plant is also equipped with a raw sewage bypass at the inlet channel of the treatment plant and a 900mm diameter outfall sewer. The outfall discharges into Georgian Bay. The ECA defines a bypass to be a diversion of sewage around one or more treatment processes, excluding preliminary treatment with bypass flows returned to the plant prior to the final effluent sampling point. The ECA identifies that bypasses are prohibited except under the following circumstances:

- When a structural, mechanical or electrical failure causes a temporary reduction in the capacity of a treatment process or when an unforeseen flow condition exceeds the design capacity of a treatment process that is likely to result in personal injury, loss of life, health hazard, basement flooding, severe property damage, or treatment process upset, if a portion of the flow is not bypassed.
- A planned bypass that is the direct and unavoidable result of a planned repair and/ or maintenance procedure.

The ECA requires notification of the Spills Action Centre and information be reported on the type of bypass, date and time of bypass, identification of treatment processes that the bypassed volume has received, and what efforts were completed to maximize the flow receiving treatment.

Table 4.3 Collingwood WWTP Unit Processes

Unit Process and Description	C of A Capacity
Bar Screens	One mechanical bar screen in main channel and one manual bar screen in the bypass channel, each with a capacity of 60,900m^3/d together with one screenings screw conveyor with screenings dewatering capacity.
Pumping	Three (3) pumps, each with a capacity of 392L/s at 11.0m TDH.
Vortex Grit Separators	Two (2) free vortex grit separators, each with a hydraulic capacity of 30,450m^3/d with one grit classifier and dewatering device.

Table 4.3 Collingwood WWTP Unit Processes

Unit Process and Description	C of A Capacity
Flow Measurement	One ultrasonic flow measurement device at the headworks building and an automatic sampler at the headworks building.
Primary Treatment	Three (3) primary clarifiers, each with a capacity of 629m^3 and a surface area of 177m^2. One 3.8L/s capacity scum pump and discharge piping to the primary digesters. A raw sludge pumping station equipped with three raw sludge pumps and discharge piping to the primary digesters.
Aeration	Two (2) aeration tanks with three 14.6m x 14.6m x 4.6m (SWD) compartments, each compartment has a capacity of 978m^3 and equipped with fine bubble air diffusion system including air supply lines, distributors and membrane diffusers. Three air blowers each having a capacity of 850L/s at 480 kPa.
Secondary Clarifiers	Two secondary clarifiers (61m x 12.2m x 3.7m (SWD)) with an approximately capacity of 2,718m^3 and a surface area of 743m^2, complete with travelling scapers and effluent launders. One scump pump and discharge piping to primary digesters. An activated sludge pumping station equipped with two 1,100mm diameter screw pumps, each with a capacity of 106L/s with discharge piping for return activated sludge to the aeration tanks and waste activated sludge to the sludge thickener.
Disinfection	One duty open channel with three duty UV banks in series, each bank has a total of 12 UV lamps and a total of 36 UV lamps in the three banks that together provide a minimum dosage of 29 mL/cm^2. The channel is 10.5m x 1.0m x 2.38m and is equipped with a level control weir, UV sensors, automatic level sensors and an existing bypass channel with chlorine tablet disinfection.
Phosphorus Removal	Two 25,000 L capacity chemical storage tanks, one 100L/h capacity duty chemical feed pump and one 90L/h capacity standby pump with feed lines to the dosing point at the end of the aeration tanks.
Final Effluent Flow Measurement and Sampling	An ultrasonic flow measurement device at the effluent discharge weir. An automatic composite sampler drawing from the effluent channel before UV disinfection.

The Town's sanitary sewers range in size from 150mm to 1050mm and there are seven pumping stations. **Table 4.4** presents details on the capacity and status of each station and forcemain.

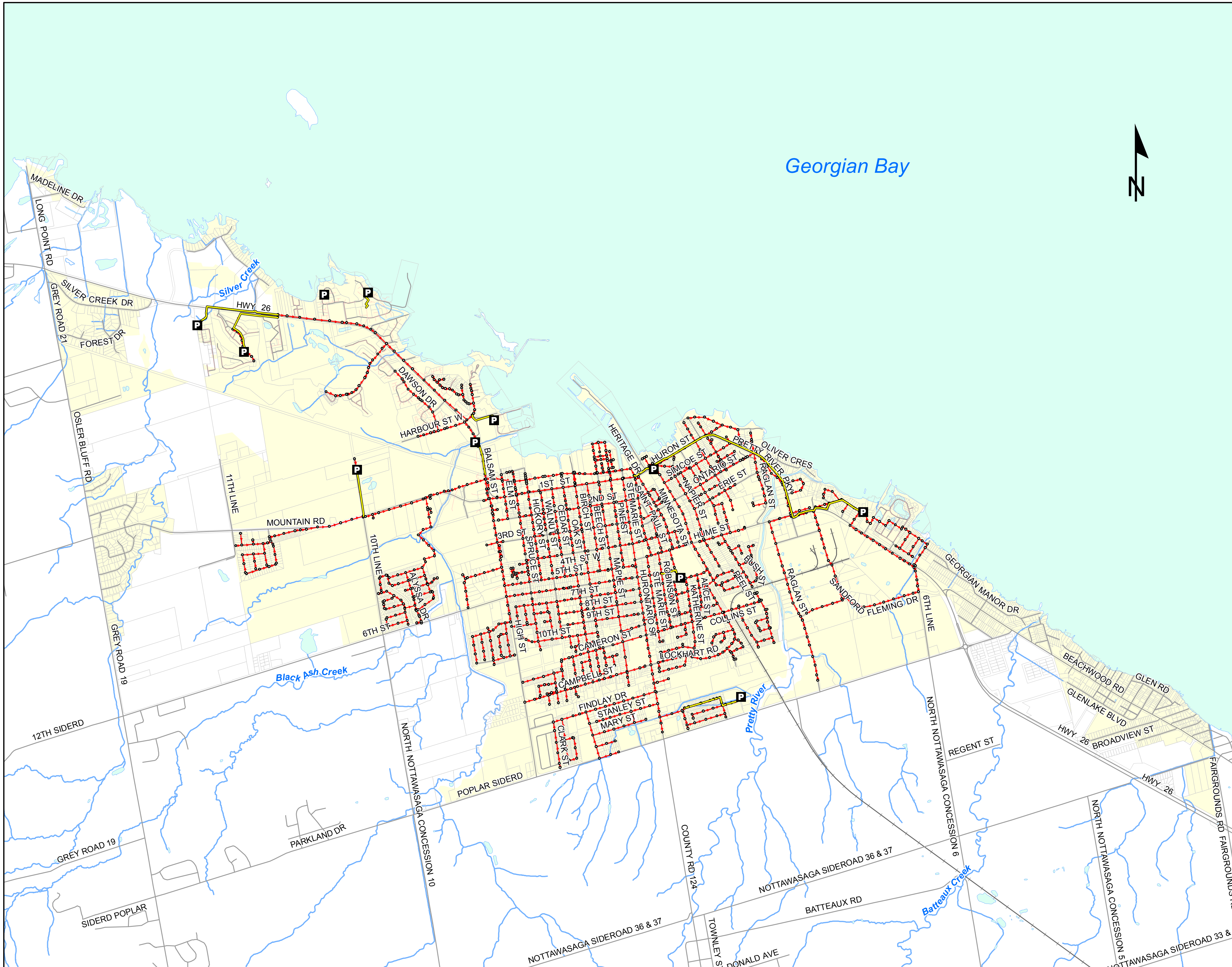


Figure 4.3
Town of Collingwood Sanitary System

- Sanitary Nodes
- Pumping Stations
- Forcemains
- Sanitary Sewers
- Private Sewers

COLE

0 250 500 1,000 Meters

Table 4.4 Collingwood Sanitary Pumping Stations

Pumping Station	Description	Capacity
Black Ash SPS	MECP ECA 1908-B97UD8, issued on March 5, 2019. Pumping station details are: • Inlet channel with grinder, handling capacity of 240L/s. • Two wet wells, with a combined unsurcharged volume of $64m^3$. • Station is equipped with piping and valves that allow a discharge to the existing wet well during overflow events. Existing wet well is equipped with one 30L/s pump that allows discharge into Sanitary MH#2. • Existing wet well has a storage volume of $62m^3$ before an overflow to Black Ash Creek occurs through an existing 600mm diameter overflow pipe. • Three submersible raw sewage pumps with a firm capacity of 212L/s. • One existing 300mm diameter forcemain. A section of new 500mm diameter forcemain has been constructed as part of the current expansion (currently capped at both ends). In period from 2012 to 2016, average pumped flow was 25.9L/s.	Station has firm capacity of 212L/s and maximum station capacity of 318L/s. Current forcemain capacity is estimated as 212L/s which is equal to the station's firm capacity but less than station capacity. Maximum water depth in wet well of 3.05m.
Cranberry Trail SPS	C of A 5925-5EATK8 issued on October 8, 2002. Pumping station information obtained from Cranberry Resort – Sewage Pumping Station Details (Dwg.PS-1). Pumping station details include: • Two constant speed centrifugal pumps with each as capacity of 32.8L/s at 7.5m TDH. • Wet well diameter of 2.4m at a total depth of 7.15m. • Station has a high wet well water alarm level of 175.75m. • Elevations and inverts: 300mm diameter incoming sewer-175.65, wet well invert-174.0, duty pump on-175.65, duty pump off-174.46, standby pump on-175.55, standby pump off-174.36, high level alarm-175.75. • 833m – 200mm diameter forcemain In period from 2012 to 2016, average pumped flow was 1.6L/s.	Station has firm capacity of 32.8L/s and maximum station capacity of 65.6L/s. Forcemain capacity is greater than station capacity. Current forcemain has an estimated capacity of 94L/s. Maximum water level in wet well of 1.75m.

Table 4.4 Collingwood Sanitary Pumping Stations

<table>
<tr><th>Pumping Station</th><th>Description</th><th>Capacity</th></tr>
<tr><td>Minnesota SPS</td><td>Pumping station recently expanded. ECA 8852-AUTS83 issued on January 18, 2018. Information obtained from Minnesota Pumping Station Detailed Design Report (January 2018). Pumping station details include:
• Three submersible pumps, with VFDs, with a firm capacity of 210L/s at a TDH of 7m.
• High level wet well alarm at 175.3m.
• Total storage available in wet well of 48.5m^3.
• Elevations and invert: 600mm incoming sewer invert-174.8, high level alarm – 175.3, lead pump on – 174.71, lead pump off-174.21, lag pump on – 174.91, lag pump off-174.21, wet well invert-172.605.
• 235m - 400m diameter forcemain
No historical flow data is available at this station.</td><td>Station has firm capacity of 210L/s and maximum station capacity of 315L/s.
Forcemain capacity is greater than station capacity. Current forcemain has an estimated capacity of 377L/s.
Maximum depth in wet well of 2.69m.</td></tr>
<tr><td>Patterson SPS</td><td>C of A 2905-655M6H issued on October 4, 2004. Pumping station details include:
• Three submersible pumps, each with a capacity of 36L/s at a TDH of 11.3m. Combined pumping capacity of 72L/s with two pumps operating in parallel.
• Wet well dimensions of 3m x 3m x 6.5 m depth.
• 256m- 250mm diameter forcemain
In period from 2012 to 2016, average pumped flow was 9.6L/s.</td><td>Station has firm capacity of 72L/s and maximum station capacity of 108L/s.
Forcemain capacity exceeds station capacity. Current forcemain has an estimated capacity of 147L/s.
Maximum depth in wet well of 2.13m.</td></tr>
<tr><td>Pretty River Estates SPS</td><td>ECA 2372-7PRP2Z issued on May 7, 2009. Pumping station details include:
• Two submersible pumps, each with a capacity of 29L/s with VFDs at a TDH of 24.3m.
• Wet well diameter of 2.4m.
• Elevations and inverts: incoming 250mm invert – 185.80,
• 150mm diameter forcemain
In period from 2012 to 2016, average pumped flow was 1L/s.</td><td>Station has firm capacity of 29L/s and maximum station capacity of 58L/s.
Forcemain capacity approaches station capacity. Current forcemain has an estimated capacity of 53L/s.
Maximum depth in wet well of 2.33m.</td></tr>
</table>

Table 4.4 Collingwood Sanitary Pumping Stations

Pumping Station	Description	Capacity
Silver Glen Preserve	ECA 1809-7GMQ32 issued on July 18, 2008. Pumping station details include: • Two submersible pumps, each with a capacity of 16L/s at a TDH of 10m. • Wet well diameter of 2.4m. • 150mm diameter forcemain In period from 2012 to 2016, average pumped flow was 0.2L/s.	Station has firm capacity of 16L/s and maximum station capacity of 32L/s. Current forcemain has an estimated capacity of 53L/s. Forcemain capacity exceeds station capacity.
St. Clair SPS	C of A 1434-622JRK issued on June 21, 2004. Pumping station drawings: 968-03047-20. Pumping station details include: • Two pumps in dry well, each with a rated capacity of 155L/s at a TDH of 15.8m. • Two wet wells dimensions of 4.3m x 2.4m (wet well#1) and 4.8m x 3.7m (wet well #2) • Elevations and inverts: 900mm inlet invert 173.11m, wet well invert- 171.65m, ground surface- 178.25m, pump off elevation - 171.95, pump on elevation - 172.95, emergency overflow elevation - 176.6m. • Twin 3,000 m - 450mm diameter forcemains. Each forcemain is dedicated to a pump. In period from 2012 to 2016, average pumped flow was 34.9L/s.	Station has firm capacity of 155L/s and a maximum station capacity of 310L/s. Twin forcemain capacity exceeds station capacity. Each of the two forcemains has an estimated capacity of 477L/s. Maximum depth in wet well of 4.95m.

Notes:

1. Firm capacity calculated as pumping capacity with largest pump out of service.
2. Maximum wet well depth set to high level alarm elevation for Cranberry, Minnesota and St.Clair Pumping Stations. Maximum wet well depth set to obvert of lowest incoming sewer at Black Ash Pumping Station, Patterson Pumping Station and Pretty River Pumping Station.
3. Forcemain capacity estimated using Hazen Williams equation. Forcemain capacity either exceeds or approaches pumping station capacity for each station except the Black Ash SPS, where forcemain capacity is equal to the firm capacity.
4. Information provided for the Black Ash SPS represents the planned upgrade of the station which will be completed by 2020.

In addition to the above pumping stations, the performance of the Collingwood WWTP was also considered. Based on the information included in **Table 4.3**, the firm capacity of the Collingwood WWTP Pumping Station was set to the peak flow capacity of the treatment plant of 60,900m^3/d or 705L/s. The maximum depth in the wet well was set to the elevation where incoming flow would crest over the stop log weir at the plant bypass chamber, located upstream of the pumping station. Based on measurements taken in 2017, the weir crest elevation is 176.53m and equates to a maximum depth in the wet well of 2.92m. To assess the performance of the existing system, a flow monitoring program was completed and a sanitary hydraulic model was developed and calibrated. Further details of the flow monitoring program results as well as the hydraulic model development and calibration can be found in **Appendices D2 and D3.**

5 Existing Water System Performance

To assess the performance of the existing water system, the updated water system hydraulic model was utilized. The following sections present the criteria used to assess performance as well as the results of the assessment.

5.1 Water System Criteria

The following performance criteria was developed for evaluating the water system and identifying required upgrades to service future growth. These criteria were developed from several sources including the Collingwood Development Standards (2007) and MECP Guidelines (2011). The criteria proposed to evaluate deficiencies in the system include:

1. Pressure Requirements:
   The normal operating pressure should range between 345kPa (50psi) and 550kPa (80psi);
   The minimum pressure during the peak hourly demand shall be greater than 275kPa (40psi);
   The maximum pressure under any conditions shall be 690kPa (100psi); and,
   The minimum pressure when the system is tested for fire flow in conjunction with the design maximum daily demand shall be 140kPa (20psi).
2. Fire Flow Requirements should meet one (1) of the following:
   - The Town's Development Standards, as amended (recommended for Master Planning); and,
     - Residential of 57L/s minimum and 76L/s preferred;
     - Institutional/Convenience Commercial – minimum of 91L/s or 114L/s preferred;
     - Institutional/Commercial – 136L/s minimum or 152L/s preferred; and,
     - Downtown Commercial – 136L/s minimum or 189L/s preferred.
   - The Fire Underwriters Survey (FUS) (recommended for development applications and specific site analysis).
3. Water Storage should be available to meet MECP Guidelines:
   - Fire storage – Volume for 2-hour fire event at required fire flow from Standards or FUS;
   - Balancing storage – 25% of Maximum Day Demands (MDD) or as calculated through system performance; and,
   - Emergency storage – 25% of fire storage plus balancing storage.
4. Existing water demands used in assessing system performance should be based on current consumption data for the Town of Collingwood:
   - 2016 Billing records indicate demands of approximately 500L/unit/day for residential accounts.
5. Future domestic demands used for development and master planning should be established based on existing demands, consumptions trends, and water loss:
   - The Town's Development Standards recommend using 450L/capita/d; and,
   - Peaking factor are stated to be 2 for MDD and 4.5 for Peak Hour Demand (PHD).
6. Pipe Velocities should not exceed a maximum of 1.5m/s during normal operation and 5.0m/s during emergency conditions (Current Collingwood criteria is 4.0m/s).
7. Head Loss Gradients should not exceed a maximum of 2.0m/km in transmission mains.
8. Firm Pumping Capacity (calculated with largest pump out of service) should meet maximum day demands and fire flows.

9. Standby Power should be provided to meet a minimum servicing requirement of average day demands.
10. System upgrades should be scheduled to coincide with a projected capacity trigger of 80% of firm capacity for maximum day demands.
11. Minimum Sizing Standards:
    - The minimum size of watermains shall be 150mm in diameter in residential subdivisions and 200mm diameter Industrial / Commercial / Institutional developments. Adequate sizing to be confirmed to supply an appropriate maximum day plus fire flow demand while maintaining adequate average pressures.

5.2 Water System Capacity

5.2.1 Water Supply and Demand

The WTP provides water to the Town of Collingwood, as well as four (4) other municipalities. The Town of Blue Mountains (ToBM) is serviced through a connection to the distribution system at the town boundary, and has a usage limit of 1,250m^3/day. A 58km regional pipeline also provides water directly from the WTP to Clearview Township (New Lowell), Essa Township (Village of Baxter and Town of Angus), and the Town of New Tecumseth (Alliston) at a rate of 9,500 m^3/day (referred to as New Tecumseth Supply herein). The treatment facility is currently rated for 31,140m^3/d (Municipal Drinking Water License).

Historical data was used to compare the Maximum Day Demands (MDD) and Average Day Demands (ADD) for water supply in Collingwood and the surrounding municipalities to existing capacity. **Figure 5-1** shows that the Town of Collingwood's current total supply commitments during MDD are approaching 80% of the WTP's capacity.

For the current analysis, the supply commitment on MDD for the Town of Collingwood was taken to be the historical average from 2011 – 2016 of 15,152 m^3/day. This value was used instead of the highest historical MDD over the last 5-years for several reasons. Firstly, per capita water usage is dropping in most Municipalities due to technology improvements, consumer awareness and increasing cost of water. Utilizing the highest historical MDD from 2012 would create an unrealistic current MDD. The most recent recorded MDD in 2016 was also found to be close to the 5-year average, and since it was considered to be a hot and dry summer it provides a strong indicator of a current MDD.

In comparison, the Average Day water demands ranged from 8,438 – 10,025m^3 for Collingwood and 17,643 – 19,828m^3 in total (**Figure 5-2**). The total supply commitments for an average day scenario represent approximately 58% of the existing capacity at the WTP.

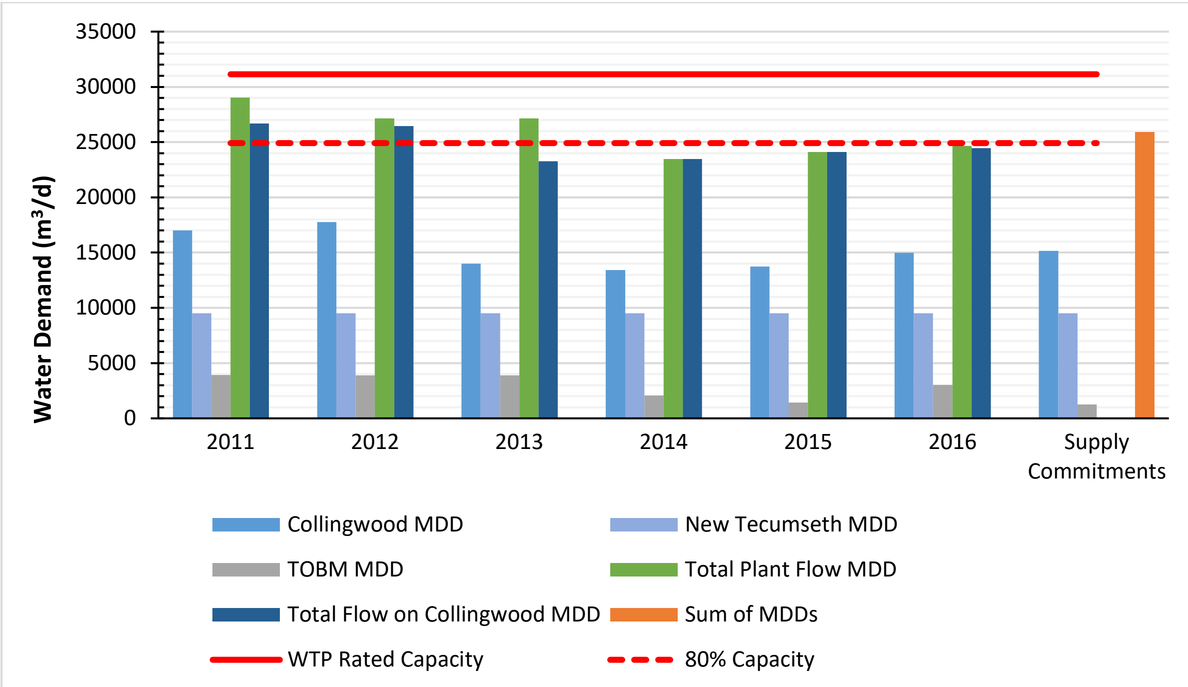


Figure 5-1 Historical Maximum Day Demand and Capacity

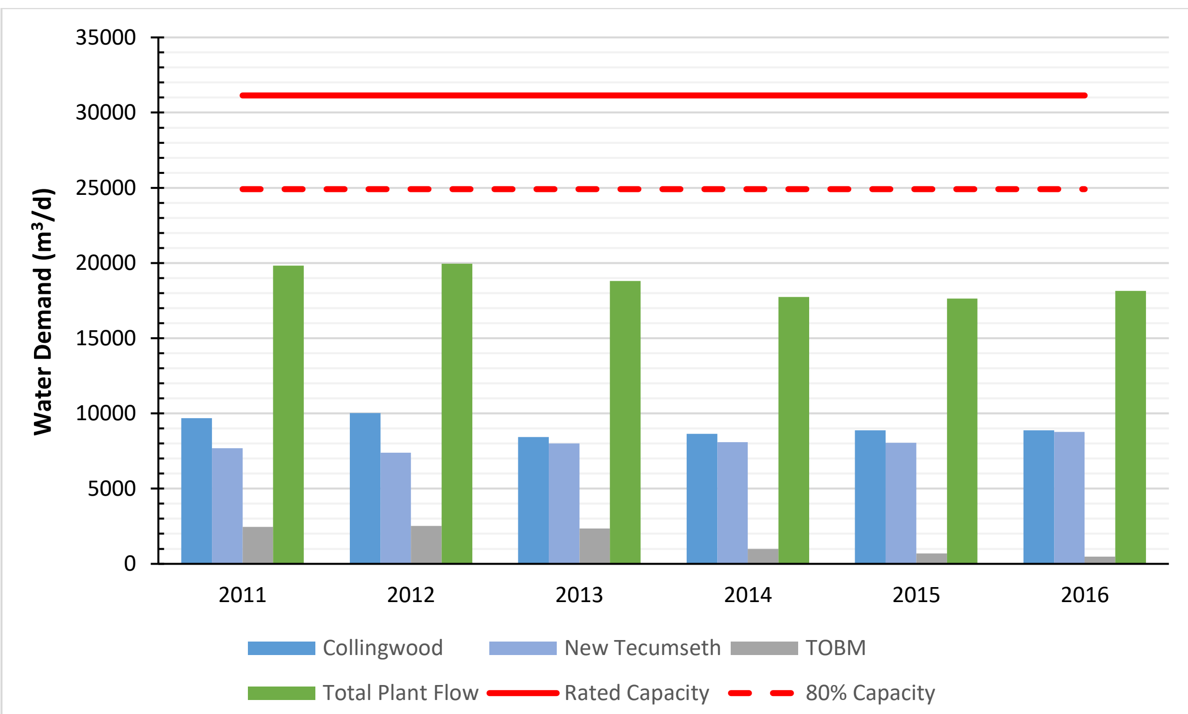


Figure 5-2 Historical Average Day Demand and Capacity

5.2.2 Water Storage

5.2.2.1 Available Storage

The Town of Collingwood has three main locations for storing water in the distribution system in addition to the clearwell at the water treatment plant. **Table 5.1** summarizes the available storage quantities from the 2016 Drinking Water Works Permit.

Table 5.1 Collingwood Total Available Storage

Facility	Volume
WTP Clearwell	$797m^3$
Carmichael West End Reservoir	$6,800m^3$
Collingwood Water Tower	$1,685m^3$ (1)
Bob Davey South Collingwood Reservoir	$2,565m^3$ (2)
Stewart Road (Future)	$4,705m^3$ (3)
Available Existing Storage Zone 1	$9,282m^3$
Available Existing Storage Zone 2	$2,565m^3$
Total Available Storage	$11,847m^3$ (4)

Notes:
(1) $1,685m^3$ or 95% of total available was used in storage analysis. A volume of $2,273m^3$ is provided for the elevated tank in the 2016 Drinking Water Works Permit. Drawings of the elevated tank indicate that the maximum level is 7.3m, resulting in a total usable storage of approximately $1,773m^3$. SCADA records indicate that tank levels are typically maintained between 5m-7m, providing a volume of $1,040m^3$-$1,685m^3$, which is reflected in the total available storage above.
(2) The Davey Reservoir services Zone 2 under standard operating conditions.
(3) The Stewart Road Pump Station is expected to be in operation in the next five years.
(4) In the event of a low pressure event, valves along the zone boundaries can open to allow water to flow from Zone 1 into Zone 2 or vice versa. The whole system therefore has access to the total storage volume of the system, but may be limited by watermain capacity.

5.2.2.2 Required Storage

MECP Fire and Storage Calculation

The MECP formula for sizing water storage systems was used to determine the requirements for the Town of Collingwood. The total storage requirement is made up of three (3) components: A, B, and C. Component A represents storage allocated for fire flow, while components B and C represent equalization and emergency storage, respectively. **Table 5.2** demonstrates the fire and storage calculations.

Table 5.2 Fire and Storage Calculation

Total Water Storage Volume Required = A (S) + B + C		
A	=	Storage volume required for fire-fighting (m^3)
	=	Fire Flow (L/s) x Duration (h)
B	=	Equalization Storage (m^3)
	=	Storage volume to meet the diurnal variation of the maximum day condition
	=	25% of MDD
C	=	Emergency Storage (m^3)
	=	Additional storage for emergency events (i.e. prolonged power loss, watermain breaks, higher than usual demands, unusual fire demands, etc.)
	=	25% of (A + B)

Source: MECP Guidelines for Drinking-Water Systems 2008

Fire Flow

The value for fire flow storage volume (A) was calculated based on the Town of Collingwood's Development Standards and the Fire Underwriters Survey (FUS). The highest preferred hydrant fire flow criteria from the Standards is 189L/s (11,340L/min) for Downtown Commercial sites. The FUS indicates a required duration of 2.0-hours – 2.5-hours for fire flows of 10,000 – 12,000L/min. The resulting Fire flow storage is:

A = Fire Flow (L/s) x Duration (h)
= 189 (L/s) x 2.5 (h) x 60 (min/h) x 60 (s/min) / 1000 (L/m^3)
= **1,701m^3**

Equalization Storage

The Maximum Day Demand for the Town of Collingwood was taken to be 15,152m^3. The storage volume required to meet diurnal variation is calculated to be 25% of the MDD, or **3,788m^3**.

The ToNT supply was not considered part of the storage analysis as these demands are typically very consistent and the regional pipeline is not directly connected to the distribution system other than at the WTP, and at Davey BPS through a flow control valve. This pipeline can impact supply, pumping and watermain capacities but should not impact storage requirements. The ToBM water taking amount was also excluded from the storage calculation.

Emergency Storage

The emergency storage is equal to 25% of the sum of fire flow storage and equalization storage. This value was calculated to be **1,372m^3**.

Total Storage Requirements

The total storage requirement is the sum of A+B+C. The total storage requirement for the Town of Collingwood based on the MECP guideline methodology is **6,861m^3**. The same procedure was carried out for Zone 2 resulting in a required storage of **2,747.3m^3**.

The total storage requirements (Zone 1 & 2) can be compared to the system's total available storage. The total required storage accounts for approximately 61% of the available 11,847m^3 in the system. There is also adequate available storage in Zone 1 to meet the full system's requirements. The additional 4,705m^3 of storage from the Stewart Road Pumping Station will provide an even larger buffer for storage in the system in the future.

Zone 2 also has access to 11,847m^3 of storage within the entire system through zone boundary valves and pumping stations. The required storage in Zone 2 represents about 23% of the total available storage in the system. If Zone 2 remained isolated from Zone 1 in an emergency, the total available storage of 2,565m^3 in Zone 2 is slightly below the required 2,747m^3. However, the additional storage from Stewart Road Pumping Station and Reservoir will also be accessible to a large portion of Zone 2 in the future.

5.2.3 Pumping Capacity

There are currently three Pumping Stations and two booster stations in the Town's distribution system. When it is complete, the Stewart Road pumping station will impact the west regions of the Town and the Georgian Meadows BPS will be decommissioned. The pumping capacity within the system is shown in **Table 5.3**.

Table 5.3 Pumping Capacity

Supply	Zone Supplied	# Units	Pump Type	Rated Flow (L/s)	Firm Capacity (L/s)	Rated Head (m)	HGL (m)	Drive Type
Raymond A. Baker WTP	Zone 1	2	vertical turbine	138.6	333.9	55	227	variable speed
		1	vertical turbine (standby)	138.6		55		constant speed
		1	vertical turbine (jockey)	56.7		37		variable speed
	Regional Transmission	3	vertical turbine	136.1	272.2	55	222	variable speed
Carmichael BPS	Zone 1	1	vertical turbine	100	300	45.6	227	constant speed
		2	vertical turbine	200		42.7		constant speed
Osler Bluff BPS	Zone 2	3	horizontal in-line booster	43.9	87.8	27.6	250	constant speed
Georgian Meadows BPS	Zone 2	1	horizontal in-line booster	2.3	11.8	22	250	constant speed
		2	horizontal in-line booster	9.5		22.4		constant speed
Davey BPS (pump upgrade design under review)	Zone 2	1	vertical high lift turbine	25	170	60	250	variable speed
		1	vertical high lift turbine	55		60		constant speed
		2	vertical high lift turbine	92		60		constant speed
Stewart Road BPS	Zone 2	TBD (Future)						

Total pumping capacity for Zone 1 is 633.9L/s with the combination of the WTP and the Carmichael BPS. Total MDD when including the ToBM is approximately 190L/s, and 175L/s for only the Town of Collingwood. Since there is limited elevated storage, the majority of the 189L/s fire flow are required to be supplied through pumping. The total required pumping capacity in the system is the sum of MDD and Fire Flow requirements (190L/s plus 189L/s) for a total of 379L/s. The Zone 1 firm pump capacity of 633.9L/s greatly exceeds the required pumping of 379L/s, and is therefore not a cause for concern.

The existing Zone 2 pumping capacity is 269.6L/s. With an MDD of 23L/s and a potential fire flow of 189L/s the total required Zone 2 pumping capacity is 212L/s. The available pumping capacity in Zone 2 also exceeds the required flow rate. **Table 5.4** presents pumping requirements.

Table 5.4 Pumping Requirements

Pressure Zone	Demands (L/s)	Available Pumping Capacity (L/s)
Zone 1, Zone 2 & ToBM		
MDD (incl. ToBM)	190	633.9
Fire	189	
Total	379	
Zone 2		
MDD (Zone 2)	23	269.6
Fire	189	
Total	212	

5.2.4 Watermain Capacity

Figure 5-3 shows the diameters of watermain in the Town. This data was updated in the hydraulic model based on information from the Town's GIS records for active watermains at the time the data was supplied. The system consists of mostly 150mm diameter watermains in residential areas, and 200mm or 300mm pipes along major roadways with several 400 and 450mm watermains in key areas.

Part of the water produced at the WTP is directed to the regional pipeline via a 600mm watermain at a firm capacity of 272.2L/s. This pipeline is also connected to a 500mm watermain that feeds the Davey BPS.

Typical feedermain capacity can be calculated for different pipe sizes based on head loss criteria, C-factors and other hydraulic parameters. At a head loss of 2.0m/km and C-factor of 130, the approximate capacity of watermains that exist in the Town are shown in **Table 5.5**.

According to these values, the 600mm Regional Pipeline is capable of transmitting 329L/s with reasonable headloss, which exceeds the Regional pumps' firm capacity of 272L/s. The 300mm and 400mm watermains that supply the Town of Collingwood have a capacity of only 53L/s and 113L/s, respectively, if they are to meet the Town's headloss criteria. This is less than the firm capacity of 333.9L/s that can be produced from the WTP. Furthermore, the distribution does not contain a large watermain loop to provide redundancy in case of watermain failure or emergency.

Table 5.5 Feedermain Capacity

Pipe Diameter (mm)	Capacity (L/s)
150	18
200	33
300	53
400	113
450	155
500	204
600	329

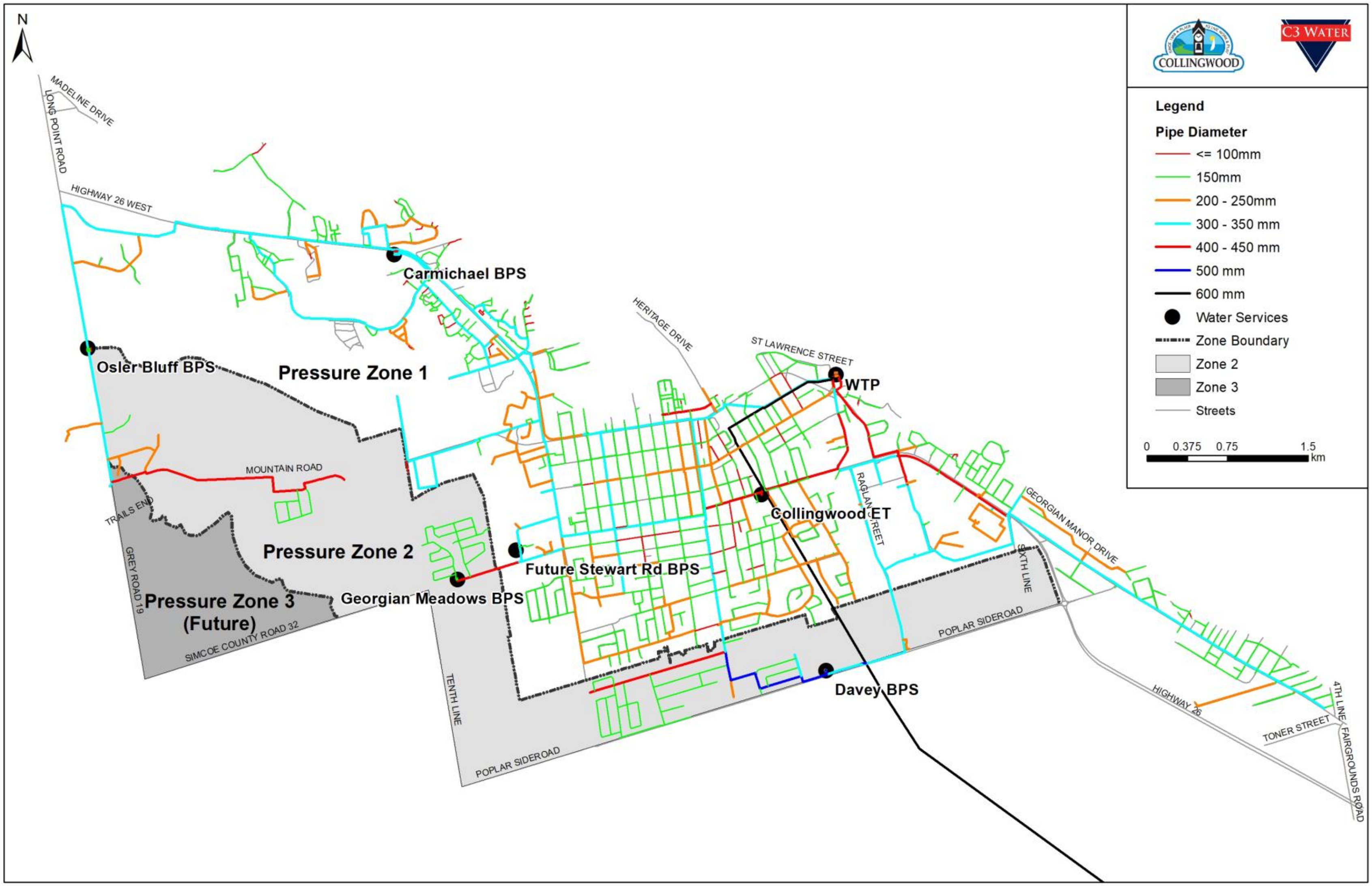


Figure 5-3 Existing Watermain Sizes and Water Services

5.3 Water System Hydraulic Performance

The Town's water system model was used to carry out a hydraulic and water quality assessment to identify potential deficiencies within the existing conditions. System pressures, watermain capacity, fire flow capacity and water quality were evaluated in the model during Average Day Demand (ADD) and Maximum Day Demand (MDD) scenarios depending on the type of analysis.

5.3.1 System Pressures and Zone Boundary Analysis

Zone Boundary Analysis

A zone boundary is defined by the elevations in a service area and the Municipality's defined level of service. Locations with higher elevations are operated at a higher hydraulic grade line (HGL) to maintain target pressures from the Design Criteria. The HGL in a pressure zone is maintained by BPS feeding the zone and closed valves along the borders. The Town of Collingwood is currently divided into two (2) pressure zones. Pressure Zone 1 operates at an HGL of 227m and Zone 2 operates at a target HGL of 250m.

Figure 5-4 provides an overview of the elevations in the Town. The ranges in elevation were defined based on pressure requirements described in the Design Criteria:

- Range 1: < 171m – Static pressures greater than 80psi in Zone 1;
- Range 2: 171 – 192m – Ideal Zone 1 static pressures between 50 and 80psi;
- Range 3: 192 – 215m – Ideal Zone 2 static pressures between 50 and 80psi; and,
- Range 4: >215m – Static pressures greater than 80psi in Zone 2.

Figure 5-4 shows that the pressure zones are well suited for their pressure designation with static pressures between 50 and 80psi in both pressure zones. Areas identified as A and B show elevations that would result in pressures below 50psi for their respective zones.

Area A

This is an area of future development that will be serviced by Zone 2 based on the existing elevations of around 190mAsL and proximity to Zone 2 infrastructure.

Area B

Area B could be moved into Zone 2 to provide more acceptable pressures for residents located at higher elevations. The HGL in this location ranges from 189 – 195mASL and would be better suited for the HGL of 250mASL in Zone 2. The watermains in this area are mostly ductile iron pipes installed since 1980, therefore watermain capacity is not the expected cause of pressure concerns.

This could be accomplished by making a connection along High Street between Campbell Street and Findlay Drive. Two zone boundary valves near High Street have already been installed in anticipation of this zone boundary change, but currently remain open. One is located on High Street north of Telfer Road and the other is on Campbell Street east of Herrington Court. Once the boundary change is made, the features and setting of these valves should be tested so that they maintain Zone 2 pressures and open in emergency conditions if required. In the future, this area is also expected to be connected to the Creekside development located in the north part of Area B and on the west side of High Street.

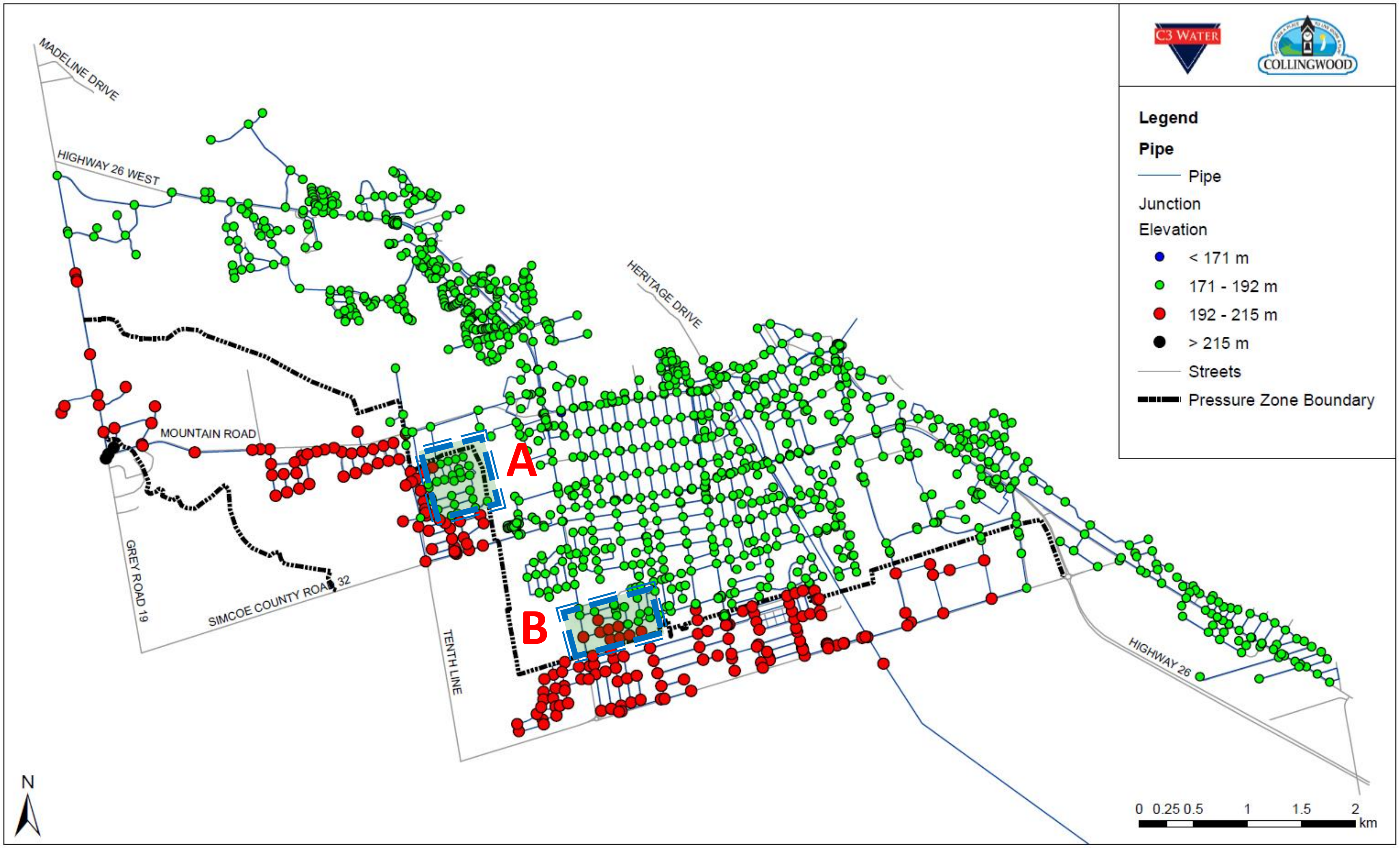


Figure 5-4 Elevations

System Pressures

The minimum pressures in the system were evaluated during maximum day demands, and typically occur at the peak hourly demand. The results are shown in **Figure 5-5** and are colour coded based on the Town's pressure criteria. Areas of concern are indicated where minimum pressure during the peak hourly demand are less than 40psi.

Based on the model results, some areas in the north-west portion (A) of the Town experience below 40psi under maximum day demands. The south-west corner (B & C) of the network also experiences low pressures near the zone boundary.

The normal operating pressures were also evaluated during average day demands, and should be in the range of 50 – 80psi. The results of this analysis show that these criteria are met in most of Zone 1 based on average pressures. Portions of the system along the Zone boundary (A, B &C) were found to be slightly lower, with average pressures of 40 – 50psi. Many areas in Zone 2 are operated at an average pressure of 80 – 100psi, which is slightly above the pressure criteria.

The maximum pressure under any condition should be 100psi. Based on the model results for MDD and ADD scenarios, there is only one location that experiences pressures slightly above 100psi, just after the Osler Bluff BPS in the far west portion of the network. There does not appear to be customers serviced at this location. This result is not shown in **Figure 5-5** since the map displays minimum pressures only. The following is a summary of each identified area.

Areas A, B, C and E

Minimum pressures are reduced in these areas due to head losses in the water system to service peak hour flows.

Area D

Minimum pressures are above 80psi in Area D but less than 90psi. Pressures are set by the Davey BPS and could be modified to reduce to less than 80psi if necessary.

Figure 5-6 provides an overview of average pressure during Average Day Demands which are indicative of normal operating pressures. During an average day scenario, there is also less headloss in the system which provides an increase in system pressures throughout the system. The following is a summary of each identified area.

Area A

This area experiences normal operating pressures during average conditions.

Areas B, C and G

Areas B and C continue to be locations of low pressures during average conditions, likely due to some headlosses across the system. Area G also has less than optimal operating pressures during average day demands.

Areas D, E and F

Areas D, E and F are areas of high pressure with pressures above 80psi. The pressures in these areas could be reduced by reducing the discharge pressures at each pumping station.

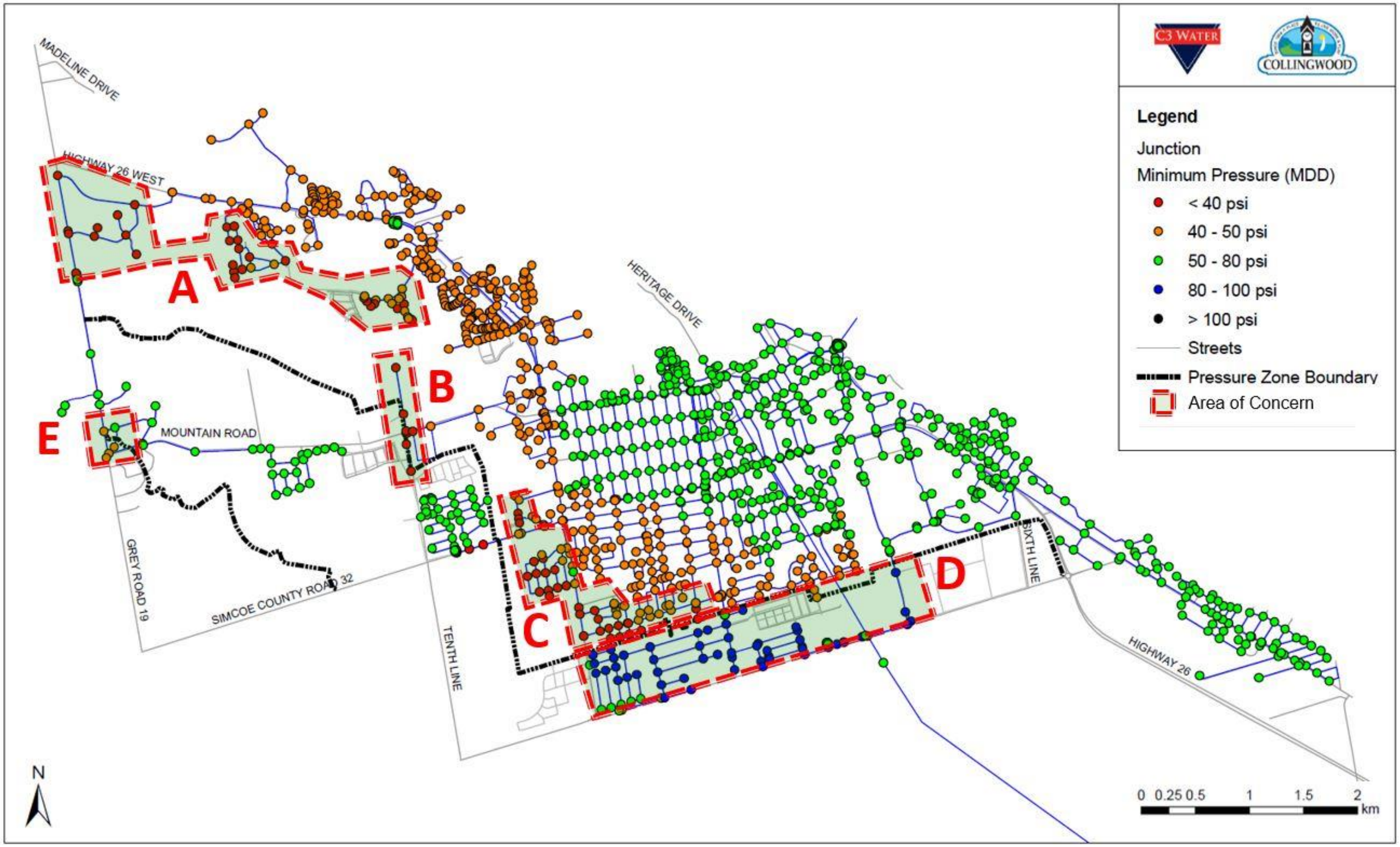


Figure 5-5 Minimum Pressures during Maximum Day Demand 2016

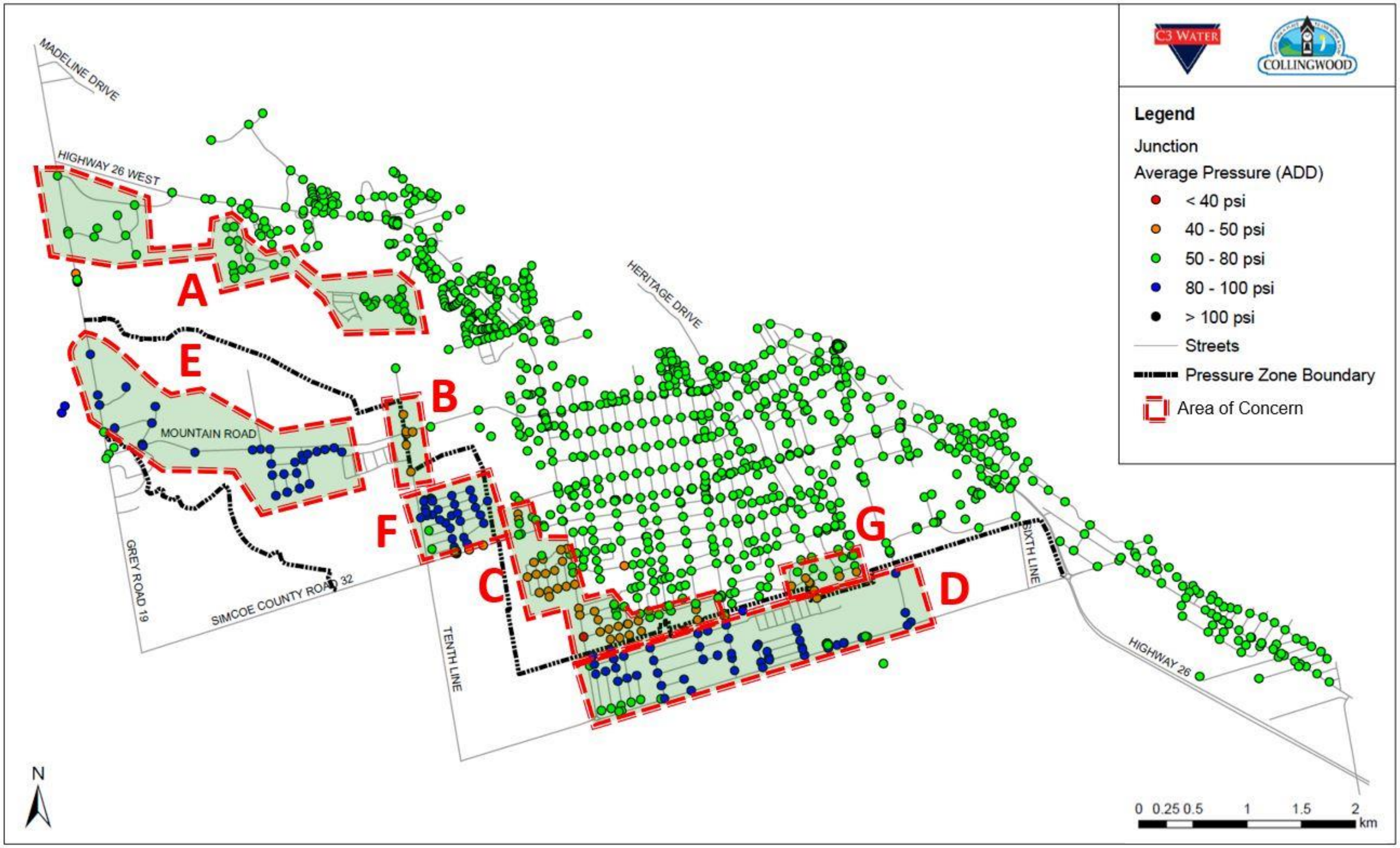


Figure 5-6 Average Pressures during Average Day Demand 2016

5.3.2 Watermain Capacity

According to the stated criteria, head loss gradients should not exceed a maximum of 2.0m/km in transmission mains. **Figure 5-7** shows head losses through the system during the Peak Hour of Maximum Day Demands, which occurs around 3:00am in the summer months. The results of the hydraulic modelling demonstrate the restrictions identified in the benchtop analysis for watermain capacity discussed previously.

There are many portions of watermain with high head loss (>2.0m/km). Particularly, head losses are high along the 300mm and 400mm feedermains that direct water from the WFP across the downtown area to the Carmichael BPS, Water Tower, and future Stewart Road Pumping Station.

As noted in **Section 5.3.1** pressures are reduced in several areas due to high head losses in the distribution system. Two (2) locations are used to demonstrate this trend in **Figure 5-8,** which shows the variation in pressure during an MDD scenario. The rapid change in pressures are caused by head losses in the system as well as large pumps turning on.

There are several ways to mitigate pressure fluctuations during periods of high demand. Increasing watermain capacity can help to decrease pressure during peak hours. Furthermore, operating a different pump at the Carmichael BPS or utilizing a variable frequency drive could reduce the impact of pump starts and stops on the system.

