2022

Strathroy-Caradoc

SERVICING CAPACITY AND CONTRAINTS STUDY

GROWING TOGETHER
STRATHROY-CARADOC
URBAN OPPORTUNITY – RURAL HOSPITALITY
Comprehensive Planning for Tomorrow

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1 Executive Summary

Water

Using the Municipalities InfoWater Model, WSP modeled the current and planned water network capacity in both Strathroy and Mount Brydges. This was done for three planning horizons including the existing, interim (2036) and full buildout (2046) demand conditions, for the Average Day Demand (ADD), Maximum Day Demand (MDD) and Peak Hour Demand (PHD) scenarios. In the modeling, WSP considered the future known and planned developments while considering the Official Plan reviewed development areas when suggesting network improvements overall.

Based on the simulations completed, WSP made two (2) types of network suggestions: linear upgrades to maintain/improve service throughout the network and supply improvements to increase the overall hydraulic head throughout the system.

Simulations from the Strathroy network assessments demonstrated that the existing system can generally support growth, up to full buildout, with some challenges with maintaining the minimum pressure targets during the full buildout PHD scenario. WSP considered two operating conditions for this scenario: with the PRV between Zone 1 and 2 open and with the PRV closed. With the PRV open; the pressure can be maintained in all scenarios, but the elevated storage volume drops rapidly. During PHD: when the PRV is closed, the pump station must operate at or above firm capacity to supply sufficient head to maintain pressures. As recommendations, an elevated storage should be considered in Zone 1 to increase the water supply volume and hydraulic head across the zone. A new storage would also create an additional source of supply into the system and can be recharged at night. Other methods, outside of a new elevated storage, to increase head include increasing the impeller size of each pump at the pump station or adding additional pumps to increase firm capacity. Alternatively, pump replacement can be considered. Both sets of solutions, adding storage or increasing capacity of the pump station, should be the focused scope of an Environmental Assessment (EA).

Simulations for Mount Brydges yielded similar results to the Strathroy findings. The network can generally support growth, up to full buildout, with some challenges with maintaining the minimum pressure targets during the full buildout PHD scenario. To support growth, WSP recommends that a pump station focus study (EA) be completed in Mount Brydges. The objective would be to increase the pumping capacity of the station – current simulations indicated that during PHD full buildout conditions the existing station has to operate at or above firm capacity to maintain pressures. To increase pumping capacity, we recommend investigating whether or not to include increasing the impeller size of each pump at the pump station or adding additional pumps to increase firm capacity. Alternatively, pump replacement can be considered. It is also recommended that the Official Plan be updated to include a potential requirement to submit fire flow demand applications as part of certain development applications, as identified through the pre-consultation process.

Wastewater Conveyance

A spreadsheet model was developed for the Mount Brydges and Strathroy sanitary sewer system. The capacity analysis was performed on the existing condition scenario and future development scenario. Based on the capacity analysis for Mount Brydges and Strathroy, the analyzed existing sanitary sewer segments have sufficient capacity to service the existing condition and the planned development.

The capacity analysis for Mount Brydges suggests that the Northwest SPS has adequate capacity to handle current flows. The analysis further indicated that the Northwest SPS does not have adequate capacity to handle future flows (2046). However, the actual pump performance via pump test is recommended to determine whether any upgrades are required. Main SPS is identified with sufficient capacity to handle existing and future flows (2046).

The capacity analysis for Strathroy identifies that the Albert St SPS and Metcalfe St SPS have adequate capacity to handle existing flows. Based on the future peak inflows (2046) it seems these pump stations do not have adequate capacity, however the actual pump performance via pump test is recommended to determine whether any upgrades are required.

To support future connections to the wastewater conveyance system, it is recommended that the Municipality's Official Plan include policies to provide guidance related to partial servicing conditions across the Municipality.

Wastewater Treatment

A capacity assessment of the Strathroy Wastewater Treatment Plant (WWTP) and the Mount Brydges Sewage Treatment Plant (STP) was conducted to determine if both plants had sufficient capacity to service the needs to 2046; and to identify any major maintenance needs and rehabilitation strategies. The assessment was conducted in accordance with the MOE Design Guidelines which provides recommendation for process units based on different flow parameters such as average day flow, peak hourly, or peak instantaneous flow. To assess the capacity of each process unit, the equivalent average day flow was calculated using the peaking factors associated with the future flows.

Based on the findings of the capacity assessment for the Strathroy WWTP, the plant is currently at 53% of its rated capacity. All the process components except for the screens in the inlet works have sufficient capacity to meet the future flow. As such, it is required to increase the capacity of the inlet screens such that they are sized to accommodate the future peak flow of 31,678 m^3/d. Additionally, due to the current maintenance challenges experienced with the single aeration lagoon, construction of an additional lagoon to provide redundancy would allow the existing aeration lagoon to be taken offline for cleaning and maintenance. The Municipality should also consider replacing the existing blowers for more energy efficient options.

The capacity review for the Mount Brydges STP, which is at 33% of its rated capacity, has shown that for a future average day flow of 1,059 m^3/d, all the process units have sufficient capacity except the rotating biological contactor (RBC) units. Given that the current RBCs would not provide adequate capacity for the future flow, replacement of the RBCs with an alternative treatment approach was concluded to be a suitable approach. The extended aeration process recommended to replace the RBCs as part of the conceptual design for upgrades to the plant eliminates the need for primary clarification while capitalizing on the reusing the existing RBC tanks. Additionally, given the small capacity of the plant, aerobic digestion could be used for sludge management and stabilization at the plant.

Further, it is recommended that the plant expansion policies of the Municipality's current Official Plan be reviewed to ensure that future system upgrades and expansions in Mount Brydges and Strathroy keep pace with growth and development.

Stormwater

A high-level review of the existing storm sewer system / municipal drains in the built-up urban areas within Strathroy and Mount Brydges has been carried out by setting up a spreadsheet model for the existing storm sewer system. The capacity review of the storm sewers in the study areas are carried out as per Municipality's current Servicing Standards (2021).

The results of capacity analysis indicate that the storm sewers of the Strathroy and Mount Brydges generally do not have adequate capacity to convey the 5-year design storms from the contributing areas due to lower level of servicing standard at the time of installation and/or lack of stormwater quantity controls for the developments since the installation of the storm sewers.

It is recommended that a master drainage plan (MDP) including comprehensive hydrology and hydraulic analysis of the storm drainage system (both minor and major) be carried out to identify the location of urban flooding and to inform the necessity of upgrades on the storm infrastructure.

It the recommended that more stringent stormwater management policies be implemented for the future developments, in part through the Official Plan Review, to minimize the impacts on downstream flooding and overflows of the infrastructure.

2 Water

2.1 Introduction

WSP was retained by the Municipality of Strathroy-Caradoc to examine the water servicing capacity of the Strathroy and Mount Brydges community water distribution system in conjunction with the Official Plan Review. As part of the expected growth in the community, the Municipality of Strathroy-Caradoc is anticipating multiple residential, commercial and industrial developments spread throughout Strathroy and Mount Brydges. The purpose of this study was to update the hydraulic model (InfoWater) and evaluate the system capacity to deliver flow and pressure to existing and planned services while also advising on potential growth areas based on the water system performance. Where applicable, WSP was to recommend water projects (capital infrastructure projects) that would be required to support growth. WSP has also suggested prioritization of these projects for future consideration and implementation based on their immediate or long term need in the respective networks.

This Report summarizes the proposed developments in the near term and the available lands slated for development in the future planning horizon, as well as provide an overview and analysis of the proposed upgrades to the water system network that would be required, based on hydraulic model simulations, to support this growth. These upgrades will be based on the network's capacity with regards to fire flows and pressures and will be recommended based on priority levels and project phasing.

The planning horizons include the review of the current (2021) planning horizon, an interim planning horizon (2036), and the future planning horizon (2046), in which WSP simulated the existing system and demands in addition to the proposed intensified demands and project recommendations. The 2036 planning horizon had demands intensified based on the forecasted developments and their resulting and estimated populations at that time. To satisfy the scope of this analysis, WSP simulated a "Full Buildout" scenario of the year 2046 for which the planning horizon was loaded with demands that reflected the full buildout development pipeline information available at the time of this analysis including the vacant lands slatted for development with their anticipated water demand consumptions. We understand these to reflect the 2046 planning horizon.

Figure 1 and **Figure 2** below displays the existing water network system for Strathroy and Mount Brydges, respectively, with further detailed figures showing the system layout included in **Appendix A** and **Appendix B**.

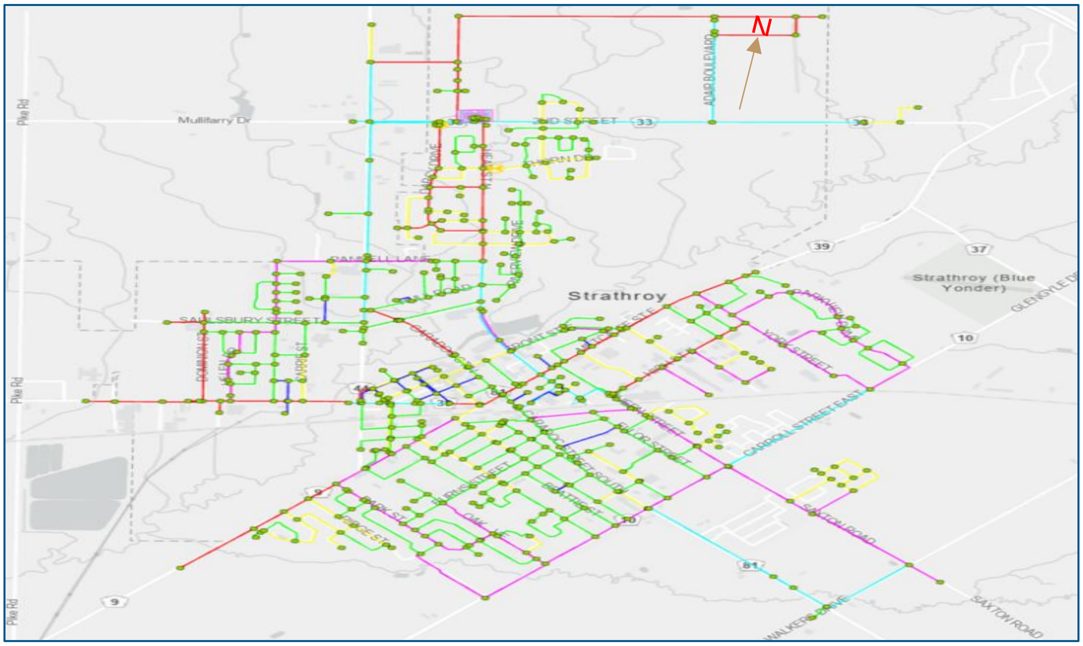

Figure 1: Overview of the Existing Strathroy Watermain Network

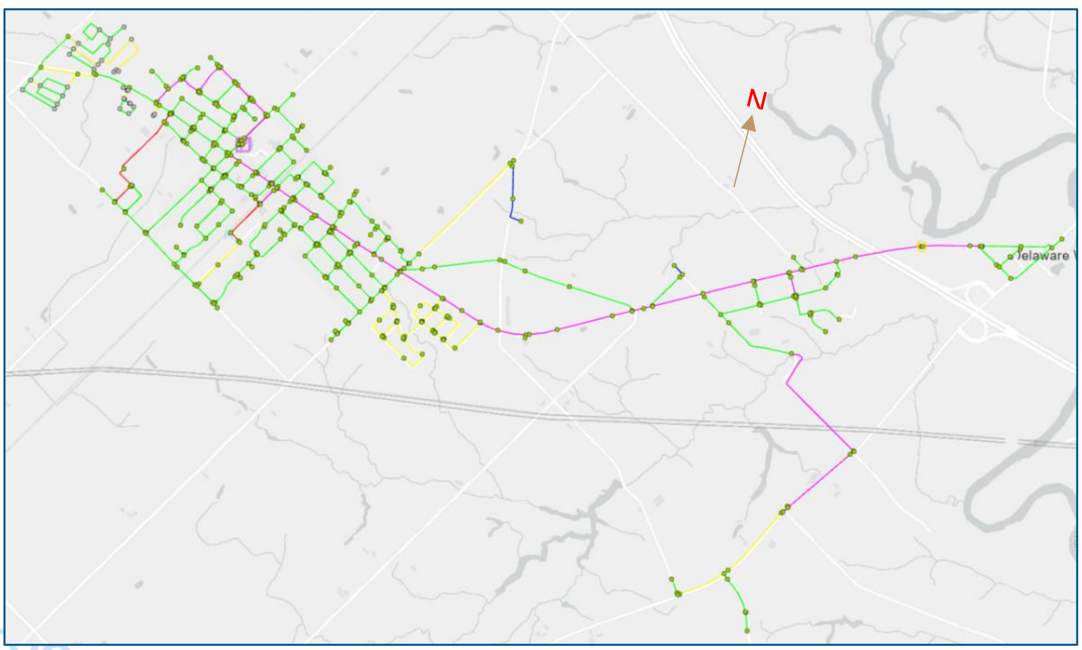

Figure 2: Overview of the Existing Mount Brydges Watermain Network

2.2 Criteria

2.2.1 Domestic Demand

Demands for developments throughout both Strathroy and Mount-Brydges were calculated using the Strathroy-Caradoc Servicing Standards, October 2021. Populations and demands allocation were determined according to the Middlesex County Housing Growth Forecast and Allocations by Local Municipality by Watson & Associates Economists Ltd, January 19, 2021 and Strathroy-Caradoc Residential Land Needs Assessment, August 2020. **Table 1** lists the factors used to determine the demands for the development.