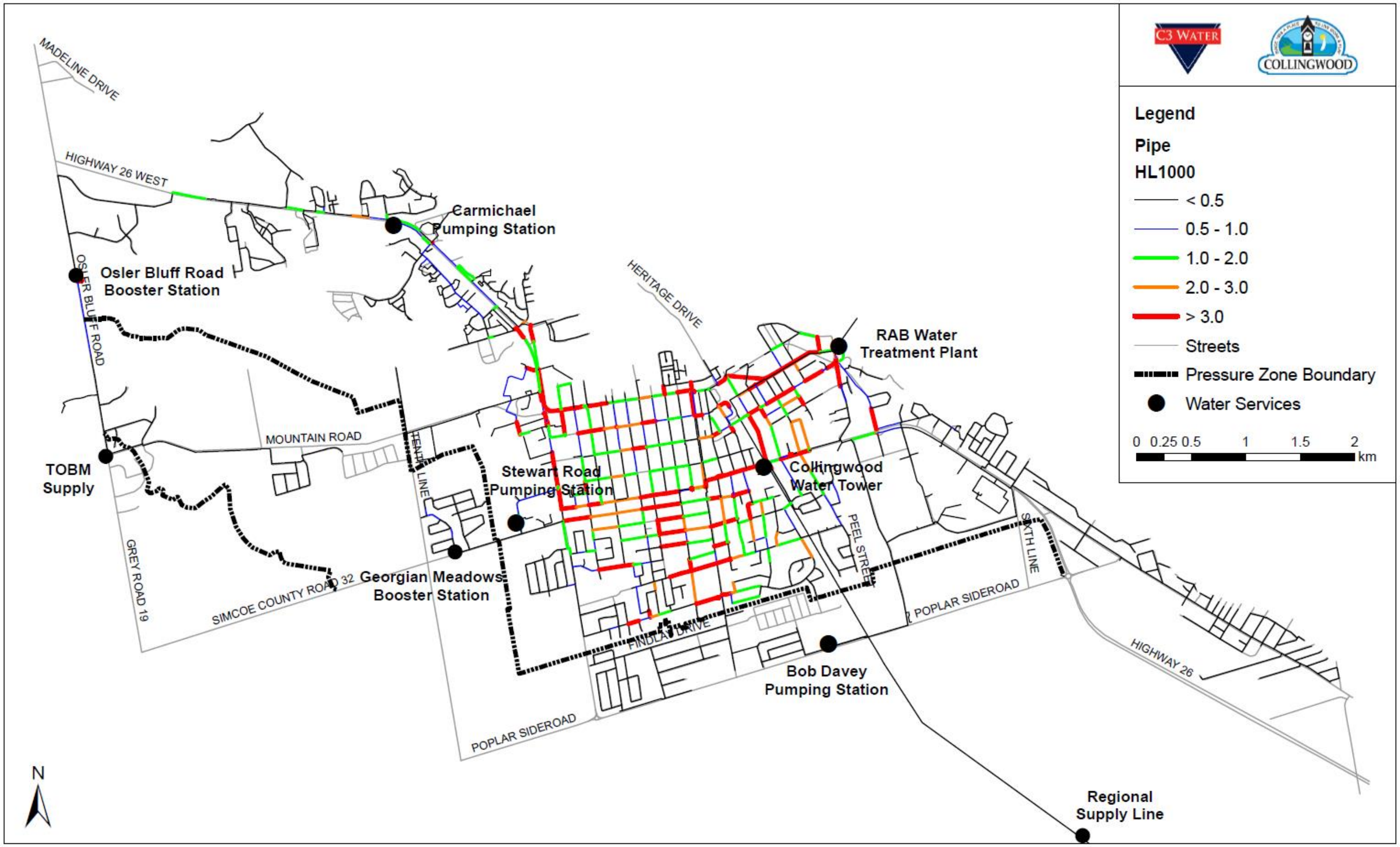


Figure 5-7 Maximum Head Loss during Maximum Day Demands (03:00 Hours) 2016

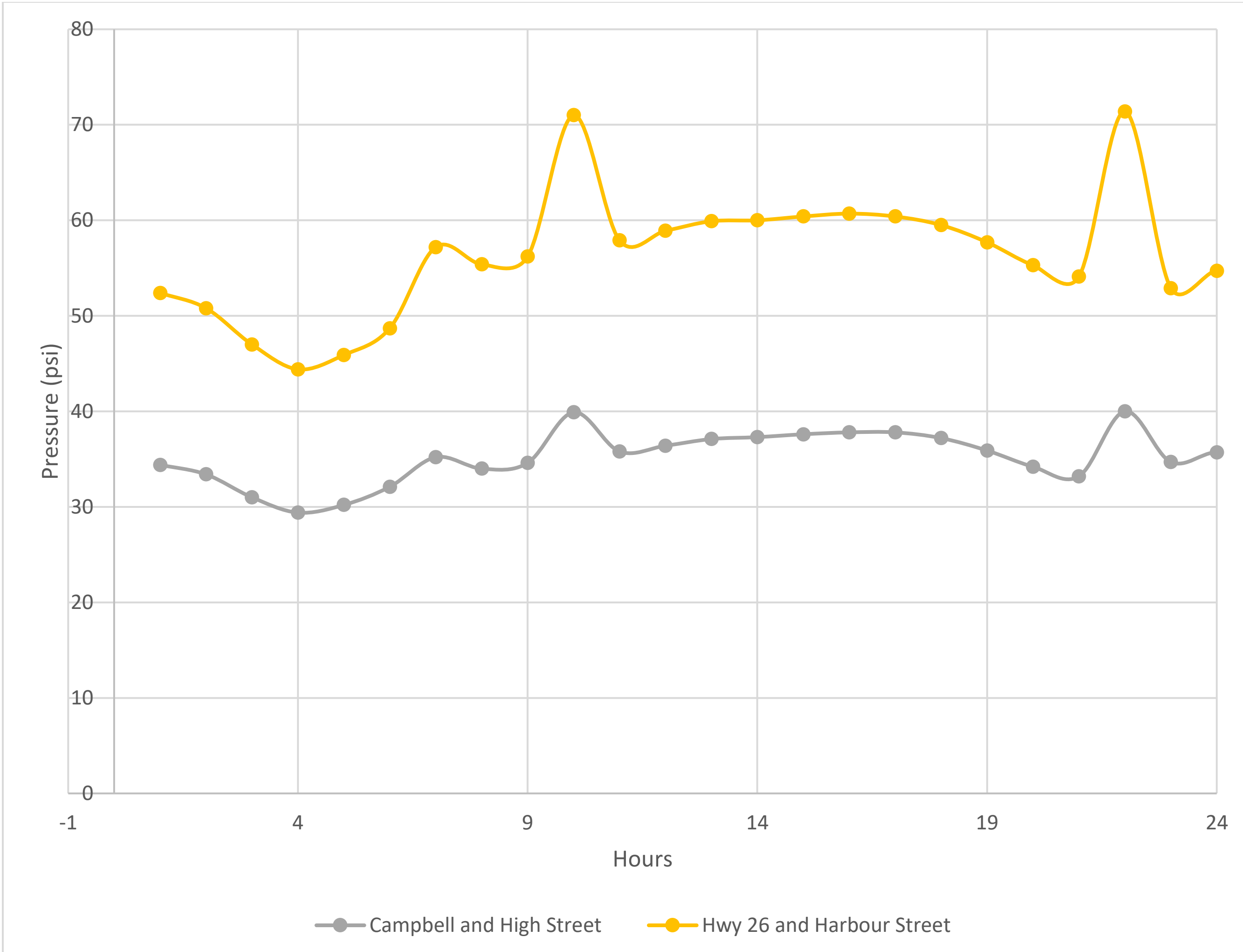


Figure 5-8 MDD Zone 1 Pressures

5.3.3 Fire Flow Capacity

Fire Flow requirements in the Town are associated with land use type; where residential zones should have a minimum fire flow of 57L/s and Industrial, Commercial, & Institutional (ICI) zone should have a fire flow of 136L/s. The recommended fire flow for ICI is 189L/s according to the Town Development Standards.

Modelling was conducted using a steady-state analysis of available fire flows at a residual pressure of 20psi for a 2.5-hour fire flow scenario at 12:00pm under MDD conditions. To simulate typical operation of the Town's pumping stations, two (2) pumps at Carmichael BPS were triggered to start during the fire flow event in the model, in addition to the pumping capacity at the WTP. The results for each node in the model are shown in **Figure 5-9** and areas of concern are outlined in red. The available fire flows are colour coded according to the Town's criteria and can be compared for each land use type. Dead-end nodes were excluded from the analysis since they typically show low fire flow values, but customers are most often serviced by hydrants located along the main roadway.

The main areas of concern are located in Zone 2.

Area A

The residential area south of the Osler Bluff BPS (Area A) is subject to fire flows less than the minimum 57L/s. Once constructed, the Stewart Road PS will service this portion of Zone 2 via a new feedermain along Tenth Line connected to Thomas Drive. Pumps at Stewart Road PS should be selected to meet fire flow requirements in this area.

Area B

The area west of the Georgian Meadows BPS (Area B) also falls below acceptable fire flow criteria, but will be serviced by the Stewart Road Pumping Station once completed.

Area C

ICI customers at the end of the Mountain Road watermain do not have acceptable fire flows according to the recommended values for this land use type. There are also several points scattered throughout the residential areas that have fire flows below 57L/s.

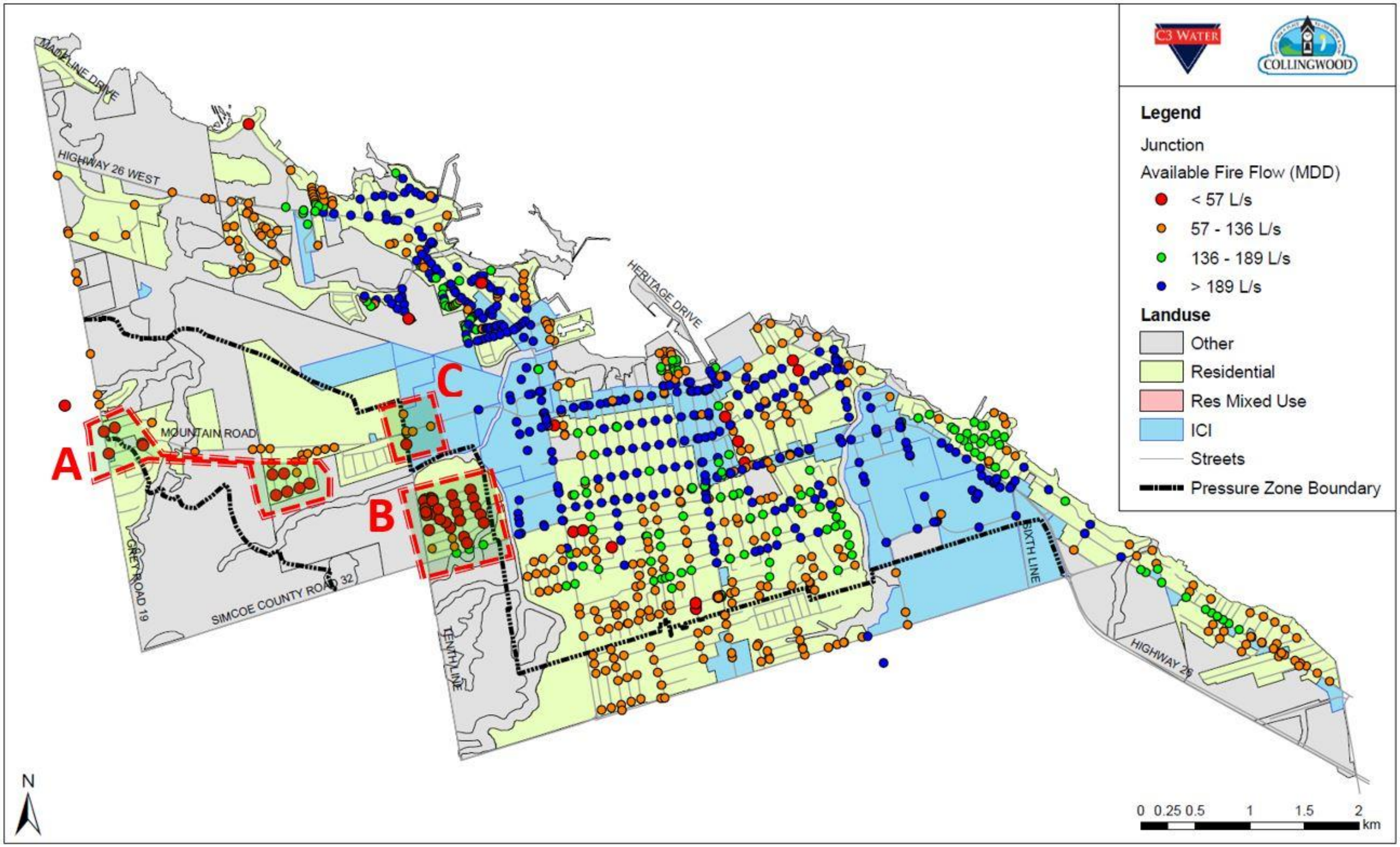


Figure 5-9 Available Fire Flow (MDD) 2016

5.3.4 Water Quality

Water quality analysis was conducted based on water age in the distribution system during average day demands. The hydraulic model was simulated over a 10-day period and the average age was calculated during the ninth day for each node in the system. Water age was generally greater in areas outside of Pressure Zone 1 since these areas are located farther from source water at the WTP. However, because water is refreshed with chlorine at booster pumping stations along the pressure boundary, the results of this analysis are not a direct indication of low chlorine residuals.

Figure 5-10 shows the results of this analysis, where black nodes indicate areas with older water, but not necessarily poorer quality.

Nodes located at the end of distribution mains typically have higher water age values. Dead-end street and sections of the model that have low demand also are subject to higher water age. This explains why certain nodes can experience higher water age compared to adjacent nodes. Based on model results, the following areas experienced water age.

Area A, C, E

These areas are located farthest from the WTP and therefore show higher water age. Area C is supplied by the Davey BPS, which provides re-chlorination of the water.

Area B, D

The residential area B in the north part of Town and the industrial zone south of the WTP (D) also has several locations with high water age values. In the existing water model, these nodes have little, or no water demand based on the past allocation method. In the industrial area, many of the demands were allocated to the main road intersections, resulting in high water age at internal nodes on the large 300mm watermains. This is not necessarily representative of field conditions and can be verified during future demand allocation and model updates.

Area B consists of a recent development area. In 2016, all phases of the housing development had not been completed, but watermain infrastructure was input to the model. This is also true for the most western portion of Area C. The lack of demand in these areas resulted in high water age, but can be addressed when water billing data beyond 2016 is updated in the model.

5.4 Summary

An overview of the existing water system provides the following conclusions:

- Recent Max Day Demands and the Town’s supply commitments to Collingwood, ToBM and ToNT appear to be stabilizing, but are reaching 80% of the WTP’s available capacity;
- Average Day Demands represent approximately 58% of the WTP’s rated capacity;
- Diurnal curve analysis showed that a nighttime peak occurs under MDD. It is recommended that this trend be further investigated, as it can cause some issues of low-pressure;
- Available storage is sufficient for existing conditions, although most storage is held by in-ground reservoirs which rely on pumping capacity to access storage;
- There is sufficient pumping capacity to meet existing conditions;
- Watermain capacity appears restricted by a lack of large watermains in the distribution system;

- Based on elevations, the pressure zones are well defined;
- Pressures are highly variable in the distribution system due to the headlosses across the water system as well as the impact of the large Carmichael BPS pump turning on / off;
- Fire flow capacity is adequate in the locations where high fire flows are required such as the downtown or the industrial area, but there are several pockets of concern identified. Two of the three areas identified should be addressed through the construction of the Stewart Road Pumping Station; and,
- Water Age results provide a baseline for reviewing the impact of future infrastructure.

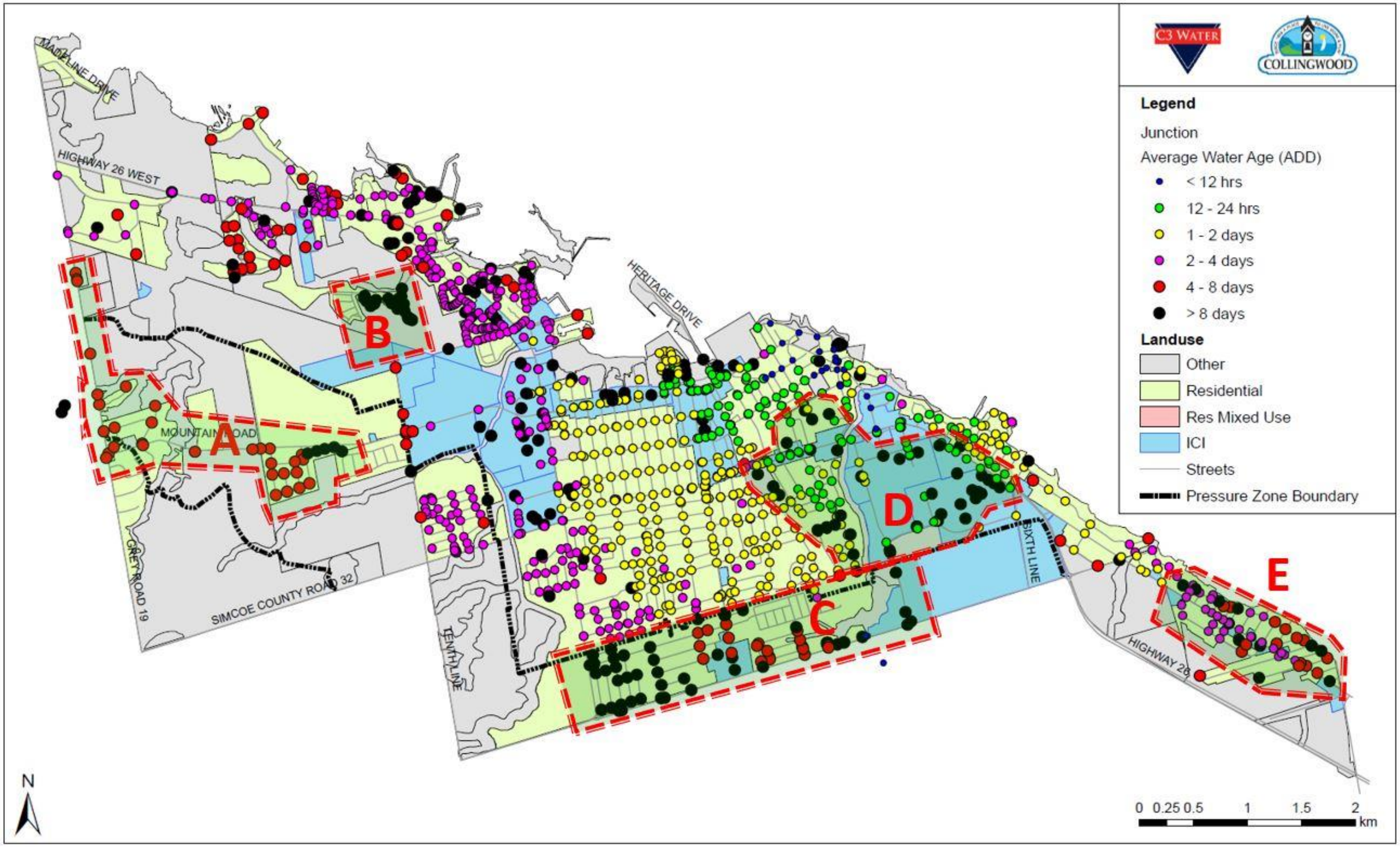


Figure 5-10 Average Water Age 2016

6 Existing Sanitary System Performance

The performance of the existing sanitary system was assessed using a variety of tools and analyses. The following sections present the criteria used to evaluate system performance as well as the performance assessment results.

6.1 Sanitary System Criteria

The following performance criteria were developed for evaluating the sanitary system performance. These criteria were applied to existing and future system performance to identify required upgrades to service future growth. The criteria were developed from several sources including the Collingwood Development Standards (2007) and the Design Guidelines for Sewage Works (MECP, 2008). To assess the performance of the existing sanitary sewer system and pumping stations, the hydraulic model was used with design storm events (2, 5, 10 and 25 year) as well as a historical event that caused isolated flooding in the Town (June 17, 2017). The June 17, 2017 event had a rainfall volume of 67.8mm in 22 hours and had a return period of approximately 25 years based on event volume.

The criteria proposed to evaluate deficiencies in the system include:

1. Treatment:
    a. Rated capacity expansion is triggered when the three year projected average flow reaches 80% of the rated capacity of the facility (average flow);
2. Pumping Stations:
    a. Stations should have sufficient firm capacity to pump incoming peak flows under peak dry weather flow conditions;
    b. Stations should have sufficient firm capacity to pump the incoming wet weather flow and/or maintain the maximum water level in the wet well below the high level alarm level during the 2-year, 5-year and 10-year event and the June 17, 2017 historical event.
    c. Stations should have sufficient station capacity to pump peak wet weather flows and maintain the maximum water level in the wet well below the high level alarm level during a 25-year storm event;
    d. No bypass to the environment should occur during wet weather conditions for storm events up to and including a 25-year design storm event.
    e. For pumping station forcemains, the forcemain capacity is to exceed or match the pumping station capacity.
3. Sanitary Sewers:
    a. Under peak dry weather flow conditions, design flow conditions, and peak wet weather flow conditions for the 2-year, 5-year and 10-year storm and the June 17, 2017 historical event, sanitary sewers should have a d/D ratio of 0.85 or less;
    b. Under the peak wet flow conditions for a 25-year design storm event, surcharge of sanitary sewers is acceptable as long the peak hydraulic gradeline is 1.8m or more below the ground surface. For shallow maintenance holes, the peak hydraulic gradeline may be within 1.8m of the ground surface as long as the incoming and outgoing sanitary sewers are not surcharged.

c. The range of acceptable velocities in all sanitary sewers is between 0.6m/s to 3.0m/s.

4. Siphons:
   a. Surcharge conditions are allowed in siphons.

6.2 System Performance Assessment

To assess system performance, the following was completed:

- Existing reports and data were reviewed to assess the capacity of the Collingwood WWTP, sanitary sewers and pumping stations; and,
- To assess the capacity of the sanitary sewer system, collected flow monitoring data were analyzed and reviewed and modelling was completed to assess capacity constraints within the existing sanitary sewers, pumping stations and forcemains under a number of different conditions including design flows and storm events including a 2-year, 5-year, 10-year and 25-year storm event and the historical rainfall event that occurred on June 17th, 2017 storm event.

The following sections presents the results of existing capacity assessments.

6.2.1 Collingwood WWTP – Historical Performance Data

The Collingwood WWTP has an average rate flow capacity of 24,548m^3/d and a peak flow capacity of 60,900m^3/d. Effluent objectives and criteria for the facility are shown in **Table 6.1**. **Table 6.2** presents a summary of the average and peak flows recorded at the facility from 2012 to 2016 and notes the number of bypass events recorded during each year. It is noted that during the period from 2012 to 2016, only one bypass event was recorded.

Table 6.1 Collingwood WWTP Effluent Limits and Objectives

Parameter	Effluent Limit Average Concentration	Effluent Limit – Average Waste Load (kg/d)	Effluent Objective
$CBOD_5$	25.0mg/L	613.7kg/d	15mg/L
Total Suspended Solids	25.0mg/L	613.7kg/d	15mg/L
Total Phosphorus	1.0mg/L	24.5kg/d	0.8mg/L
E. Coli	-	-	100 organisms per 100mL (monthly geometric mean density)
pH	To be maintained between 6.0 and 9.5		-

Table 6.2 Collingwood WWTP Historical Flows

Year	Average Flow (m^3/d)	Peak Flow (m^3/d)	Notes
2012	17,701	38,160	No bypasses recorded. One exceedance of E.Coli objective reported.
2013	17,774	44,980	No bypasses recorded. Two exceedances of E.Coli objective reported.
2014	16,180	41,610	No bypasses recorded. No exceedances of effluent objectives or limits reports.
2015	13,658	31,500	No bypasses recorded. One exceedance of E.Coli objective reported.
2016	16,189	60,310	One bypass event recorded. One exceedance of E.Coli objective reported.
Average	16,300	43,312	
Maximum	17,774	60,310	

A review of **Table 6.2** indicates the following:

- In the period from 2012 to 2016, the average day flow recorded at the Collingwood WWTP was 16,300m^3/d. This value equates to 63% of the rated capacity of the facility of 25,548m^3/d; and,
- Over the period from 2012 to 2016, one bypass event was recorded in 2016. This event occurred on March 28, 2016 as a result of a rainfall / snowmelt event. A bypass discharge of 50,000m^3 over 11 hours to Georgian Bay was recorded.

In 2017, Town Staff reported that high lake levels interfered with the operation of the wastewater plant. The design Hydraulic Profile for WWTP (Ainley & Associates Limited Dwg: 197014-G2RD) indicates that a high lake level of 177.44m was considered in the hydraulic design of the 1999 plant upgrades. According to Environment Canada, the long term average lake level in Lake Huron is 176.42m. The drawings show a weir elevation at the bypass at 176.524m. In 2017, average month lake levels ranged from 176.47m in January to 177.0m in August 2017.

6.2.2 Future Expansion of the Collingwood WWTP

In May 2011, the Town completed a Schedule C Class Environmental Assessment (EA) for a future expansion of the Collingwood WWTP. The EA study was initiated after a 2005 capacity assessment identified that the average day flow to the plant had reached 85% of the rated capacity. As a condition of the current ECA, MECP now requires proponents to initiate an EA study for additional capacity once a threshold of 80% is reached. The study developed and evaluated options to provide an additional 12,000m^3/d of rated capacity and identified a preferred alternative of maintaining the current facility and providing additional treatment capacity through a compact treatment technology that could be implemented in two (2) 6,000m^3/d increments. The study concluded that expansion would be needed between 2016 and 2028 depending on growth rates and identified that expansion should be triggered when the 3-year average flow reached 80% of plant capacity or 20,438m^3/d.

As part of the EA document, an existing process performance evaluation was completed. The process evaluation identified the status of the Collingwood WWTP, in 2011, in terms of flows, loadings, process capacity, bottlenecks and opportunities and compared the performance of the facility against current guidelines (MECP, Sewage Design Guidelines, 2008). As the plant has not been modified since the completion of the EA study, the study results were reviewed and the major findings discussed below, in context of flows over the past 5-years. It is noted that the EA study considered historical flows from 2004 to 2010, during which time the average flow to the Collingwood WWTP was 16,931m^3/d. During the period from 2011 to 2016, the average flow to the Collingwood WWTP has decreased to a 5-year average of 16,300m^3/d. Major findings of the 2011 study were as follows:

- BOD and TSS removal of 20% and 60% for primary clarification were lower than current design guidelines of 35% and 65%. BOD removals greater than 35% were achieved in selected time periods;
- Aeration basin loading and operating parameters were within the current design guidelines for a non-nitrifying system. The report noted that although the system was not designed for nitrification, the system was currently nitrifying. It was noted that the plant consistently achieved effluent requirements;
- It was noted that the secondary clarifier surface overflow rate exceeded the current design guidelines; and,
- The Dissolved Air Flotation unit provided a good thickened sludge and was operating under parameters less than current design guidelines.

The 2011 ESR identified an expanded Collingwood WWTP would be subject to lower effluent limits and objectives as per the Assimilative Capacity Assessment, completed in 2011.

6.2.2.1 Collingwood WWTP Predicted Peak Flows

The model was used to predict peak flow conveyed to the Collingwood WWTP, predicted bypass flows and the peak wet well depth. **Table 6.3** presents the predicted peak flow reaching the treatment plant bypass chamber, the peak predicted bypass flow and the peak wet well depth under design flow conditions as well as the 2-year 5-year, 10-year, 25-year and historical June 17, 2017 rainfall events. It is important to note that the model is a fully dynamic model and considers the impact of flow attenuation and available storage within the system. As a result, the peak flows predicted to the bypass chamber do not increase significantly for the larger rainfall events. A review of the hydraulic conditions in the upstream sanitary sewer system indicate surcharge conditions which act to attenuate the peak flow predicted at the bypass chamber. It is also noted that the June 17, 2017 event had a return period based on volume of 25 years while the return period based on peak intensity in the range of a 2 year design storm. Hydraulic analyses results for this event show widespread surcharge conditions in the upstream sanitary sewer system with lower peak flows.

Table 6.3 Estimated Existing Peak Flows to Collingwood WWTP

Conditions	Peak Flow to the Bypass Chamber (L/s)	Peak Bypassed Flow (L/s)	Peak Wet Well Depth (m)
Design Flow	473	0	1.62
2-Year Storm	1,368	0	2.98
5-Year Storm	1,413	76.0	3.50

Table 6.3 Estimated Existing Peak Flows to Collingwood WWTP

Conditions	Peak Flow to the Bypass Chamber (L/s)	Peak Bypassed Flow (L/s)	Peak Wet Well Depth (m)
10-Year Storm	1,503	89.5	3.50
25-Year Storm	1,535	146.4	3.96
June 17, 2017 Event	1,427	103.0	3.50

Model results presented in **Table 6.3** indicate that treatment bypass will occur as a result of a 5-year storm event. This is consistent with the historical data which showed one recorded bypass between 2012 and 2017. As the peak treatment capacity of the Collingwood WWTP is 705L/s (60,900m^3/d), **Table 6.3** shows that the peak flow conveyed through the trunk sewer system to the WWTP exceeds the peak treatment capacity during all design storm events. The capacity limitation of the plant impacts the upstream sanitary sewer system and does result in surcharge conditions. **Section 6.2.3** provides further details on these impacts.

6.2.3 Existing Sanitary Sewer System, Pumping Stations and Forcemains

The performance of the existing sanitary sewer system, which encompasses all sanitary sewers, pumping stations and forcemains, was assessed using collected flow monitoring data and the calibrated hydraulic model. Model analyses were completed under the following conditions:

- Peak design flows calculated using the Town's per capita wastewater flow of 450Lpcd and infiltration allowance of 0.23L/s/ha.
- Existing peak dry weather flow plus response to a 2-year design storm event;
- Existing peak dry weather flow plus response to a 5-year design storm event;
- Existing peak dry weather flow plus response to a 10-year design storm event;
- Existing peak dry weather flow plus response to a 25-year design storm event; and
- Existing peak dry weather flow plus response to the June 17, 2017 Assessment event.

It is noted that the Black Ash SPS will be upgraded in 2019. All existing system analyses were completed with the upgraded pumping station in place. The Collingwood WWTP Pumping Station was included in the model so that the impact of the pumping station on the sanitary sewer system could be fully understood. For the purposes of the analysis, the capacity of the Collingwood WWTP Pumping Station was set to the peak capacity of the treatment plant of 60,900m^3/d or 705L/s. The following sections describe the results of the analyses.

The Town has two siphons, located on Spruce and Hickory Streets. These siphons allow wastewater from small areas to be conveyed underneath the Harbourview Trail Trunk Sewer to the First Street sanitary sewer. Discussions with Town Staff have indicated that basement flooding has occurred in the areas served by these siphons as the siphons are prone to plugging.

6.2.3.1 Flow Monitoring Results

Flow monitoring data and rainfall data were collected at a total of 12 locations between May 4, 2017 and October 6, 2017. Details of the flow and rainfall monitoring program are presented in **Appendix D2**. **Figure 6-1** presents the flow monitoring locations and the areas monitored. Over the monitoring period, a total of eight rainfall events occurred which provided sufficient data to characterize wet weather flows at each site. The data collected was analyzed for dry and wet weather flow patterns and results were against the Town's design criteria to identify areas that contribute excessive infiltration and inflow to the system. Three key values were identify to assess inflow and infiltration at each site. These included the dry weather groundwater infiltration, the range of peak infiltration and inflow values measured during rainfall events and the average Cv value. The average Cv value is the percentage of rainfall that flows into the sanitary sewer system. **Table 6.4** presents a summary of the results.

Table 6.4 Flow Monitoring Results Summary

Monitor Site ID	Measured Dry Weather Groundwater Infiltration (L/s/ha)	Range of Measured Peak Infiltration and Inflows (L/s/ha)	Number of Events Where the Measured Peak Infiltration and Inflow Exceeded the Town's Design Allowance of 0.23 L/s/ha	Range of Cv Values (Average Cv Value Over All Events) (%)
FM01	0.05	0.03 – 0.35	1 of 8	0.4 – 3.3 (1.6)
FM02	0.10	0.08 – 0.20	0 of 8	0.2 – 3.3 (1.6)
FM03	0.03	0.06 – 1.19	2 of 8	0.3 – 5.5 (2.9)
FM04	0.04	0.03 – 0.86	3 of 8	0.3 – 12.4 (2.8)
FM05	0.02	0.12 – 1.73	6 of 8	0.8 – 9.4 (3.0)
FM06	0.003	0.13 – 0.49	1 of 5	0.6 – 2.7 (2.2)
FM07	0.02	0.09 – 0.89	2 of 8	1.4 – 8.4 (3.5)
FM08	0.03	0.03 – 0.30	1 of 8	0.1 – 3.7 (1.0)
FM09	0.10	0.12 – 0.77	2 of 5	2.8 – 18.4 (9.6)
FM10	0.10	0.16 – 0.87	2 of 7	1.1 – 6.9 (2.9)
FM11	0.02	0.05 – 0.89	1 of 8	0.3 – 8.5 (2.4)
FM12	0.01	0.12 – 0.50	1 of 2	2.0 – 3.5 (2.8)

Based on the results of the flow monitoring data analysis, all of the monitored areas contributed excessive inflow and infiltration to the system during at least one rainfall event. One site, FM05, was identified as consistently contributing excessive inflow and infiltration to the system over a number of events. The monitored area for FM05 was centered around Hurontario Street from south of First Street to north of Campbell Street.

6.2.3.2 Existing Peak Design Flow Conditions

The performance of the existing sanitary sewer system was assessed under design flow conditions. For this assessment, a per capita wastewater flow of 450Lpcd and an infiltration allowance of 0.23L/s/ha were used to calculate flows. For this condition, the following performance criteria were applied:

- For sanitary sewers, d/D < 0.85
- For pumping stations, firm capacity should exceed the peak incoming flow.

These criteria are the same as those identified in **Section 6.2**.

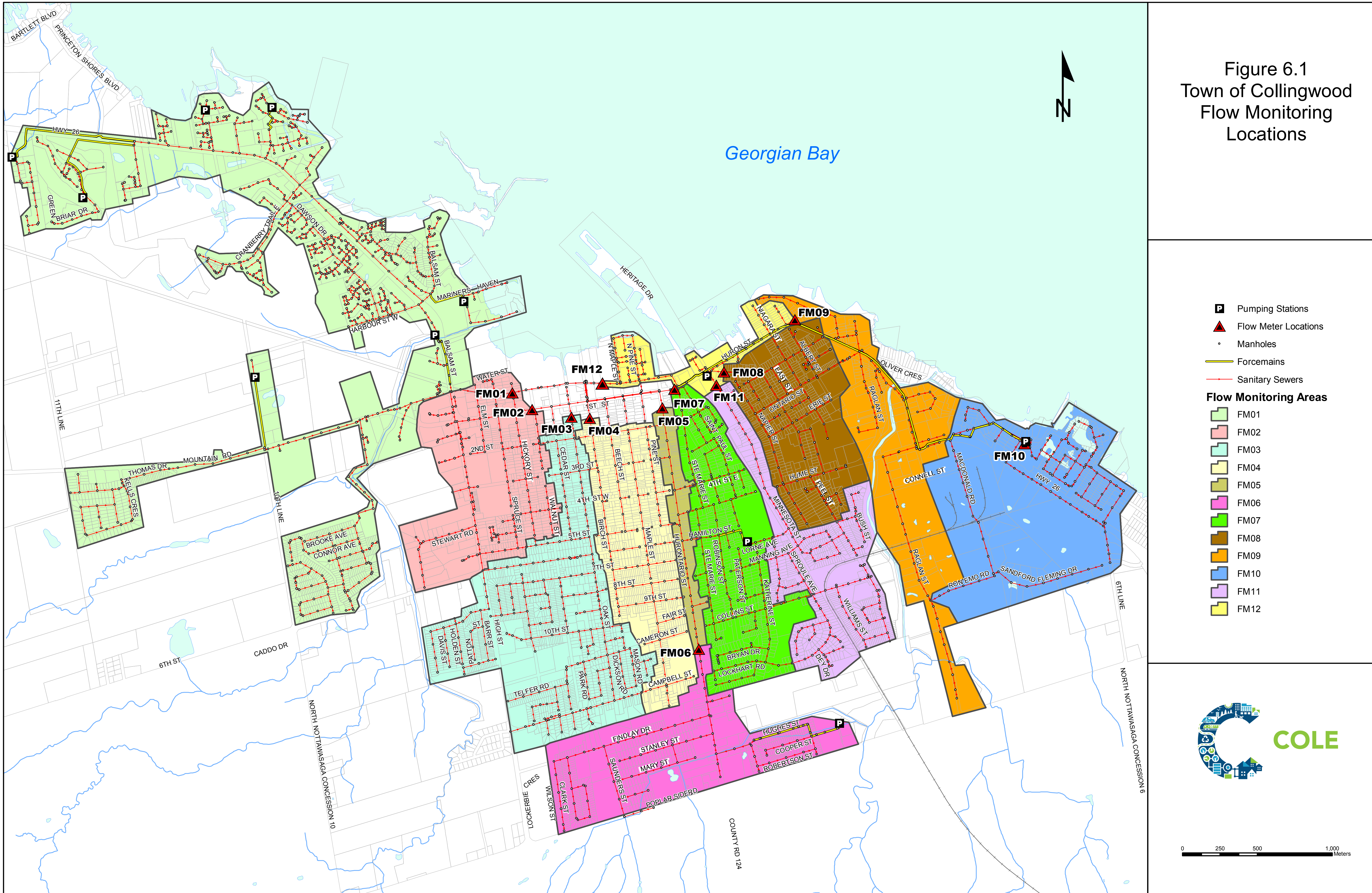


Figure 6.1
Town of Collingwood Flow Monitoring Locations

Figure 6-2 presents the location of sanitary sewers where the d/D ratio exceeded 0.85. It is noted that all sanitary sewers had predicted d/D ratios less than 0.85. There are maintenance holes where the peak hydraulic grade line is within 1.8m of the ground elevation, however, these are all shallow maintenance holes where the upstream and downstream sewers are not surcharged. Therefore, all sanitary sewers met the performance criteria. **Table 6.5** presents the performance of the pumping stations and forcemains.

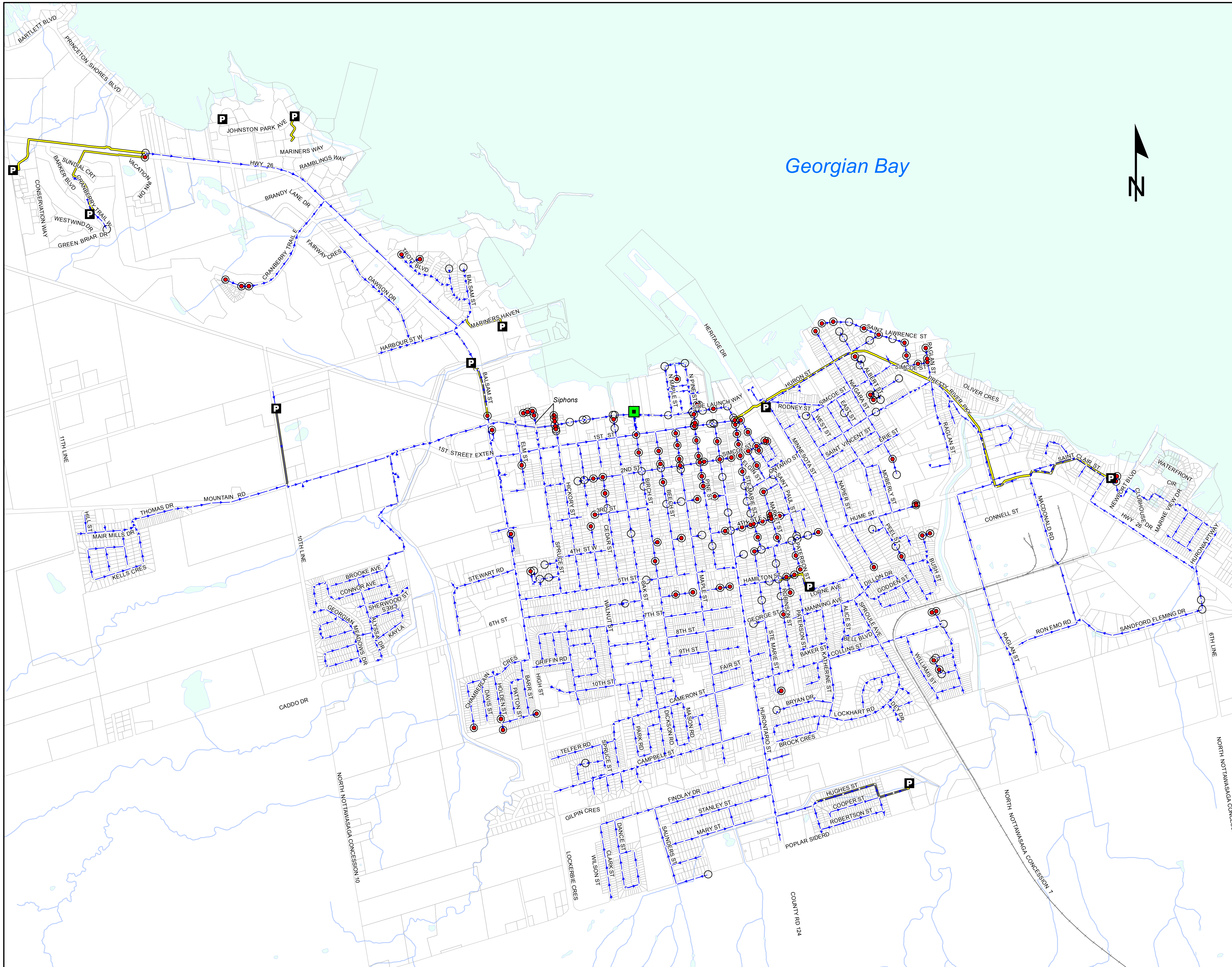
Table 6.5 Pumping Station Performance – Design Flow Conditions

Pumping Station	Peak Modelled Flow Entering Pumping Station (L/s)	Wet Well Depth (m)	Firm Capacity (L/s)	Notes
Black Ash SPS	66	0.8	212	One of three pumps are required to pump design flows. Station has sufficient firm capacity.
Cranberry Trail SPS	5	1.55	32.8	One of two pumps are required to pump design flows. Station has sufficient firm capacity.
Minnesota SPS	113	2.1	210	One of three pumps are required to pump design flows. Station has sufficient firm capacity.
Patterson SPS	23	1.55	72	Two of three pumps are required to pump design flows. Station has sufficient firm capacity.
Pretty River Estates SPS	7	1.25	29	One of two pumps are required to pump design flows. Station has sufficient firm capacity.
St. Clair SPS	31	1.0	155	One of two pumps are required to pump design flows. Station has sufficient firm capacity.
Silver Glen Preserve SPS	4	-	16	One of two pumps are required to pump design flows. Station has sufficient firm capacity.
Collingwood WWTP PS	470	1.62	704	Two of three pumps are required to pump design flows. Station has sufficient firm capacity.

1. Firm capacity of all pumping stations calculated assuming largest pump is out of service.

For all pumping stations, the modelled peak entering the station was less than the firm capacity. The assessment identified that all sanitary sewers and pumping stations meet performance requirements under existing peak design flow conditions.

Figure 6.2
Town of Collingwood
Existing Sanitary Sewer System Performance
Design Flow Conditions



- Pumping Stations
- Birch Street WWTP
- Sanitary Maintenance Holes (<1.8m Freeboard)
- Sanitary Maintenance Holes
- Shallow Maintenance Holes
- Sanitary Sewers
- Forcemains

COLE

0 250 500 1,000 Meters

6.2.3.3 Existing Wet Weather Performance – 2-Year Design Storm Conditions

The performance of the existing sanitary sewer system was assessed under peak existing dry weather flow conditions with a wet weather response resulting from a 2-year design storm event. Model assessments were completed using calibrated peak dry weather flows and calibrated wet weather flows. It should be noted the calibrated peak dry weather flows used differ from the Town's design wastewater allowance for residential areas of 450Lpcd. For the 2-year storm event, performance criteria identified in **Section 6.2** were considered. These criteria are as follows:

- For sanitary sewers, d/D < 0.85
- For pumping stations, the station should have sufficient firm capacity to pump peak flows while maintaining the peak wet well depth below the maximum wet well depth.

These criteria are the same as those identified in **Section 6.2**.

Figure 6-3 presents sanitary sewers where the d/D ratio exceeded 0.85. **Table 6.6** presents the pumping station performance results.

Table 6.6 Pumping Station Performance – Existing Peak Dry Weather Flow with a 2-Year Design Storm

Pumping Station	Peak Modelled Flow Entering Pumping Station (L/s)	Peak Wet Modelled Wet Well Depth (m)	Firm Capacity (L/s)	Maximum Wet Well Depth (m)
Black Ash SPS	90	0.61	212	3.05
Cranberry Trail SPS	8	0.94	32.8	1.75
Minnesota SPS	234	2.12	210	2.69
Patterson SPS	24	1.55	72	2.13
Pretty River Estates SPS	7	0.86	29	2.33
St. Clair SPS	79	0.53	155	4.95
Silver Glen Preserve SPS	7	-	16	-

1. Model does not predict bypass at Black Ash, Minnesota or St. Clair SPS.
2. Firm capacity of all pumping stations calculated assuming largest pump is out of service.

A review of the results shown in **Table 6.6** and **Figure 6-3** indicates performance criteria were met at all pumping stations. Although the peak flow entering the Minnesota SPS is greater than the firm capacity of the station, the storage provided in the wet well equalizes peak flows and the wet well depth does not exceed the maximum wet well depth. Therefore, the criteria is met at this station.

One section of sanitary sewer on 6th Street between Hickory Street and Spruce Street and two sections of the Minnesota Street sanitary sewer south of Simcoe Street were identified as having a d/D of 0.85. The section on 6th Street had a d/D of 0.85.

6.2.3.4 Existing Wet Weather Performance – 5-Year Design Storm Conditions

The performance of the existing sanitary sewer system was assessed under peak existing dry weather flow conditions with a wet weather response resulting from a 5-year design storm event. Model assessments were completed using calibrated peak dry weather flows and calibrated wet weather flows. It should be noted the calibrated peak dry weather flows used differ from the Town's design wastewater allowance for residential areas of 450Lpcd. For the 5-year storm event, performance criteria identified in **Section 6.2** were considered. These criteria are as follows:

- For sanitary sewers, d/D < 0.85
- For pumping stations, the station should have sufficient firm capacity to pump peak flows while maintaining the peak wet well depth below the maximum wet well depth.

Figure 6-4 presents sanitary sewers where the d/D ratio exceeded 0.85. **Table 6.7** presents the pumping station performance results.

Table 6.7 Pumping Station Performance – Existing Peak Dry Weather Flow with a 5-Year Design Storm

Pumping Station	Peak Modelled Flow Entering Pumping Station (L/s)	Peak Wet Modelled Wet Well Depth (m)	Firm Capacity (L/s)	Maximum Wet Well Depth (m)
Black Ash SPS	112	1.05	212	3.05
Cranberry Trail SPS	9	1.55	32.8	1.75
Minnesota SPS	281	2.31	210	2.69
Patterson SPS	34	1.55	72	2.13
Pretty River Estates SPS	9.0	1.26	29	2.33
St. Clair SPS	101	1.01	155	4.95
Silver Glen Preserve	8	-	16	-

1. Model does not predict bypass at Black Ash, Minnesota or St. Clair SPS.
2. Firm capacity of all pumping stations calculated assuming largest pump is out of service.

Performance criteria for this event are met at all pumping stations. It is noted that the peak incoming flow to the Minnesota SPS exceeds the firm capacity of the station. However, the storage provided in the wet well equalizes the peak flow and the peak wet well depth does not exceed the maximum wet well depth. Therefore the criteria is met at this station. A review of the results presented in **Figure 6-4** indicate the following:

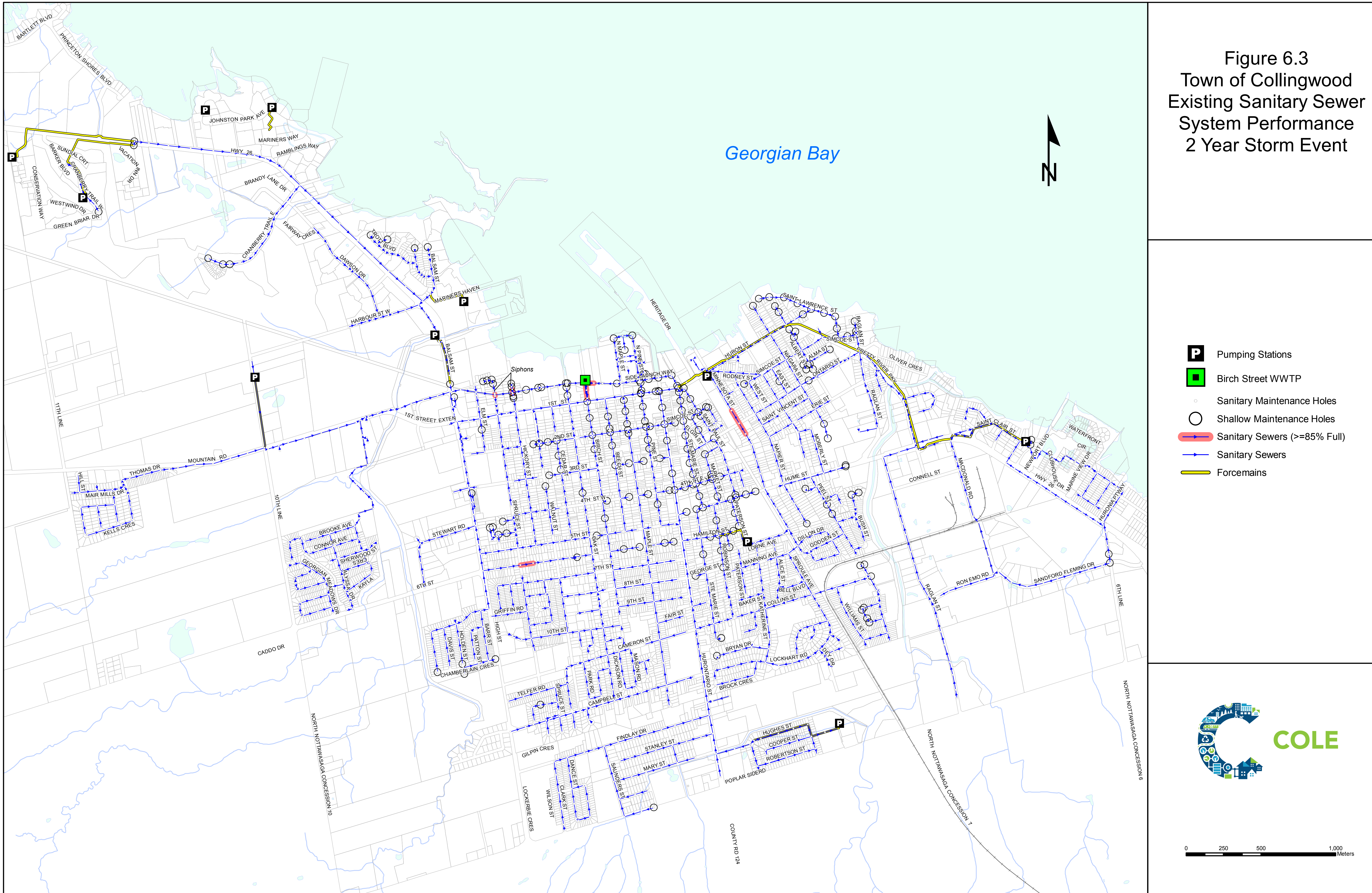


Figure 6.3
Town of Collingwood
Existing Sanitary Sewer
System Performance
2 Year Storm Event

COLE

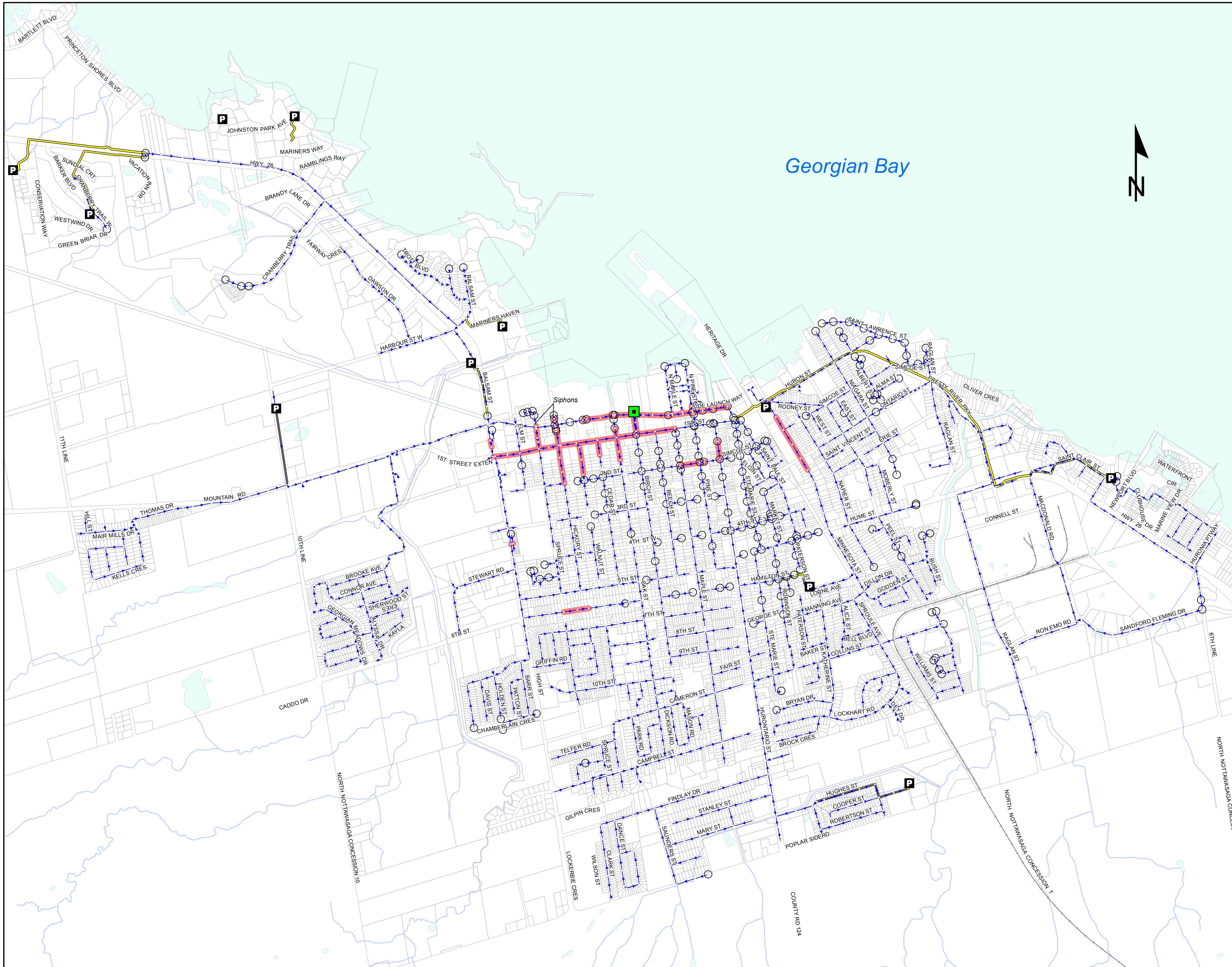


Figure 6.4
Town of Collingwood
Existing Sanitary Sewer System Performance
5 Year Storm Event

- Pumping Stations
- Birch Street WWTP
- Sanitary Maintenance Holes
- Shallow Maintenance Holes
- Sanitary Sewers (>=85% Full)
- Sanitary Sewers
- Forcemains

COLE

0 250 500 1,000 Meters

- Hydraulic limitations at the Collingwood WWTP resulted in surcharge conditions in sections of sanitary trunk. Affected trunk sewers included the Harbourview Trail Trunk Sewer from Cedar Street to Birch Street, the Harbourview Trail Trunk Sewer from Hurontario Street to Birch Street, the First Street sanitary sewer from High Street to Birch Street, the First Street sanitary sewer from Beech Street to Birch Street and the Birch Street sanitary sewer from First Street to the WWTP.
- The criteria were not met in selected sections of sanitary sewer including three sections on Minnesota Street (south of Simcoe Street), Second Street (between Pine and Maple), Hurontario Street (Second Street to First Street), two sections on Sixth Street (Spruce to Walnut) and one section on High (north of Stewart Road).

6.2.3.5 Existing Wet Weather Performance – 10 Year Design Storm Conditions

The performance of the existing sanitary sewer system was assessed under peak existing dry weather flow conditions with a wet weather response resulting from a 10-year design storm event. Model assessments were completed using calibrated peak dry weather flows and calibrated wet weather flows. It should be noted the calibrated peak dry weather flows used differ from the Town's design wastewater allowance for residential areas of 450Lpcd. For the 10-year storm event, performance criteria identified in **Section 6.2** were considered. These criteria are as follows:

- For sanitary sewers, d/D < 0.85
- For pumping stations, the station should have sufficient firm capacity to pump peak flows while maintaining the peak wet well depth below the maximum wet well depth.

Figure 6-5 presents sanitary sewers where the above criteria is not met. **Table 6.8** presents the pumping station performance results.

Table 6.8 Pumping Station Performance – Existing Peak Dry Weather Flow with a 10-Year Design Storm

Pumping Station	Peak Modelled Flow Entering Pumping Station (L/s)	Peak Wet Modelled Wet Well Depth (m)	Firm Capacity (L/s)	Maximum Wet Well Depth (m) Notes
Black Ash SPS	128	1.05	212	3.05
Cranberry Trail SPS	11	1.55	32.8	1.75
Minnesota SPS	314	2.31	210	2.69
Patterson SPS	42	1.55	72	2.13
Pretty River Estates SPS	10	1.26	29	2.33
St. Clair SPS	119	1.01	155	4.95
Silver Glen Preserve SPS	9	-	16	-

1. Model does not predict bypass at Black Ash, Minnesota or St. Clair SPS.
2. Firm capacity of all pumping stations calculated assuming largest pump is out of service.

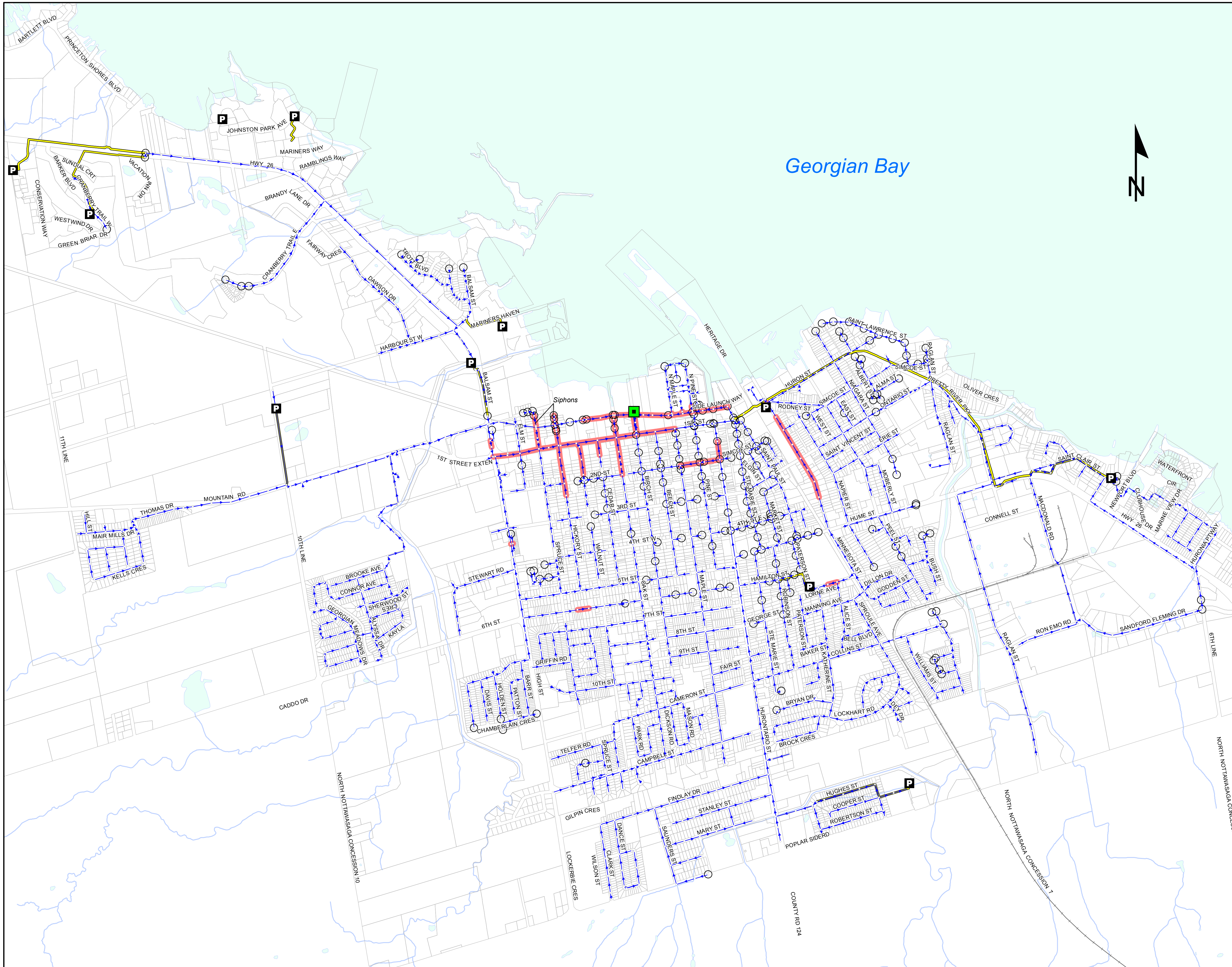


Figure 6.5
Town of Collingwood
Existing Sanitary Sewer System Performance
10 Year Storm Event

- Pumping Stations
- Birch Street WWTP
- Sanitary Maintenance Holes
- Shallow Maintenance Holes
- Sanitary Sewers (>=85% Full)
- Sanitary Sewers
- Forcemains

Performance criteria for this event are met at all pumping stations. At the Minnesota SPS, all three pumps are required to pump the incoming peak flow and maintain the peak wet well depth below the maximum wet well depth. A review of the results presented in **Figure 6-5** indicate the following:

- Similar to the results of the 5-year storm event, hydraulic limitations at the Collingwood WWTP resulted in surcharge conditions in sections of sanitary trunk sewers. Affected trunk sewers included the Harbourview Trail Trunk Sewer from Cedar Street to Birch Street, the Harbourview Trail Trunk Sewer from Ste. Marie to Birch, the First Street sanitary sewer from High to Maple and the Birch Street sanitary sewer from First Street to the WWTP. In sections of these trunk sewers, the peak hydraulic grade line was located within 1.8m of the ground surface. Sanitary sewers located on High Street, Spruce Street, Hickory Street, Walnut Street, Cedar Street and Oak Street which discharge into the First Street sanitary sewer also did meet not this criteria.
- The criteria were not met in selected sections of sanitary sewer including three sections on Minnesota Street (south of Simcoe), Hurontario Street (north of Second), Second Street (Hurontario to Maple), 6th Street (between Spruce and Walnut), High Street (between Mountain Street and First Street) and Lorne Avenue (Alice to Katherine).

6.2.3.6 Existing Wet Weather Performance – 25-Year Design Storm Conditions

The performance of the existing sanitary sewer system was assessed under peak existing dry weather flow conditions with a wet weather response resulting from a 25-year design storm event. Model assessments were completed using calibrated peak dry weather flows and calibrated wet weather flows. It should be noted the calibrated peak dry weather flows used differ from the Town's design wastewater allowance for residential areas of 450Lpcd. For the 25-year storm event, performance criteria identified in **Section 6.2** were considered. These criteria are as follows:

- For sanitary sewers, surcharge conditions are acceptable if the peak hydraulic grade line remains 1.8m or below the ground surface. Where this criteria is not met due to a shallow sewer, the sanitary sewer should not be surcharged.
- For pumping stations, the station is to have sufficient station capacity to pump incoming flows while maintaining the peak predicted water level in the wet well below the maximum wet well depth.

Figure 6-6 presents sanitary sewers where the above criteria is not met.

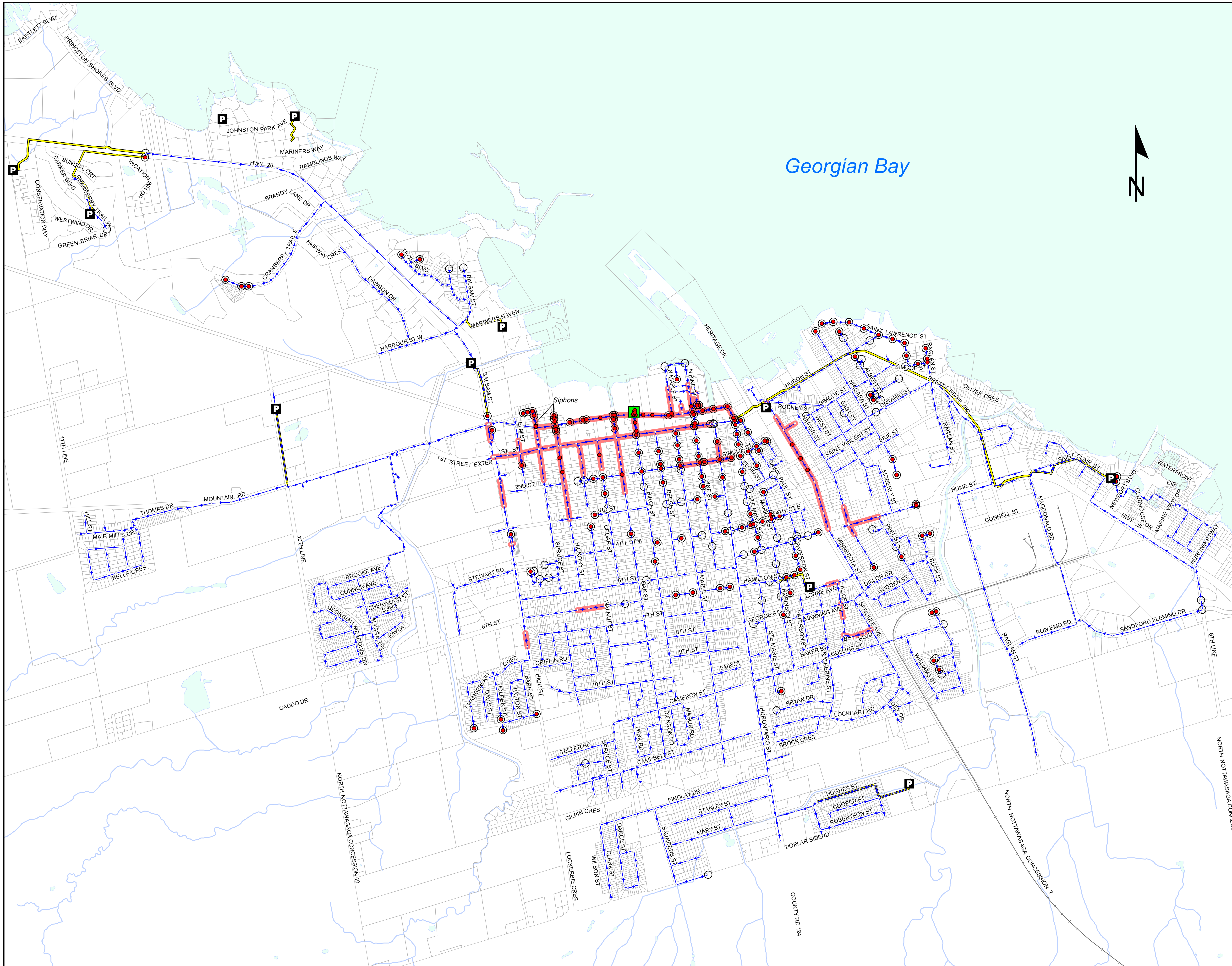


Figure 6.6
Town of Collingwood
Existing Sanitary Sewer System Performance
25 Year Storm Event

- Pumping Stations
- Birch Street WWTP
- Sanitary Maintenance Holes (<1.8m Freeboard)
- Sanitary Maintenance Holes
- Shallow Maintenance Holes
- Sanitary Sewers (>=85% Full)
- Sanitary Sewers
- Forcemains

0 250 500 1,000 Meters

Table 6.9 presents the pumping station performance results.

Table 6.9 Pumping Station Performance – Existing Peak Dry Weather Flow with a 25-Year Design Storm

Pumping Station	Peak Modelled Flow Entering Pumping Station (L/s)	Peak Wet Modelled Wet Well Depth (m)	Firm Capacity (L/s)	Maximum Wet Well Depth (m) Notes
Black Ash SPS	144	1.05	212	3.05
Cranberry Trail SPS	12	1.55	32.8	1.75
Minnesota SPS	340	2.31	210	2.69
Patterson SPS	50	1.55	72	2.13
Pretty River Estates SPS	12	1.26	29	2.33
St. Clair SPS	136	1.01	155	4.95
Silver Glen Preserve SPS	10	-	16	-

1. Model does not predict bypass at Black Ash, Minnesota or St. Clair SPS.
2. Firm capacity of all pumping stations calculated assuming largest pump is out of service.

At the Minnesota SPS, all three pumps are required to peak the incoming peak flow and maintain the peak wet well level below the maximum wet well level. Performance criteria for this event are met at all pumping stations. A review of the results presented in **Figure 6-6** indicate the follows:

- Similar to the results of the 5-year and 10-year storm events, hydraulic limitations at the Collingwood WWTP resulted in surcharge conditions in sections of sanitary trunk sewers. Affected trunk sewers included the Harbourview Trail Trunk Sewer from Hickory to Hurontario, the First Street sanitary sewer from High to Hurontario and the Birch Street sanitary sewer from First to the Collingwood WWTP. In some sections of these trunk sewers, the peak hydraulic grade line was located within 1.8m of the ground surface.
- For the 25-year storm event, surcharge conditions are acceptable as long as the peak hydraulic grade line is more than 1.8m below the ground surface. These criteria were not met for selected sections of sanitary sewer including Second Street (Hurontario to Maple).

6.2.3.7 June 17, 2017 Assessment Event

The performance of the existing sanitary sewer system was assessed under peak existing dry weather flow conditions with a wet weather response resulting from the historical rainfall event that occurred on June 17, 2017. Model assessments were completed using calibrated peak dry weather flows and calibrated wet weather flows. It should be noted the calibrated peak dry weather flows used differ from the Town's design wastewater allowance for residential areas of 450Lpcd. For this event, the performance criteria identified in **Section 6.2** were considered. These criteria are as follows:

- For sanitary sewers, surcharge conditions are acceptable if the peak hydraulic grade line remains 1.8m or below the ground surface. Where this criteria is not met due to a shallow sewer, the sanitary sewer should not be surcharged.

- For pumping stations, the station is to have sufficient station capacity to pump incoming flows while maintaining the peak predicted water level in the wet well below the maximum wet well depth.

Figure 6-7 presents sanitary sewers where the above criteria is not met. **Table 6.10** presents the pumping station performance results.

Table 6.10 Pumping Station Performance – Existing Peak Dry Weather Flow with the June 17, 2017 Storm Event

Pumping Station	Peak Modelled Flow Entering Pumping Station (L/s)	Peak Wet Modelled Wet Well Depth (m)	Firm Capacity (L/s)	Maximum Wet Well Depth (m) Notes
Black Ash SPS	115	1.05	212	3.05
Cranberry Trail SPS	10	1.55	32.8	1.75
Minnesota SPS	286	2.31	210	2.69
Patterson SPS	33	1.55	72	2.13
Pretty River Estates SPS	7	1.25	29	2.33
St. Clair SPS	101	1.01	155	4.95
Silver Glen Preserve SPS	8	-	16	-

1. Model does not predict bypass at Black Ash, Minnesota or St. Clair SPS.
2. Firm capacity of all pumping stations calculated assuming largest pump is out of service.

At the Minnesota SPS, all three pumps are required to peak the incoming peak flow and maintain the peak wet well level below the maximum wet well level. Performance criteria for this event are met at all pumping stations. A review of the results presented in **Figure 6-7** indicate the follows:

- Similar to the results of the 5-year and 10-year storm events, hydraulic limitations at the Collingwood WWTP resulted in surcharge conditions in sections of sanitary trunk sewers. Affected trunk sewers included the Harbourview Trail Trunk Sewer from Spruce to Ste. Marie, the First Street sanitary sewer from High to Maple and the Birch Street sanitary sewer from First to the Collingwood WWTP. In some sections of the Harbourview Trail Trunk Sewer and the Birch Street Sewer, the peak hydraulic grade line was located within 1.8m of the ground surface. Surcharge conditions extended into the Shipyards area on North Pine and North Maple. These results are consistent with the reports of basement flooding that occurred in the Shipyards area.

- The criteria were not met in selected sections of sanitary sewer including six sections on Minnesota Street (south of Simcoe).

6.2.3.8 Impact of the Collingwood WWTP on Sanitary Sewer System

For the analyses completed of existing conditions, surcharge conditions were identified in the sanitary collection system upstream of the Collingwood WWTP as a result of all wet weather events. To identify sanitary collection system limitations, independent of the treatment plant, a model scenario was created to analyze the impact of a 25-year design storm event with a free outfall, or no restriction, at the Collingwood WWTP. **Figure 6-8** presents the sanitary sewers where surcharge conditions are predicted.

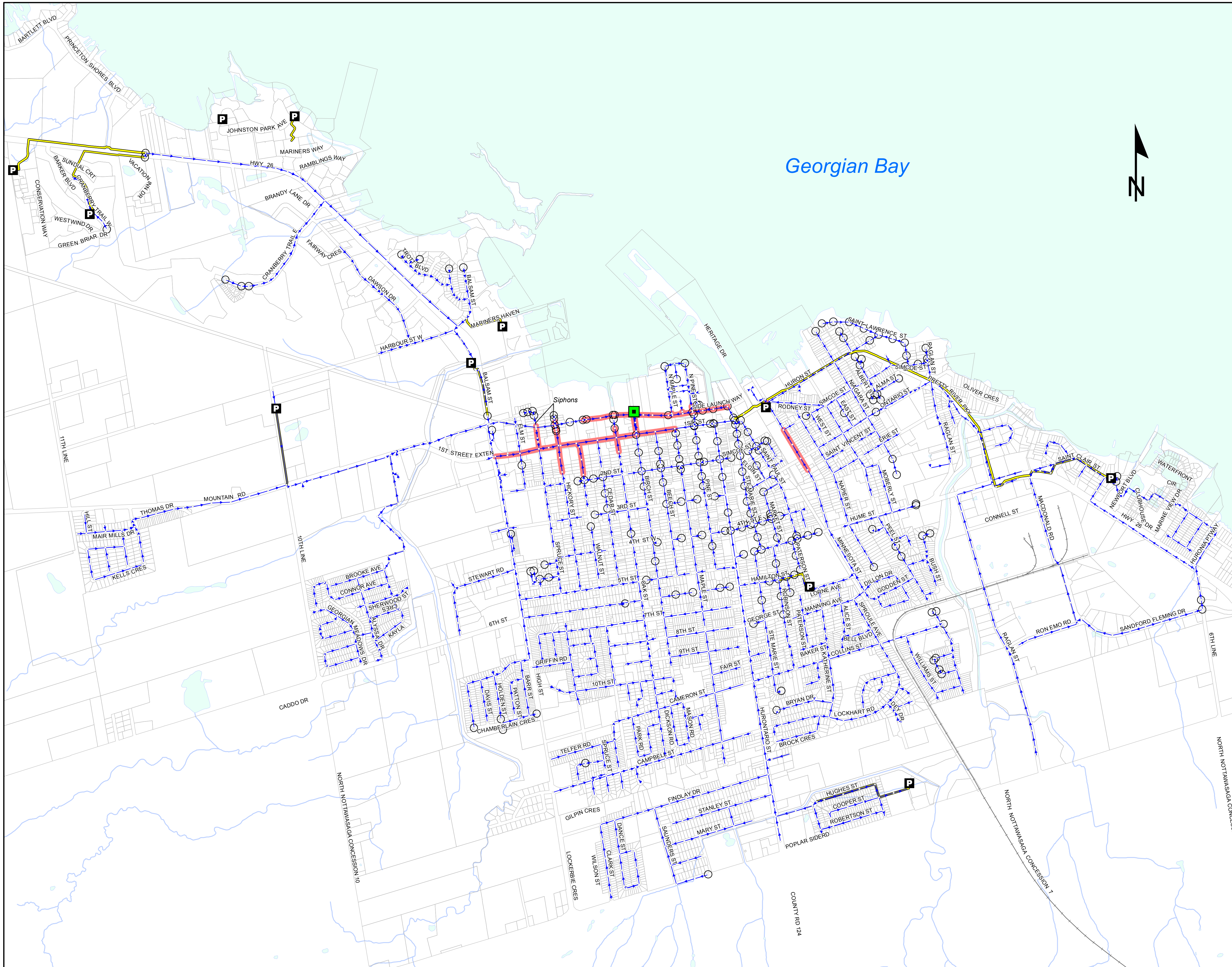
Figure 6-8 shows that in general, Collingwood's trunk sanitary sewer system has adequate capacity to convey peak flows under existing conditions for a 25-year design storm event as surcharge conditions were not predicted in the Harbourview Trail Trunk Sewer system upstream of the WWTP. Localized surcharge conditions are noted in selected sewers located on Minnesota Street, First Street, Second Street, Hurontario Street and Hume Street. Surcharge conditions were also noted in the First Street sanitary sewer.

6.2.4 Summary

An overview of the existing sanitary system identified the following:

- The Collingwood WWTP is currently operating at 63% of the average rated capacity. Data from 2012 – 2016 indicates that there is a slight downward trend in average day flows.
- Existing siphon structures at Spruce and Hickory Streets have had operational issues which has resulted in flooding of upstream properties.
- Peak flows entering the Collingwood WWTP during wet well events exceeds the peak flow capacity of the plant. This limitation does result in surcharge conditions in the trunk sewer system.
- All of the Town's pumping stations have adequate capacity to pump peak flows and meet performance criteria under a series of storm events.
- The Black Ash SPS forcemain capacity is equal to the firm capacity of the station and is lower than the station capacity. All other forcemains have sufficient capacity.
- For the most part, the existing sanitary sewer system meets performance criteria. The capacity of the downstream trunk sewers (Harbourfront Trail Trunk Sewer and First Street sanitary sewer) are limited by Collingwood WWTP.
- There are limitations within existing sanitary sewers where performance criteria is not met, when the capacity limitation at the Collingwood WWTP is not considered. These capacity limited sanitary sewers are located on Minnesota Street, Hurontario Street and Second Street. Minor exceedance of the criteria was noted for two sections located on Lorne Avenue and on Sixth Street.

Figure 6.7
Town of Collingwood
Existing Sanitary Sewer System Performance
June 17, 2017
Storm Event



P Pumping Stations

Birch Street WWTP

Sanitary Maintenance Holes

Shallow Maintenance Holes

Sanitary Sewers (>=85% Full)

Sanitary Sewers

Forcemains

COLE

0 250 500 1,000 Meters

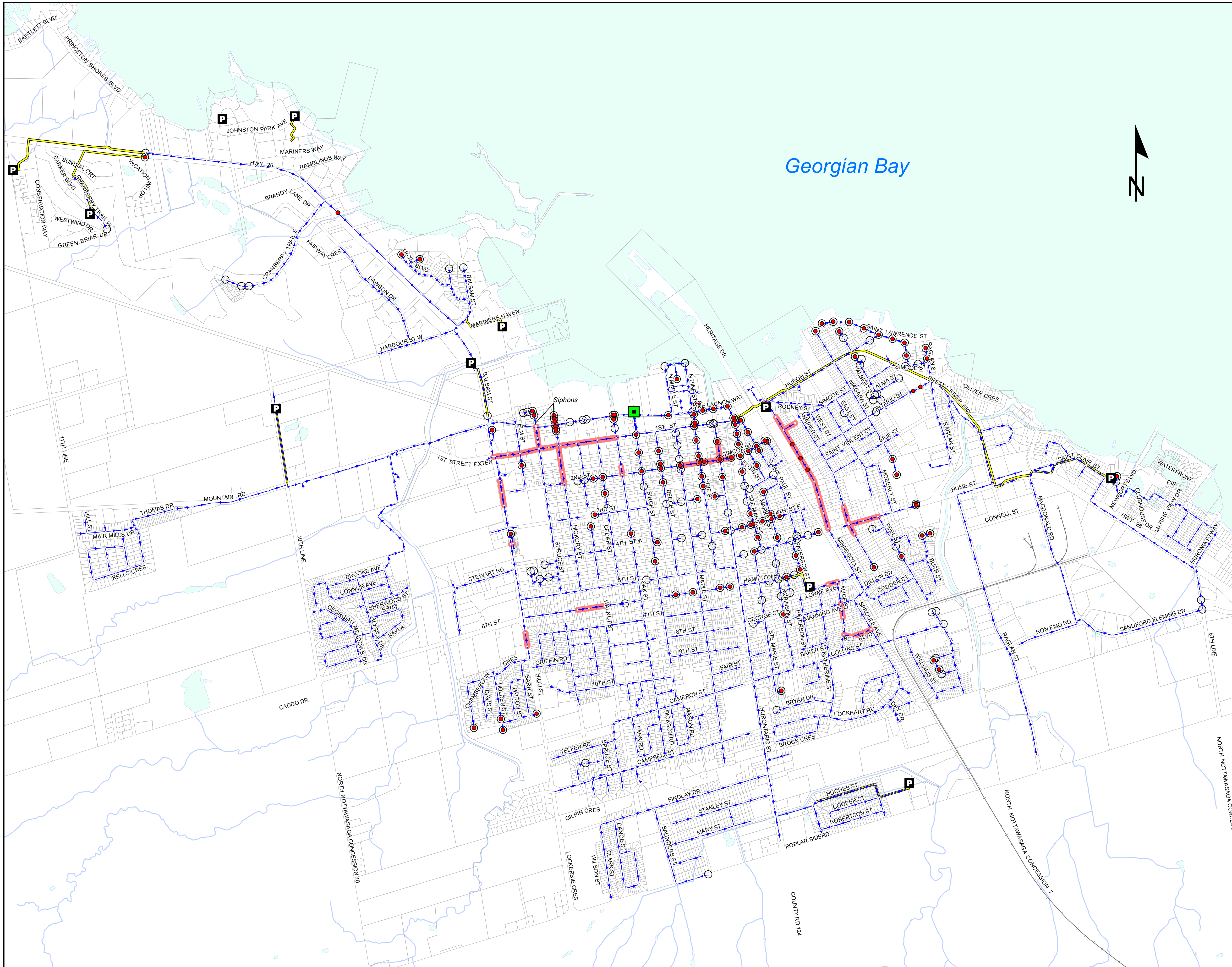


Figure 6.8
Town of Collingwood
Existing Sanitary Sewer
System Performance
25 Year Storm with Free
Outfall at Collingwood
WWTP

- Pumping Stations
- Birch Street WWTP
- Sanitary Maintenance Holes (<1.8m Freeboard)
- Sanitary Maintenance Holes
- Shallow Maintenance Holes
- Sanitary Sewers (>=85% Full)
- Sanitary Sewers
- Forcemains

COLE

0 250 500 1,000 Meters

7 Future Growth and Needs Assessment

Water and sanitary future needs to service future growth were assessed using the tools and models used to assess existing performance. Prior to completing the assessment, analysis of current demands was completed to rationalize the selection of water use and sanitary generation rates for use in this study. **Appendix E** contains the results of the assessment. **Table 7.1** presents the unit rates utilized as part of this study. **Table 7.2** presents the peaking factor used for the assessments.

Table 7.1 Master Plan Unit Water and Sanitary Rates

Demand Type	Historical	Existing Guidelines	Rates Used in this Master Plan
Water System			
Residential	210L/cap/d	450L/cap/d	210L/cap/d
ICI	150L/cap/d		150L/cap/d
Non-Revenue Water	50L/cap/d		50L/cap/d
Total	410L/cap/d		410L/cap/d
Sanitary System			
Residential	249L/cap/d	450L/cap/d	260L/cap/d
ICI	15.3m^3/ha/d		21.6m^3/ha/d
Infiltration for Pumping and Treatment		90L/cap/d (Residential) 6.4m^3/ha/d (ICI)	90L/cap/d (residential) 6.4m^3/ha/d (ICI)
Peak infiltration allowance for new developments	-	0.23 /s/ha	0.23L/s/ha

Table 7.2 Recommended Multiplication Factors

Criteria	Historic	Source	Existing Guidelines	Recommended
Residential Population Density	2.2 /unit	2016 Census	2.9 /unit (single family) 2.7 /unit (semi-detched) 2.4 /unit (townhouse) 1.9 /unit (apartment)	2.9 /unit 2.7 /unit 2.4 /unit 1.9 /unit
MDD Peaking Factor	1.55 – 1.77 (MDD/ADD)	WTP Flow Data 2011 - 2016	2.0 (MDD/ADD)	1.77 (MDD/ADD)
PHD Peaking Factor	1.3 (PHD/MDD)	WTP Flow Data 2011 - 2016	4.5 (PHD/ADD) or 2.25 (PHD/MDD)	1.3 (PHD/MDD)
Sanitary Peaking Factor				Harmon equation

7.1 Water System Needs Assessment

7.1.1 Water Supply and Demand

The calculated demands for future phases of development in the Town of Collingwood and the existing water taking rates for neighbouring municipalities were considered in the analysis of future water supply and demand. This approach was taken to first determine deficiencies and requirements for the Town of Collingwood alone before considering needs of neighbouring municipalities. Requests for additional water taking are addressed separately in **Section 9**. **Table 7.3** lists the calculated demands for Collingwood, and the existing demands for ToBM and New Tecumseth used in the future scenarios.

Table 7.3 Existing and Future Water Demands (m^3/d)

Location	Existing	Planned (2032)	Potential (2044)	Built Boundary
Collingwood ADD	8,884	13,954	17,903	26,490
Collingwood MDD	15,152	24,126	31,116	46,315
ToBM	1,250	1,250	1,250	1,250
New Tecumseth	9,500	9,500	9,500	9,500
Total ADD	19,634	24,704	28,653	37,240
Total MDD	25,902	34,876	41,866	57,065

The future ADD and MDD requirements were compared to the available supply of 31,140m^3/day from the WTP, and are shown in **Figure 7-1. Figure 7-2** shows that the total MDD for Collingwood, ToBM and NT and combined is expected to exceed the capacity of the WTP before the Planned (2032) scenario. The Town has adopted an 80% trigger for planned upgrades to provide a conservative time buffer. The 2016 MDD are relatively consistent with previous years and are considered a reasonable estimate of the existing MDD conditions. The summer of 2016 was an extremely hot and dry summer and is therefore a conservative indicator of MDD.

To forecast a WTP upgrade timeline, existing maximum WTP production (31,140m^3/day) was compared with 2032 MDD (34,876m^3/day). Assuming a linear development projection between 2016 and 2032, the MDD is expected to hit the WTP 80% factor approximately 1 year from existing demands. If a 90% factor was utilized, then the WTP upgrades would be required by the end of 2019. If a 100% factor was used, then WTP upgrades would be required by early 2025. By the end of the current planning horizon, if the Town is developed up to the built boundary the total required treatment capacity to supply MDD is projected to be 57,065m^3/day. The ADD requirement is expected to be approximately 37,240m^3/day.

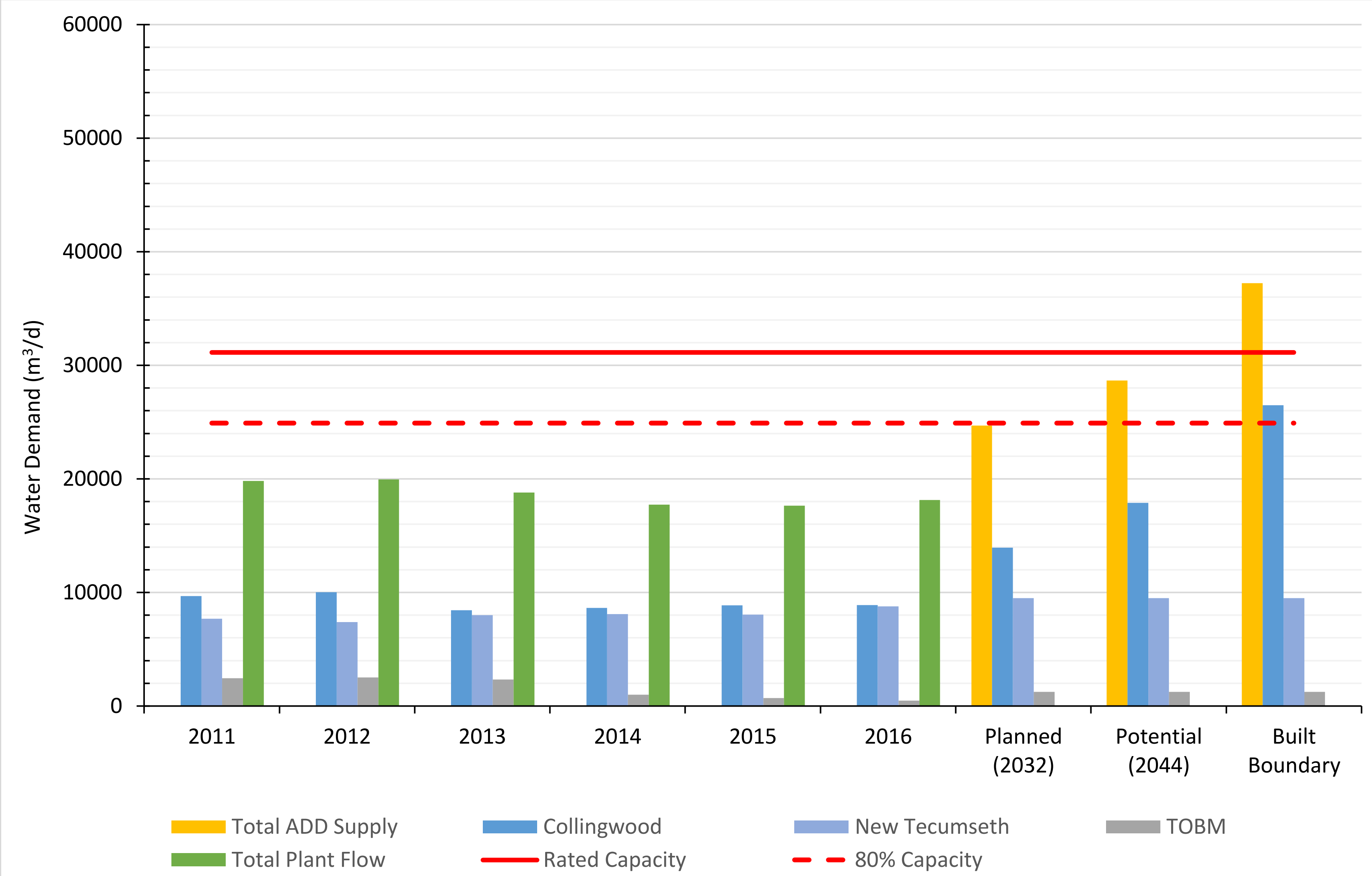


Figure 7-1 Future ADD Demand and Supply Comparison

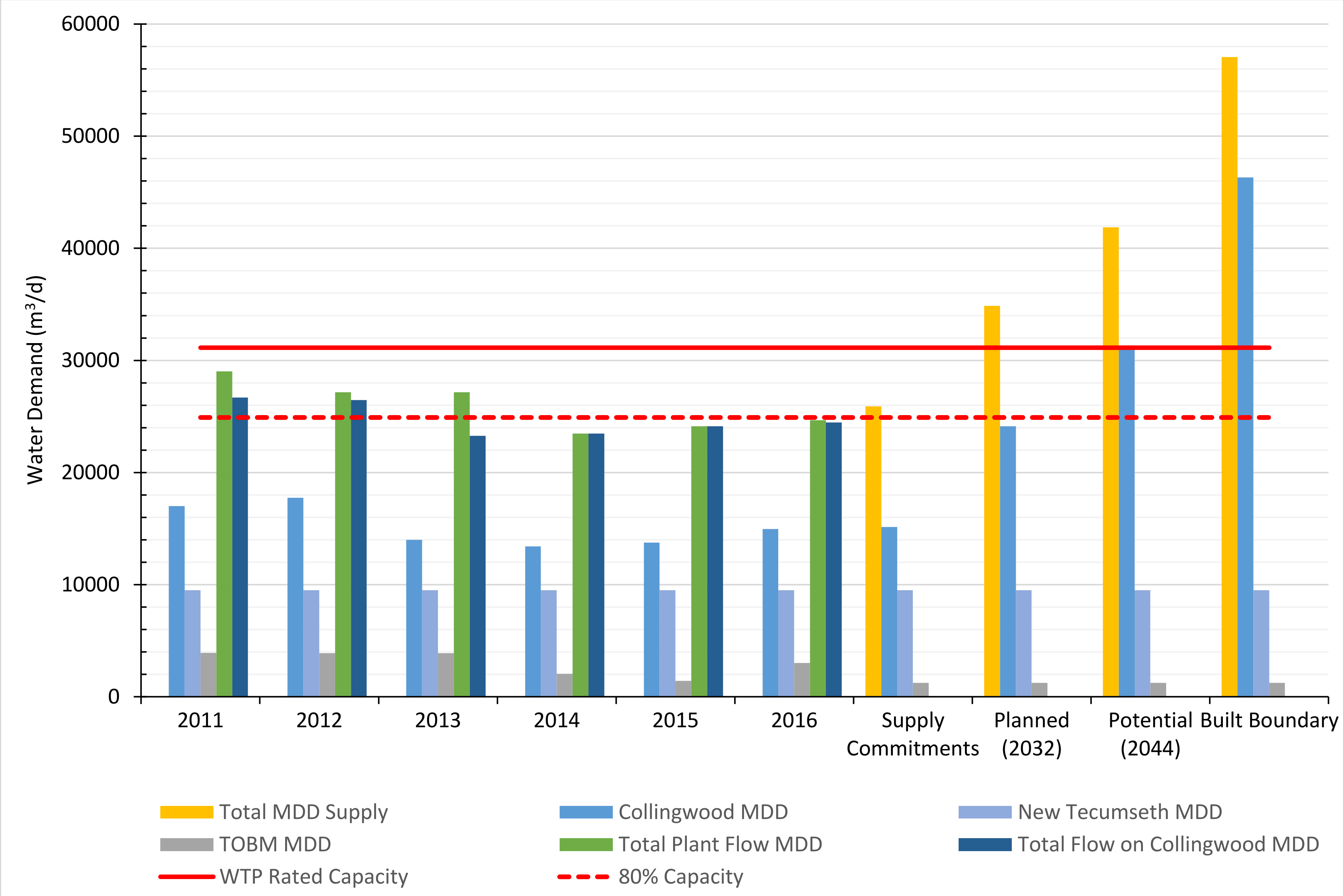


Figure 7-2 Future MDD Demand and Supply Comparison

7.1.2 Water Storage

Table 7.4 summarizes existing and anticipated water storage volumes. In this section, the term 'anticipated' refers to storage that has been designed prior to the Master Plan and is being planned for construction. Future phases and possible expansions of existing and planned storage facilities have not been included in the baseline desktop storage analysis so that the timing and size requirements of additional storage can be determined. Storage upgrades, including addition of new cells at Carmichael, Davey and Stewart Road Reservoirs, are analyzed in **Section 8.2** as alternatives to address deficiencies.

The WTP, Carmichael Reservoir and Collingwood ET have no anticipated storage increases prior to the Master Plan. The Carmichael Reservoir has space for the addition of a storage new cell, but expansion is not yet anticipated. The first phase of the Stewart Road Reservoir has been designed to have a storage volume of 1,540m^3. The Stewart Road BPS has been designed to allow for future reservoir expansion of an additional 1,615m^3 in each of Phase 2 and 3, resulting in a total of 4,770m^3. The storage values for Phase 2 and 3 have not been included in the baseline analysis as explained above. Davey Reservoir currently has 2,565m^3 of storage between two cells, with room to add a third and fourth cell to the reservoir of similar size, but this expansion has not yet been designed.

Table 7.4 Existing and Anticipated Storage Capacity (m^3)

Facility	Existing	Planned (2032)	Potential (2044)	Built Boundary
Zone 1				
WTP Clearwell *(excluded from total available since volume is needed for contact time)*	*797*	*797*	*797*	*797*
Existing Carmichael Reservoir	6,800	6,800	6,800	6,800
Existing Collingwood Water Tower	1,685	1,685	1,685	1,685
Zone 2				
Existing Davey Reservoir	2,565	2,565	2,565	2,565
Future Stewart Road Phase 1	-	1,540	1,540	1,540
Summary				
Total (Zone 1 & 2)	11,050	12,590	12,590	12,590
Zone 1	8,485	8,485	8,485	8,485
Zone 2	2,565	4,105	4,105	4,105
Zone 3	-	-	-	-

The available storage was compared to the future storage requirements. Required storage was calculated using the MECP method (MECP Guidelines for Drinking Water Systems 2008), including fire, equalization and emergency storage. The amount of storage required by the Town is calculated as follows, and values are provided in **Table 7.5;**

- A – Fire Flow Storage = 189L/s for 2.5 hours
- B – Equalization Storage = MDD * 25%
- C – Emergency Storage = (A + B) * 25%

Table 7.5 Required Storage (m^3)

Location	Existing	Planned (2032)	Potential (2044)	Built Boundary
Total Zone 1, 2 &3	6,861	9,666	11,850	16,600
Zone 1	6,387	8,641	10,092	10,802
Zone 2	2,747	3,714	5,170	8,273
Zone 3	-	-	-	2,992

Note that the total requirements for Zone 1, 2 and 3 is not equal to the sum of the total for each individual zone. Summing the individual zone requirements would duplicate the fire flow and emergency storage requirements. The amount of storage required to supply the ToBM's water taking rate was not included in the calculations. Calculations of required storage for each scenario are provided in **Appendix F**. The difference between available and required storage for each zone is shown in **Table 7.6**, and the comparisons are shown graphically in **Figure 7-3**. **Figures 7-4, 7-5** and **7-6** present the comparison of available and required storage for each zone.

The total available and anticipated storage in the system allows for future growth up until the Potential (2044) scenario.

There is currently no additional storage anticipated for Zone 1. The available storage does not meet the zone's storage requirements by 2032 and additional storage would be required. It is anticipated that new storage will be required in Zone 1 by approximately 2030 based on a linear growth rate between 2016 and 2032.

The addition of storage in Phase 1 of the Stewart Road BPS and reservoir construction is expected to meet requirements for Zone 2 in the Planned 2032 scenarios. It is anticipated that new storage will be required in Zone 2 by approximately 2035 based on a linear growth rate between 2032 and 2044.

Zone 3 has no anticipated storage at this time but is estimated to require approximately 3,000m^3 if the Town develops all of the available land up to the built boundary. It is anticipated that new storage will be required in Zone 3 by whenever the area defined as Zone 3 develops.

Table 7.6 Difference in Storage (Available – Required) (m^3)

Location	Existing	Planned (2032)	Potential (2044)	Built Boundary
Total Zone 1, 2 &3	4,189	2,924	740	-4,010
Zone 1	2,098	-156	-1,607	-2,317
Zone 2	-182	391	-1,065	-4,168
Zone 3	-	-	-	-2,992

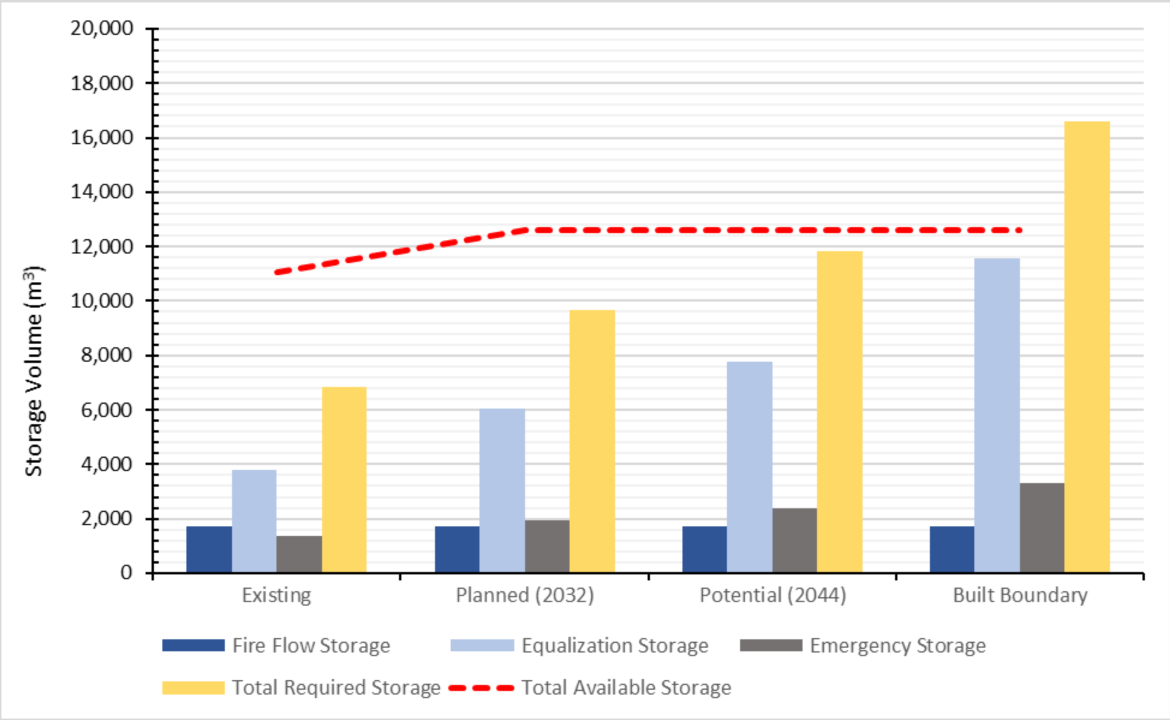


Figure 7-3 Total Storage Requirements (Zones 1, 2, and 3)

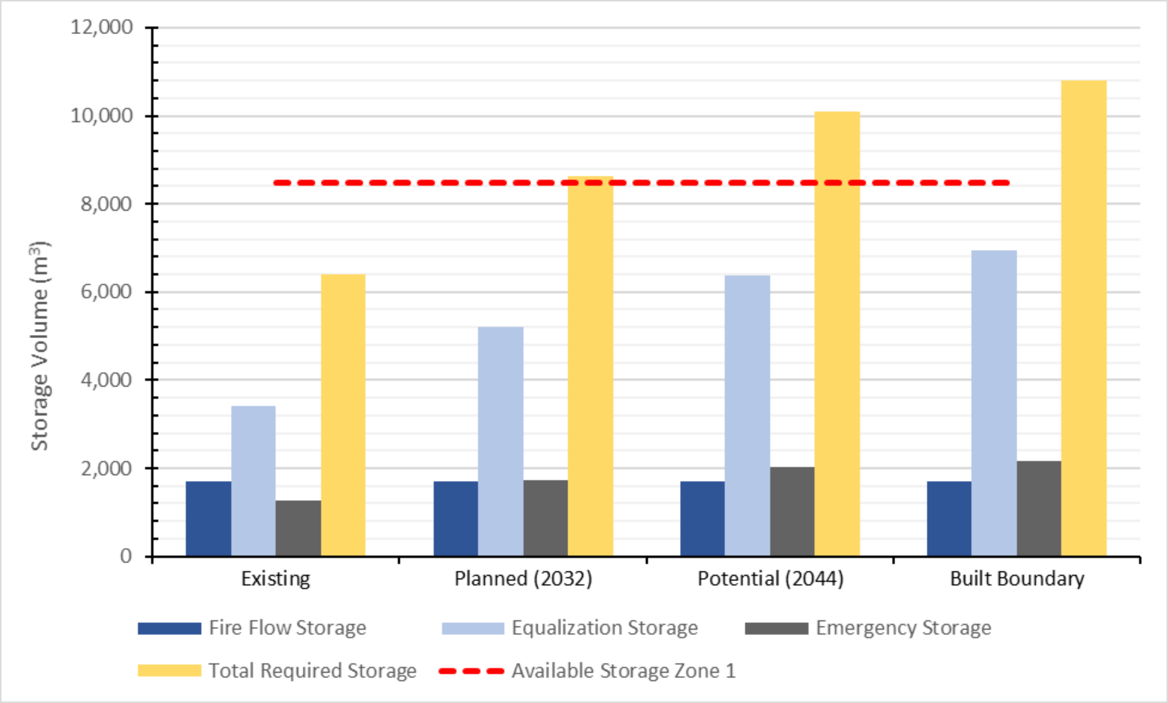


Figure 7-4 Zone 1 Storage Requirements

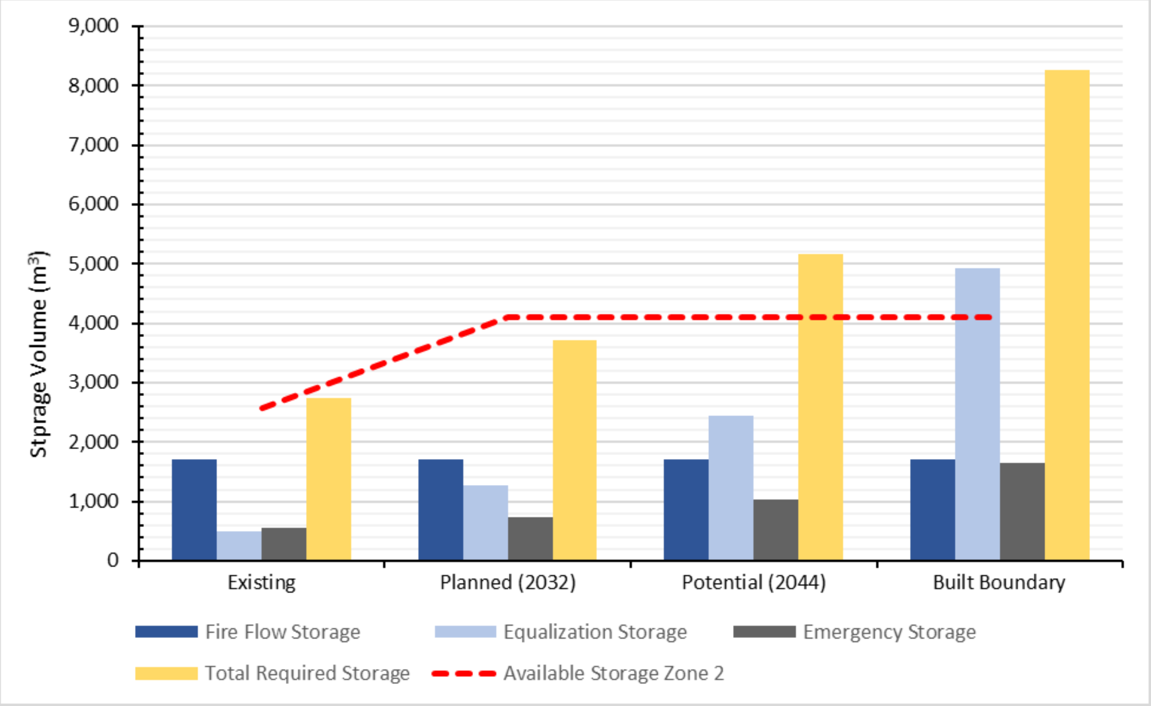


Figure 7-5 Zone 2 Storage Requirements

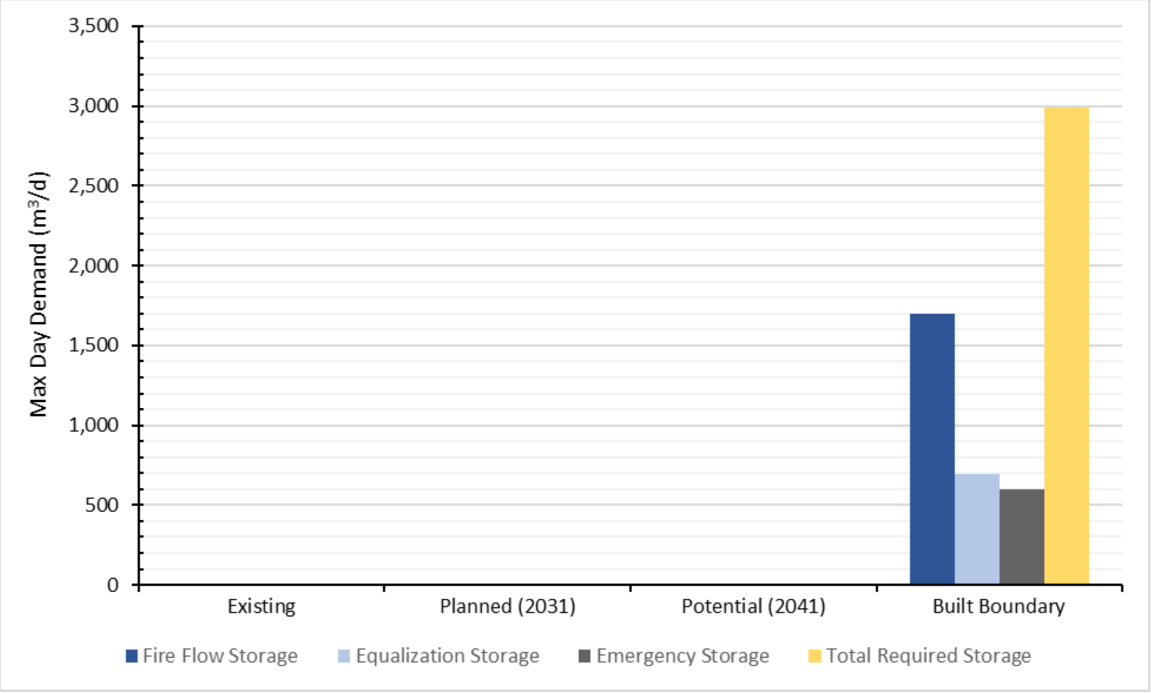


Figure 7-6 Zone 3 Storage Requirements

7.1.3 Pumping Capacity

The existing and planned pumping capacity at each facility is summarized in **Table 7.7**. Future phases and possible expansions of these facilities have not been included in the baseline analysis so that the timing and size requirements of additional pumping can be determined. In this section, the term 'anticipated' refers to pumping that has been designed prior to the Master Plan and is being planned for construction.