The anticipated developments in Strathroy and Mount Brydges are of two types: proposed developments and vacant residential lands that are designated for potential developments. Residential demands for the proposed developments, with provided site plans, were calculated by accounting for the number of units on the site plan. Commercial demands were calculated by accounting for the site area, modeling these demands in a conservative manner. The calculations of these demands were then allocated to the closest node in the water model.

The water modeling study began by updating the hydraulic model's demand to account for future approved and planned developments and lands that are zoned for future development – this was defined as the interim 2036 planning horizon. Following this, WSP added expected demands for the vacant lands that make up the remaining development areas in both system – this made up the 2046 planning horizon (full buildout). In doing this, WSP relied on development pipeline information from the Municipality, available site plans, planning maps and the "Middlesex County Housing Allocations Memo". Applicable maps used as a basis for demand updates have been appended to this report (Volume 1).

Table 1: Demand Factors and Inputs

DEMAND FACTORS AND INPUTS	VALUE
Low Density Residential	2.4 PPU
Medium Density Residential	2.4 PPU
High Density Residential	1.6 PPU
Average Day Demand	250 L/Person/day
Maximum Day Peaking Factor	3.5
Peak Hour Factor	7.8

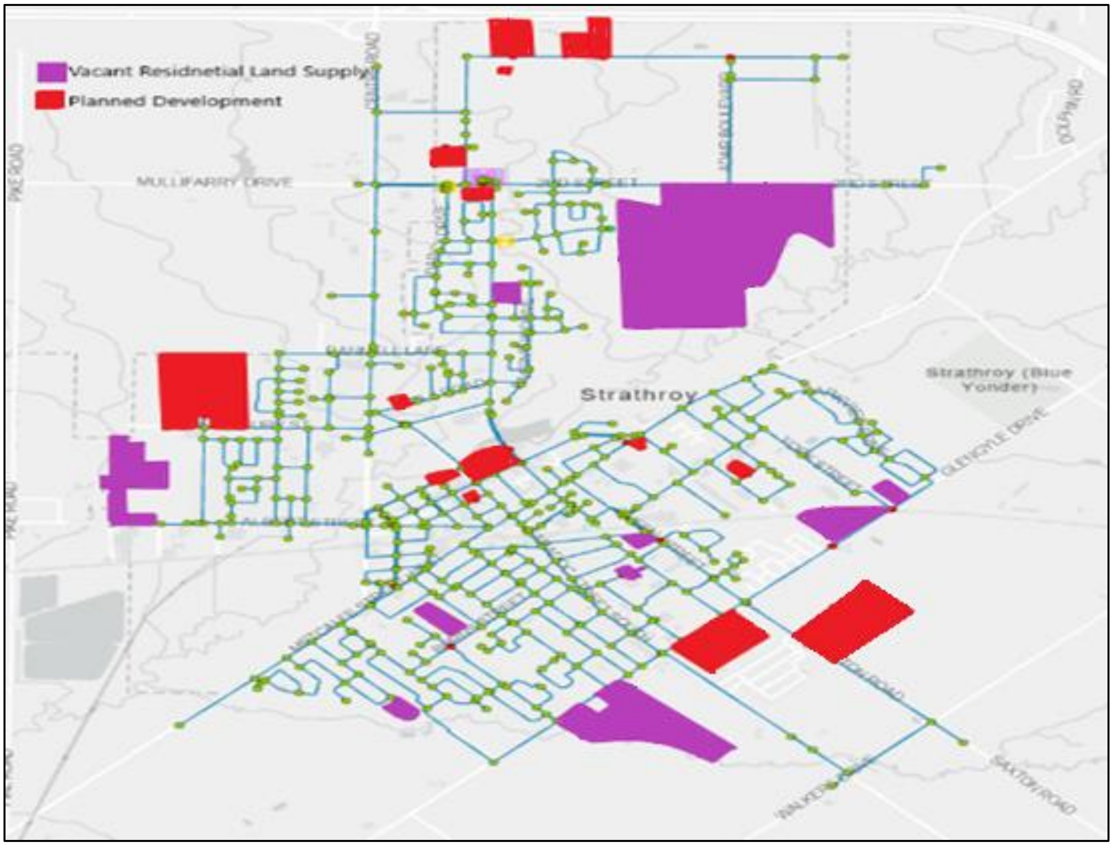

Figure 3: Land Areas for Developments in Strathroy

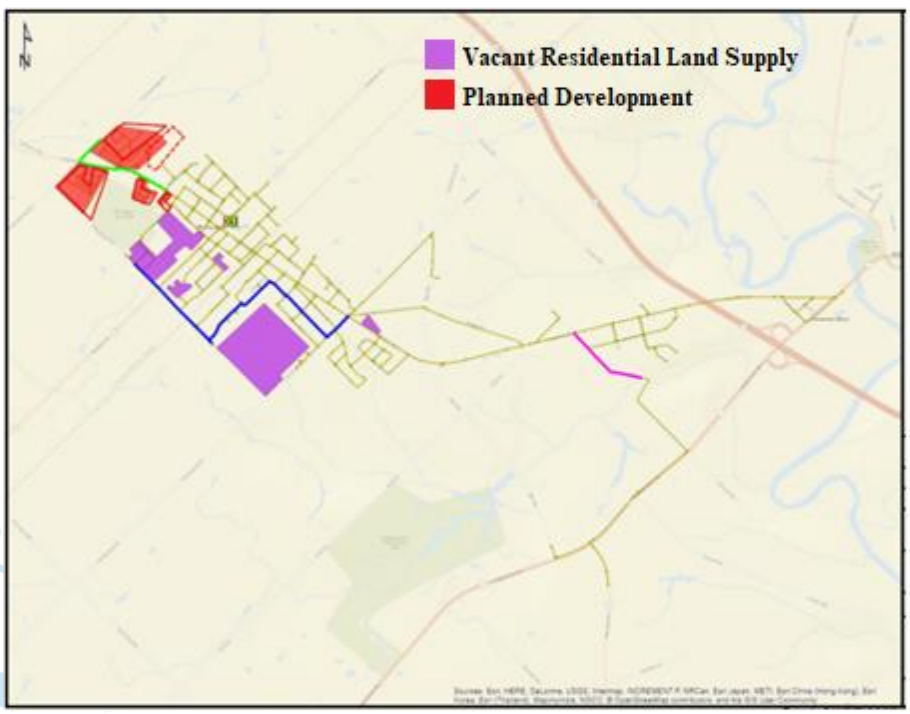

Figure 4: Land Areas for Developments in Mount Brydges

For the vacant residential lands that are expected to be developed in the next 25 years; demands were calculated by first determining the growth in low, medium and high-density housing units from 2021 to 2036 using the Strathroy-Caradoc housing projection found in the *Middlesex County Housing Allocations Memo*. Using the high scenario projection, to be conservative, the number of housing units were divided between the Strathroy and Mount Brydges communities based on the historic 70/30 split mentioned in the *Strathroy-Caradoc Residential Land Needs Assessment*. Following this, the number and type of housing units were distributed throughout the vacant residential lands proportionally to the amount of net developable area in 25 years for each land area, as well as the type of housing units for which the area was zoned. Domestic demands were then calculated from this housing unit distribution and allocated to the closest node in the water model. Detailed calculations for all demands are shown in **Appendix A** and **Appendix B**.

The InfoWater model used had built in peaking factors for the Maximum Day Demand (MDD) and Peak Hour Demand (PHD) scenarios. That is, WSP updated the Average Day Demand (ADD) demands and the built-in peaking factors peaked demands during the MDD and PHD simulations. At the time of receipt, the Strathroy model had peaking factors of 1.9 and 2.85 (2.7 in 2036) built in for the MDD and PHD scenarios and the Mount Brydges model had peaking factors of 3.0 and 4.5 built in for the MDD and PHD scenarios, which differ from the peaking factors in the previously quoted design guidelines. For this analysis, WSP used the peaking factors set in the standards in order to be conservative.

In the model that WSP received and used for the modeling assignment, the total demands for existing services were divided and loaded equally throughout the model. This can be significant since the simulated hydraulic grade line throughout the system may not drop or "dip" the same as it does in the system.