There are currently no anticipated pumping upgrades at the WTP and Carmichael BPS. Upgrades that are being assessed as part of the WTP Environment Assessment (EA) and Carmichael conceptual planning are not included in the baseline analysis. The existing firm capacity at Davey BPS was included in the pumping analysis, and the design report indicates that there is an allocated space for a future pump and a planned replacement of a small pump. The first phase of the Stewart Road BPS will have a firm capacity of 105L/s and is included in this analysis. The final phase of Stewart Road BPS per the design report is anticipated to have a firm capacity of 150L/s, but is not included in the calculation to demonstrate when additional pumping will be required. Pumping upgrades are analyzed in **Section 8.2.4** as alternatives to address pumping deficiencies.

Table 7.7 Existing and Anticipated Available Pumping Capacity (L/s)

Facility	Existing	Planned (2032)	Potential (2044)	Built Boundary
Zone 1				
Existing WTP	334	334	334	334
Existing Carmichael BPS	300	300	300	300
Zone 2				
Osler Bluff BPS	87.8	-	-	-
Georgian Meadows BPS	11.8	-	-	-
Existing Davey BPS	170	170	170	170
Stewart Road Phase 1	-	105	105	105
Summary				
Zone 1	634	634	634	634
Zone 2	270	275	275	275
Zone 3	-	-	-	-

Required pumping capacity was calculated as the total of MDD pumping plus fire flow pumping requirements and the values are shown in **Table 7.8**. The MDD pumping requirements for the ToBM were not included in these calculations and are evaluated separately in **Section 9**. The difference between available and required pumping capacity was analyzed by pressure zone and is shown in **Table 7.9** and the comparisons are shown graphically in **Figure 7-7**. **Figures 7-8, 7-9** and **7-10** present the comparisons for each zone.

Table 7.8 Required Pumping (L/s)

Zone	Existing	Planned (2032)	Potential (2044)	Built Boundary
WTP Pumping Supply for MDD	175	279	360	536
MDD Plus Fire Pumping Comparison				
Zone 1	364	468	549	725
Zone 2	212	248	302	449
Zone 3	-	-	-	221

Note that the total requirements for Zone 1, 2 and 3 is not equal to the sum of the total for each individual zone. Summing the individual zone requirements would duplicate the fire flow pumping calculations. The detailed pumping requirement calculations are provided in **Appendix F**.

The pumping capacity required to supply the MDD for the entire system must be supplied by the WTP pumping station. The MDD requirements were compared to the available pumping capacity at the WTP. The analysis shows that the WTP has a pumping capacity of 334L/s and is able to supply MDDs to approximately 2040 assuming linear growth from 2032 to 2044.

The total pumping requirement to supply MDD and Fire Flow were compared for each pressure zone.

The pumping requirements in Zone 1 include the Town's total MDD plus a fire event in Zone 1. The available capacity in Zone 1 was found to be adequate until the Built Boundary scenario beyond 2044.

Pumping requirements in Zone 2 includes MDD for pressure Zones 2 and 3 plus Zone 2 fire flows. The total MDD of Zone 2 and 3 plus fire pumping requirement is 302L/s by the Planned (2032) scenario. The available pumping capacity is exceeded by approximately 2038. This demonstrates that the combination of the existing Davey BPS and the planned Stewart Road BPS are not adequate to supply future demands.

Pumping requirements in Zone 3 of 221L/s are expected to be triggered beyond 2044 or whenever the area identified as Zone 3 develops.

Table 7.9 Difference in Pumping Capacity (Required – Available)

Zone	Existing	Planned (2032)	Potential (2044)	Built Boundary
WTP Pumping Supply for MDD	159	55	-26	-202
MDD Plus Fire Pumping Comparison				
Zone 1	270	166	85	-91
Zone 2	58	27	-27	-174
Zone 3	0	0	0	-221

Note that the above analysis is based on desktop calculations. Pumping capacities and watermain capacity were also verified in the model.

7.1.4 Watermain Capacity

A map of Collingwood's drinking water distribution network with proposed watermains for planned and proposed developments is shown in **Figure 7-11** highlighting watermain diameters. There are two proposed watermain extensions that will connect portions of Zone 2 that are currently isolated. The first connection is the 400mm watermain that will extend from Stewart Road BPS, past Georgian Meadows subdivision, up the Tenth Line, and connect to the watermain on Thomas Drive. This will create a connection between the 400mm watermains on Sixth Street and Mountain Road, joining the portions of Zone 2 currently serviced by Osler Bluff Rd BPS and Georgian Meadows BPS so that both can be connected to the future Stewart Road BPS.

Another connection will be made to join the Davey BPS service area of Zone 2 to the future Stewart Road BPS Service area. As part a separate hydraulic assessment for the development at 580 & 590 Sixth St., it was recommended that 200mm watermains be installed to connect to Sixth Street just west of Stewart Road, and to the stub on Holden Street. A 300mm watermain on High Street from Campbell to Findlay is also anticipated. These projects would form a connection from Stewart Road BPS, through the Creekside subdivision and along High Street to Findlay Drive. Once both of these connections are made and the valves on High Street are set, the zone boundary will be altered to connect all portions of Zone 2 including the areas currently serviced by Osler Bluff BPS, the future Stewart Road BPS, and Davey BPS.

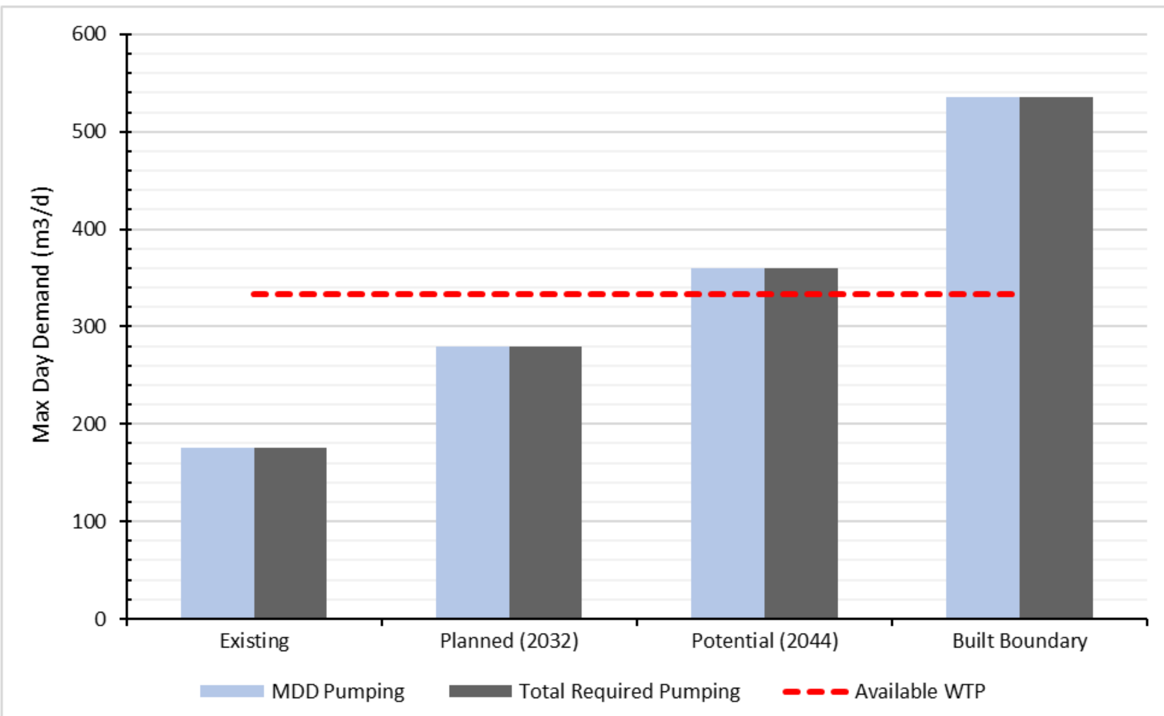


Figure 7-7 Required Pumping Comparison (WTP Supply for Total MDD)

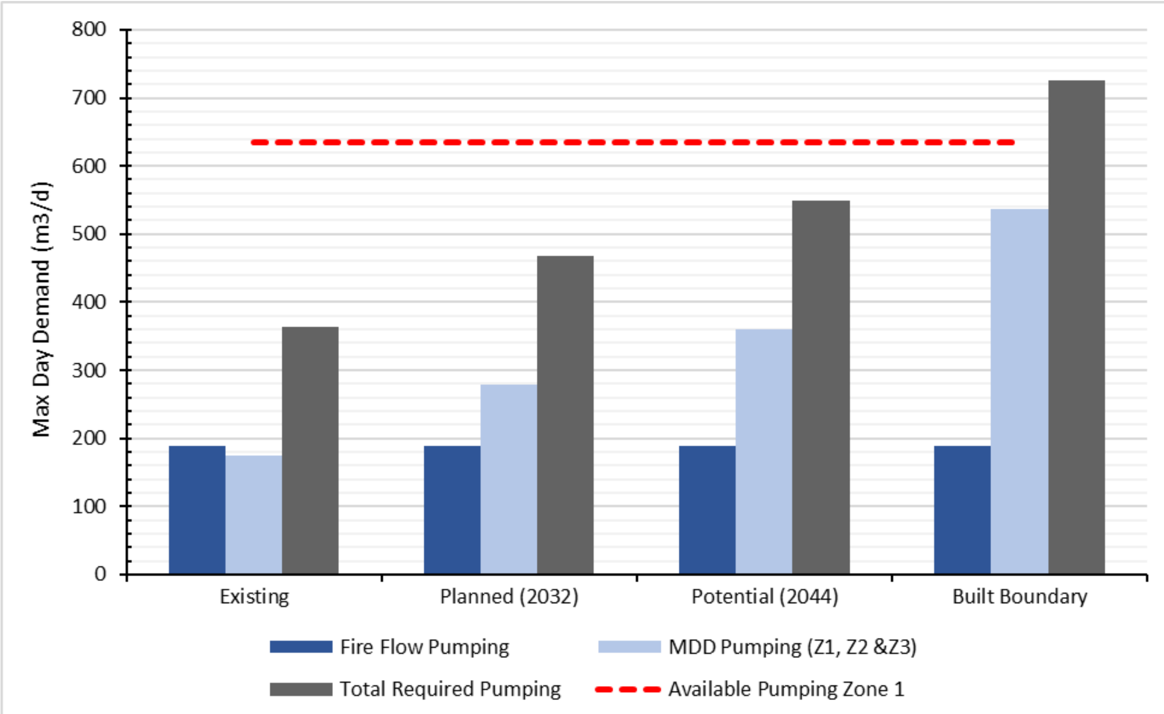


Figure 7-8 Required Pumping Comparison (Zone 1)

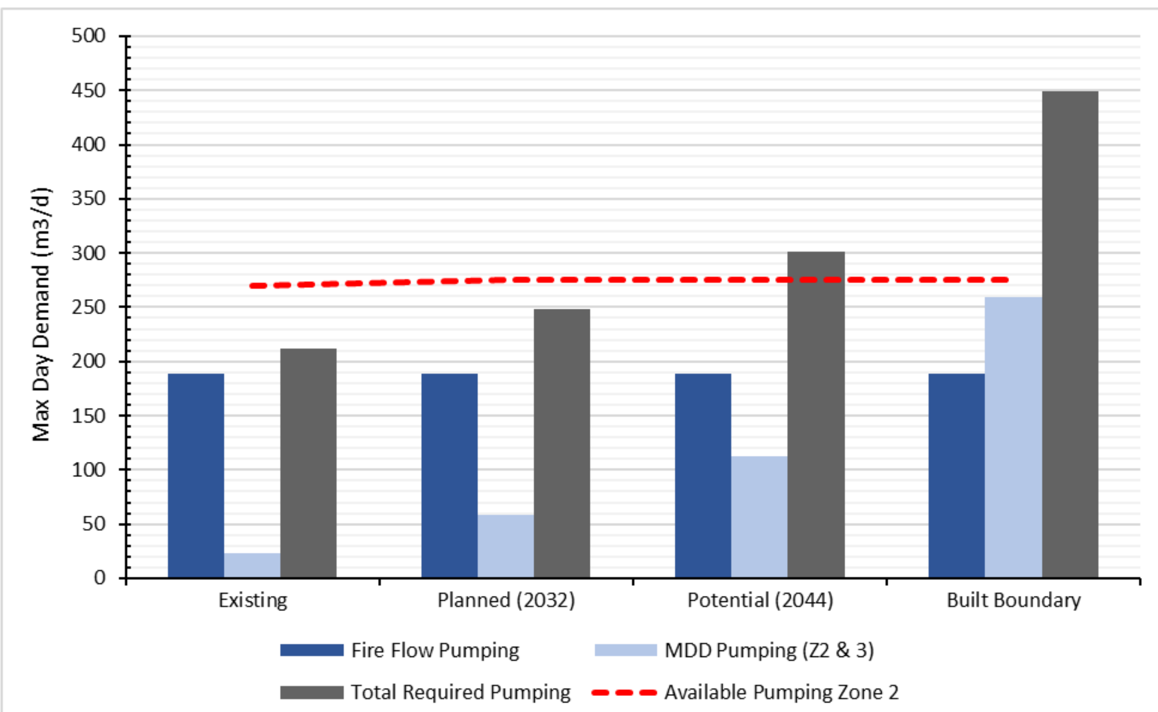


Figure 7-9 Required Pumping Comparison (Zone 2)

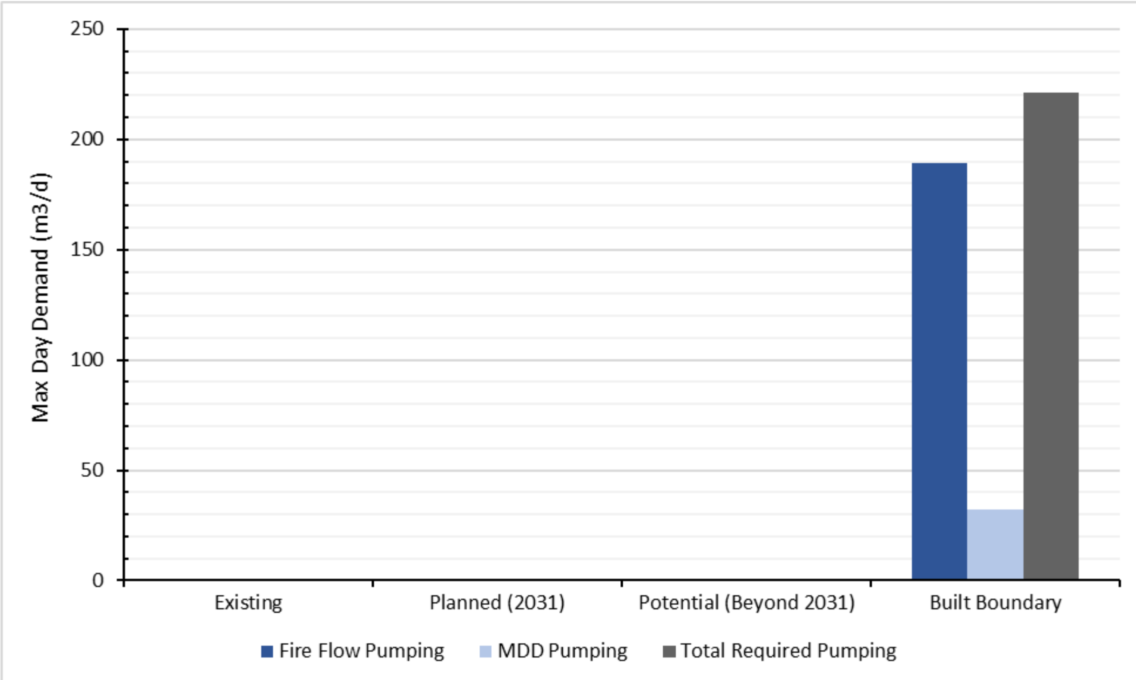


Figure 7-10 Required Pumping Comparison (Zone 3)

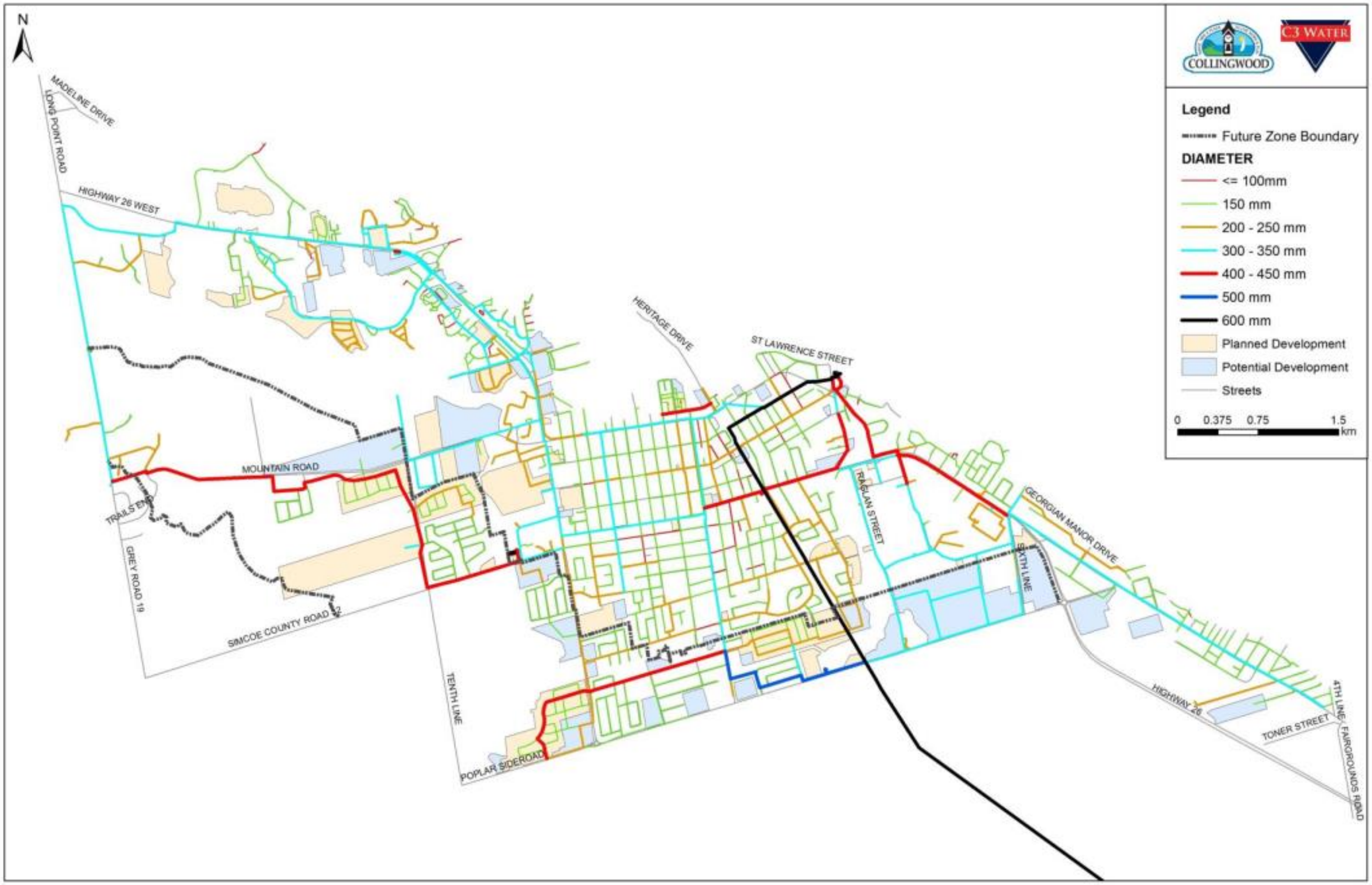


Figure 7-11 Town of Collingwood Water Distribution System, including Planned and Proposed Watermains

Prior to modeling the water system, it was a useful exercise to determine the required watermain sizing through a desktop analysis. Typical watermain capacity can be calculated for different pipe sizes based on head loss criteria, C-factors and other hydraulic parameters. At a head loss gradient of 2.0m/km and C-factor of 130, the approximate capacity of watermains that exist in the Town are shown in **Table 7.10**. Although a higher head loss gradient can convey water adequately, typically the energy consumed to overcome the head loss becomes a concern and pressure issues are noted because of the loss of pressure.

Table 7.10 Watermain Capacity by Diameter

Diameter	Capacity (L/s)
150	18
200	33
300	53
400	113
450	155
500	204
600	329

The required watermain capacity was estimated based on the amount of flow required to supply MDD and fire flows for each zone. **Table 7.11** shows the estimated pumping rates that the watermain would be expected to support.

In the final Built Boundary scenario, the WTP and Carmichael BPS would be expected to transmit approximately 510L/s to supply Zone 1 MDD and fire flows. This capacity is expected to be supplied from both the WTP and the Carmichael BPS. The existing WTP has a pumping firm capacity of 334L/s, but this may be altered depending on the proposed WTP upgrades. The existing WTP discharge connects to a tee 450mm watermain that supplies the system, providing a watermain capacity of approximately 310L/s based on values shown in **Table 7.3**.

The pumping firm capacity at Carmichael BPS is currently 300L/s, with a tee 300mm discharge pipe. This configuration provides approximately 106L/s in watermain capacity at a head loss of 2.0 m/km. In order to transmit the total pumping capacity from both stations, and the total 510L/s required for fire flow and MDD in Zone 1, watermain upgrades at the WTP and Carmichael BPS are recommended.

The addition of the Stewart Road BPS and Reservoir creates a greater requirement on Zone 1 to convey water. As can be seen in **Figure 7-11**, there is a gap in large watermain in Zone 1 between Hurontario Street and the proposed location of the Stewart Road Reservoir. As noted in the Collingwood Master Servicing Plan for Water and Sanitary Sewer System Technical Memorandum# 4 – Existing Conditions (TM4), Zone 1 has existing pressure concerns which will be worsened by the flow required to fill the Stewart Road Reservoir. Watermain upgrades are required to fill the Stewart Road Reservoir while maintaining adequate Zone 1 pressures.

Zone 2 is expected to require a total watermain capacity of 417L/s. There is currently a 500mm discharge watermain at the Davey BPS and a 400mm at the proposed Stewart Road PS, giving a combined capacity of 317L/s based on **Table 7.10**, above. This would support the Planned and Potential developments, but additional capacity would be needed to supply up to the Built Boundary.

Additional linkage between the Stewart Road BPS and the Davey BPS is required in the future to allow for transfer of water between these areas and to service new developments on the west side of Town.

Watermain capacity out of proposed pumping station in Zone 3 would be expected to convey over 220L/s.

Table 7.11 Required Pumping (L/s)

Location	Existing	Planned (2032)	Potential (2044)	Built Boundary
Zone 1	347	430	484	510
Zone 2	212	248	302	417
Zone 3	-	-	-	221

7.2 Water System Hydraulic Performance Under Future Demand Scenarios

The water system's hydraulic performance was modelled with existing infrastructure and planned 2032 MDD demands to analyze the system's response to the increased development. Preliminary results showed that the system experienced performance issues under 2032 conditioned, which were worsened with 2044 demands. The 2032 scenario therefore formed the baseline for addressing future deficiencies, and then alternatives were tested with both Planned and Potential demands.

Watermains that are currently planned to service the 2032 and 2044 developments were included in the model as listed below and shown in **Figure 7-11**. This scenario included the anticipated pressure zone boundary change and watermains connect the portions of Zone 2 serviced by Stewart Road BPS and Davey BPS. The following changes were made to the existing system in the 2032 planned scenario:

- 400mm feedermain on Tenth Line, through Mair Mill Villages development to Mountain Road
- Decommissioning of Georgian Meadows BPS
- Decommissioning of Olser BPS (acts as pressure sustaining boundary valve)
- Phase 1 of Stewart Road BPS
- 200m watermain on High Street between Findlay Drive and Campbell Street
- Various watermains to service the following developments:
  - Red Maple
  - Mair Mill Villages
  - 580 & 590 Sixth Street
  - Fumo Property
  - Phase 1, 2, 3 of Summit View
  - Eden Oak McNabb
  - Pretty River Village (King)
  - Eden Oaks Industrial developments

7.2.1 System Pressure and Zone Boundary Analysis

The minimum pressure was modelled for the 2032 future scenario under MDD conditions with the addition of planned development demands and anticipated zone boundary changes. Minimum pressures were recorded for the 24-hour simulation, but typically occur at the peak hourly demand. The results are shown in **Figure 7-12** and are colour coded based on the Town's pressure criteria. Areas of concern are indicated in red where minimum pressure are less than 40psi**.**

The results indicate that a large area of low pressure develops in the west side of Zone 1 under the 2032 MDD conditions. With the addition of the Stewart Rd PS and Reservoir and increased demands in Zone 2, as expected there is limited capacity within the existing system to supply the reservoir at Stewart Road. This increase in flow increases the head loss in Zone 1, reducing system pressures.

The normal operating pressures were also evaluated during ADD and should be in the range of 50psi - 80psi. The results of this analysis presented in **Figure 7-13**. This figure show acceptable ranges under future average demand conditions. The maximum pressure under average future conditions was modelled and confirmed that pressures did not exceed 100 psi.

7.2.2 Watermain Capacity

The watermain capacity was modelled for the 2032 MDD scenario. Typically, a large watermain should have head losses of less than 2m/km. The maximum head loss gradients are shown in **Section 7.14.** Head losses through the system typically occur during the Peak Hour of MDD, which is shown to be 9:00am in this scenario. It should be noted that the Carmichael Reservoir is also filling at this time. The desktop analysis completed in the previous section can also be observed in the modeled watermain capacity results. There are many portions of watermain with high head loss (>2.0m/km). Particularly, head losses are high throughout the core of the system as water is directed from the WTP across the downtown area to the Carmichael BPS, Water Tower, and Stewart Road Pumping Station. High head losses contribute to the low pressures.

The requirements in the Town are associated with land use type; where residential zones should have a minimum fire flow of 57L/s and ICI zone should have a fire flow of 136L/s. The recommended fire flow for ICI is 189L/s according to the Town Development Standards. Modelling was conducted using a steady-state analysis of available fire flows at a residual pressure of 20psi for a 2-hour fire flow scenario at 12:00pm under Maximum Day Demand (MDD) conditions.

The operation of the pumping stations that were modelled during MDD and fire flow conditions are provided in **Table 7.12**. The results for each node are shown in **Figure 7-15.** The available fire flows are colour coded according to the Town's criteria and can be compared for each land use type. Dead-end nodes without hydrants were excluded from the analysis. Some locations along the zone boundary, near old small diameter watermains, and dead-ends with fire hydrants were found to experience fire flows below 57L/s. These areas will be investigated and addressed through detailed linear infrastructure upgrade projects outlined in the implementation planning phase. The majority of the concerns will be addressed through the addition of large watermains. The remaining concerns will be addressed through local watermain improvements.

Table 7.12 Pump Operation in Model, 2032

Location	Number of Pumps Operated at MDD	Pump Rated Capacity (L/s)	MDD Flow (Planned 2032)	Additional Fire Flow Pumps at 12:00-14:00
Raymond A. Baker WTP	3 Pumps	138.6L/s each	150 - 300L/s	-
Carmichael BPS	1 Pump	100L/s	30 - 80L/s	200L/s Pumps
Davey BPS	1 Pump 1 Pump	25L/s 55L/s	15 - 30L/s	92L/s Pump
Stewart Road BPS	1 Pump 2 Pumps	15L/s 35L/s	55 - 60L/s	35L/s Pump
Osler Bluff BPS	-	-	-	-

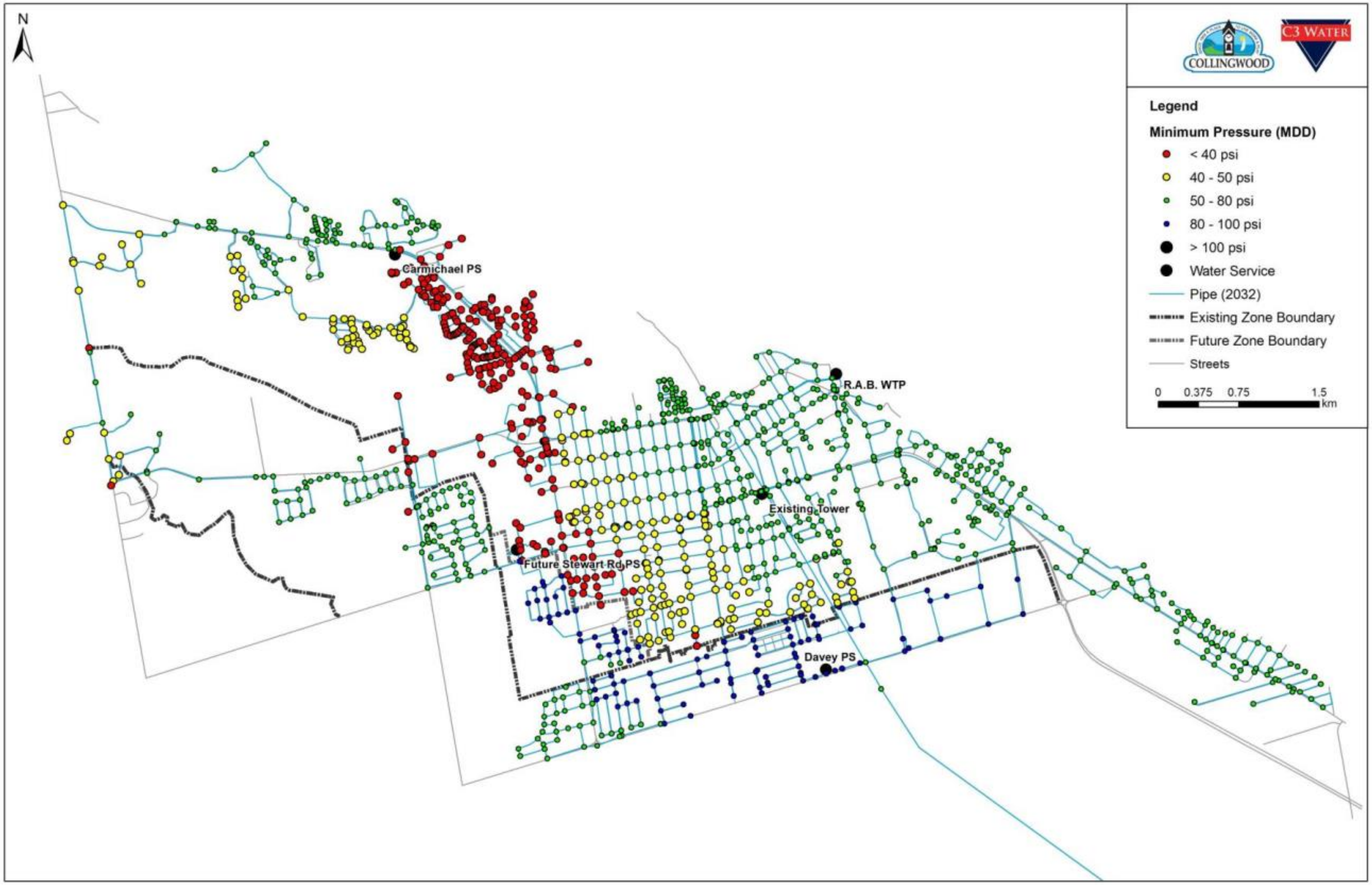


Figure 7-12 Minimum Pressures MDD, 2032

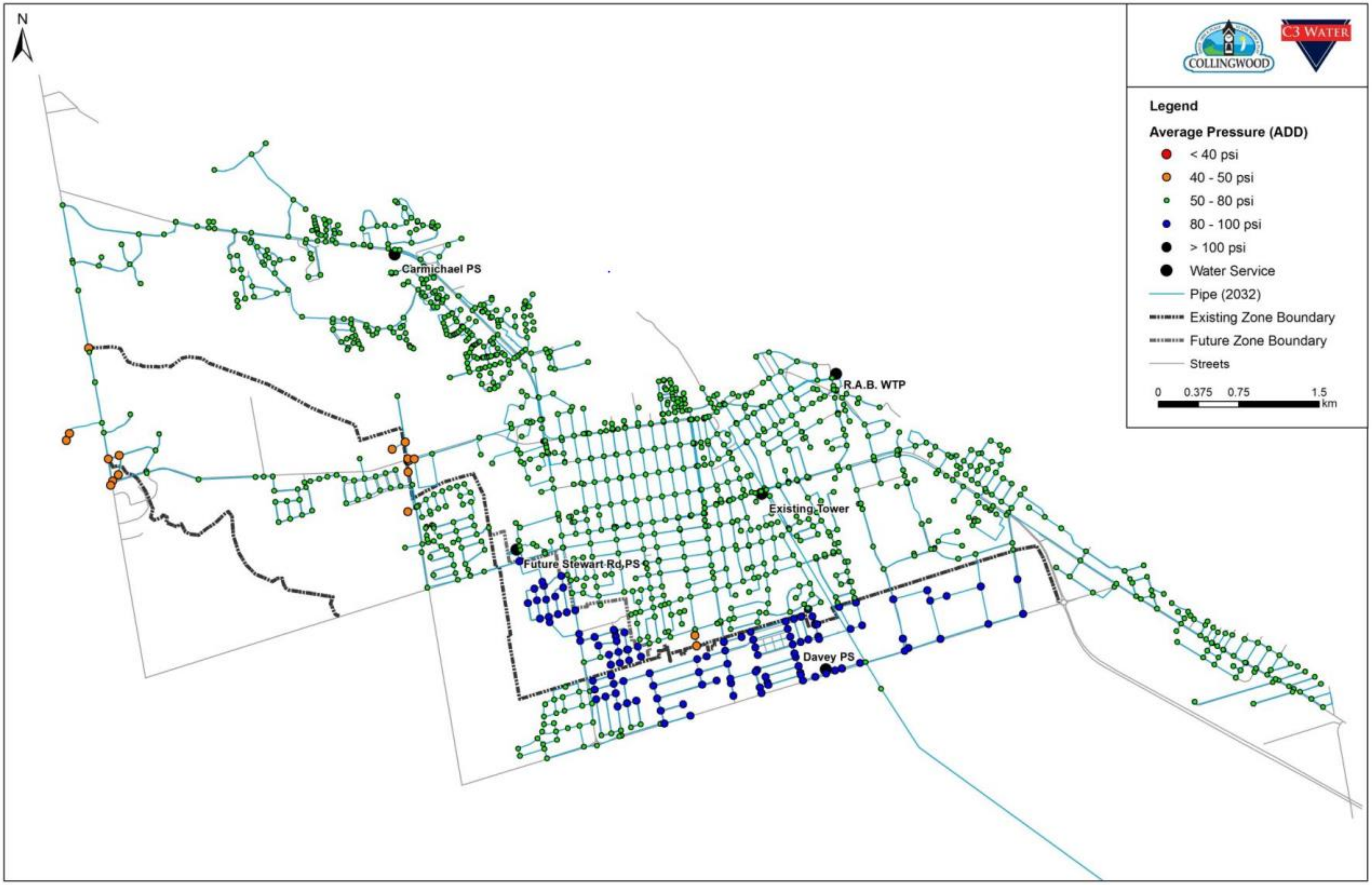


Figure 7-13 Average Pressures ADD, 2032

Figure 7-14 Maximum Head Loss MDD, 2032

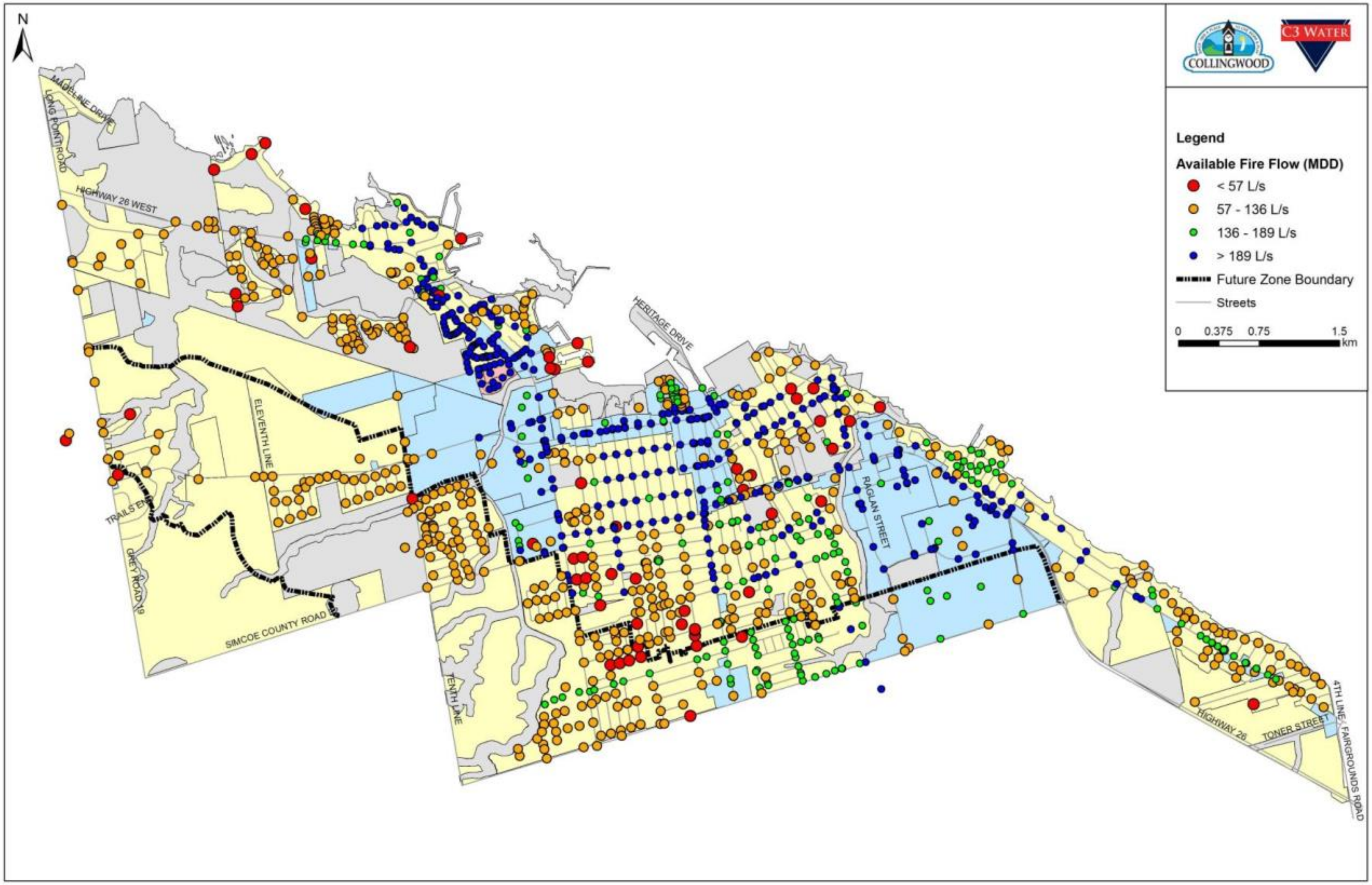


Figure 7-15 Available Fire Flow MDD, 2032

7.3 Needs Assessment - Sanitary Systems

The impact of planned growth on the existing sanitary sewer system and the Collingwood WWTP was assessed. The following sections present the results of the analysis.

7.3.1 Collingwood WWTP

Planned growth is anticipated to increase the residential population by 12,366 persons. Growth is also anticipated in non-residential lands, where a total of 48ha of lands are anticipated to be developed or re-developed.

To assess future flows for planned development, the flow generation rates listed in **Table 7.1** were used. Residential development is anticipated to contribute an average sanitary flow of 260Lpcd plus an average infiltration of 90Lpcd, while non-residential development is anticipated to contribute an average flow of 21.6m^3/ha/d plus average infiltration of 6.4m^3/ha/d. **Table 7.13** presents the projected flows at the Collingwood WWTP for planned development.

Table 7.13 Projected Flow at the Collingwood WWTP for Planned Development

Description	Anticipated Residential Population Growth	Anticipated ICI Area Growth (ha)	Recommended Per Capita or Area Flow Generation, including I/I	Projected Flow (m^3/d)	Current Rated Capacity (m^3/d)
Existing Flow	-	-		16,300	24,548
Planned Development (2032)	12,366	48.0	350Lpcd (residential) and 28m^3/ha/d (non-residential)	21,973	24,548

Based on the results shown in **Table 7.13**, planned growth will increase average day flows at the Collingwood WWTP by 5,673m^3/d and result in a projected flow at the facility of 21,973m^3/d. This planned growth flow represents 89% of the rated capacity of the existing facility. An expansion project to the Collingwood WWTP would be triggered when the flow reaches 80% of the rated capacity or 19,638 m^3/d. The average flow at the Collingwood WWTP will reach 19,638 m^3/d when 64% of all planned growth is serviced. Assuming a constant annual growth rate over this period, this would occur in 2026. A plant expansion project would include an Environmental Assessment, preliminary and detailed design and construction. As the Town has already completed a Schedule C EA for an expansion to the Collingwood WWTP, an EA addendum will be needed to update the 2011 findings based on updated and new information and would include re-evaluation of alternatives for expansion. Following completion of the addendum, the Town could proceed with preliminary and detailed design.

The model was also used to predict peak flow conveyed to the Collingwood WWTP, predicted bypass flows and the peak wet well depth with future development in place. **Table 7.14** presents the predicted peak flow reaching the treatment plant bypass chamber, the peak predicted bypass flow and the peak wet well depth under design flow conditions as well as the 2-year 5-year, 10-year, 25-year and historical June 17, 2017 rainfall events for planned growth.

Table 7.14 Peak Flows to Collingwood WWTP – Planned Growth

Conditions	Peak Flow to the Bypass Chamber (L/s)	Peak Bypassed Flow (L/s)	Peak Wet Well Depth (m)
Design Flow	777	0	2.57
2-Year Storm	1409	84	3.50
5-Year Storm	1,537	95	3.50
10-Year Storm	1,576	165	4.01
25-Year Storm	1,668	309	4.08
June 17, 2017 Event	1,422	105	3.50

Model results presented in **Table 7.14** indicate that treatment bypass will occur as a result of a 2-year storm event for planned growth conditions. As the current peak treatment capacity of the Collingwood WWTP is 705L/s (60,900m^3/d), **Table 7.14** shows that the peak flow conveyed through the sanitary trunk sewer system to the WWTP is predicted to exceed the peak treatment capacity during design flow conditions and for all design storm events. For the June 17, 2017 event, the planned growth peak flow from the sanitary sewer collection system to the treatment plant of 1,422L/s is more than two times the current peak treatment capacity flow of 705L/s. Any expansion of the Collingwood WWTP will need to address the high peak flows reaching the plant.

7.3.2 Sanitary Sewer System and Pumping Stations

To assess the impact of planned growth of the sanitary sewer system and pumping stations, planned development populations and areas were added to the hydraulic model to reflect how these new developments would be serviced. **Appendix B** presents how planned growth developments were assigned to model nodes in the hydraulic model.

The performance of the sanitary sewer system and pumping stations with planned growth was assessed using the June 17, 2017 storm event. This event was selected as it represents a historical event that did result in hydraulic issues in the system. **Table 7.15** presents a comparison of the peak flow and peak wet well depth predicted at each pumping station with pumping firm station capacities and maximum wet well depths.

All of the Town's existing pumping stations have sufficient firm capacity to pump planned growth. Planned growth is anticipated to increase peak flows at the Black Ash SPS, while small increases in peak flow are anticipated at Minnesota SPS, St. Clair SPS, Pretty River SPS and Patterson SPS. No planned growth is anticipated to occur in the Cranberry SPS tributary area. As noted in **Table 7.15**, the Silver Glen Preserve SPS will be replaced by the Developer of the Preserve at Georgian Bay and will receive flows from the planned developments of the Preserve at Georgian Bay (18-Planned), Huntingwood (34-Planned) and Silver Glen (19-Planned). The station will be designed with sufficient firm capacity to accept flows from new development as well as flows which are currently directed to the existing Silver Glen Preserve SPS. A nominal firm capacity of 30L/s has been assumed. This value will be confirmed through detailed design.

Table 7.15 Planned Growth - Pumping Station Performance (June 17, 2017 Event)

Pumping Station	Peak Predicted Flow Entering Station (L/s)	Peak Predicted Wet Well Depth (m)	Station Firm Capacity (L/s)	Maximum Wet Well Depth (m)
Black Ash SPS	143	1.05	212	3.05
Cranberry Trail SPS	10	1.55	32.8	1.75
Minnesota SPS	294	2.31	210	2.69
Patterson SPS	37	1.55	72	2.13
Pretty River Estates SPS	10	1.25	29	2.33
St. Clair SPS	102	1.01	155	4.95
Silver Glen Preserve	29	-	30	

1. As part of the development of the Preserve at Georgian Bay, Huntingwood and Silver Glen Developments, the existing pumping station is planned to be replaced with a new station by the Preserve at Georgian Bay Developer. A nominal firm capacity of 30L/s has been assumed for this station. This value will be confirmed through the detailed design of the station.

The performance of the sanitary sewer system was assessed using the calibrated hydraulic model and the June 17, 2017 storm event. **Figure 7-16** presents the location of sanitary sewers where peak depth exceeded 85% of pipe depth and where surcharge conditions were predicted. In total approximately 5%, or 68 of 1462 sanitary sewers were found to be surcharged. A total of 6%, or 85 of 1462 sanitary sewers, were found to have a peak depth greater than 85% of the sewer depth. The following provides additional details:

- Hydraulic limitations at the Collingwood WWTP resulted in surcharge conditions in sections of sanitary trunk sewer located upstream of the Collingwood WWTP. Affected trunk sewers included the Harbourview Trail Trunk Sewer from Cedar Street to Birch Street, the Harbourview Trail Trunk Sewer from Ste Marie Street to Birch Street, the First Street sanitary sewer from High Street to Beech Street, the Birch Street sanitary sewer from First Street to the WWTP and the Hickory and Walnut Street sanitary sewers from Second Street to First Street. This is consistent with the existing conditions assessment.

- The criteria were not met in selected sanitary sewers including several sections on Minnesota Street (south of Simcoe Street). These results are consistent with the existing conditions assessment.

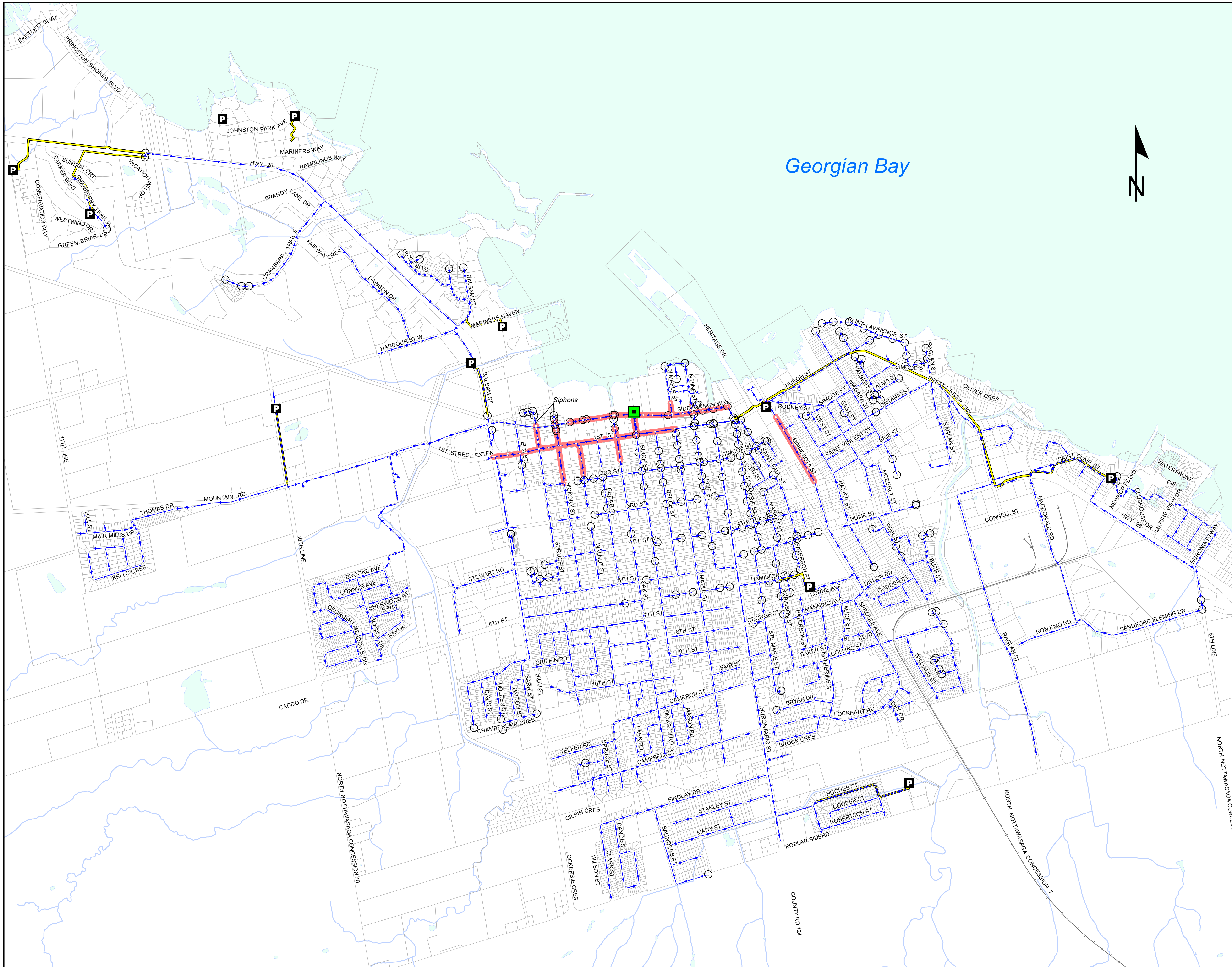


Figure 7.16
Town of Collingwood Sanitary Sewer System Performance Planned Development June 17, 2017 Event

- P Pumping Stations
- Birch Street WWTP
- Sanitary Maintenance Holes
- Shallow Maintenance Holes
- Sanitary Sewers (>=85% Full)
- Sanitary Sewers
- Forcemains

COLE

0 250 500 1,000 Meters

In summary, the Town's pumping stations all have adequate firm capacity to pump planned growth flows. It is noted that the Black Ash SPS forcemain capacity is less than the station capacity. In general, the Town's sanitary sewers have sufficient capacity to convey peak flows resulting from planned growth.

The Collingwood WWTP has adequate rated capacity to treat flows from planned growth. However, an expansion to the Collingwood WWTP will need to be triggered when the average flow to the plant reaches 80% of the rated capacity. This condition is predicted to occur when 64% of planned growth has been completed.

7.4 Assessment of Planned and Potential Development

The impact of planned and potential growth on the existing sanitary sewer system and the Collingwood WWTP was assessed. The following sections present the results of the analysis.

7.4.1 Collingwood WWTP

Planned and potential growth is anticipated to increase the residential population by 21,894 persons by 2044. Growth is also anticipated in non-residential lands, where a total of 130ha of lands are anticipated to be developed or re-developed. There are two major non-residential development properties including the Eden Oak Industrial lands which are 50.7ha in area and the underutilized industrial lands located north of Mountain Road and east of 10th Line. The underutilized industrial lands are 24.2ha in area.

To assess future flows for planned and potential development, the flow generation rates listed in **Table 7.1** were utilized. Residential development is anticipated to contribute an average sanitary flow of 260Lpcd plus average infiltration of 90Lpcd, while non-residential development is anticipated to contribute an average sanitary flow of 21.6m^3/ha/d plus average infiltration of 4.6m^3/ha/d. **Table 7.16** presents the average projected flows at the Collingwood WWTP for planned and potential development.

Section 7.3.1 noted that an expansion of the Collingwood WWTP would need to be triggered when average flow at the Collingwood WWTP reach 80% of the rated capacity. This is projected to occur when 64% of all planned growth is completed. A future expansion would need to be sized to service both planned and potential developments.

Based on the results shown in **Table 7.16**, planned and potential growth will increase average day flow at the Collingwood WWTP by 12,648m^3/d and result in a projected flow at the facility of 28,948m^3/d. This value exceeds the rated capacity of the Collingwood WWTP. To service all planned and potential growth, additional capacity of 4,400m^3/d would be required.

The model was used to predict peak flow conveyed to the Collingwood WWTP, predicted bypass flows and peak wet well depth with planned and potential developments completed. **Table 7.17** presents these values for design flow conditions and the series of wet weather events considered throughout this study. These values were generated with the existing peak flow capacity of the Collingwood WWTP set to 705L/s.

Table 7.16 Projected Flow at the Collingwood WWTP for Planned and Potential Development

Description	Anticipated Residential Population Growth	Anticipated ICI Area Growth (ha)	Recommended Per Capita or Area Flow Generation, including I/I	Projected Flow (m^3/d)	Current Rated Capacity (m^3/d)
Existing Flow	-	-		16,300	24,548
Planned Development (2032)	12,366	48.0	350Lpcd (residential) and 28m^3/ha/d (non-residential)	21,973	24,548
Potential Development (2044)	9,528	130.0	350 Lpcd (residential) and 28m^3/ha/d (non-residential)	28,948	24,548

Table 7.17 Peak Flows to Collingwood WWTP – Planned and Potential Growth

Conditions	Peak Flow to the Bypass Chamber (L/s)	Peak Bypassed Flow (L/s)	Peak Wet Well Depth (m)
Design Flow	1,580	97	3.35
2-Year Storm	1,530	120	3.50
5-Year Storm	1,513	204	4.04
10-Year Storm	1,617	274	4.06
25-Year Storm	1,587	416	4.10
June 17, 2017 Event	1,438	73	3.63

Model results indicate that treatment plant bypass will occur as a result of design flow conditions as well as all of the storm events considered for planned and potential growth conditions. As the current peak treatment capacity of the Collingwood WWTP is 705L/s, the model predicts that peak flows more than two times this capacity will be conveyed by the Collingwood sanitary sewer system. For the level of service selected in **Section 6.1**, the planned and potential growth peak flow to the plant is more than twice the peak treatment capacity. High wet weather flows to the Collingwood WWTP under planned and potential development conditions will need to be addressed either through improvements at the WWTP or in the sanitary sewer system. The existing service contributes excessive wet weather flows to the system.

7.4.2 Sanitary Sewers, Pumping Stations

To assess the impact of planned and potential growth on the sanitary sewer system and pumping stations, planned and potential growth populations were added to the hydraulic model to reflect how these new developments would be serviced. **Appendix B** presents the growth area model node assignments.

The performance of the existing sanitary sewer system and pumping stations with planned and potential growth was assessed for the June 17, 2017 storm event. **Table 7.18** presents a comparison of the peak flow predicted at each pumping station with pumping station capacities.

All of the Town's existing pumping stations have adequate firm capacity to pump peak flows resulting from planned and potential growth. It is noted that the Black Ash SPS forcemain capacity is less than the station capacity.

Table 7.18 Pumping Station Performance – Planned and Potential Growth

Pumping Station	Peak Predicted Flow Entering Station (L/s)	Peak Predicted Wet Well Depth (m)	Station Firm Capacity (L/s)	Maximum Wet Well Depth (m)
Black Ash SPS	157	1.05	212	3.05
Cranberry Trail SPS	10	1.55	32.8	1.75
Minnesota SPS	310	2.31	210	2.69
Patterson SPS	40	1.55	72	2.13
Pretty River Estates SPS	10	1.25	29	2.33
St. Clair SPS	128	1.01	155	4.95
Silver Glen Preserve	29	-	30	-

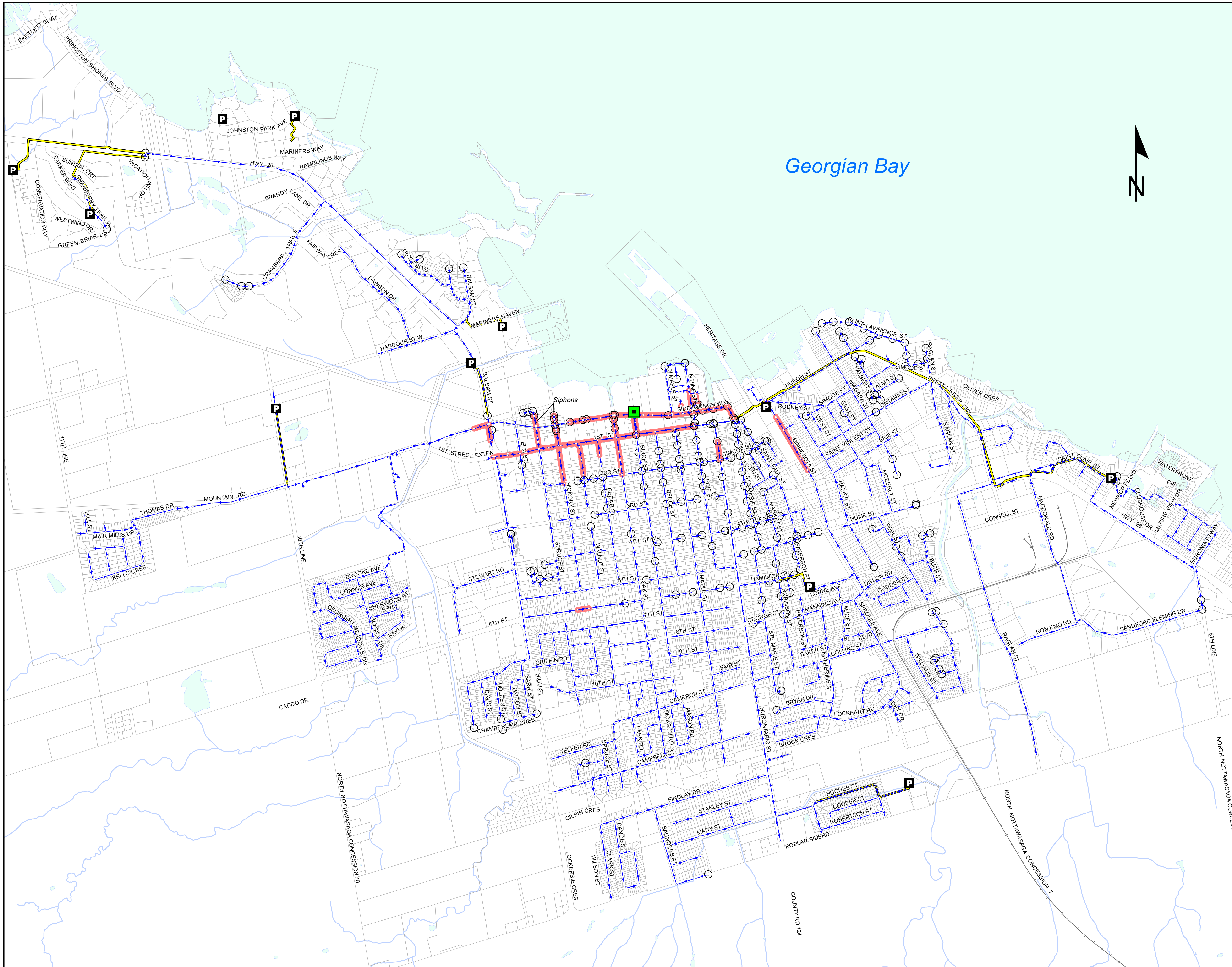
1. As part of the development of the Preserve at Georgian Bay, Huntingwood and Silver Glen Developments, the existing pumping station is planned to be replaced with a new station by the Preserve at Georgian Bay Developer. A nominal firm capacity of 30L/s has been assumed for this station. This value will be confirmed through the detailed design of the station.

The performance of the sanitary sewer system was also assessed using the calibrated hydraulic model. **Figure 7-17** presents the location of sanitary sewers where the peak depth exceeded 85% of the pipe depth. In total approximately 7%, or 110 of 1462 sanitary sewers, were found to have a predicted peak depth which was greater than 85% of the pipe depth. A total of 91 sanitary sewers were found to be full or surcharged. The following provides additional details:

- Hydraulic limitations at the Collingwood WWTP resulted in surcharge conditions in sections of sanitary trunk sewer located upstream of the plant. Affected trunk sewers included the Harbourview Trail Trunk Sewer from Hickory to Birch, the Harbourview Trail Trunk Sewer from Ste. Marie to Birch, the First Street Sanitary Sewer from High Street to Maple, two sections of sanitary sewer on High Street north of First Street, the Spruce Street Sanitary Sewer north of First Street, the Hickory Street sanitary sewer from north of First Street to Second Street, the Walnut Street sanitary sewer from Second to First, the Cedar Street sanitary sewer from Second to First, the Oak Street sanitary sewer from north of First Street to Second Street, and surcharge conditions extended into local sewers on North Pine Street and North Maple Street.

- Two sections of the 500mm diameter sanitary sewer on Mountain Road, located from 282m west of High Street to High Street were identified as operating at full flow conditions. Five sections of sanitary sewer on Minnesota Street north of Simcoe also did not meet the performance criteria.

Figure 7.17
Town of Collingwood
Sanitary Sewer System Performance
Planned and Potential Development
June 17, 2017 Event



Pumping Stations
Birch Street WWTP
Sanitary Maintenance Holes
Shallow Maintenance Holes
Sanitary Sewers (>=85% Full)
Sanitary Sewers
Forcemains

COLE

- Performance criteria were not met in three sections of sanitary sewer on Minnesota Street (south of Simcoe). These conditions are consistent with findings from the existing conditions assessment and the planned growth assessment.

In summary, the Collingwood WWTP does not have adequate rated capacity to treat flows from planned and potential growth and an expansion project would need to be sized to accommodate both planned and potential growth. All of the Town's pumping stations have adequate capacity for planned and potential growth. However, there are capacity limitations associated with peak flows at the Collingwood WWTP and within the sanitary sewer system.

7.5 Built Boundary Growth

The impact of planned, potential and built boundary growth on the existing sanitary sewer system and the Collingwood WWTP was assessed. Within the built boundary, growth is anticipated in several areas. These areas are shown in **Figure 3-2** and anticipated servicing needs are presented below:

- Area A – lands south of Mountain Road, east of Osler Buff Road, and west of 11th Line. The developable area is estimated to be 193ha. Sanitary flows from these areas would be directed to the Mountain Road sanitary sewer by gravity. A new 450mm diameter sanitary sewer with a slope of 0.5% will be adequate for servicing of this area.
- Area B – south of Poplar Road, east of Tenth Line, west of High Street. The developable area is estimated to be 97ha. Servicing of these lands requires a new trunk sewer which would discharge to the Black Ash sewer and ultimately to the Mountain Road sanitary sewer. A new 375mm diameter sanitary sewer with a slope of 0.4% will be adequate for servicing of this area.
- Area F – lands south of Poplar Sideroad, south of Georgian Bay and north of Highway 26. The developable area is estimated to be 51ha. Servicing of these lands would require the construction of new sanitary sewers and one new pumping station to convey flows to the St. Clair SPS.A new 375mm diameter sanitary sewer with a slope of 0.2% will be adequate for servicing of this area. A new pumping stations will also be required.
- Area G1 – lands north of Mountain Road and west of Silver Creek. The developable area is estimated to be 56ha. A Silver Creek Pumping Station would be required to pump flows to the trunk sewer on Highway 26. A new 375mm diameter sanitary sewer with a slope of 0.2% will be adequate for servicing of this area.
- Area G2 – Lands north of Mountain Road, south of Georgian Trail and east of Silver Creek. The developable area is estimated to be 41ha. These lands would be serviced through the Harbour Street sanitary sewer by gravity. A new 375mm diameter sanitary sewer with a slope of 0.2% will be adequate for servicing of this area.
- Area G3 – Lands south of Georgian Trail, north of Mountain Road and east of Silver Creek. The developable area is estimated to be 35ha. Servicing of these lands would require a pumping station which would discharge into the Mountain Road sanitary sewer. A new pumping station with a firm capacity of 45 L/s and a 200mm diameter forcemain will be adequate for servicing of this area.
- Area G4 – Lands north of Highway 26 and immediately east of Osler Bluff Road. The developable area is estimated to be 11ha. These lands would be serviced by the construction of a new pumping station which would direct the sanitary flow to Highway 26 Trunk Sewer. A new pumping station

with a firm capacity of 15 L/s and a new 100mm diameter forcemain will be adequate for servicing of this area.

The following sections present the results of the analysis.

7.5.1 Collingwood WWTP

Growth to the Built Boundary will add a service area of 484ha and an additional residential population of 16,104 and an employment population of 8,052 persons. This would equate to an additional average flow at the Collingwood WWTP of 10,143m^3/d. **Table 7.19** presents the projected flow at the Collingwood WWTP for planned, potential developments and buildout growth within the Built boundary.

Table 7.19 Projected Flow at the Collingwood WWTP for Planned, Potential and Built Boundary Growth

Description	Anticipated Residential Population Growth	Anticipated ICI Area Growth (ha)	Recommended Per Capita or Area Flow Generation, including I/I	Projected Flow (m^3/d)	Current Rated Capacity (m^3/d)
Existing Flow	-	-		16,300	24,548
Planned Development (2032)	12,366	48.0	350Lpcd (residential) and 28m^3/ha/d (non-residential)	21,973	24,548
Potential Development (2044)	9,528	130.0	350Lpcd (residential) and 28m^3/ha/d (non-residential)	28,948	24,548
Built Boundary (2064)	16,104	161	350Lpcd (residential and employment)	39,091	24,548

1. Built boundary growth residential growth population calculated based on assumption that 66% of lands in Built Boundary would be developed as residential lands and 33% of lands in Built Boundary would be developed as non-residential lands.

It is noted that growth to the built boundary will require a significant expansion to the Collingwood WWTP.

7.5.2 Sanitary Sewers, Pumping Stations

The performance of the existing sanitary sewer system, which encompasses all sanitary sewers and pumping stations was assessed for planned, potential and built boundary growth using the calibrated hydraulic model. A built boundary growth model scenario was created by adding new population and serviced areas to the model to represent new growth. Growth populations and areas were added to the model to reflect how these lands would be serviced. Section 7.5 provided information on how Areas A, B, F, G1, G2, G32 and G4 are anticipated to be serviced.

Performance assessments were completed using the June 17, 2017 rainfall as input. **Table 7.20** presents a comparison of the peak flow predicted at each pumping station with pumping station capacities.

Table 7.20 Pumping Station Performance – Planned, Potential and Built Boundary Growth

Pumping Station	Peak Predicted Flow Entering Station (L/s)	Peak Predicted Wet Well Depth (m)	Station Firm Capacity (L/s)	Maximum Wet Well Depth (m)
Black Ash SPS	198	1.05	212	3.05
Cranberry Trail SPS	10	1.56	32.8	1.75
Minnesota SPS	310	2.31	210	2.69
Patterson SPS	40	1.69	72	2.13
Pretty River Estates SPS	10	1.26	29	2.33
St. Clair SPS	147	1.02	155	4.95
Silver Glen Preserve SPS	29	-	30	-

1. As part of the development of the Preserve at Georgian Bay, Huntingwood and Silver Glen Developments, the existing pumping station is planned to be replaced with a new station by the Preserve at Georgian Bay Developer. A nominal firm capacity of 30L/s has been assumed for this station. This value will be confirmed through the detailed design of the station.

All of the Town's pumping stations have sufficient capacity to pump incoming flows from built boundary growth while maintaining the peak predicted wet well depth below maximum wet well depth. The peak flow entering the Minnesota SPS is predicted to be higher than the firm capacity. It is noted that the Black Ash SPS forcemain capacity is less than the station capacity. The performance of the sanitary sewer system was also assessed using the calibrated hydraulic model. **Figure 7-18** presents the location of sanitary sewers where the peak depth exceeded 85% of the pipe depth. In total approximately 13%, or 183 of 1462 sanitary sewers, were found to have a predicted peak depth which was greater than 85% of the pipe depth. A total of 168, or 12% of sanitary sewers, were found to be surcharged. The following provides additional details:

- Similar to the planned and potential growth assessment, limitations at the Collingwood WWTP result in predicted surcharge conditions in the Town's trunk sewer system. The extent of surcharge conditions is predicted to increase with built boundary development.
- The Mountain Road sanitary sewer is predicted to be surcharged from immediately west of Tenth Line to High Street. Surcharge conditions in the Mountain Road sanitary sewer could restrict the capacity of upstream sewers including the Black Ash Trunk Sewer and the Tenth Road sanitary sewer.

In summary, additional capacity would be required at the Collingwood WWTP and in the Mountain Road sanitary sewer to service planned, potential and built boundary growth.

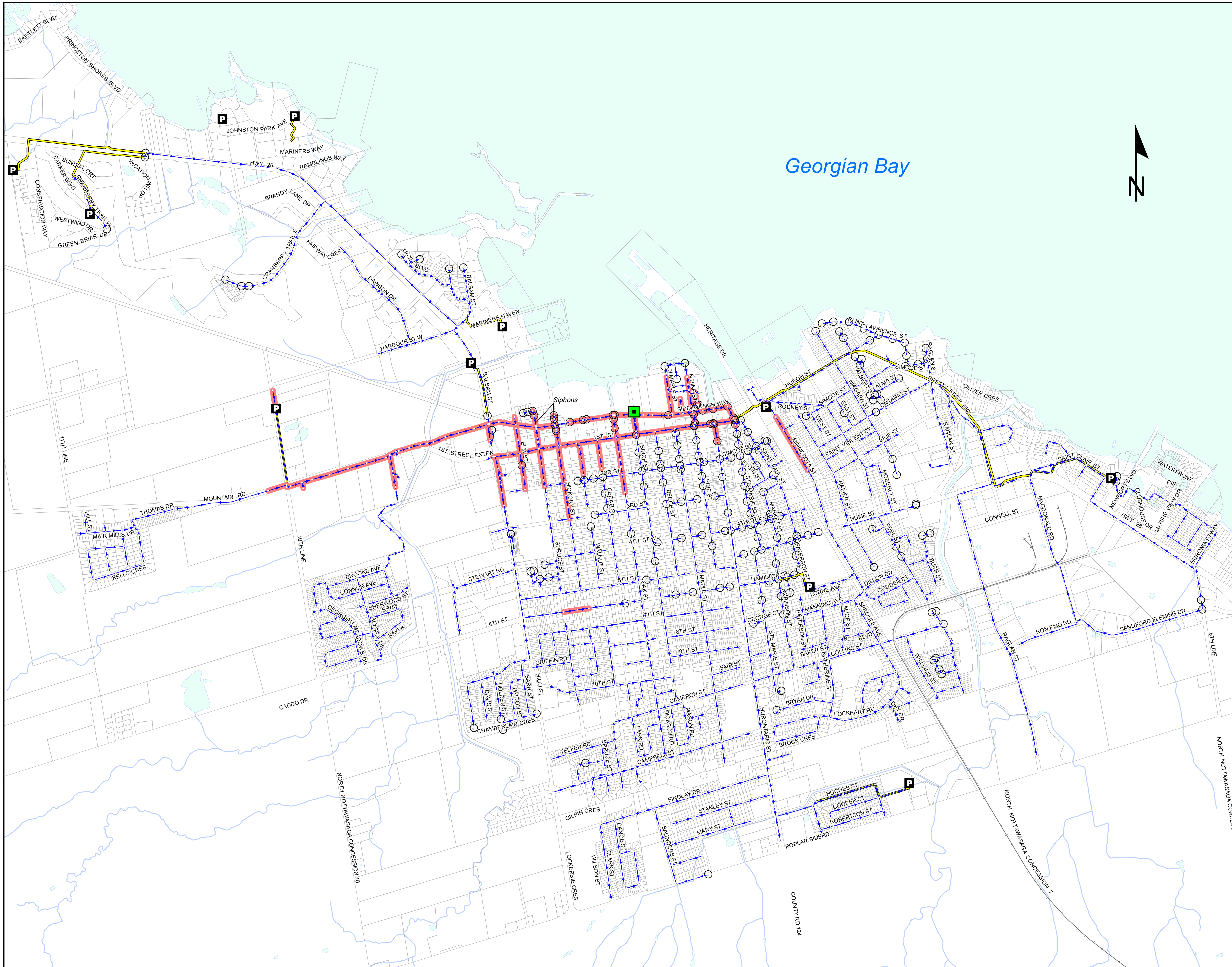


Figure 7.18
Town of Collingwood
Sanitary Sewer System
Performance Planned,
Potential and Built
Boundary Development
June 17, 2017 Event

- P Pumping Stations
- Birch Street WWTP
- Sanitary Maintenance Holes
- Shallow Maintenance Holes
- Sanitary Sewers (>=85% Full)
- Sanitary Sewers
- Forcemains

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0 250 500 1,000 Meters

8 Alternatives Evaluation

Alternatives were developed and evaluated to provide water and sanitary servicing for future development. The following sections discuss the evaluation criteria, the development and evaluation of water system and sanitary system alternatives and the selection of preferred alternatives.

8.1 Evaluation Criteria

The evaluation process utilized a two-step process where an initial evaluation was conducted to assess feasibility and ability to meet future requirements. Alternatives identified as not feasible were eliminated from further consideration. Feasible alternatives were carried forward into evaluation based on the criteria and numeric scoring method presented in **Table 8.1**. An even weighting system was also applied to each category to develop an overall score for each alternative and recommend a preferred alternative.

Table 8.1 Evaluation Criteria and Weighting

Category	Criteria Description	Weight	Scoring
Natural Environment	Impacts on fish habitat, terrestrial habitat, species of concern and groundwater	1	No impact (score=1) Moderate impact (score=3) Major impact (score=5)
Social/ Cultural Environment	Construction related impacts on communities, disruption to existing community and land uses, need for property acquisition, impacts on parks, cultural landscape, heritage resources and aesthetics	1	No impact (score=1) Moderate impact (score=3) Major impact (score=5)
Technical Environment	Constructability, Integration with existing systems, utility conflicts, ability to maintain current operation during construction, infrastructure security, flexibility for future expansion, construction risks, timelines and approvals	1	No impact (score=1) Moderate impact (score=3) Major impact (score=5)
Financial Environment	Capital and lifecycle costs and cost sharing feasibility	1	No impact (score=1) Moderate impact (score=3) Major impact (score=5)

8.2 Water System Alternatives

Alternatives were developed to address the existing and future deficiencies in the water system. The alternatives were divided into sections according to problem area, including total system supply, Zone 1 storage, Zone 2 & 3 storage, and Zone 2 &3 pumping. Watermain capacity was addressed through the identification of projects following the selection of alternatives. Options for addressing water supply deficiencies included do nothing, limit future growth, implement water efficiency measures, and upgrade the WTP. The storage and pumping alternatives were developed based on opportunities for improvement in existing infrastructure, or new infrastructure requirements. Information was collected for each alternative, including infrastructure locations and costs, and existing information was used to assess potential natural environment, social and cultural environment, technical environment and financial environment impacts.

8.2.1 Supply Alternatives

As previously identified, if all Planned and Potential developments are built by 2044, the system will have an MDD of 41,866m^3/day. Comparatively, when the system develops to the Built boundary it will have an MDD of 57,065m^3/day. The current WTP has a rated capacity of 31,140m^3/day and system's MDD supply commitments has reached 80% of the WTP's capacity. The following alternatives were evaluated to supply the future system MDDs.

8.2.1.1 W-R-1: Do Nothing

The Do Nothing alternative is that in which no changes would be made to address the existing and future water supply deficiencies. This alternative represents what would occur if none of the alternative solutions were implemented. This is not recommended as a viable solution as it would have a significant impact on the growth of the distribution system. This is not a feasible alternative.

8.2.1.2 W-R-2: Limit Growth

This alternative comprises of reducing the future water supply requirements by limiting distribution system demands. This would involve limiting future residential, industrial, commercial and institutional growth and does not conform with the Town's strategic growth plan. This is not a feasible alternative.

8.2.1.3 W-R-3: Water Conservation

The Water Conservation alternative involves reducing water usage to decrease the system demand. Typically, water conservation is an economical method of delaying infrastructure costs. Examples of measures that can be taken include public education programs, irrigation reduction incentives, switching to water efficient water softeners and increasing water efficiency in gardens and pools. Improving water efficiency would help to reduce peak demands and overall water usage in the system. Additionally, water conservation would decrease the volume of sanitary produced. Encouragement to conserve water can be achieved by increasing water costs and providing infrastructure improvement incentives such as toilet rebate programs.

While water conservation could partially address the future supply deficiency, this alternative would be implemented in conjunction with other system improvements to meet demands. Additional supply would still be required but timelines would be adjusted.

8.2.1.4 W-R-4: WTP Upgrade

To provide the total supply of water required by the Town of Collingwood and water taking allowances of neighbouring municipalities, the WTP is expected to expand. The current capacity of the WTP is rated for 31,140m^3/day; however, the intake pipe is designed for up to 90,000m^3/day. The existing PTTW allows up to 68,200m^3/day.

To meet Planned and Proposed development requirements, an additional capacity of 10,726m^3/day would be required. To meet Built Boundary conditions, the capacity would need to be increased by 25,860m^3/day. This can be achieved by installing additional membrane filter modules, high lift pumps and potentially clearwell expansion. Expanding the clearwell may be a concern due to the limited space on the existing WTP site and its location on the shore in the central area of Town.

In addition to improving infrastructure at the WTP, increased water supply would require an upgrade of discharge watermains. This alternative is expected to address the MDD supply pumping deficiency of 26L/s in Potential (2044) scenario and 202L/s in the Built Boundary scenario.

8.2.2 Zone 1 Storage Alternatives

Storage alternatives were evaluated on a zone basis. Approximately 2,317m^3 of storage is required in Zone 1 by the Built Boundary scenario. Four storage alternatives were considered for Zone 1. **Table 8.2** presents the difference in available and required storage in Zone 1. **Figure 8-1** shows the location of storage alternatives.

Table 8.2 Difference in Zone 1 Storage (m^3) (Available – Required)

Location	Existing	Planned (2032)	Potential (2044)	Built Boundary
Zone 1	2,098	-156	-1,607	-2,317

8.2.2.1 W-S1-1: New Zone 1 Elevated Tank and Feedermains

The first alternative considered for addressing storage requirements is to build a new elevated tank in Zone 1. Elevated tanks provide the benefit of floating storage, which maintain system pressure during power failure events and emergencies. Elevated tanks also provide an additional source of water during peak demand events such as fire events and other emergencies. Furthermore, elevated tanks act as a buffer to absorb pressure surges and transients which help maintain the integrity of the water distribution system.

The existing elevated tank has a capacity of approximately 1,685m^3 and is located near the centre of Town. The tank was purchased by the Town in the 1950's in a used condition, so it is nearing the end of its life cycle. The ideal location for an elevated tank is near the edge of a pressure zone a location far away from the main water source to help maintain system pressures across the water system. Due to the age, location and size of the existing elevated tank the eventual replacement should be considered. This alternative would include decommissioning the existing tank and building a new elevated tank to provide a total of 4,002m^3 to replace the existing volume and cover the Zone 1 storage deficiency under Built Boundary conditions.

Two locations have been highlighted based on high points in Zone 1, and proximity to existing areas of low pressure concerns. **Figure 8-1** shows the approximate location of the Elevated Tank Option 1 and Elevated Tank Option 2. Both options would require the construction of a new elevated tank as well as upgrades to watermains from the WTP to the tank location. Finding a site for a new elevated tank can be challenging due to public acceptance. It is most successful to add to a new development area, prior to development or an ICI area.

8.2.2.2 W-S1-2: Carmichael Reservoir Expansion

The second storage alternative for Zone 1 considers the addition of in-ground storage at the Carmichael Reservoir. This alternative would require construction at the existing reservoir site to increase the volume. This option would not require the Town to acquire new land, and the existing pumps would likely provide suitable pumping capacity. Challenges with this alternative include maintaining operations during construction and increased lifecycle costs due to energy usage. The location and accessibility of the storage is also a concern due to Carmichael Reservoir's position in a western part of Zone 1, away from the majority of central residential and industrial demands. Access to this reservoir from the main portion of Zone 1 is also limited to two watermains under 400mm along Hwy 26.

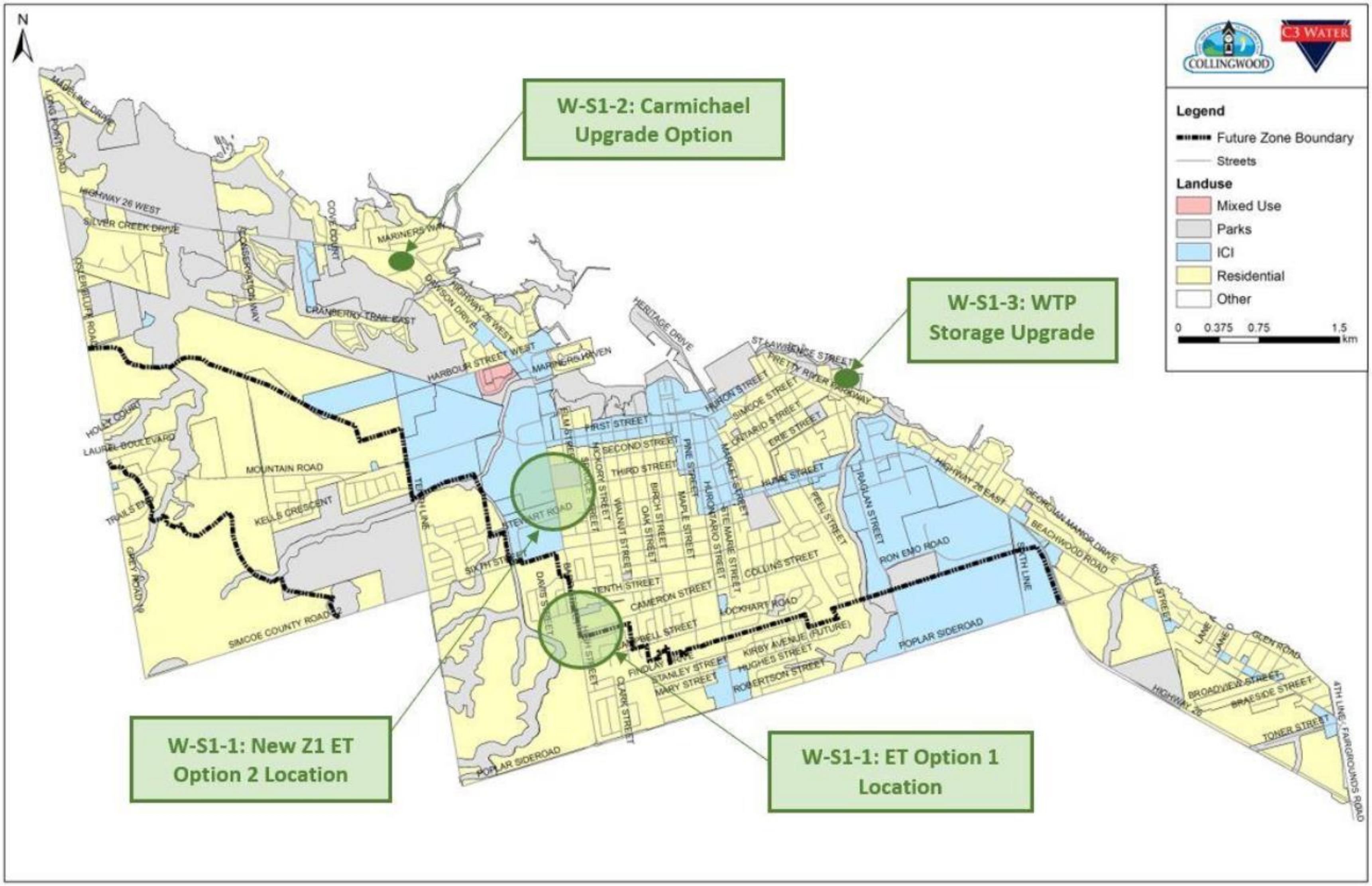


Figure 8-1 Location of Storage Alternatives

8.2.2.3 W-S1-3 WTP Storage and Pumping Station

The third alternative for increasing storage in Zone 1 is to increase the storage at the WTP. This would likely also require an expansion of the WTP PS. This option would involve construction of a new reservoir cell at the existing WTP site to increase the volume of the existing in-ground reservoir. Additionally, the pumping capacity may need to be increased to be able to achieve required fire flows. Construction would be required at the existing site. Challenges with this alternative would include finding the available site capacity to expand the existing reservoir.

8.2.3 Zone 2 and 3 Storage Alternatives

Zone 2 has a small existing deficiency in storage that will be addressed through the addition of the Stewart Road Reservoir with a Phase 1 volume of 1,540m^3. After the development of Phase 1 of the Stewart Road Reservoir, Zone 2 storage should be adequate until approximately 2035, when additional storage will be required.

The future Zone 3 is expected to require approximately 2,992m^3 of storage if all the available lands to the Built Boundary are developed. It is also possible that the ToBM connection would fall into Zone 3, providing another incentive to store additional water. **Table 8.3** presents the difference in available versus required storage in Zones 2 and 3.

Table 8.3 Difference in Storage in Zones 2 and 3 (Available – Required)

Location	Existing	Planned (2032)	Potential (2044)	Built Boundary
Zone 2	-182	391	-1,065	-4,168
Zone 3	0	0	0	-2,992

8.2.3.1 W-S2-1 and W-S2-2: Stewart Road Phase 2 and Phase 3 Reservoir Expansion

The proposed Stewart Road Reservoir has been designed in three phases. Phase 1 has a total volume of 1,540m^3. An additional 1,615m^3, is anticipated for Phase 2 and would cover the 1,065m^3 deficiency in the Potential 2044 scenario. Phase 3 is expected to add another 1,615m^3 or storage resulting in an ultimate available storage of 4,770m^3 at the Stewart Road Pump Station. The Phase 3 expansion would provide storage for a portion of the Built Boundary requirements, but additional storage would be required beyond 2044 and is discussed in the following alternatives.

8.2.3.2 W-S2-3 Stewart Road Additional Reservoir Expansion

Assuming Phase 2 and 3 of the Stewart Road reservoir are built, there would still be a 938m^3 deficiency to meet Built Boundary storage recommendations (Required = 8,273m^3, Available = 2,565m^3 at Davey BPS + 4,770m^3 at Stewart Rd). An additional phase could be added that increases the ultimate storage capacity at Stewart Road Reservoir to 5,708m^3. Additional pumping capacity would also be required at the Stewart Road PS to access the storage. A review of Zone 1 and 2 watermains would also be required to determine if sufficient capacity exists to service the Stewart Road PS and Reservoir.

8.2.3.3 W-S2-4 Davey Reservoir Expansion

Another storage alternative in Zone 2 for the Built Boundary scenario is to expand the existing Davey Reservoir. This alternative assumes that the Phase 2 and Phase 3 reservoir expansions at Stewart Road would be in place, and the additional storage deficiency of 938m^3 from the Built Boundary scenario would be added to the Davey Reservoir. The existing reservoir has a capacity of 2,565m^3 and there is space available to expand based on facility drawings. This alternative would have minimal impact on the community and nearby land uses as the site was designed to accommodate reservoir expansion. Increasing storage at the Davey BPS would require additional water taking from the Regional Pipeline. Capacity and water taking from the Regional Pipeline is discussed in **Section 9.1**.

8.2.3.4 W-S3-1: New Zone 3 Elevated Tank

Since Zone 3 will be located at the end of the water system and at higher elevations, an elevated tank could be built to supply Zone 3. This alternative would involve building on a new site in Zone 3 and would address future built boundary storage restraints. In addition to storage construction, this would require installation of associated watermains.

Challenges associated with implementing new elevated tanks include public acceptance and design approvals. One benefit of building an elevated tank in Zone 3 as opposed to Zone 2 is that the majority of the land is not yet developed, and a site could be easily secured. Depending on the location of the proposed tank, there may be associated impacts on land uses. With the dramatic elevation changes nearby, it may be possible to site in-ground storage at a location to still provide floating storage.

8.2.3.5 W-S3-2: New In-ground Storage Reservoir

An alternative to building a new elevated tank to supply Build Boundary demands in Zone 3 is to build a new in-ground storage facility.

From a social stance, in-ground storage is preferential to floating as it cannot be as easily seen by the public and would face less opposition. However, in-ground storage does not maintain the HGL and would require pumping with increased lifecycle costs.

8.2.3.6 W-S3-3: Combination of System Storage

Realistically the storage requirements for Zones 2 and 3 could be a combination of several of the above alternatives. Staging storage could be coordinated with proposed development needs and locations.

Assuming that the Phase 2 and 3 expansion of the Stewart Road reservoir occur, a new elevated tank or high elevation reservoir could be designed to meet the needs of Zone 2 and Zone 3 with a capacity of approximately 3,930m^3 to meet demands up to the Built Boundary. This covers the Zone 2 deficit of 938m^3 and the total Zone 3 storage requirements of 2,992m^3. This alternative would require a new site and watermains.

A combination of Stewart Road Reservoir expansion and a Zone 2/3 elevated tank would provide the Town with an excellent combination of energy efficient and publicly accepted infrastructure to meet water system requirements. The elevated tank could be sited to provide floating storage to Zone 2 and pumped storage to Zone 3 in the future. The building site should be chosen with sufficient space to place a future Zone 3 PS.

8.2.4 Zone 2 and 3 Pumping Alternatives

The projected Planned and Potential (2044) demands do not require increased pumping capacity in addition to the planned build-out of the Stewart Road pumping station. The Built Boundary demand would require additional pumping of 97L/s in Zone 2 and 221L/S in Zone 3. **Table 8.4** presents the difference in available versus required pumping in Zones 2 and 3.

Table 8.4 Difference in Pumping Capacity in Zones 2 and 3 (Required – Available)

Zone	Existing	Planned (2032)	Potential (2044)	Built Boundary
Zone 2	58	27	-27	-174
Zone 3	0	0	0	-221

8.2.4.1 W-P2-1: Stewart Road Pumping Station Ultimate Pump Upgrades

The Stewart Rd PS project includes two phases of pumping capacity. Phase 1 is designed to have a firm capacity of 105L/s and was included in the initial storage calculations for the entire planning horizon. The Ultimate firm pumping capacity is designed to be 150L/s. Moving forward with this alternative would provide adequate pumping to supply the Zone 2 requirement of the Potential (2044) demands, which has a deficiency of 27L/s. Additional pumping capacity will be required for the Built Boundary Scenario, and alternatives are discussed below.

In the short-term, it is possible that the Stewart Road BPS would supply a portion of Zone 2 before a connection to the Davey BPS service area is in place. In this scenario, the pumping capacity of both stations would not be combined to supply a potential commercial fire in Zone 2 of up to 189L/s. Currently, the developments served by the Stewart Road BPS are not expected to contain many ICI land uses, but the Phase 1 and Phase 2 pumping capacities should be reviewed to supply required fire flows in the area. The construction of the Stewart Road BPS is dependent on development funding timing, so planned capacity should be reviewed as developments are confirmed.

8.2.4.2 W-P2-2: Stewart Road Pumping Station Additional Expansion

The first alternative evaluated to increase pumping capacity in Zone 2 for the Built Boundary was to expand the Stewart Road pumping station beyond the Ultimate phase capacity. If done simultaneously with the planned expansion, this would not require an additional construction project and would have no additional impact on the community. The amount of expansion would be dependent on the selected storage alternative. This alternative would also require review of the pump station’s planned discharge capacity.

8.2.4.3 W-P2-3: Davey Additional Pumping

An alternative to increase pumping capacity in Zone 2 is to increase pumping at the existing Davey pumping station. Davey currently has four operating high lift pumps. The station was built with space for a potential future pump if required. Implementing an additional pump would require construction at the existing site involving demolition and reconstruction of the roof and floor. The new pump would then be tied into the existing water main. This alternative would not require a new site or expansion of the existing pumping station building. The amount of expansion would be dependent on the selected storage alternative. This alternative could be combined with W-P2-2 to optimize pumping capacity in Zone 2.

8.2.4.4 W-P2-4: Retrofit Osler Bluff BPS

Another alternative to increase pumping capacity in Zone 2 is to retrofit the existing Osler Bluff BPS. This would require construction at the existing site and design approvals.

8.2.4.5 W-P3-1: New Zone 3 Booster Station

Zone 3 is a future pressure zone, and currently does not have a booster pumping station. A new station in Zone 3 with 221L/s of capacity would meet the proposed built boundary demands. Similar to the existing Davey Station, the BPS could be initially built with a smaller pumping capacity and space for future pump installation to meet short-term development demands and reduce capital costs until additional pumping is required.

This alternative would require construction of a new site and may have an impact on land uses. As previously mentioned, Zone 3 is currently mostly undeveloped land and therefore site construction and impact on community are not considered major challenges.

8.2.5 Summary of Alternatives Evaluation

Each of the above discussed alternatives were evaluated based on the criteria of natural, social/heritage, technical and financial environments. Alternatives were given a score between one and five for each criterion. The alternative with the lowest overall score is considered the preferred alternative. The evaluation scoring of each alternative is summarized in **Table 8.5**.

Table 8.5 Water System Alternatives Evaluation

	Alternative #	Description	Meets Quantity Requirements?	Natural Environment		Social / Heritage Environment		Technical Environment		Financial Environment		Overall Score
Water Treatment/Supply Capacity				**Weight:**	**1**		**1**		**1**		**1**	
Total System Supply	W-R-1	Do Nothing	no	No impact.	1	No construction impacts. No impact on land uses.	1	No construction required.	1	No capital costs. Expected increased lifecycle costs due to system aging and replacement/emergency needs.	1	**4**
	W-R-2	Limit Future Growth	partial	Limited impact due to reduced development.	2	No construction impacts. Limited impact on land uses. Does not meet Town's Official Plan	5	Limited construction required. Does not meet growth targets.	5	Limited capital costs. Expected increased lifecycle costs due to system aging and replacement/emergency needs. Reduced revenue from DC and taxes.	3	**15**
	W-R-3	Water Efficiency Measures	partial	Lessen the impact on water resources. Decrease in sanitary production. Increase WW Concentrations	2	Delay major construction projects. May have opposition from public to implement efficiency upgrades. Long term effects on the system thru education	3	Delay the need for infrastructure	3	Limited implementation costs, depending on the programs put in place.	3	**11**
	W-R-4	WTP Upgrade	yes	Increase water taking. Construction related impacts. Footprint is limited.	3	Construction at existing site. May impact nearby community. Expect work to be limited to the site.	3	Major construction at existing WTP. Design requiring approval. Potential for supply impacts during construction. Provides security, redundancy and flexibility and meets all demand needs.	3	Major capital costs. Reduce increase in lifecycle costs due to upgraded WTP infrastructure. Cost sharing with other municipalities.	5	**14**
Storage												
Zone 1 Storage	W-S1-1	New Z1 Elevated Tank	yes	Requires new site with potential impact on natural environment. Small footprint.	4	Potential social opposition to new elevated tank. Potential location in industrial area	3	Moderate construction at new site. Design requiring approval. Improved system performance and protection.	1	High capital cost, minimal life cycle cost. Improved system efficiency and flexibility.	3	11
	W-S1-2	Carmichael Reservoir Expansion	yes	Existing site, minimal impact on environment. Small footprint.	2	Construction at existing site. Limited impact on community.	2	Limited space on existing site for expansion. Less redundant as pumping required. Location in distribution system is not ideal.	5	Moderate capital cost, increased life cycle cost.	3	**13**
	W-S1-3	WTP Storage and PS	yes	Existing site, minimal impact on environment. Small footprint. Near lake and lake impacts.	2	Construction at existing site. Limited impact on community.	2	Limited space on existing site for expansion. Less redundant as pumping required.	5	Moderate capital cost, significant life cycle cost.	3	**13**
Zone 2 & 3 Storage	W-S2-1 W-S2-2	Stewart Rd Phase 1 and Phase 2 Reservoir Expansion	yes	Existing site, minimal impact on environment. Small footprint	3	Construction at existing site. Will not impact community or other land uses.	2	Limited space on existing site for expansion. Less redundant as pumping required. Does not protect system from pressure spikes.	5	Moderate capital cost, significant life cycle cost.	3	**13**

	Alternative #	Description	Meets Quantity Requirements?	Natural Environment		Social / Heritage Environment		Technical Environment		Financial Environment		Overall Score
	W-S2-3	Stewart Rd Additional Reservoir Expansion	yes	Existing site, minimal impact on environment. Small footprint	3	Construction at existing site. Will not impact community or other land uses.	2	Limited space on existing site for expansion. Less redundant as pumping required. Does not protect system from pressure spikes.	5	Moderate capital cost, significant life cycle cost.	3	**13**
	W-S2-4	Davey Reservoir Expansion	yes	Existing site, minimal impact on environment. Small footprint	3	Construction at existing site. Will not impact community or other land uses.	2	Limited space on existing site for expansion. Less redundant as pumping required. Does not protect system from pressure spikes.	5	Moderate capital cost, significant life cycle cost.	3	**13**
	W-S3-1	New Zone 3 Elevated Tank	yes	Requires new site with potential impact on natural environment. Small footprint	3	Potential social opposition to new elevated tank. New site required.	3	Moderate construction at new site. Design requiring approval.	3	High capital cost, minimal life cycle cost.	5	**14**
	W-S3-2	New Zone 3 In-ground storage	yes	Requires new site, potential impact on environment. Large footprint	3	Construction at new site. Potential impact on community	3	Flexibility in location and future expansion. Design requiring approval.	5	Moderate capital cost, significant life cycle cost	3	**14**
	W-S3-3	Combination of Zone 2/3 System Storage (Floating Reservoir)	yes	Requires new site, potential impact on environment. Large footprint	3	Construction at new site. Potential impact on community	3	Flexibility in location and future expansion. Design requiring approval. Improved system performance and protection.	4	Moderate capital cost, minimal life cycle cost. Improved system efficiency and flexibility.	2	**12**
Pumping												
Zone 2 & 3 Pumping	W-P2-1	Stewart Road Ultimate Pump Station Capacity	partial	Limited impact.	1	Limited impact.	1	Minimal construction required. No approvals required	2	minor capital cost, moderate life cycle cost	2	**6**
Zone 2 & 3 Pumping	W-P2-2	Stewart Road PS Additional Expansion	partial	Limited impact.	1	Limited impact.	1	Minimal construction required. No approvals required	2	minor capital cost, moderate life cycle cost	2	**6**
Zone 2 & 3 Pumping	W-P2-3	Davey Additional Pumping	partial	Limited impact.	1	Limited impact.	1	Minimal construction required. No approvals required	2	minor capital cost, moderate life cycle cost	2	**6**
Zone 2 & 3 Pumping	W-P2-4	Retrofit Osler Bluff BPS	partial	Existing site. Small footprint, may need a building.	3	Construction at existing site. Potential impact on community.	4	Moderate construction required. Design requiring potential approval	4	minor capital cost, moderate life cycle cost	3	**14**
	W-P3-1	New Zone 3 BPS	yes	Requires new site, potential impact on environment. Large footprint	5	Construction at new site. Potential impact on community	3	Flexibility in location and future expansion. Design requiring approval.	3	Moderate capital cost.	3	**14**

8.2.6 Preferred Alternatives

A summary of the preferred alternatives is shown in **Table 8.6**. The following sections provide additional information on the preferred alternatives.

Table 8.6 Preferred Alternatives

Deficiency	Preferred Alternative for Planned and Potential Growth (2044)	Preferred Alternative for Built Boundary
Supply – Total System	W-R-3: Water Efficiency Measures and W-R-4: WTP Upgrade	
Storage – Zone 1	W-S-1: New Zone 1 Elevated Tank	
Storage – Zone 2	W-S2-1 and W-S2-2 and Phase 3 Reservoir Expansion	W-S2-4: Davey Reservoir Expansion
Storage – Zone 3	W-S2-1 and W-S2-2 and Phase 3 Reservoir Expansion	W-S3-3: Combination of System Storage
Pumping – Zone 2	W-P2-1: Stewart Road Pumping Station Ultimate Pump Upgrades	W-P2-2: Davey Additional Pumping
Pumping – Zone 3	W-P2-1: Stewart Road Pumping Station Ultimate Pump Upgrades	W-P3-4: New Zone 3 Booster Pumping Station

8.2.6.1 Supply

The alternative of W-R-4 to upgrade the existing WTP was found to be the preferred alternative for meeting system supply deficiencies as it is the only alternative that would be able to fully meet future demands without limiting the growth of the system. It is recommended that the Town also undertake Water Efficiency Measures (W-R-3) to reduce the peak and total demand as much as possible in the future.

Possible water efficiency and conservation measures include enforcing existing irrigation by-laws, educating landscaping professionals about efficient irrigation practices, and enhancing public education efforts. Potential incentives for water efficient water softeners, garden features and pools could also improve conservation. A detailed Water Efficiency Study (W-O-3) is recommended to determine cost effective strategies specific to the Town of Collingwood.

8.2.6.2 Storage

The preferred alternative to meet storage demands in Zone 1 is W-S1-1B to build a new elevated tank in location 2. This option had a similar score to others for impacts on natural, social/heritage and technical environments but is expected to have a lower financial impact. This is a result of lower lifecycle costs because no additional pumping would be required. This option is preferable to W-S1-1A because the location is situated near industrial land use rather than residential and would require a less extensive feedermain network. The location is also well situated to address existing low-pressure areas of concern and supply improved fire flow to ICI customers. The preferred alternative for a new elevated tank includes taking the existing Collingwood ET offline.

To meet storage requirements in Zones 2 and 3, the preferred alternative was found to be a combination of increased storage at Davey in the short term, and floating storage to serve zone 2 and 3. New floating storage is preferable to in-ground storage because it maintains the HGL and does not require additional pumping. This reduces lifecycle costs. Additionally, floating storage has a smaller footprint than in-ground reservoirs. Elevated tanks and reservoirs should be designed to provide adequate water turnover to reduce water age and maintain quality.

8.2.6.3 Pumping

To meet pumping requirements in Zones 2 and 3 the preferred alternative was found to make use of the proposed floating storage near the Zone 2/3 boundary and add a new Zone 3 booster pumping station to service demands up to the built boundary. Additional pumping at Davey BPS is also recommended to service increased demand in Zone 2 up to the built boundary.

8.2.7 Hydraulic Performance of Preferred Alternative

The hydraulic performance of the system was assessed with the implementation of the proposed alternatives. The results from the 2032 and 2044 scenarios are provided for the minimum pressure, head loss, and fire flow analysis. **Figure 8-2** to **Figure 8-9** demonstrates the performance of the preferred alternatives with respect to minimum pressures, head losses and fire flow conditions for 2032 and 2044 conditions.

Only the supply, storage, pumping and major linear infrastructure projects were implemented in the model to demonstrate the impact on results. Local linear infrastructure projects and valve projects are expected to provide improvements at a local level and can be examined case by case as required.

The minimum pressure results for 2032 and 2044 MDD conditions showed significant improvement with the proposed new watermains and Zone 1 ET. The analysis included the proposed Zone 1 ET and the decommissioning of the existing Collingwood ET.

The head loss in terms of m/km was also improved by the addition of new watermains and location of the New Zone 1 ET. The central core of the Town no longer experienced high headloss over 3m/km during peak demand on MDD. The areas of red that are still visible are likely due to low C-factors in the model, which will be reviewed as part of the C-factor Testing and Calibration exercise. It is anticipated that the existing 400mm watermains near the WTP and along Hume Street can support more watermain capacity than demonstrated in the model, but unreasonably low C-factors show restricted flow. The watermains on Hwy 26 towards Carmichael BPS should also be reviewed, and the results may warrant offsetting the proposed Major Linear Project W-L1-1B scheduled for 2045.

The fire flow results were slightly improved from existing conditions. Many of the red nodes are located on dead-end street that have an existing hydrant. Local fire flow issues will be addressed primarily by local linear improvements and valve projects, which were not modelled in this scenario. Impact to local fire flows can be modelled on a case by case basis as required. Furthermore, local fire flow results in the model may be improved through the C-factor Testing and Model Calibration Project.