Table 2 is a summary of the demands considered in this study. The *"Demand @ Time of Receipt"* columns reflect the water demands in the model at the time that WSP received it, the following *"Demands Calculated"* reflect those calculated demands for the future development areas, which were added in the model incrementally increasing the total water loading to what is shown in the *"Total Demands (Sum Existing + Proposed)"*. It is to be noted that the demands for the existing and ultimate buildout demands at time of receipt are calculated with the peaking factors found in the standards, and not the pre-set peaking factors in the model at the time of receipt.

Table 2: Summary Table of Demands

	2021 DEMAND @ TIME OF RECEIPT (L/S)			2036 DEMAND @ TIME OF RECEIPT (L/S)			2021 DEMANDS CALCULATED (L/S)			2046 DEMANDS CALCULATED (L/S)			2021 TOTAL DEMANDS (SUM EXISTING + PROPOSED) (L/S)			2046 TOTAL DEMANDS (SUM EXISTING + PROPOSED) (L/S)		
Demand Scenarios	ADD	MDD	PHD	ADD	MDD	PHD	ADD	MDD	PHD	ADD	MDD	PHD	ADD	MDD	PHD	ADD	MDD	PHD
Strathroy	54	190	424	54	190	424	0	0	0	23	80	178	54	190	424	77	270	602
Mount Brydges	8	29	65	8	29	65	4	13	30	8	28	62	12	43	95	16	57	127

Note: Demands for 2046 conditions were generated by applying residential unit growth projected for 2046 to the vacant residential land supply areas.

2.2.2 System Pressures

As outlined in the Municipality of Strathroy-Caradoc's Servicing Standards (2021), the acceptable pressures under normal conditions are between 275 kPa (40 psi) and 700 kPa (100 psi). The minimum allowable pressure under Maximum Day Demand plus Fire Flow is 140 kPa (20 psi) at the location of the fire and everywhere else in the water system.

2.2.3 Fire Flow Demand

The required fire flows (RFF) used in the model were assumed based on the procedure outlined in the Ontario Building Code Compendium (OBC) 2012. The guideline was used to estimate the general minimum fire flow requirement for the various building types in Strathroy.

Following the historic application of the OBC method on a wide variety of buildings, the general RFFs are assumed to be 45 L/s for residential single homes, 150 L/s for residential multi, and 100-150 L/s for industrial, commercial and institutional (ICI) buildings. These RFFs are generally applied on a large-scale analysis such as this one.

WSP recommends that when development re-zoning or site plan applications are submitted to the Municipality, individual fire flow calculations for specific building types or land use areas should be completed. These RFF calculations should be provided by developers at the time of site plan and Form 1 applications.

2.3 Hydraulic Model

WSP used a Strathroy-Wide InfoWaters model obtained through the Municipality of Strathroy-Caradoc. This model has water network operating at boundary conditions pre-set by the Municipality. The water network is serviced by one (1) booster pumping station, located on the intersection of Head Street North and Second Street, and is supplied by a fixed head reservoir. The network is also supplied by an elevated water tank on the Head Street South and Tanton Street intersection.

2.3.1 Strathroy Boundary Conditions

The Strathroy community water network is made up of two (2) pressure districts, both of which are supplied by one (1) booster pumping stations, which is supplied by a fixed head reservoir with a hydraulic grade line (HGL) set to 239.40m. The analysis was carried out with this HGL for all existing and ultimate buildout scenarios for the Strathroy community. The two pressure districts correspond to Zone 1, located south of the booster pumping station where lower elevations are present, and Zone 2, situated north of the pumping station and on higher elevated ground.

The status of each pump during all modeled scenarios of this analysis is outlined in **Table 3**.

Table 3: Pump Status at the Strathroy pumping stations for 2021 and 2036 scenarios

	Pump	2021 AVG	2021 MDD	2021 PHD	2021 MDD+FF	2036 AVG	2036 MDD	2036 PHD	2036 MDD+FF
Pump Station	P1	OFF	OFF	OFF	OFF	OFF	OFF	OFF	OFF
	P2	ON	ON	ON	OFF	ON	ON	OFF	OFF
	P3	OFF	OFF	OFF	ON	OFF	OFF	ON	ON
	P4	OFF	OFF	OFF	OFF	OFF	OFF	OFF	OFF
	P5	ON	ON	ON	OFF	ON	ON	OFF	OFF

In this pump station, Pump 3 is the largest pump. Firm capacity of this station if with Pump 3 “OFF” – this is the case that was simulated for all, but one (1), domestic demand planning horizon. More on this planning horizon in the result section. During the fire flow simulation, WSP considered Pump 3 as the “fire pump” and simulated fire flows with this pump “ON” understanding this to be an “emergency condition” simulation.

For the basis of this analysis, fire flows and pipe headloss were used as trigger points for upgrade and performance evaluations. Junctions with fire flows that were deemed to be below required fire flows based on land use were marked for upgrade recommendations, along with pipes with headloss greater than 2m.

2.3.2 Mount Brydges Boundary Conditions

The Mount Brydges community water network is made up of one (1) pressure district that is supplied by one (1) booster pumping stations, which is supplied by a fixed head reservoir with a hydraulic grade line (HGL) set to 250.10m. The analysis was carried out with this HGL for all existing and ultimate buildout scenarios Watermain analysis for the Mount Brydges water network is conducted under both planning horizons correspondingly. The status of each pump during all modeled scenarios of this analysis is outlined in **Table 4**.

Table 4: Pump Status at the Mount Brydges pumping stations for 2021 and 2036 scenarios

	Pump	2021 AVG	2021 MDD	2021 PHD	2021 MDD+FF	2036 AVG	2036 MDD	2036 PHD	2036 MDD+FF
Pump Station	P1	OFF	OFF	OFF	ON	OFF	OFF	OFF	ON
	P2	OFF	OFF	OFF	ON	OFF	OFF	OFF	ON
	P3	ON	ON	ON	ON	ON	ON	ON	ON
	P4	ON	ON	ON	OFF	ON	ON	ON	OFF

In this pump station, Pump 1 is the largest pump with the highest static head. Firm capacity of this station if with Pump 1“OFF” – this is the case that was simulated for all but domestic demand planning horizon. During the fire flow simulation, WSP considered Pump 1 as the “fire pump” and simulated fire flows with this pump “ON” understanding this to be an “emergency condition” simulation.

For the basis of this analysis, fire flows and pipe headloss were used as trigger points for upgrade and performance evaluations. Fire flows that were deemed to be below required fire flows based on land use, as well as pipes with headloss greater than 2m were highlighted for upgrade recommendations.

2.3.3 Strathroy Community

The proposed municipal watermains were added to the Strathroy and Mount Brydges model along the street layout of the proposed development based on the overall servicing plan. Friction factors for all new pipes added to the model were assigned according to the Ministry of the Environment, Conservation and Parks (MECP) watermain Design Criteria as listed in **Table** 5.

Table 5: Hazen-Williams C-Factors

DIAMETER – NOMINAL	C-FACTOR
150mm	100
200mm to 250mm	110
300mm to 600mm	120

The proposed layout of the water distribution system is intended to satisfy the requirements of the Municipality of Strathroy-Caradoc. All pipes and nodes added for the development are shown and identified in **Appendix A** and **Appendix B**.

2.3.4 Model Validation

R.V. Anderson Associates Ltd. conducted hydrant flow tests in Strathroy-Caradoc, with the first 10 tests conducted in Strathroy and the tests #11 to #17 taken in Mount Brydges. All hydrant test sheets for Strathroy are attached in **Appendix G** and **Appendix H.**

The calibration of the Strathroy and Mount Brydges models were verified using results of the flow tests provided by the Municipality of Strathroy-Caradoc. The test was performed on and at:

Table 6: Summary of Hydrant Test Results in Strathroy

HYDRANT TEST #	DATE & TIME	STREET NAME	TEST HYDRANT #	TEST STATIC PRESSURE (KPA)	MODEL STATIC PRESSURE (KPA)	TEST FLOW AT 140 KPA (L/S)	MODEL FLOW AT 140 KPA (L/S)
1	2019/07/09 @ 9:25 AM	Adelaide Rd.	454	386	404	282	280
2	2019/07/09 @ 9:32 AM	Moffatt Ln.	287	310	325	112	110
3[1]	2019/07/09 @ 9:42 AM	Park St.	327	372	355	299	230
4	2019/07/09 @ 10:00 AM	Colborne St.	327	441	450	665	250
5	2019/07/09 @ 10:10 AM	Egerton St.	282	414	425	285	220
6	2019/07/09 @ 10:20 AM	Helen Dr.	120	441	446	264	300
7	2019/07/09 @ 10:35 AM	Acton St.	662	359	378	298	285
8	2019/07/17 @ 9:17 AM	Deborah Dr.	349	469	477	93	95
9[1]	2019/07/17 @ 9:55 AM	Wright St.	349	421	433	84	100
10	2019/07/17 @ 9:55 AM	Wright St.	593	414	420	74	99

Note 1 – The test hydrants listed on tests #3 and #9 were not found on their respective streets and are assumed to have been an error.

Table 7: Summary of Hydrant Test Results in Mount Brydges

HYDRANT TEST #	DATE & TIME	STREET NAME	TEST HYDRANT #	TEST STATIC PRESSURE (KPA)	MODEL STATIC PRESSURE (KPA)	TEST FLOW AT 140 KPA (L/S)	MODEL FLOW AT 140 KPA (L/S)
11	2019/07/17 @ 10:59 AM	Emerson St.	53	359	422	120	200
12	2019/07/17 @ 11:05 AM	King St.	86	345	383	113	110
13	2019/07/17 @ 11:17 AM	Bentim Rd.	28	359	384	112	105
14	2019/07/24 @ 8:43 AM	Bennet Cr.	146	400	425	110	130
15	2019/07/24 @ 8:54 AM	Clark St.	10	517	549	75	74
16	2019/07/24 @ 9:04 AM	Birmingham St.	113	483	515	69	66
17	2019/07/24 @ 9:18 AM	Longwoods Rd.	125	483	499	43	40

WSP used the hydrant flow tests to validate the InfoWaters model but did not conduct any model calibrations using these tests. The hydrant flow test results and the model simulated hydrant flow curves were compared at the location listed using the existing PHD scenario given the time of year and time of day of the tests. It was found that the modeled static pressures and modeled flow at 140kPa (20 psi) were reasonable relative to the hydrant flow tests static pressures. Accordingly, WSP model results are deemed to be practical but may be subject to update based on further investigation of model calibration, which falls outside the scope of this study.