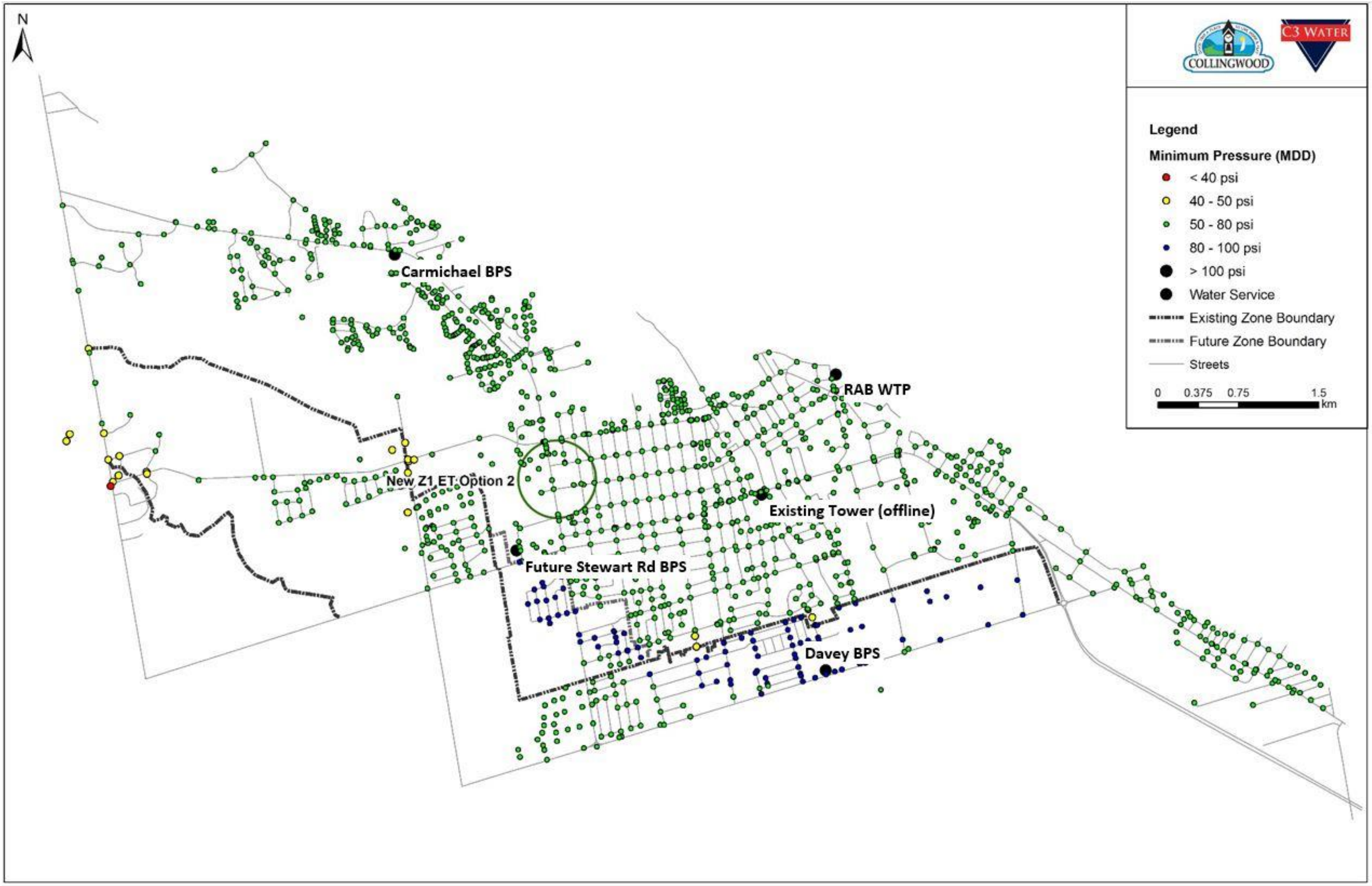


Figure 8-2 Preferred Alternatives - Minimum Pressures MDD, 2032 Conditions

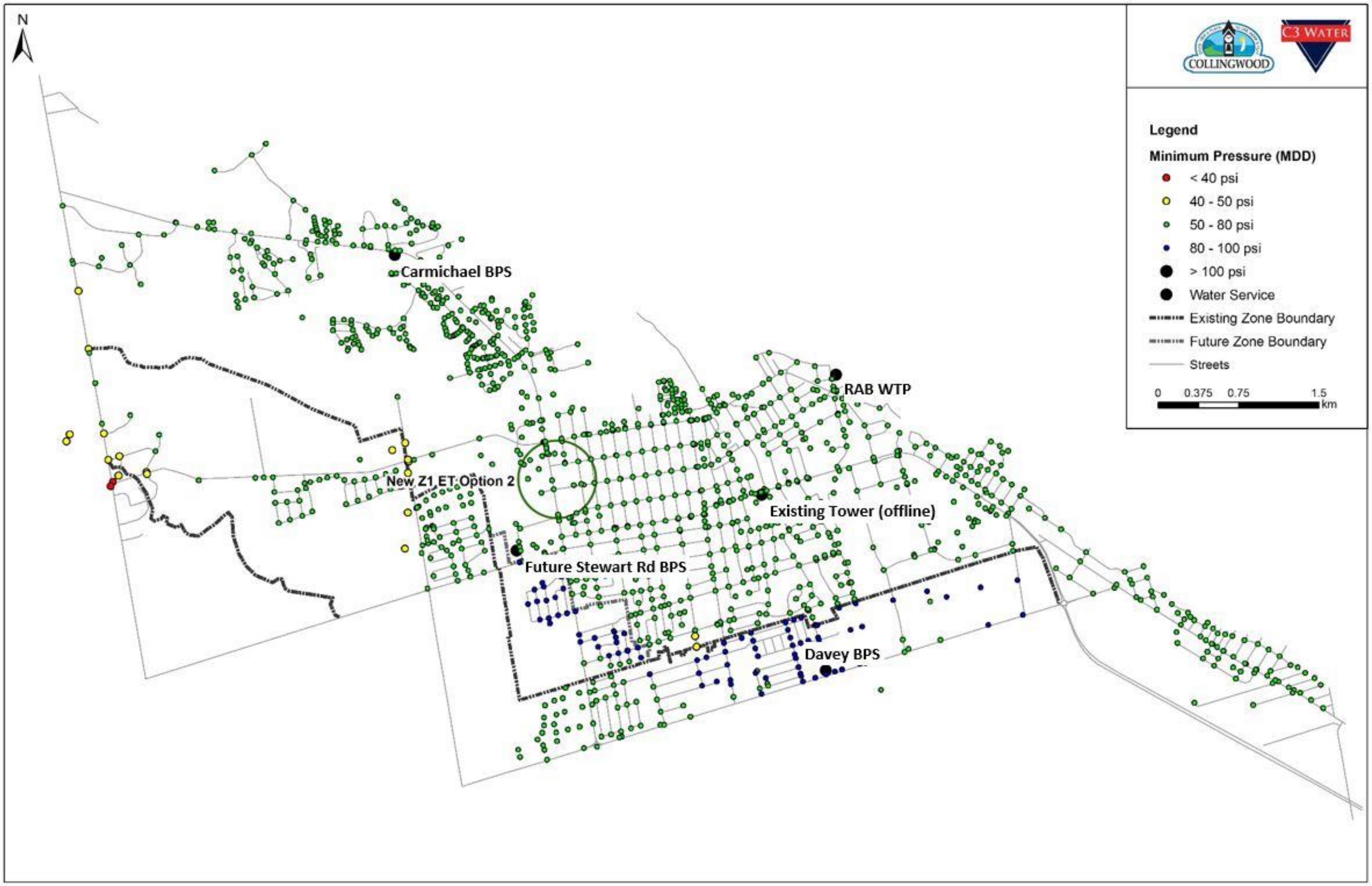


Figure 8-3 Preferred Alternatives - Minimum Pressures MDD, 2044 Conditions

Figure 8-4 Preferred Alternatives - Maximum Head Loss MDD, 2032 Conditions

Figure 8-5 Preferred Alternatives - Maximum Head Loss MDD, 2044 Conditions

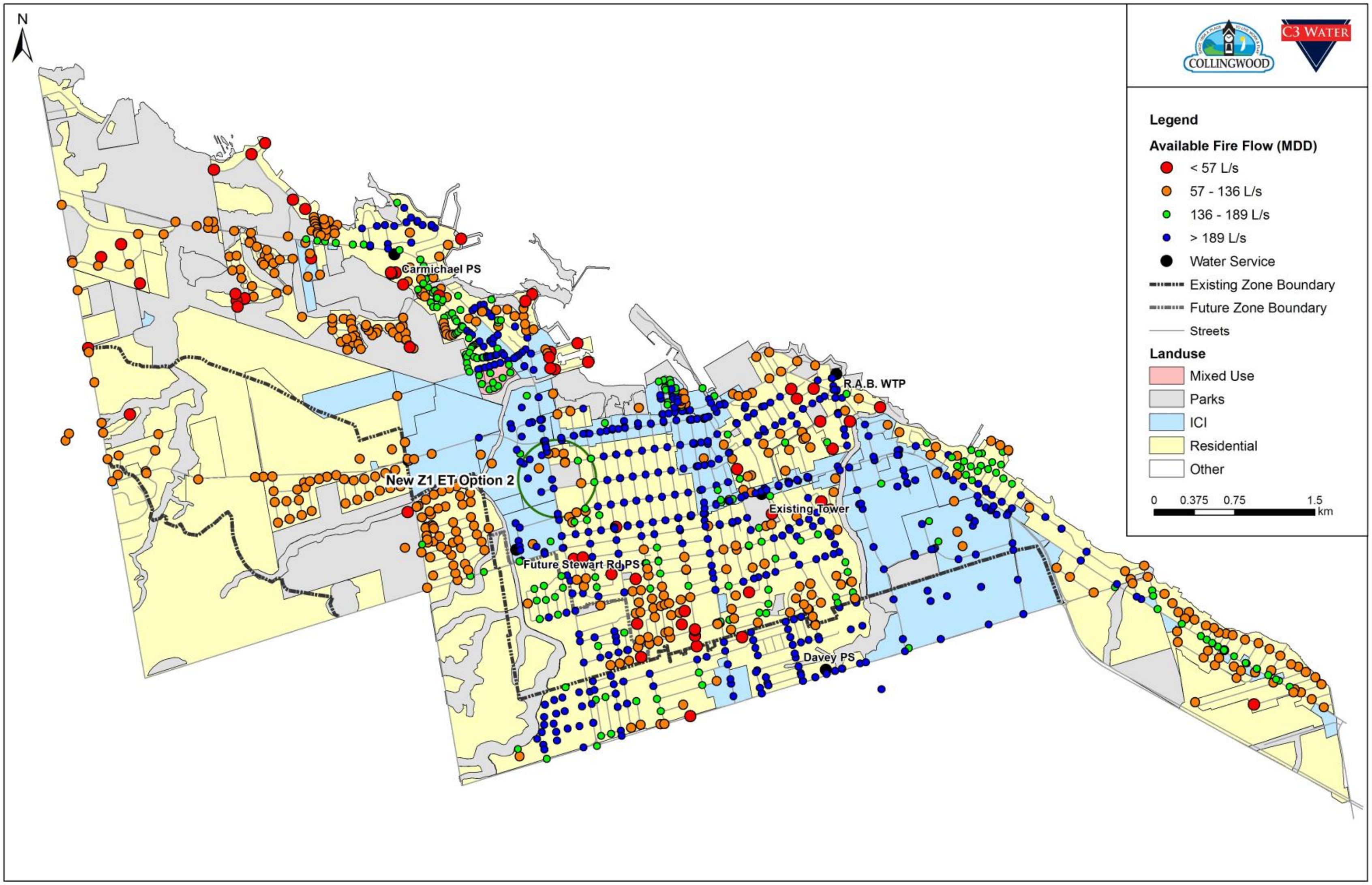


Figure 8-6 Preferred Alternatives – Available Fire Flow MDD, 2032 Conditions

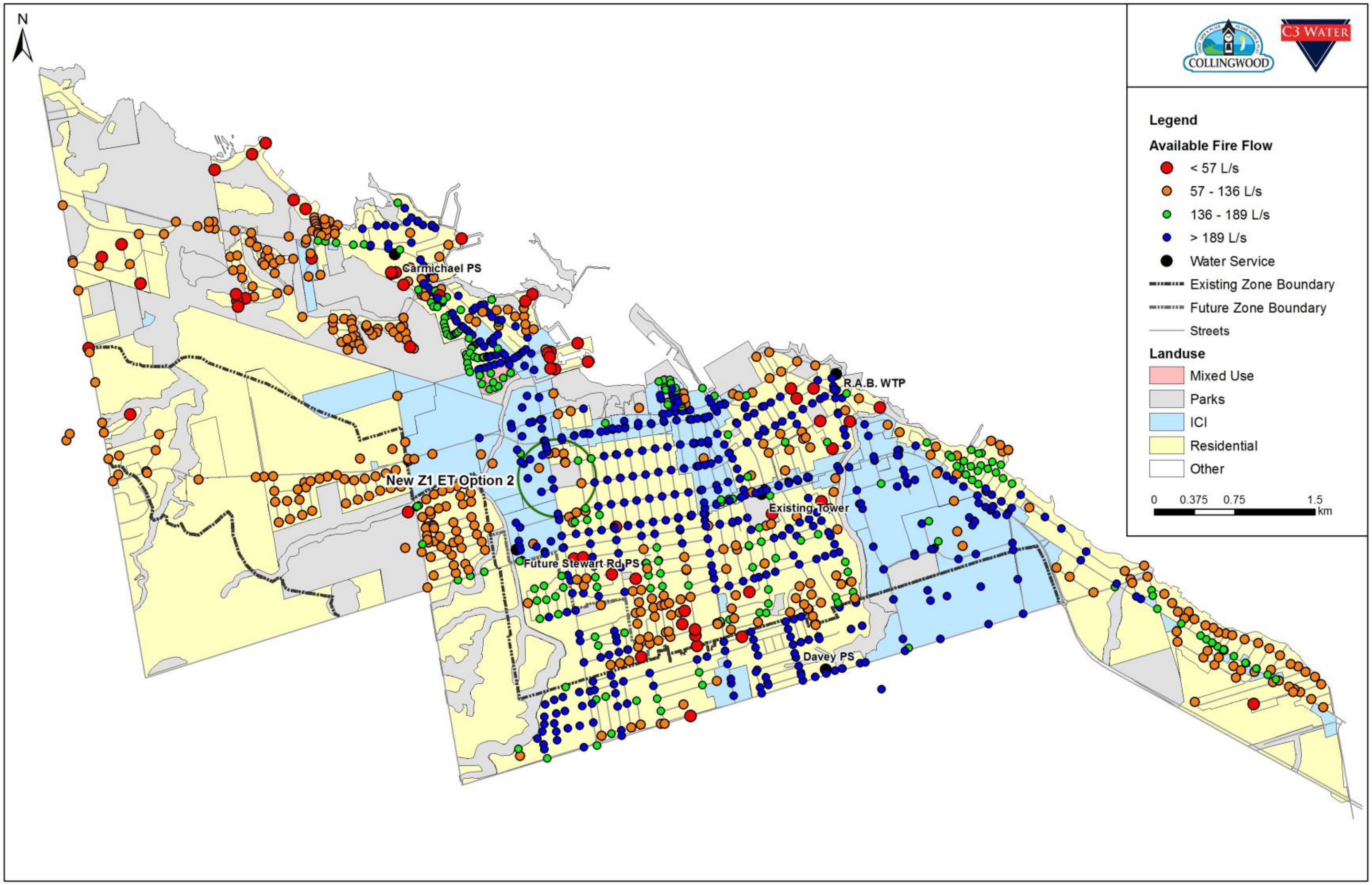


Figure 8-7 Preferred Alternatives – Available Fire Flow Results MDD for 2044 Conditions

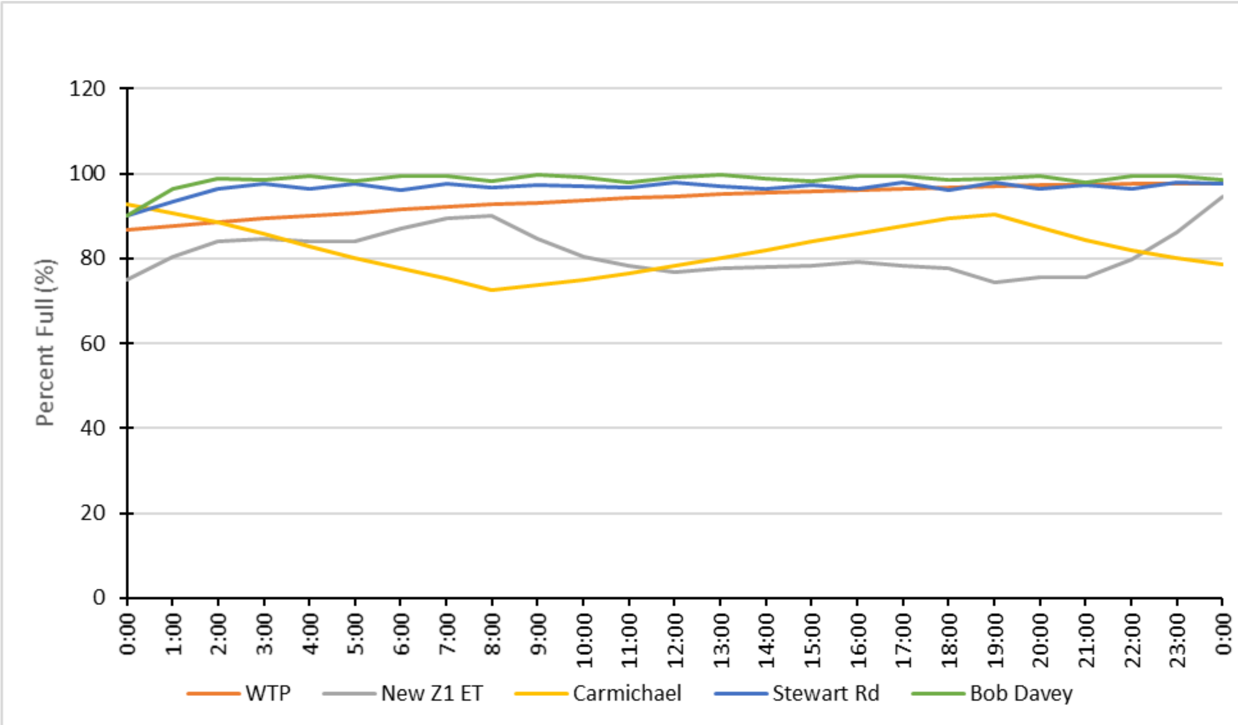


Figure 8-8 Storage Usage MDD 2032 with Alternatives

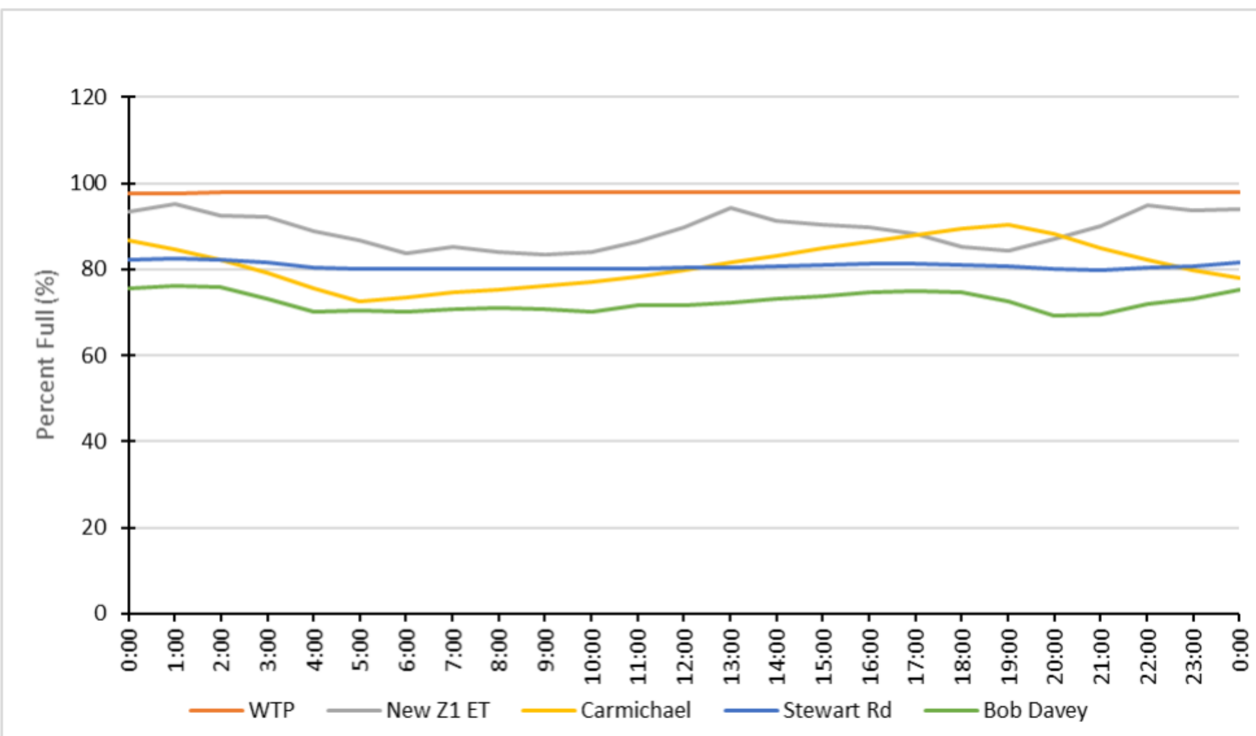


Figure 8-9 Storage Usage MDD 2044 with Alternatives

8.3 Sanitary System Alternatives

A series of alternatives have been developed to address future sanitary servicing requirements. Alternatives were developed to address planned and potential growth. Consideration of oversizing to allow for servicing of the built boundary, servicing of flows from Nottawa and servicing of unserviced properties is discussed in **Section 9**. Alternatives were developed for the treatment system and for conveyance separately.

8.3.1 Treatment Alternatives

Treatment alternatives are necessary to provide treatment capacity for flows resulting from existing development, planned development and potential development. Upon completion of all planned and potential developments, the projected average flow will be 28,948m^3/d. All treatment alternatives were sized assuming that this value will equate to 80% of the rated treatment capacity. Therefore, all treatment alternatives were developed to provide a capacity of 36,185m^3/d. Alternatives evaluated are described in the following sections.

8.3.1.1 Alternative ST-1: Expansion of Collingwood WWTP and Retrofit of A.G. Global WWTP

This alternative involves the expansion of the Collingwood WWTP and retrofit of the existing A.G. Global WWTP to process a total treatment capacity of 36,185m^3/d.

The A.G. Global Wastewater Treatment Facility was formerly used as an industrial wastewater pre-treatment plant associated with an ethanol and starch production facility. The plant provided pre-treatment of production wastewater prior to discharge into the Town of Collingwood's sanitary sewer system until 2012. It was a two-stage activated sludge facility consisting of two stages. Stage 1 consisted of pre-treatment clarifiers, aeration tanks, and secondary clarifiers. Stage II consisted of sedimentation DAF tanks, lift stations, aerobic digesters, sludge dewatering and a nutrient dosing system. An inspection completed in 2015 identified that plant infrastructure has been degraded. The pre-treatment clarifier and aerobic digesters were classified as being in fair condition while the Stage1 aeration tanks were classified as being in poor condition. The plant site is also equipped with an existing outfall sewer that currently conveys stormwater and non-process water to Georgian Bay. No information is available on the elevation of this outfall sewer. A 2015 Preliminary Capacity Assessment identified retrofit requirements with an estimated capital cost of $5.4M. The Preliminary Capacity Assessment also identified that a retrofit of the existing facility would result in an available rated capacity of 1,100m^3/d. The report identified the following works/upgrades/ studies would be required to utilize this facility as a municipal wastewater treatment facility:

- A detailed hydraulic and assimilative capacity assessment, including flow monitoring and confirmation of all connections, modelling and mixing zone assessment would be required for the existing outfall that extends into Georgian Bay. The assessment would be needed to confirm that the current outfall has sufficient capacity to convey treated wastewater and identify effluent criteria for the A.G. Global WWTP facility.
- A Schedule C Class EA would be required to complete Phases 3 and 4 of the EA process. This Master Plan would meet the requirements of Phases 1 and 2 but Phases 3 and 4 would be required for a Schedule C project. Phases 3 of the EA process would develop and evaluate alternative design concepts for retrofitting the existing facility.

- A water resources impact assessment would be required to confirm the requirements for an extension of the outfall into Georgian Bay.

This facility is located on MacDonald Street and modifications to the sanitary sewer system would be necessary to utilize this facility. The Preliminary Capacity Assessment suggested that this plant could be used to provide servicing to the community of Nottawa.

Based on the capacity requirements identified in **Section 7**, the additional capacity that could be provided by this facility is not sufficient to meet the needs of planned growth, planned and potential growth or servicing of Nottawa. Therefore, a rated capacity increase at the Collingwood WWTP to a rated capacity of 36,185m^3/d would be required. As the plant has historically been designed with a peaking factor of 2.5, the peak flow capacity of the Collingwood WWTP would be increased to 90,463m^3/d. To address high peak flows in the sanitary sewer system, this alternative would include either storage tanks or wet weather treatment.

An Addendum to the 2011 Schedule ‘C’ Class EA for the expansion of the Collingwood WWTP would be needed. The 2011 Schedule ‘C’ Class EA recommended compact treatment technology for a 12,000m^3/d capacity expansion which could be implemented in two 6,000m^3/d increments to match growth. Expansion to a rated capacity of 36,185m^3/d will ensure that the WWTP operates at 80% of its rated capacity at the end of the planned and potential growth period. Relocation of the plant outfall to deeper waters in the Harbour would also be needed to comply with MECP policy B-1-5.

8.3.1.2 Alternative ST-2: Expansion of Collingwood WWTP

This alternative would involve expansion of the Collingwood WWTP to provide a rated capacity of 36,185m^3/d and a peak flow capacity of 90,463m^3/d (based on a 2.5 peaking factor). The 2011 Schedule ‘C’ Class EA for the expansion of the Collingwood WWTP recommended compact treatment technology for a 12,000m^3/d capacity expansion which could be implemented in two 6,000m^3/d increments to match growth. To address high peak wet weather, an equalization storage tank may be required at the influent to the WWTP. A storage volume of less than 1,000 m^3 will be sufficient to meet peak flow needs. Expansion to a rated capacity of 36,185m^3/d will ensure that the WWTP operates at 80% at completion of all planned and potential development. Relocation of the plant outfall to deeper waters in the Harbour would also be needed to comply with MECP policy B-1-5. As part of this alternative, the A.G. Global WWTP would not be retrofitted and would not be utilized for treatment of municipal wastewater.

8.3.1.3 Alternative ST-3: Do Nothing

The Do Nothing alternative would allow growth to proceed without any upgrades to treatment capacity. This alternative would have significant impact on the environment as insufficient treatment capacity would be available to treat the sanitary flows generated. This alternative was determined to be infeasible and was eliminated from consideration.

8.3.1.4 Alternative ST-4: Limit Growth

This alternative would involve reducing future growth to within the capacity of current systems. This would involve limiting future residential, industrial, and commercial and institutional growth. This alternative does not comply with the Town’s Official Plan and is considered infeasible. This alternative was eliminated from further consideration.

8.3.1.5 Alternative ST-5: Demand Management

This alternative would consist of reducing flows from existing developments through water conservation and through inflow and infiltration (I/I) reduction to provide sufficient capacity to service future growth. Water demand reductions could be achieved through a variety of means including promoting the installation of water efficient fixtures, public education and increased water rates. I/I reduction could be achieved through repairs to the Town's sanitary sewers and maintenance holes and repairs to sanitary sewer laterals on private property. Private property programs would be geared to reducing I/I from private property sources and would be required to achieve the necessary reductions.

An aggressive I/I reduction program could also be undertaken on a pilot area basis to address private property and Town of Collingwood infrastructure. The pilot approach would enable the Town to identify and complete needed modifications and repairs and assess the cost effectiveness of the repairs through post construction monitoring. Program activities would include private property inspections and smoke and dye tests to locate direct connections, mandatory disconnection of any identified direct connections, additional CCTV and maintenance hole inspections to identify deficiencies in the Town's sewer system, completion of sewer system rehabilitation and post construction flow monitoring to assess the cost effectiveness of the rehabilitation completed.

It has been noted that peak flows to the Collingwood WWTP are high and can exceed the peak flow capacity of the plant. Implementation of this alternative would result in reductions in the peak flow reaching the plant and could mitigate the need to address peak wet weather flows at the plant. However, significant reductions in water use and average day flows are unlikely to be achieved based on experience in other municipalities. A reduction in water use and average day flows in the range of 0-10% will not be sufficient to eliminate the need for additional capacity. Therefore, demand management will not be sufficient to fully meet servicing needs for planned and potential growth. This alternative can be implemented in conjunction with other system improvements to meet future needs. Additional wastewater treatment would be still be required.

8.3.2 Sanitary Sewer System Alternatives

Sanitary sewer system alternatives were developed and evaluated to address specific capacity constraints identified during the analysis of future needs. These alternatives address future needs which are not related to the capacity constraint of the Collingwood WWTP. These alternatives address:

- Local improvements in areas where performance criteria was not met; and
- Improvements/ modifications to the trunk sewer system designed to reduce surcharge conditions within the trunk sewer system.

8.3.2.1 Local Improvement Alternatives

A review of the results for the June 17, 2017 event for planned and potential growth identified the following constraints in the local sanitary sewer system:

- Sections of the Mountain Road sanitary sewer immediately west of High Street were identified as not meeting performance criteria;
- One section of sanitary sewer on Huron Street immediately upstream of the Minnesota SPS;
- Three sections of the Minnesota Street sanitary sewer south of Simcoe Street did not meet performance criteria;

- Two sections of the Hurontario sanitary sewer between Lockhart and Collins Street did not meet performance criteria; and
- Two existing siphons on Spruce Street and Hickory Street meet performance criteria but have been prone to plugging and have caused basement flooding upstream when plugged.

Capacity constraints at the Collingwood WWTP are not responsible for these local system constraints. The following alternatives were developed:

Local Alternative L1 – Sewer Capacity Increases

Table 8.7 presents the sewer improvements included in Alternative L1.

Table 8.7 Alternative L1 - Required Infrastructure Improvements

Location	Description
Minnesota Street	Replacement of 380 of existing 300mm diameter sanitary sewer on Minnesota Street south of Simcoe Street with new 375mm diameter sanitary sewer
Hurontario Street	Replacement of 221m of existing 350mm diameter sanitary sewer on Hurontario Street between First and Second Streets with new 450mm diameter sanitary sewer Replacement of 368m of existing 350mm diameter sanitary on Hurontario Street between Collins and Lockhart with a new 375mm diameter sanitary sewer
Mountain Road	Replacement of 96m of existing 450mm and 500mm diameter sanitary sewer on Mountain Road west of High Street with new 600mm diameter sanitary sewer
Huron Street	Replacement of 19m of existing 450mm diameter sanitary sewer on Huron Street immediately upstream of the Minnesota SPS with new 750mm diameter sanitary sewer
Hickory and Spruce Street siphons	Decommissioning of existing siphon structures and construct new pumping stations and forcemains with connection to the Habourfront Trail Sanitary Trunk Sewer

Hydraulic modelling confirmed that this alternative would meet the level of service requirements for planned and potential growth. Impacts on downstream infrastructure would be minimal. This alternative does result in an increase to the peak flow entering the Minnesota SPS. At the Minnesota SPS, the peak flow entering the station would be 291L/s, which is greater than the station's firm capacity but less than the station capacity.

Local Alternative L2 – Flow Diversion Modification

Flow diversion modifications could be used to improve hydraulic conditions. There is no opportunity to modify existing flow diversion chambers to address conditions in the Mountain Road, Huron Street or Minnesota Street sanitary sewers. There is an opportunity to modify the flow diversion chamber at Hurontario and Second Street to divert additional flow to the Second Street sanitary sewer. **Table 8.8** presents the improvements identified in this alternative.

Table 8.8 Alternative L2 – Required Infrastructure Improvements

Location	Description
Minnesota Street	Replacement of 380 of existing 300mm diameter sanitary sewer on Minnesota Street south of Simcoe Street with new 375mm diameter sanitary sewer
Hurontario and Second Street	Installation of a weir to control flows into the downstream Hurontario Street sanitary sewer and direct additional flow towards Second Street sanitary sewer
Hurontario Street	Replacement of 368m of existing 350mm diameter sanitary on Hurontario Street between Collins and Lockhart with a new 375mm diameter sanitary sewer
Mountain Road	Replacement of 96m of existing 450mm and 500mm diameter sanitary sewer on Mountain Road west of High Street with new 600mm diameter sanitary Sewer
Huron Street	Replacement of 19m of existing 450mm diameter sanitary sewer on Huron Street immediately upstream of the Minnesota SPS with new 750mm diameter sanitary sewer
Hickory and Spruce Street siphons	Decommissioning of existing siphon structures and construct new pumping stations and forcemains with connection to the Harbourview Trail Trunk Sewer

Hydraulic modelling confirmed that this alternative would meet the level of service requirements for planned and potential growth. Impacts on downstream infrastructure would be minimal. This alternative does result in an increase to the peak flow entering the Minnesota SPS. At the Minnesota SPS, the peak flow entering the station would be 291L/s, which is greater than the station's firm capacity but less than the station capacity.

Local Alternative L3 – Inflow and Infiltration Reduction

Local Alternative L3 is a local targeted inflow and infiltration reduction program to address high wet weather flows. The program would be targeted to address the Minnesota Street, Hurontario Street and Huron Street sanitary sewers and would achieve reductions in peak flow. In the Mountain Road area, the program would be targeted to achieve lower infiltration values than the Town's design criteria of 0.23L/s/ha for new developments.

For the Minnesota Street and Hurontario Street areas, the program would consist of the following major tasks:

- Small area flow monitoring or micromonitoring to identify specific streets where excessive infiltration and inflow enters the sanitary sewer system. Data analysis would be completed to compare dry and wet weather monitored flows against the Town's design criteria to identify areas of concern. Further review of the data would also identify whether the issue is likely related to direct connections (quick wet weather response) or infiltration (slower and longer lasting response).

- For each area of concern identified through the flow monitoring data analysis, available data would be reviewed to identify sources. This data may include historical CCTV or maintenance hole inspection data. A field program would also be designed to collect additional data. If direct connections are probable based on data analysis results, a program of smoke and dye testing would be necessary to identify direct connections on private property or within the municipal system. Where infiltration is suspected, additional and up to date CCTV inspection and maintenance hole inspections would be necessary to identify cracks in pipes and maintenance holes. CCTV inspections can also be undertaken during rainfall events to clearly identify pipe defects which allow infiltration flows to enter the sanitary sewer. Wet weather CCTV inspections, while difficult to schedule, can pinpoint the defects which contribute the most infiltration to the sanitary sewer and contribute valuable information for remediation program development.

- For existing areas, a remediation program would be developed to identify and prioritize required works. The remediation program would consist of rehabilitation capital improvements and programs to reduce I/I. For each defect identified, rehabilitation techniques would be identified to address specific sources. For the municipal system, this is likely to take the form of sanitary sewer spot lining, full pipe lining or replacement, sanitary maintenance hole spot repairs or lining. The remediation plan should consider low cost works such as spot repairs first. In cases where a pipe requires an excess number of spot repairs or if there is a significant joint displacement, full pipe lining or replacement may be the only options. Programs to reduce I/I would be focused on private property and could include incentives or rebates to encourage property owners to address private property sources. A number of municipalities have had rebate programs for downspout disconnection, sump pump removal and foundation drain disconnection and most of these municipalities have offered rebates to cover approximately 50% of the total costs. Strict By-Law enforcement may be used to remove any remaining direct connections.

As the Mountain Road area is a new development area, a program aimed at developers would be needed. The Town could require engineering consultants to provide post construction flow monitoring and assessment to prove that peak flows from a development area are below the Town's design standards following construction and occupancy. Several municipalities have made this type of program part of their development assumption process and require a certification from the development engineer prior to release of letters of credit.

To assess whether the above program could achieve needed results, the hydraulic model was modified to remove the fast and medium infiltration component of the wet weather response. This is a reasonable approach used for master planning. This change was made for a total of 3.4ha within the Hurontario Street area, 59ha within the Minnesota Street tributary area. Model results indicate that an aggressive I/I program will be sufficient to meet level of service requirements on Hurontario and Minnesota Streets. This alternative also includes sewer replacements on Hurontario Street, Huron Street, Mountain Road and decommissioning of the existing siphons on Hickory and Spruce Streets. **Table 8.9** provides details on the infrastructure improvements necessary.

Table 8.9 Alternative L3 – Required Infrastructure Improvements

Location	Description
Hurontario Street	Replacement of 368m of existing 350mm diameter sanitary on Hurontario Street between Collins and Lockhart with a new 375mm diameter sanitary sewer
Mountain Road	Replacement of 96m of existing 450mm and 500mm diameter sanitary sewer on Mountain Road west of High Street with new 600mm diameter sanitary Sewer

Table 8.9 Alternative L3 – Required Infrastructure Improvements

Location	Description
Huron Street	Replacement of 19m of existing 450mm diameter sanitary sewer on Huron Street immediately upstream of the Minnesota SPS with new 750mm diameter sanitary sewer
Hickory and Spruce Street siphons	Decommissioning of existing siphon structures and construct new pumping stations and forcemains with connection to the Harbourview Trail Trunk Sewer
I/I Reduction Program	Program of I/I reduction (sewer flow monitoring, inspection, rehabilitation program development and construction) for 63ha of property in the vicinity of Hurontario Street and upstream of the Minnesota SPS

8.3.3 Trunk Sewer System Improvement Alternatives

Trunk sanitary sewer system improvements will be necessary, in combination with a treatment plant upgrade, to reduce peak hydraulic gradelines and meet performance criteria in the Town's trunk sewer system. A total of five alternatives were analyzed assuming that the treatment plant would be upgraded to provide a peak capacity of 90,463m^3/d or 1,047L/s. This peak capacity was calculated based on a 2.5 peaking factor, which is consistent with the design of the current WWTP and consistent with treatment plant design guidelines (Ontario, 2008). Trunk sewer improvements alternatives include a new Harbourview Trail Trunk Sewer, a new Black Ash SPS forcemain to direct flow to the headworks of the Collingwood WWTP, a Town wide demand management program, limit growth and do nothing. The following sections provide further detail on each alternative.

8.3.3.1 Alternative SC-1: New Harbourview Trail Trunk Sewer

This alternative would consist of a new Harbourview Trail Trunk Sewer, extending from High Street to the Collingwood WWTP Pumping Station at Birch Street. As part of this alternative, the existing Harbourview Trail Trunk Sewer would remain in service. The new trunk sewer would be located within the Harbourview Trail corridor and would provide an additional 530L/s of capacity. This new trunk sewer, with a diameter of 750mm, would be constructed within a narrow trail corridor. In order to ensure sufficient space, it is anticipated that this trunk sewer will need to be constructed using tunneling methods. This alternative would also include the construction of new Black Ash SPS forcemain from the existing 500mm diameter forcemain section located south of the Black Ash SPS (currently capped) to the Harbourview Trail Trunk Sewer at Balsam. This new 500mm diameter forcemain would have a capacity of 590L/s.

Hydraulic modelling identified the need for additional works at the intersection of Balsam and High Street to divert sufficient flow to a twin Harbourview Trail Trunk Sewer to eliminate surcharge conditions within the trunk sewer system. These improvements would include the following:

- Removal of the existing sanitary sewer which allows flow to travel westward from MH122-001; and,
- Replacement of the existing 450mm diameter sanitary sewer that connects the Mountain Road sanitary sewer to the Harbourview Trail Trunk sewer from MH H26N-31 to MH122-001 with a new 750mm diameter sanitary sewer.

Hydraulic modelling identified that implementation of this alternative would effectively control peak hydraulic grade lines within the Town's trunk sewer system. For this alternative, the d/D value for all trunk sanitary sewers would be 0.85 or less, which meets the performance criteria. A peak flow of 1,046L/s was predicted through the Collingwood WWTP. A bypass was not predicted to occur as a result of the June 17, 2017 event.

8.3.3.2 Alternative SC-2: New Black Ash SPS Forcemain

This alternative would consist of the construction of a new 500mm diameter forcemain connecting the Black Ash SPS to MH-C at the Collingwood WWTP. MH-C is the plant bypass chamber located upstream of the Collingwood WWTP pumping station. The Town has already constructed a short section of 500mm diameter forcemain that crosses underneath Black Ash Creek. This new section of forcemain is currently capped at both ends. The new forcemain would utilize this existing 500mm diameter section of forcemain and be constructed in parallel to the existing forcemain along Highway 26 and then continue eastwards along the Harbourview Trail Corridor to MH-C. The new forcemain would have a maximum capacity of 590L/s, which exceeds the Black Ash SPS station capacity. Higher head pumps would not be required at the Black Ash SPS as the new forcemain would have a similar head loss to the existing 300mm diameter forcemain. To analyze this alternative, it was assumed that the existing forcemain would be decommissioned. However, the Town could chose to retain the existing forcemain to provide additional redundancy.

Hydraulic modelling identified that implementation of this alternative would effectively control peak hydraulic grade lines within the Town's trunk sewer system. For this alternative, the d/D value for all trunk sanitary sewers would be 0.85 or less, which meets the performance criteria. A peak flow of 1,046L/s was predicted through the Collingwood WWTP. A bypass was not predicted to occur as a result of the June 17, 2017 event.

8.3.3.3 Alternative SC-3: Demand Management

This alternative would consist of reducing flows from existing developments through water conservation and through inflow and infiltration (I/I) reduction to provide sufficient capacity to service future growth. Water demand reductions could be achieved through a variety of means including promoting the installation of water efficient fixtures, public education and increased water rates. I/I reduction could be achieved through repairs to the Town's sanitary sewers and maintenance holes and repairs to sanitary sewer laterals on private property. Private property programs geared towards reducing I/I from sources located on private property would be needed to achieve required reductions.

Specific elements of a Town wide demand management program would include a comprehensive water conservation program aimed at both residential and ICI water users. The program would By-Laws to enforce the installation of water efficient fixtures, rebates to existing residential users to encourage the replacement of old fixtures with new water efficient fixtures, incentives for industrial water uses to encourage reuse for industrial processes. For new development, standards and guidelines could be altered to require higher water efficiency fixtures.

An I/I reduction program would also be needed. The Town wide program would be similar in nature to the one described for Local System Alternative L1 but would be implemented on a Town-wide basis.

8.3.3.4 Alternative SC-4: Do Nothing

The Do Nothing alternative would allow growth to continue without any upgrades to the Town's sanitary systems. This is not recommended as a feasible alternative as it would have significant impact on the environment as insufficient treatment capacity would be available to treat the sanitary flows generated. This alternative was eliminated from further consideration.

8.3.3.5 Alternative SC-5: Limit Growth

This alternative would involve reducing future growth to within the capacity of the current systems. This would involve limiting future residential, industrial, and commercial and institutional growth. This alternative does not comply with the Town's Official Plan and is not considered feasible. This alternative was eliminated from further consideration.

8.3.4 Evaluation of Alternatives

Each of the above alternatives were evaluated based on the natural, social, technical and financial criteria. Scoring was assigned on a scale from one to five as outlined in **Table 8.1**. The alternatives with the lowest overall scores were identified as the recommended preferred alternatives. **Table 8.10** presents the evaluation of alternatives.

Table 8.10 Evaluation of Sanitary System Alternatives

	Alternative #	Description	Meets Quantity Requirements?	Natural Environment		Social / Heritage Environment		Technical Environment		Financial Environment		Overall Score
Wastewater Treatment				**Weight:**	**1**		**1**		**1**		**1**	
Wastewater Treatment	ST-1	Expansion of Collingwood WWTP and Retrofit of A.G. Global WWTP	Yes	Expansion of Collingwood WWTP will have impact on local environment surrounding the facility.	3	Construction impacts at two WWTP sites. Improvements and impacts may also occur due to collection system improvements needed to connect A.G. Global WWTP to the Collingwood sanitary sewer system. Potential for odour issues and nuisance complaints from residents possible at the A.G. Global facility due to the close proximity of residents and commercial properties.	3	Significant additional study will be required to confirm state of the current A.G. Global WWTP and assess requirements. Condition assessment indicates that significant retrofits and replacement of existing equipment and tanks will be necessary. EA Addendum, preliminary and detailed design of improvements at the Collingwood WWTP will need to be completed. Design of improvements at Collingwood WWTP will need to ensure adequate land area is available to accommodate future expansions to allow for servicing of built boundary growth.	5	High capital costs and significant increase in lifecycle costs due to operation of two wastewater treatment facilities.	5	**14**
	ST-2	Expansion of Collingwood WWTP	Yes	Expansion of Collingwood WWTP will have impact on local environment surrounding the facility.	3	Construction impacts at the Collingwood WWTP site.	3	EA Addendum, preliminary and detailed design of improvements at the Collingwood WWTP will need to be completed. Design of improvements at Collingwood WWTP will need to ensure adequate land area is available to accommodate future expansions to allow for servicing of built boundary growth.	3	High capital costs. Lifecycle costs for this alternative will be lower than S-T-1 as only one plant will be operated.	3	**12**
	ST-3	Do Nothing	No	Significant impact on the environment as all of the wastewater generated in the service area will not be treated	5	No construction impacts. No impact on existing land uses. Alternative does not meet provisions of Town's Official Plan as servicing will not be provided for future growth.	5	The Collingwood WWTP will be unable to meet Provincial regulations without additional capacity.	5	No additional costs.	1	**16**
	ST-4	Limit Growth	No	No Impact are anticipated.	1	No construction impacts. No impact on existing land uses. Does not meet Town's and County's Official Plans as well as Growth Plan for Greater Golden Horseshoe provisions.	5	Alternative does not meet project requirements.	5	No additional costs.	1	**12**

Table 8.10 Evaluation of Sanitary System Alternatives

	Alternative #	Description	Meets Quantity Requirements?	Natural Environment		Social / Heritage Environment		Technical Environment		Financial Environment		Overall Score
	ST-5	Demand Management	Partial	Demand management systems will not be sufficient to reduce projected flows to within capacity of the Collingwood WWTP. Alternative will have environmental impacts as the wastewater system will not be sufficient to provide treatment to future flows.	5	Construction impacts will be limited to local impacts associated with sewer rehabilitation. On-lot improvements may also be necessary with impacts to local property owners.	5	Alternative will require significant public programs aimed at residents. As the success of these programs is unknown in Collingwood, this alternative has significant risks. Alternative only partially meets requirements.	5	Additional costs associated with programs and with sewer and maintenance rehabilitation. Alternative could also reduce available rate based funding if the Town maintains current water rates. This will occur as the volume of water purchased by customers will be reduced.	2	**17**
Local System Alternatives	L-1	Local Sewer Capacity Increases	Yes	Minimal impacts on the natural environment as alternative involves replacement of existing sanitary sewers located within existing road allowances.	1	Construction impacts such as noise, dust, traffic impacts can be mitigated through good construction practices. Replacement of 121m of sanitary sewer on Hurontario Street between First and Second Street could impact businesses in the downtown district along Hurontario Street.	3	All sewer replacements are located within existing road allowances. Design of improvements would consider twin construction to maintain current operation of system. Pumping stations necessary to replace Hickory and Spruce Street siphons will be small and can be located within existing road allowances.	2	Estimated capital of $1.2M for improvements. Increase in O&M costs with small pumping stations on Hickory and Spruce Street	2	**8**
	L-2	Flow Diversion Modification on Hurontario Street at Second Street and Local Sewer Capacity Increases	Yes	Minimal impacts on the natural environment as alternative involves replacement of existing sanitary sewers located within existing road allowances and modifications to flow diversion chamber located within existing road allowance.	1	Construction impacts are anticipated due to noise, dust and traffic. These impacts will be less than Alternative L-1 as no sewer construction is required on Hurontario Street between First and Second Streets. Minor construction activities will be associated with changes to the existing flow diversion chamber at Hurontario Street and Second Street.	2	Construction impacts are anticipated due to noise, dust and traffic. These impacts will be less than Alternative L-1 as only minor construction would be required at the intersection of Hurontario and Second Street to modify the existing flow diversion chamber. Trenchless repairs may also be necessary on Hurontario Street to the existing sewer.	1	Estimated capital of $1.1M for improvements. Increase in O&M costs with small pumping stations on Hickory and Spruce Street	2	**6**
	L-3	Targetted I/I Reduction Program for Hurontario and Alice Street Target Areas and Local Sewer Capacity Increase	Partial	Minimal impacts on the natural environment as alternative involves replacement of existing sanitary sewers located within existing road allowances and repairs to existing infrastructure to address wet weather sources in target areas	1	Construction impacts are anticipated due to noise, dust and traffic. These impacts will be less than Alternative L-1 as no construction is required on Hurontario between First and Second Streets. Completion of smoke and dye tests will require notification of residents.	2	Construction impacts are anticipated due to noise, dust and traffic. These impacts will be less than Alternative L-1 as no construction is required on Hurontario between First and Second Streets. Repairs to existing infrastructure to reduce I/I will generally be completed using trenchless methods.	1	Estimated capital of $1.9M for improvements and I/I reduction program. Increase in O&M costs with small pumping stations on Hickory and Spruce Street	3	**7**

Table 8.11 Evaluation of Sanitary System Alternatives

	Alternative #	Description	Meets Quantity Requirements?	Natural Environment		Social / Heritage Environment		Technical Environment		Financial Environment		Overall Score
Pumping Station Improvements				**Weight:**	**1**		**1**		**1**		**1**	
Trunk Sanitary Sewer System Alternatives	SC-1	New Harbourview Trail Trunk Interceptor Sewer with Local Improvements	Yes	New Harbourview Trail Trunk Sewer would be constructed within existing easement and utility corridor. Any environmental impacts can be mitigated through best practice design and construction	3	Construction will occur within utility corridor and will impact on users of existing trails during construction.	3	Space is limited in existing utility corridor for construction of new trunk sewer. To construct the trunk sewer within the available space will require tunnelling construction. Detailed design will require additional time and additional geotechnical and hydrogeological information and data collection to support design. Higher construction risks associated with this type of construction.	3	High capital costs due to tunnelling construction. Minimal change to existing O&M costs.	5	**14**
	SC-2	New Black Ash SPS Forcemain and Local Improvements	Yes	New Black Ash SPS Forcemain will be constructed within existing easement and utility corridor. Any environmental impacts can be mitigated through best practice design and construction.	2	Construction will occur within utility corridor and will impact on users of existing trails during construction.	3	Space is limited in existing utility corridor but corridor can accommodate new forcemain constructed using open cut methods. New forcemain will also increase system redundancy if the Town elects to maintain the existing 300mm diameter Black Ash SPS forcemain.	1	Moderate capital cost. O&M costs will increase due to increased head required for pumping station.	3	**11**
	SC-3	Do Nothing	No	Existing sanitary sewers have inadequate capacity to service future growth. Overflows and environmental discharges will occur.	5	Alternative does not meet project requirements.	5	Alternative does not meet requirements	5	No additional costs	1	**16**
	SC-4	Limit Growth	No	No Impacts	1	Alternative does not meet project requirements.	5	Alternative does not meet requirements	5	No additional costs	1	**12**
	SC-5	Demand Management	Partial	Existing sanitary sewers with demand management will still not have inadequate capacity to service future growth. Overflows and environmental discharges will occur.	5	Construction impacts will be limited to local impacts associated with sewer rehabilitation. On-lot improvements may also be necessary with impacts to local property owners.	3	Alternative will only partially meet project requirements	3	Additional costs associated with programs and with sewer and maintenance rehabilitation. Alternative could also reduce available funding for water and sanitary systems, if the Town elects to maintain current water rates. This will occur as the volume of water required by customers would be reduced.	3	**14**

8.3.5 Preferred Sanitary System Alternatives

A summary of the preferred sanitary system alternatives is shown in **Table 8.12**. The following sections provide additional information on the preferred alternatives. **Figure 8-10** presents the location of the preferred sanitary system alternatives.

Table 8.12 Preferred Sanitary System Alternatives

Deficiency	Preferred Alternative for Planned and Potential Growth (2044) and Consideration for Future Expansion
Treatment	ST-2 (Expansion of Collingwood WWTP). The expansion is to be sited such that there will be sufficient land available for future expansion, where future expansion would service built boundary growth, servicing of neighbouring communities, and servicing of currently unserviced areas within Collingwood.
Local Sewers	L-2 (Flow Diversion Modification on Hurontario Street at Second Street and Local Sewer Capacity Increases).
Sanitary Trunk Sewer System	SC-2 (New Black Ash SPS Forcemain and Local Improvements). Projects can be oversized to provide servicing for built boundary growth, servicing of neighbouring community and for servicing of currently unserviced areas within Collingwood.

8.3.5.1 Treatment

The ST-2 alternative was found to be the preferred alternative for providing wastewater treatment capacity. This alternative fully meets future demands at a lower capital cost than the other alternatives evaluated. The preferred alternative includes an expansion to the existing Collingwood WWTP to provide a rated capacity of 36,185m^3/d and a peak flow capacity of 90,463m^3/d. As part of the preferred alternative, it is also recommended that the Town undertake demand management measures to reduce average and peak wastewater flows at the Collingwood WWTP as much as possible. Demand management programs will be considered as am implementation measure in Section 9 and an infiltration and inflow reduction strategy has been developed and included in Section 10. Two areas have been identified as pilot areas and are centred on Hurontario Street and the Minnesota Street area.

8.3.5.2 Local Sewers

To improve capacities within the local sewer system, Alternative L-2 has been identified as the preferred alternative. This alternative includes modifications to the existing flow diversion chamber at Hurontario Street and Second Street to divert additional flow into the Second Street sanitary sewer. An orifice plate or weir can be installed to divert additional flow. A number of local sewer improvements and replacements were also identified on Minnesota Street south of Simcoe, Hurontario Street between Collins and Lockhart, Mountain Road west of High and on Huron Street upstream of the Minnesota SPS. Replacement of the existing siphons at Hickory and Spruce with new small pumping stations has also been included in the preferred alternative. Demand management programs could delay the timing and extent of required upgrades and should be carried forward in implementation.

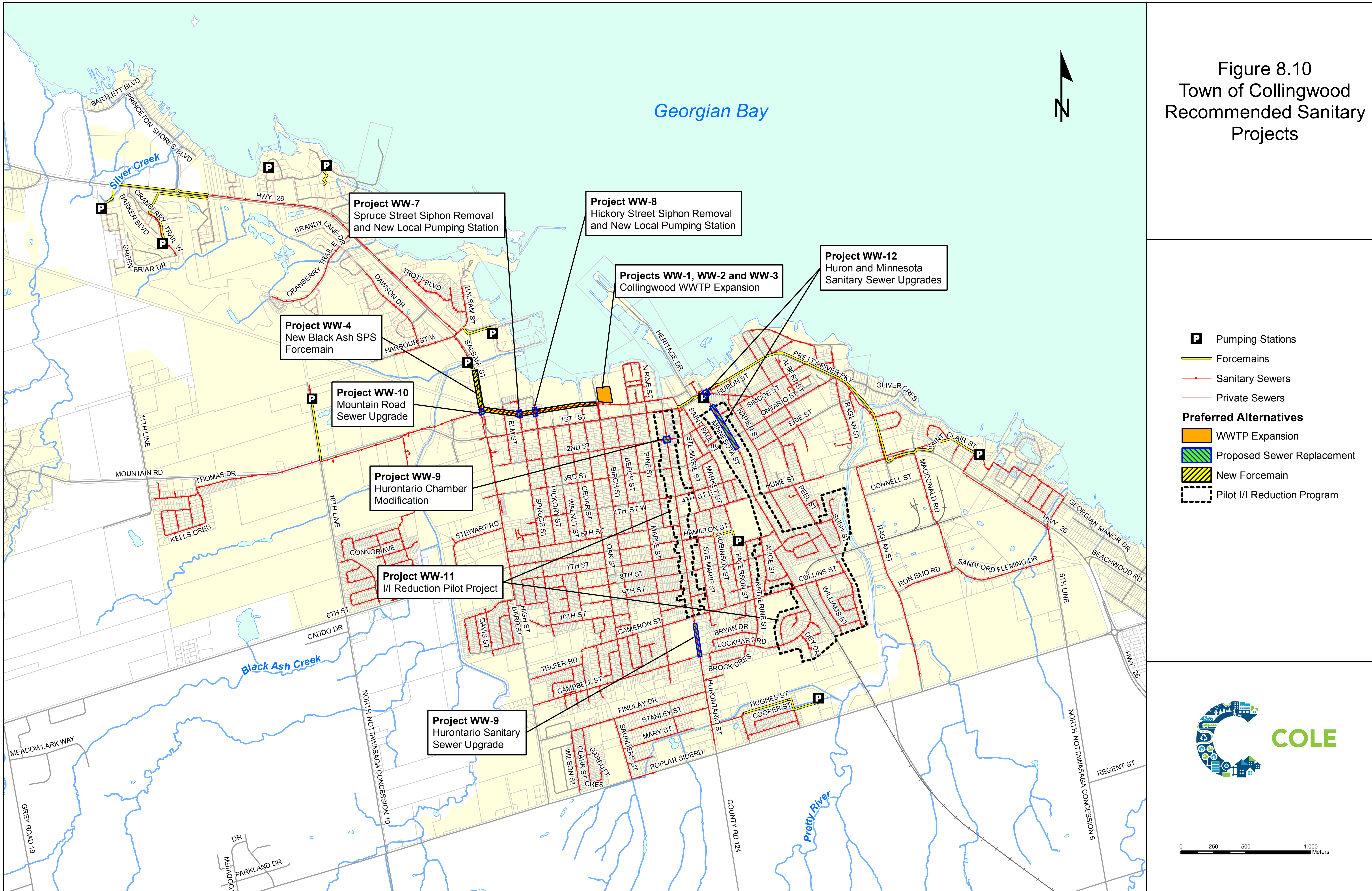


Figure 8.10
Town of Collingwood
Recommended Sanitary Projects

COLE

8.3.5.3 Sanitary Trunk Sewer System

The preferred alternative for the sanitary trunk sewer system is Alternative SC-2. This alternative involves the construction of a new 500mm diameter forcemain from downstream of the Black Ash SPS to the bypass chamber at the Collingwood WWTP (MH-C). This alternative fully meets future demands and will improve system redundancy, if the Town elects to maintain the existing 300mm diameter Black Ash SPS forcemain in service. A new forcemain can also be constructed at a lower cost, relative to the trunk sewer alternative, as open cut methods can be used.

8.3.6 Hydraulic Performance of Preferred Sanitary Alternatives

The hydraulic performance of the system was assessed with the implementation of the preferred alternatives.

Table 8.13 presents a comparison of the peak flow predicted at each pumping station with the station's firm capacity. **Figure 8-11** presents the hydraulic analysis results for the June 17, 2017 assessment event. Review of **Figure 8-11** shows that with the preferred alternatives, the predicted d/D value for all pipes is less than 0.85.