The results of these tests and how they compare to the model simulated results can be seen in **Appendix G** and **Appendix H.**

From the resulting Strathroy tests, it was found that the modeled static pressures for the tests were higher than the tested static pressures. As shown in **Appendix G**, the simulated flow curves generally follow the same curvature or behaviour as the tested flow curves, but show higher flow and pressure, while the simulated curves intersect the tested flow curves for the majority of the tests, with a portion intersecting before the tested residual point and dropping at a steeper rate than the tested flow curve.

The Mount Brydges hydrant flow test results and the model simulated hydrant flow curves were compared at the location of the test hydrants. We note that all test provided were located south of Lions Park Drive. As there was no pump station flow or reservoir information provided to match the hydrant tests, the model was simulated to the assumed PHD scenario given the time of year and day of the test. From this, it was found that the modeled static pressures for both tests were higher than the tested static pressures. Similarly, the simulated fire flow at hydrant #53 is higher than the theoretical fire flow from the hydrant flow test while the simulated fire flow at hydrant #86 is conservative relative to the theoretical fire flow from the test. As shown in **Appendix H**, the simulated flow curve at hydrant #53 generally follows the same curvature or behavior as the simulated flow curve, but shows higher flow and pressure, while the simulated flow curve at hydrant #86 intersects the tested flow curve before the tested residual point and drops at a steeper rate than the tested flow curve.

The model's calibration relative to the test hydrant flow curves can be explained by:

- The C-Factors in the model, for existing pipes, may be smoother than the actual pipes. Many of the existing 150mm watermains in the model have C-Factors of 120 or more while most main larger than 200mm have C-Factors of 140. C-Factor testing can be completed to validate these values;
- As discussed in section 2.1, the demand allocation may result in less local drops of the hydraulic grade line. Given that the flow curves were completed in residential neighborhoods, the approach taken to load the model may not generate a large enough drop in static pressures;
- The source of water in the model is a fixed head reservoir set to 239.4m of head. This may be higher than the actual source of head. Pressure monitoring can be completed to validate this; and,
- A lack of inputted minor loss information in the model that would contribute to lower static pressures in the hydrant testing.

WSP recommends that more recent testing be done and that a calibration exercise be completed ahead of possible future EAs or as part of a Master Plan. As they are currently assembled, the models over predicts static pressures by 1% to 5% in Strathroy and by 3% to 15% in Mount Brydges. This means we expect the modeled results presented in this analysis to be high relative to the actual pressures that may be currently experienced in the system.

2.4 Analysis

The proposed municipal watermains within Strathroy and Mount Brydges were sized to satisfy the greater of either Peak Hour or Maximum Day plus Fire Flow demands. Modeling was carried out for Average Day, Maximum Day, Maximum Day plus Fire Flow and Peak Hour demand conditions for the existing and proposed watermain network under the existing and future planning horizons using the InfoWaters models of the Strathroy and Mount Brydges networks respectively, as described in Section 3.

Projects considered in this study were suggested to support the full buildout 2046 demand conditions. They should be implemented as opportunities come up and validated prior to constructions. Validating these projects should be done in a Master Plan and coordinated with the Transportation Master Plan, where possible, to align capital projects.

2.4.1 Strathroy Baseline Condition Results

Modeled service pressures in Strathroy for the existing network are summarized in **Table 8**. Detail pipe and node results tables are found in **Appendix C**.

The modeling indicates that under the existing watermain network, the expected service pressures range between 291 kPa and 521 kPa for the existing planning horizon and between 274 kPa to 521 kPa for the future planning horizon.

Table 8: Simulated Pressure Range Under Existing Strathroy Watermain Network

DEMAND SCENARIOS	AVERAGE DAY (KPA)	MAXIMUM DAY (KPA)[1]	PEAK HOUR* (KPA)
2021	332-521	320-521	291-498
2036	330-521	286-468	274-533

*Note: The Peak Hour scenario was simulated with the Zone 1 to 2 zone boundary PRV open

During the Ultimate/Future buildout (2046) Peak Hour (PHD) demand scenario, junction *J54* falls under the minimum pressure by 1 kPa, and without it the scenario would simulate a pressure range between 276 kPa to 533 kPa. Junction 54 is connected to a trunk watermain and located near the pressure reducing valve found near Strathroy's booster pumping station, detailed in a figure in **Appendix C**.

2.4.2 Recommended Strathroy Projects

In support of the increasing growth expected for the Strathroy community, upgrades and extensions to the existing water distribution network will help in satisfying the necessary fire flows and pressures in order to sustain the increasing demands on the network in the next 15 years and foreseeable future.

The following section presents projects that can be considered to help support growth. These are presented based on their priority and phasing, which will be subject to the timing of the development projects across Strathroy. That being said, projects should be considered on an opportunity basis. As road reconstruction projects are expected, the Municipality should consider reviewing the size of watermains.

Projects presented below should be considered, evaluated and validated by a formal Water Master Plan and coordinated with the Transportation Master Plan to identify opportunities for upgrades. Once validated, these projects can be considered and included in future Development Charges (DC) studies based on needs and budget considerations.

Priority 1

Priority 1 encompasses the watermain extensions that parallel the road extensions recommended in the preferred roads improvement scenario from the Transportation Master Plan. The proposed extensions of Pannell Lane, Dominion Street, and Jenna Drive are in anticipation of the planned developments in the area. Similar to the expected growth in traffic demand, an increase in water demands from these developments requires an expansion in water servicing, and thus parallel extensions of 300mm watermains are prioritized in the upgrade of the water network.

As part of Priority 1, WSP also recommends that further pump station studies be completed. The simulations herein demonstrate that the largest pump (Pump 3) may be required to service the 2036 Peak Hour demand condition in the future. These same findings would apply to the “Future Buildout” condition. As part of this, WSP recommends that investigations be completed to increase Pump 5 to be the same size or larger than Pump 3. This would allow for firm capacity to include Pump 3 in normal operations and service the projected 2036 and Full Buildout PHD conditions. These conditions and pump evaluation are discussed further in section 2.5.

Figure 5: Scope of Work for Strathroy Priority 1

Project No.	Street	Proposed Solution
Priority-1-1	Pannell Lane	Extension of 300mm watermain for the approximate length of 500m from Drury Lane west bound. The stop location is aligned with Dominion St.
Priority-1-2	Dominion Street	Extension of 300mm watermain for the approximate length of 500m from the end of the Dominion St main to the suggested Pannell Lane extension (Priority 1-1).

Priority 2

Priority 2 focuses on the upgrade of watermains in order to prevent bottlenecking in the core of Strathroy. These short watermain segment upgrades to 250mm (or closest commercially available size without going below 250mm) on Pannell Lane, Maple Street, and Metcalfe Street West were proposed as they will match the size of the remaining watermain pipes on these roads. Implementing these upgrades ensures the continuation of watermain sizes in order to provide improved, uninterrupted fire flow capacity in the core of the community.

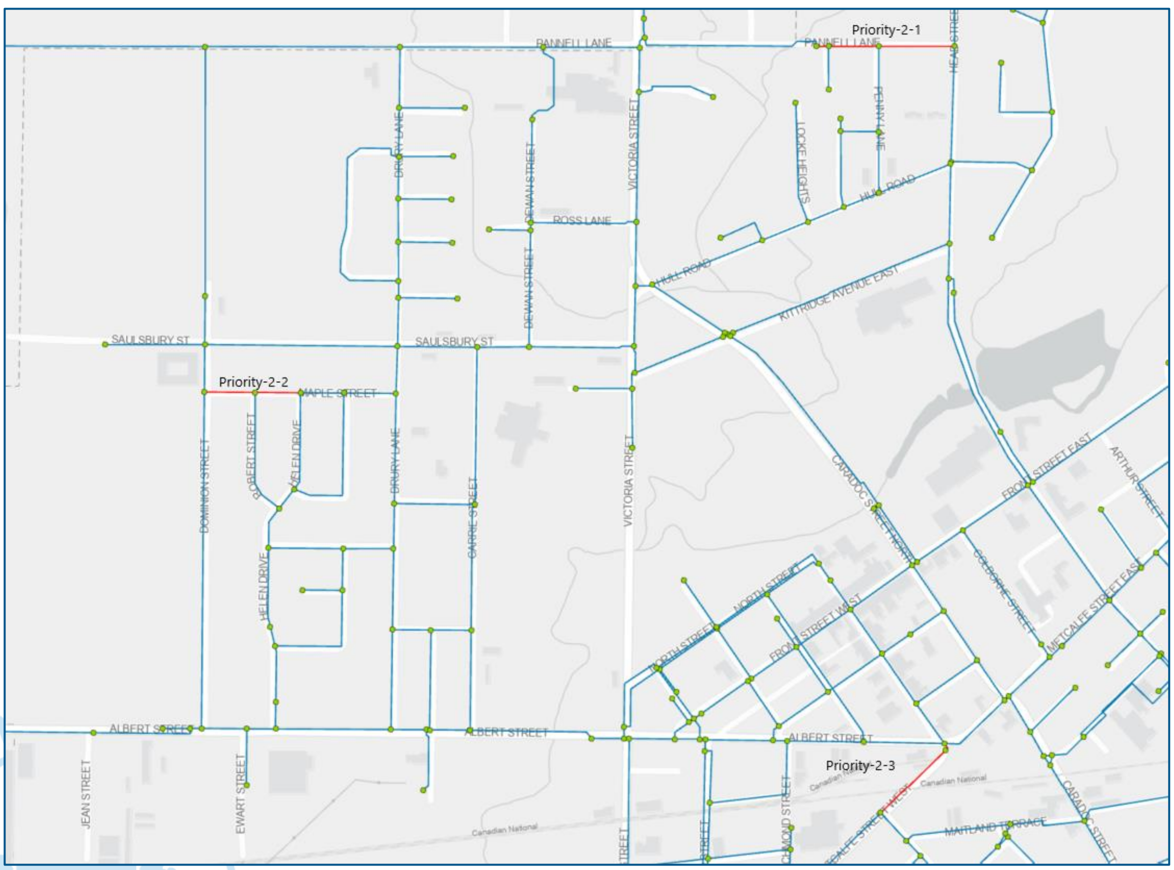

Figure 6: Scope of Work for Strathroy Priority 2

Project No.	Street	Proposed Solution
Priority-2-1	Pannell Lane	Upgrade watermain from 150mm to 250mm for the length of 277m from Head Street North to Martin Crescent.
Priority-2-2	Maple Street	Upgrade watermain from 150mm to 250mm for the length of 192m from Helen Drive to Dominion Street.
Priority-2-3	Metcalfe Street West	Upgrade watermain from 200mm to 250mm for the length of 199m from Adelaide Street to Frank Street.

Priority 3

Priority 3 is based on facilitating fire flows that would be adequate for commercial areas in Strathroy. The upgrades prioritized in this phase covers commercial areas that have available fire flows of 100 L/s in the existing watermain network. The watermain upgrades on Centre Street West, Canaan Street, Head Street South, and Ellen Street, as well as the upgrade and extension on McVicar Street target these areas to ensure the required fire flow for commercial buildings are met.