Table 8.13 Pumping Station Performance – Preferred Sanitary Alternatives

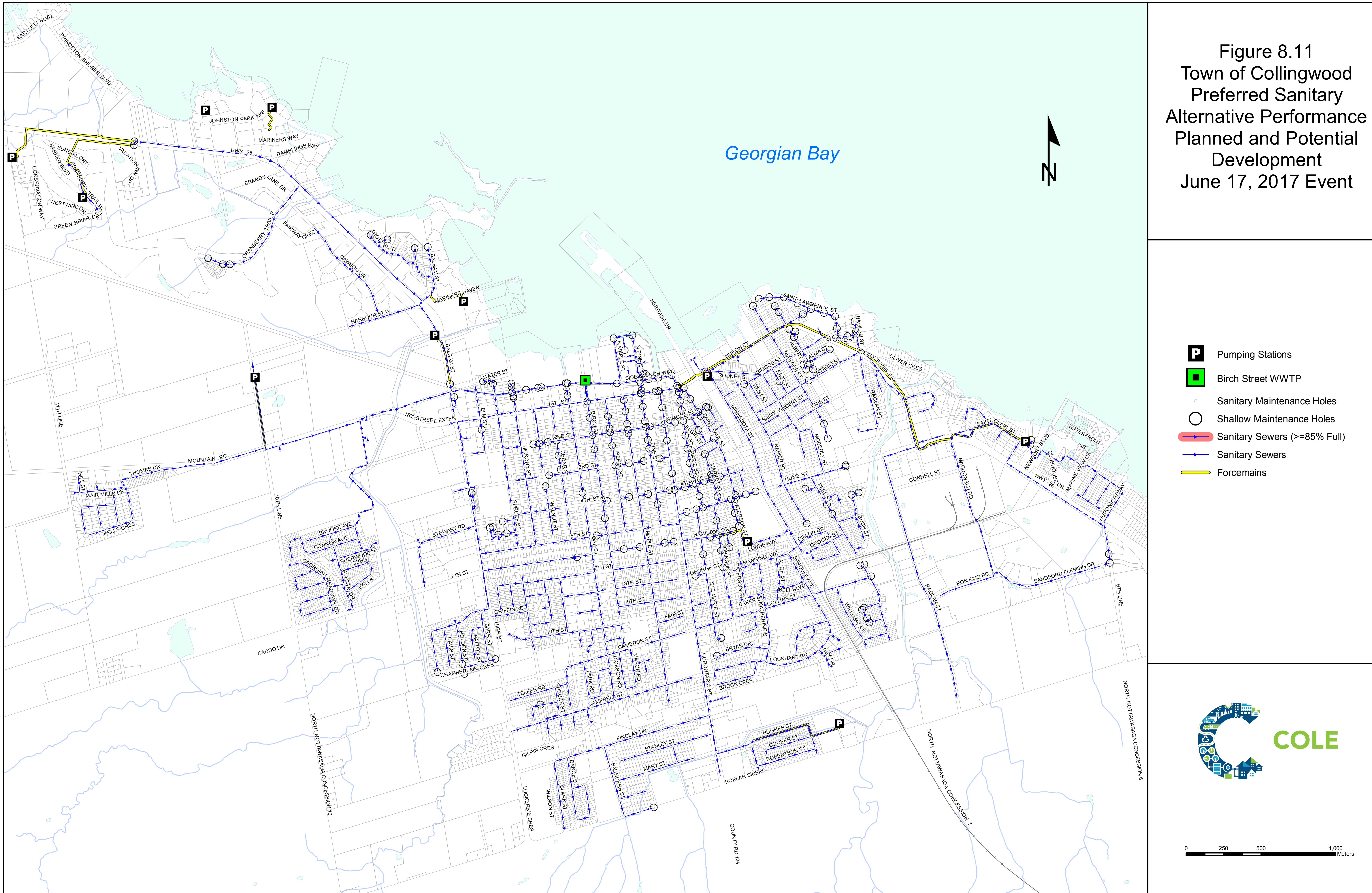
Pumping Station	Peak Modelled Flow Entering the Pumping Station (L/s)	Peak Modelled Wet Well Depth (m)	Firm Capacity (L/s)	Maximum Wet Well Depth (m)
Black Ash SPS	157	1.05	212	3.05
Cranberry Trail SPS	10	1.55	32.8	1.75
Minnesota SPS	288	2.41	210	2.69
Patterson SPS	41	1.55	72	2.13
Pretty River Estates SPS	10	1.25	29	2.33
St. Clair SPS	127	1.01	155	4.95
Silver Glen Preserve	29	-	30	-

1. As part of the development of the Preserve at Georgian Bay, Huntingwood and Silver Glen Developments, the existing pumping station is planned to be replaced with a new station by the Preserve at Georgian Bay Developer. A nominal firm capacity of 30L/s has been assumed for this station. This value will be confirmed through the detailed design of the station.

Table 8.13 shows that all pumping stations have sufficient firm capacity to pump incoming flows for the June 17, 2017 event for the preferred alternatives. All pumping stations have adequate firm capacity to pump incoming flows, except the Minnesota SPS. The Minnesota SPS has adequate station capacity and the peak water level is maintained below the maximum wet well depth. Therefore, performance criteria is met.

At the Collingwood WWTP, a peak flow of 1,194L/s is predicted to reach the plant bypass chamber and the maximum wet well depth predicted is 3.08m for the June 17, 2017 event. Although the predicted peak flow exceeds the future peak flow capacity of Collingwood WWTP of 1,047L/s, there is sufficient storage in the wet well to equalize peak flows. A bypass at the plant is not predicted with implementation of the preferred alternatives for the June 17, 2017 event.

Figure 8.11
Town of Collingwood
Preferred Sanitary
Alternative Performance
Planned and Potential
Development
June 17, 2017 Event



8.4 Built Boundary

The sanitary sewer system with the preferred alternatives was assessed with growth to the built boundary. Growth to the Town's Built Boundary is anticipated to increase the Town's residential population beyond the planned and potential growth population. Development of the built boundary would result in a residential population of 59,894 persons and is anticipated to occur by 2064. **Section 7.5** provided details on the location and distribution of the built boundary growth and also provided information on servicing of Area A, B, F, G1, G2, G3 and G4.

Figure 8-12 presents the performance of the Town's sanitary sewer system with the preferred alternatives in place and built boundary growth. **Table 8.14** presents the performance of the Town's pumping stations with the preferred alternative and built boundary growth.

Table 8.14 Pumping Station Performance – Preferred Sanitary Alternatives

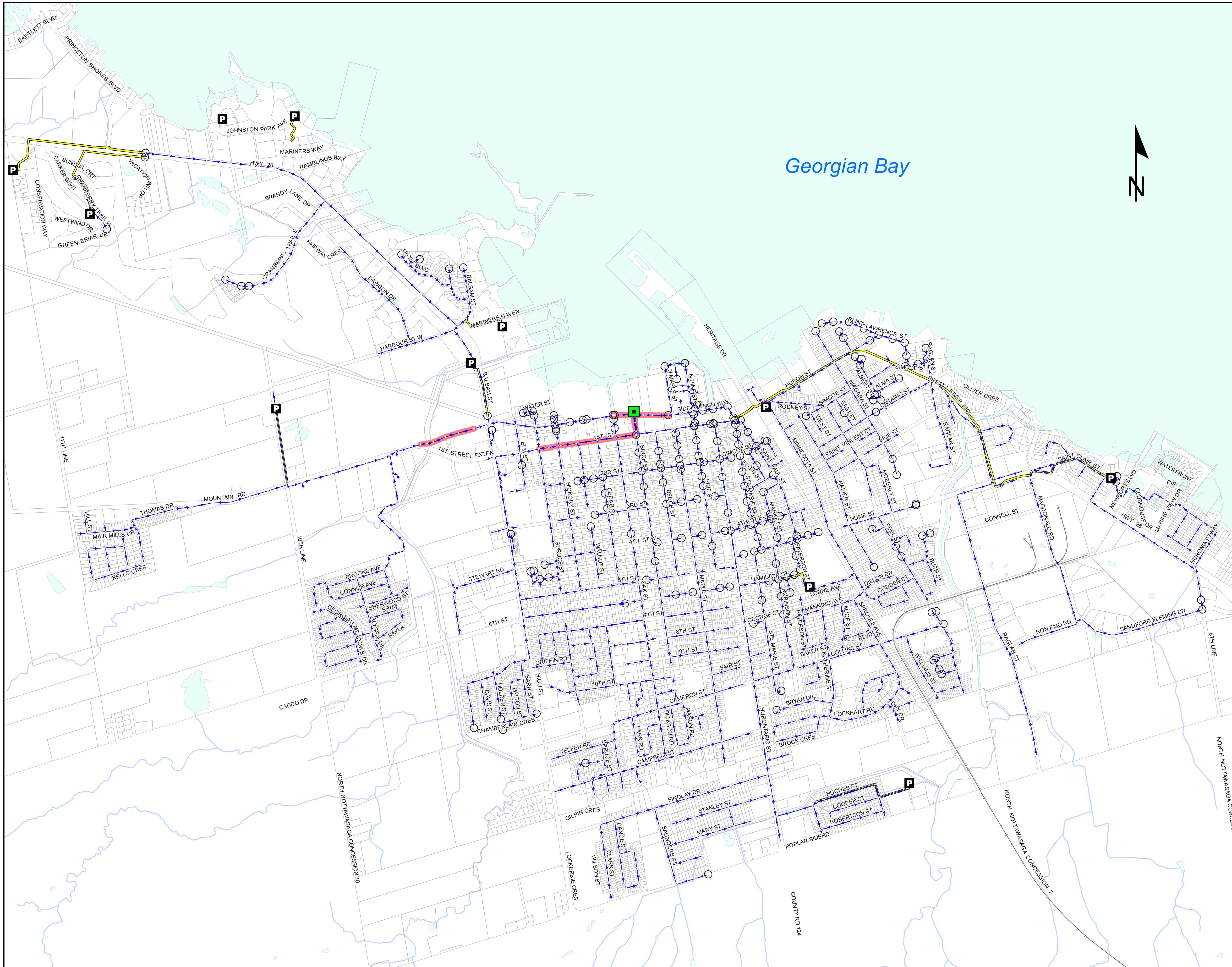
Pumping Station	Peak Modelled Flow Entering the Pumping Station (L/s)	Peak Modelled Wet Well Depth (m)	Firm Capacity (L/s)	Maximum Wet Well Depth (m)
Black Ash SPS	198	1.05	212	3.05
Cranberry Trail SPS	10	1.55	32.8	1.75
Minnesota SPS	287	2.41	210	2.69
Patterson SPS	41	1.55	72	2.13
Pretty River Estates SPS	10	1.25	29	2.33
St. Clair SPS	146	1.02	155	4.95
Silver Glen Preserve	29	-	30	-

1. As part of the development of the Preserve at Georgian Bay, Huntingwood and Silver Glen Developments, the existing pumping station is planned to be replaced with a new station by the Preserve at Georgian Bay Developer. A nominal firm capacity of 30L/s has been assumed for this station. This value will be confirmed through the detailed design of the station.

Figure 8-12 shows that the Mountain Road sanitary sewer will have insufficient capacity to service built boundary growth**.** In addition, surcharge conditions are predicted in a number of trunk sewers including the Harbourview Trail Trunk Sewer, the First Street Sanitary and the Birch Street sanitary sewer. The peak hydraulic grade will be 1.8m below the ground surface. Additional treatment capacity would be needed at the Collingwood WWTP where a minimum rated capacity of 39,091m^3/d will be needed. Other servicing requirements will include:

- An upgrade to the Mountain Road sanitary sewer will be needed to service built boundary growth. The preferred alternative includes a replacement of 96m of existing 450mm and 500mm diameter sanitary sewer with a new 600mm diameter sanitary sewer located immediately west of High Street. To service built boundary growth, an additional 1,183m of existing 375mm, 500mm and 525mm diameter sanitary sewer from Tenth Line to 96m west of High Street will need to be replaced with a new 600mm diameter sanitary sewer.

Figure 8.12
Town of Collingwood
Preferred Sanitary
Alternative Performance
Planned, Potential and
Built Boundary
Development
June 17, 2017 Event



P	Pumping Stations
■	Birch Street WWTP
○	Sanitary Maintenance Holes
◯	Shallow Maintenance Holes
	Sanitary Sewers (>=85% Full)
	Sanitary Sewers
	Forcemains

0 250 500 1,000 Meters

9 Servicing Neighbouring Municipalities

The provision of water and sanitary servicing to neighbouring communities was considered in this Master Plan. In addition, as there are areas within the Town with municipal water servicing and private sanitary servicing, consideration was also given to providing sanitary servicing to these areas. The following sections address both water and sanitary servicing needs in neighbouring communities and areas with private sanitary servicing.

9.1 Water Servicing for Neighbouring Municipalities

The Town of Collingwood currently provides water to neighbouring municipalities. This includes the group of municipalities, Clearview Township (New Lowell), Essa Township (Village of Baxter and Town of Angus), and the Town of New Tecumseth (Alliston) that are serviced by the New Tecumseth Supply, and referred to as New Tecumseth herein. The Town of Blue Mountains (ToBM) is serviced through a connection to the distribution system at the town boundary. The Township of Clearview has also made inquiries about future water supply to Nottawa from the Town of Collingwood, and is therefore addressed below.

9.1.1 New Tecumseth

The Town supplies water to New Tecumseth through a Regional Pipeline which has been operational since May 2000. The Regional Pipeline is supplied by dedicated pumps at the Collingwood WTP to deliver the permitted flow of 9,500m^3/day. There have been informal inquiries about increasing water taking to 10,000 -13,000m^3/day in the near future, and ultimately to 23,500 – 33,500m^3/day. The 2004 RAB WTP Expansion Environmental Study Report (2004 ESR) included demand projections of up to 23,500m^3/d for New Tecumseth by 2030. **Table 9.1** summarizes Town of New Tecumseth requests.

Table 9.1 New Tecumseth Requests

Requirement	Existing	Planned (2032)	Potential (2044)	Built Boundary
Demand	9,500m^3/d	13,000m^3/d	23,500m^3/d	33,500m^3/d
Pumping	110L/s	151L/s	272L/s	388L/s

9.1.1.1 Supply/ Treatment Capacity

The existing WTP is approaching its supply capacity to serve the Town of Collingwood and neighbouring municipalities at current water taking rates. To achieve New Tecumseth's requested supply of 33,500m^3/d, the increase in demand at the WTP would be 24,000m^3/d from the existing 9,500m^3/d. An opinion of probable cost (OPC) for the requested water supply and treatment was developed based on a linear application of the cost to upgrade the WTP from the analysis for the Town of Collingwood. The OPCs are summarized in **Table 9.2.**

Table 9.2 New Tecumseth Supply Costs

	Collingwood	New Tecumseth		
	WTP Upgrade (W-R-4)	Option 1	Option 2	Option 3
Existing Capacity (m^3/d)	31,400	9,500	9,500	9,500
Required Capacity (m^3/d)	47,000	13,000	23,500	33,500
Increase in Capacity (m^3/d)	15,860	3,500	14,000	24,000

Table 9.2 New Tecumseth Supply Costs

	Collingwood	New Tecumseth		
	WTP Upgrade (W-R-4)	Option 1	Option 2	Option 3
Total OPC	$40M	$8.9M	$35.4M	$60.6M
Unit OPC (per m^3/d increase)	$2,522	Total OPC above based on $2,522 x increase in capacity, rounded		

9.1.1.2 Storage Capacity

New Tecumseth does not require storage in the Town's system since they have local storage.

9.1.1.3 Pumping Capacity

The Regional supply system currently has three vertical turbine pumps with variable speed drives each rated at 136.1L/s at 55m TDH to provide an existing firm capacity of 272L/s. The required firm capacity to supply the ultimate demand of 33,500m^3/d is 388L/s. This increase of 116L/s in pumping capacity could be accommodated by increasing the pumping capacity at the WTP, and the costs are in provided in **Table** 9.3**.**

Table 9.3 New Tecumseth Pumping Costs

	Collingwood	New Tecumseth		
	Stewart Rd PS (W-P2-1) used to determine Unit OPC	Planned (2032)	Potential (2044)	Built Boundary
Water Taking (m^3/d)	-	13,000	23,500	33,500
Current Capacity (m^3/d)	-	272	272	272
Design Capacity (m^3/d)	45	150	272	388
Increase in Capacity (m^3/d)	45	-	-	116
Total OPC	$0.5M	$-	$-	$1.3M
Unit OPC (perL/s increase)	$11,111	Total OPC above based on $11,111 x increase in capacity, rounded		

9.1.1.4 Watermain Capacity

The Regional Pipeline is constructed of 600mm concrete pressure pipe to the Town's boundary. It has a total length of 58km. The initial capacity limit of the Regional Pipeline was designed to supply 23,500m^3/d; however, the addition of an inline booster station and upgrade of the existing transmission pumps was expected to further increase capacity. The capacity of a 600mm pipe is approximately 329L/s while maintaining less than 2m head loss per km. A constant flow rate of 388L/s would result in slightly higher headloss but could be overcome by additional booster stations beyond the Town's boundary.

Within the Town's limits, the Regional Pipeline is also used to supply water from the WTP to Davey BPS. Currently, the Davey BPS uses approximately 850m^3/d (10L/s) on MDD, but this amount is expected to increase with new developments in Zone 2. The Town's agreement with New Tecumseth entitles Collingwood to reserve watermain capacity for their domestic demands. Based on planning data, the Zone 2 demands from Davey BPS are expected to reach approximately 130L/s by the Built Boundary scenario. This value was estimated by splitting the total demands of Zone 2 and Zone 3 between Davey BPS and Stewart Road as shown in **Table 9.4**.

Developing a watermain and pumping plan to provide this capacity to New Tecumseth is not within the scope of this assignment. The Town's ultimate Zone 2 capacity requirements at the Davey BPS and Reservoir of 130L/s should be reserved in the existing New Tecumseth watermain. Demand increases from Clearview Township may also be considered.

Table 9.4 Collingwood Zone 2 and 3 Ultimate MDD

	Existing	Built Boundary
Zone 2	23L/s	227.7L/s
Zone 3	XXXX	32.1L/s
Total	23L/s	259.8L/s
Total Split (50%)		130L/s

9.1.2 Clearview Township

The Township of Clearview (Clearview) is located to the south of Collingwood and includes Stayner, Nottawa, New Lowell and Brentwood. Nottawa is the closest of these municipalities and is located approximately 2.6km from Poplar Side Road. Clearview was considered as a potential customer for treated water from Collingwood in the 2004 ESR. To date, they have not been supplied by the Town, but have requested a potential future supply of 4,854m^3/d to service the Nottawa community. Pumping capacity of 56L/s would be required. **Table 9.5** presents Clearview Township requests.

Table 9.5 Clearview Township Requests

Requirement	Existing	Planned (2032)	Potential (2044)	Built Boundary
Demand	-	-	-	4,854 m^3/d
Pumping	-	-	-	56L/s

9.1.2.1 Supply/ Treatment

The total MDD of 4,854m^3/d would need to be added to the existing supply and treatment at the WTP. Based on preliminary discussions, it is expected that Clearview would be supplied via the Regional Pipeline to the Davey BPS and Reservoir, then pumped from Davey BPS to a connection point along Poplar Sideroad. The Opinion of Probable Cost (OPC) for this increase in demand at the WTP is provided in **Table 9.6.**

Table 9.6 Clearview Township Supply Costs

	Collingwood	Clearview
	WTP Upgrade (W-R-4)	
Existing Capacity (m^3/d)	31,400	-
Required Capacity (m^3/d)	47,000	4,854
Increase in Capacity (m^3/d)	15,860	4,854
Total OPC	$40M	$12.3M
Unit OPC (per m^3/d increase)	$2,522	Total OPC above based on $2,522 x increase in capacity, rounded

9.1.2.2 Storage Capacity

Storage is not expected to be required for servicing Clearview. Storage to provide peak flows and fire or emergency flows should be provided locally by Clearview.

9.1.2.3 Pumping Capacity

Supplying Clearview would require pumping at the WTP and the Davey BPS. The total pumping requirement is 56L/s which could be supplied by additional pumping capacity at both the WTP and Davey BPS or could be combined with the required additional pumping capacity for New Tecumseth. The approximate cost of adding 56L/s is provided in **Table 9.7**. It is expected that dedicated pumps would be required at Davey BPS to supply Clearview. Lower pressure pumps should be utilized to fill a local Clearview reservoir.

Table 9.7 Clearview Township Pumping Costs

	Collingwood	Clearview
	Stewart Rd PS (WP2-1)	
Water Request (m^3/d)		4,854
Existing Capacity (L/s)		-
Required Capacity (L/s)	45	56
Increase in Capacity (L/s)	45	56 x 2 (capacity required at both WTP and Davey BPS)
Total OPC	$0.5M	$1.3M
Unit OPC (perL/s increase)	$11,111	Total OPC above based on $11,111 x increase in capacity, rounded

9.1.2.4 Watermain Capacity

The additional demand would impact watermain capacity of the Regional Pipeline to Davey BPS, and from Davey BPS to Nottawa. A new dedicated watermain should be built from the Bob Davey BPS to Clearview. It is expected that a 300mm watermain would be required to supply an MDD of 56L/s. It is recommended that the Clearview watermain capacity be kept separate from the existing water distribution system to maintain the capacity requirements in the Town. Cost sharing should be considered for upgrades to the Regional Pipeline if required to supply the future demands of Collingwood, New Tecumseth and Clearview.

9.1.3 Town of Blue Mountains (ToBM)

The Town of Blue Mountains (ToBM) is currently supplied at a connection point at Mountain Road and Osler Bluff Road. The existing agreement is to supply 1,250m^3/d but the ToBM has discussed increasing this rate up to 16,400m^3/d. Water taking amounts up to 8,000m^3/d beyond 2008 was considered in the 2004 ESR. **Table 9.8** provides information on ToBM requests.

Table 9.8 ToBM Requests

Requirement	Existing	Planned (2032)	Potential (2044)	Built Boundary
Demand	1,250m^3/d	4,000m^3/d	8,000m^3/d	16,400m^3/d
Pumping	14L/s	46L/s	93L/s	190L/s

9.1.3.1 Supply and Treatment Capacity

ToBM's ultimate request would involve a supply and treatment increase of 15,150m^3/d at the WTP. The cost for this additional capacity is provided in **Table 9.9**.

Table 9.9 ToBM Supply Costs

	Collingwood	ToBM		
	WTP Upgrade (W-R-4)	Option 1	Option 2	Option 3
Existing Capacity (m^3/d)	31,140	1,250	1,250	1,250
Required Capacity (m^3/d)	47,000	4,000	8,000	16,400
Increase in Capacity (m^3/d)	15,860	2,750	6,750	15,150
Total OPC	$40M	$7M	$17.1M	$38.3M
Unit OPC (per m^3/d increase)	$2,522	Total OPC above based on $2,522 x increase in capacity, rounded		

9.1.3.2 Storage Capacity

Storage to supply the ToBM's MDD is not calculated in this study as ToBM is expected to supply their own storage.

9.1.3.3 Pumping Capacity

The ultimate pumping requirement to supply 16,400 m3/d would be 190L/s, resulting in an increase of 176L/s from the existing 14L/s. Additional pumping capacity would be required at the WTP and at Stewart Road if the ToBM expects to draw from the existing connection in Zone 2. The cost to provide the additional pumping capacity at both locations is listed in **Table 9.10**. Discussions with ToBM should include the addition of redundancy and consider a secondary supply point and backup plan if required.

Table 9.10 ToBM Pumping Costs

	Collingwood	ToBM		
	WTP Upgrade (W-R-4)	Option 1	Option 2	Option 3
Water Request (m^3/d)		4,000	8,000	16,400
Existing Capacity (L/s)		14	14	14
Recommended Capacity (L/s)	45	46	93	190
Increase in Capacity (L/s)	45	32 x 2	79 x 2	176 x 2
		(capacity required at both WTP and Stewart Rd BPS)		
Total OPC	$0.5M	$0.72M	$1.8M	$3.9M
Unit OPC (per L/s increase)	$11,111	Total OPC above based on $11,111 x increase in capacity, rounded		

9.1.3.4 Watermain Capacity

Additional watermain capacity from the WTP to the ToBM's connection point at Mountain Road and Osler Bluff Road (ToBM Connection) would be required to supply the ultimate demand requested by the ToBM. Costs were assessed for two options. Option A is to build a dedicated watermain from the WTP to the ToBM Connection. Option B would be to increase the capacity of the proposed 400mm watermain from the WTP and along Sixth Street (W-L1-1C) to Stewart Road Reservoir and build a dedicated watermain from Stewart Road BPS to the ToBM Connection. (Option B). **Table 9.11** and **Table 9.12** provides costs for the two options for the requested water values.

In the second alternative, a new 400mm watermain from the Stewart Road BPS to the ToBM Connection would also be required. The new 400mm watermain along 10th line from Stewart Road to Mountain Road has already been designed and is required to supply Collingwood demands. **Table 9.12** provides costs for both watermains.

Table 9.11 ToBM Costs for New Dedicated Watermain from WTP to ToBM Connection

	Planned (2032)	Potential (2044)	Built Boundary
Water Request (m^3/d)	4,000	8,000	16,400
Required Capacity (L/s)	47	93	190
Recommended Diameter (mm)	300	400	500
Length (m)	10,000	10,000	10,000
Unit OPC (per m)	$1,800	$2,000	$2,400
Total OPC	$18M	$20.0M	$24.0M

Table 9.12 ToBM Costs for Watermain Capacity Increase to W-L1-1C

	Planned (2032)	Potential (2044)	Built Boundary
Water Request (m^3/d)	4,000	8,000	16,400
Combined Watermain from WTP To Stewart Rd			
Total Required Capacity (L/s)	177 (130 + 47)	223 (130 + 93)	320 (130 + 190)
	The required capacity of W-L1-1C alone is 130L/s		
Required Diameter (mm)	500	600	600
Length (m)	5,000	5,000	5,000

Table 9.12 ToBM Costs for Watermain Capacity Increase to W-L1-1C

	Planned (2032)	Potential (2044)	Built Boundary
Unit OPC/m	$2,400	$2,600	$2,600
Unit OPC of upgrade /m	$400	$600	$600
	The OPC for W-L1-1C original diameter of 400mm is $2,000/m		
Total OPC of upgrade	$2.0M	$3.0M	$3.0M
Dedicated Watermain from Stewart Rd to ToBM Connection			
Total Required Capacity (L/s)	47	93	190
Recommended Diameter (mm)	300	400	500
Length (m)	5,000	5,000	5,000
Unit OPC/m	$1,800	$2,000	$2,400
Total OPC	$9.0M	$10.0M	$12.0M
Overall Total OPC	$11.0M	$13.0M	$15.0M

9.2 Sanitary Servicing for Neighbouring Municipalities

For the sanitary system, only servicing of the community of Nottawa in Clearview Township would contribute additional flow to the sanitary system. The impact of planned, potential and built boundary growth with servicing of Nottawa on the preferred sanitary alternatives system was assessed. Based on information provided by the Clearview Township, the anticipated average sanitary flow from Nottawa would be 2,800m^3/d and the peak sanitary flow would be 8,820m^3/d.

Two alternatives were considered for servicing of Nottawa. Alternative Nottawa1 would allow discharge of flows from Nottawa into new sewers that will be constructed as part of the servicing of the Eden Oak Industrial lands. These new sewers would discharge into the existing 300mm diameter sewer on Raglan Street. Alternative Nottawa2 would allow flows to be discharged into the existing Sixth Line sanitary sewer. The Sixth Line Sewer discharges into a trunk sewer which conveys flows to the St. Clair SPS. **Table 9.13** presents the projected future flows at the Collingwood WWTP with servicing of Nottawa.

Table 9.13 Projected Flows at the Collingwood WWTP with Planned, Potential, Built Boundary and Servicing of Neighbouring Communities

	Anticipated Residential Population Growth	Anticipated ICI Area Growth (ha)	Recommended Per Capita or Area Flow Generation, including I/I	Projected Flow (m3/d)	Current Rated Capacity (m3/d)
Existing Flow	-	-		16,300	24,548
Planned Development (2032)	12,366	48.0	350 Lpcd (residential) and 28m^3/ha/d (non-residential)	21,973	24,548
Potential Development (2044)	9,528	130.0	350 Lpcd (residential) and 28m^3/ha/d (non-residential)	28,948	24,548
Built Boundary (2064)	20,944	161	350Lpcd (residential and employment)	38,301	24,548

Table 9.13 Projected Flows at the Collingwood WWTP with Planned, Potential, Built Boundary and Servicing of Neighbouring Communities

	Anticipated Residential Population Growth	Anticipated ICI Area Growth (ha)	Recommended Per Capita or Area Flow Generation, including I/I	Projected Flow (m3/d)	Current Rated Capacity (m3/d)
Nottawa Servicing				41,891	24,548

Servicing of Nottawa, would increase the required rated capacity at the Collingwood WWTP by 2,800m^3/d. Servicing of Planned, Potential, Built Boundary and Nottawa would require the Collingwood WWTP to be expanded to a rated capacity of 41,891 m^3/d.

The performance of the preferred alternatives sanitary system, which encompasses all sanitary sewers, pumping stations and forcemains was assessed for planned, potential and built boundary growth and Nottawa servicing using the calibrated hydraulic model. A Nottawa servicing growth model scenario was created by adding new population and serviced areas to the model to represent new growth and by adding the flow contributed by Nottawa. The assessment built on the results already generated for the planned, potential and built boundary development scenarios. The following describes the servicing options considered for Nottawa:

- Alternative Nottawa 1 – Flow from Nottawa would be discharged into the Town's 300mm diameter sanitary sewer on Raglan Street south of Poplar Road. This location was identified as the preferred connection point in the Nottawa Municipal Class Environmental Assessment, completed in 2009. The Raglan Street sewer extends from south of Poplar Sideroad to Hume Street. For this alternative, an upgrade to the Raglan Street sanitary sewer is required to provide sufficient capacity. At a minimum, a total 942m of existing 300mm and 773m of existing 375mm diameter sanitary sewer on Raglan Street would need to be replaced with a new 450mm diameter sanitary sewer to provide the required capacity. In addition to the above, Figure 9.1 shows that the predicted peak depth ratio will be greater than 0.85 for this alternative. The estimated cost to upgrade this sewer is $2.7M.
- Alternative Nottawa 2 – Flow from Nottawa would be discharged into the existing 450mm diameter sanitary sewer on Sixth Line at Sanford Fleming Road. Flows would be conveyed by the 450mm diameter sanitary sewer on Huronia Parkway and the existing 675mm diameter sanitary trunk sewer that extends from Huronia Parkway to the St. Clair SPS. Hydraulic analysis identified that this existing sewer has sufficient capacity to convey the additional peak flow from Nottawa of 102L/s. The flow entering the St. Clair SPS is predicted to increase to 246L/s which exceeds the firm capacity of the station. A review of the St. Clair SPS drawings identified that the existing pumping station was constructed with space available for a third pump. To service growth from Nottawa through this Alternative will require the installation of a third pump with a capacity of 155L/s. The upgraded St. Clair SPS would have sufficient firm capacity to pump peak flows. The estimated cost for installation of a third pump is $1M.

Both of the above alternatives include additional capacity at the Collingwood WWTP to provide capacity for Nottawa. An additional rated capacity of 2,800m^3/d would be required. **Figure 9-1** presents the location of works for Alternative Nottawa1. **Figure 9-2** presents the location of works for Alternative Nottawa2. Both of these figures also show system performance with the alternative in place.

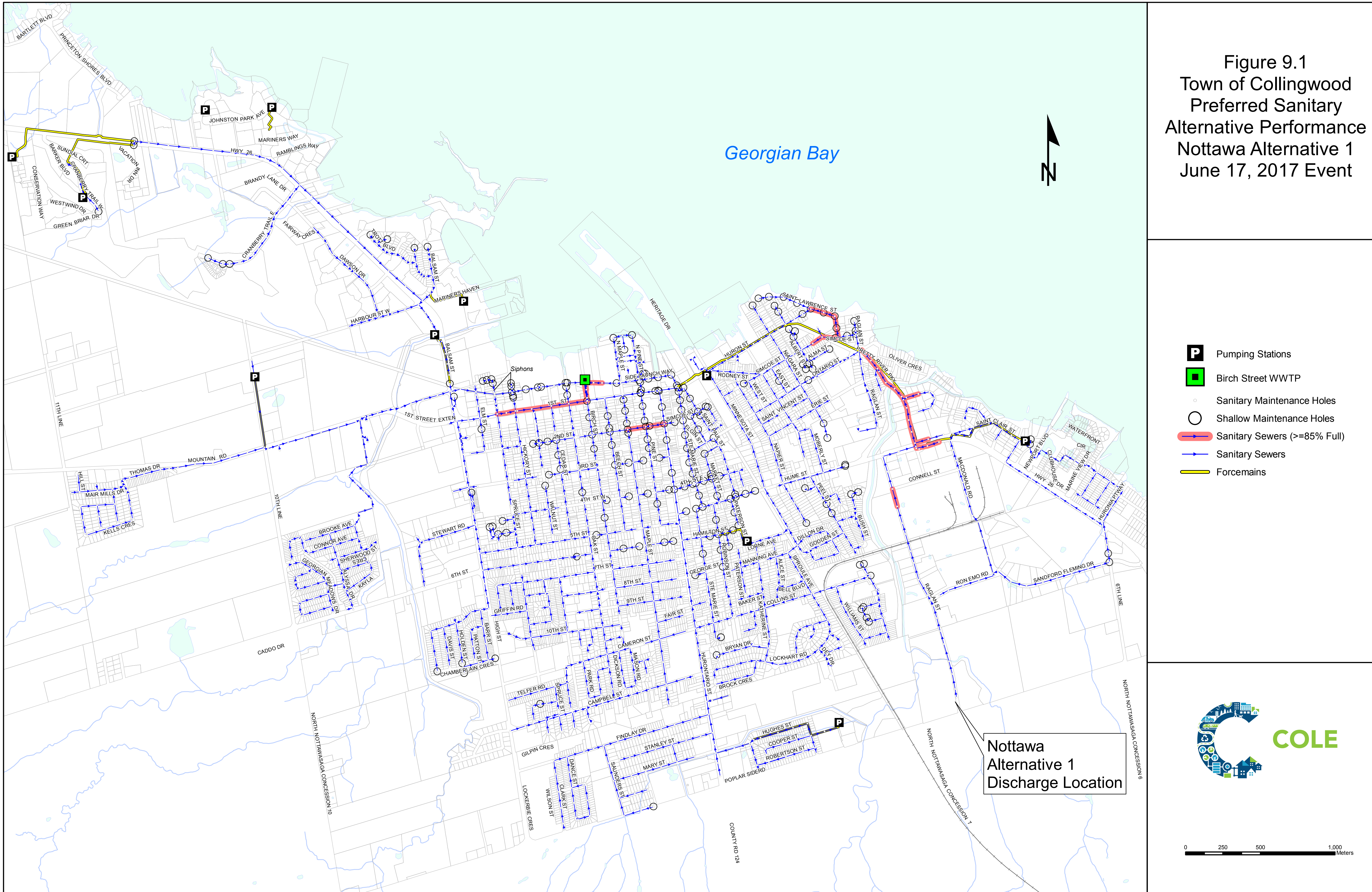


Figure 9.1
Town of Collingwood
Preferred Sanitary
Alternative Performance
Nottawa Alternative 1
June 17, 2017 Event

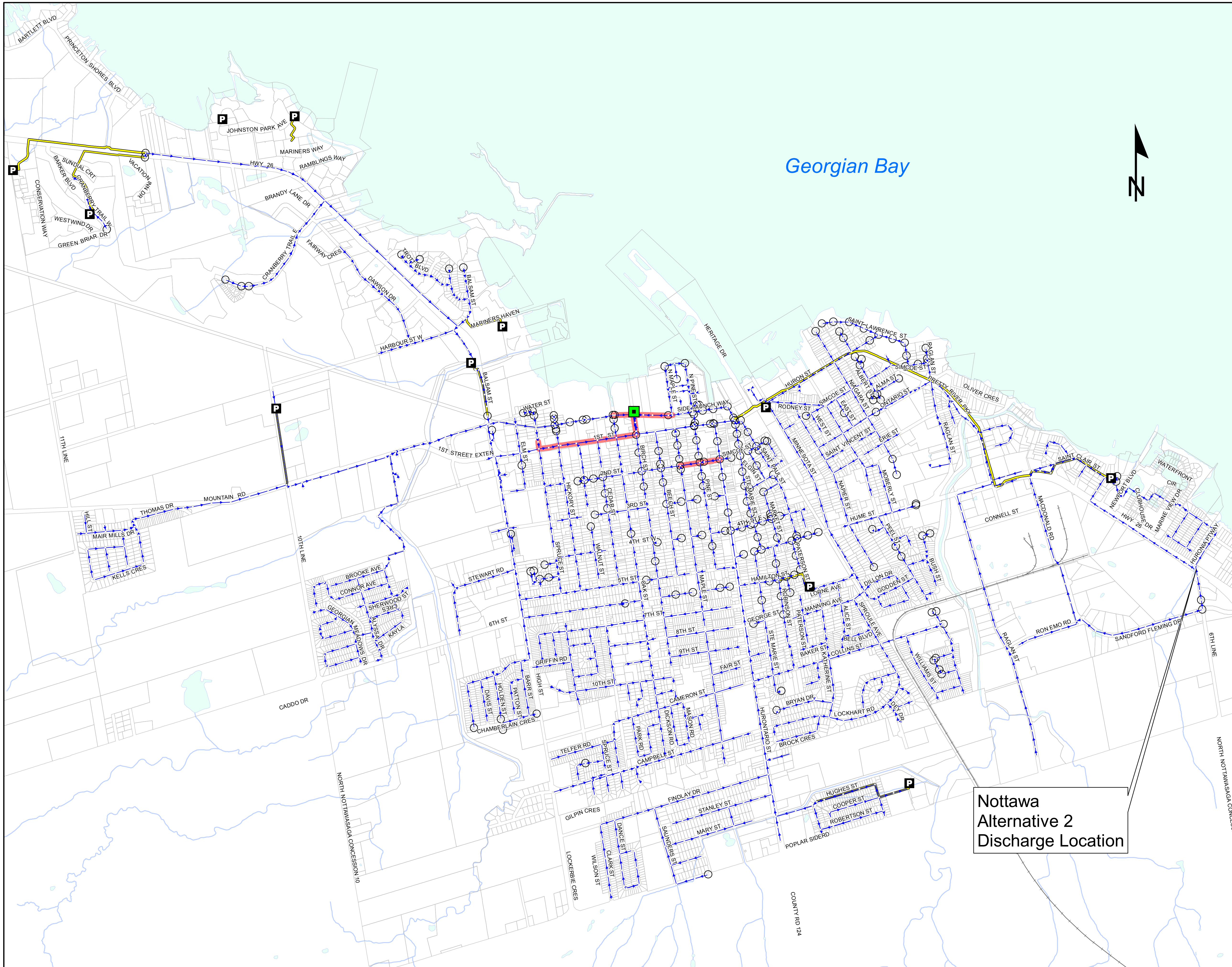


Figure 9.2
Town of Collingwood
Preferred Sanitary
Alternative Performance
Nottawa Alternative 2
June 17, 2017 Event

Pumping Stations
Birch Street WWTP
Sanitary Maintenance Holes
Shallow Maintenance Holes
Sanitary Sewers (>=85% Full)
Sanitary Sewers
Forcemains

COLE

0 250 500 1,000 Meters

Table 9.14 presents the evaluation of alternatives for servicing of Nottawa.

Table 9.14 Nottawa Servicing Alternatives Evaluation

Evaluation Criteria/ Alternative	Alternative Nottawa 1 – Servicing of Nottawa Through Raglan Street Sanitary Sewer	Alternative Nottawa 2 – Servicing of Nottawa Through Sixth Line Sanitary Sewer
Natural Environment	Alternative requires upgrade of the Raglan Street sanitary sewer. Upgraded sewer to be located within existing Raglan Street road allowance. Construction will require crossing of one watercourse north of Poplar Sideroad. Raglan Street generally has a rural road cross section with ditches. Any Impacts to existing trees can be mitigated as the tree canopy does not extend over the roadway.	Alternative will require the installation of a third pump within the existing dry well at the St. Clair SPS. The pumping station was designed for the installation of a future pump. Therefore, alternative will consist of modifications to the existing pumping station to allow for pump installation. No impacts to the natural environmental are anticipated.
Natural Environment Score	2	1
Social Environment/ Heritage impacts	Alternative will have noise, dust and traffic impacts on local residents and businesses. Raglan Street is also the access road for St. Mary’s Cemetery and several industrial/ commercial facilities. Construction impacts can be mitigated through good construction practices.	Alternative will have minimal impacts on local residents and businesses. Construction activity will be limited to installation of third pump at St. Clair SPS.
Social/ Heritage Environment Score	2	1
Technical Environmental Impacts	Alternative will require upgrade to existing sewer. Alternative will not increase system complexity or operating and maintenance requirements	Alternative will increase the operating complexity of the St. Clair SPS, increase energy use and pump monitoring at the St. Clair SPS
Technical Environment Score	1	2
Financial Environment	Capital cost for sewer replacement is estimated to be $2.7M. No increase over existing operating and maintenance costs is anticipated.	Capital cost for pump installation including changes required to facilitate operation is estimated to be $1M. Increased operating and maintenance costs are anticipated with additional energy use from third pump.

Table 9.14 Nottawa Servicing Alternatives Evaluation

Evaluation Criteria/ Alternative	Alternative Nottawa 1 – Servicing of Nottawa Through Raglan Street Sanitary Sewer	Alternative Nottawa 2 – Servicing of Nottawa Through Sixth Line Sanitary Sewer
Financial Environment Score	2	2
Overall Score	7	6

Based on the above, Alternative Nottawa2 is the preferred alternative as it has a lower cost to implement with minimal construction impacts. In summary, to service Nottawa, the following works would be required:

- A third pump with a capacity of 155L/s will need to be installed at the St. Clair SPS. The design of the pumping station did allow for the installation of a third pump.
- The Collingwood WWTP would need to be expanded to provide an additional rated capacity of 2,800m^3/d. If Nottawa is to be serviced before 2044, the expansion needed to provide capacity for planned and potential development identified in the preferred alternative could be increased to provide the capacity. As a result, the Town would consider a plant expansion to a rated capacity of 38,195m^3/d. If Nottawa is to be serviced after 2044, expansion of Collingwood WWTP could proceed to a rated capacity of 36,185m^3/d. If Nottawa is be to serviced after 2044, the detailed design of an expansion to accommodate planned and potential growth should ensure that sufficient land area is available at the site to accommodate a future expansion to provide servicing for built boundary growth and Nottawa servicing. The long term site capacity of the Collingwood WWTP should be a minimum of 41,891 m^3/d.

9.3 Extension of Sanitary Servicing to Areas With Private Servicing

A total of five areas were identified where residents have municipal water servicing but private sanitary servicing. Each of these areas was considered to determine requirements to provide future sanitary servicing.

Figure 9-3 presents the location of these areas. The following sections provide further information.

9.3.1 Oliver Crescent

The Oliver Crescent area consists of 46 residential properties on 10.1ha located on Oliver Crescent. Oliver Crescent is located adjacent to Georgian Bay and extends from the intersection of Ontario Street and Raglan Street to a dead end. The properties on Oliver Crescent are currently serviced by private septic systems. Based on a PPU for single residential properties of 2.9, the population of Oliver Crescent is estimated to be 133 persons.

To service this area, two options were considered. Option 1 would consist of a combination of gravity sewer, a pumping station and new forcemain. A new 200mm diameter sanitary sewer would be constructed on Oliver Crescent eastward. A new pumping station would be needed at the east end of Oliver Crescent and a new 100mm diameter forcemain would extend westward on Oliver Crescent to the existing 450mm diameter sanitary sewer on Pretty River Parkway. The project would include construction of new service connections to the property for each of the 46 properties. Individual property owners would then be responsible for the construction of service connections on private property. The new sanitary sewer would discharge into the existing 450mm diameter sanitary sewer located immediately north of Pretty River Parkway at Oliver Crescent.

Option 2 would involve the construction of an alternative sanitary collection such as a Septic Tank Effluent Gravity Sewer (STEG), Septic Tank Effluent Pump or Vacuum Sewer system.

Figure 9-4 presents a schematic of the elements of the STEG, STEP and vacuum sewer systems as well as the elements of a conventional gravity sewer system.

A new sanitary system for this area would discharge into the existing 450mm diameter sanitary sewer on Pretty River Parkway, would be pumped at the Minnesota SPS and would receive treatment at the Collingwood WWTP. Based on a PPU for single family residential of 2.9, and unit flow rates assigned for new growth, servicing of Oliver Crescent with a conventional sewer system would result in an average day flow of 47m^3/d and a peak flow of 4.0L/s.

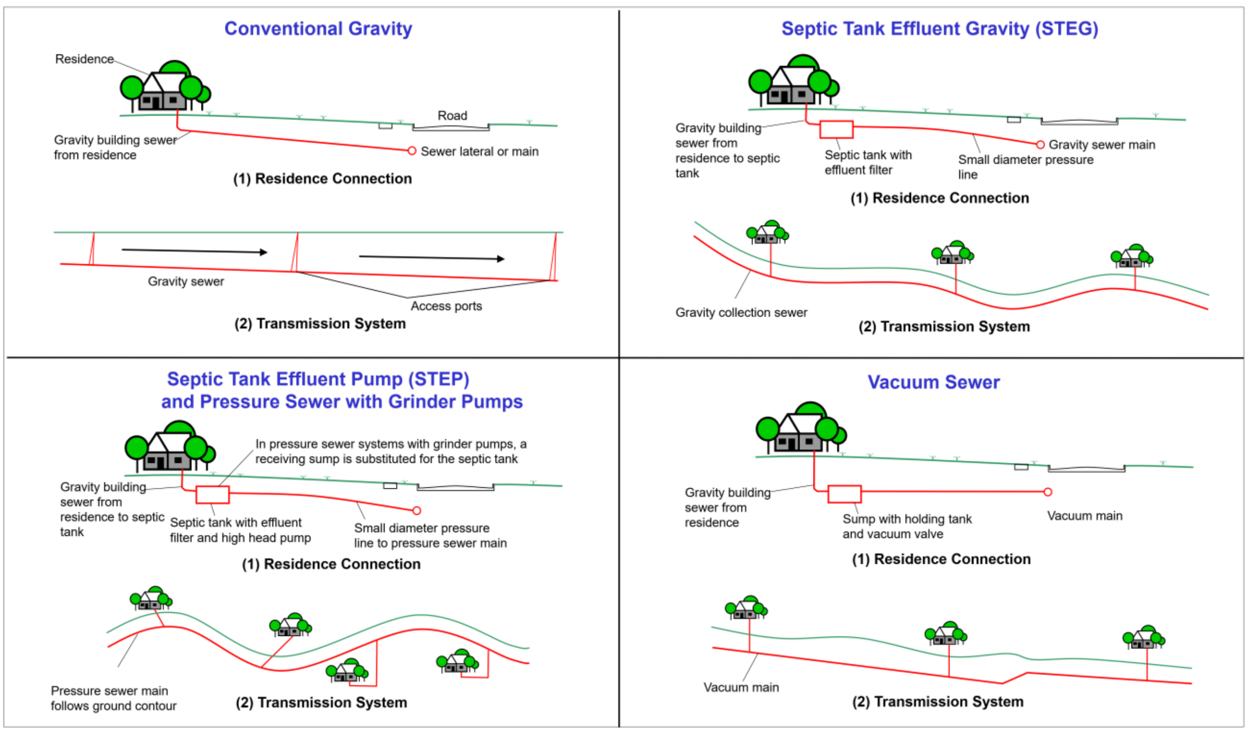


Figure 9-3 Elements of Alternative Sanitary Sewer Collection Systems (STEG, STEP, Vacuum Systems)

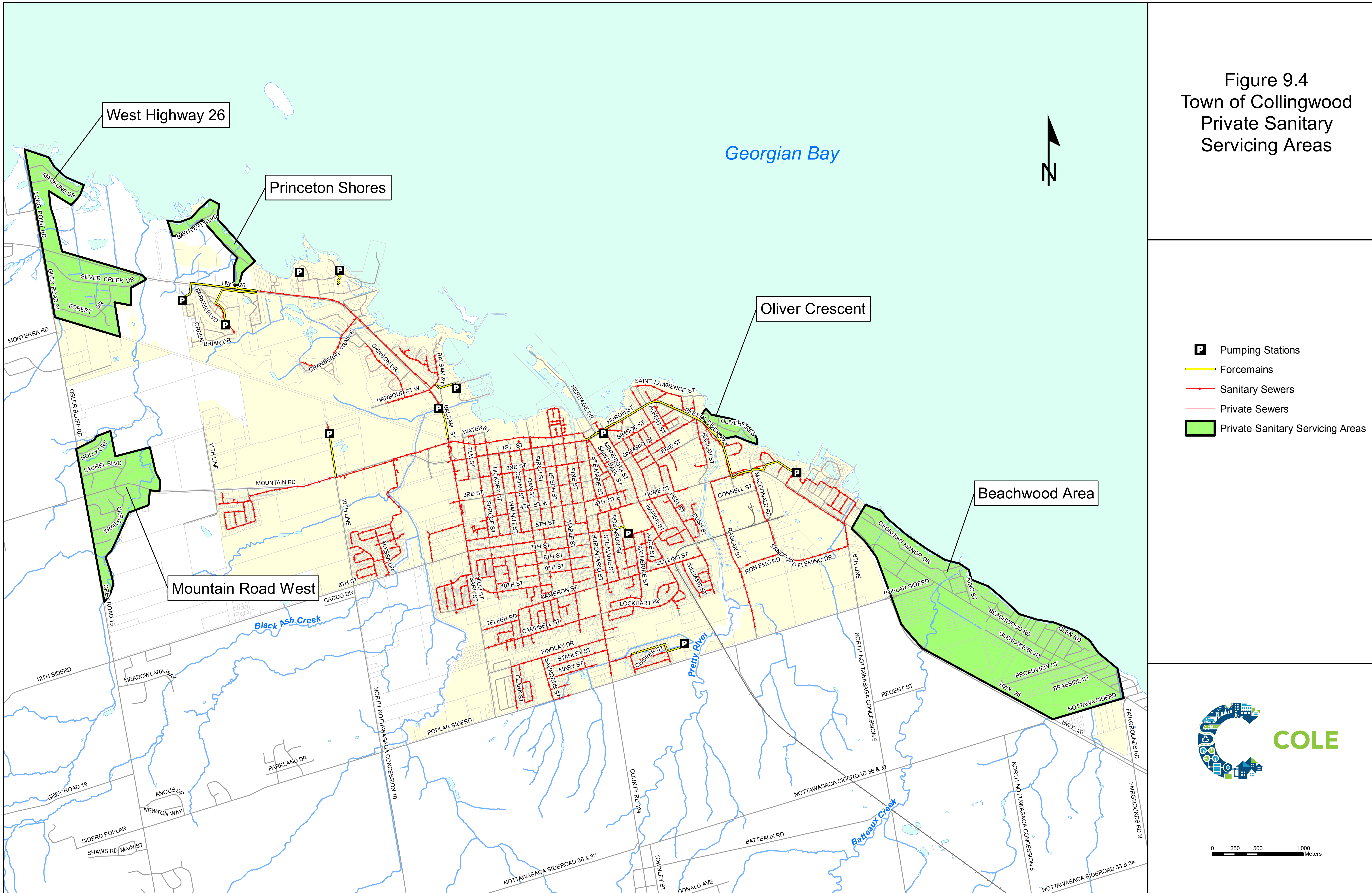


Figure 9.4
Town of Collingwood
Private Sanitary
Servicing Areas

Under Option 2, an alternative sanitary system would be constructed. There are a number of options and these are discussed below. Septic Tank Effluent Gravity (STEG) systems are also referred to as small diameter variable grade gravity sewers. Raw sewage flows from the house into a septic tank (or primary treatment tank), where the liquid and solid portions of the wastewater are separated. The effluent from the septic system flows is discharged by gravity to a small diameter collection system. As the wastewater has been settled prior to transport, the wastewater is not as strong as typical wastewater. Solids in the primary treatment tanks must be removed for treatment every three to five years, similar to a conventional Class 4 sewage system. This must be done to avoid blockages in the small diameter collection system.

With a Septic Tank Effluent Pump (STEP) system, raw sewage is conveyed from the house to the septic tank (or primary treatment tank). The solids and liquid portions of the wastewater are separated in the primary treatment tank. A pump is used to pump effluent from the primary treatment tank into a pressurized collection system. Because the lines are under pressure, they can be used in situations with larger grade variations than a STEG system. As the wastewater has been settled prior to transport, the wastewater is not as strong as typical wastewater. Solids in the primary treatment tank must be removed for treatment every three to five years, as with a conventional septic system to ensure blockages do not occur in the small diameter collection system pipes. A STEP system can also be implemented with grinder pumps. Household sewage is directed to the grinder pump which grinds solids and discharges into a low pressure sewer system. Higher solids and oil and grease concentrations are typically encountered. Since the line is pressurized, this system is applicable to variable grades. Both the solid and liquid portion of the wastewater is conveyed in the small diameter pressurized pipes to a treatment facility. Because the collection system is pressurized, inflow and infiltration volumes are minimized. The wastewater will also have higher solids concentrations. This type of system has been implemented in Oxford County to provide servicing to small communities. Oxford County developed a Grinder Pump Policy that identifies that individual property owners are responsible for the installation of grinder pumps and that the County will be responsible for routine maintenance, repair and replacement of grinder pumps.

Vacuum sewers use the suction of a vacuum, created by a central vacuum source and maintained in the small-diameter pipes, to draw and convey wastewater through the system to the treatment plant. A central vacuum pump station maintains a 380 to 500-mm (15 to 20 in) vacuum in the small-diameter collection mains to convey the wastewater. Wastewater flow is created as a result of the differential pressure between the atmospheric air pressure in the sump and the vacuum in the sewer. Both the solid and liquid portion of the wastewater are conveyed via the small diameter pressurized pipes for treatment. Because the collection system is pressurized, inflow and infiltration volumes are minimized. The wastewater will also have higher solids concentrations. Wastewater from each household is discharged to a sump that is isolated from the main vacuum line by a valve. The valve is normally closed to seal the vacuum lines so that vacuum can be maintained throughout the system. This valve opens automatically when a predetermined volume of wastewater has accumulated in the collecting sump, admits the sewage and air and then closes. Wastewater is collected in a receiving tank at a collection station and then pumped under pressure to the treatment plant. Applications for vacuum sewers include flat or slightly rolling terrain with small elevation changes.

Table 9.15 presents the advantages and disadvantages of each type of alternative collection system.

Table 9.15 Advantages and Disadvantages of Alternative Sanitary Sewers

System Type	Advantages	Disadvantages
STEG	• Lower potential for infiltration and inflow due to smaller system and the use of cleanouts instead of manholes. • Smaller diameter pipes could be installed in smaller trenches and reduces need for extensive road reconstruction. ` • Lower capital costs than conventional system. This type of system can be installed using trenchless methods. • Primary treatment would be provided in individual primary treatment tanks. Existing septic tanks could potentially be retrofit to provide primary treatment on-lot, if the tanks are in sufficient condition.	• System would require local pumping stations to pump wastewater from low lying lots. • System requires use of private property primary treatment tanks. Retrofit of all existing septic tanks may not be possible due to age. • Capital and O&M costs associated with pumping station. • O&M requirements of system are less well defined than conventional system. ' • STEG systems are not suitable for use where wastewater will be directed to a full mechanical treatment facility with primary treatment as reduced raw sewage solids loadings could reduce the effectiveness of the treatment process.
STEP	• Much lower potential for infiltration and inflow as constructed system will operate under pressure. • Smaller diameter pipes could be installed in smaller trenches and reduces need for extensive road reconstruction. • Lower capital costs than conventional system. This type of system can be installed using trenchless methods. • Primary treatment would be provided in individual primary treatment tanks. Existing septic tanks could potentially be retrofit to provide primary treatment on-lot, if the tanks are in sufficient condition. • No need to construct centralized primary treatment tank.	• System requires use of private property primary treatment tanks. Retrofit of all existing septic tanks may not be possible due to age. • Limited capacity during power outages. • Individual owners would be responsible for pumping costs. • O&M requirements of system are less well defined than conventional system. • STEG systems are not suitable for use where wastewater will be directed to a full mechanical treatment facility with primary treatment as reduced raw sewage solids loadings could reduce the effectiveness of the treatment process.
STEP with Grinder Pumps	• Much lower potential for infiltration and inflow as constructed system will operate under pressure. • Smaller diameter pipes could be installed in smaller trenches and reduce need for extensive road reconstruction. This type of system	• Limited capacity during power outages. • Individual owners would be responsible for pumping costs. • O&M requirements of system are less well defined than conventional system. • Grinder pump systems may not be suitable for use where wastewater will

Table 9.15 Advantages and Disadvantages of Alternative Sanitary Sewers

System Type	Advantages	Disadvantages
	can be installed using trenchless methods. • Lower capital costs than conventional system.	be pumped through a long forcemain as the higher raw wastewater solids concentrations would increase the potential and severity of odour issues at the downstream forcemain discharge locations.
Vacuum Sewers	• Much lower potential for infiltration and inflow as constructed system will operate under pressure. • Smaller diameter pipes could be installed in smaller trenches and reduce need for extensive road reconstruction. This type of system can be installed using trenchless methods.	• Central vacuum pumping station would be required. • O&M requirements of system are less well defined than conventional system. • Would require primary treatment at a centralized treatment facility if RSF treatment process is used. • Vacuum systems may not be suitable for use where wastewater will be pumped through a long forcemain as the higher raw wastewater solids concentrations would increase the potential and severity of odour issues at the downstream forcemain discharge locations.

Based on the advantages and disadvantages information provided in **Table 9.15**, a STEP system with grinder pumps is the most promising alternative sanitary system for servicing unserviced areas in Collingwood as STEG systems are generally not recommended in areas where wastewater will be treated at a conventional wastewater plant, such as the Collingwood WWTP. Vacuum sewers were not carried forward as these systems will require the construction of a vacuum station. For the purposes of developing options for servicing of Oliver Crescent, a STEP system with grinder pumps was included in Option 2.

Option 2 for Oliver Crescent would consist of the construction of STEP system with grinder pumps. This option would include works on private property including installation of grinder pumps in existing septic tanks or construction of a new tank to house grinder pumps, if existing septic tanks are in poor condition. On the Town's property, a small diameter (100mm) low pressure sewer would be constructed with a minimum of two cleanouts. The low pressure sewer would be installed using trenchless methods (if soils conditions permit), such as directional drilling, reducing the need for open cut construction. A series of pit excavations would be needed. The estimated cost to service Oliver Crescent would be $578K under this option. However, there would be additional O&M costs, if the Town agreed to maintain the grinder pumps, similar to Oxford County's policy and practice. **Table 9.16** presents the comparison of options for Oliver Crescent.

Table 9.16 Comparison of Servicing Options for Oliver Crescent

Option	Advantages	Disadvantages
Option 1 – Conventional Sanitary Sewer	• Minimal additional O&M costs with conventional sanitary sewer system • Conventional system with centralized pumping station	• Potential for low velocities due to low flow. • Higher capital cost than Option 2. • Open cut construction required and rock excavation due to shallow bedrock.
Option 2 – STEP system with Grinder Pumps	• Can be installed using trenchless methods, such as directional drilling, with excavation required at selected locations. • Infiltration should be negligible for this type of system as it is under pressure. Will reduce flow requiring treatment. • Lower capital cost than Option 1.	• Potential increase in O&M costs as Town could assume responsibility for maintaining grinder pumps • Use of trenchless methods is limited in areas where bedrock excavation is required. Based on available soils information, there is shallow bedrock along Oliver Crescent.

Based on the information provided in **Table 9.16**, servicing of Oliver Crescent with a low pressure sanitary sewer with grinder pumps is recommended. The estimated cost of servicing this area is estimated to be \$578K. This is a conservative cost estimate and assumes open cut construction of the low pressure sewer due to the presence of shallow bedrock. With a low pressure sanitary sewer system, infiltration will be minimal. As a result, servicing Oliver Crescent will require additional treatment capacity of 35m^3/d at the Collingwood WWTP.

9.3.2 Princeton Shores

The Princeton Shores areas consists of 44 residential properties on 10ha located on Princeton Shores Boulevard and Bartlett Boulevard. Similar to Oliver Crescent, this area is located adjacent to Georgian Bay and also has shallow bedrock. Two options have been considered including:

- Option 1 – Conventional sanitary sewers installed on Princeton Shores Boulevard, Bartlett Boulevard , a new 4 L/s pumping station at the north end of Princeton Shores Boulveard and a forcemain along Princeton Shores Boulevard and Highway 26. This new forcemain would discharge into the existing 750mm diameter sanitary sewer at the Cranberry SPS forcemain discharge location. In total, 1264m of 200mm diameter sanitary sewer, a 4L/s pumping station and 600m of 100mm diameter forcemain would be needed at an estimated cost of \$2.0M.

- Option 2 – Installation of a STEP system with grinder pumps (low pressure sewers) along Princeton Shores Boulevard, Bartlett Boulevard and Highway 26 using trenchless methods, such as directional drilling. In total, 44 grinder pumps would be installed in new interceptor tanks or in retrofitted existing septic tanks. A total of 1864m of 100mm low pressure sewer would be needed. The estimated cost for this option is \$1.4M. This is a conservative cost estimate and assumes open cut construction of the low pressure sewer on due to the presence of shallow bedrock.

Table 9.17 presents a comparison of these options for the Princeton Shores area.

Table 9.17 Comparison of Servicing Options for Princeton Shores

Option	Advantages	Disadvantages
Option 1 – Conventional Sanitary Sewer	• Minimal additional O&M costs with conventional sanitary sewer system	• Potential for low velocities due to low flow. • Higher capital cost than Option 2. • Open cut construction required and rock excavation due to shallow bedrock.
Option 2 – STEP system with Grinder Pumps	• Can be installed using trenchless methods, such as directional drilling, with excavation required at selected locations. • Infiltration should be negligible for this type of system as it is under pressure. Will reduce flow requiring treatment. • Lower capital cost than Option 1.	• Potential increase in O&M costs as Town could assume responsibility for maintaining grinder pumps • Use of trenchless methods is limited in area of shallow bedrock.

Based on the information provided in **Table 9.17**, servicing of the Princeton Shores area with a low pressure sanitary sewer with grinder pumps is recommended. The estimated cost of servicing this area is estimated to be $1.4M. With a low pressure sanitary sewer system, infiltration will be minimal. As a result, servicing the Princeton Shores area will require additional treatment capacity of 33m^3/d at the Collingwood WWTP.

9.3.3 West Highway 26

This area is located north and south of Highway 26 and east of Osler Bluff Road. Local streets include Lindsay Lane, Madeline Drive, Long Point Road, Silver Creek Drive, Georgian Court, Craigleith Court, Alpine Court and Forest Drive. A new local sewer system to a local pumping station would be required to service this area. There are a total of 123 properties on 113.4ha located on the above streets. Based on a PPU for single family residential of 2.9, and unit flow rates assigned for new growth, servicing of this area would result in an average flow of 125m^3/d and a peak flow of 30.4L/s. This area is located both south and north of Highway 26 and in addition, a crossing of Silver Creek would be needed. To service this area, local sewers, local pumping station and forcemain are recommended. Servicing would include the following:

- To service properties on Madeline Drive and Lindsay Lane, 1,806m of new 200mm diameter sanitary sewer will be required along Madeline Drive, Long Point Road and Lindsay Lane. This new sanitary sewer would discharge into the new local pumping station.
- To service properties on Silver Creek Drive, Georgian Court, Craigleith Court, Alpine Court and Forest Drive, 2,400m of new sanitary sewers will be required along Silver Creek Drive, Georgian Court, Craigleigh Court, Alpine Court and Forest Drive. These sewers will discharge into the new local pumping station.
- A new local pumping station located within the road allowance of Highway 26 immediately east of Long Point Road. A station firm capacity of 30.4L/s would be required.

- A new forcemain along Highway 26 from Long Point Road to the existing 750mm diameter sanitary sewer located east of Princeton Shores Boulevard. This forcemain would be 2.4km in length and would be a 150mm diameter.

The above noted servicing has an estimated cost of $12.4M.

As an alternative to the above projects, this area could be serviced through the neighbouring Craigleith WWTP and collection system located in ToBM. ToBM's Craigleith WWTP is located immediately west of Long Point Road and there is a gravity sewer at the intersection of Highway 26 and Osler Bluff Road that conveys wastewater to the Craigleith SPS. To service the West Highway 26 lands at Craighleith WWTP would require the following infrastructure:

- To service properties on Madeline Drive and Lindsay Lane, 1,060m of new 200mm diameter sanitary sewer will be required along Madeline Drive, Long Point Road and Lindsay Lane. This new sanitary sewer would discharge a new local pumping station. In total 2,450m of new 250mm diameter sanitary sewer would be needed.
- A new local pumping station located within the road allowance of Long Point Road adjacent to the Craigleith WWTP. A station firm capacity of 30.4L/s would be required.
- A new forcemain to the Craigleigth WWTP. The forcemain would extend 350m to the headworks of the Craigleith WWTP.
- To service properties on Silver Creek Drive, Georgian Court, Craigleith Court, Alpine Court and Court, Craigleigh Court, Alpine Court and Forest Drive. These sewers could potentially discharge by gravity into the ToBM sanitary sewer system.

The cost for the alternative servicing scheme is estimated to be $9.4M, not including any costs associated with upgrades to infrastructure in ToBM. ToBM has plans to undertake a Master Servicing Plan Study in 2020 and it is recommended that the Town request that servicing of this area through ToBM be considered as an option in the upcoming study.

9.3.4 Mountain Road West

This area is located east of Osler Bluff Road surrounding Mountain Road.Local streets include Holly Court,Laurel Boulevard, Juniper Court, Evergreen Road, Trails End, Slalom Gate Road and Mountainview Court. Servicing of this area would require a new local pumping station and forcemain that would discharge into the Mountain Road sanitary sewer. There are a total of 134 properties on 33.3ha on the above streets. Based on a PPU of 2.9, unit flow rates assigned for new growth, servicing of this area with a conventional sewer system would result in an average sanitary flow of 136m^3/d and a peak flow of 12.3L/s.

Due to the size of this area and the need to construct a new pumping station west of Silver Creek on Mountain Road, an alternative STEP grinder pump system is not considered feasible. To service this area with a conventional system would require:

- New 200mm diameter sanitary sewers on Holly Court, Laurel Boulevard, Juniper Court, Evergreen Road, Trails End, Slalom Gate Road and Mountainview Court. The total length of sanitary sewer is 2,550m.
- A new 200mm diameter sanitary sewer on Mountain Road from west of Slalom Gate to immediately east of Silver Creek. A total length of 390m would be required.

- A new local pumping station with a firm capacity of 12.3L/s. This station would receive flows from the entire area and would be located within the Mountain Road road allowance immediately west of Silver Creek.
- A new 150mm diameter forcemain from the new local pumping station to the first maintenance hole on Mountain Road located west of Tenth. In total, 1740m of forcemain will be needed.

The cost for the above noted sanitary sewers, pumping station and forcemain is estimated to be $4.9M.

As an alternative, the Town could consider servicing of this area through the ToBM. ToBM does have an existing sanitary sewer along Osler Bluff Road and servicing of this area though the ToBM sanitary sewer would eliminate the need for the new pumping station and forcemain in Collingwood. ToBM's existing sanitary sewer on Osler Bluff Road conveys flows to the Craigleith SPS. The alternative servicing would require the following:

- New 200mm diameter sanitary sewers on Holly Court, Laurel Boulevard, Juniper Court, Evergreen Road, Trails End, Slalom Gate Road and Mountainview Court. The total length of sanitary sewer is 2,550m.
- A new 200mm diameter sanitary sewer on Osler Bluff Road from Mountainview Court to the existing ToBM sanitary sewer on Osler Bluff Road. A total length of 850m would be required.

Discussions with ToBM would be need to confirm the feasibility of this alternative. The estimated cost for this alternative is $4M but does not include any costs associated with improvements to the ToBM conveyance, pumping or treatment infrastructure.

9.3.5 Beachwood Area

This area is located along Georgian Bay shoreline east of Sixth Line to Fairgrounds Road.

There are a total of 626 properties within a 150ha area. Based on a PPU of 2.9, unit flow rates assigned for new growth, servicing of this area would result in an average sanitary flow of 635m^3/d and a peak flow of 54.3L/s. It is noted that the St. Clair SPS and existing 675mm diameter sanitary sewer between Huronia Pathway and the St. Clair SPS were oversized to accommodate servicing of this area. With servicing of this area, the peak flow reaching the St. Clair SPS will be increased by 54.3L/s. A capacity increase would be required at the St. Clair SPS to service this area. A review of the record drawings for the St. Clair SPS identified that the station was designed for the installation of a third pump. To service this area would require the installation of a third pump. If this area is to be serviced after planned and potential growth, the peak flow reaching the St. Clair SPS would be 199L/s which exceeds the current firm capacity of the station of 155L/s. With the addition a third pump, sufficient capacity will be available.

Servicing of this area is challenging due to the lack of grade and the presence of shallow bedrock. To service this area, two pumping stations have been identified. To service this area will require:

- A new 300mm diameter sanitary sewer along Beachwood Road extending from west of Fairground Road to west of James Street. This new sanitary sewer would discharge into a new pumping station. The length of sanitary sewer required would be 2.3km.
- A new pumping station with a capacity of 55L/s in the vicinity of James Street and Beechwood Road. A new 200mm diameter forcemain from the new pumping station to Poplar Sideroad would also be needed. This forcemain would cross the Batteaux River.

- A new 300mm diameter sanitary sewer from Poplar Sideroad to west of Huronia road. The total length of sanitary sewer would be 1.2km.
- A second new pumping station with a capacity of 75L/s in the vicinity of Beechwood Road and West of Huronia Road would be needed with a new 200mm diameter forcemain with a 50m length would be needed.
- Local 200mm diameter sanitary sewers to provide servicing to residents. In total, 7.5km of new 200mm diameter sanitary sewer would be required on Stalker (79m), Sandell (165m), Kohl (217m), Downer (257m), Belcher(238m), Currie (178m), Edgar (219m), York (220m), Selkirk (265m), Glen (143m), McAllister(136m), Lane A (141m), Arthur (139m), Indian Trail (601m), Bellhomme (166m), King (258m), Wellington (103m), Georgian Manor (1,354m), Summerview (381m), Lakeview (168m), Glenlake (796m), Woodcrest (370m), Dellpark (153m), Broadview (749m) and Braeside (402m).
- Installation of a third 155L/s pump at the St. Clair SPS. The estimated cost to install the pump is $1M.
- The estimated cost for the above sewers and pumping station improvements is $24.9. The flatness of this area will make this solution challenging to implement.

As an alternative, a servicing scheme that includes both conventional gravity sewers and pumping stations and low pressure sanitary sewers could be considered. This option would require:

- A new low pressure sanitary sewer located along local streets and on Beachwood Avenue from Fairground Road to west of James Street. This low pressure sanitary sewer would discharge into a new pumping station. For properties serviced by the low pressure sewer system, grinder pumps, installed in a retrofitted septic tank or a new interceptor tank will be needed. In total 8.7knm of new pressure sewer would be installed using trenchless methods.
- A new pumping station with a capacity of 20L/s in the vicinity of James Street and Beechwood Road. A new 150mm diameter forcemain from the new pumping station to Poplar Sideroad would also be needed. This forcemain would cross the Batteaux River
- A new 250mm diameter sanitary sewer along Beachwood Road extending from west of Poplar Road to Huronia Parkway. This new sanitary sewer would discharge into a new pumping station. The length of sanitary sewer required would be 1.2km.
- A new 35 L/s pumping station and short forcemain would be needed to lift the sewage into the existing 675mm diameter sanitary sewer on West Huronia Road.
- Installation of a third 155L/s pump at the St. Clair SPS. The estimated cost to install the pump is $1M.
- The estimated cost for the above sewers and pumping station improvements is $17.4M

Table 9.18 presents a comparison of these options for the Beachwood Area.

Table 9.18 Comparison of Servicing Options for Beachwood Area

Option	Advantages	Disadvantages
Option 1 – Conventional Sanitary Sewer	• Minimal additional O&M costs with conventional sanitary sewer system	• Potential for low velocities due to low flow. • Higher capital cost than Option 2. • Open cut construction required and rock excavation due to shallow bedrock.
Option 2 – STEP system with Grinder Pumps with Conventional Sanitary Sewer on Beachwood	• Can be installed using trenchless methods, such as directional drilling, with excavation required at selected locations. • Infiltration should be negligible for this type of system as it is under pressure. Will reduce flow requiring treatment. • Lower capital cost than Option 1.	• Potential increase in O&M costs as Town could assume responsibility for maintaining grinder pumps • Use of trenchless methods in areas with shallow bedrock are limited.

Based on the information provided in **Table 9.18**, servicing of the Beachwood Area with a new conventional sanitary sewer on Beachwood and low pressure sewers (STEP system) on local streets is recommended. The estimated cost of servicing this area is estimated to be $17.4M. With a low pressure sanitary sewer system, infiltration will be minimal. As a result, servicing of the Beachwood Area will require additional treatment capacity of 472m^3/d at the Collingwood WWTP.

9.3.6 Additional Treatment Costs

In addition to the costs for local pumping, local sewers and local forcemains, the Collingwood WWTP would need to be expanded beyond the rated capacity of 36,185m^3/d recommended for servicing of planned and potential development. To accommodate flows from all five of the unserviced areas would require an additional 801m^3/d of treatment capacity. The cost to provide this treatment capacity would be $7.5M.

9.3.7 Summary of Unserviced Areas

Table 9.19 provides a summary of the costs for servicing of the five unserviced areas. It is noted that the costs shown for the West Highway 26 and Mountain Road West are based on servicing these areas through the Collingwood WWTP. It is recommended that the Town enter into discussions with ToBM to determine the cost and feasibility of servicing these areas through the ToBM sanitary sewer system.

Table 9.19 Summary of Costs for Servicing Currently Unserviced Areas

Area	Description of Servicing Requirements	Estimated Costs of Local Pumping, Sewers and Forcemains	Additional Treatment Capacity Required (m^3/d)	Estimated Cost of Additional Treatment Capacity ($)	Total Cost of Servicing
Oliver Crescent	513m of low pressure sanitary sewers with grinder pumps	$580K	35	$330K	$910

Table 9.19 Summary of Costs for Servicing Currently Unserviced Areas

Area	Description of Servicing Requirements	Estimated Costs of Local Pumping, Sewers and Forcemains	Additional Treatment Capacity Required (m^3/d)	Estimated Cost of Additional Treatment Capacity ($)	Total Cost of Servicing
Princeton Shores	1,864m of low pressure sanitary sewers with grinder pumps	1.4M	33	$310K	$1.7M
West Highway 26	4,200m of 200mm diameter sanitary sewer, new 31L/s pumping station and 240m of new 150mm diameter forcemain	$12.4M	125	$1.2M	$13.6M
Mountain Road West	2940m of 200mm diameter sanitary sewer, 13L/s pumping station and 1,740m of 150mm diameter forcemain	$4.9M	136	$1.3M	$6.2M
Beachwood Area	New 1200m of 250mm diameter sanitary sewer, new 35 L/s pumping station, 8.7km of new low pressure sewers with grinder pumps and new pump installation at St. Clair SPS	$17.4M	472	$4.4M	$21.8M

In total, the population in these five currently unserviced areas is 2,810 persons within a combined area of 315.5ha. Following servicing, these areas would contribute an additional 801m^3/d of average flow to the Collingwood WWTP.

10 Implementation

The following sections provides implementation plans for the water and sanitary projects included in the preferred alternatives. **Section 10.1** provides an inflow and infiltration reduction strategy for the Town. Inflow and Infiltration reduction was identified as an implementation measure following the evaluation of alternatives. **Sections 10.2** and **10.3** provide additional details on the implementation of water and sanitary projects.

10.1 Inflow and Infiltration Reduction

Through the evaluation of alternatives, demand management programs were identified as implementation measures. One key element of demand management is inflow and infiltration reduction. A successful inflow and infiltration reduction program would involve ongoing completion of repairs to the existing sanitary sewer system as well as program activities to address private property sources to reduce peak flows within the sanitary sewer system. **Figure 10-1** contains a graphic which demonstrates I/I sources with Town infrastructure and private property. A successful program would have the following benefits:

- Reduction of peak flows in sanitary sewers where upgrades have been identified as necessary to service future growth.
- Reduction in peak flows reaching the Collingwood WWTP during typical wet weather events. These reductions would reduce treatment costs.
- Reduction in peak flows entering the sanitary sewer system during significant rainfall events. These reductions would reduce the risk of basement flooding and bypasses within the existing system.
- Allow the Town flexibility for the expansion of the Collingwood WWTP.

To effectively achieve reductions in I/I in the sanitary sewer system, a strategy has been developed that includes programs and activities to reduce I/I in the Town's sanitary infrastructure, from existing private property sources and to prevent I/I from occurring in new development areas. **Table 10.1** presents the overall strategy, identifies strategic areas, identifies specific programs within each strategic area, and provides goals and objectives within each of these program areas and provides information on activities and outcomes to be achieved.

Table 10.1 Inflow and Infiltration Reduction Strategy

Strategic Area	Specific Programs	Goals and Objectives	Activities and Outcomes
Town's Sanitary Sewer Infrastructure	Investigation program including flow monitoring, CCTV inspection and maintenance hole inspection.	Programs are aimed at identifying and prioritizing sources of I/I within the Town's sanitary system.	1. Identification of priority areas for I/I reduction 2. Within priority areas, prioritize list of maintenance holes and sanitary sewers where repairs are needed 3. Develop rehabilitation plans, budgets, work assignments and tender documents. 4. Schedule tenders for release.

Table 10.1 Inflow and Infiltration Reduction Strategy

Strategic Area	Specific Programs	Goals and Objectives	Activities and Outcomes
	Repair programs	Annual programs of sewer system repairs including spot repairs, maintenance holes repairs, sewer lining and sewer replacement	1. Completion of annual capital projects for sewer and maintenance hole repairs. Can be integrated with Town's asset management activities.
Private Property	Investigations program including smoke and dye tests in priority areas for I/I reduction	Identify direct sources of inflow and infiltration on private property	1. Resident notification of smoke testing. 2. Complete smoke testing and review results to identify probable connections. 3. Confirmation dye tests to confirm connection 4. List of private property sources
	Private Property Repairs	Removal, where possible, of private property sources	1. Assess and select methods of removing sources including by-law enforcement, rebates/ incentives or completion of works by Town. 2. If incentive/ rebate programs are selected, develop program information for residents, select and develop rebate/ incentive values and process, and develop budgets. 3. If private owners undertake work, complete inspection and confirm work has been completed.
	Foundation Drain Removal Programs	Removal of Foundation Drain/ Sump Pump Direct Connections	1. Complete an assessment of historical records to identify areas where foundation drains are likely connected to the sanitary sewer system. 2. Consider household drainage surveys on individual properties and recommend to residents how to redirect any sump pump discharges to the surface.
New Development	Enhance new development standards and requirements	Prevent I/I from occurring in new development areas	1. Require developers to monitor flows prior to infrastructure assumption and provide proof that new development is not contributing I/I beyond a set threshold. The set threshold should be set below the Town's infiltration allowance, recognizing that I/I will worsen during the lifespan of the asset.

Table 10.1 Inflow and Infiltration Reduction Strategy

Strategic Area	Specific Programs	Goals and Objectives	Activities and Outcomes
			This may require changes to the standard Development Agreements. 2. Enhance requirements for inspection and certification, either by Town forces or consulting engineers. 3. Consider industry outreach to Development industry to provide education on any new requirements. 4. Consider innovative programs, including a program where developers identify and remove private property sources in exchange for allocation.

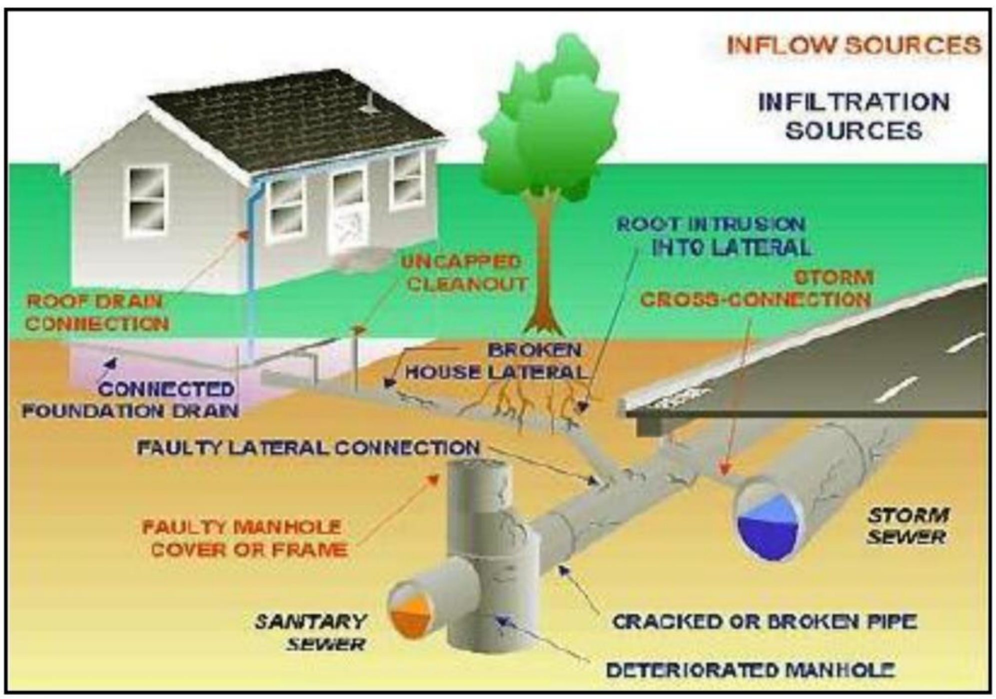


Figure 10-1 Inflow and Infiltration Sources

The above strategy contains a number of innovative approaches that the Town could implement. Further information and examples of where these approaches have been successful is contained below:

- The Town could require post construction flow monitoring and assessment of sanitary sewer infrastructure as part of the development process. Several municipalities in Ontario currently have this requirement, including the Town of New Tecumseth and Woolwich Township. Because of historically high I/I within the St. Jacobs area of Woolwich, developers are required to complete post construction monitoring and data analysis to prove that new infrastructure contributes I/I which is less than the Township's design allowance.

- As a condition of approval, the Town could require developers and their consulting engineers to identify and repair private property sources, in exchange for allocation for their developments. York Region has partnered with landowners groups and a number of local area municipalities to implement this type of program. In York Region, capacity allocation is released to local municipalities, when it can be confirmed that repairs completed by developers have been successful. The Region, through extensive monitoring and assessment, has identified target areas where excessive I/I is present. In these areas, Developers and their consultants, undertake existing data review, modelling, field inspection and field testing (smoke and dye, CCTV, etc.) to identify specific sources, to quantify I/I and to recommend repairs to reduce I/I. Upon approval of the repair lists by the Region and local municipality, the Developer completes the repairs and is awarded allocation. The Region has a formula that determines the allocation awarded based on the I/I removed from the system.
- Piloting of methodologies and technologies is recommended to ensure that the Town is continuing to achieve the results required at a reasonable cost. Pilot studies can be geared to particular areas of the Town or can be geared towards testing of technologies. In both cases, it is recommended that the effectiveness of any pilot be accessed to determine its cost effectiveness. To measure effectiveness, pre and post construction monitoring and assessment is recommended.
- Sump pumps connected to the sanitary sewer system allow groundwater to be discharged into the sanitary sewer system which is then treated at the Collingwood WWTP. In areas where these types of connections are prevalent, extended periods of high flow are often observed at the treatment plant. In 2017, the Town partnered with a number of groups and agencies in the Smart Stormwater Pilot program "Smart Pump" project. The information gained through the "Smart Pump" project can be used to design a Town wide strategy to address existing sump pump connections.
- Integration of repairs to the Town's sanitary sewer system with the Town's asset management initiatives. As part of its asset management plan, the Town considers asset condition. Where areas where high I/I is an issue, the Town can integrate I/I into its asset management planning processes and prioritize repair projects based on structural and service condition.

10.2 Water Projects

The timing of the supply, storage and pumping alternatives were estimated assuming linear growth of demands between the existing, planned, and potential scenarios. The future requirement for supply, storage and pumping was compared to the available capacity in each case, and a trigger year was estimated based on the linear interpolation between 2016, 2032 and 2044. In some cases the upgrade was recommended to be completed when an 80% capacity trigger was reached in order to provide a safety factor. The trigger year was assigned as the date of completion for each alternative and can be seen in **Table 10.2. Table 10.2** also provides the opinion of probable cost developed for each project based on high level cost estimates. Previous studies, recent local tenders, and RS Means were used to develop cost estimates specific to the Town of Collingwood. The accuracy of these costs varies according to the level of project definition. Since Master Plan costs are used for planning purposes, project contingencies were built into the cost estimates. Further details on the development of cost estimates can be found in **Appendix G**. **Figure 10-2** presents the location of recommended water projects.

Table 10.2 Summary of Water Project Costs and Timelines

Problem Area	Alternative #	Description	Diameter (mm)	Length (m)	2018 Opinion of Probable Cost	Reason for Project	Funding Source	Approximate Duration	Completion Timeline	Future Value	EA Schedule	Comments
Water Treatment/Supply Capacity												
Total System Supply	W-R-3	Water Efficiency Measures			$ 200,000	To limit future supply capacity requirements	Growth	10 years	Ongoing		NA	$20,000/year for 10 years (administration, targeted programs, public/youth education)
	W-R-4	WTP Upgrade			$ 40,000,000	To provide future supply capacity	85 Growth / 15 Non-Growth	5 years	2025	$ 48,600,000	C	Based on rounded Ainley estimate of $36 million to increase capacity to 47 MLD total.
Storage												
Zone 1 Storage	W-S1-1	New Z1 ET Option 1 New Z1 ET Option 2			$ 9,600,000	to provide future storage, improve low pressure areas, increase fire flows	50 Growth / 50 Non-Growth	3 years	2030	$ 13,200,000	B	Estimated volume of 4,000 m^3. Intended to expand storage for future growth and eventually replace existing ET
Zone 2 & 3 Storage	W-S2-1	Stewart Rd Phase 2 Reservoir			$ 2,700,000	to provide future storage	Growth	1 year	2035	$ 4,200,000	A	Costs can be updated pending Stewart Road Tender submission
	W-S2-2	Stewart Rd Phase 3 Reservoir			$ 2,700,000	to provide future storage	Growth	1 year	2044	$ 4,900,000	A	
Pumping												
Zone 1, 2 & 3 Pumping	W-P-1	Carmichael BPS Improvements			$ 1,000,000	To improve discharge from Carmichael	85 Growth / 15 Non-Growth	3 years	2022	$ 1,200,000	B	Based on preliminary Town estimate
	W-P-2	Decommission Georgian Meadows			$ 200,000	Booster pumps and isolation valves no longer needed	Growth	1 year	2022	$ 300,000	A	To be coordinated with Stewart Road Phase 1 Completion
	W-P2-1	Stewart Rd Ultimate Pump Upgrades			$ 500,000	to meet future MDD and fire flow pumping requirements	Growth	1 year	2038	$ 900,000	A	Costs can be updated pending Stewart Road Tender submission

Problem Area	Alternative #	Description	Diameter (mm)	Length (m)	2018 Opinion of Probable Cost	Reason for Project	Funding Source	Approximate Duration	Completion Timeline	Future Value	EA Schedule	Comments
Linear Infrastructure												
Major Infrastructure	W-L1-1A	WTP to Hwy 26				to create a large diameter watermain loop for capacity & storage	85 Growth / 15 Non-Growth				A	Expected to be completed in phases. Timed with other Town projects and developments.
		Maple to Hickory	400	785	$ 2,200,000			1 year	2025	$ 2,700,000		
		Hickory to Hwy 26	400	420	$ 1,200,000			1 year	2030	$ 1,700,000		
		WTP to Heritage on Simcoe	500	1,365	$ 4,500,000			1 year	2040	$ 7,600,000		
	W-L1-1B	FM: Hwy 26 (Old Mountain Rd. to Carmichael PS)	400	2,122	$ 5,800,000		85 Growth / 15 Non-Growth	1 year	2045	$ 10,700,000	A	Review if required following C-factor testing and Harbour St. connection.
	W-L1-1C	FM: Sixth St. (Hurontario St. to Stewart Rd. PS)	400	1,960	$ 5,400,000		85 Growth / 15 Non-Growth	3 year	2023	$ 6,300,000	A	Coincide with small cast iron replacement, phased over 3 years ending 2023. Coinciding with Stewart Rd PS and Reservoir Completion.
	W-L1-1D	FM: High St. (Old Mountain Rd. to Sixth St.)	400	1,225	$ 3,400,000		85 Growth / 15 Non-Growth	1 year	2030	$ 4,700,000	A	Timed with W-S1-1.
	W-L1-1E	FM: Side Launch Way to Hume St. along St Paul St.	400	830	$ 2,300,000		85 Growth / 15 Non-Growth	1 year	2045	$ 4,300,000	A	Potential to coincide with St Paul St. Upgrade (2024).
	W-L1-1F	FM: Extra to ET Option 1	400	1,468	$ 4,000,000		Growth	1 year	2030	$ 5,500,000	A	Timed with W-S1-1. Only required for Option 1.
Local Infrastructure	W-L-3	Poplar Side Rd. Connection	200	720	$ 1,600,000	Future Connectivity	Growth	1 year	2035	$ 2,500,000	A	Timed with development.
	W-L-4	Birch St. Upgrade	200	350	$ 760,000	Fire Flow & Connectivity	Non-Growth	1 year	2030	$ 1,100,000	A	
	W-L-5	Campbell St. Upgrade	200	220	$ 480,000	Fire Flow & Connectivity	50 Growth / Non-Growth 50	1 year	2019	$ 500,000	A	
	W-L-6	Collins St. Upgrade	250	130	$ 320,000	Connectivity	Non-Growth	1 year	2035	$ 490,000	A	
	W-L-7	Harbour St. Connection	300	565	$ 1,400,000	Fire Flow & Pressure	85 Growth / 15 Non-Growth	1 year	2020	$ 1,500,000	A	
	W-L-8	Hume St. Upgrade	400	410	$ 1,120,000	Connectivity	15 Growth / 85 Non-Growth	1 year	2035	$ 1,800,000	A	Review if required following C-factor testing
	W-L-9	Minnesota St. Upgrade	200	100	$ 220,000	Fire Flow & Connectivity	Non-Growth	1 year	2030	$ 310,000	A	
	W-L-10	Mountain Rd. Connection	300	450	$ 1,100,000	Fire Flow & Pressure	85 Growth / 15 Non-Growth	1 year	2025	$ 1,400,000	A	
	W-L-11	Raglan St. Upgrade	400	250	$ 680,000	Connectivity	85 Growth / 15 Non-Growth	1 year	2035	$ 1,100,000	A	Review if required following C-factor testing
	W-L-12	Third St. Upgrade	200	565	$ 1,300,000	Fire Flow & Connectivity	Non-Growth	1 year	2023	$ 1,500,000	A	
	W-L-13	Second St. Connection	150	125	$ 240,000	Fire Flow & Connectivity	Non-Growth	1 year	2025	$ 300,000	A	
	W-L-14	High St. Connection	300	450	$ 1,100,000	Connectivity & Pressure	Growth	1 year	2030	$ 1,510,000	A	Timed with Stewart Rd completion & Sixth street development. Potential to time with Summit View road & storm work.
	W-L-15	Fourth St. Connection	150	120	$ 230,000	Connectivity	Non-Growth	1 year	2035	$ 350,000	A	
	W-L-16	St Paul St. Upgrade	150	650	$ 1,300,000	Fire Flow & Connectivity	Non-Growth	1 year	2024	$ 1,600,000	A	Review timing and sizing with W-L1-1E. Option to twin 400mm FM with a 150mm, or connect services to FM.

Problem Area	Alternative #	Description	Diameter (mm)	Length (m)	2018 Opinion of Probable Cost	Reason for Project	Funding Source	Approximate Duration	Completion Timeline	Future Value	EA Schedule	Comments
Linear Infrastructure												
	Valves											
System Valves	W-V-2	Cranberry Trail PRV (10" S106-RPS-C)			Adjust Settings	Adjust settings to maintain Zone 1A and feed Zone 1 in emergency	Growth	1 year	2022		A	Timed with Carmichael BPS upgrades.
System Valves	W-V-3	Osler PRV and Check valve			$ 510,000	Create valve configuration to feed Zone 1A or 2 in emergency	Growth	1 year	2025	$ 620,000	A	After Stewart Rd Completion.
System Valves	W-V-4	Mountain Road PRV			$ 150,000	New valve to feed Mountain Rd low pressure & fire from Zone 2	85 Growth / 15 Non-Growth	1 year	2025	$ 190,000	A	Timed with W-L-10.
System Valves	W-V-5	High Street PRVs (8" & 6" Singer 106-PR-R)			Adjust Settings	Adjust settings to supply Zone 2 in emergency and regulate low pressure in Zone 1	Non-Growth	1 year	2030		A	Timed with W-L-14.
System Valves	W-V-6	Hurontario PRV			Adjust Settings	Adjust settings to supply Zone 2 in emergency and regulate low pressure in Zone 1	Non-Growth	1 year	2020		A	
System Valves	W-V-7	Dey Drive Check Valve			Completed	New check valve to supply Zone 2 in emergency	Growth	1 year	2019		A	
System Valves	W-V-8	Pretty River Check Valve			$ 50,000	New check valve to supply Zone 2 in emergency if needed	Growth	1 year	2025	$ 61,000	A	Timed with development. Review if required following W-V-7.
System Valves	W-V-9	Raglan PRV (12" Pressure Reducing)			Adjust Settings	Adjust setting to supply Zone 2 in emergency	Non-Growth	1 year	2025		A	
System Valves	W-V-10	Sixth Line PRV or Check			$ 140,000	New valve to supply Zone 2 in emergency	Growth	1 year	2035	$ 220,000	A	Timed with industrial developments. Review if required following other valves.
	Other											
Other	W-O-1	C-Factor Field Testing/ Model Calibration			$ 60,000	Test C-factors & flows of old large diameters, potentially offset linear replacement	Growth / Non-Growth	2 years	2020	$ 64,000	NA	Assumed $15,000 for field testing, $45,000 for model calibration.
Other	W-O-2	100mm and Cast Iron Replacement Program			$ 1,000,000	Replace old small diameter cast iron water main to improve fire flow & connectivity	Non-Growth	10 years	Ongoing		A	Assumed 1 project @ $100,000/ yr. for 10 yrs.
Other	W-O-3	Water Efficiency Study			$ 100,000	Review water usage and recommend location-specific efficiency measures	Growth / Non-Growth	2 years	2020	$ 110,000	NA	

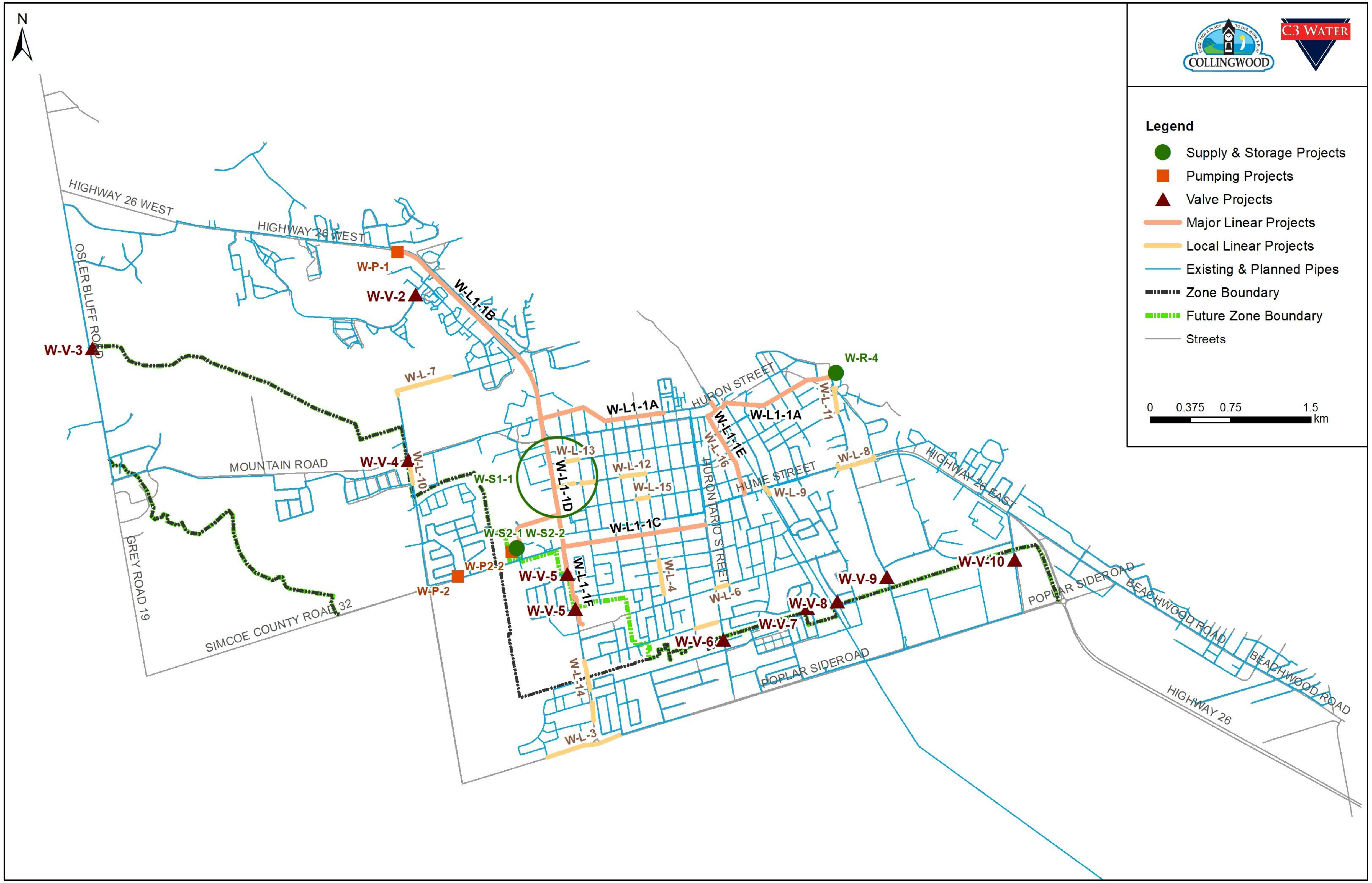


Figure 10-2 Recommended Water Projects

10.3 Sanitary Projects

The timing of the treatment and pumping and sanitary sewer recommended projects was estimated assuming linear growth of demands between the existing, planned, and potential growth scenarios. The future requirement for each project was compared to the available capacity in each case, and a trigger year was estimated based on the linear interpolation between 2019, 2032 and 2044. In some cases, the upgrade was recommended to be completed when an 80% capacity trigger was reached in order to provide a safety factor. The trigger year was assigned as the date of completion for each alternative and can be seen in **Table 10.3**. **Table 10.3** also provides estimated costs developed for each project based on high level cost estimates. Previous studies and recent local tenders were used to develop cost estimates specific to the Town of Collingwood. The accuracy of these costs varies according to the level of project definition. Since master plans costs are used for planning purposes, project contingencies were built into the cost estimates. **Figure 10-3** and **Table 10.3** presents the location of recommended sanitary projects.

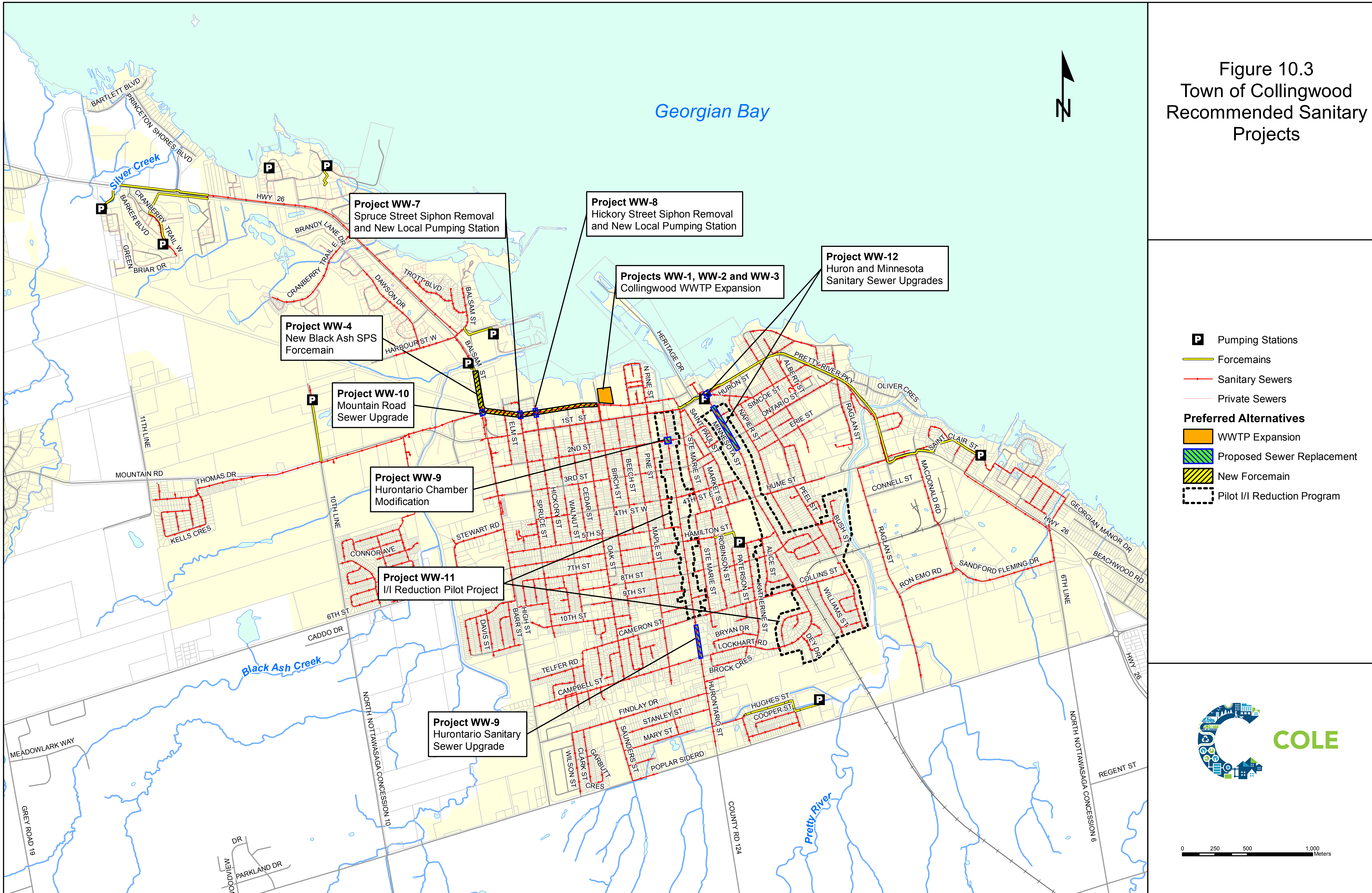


Figure 10.3
Town of Collingwood
Recommended Sanitary Projects

Table 10.3 Summary of Proposed Sanitary Projects and Timelines

	Project ID	Description	Treatment Capacity Provided (m^3/d)	Estimated Cost and Funding Source	Completion Timeline and Phasing	Reason for Project	Further Studies Required
Wastewater Treatment	WW-1	Collingwood WWTP Treatment plant expansion to provide a rated capacity of 36,185 m^3/d and a peak capacity of 90,463 m^3/d.	36,185	$89M Will be funded through growth funding	Project can implemented in two phases. Expansion project will be triggered in 2026 and will need to be in service by 2036. Expansion could be completed in two phases.	Project required to provide additional treatment capacity to support growth requirements.	Addendum to completed Schedule C Class EA is required (2011).
Wastewater Treatment	WW-2	Improvement to existing outfall to meet B-1-5 requirements,228m of new 900mm diameter outfall sewer will be needed.	-	$1.2M Will be funded through growth funding	Project cannot be phased and will be required with first phase of Collingwood WWTP expansion which will be triggered in 2026.	Project required to provide additional treatment capacity to support growth requirements.	Further study will be required to identify outfall route, discharge location and diffuser requirements. Requirements can be addressed in Addendum to completed Schedule C Class EA.
Wastewater Treatment	WW-3	Additional Studies (Class EA Addendum and assimilative capacity assessment)	-	$500K Will be funded through growth funding	Project can be initiated following completion of Master Plan	Project required to revisit preferred design concept for plant expansion and consider all approval requirements.	Assimilative capacity assessment will be needed to identify discharge limits and objectives and to identify location of new outfall discharge.
Trunk and Local Sanitary Sewers	WW-4	Twin Black Ash SPS Forcemain from Black Ash SPS to Collingwood WWTP headworks (1390m – 500mm diameter)	-	$1.2M Will be funded through growth funding	Project cannot be phased and will be required to be in service in 2036.	Project required to provide additional conveyance capacity for growth and system redundancy. New forcemain sized to have a capacity equal to or greater than the station capacity.	Schedule A+ (if located within existing utility corridor) requirements met by Master Plan
Trunk and Local Sanitary Sewers	WW-7	Spruce Street Improvements and siphon decommissioning (new local sewer, pumping station and forcemain)	-	$100K Non-growth funding	Project required in 2032.	Project required to eliminate siphon and improve level of service to existing residents	Detailed design required
Trunk and Local Sanitary Sewers	WW-8	Hickory Street Improvements and siphon decommissioning (new local sewer, pumping station and forcemain)	-	$100K Non-growth funding	Project required in 2032	Project required to eliminate siphon and improve level of service to existing residents	Detailed design required

	Project ID	Description	Treatment Capacity Provided (m^3/d)	Estimated Cost and Funding Source	Completion Timeline and Phasing	Reason for Project	Further Studies Required
	WW-9	Hurontario Street sewer replacement (Campbell to Collins) (368m of 375mm) and modification to Second Street chamber	-	\$407k Will be funded through growth funding	Project required in 2032	Project required to provide additional conveyance capacity for growth	Schedule A+ (if located within existing road allowance) requirements met by Master Plan
	WW-10	Mountain Road sewer upgrade west of High Street (96m – 600mm)	-	\$112K Will be funded through growth funding	Project required in 2032	Project required to provide additional conveyance capacity for planned and potential growth. Can consider upgrade to a larger section of Mountain Road sanitary sewer for built boundary growth.	Schedule A+ (if located within existing road allowance) requirements met by Master Plan
	WW-11	Inflow and infiltration reduction pilot program in Hurontario and Alice Street area to identify and reduce sources of I/I. If successful, implement in Minnesota area to reduce peak flows to Minnesota SPS.	-	\$200K Non-growth funding	Project can be initiated in 2020	Project required to reduce peak flows to the Collingwood WWTP and improve performance of the Hurontario Street sanitary sewer and local sanitary sewers on Alice, Manning and Lorne.	Field investigation and assessment study required to identify sources of I/I in the system and develop remediation plan for removing these sources.
	WW-12	Sewer improvements on Minnesota Street and Huron Street (19m-750 and 380m-375mm)		\$398K	Project required in 2032	Project required to provide capacity in local sewers. Need for project may be mitigated if I/I reduction initiatives in this area are successful.	Schedule A+ (if located within existing road allowance) requirements met by Master Plan

11 Conclusions and Recommendations

The Town of Collingwood has completed a Master Servicing Plan for Water and Sanitary Sewer Systems to identify water and sanitary servicing projects that will be required to accommodate growth over the planning horizon, including residential and employment growth. The Master Servicing Plan also considers servicing of neighbouring municipalities and sanitary servicing needs for currently unserviced areas in the Town. By considering water and sanitary requirements, an optimal design and delivery of services can be planned for.

A detailed analysis of existing water and sanitary systems and consideration of future growth requirements led to the development of alternatives to provide the required servicing. Following a detailed evaluation of alternatives, preferred alternatives for both water and sanitary systems were developed. Implementation is planned to occur over time.

Recommendations of this study include:

1. The Town should trigger an expansion project to provide additional treatment capacity at the Collingwood WWTP by 2026. The expansion should be sized to provide capacity to 36,185m^3/d so that the projected flow at the end of the planned and potential development period is at 80% of the expanded rated capacity. A peaking factor of 2.5 has been assumed for the design of the upgrade. The need for equalization storage at the facility should be considered in the detailed design. To implement this project, an Addendum to the 2011 Class EA will be needed. Phasing of the required capacity should be considered in the EA Addendum.
2. The Town should embark on an Infiltration and Inflow Reduction Pilot Project in the Minnesota Street and Hurontario Street areas. These areas have been selected as local capacity constraints have been identified and flow monitoring identified these areas as contributing excessive I/I. The program should consist of flow monitoring at upstream sites to pinpoint areas contributing excessive I/I, investigations of Town infrastructure including CCTV inspection and maintenance hole inspection, investigations of private property sources including smoke and dye testing, development and implementation of rehabilitation plans for Town infrastructure and implementation of private property programs to encourage or compel private property owners to remove sources of I/I located on private property.
3. The Town should consider decommissioning and replacement of the existing siphons at Hickory and Spruce Streets with new small pumping stations to reduce the risk of flooding due to siphon plugging.
4. The Town should initiate a project to construct a new forcemain from the Black Ash SPS (existing 500mm diameter section that is currently capped) to the bypass chamber at the Collingwood WWTP. The new forcemain would have a capacity greater than or equal to the Black Ash SPS station capacity. The new forcemain would be located within the existing Harbourview Trail corridor.
5. The Town should initiate projects to upgrade local sewers including existing sewers on Hurontario Street, Mountain Road, Minnesota Street, Huron Street. Improvements on Hurontario, Minnesota and Huron are growth driven. Should the I/I pilot program prove successful, the Town should implement a similar program in the Oak Street and Birch Street areas to reduce peak flows at the Minnesota SPS. These areas were selected based on flow monitoring results.

6. The Town should consider oversizing requirements for built boundary growth as part of any upgrade to the Mountain Road sewer.
7. Should the Town and Clearview Township decide to proceed with servicing of Nottawa through Collingwood, it is recommended that flows from Nottawa be directed to the existing Sixth Line sanitary sewer. To service Nottawa will require the installation of a third pump at the St. Clair SPS.
8. The Town should consider servicing currently unserviced areas within the Town. Five areas were identified including Oliver Crescent, Princeton Shores, West Highway 26, Mountain Road West and Beachwood. These areas all have municipal servicing for water and private servicing for wastewater. For the Oliver Crescent and Princeton Shores, a low pressure STEP system with grinder pumps has been identified. For the Beachwood area, a combination of gravity sewer and low pressure sewer systems has been identified. For the West Highway 26 and Mountain Road West areas, infrastructure has been identified for servicing these areas through the Collingwood WWTP. However, it is recommended that the Town consider servicing requirements needed for servicing of these areas through the ToBM.