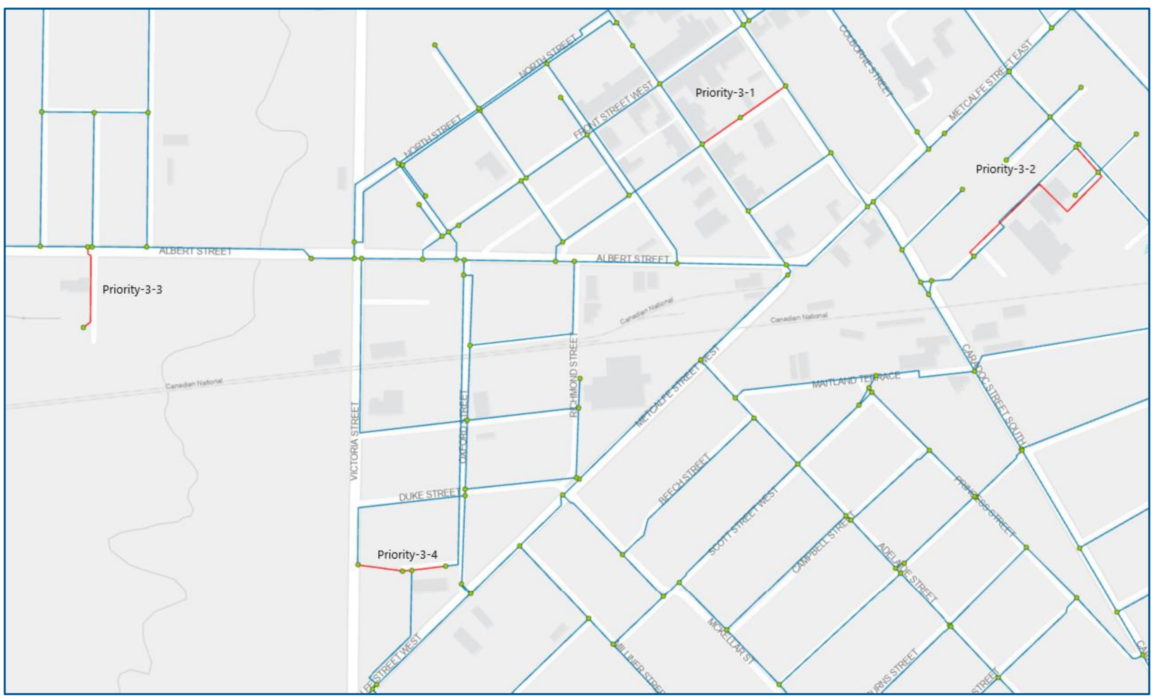

Figure 7: Scope of Work for Strathroy Priority 3

Project No.	Street	Proposed Solution
Priority-3-1	Centre Street West	Upgrade watermain from 100mm to 150mm for the length of 151m from Caradoc Street North to Frank Street.
Priority-3-2	Canaan Street, Head Street South	Upgrade service main on Canaan Street from 100mm to 150mm for the length of 292m from Dennis Street to Scott Street. Upgrade watermain on Head Street South from 100mm to 150mm for the length of 50m from Scott Street to Canaan Street. Both these projects are opportunistic options if streets are to be re-done. The purpose is to increase fire flow at these locations.
Priority-3-3	Ellen Street	Upgrade watermain from 100mm to 200mm for the length of 128m from Albert Street to the end of the watermain. This project is an opportunistic option if the street is to be re-done. The purpose is to increase fire flow at this location for the existing building.
Priority-3-4	McVicar Street	Upgrade and extend watermain from 100mm to 150mm for the length of 132m from Oxford Street to Victoria Street.

Priority 4

Priority 4 focuses on addressing residential areas serviced by fire flows under 45 L/s. In this priority phase, watermain segments across Strathroy that limit residential areas are upsized in order to provide available fire flows that would satisfy the residential single homes in these areas. These segments, along Riverview Drive, Hull Road, Victoria Street, North Street, Emily Street, Mill Lane, Thomas Street, Concord Street, and Oak Avenue, are hydraulically possible as these upsized watermains will match or be smaller in size than the feeder watermains. WSP recommends a flushing analysis to consider water quality at dead-ended mains.

Figure 8: Scope of Work for Strathroy Priority 4

Project No.	Street	Proposed Solution
Priority-4-1	Riverview Drive	Upgrade watermain from 150mm to 200mm for the length of 277m from Deruiter Drive to Joel Court.
Priority-4-3	Emily Street	Upgrade watermain from 100mm to 150mm for the length of 58m along Emily Street.
Priority-4-4	Mill Lane	Upgrade watermain from 100mm to 150mm for the length of 141m from Metcalfe Street East to the end of the watermain. This project is an opportunistic option if the street is to be re-done. The purpose is to increase fire flow at this location for the existing building.
Priority-4-5	Thomas Street	Upgrade watermain from 100mm to 150mm for the length of 68m from Front Street West to the end of the watermain. This project is an opportunistic option if the street is to be re-done. The purpose is to increase fire flow at this location for the existing building.

Project No.	Street	Proposed Solution
Priority-4-6	Concord Street	Upgrade watermain from 100mm to 150mm for the length of 127m from Caradoc Street South to the end of the watermain. This project is an opportunistic option if the street is to be re-done. The purpose is to increase fire flow at this location for the existing building.
Priority-4-7	Oak Avenue	Upgrade watermain from 150mm to 200mm for the length of 308m from Burns Street to the end of the watermain.

2.4.3 Strathroy Proposed Condition Results

2.4.3.1 Simulated Pressure Results

For the Strathroy watermain network that incorporates the priority upgrade recommendations, the service pressures expected during the existing planning horizon range between 294 kPa to 521 kPa and, while it ranges from 274 kPa and 521 kPa for the future planning horizon.

Table 9: Simulated Pressure Range Under Strathroy Watermain Network with Priority Upgrades

DEMAND SCENARIOS	AVERAGE DAY (KPA)	MAXIMUM DAY (KPA)	PEAK HOUR (KPA)
2021	333-521	320-521	294-498
2036	330-521	281-467	274-513[1]

Note 1: The Peak Hour Demand scenarios require elevated storage tank water level to be at 70%

As mentioned in the discussion of the pressure results for the existing system, junction *J54* falls under the minimum pressure by 1 kPa. This junction is connected to a trunk watermain and located near the pressure reducing valve found near Strathroy's booster pumping station, and the system would have a minimum pressure of 276 kPa when this junction is not considered. A detailed figure of this junction can be found in **Appendix C**.

Further PHD simulations were run in anticipation for Ultimate/Full buildout conditions, with demands estimated for population growth up until 2046. Although running with the current boundary conditions (one pump ON) led to junctions in the north end falling under 275 kPa, simulating with pumps 3 and 5 ON resulted in a pressure range of 275 to 512 kPa, with the exception of junctions *J54* which was previously mentioned to fall under the minimum pressure by 1 kPa and junction *J956*. The latter junction, which is located on the intersection of 200mm watermains along Collins Way and Foxen Street in the southwest part of Strathroy, falls under the minimum pressure by 2 kPa. A detailed figure of this junction can be found in **Appendix C**

2.4.3.2 Fire Flow Results

The minimum allowable pressure under Maximum Day Demand plus Fire Flow is 20 psi (140 kPa) at the location of the fire or anywhere else in the pressure district. As part of the fire flow analysis, WSP simulated the available fire flow (AFF) using the automated fire flow solver built in the InfoWaters software. The fire flow scenarios were simulated under Maximum Day Demand conditions for the existing and future planning horizons for both the existing and proposed watermain networks.

The hydraulic water model provided by the Municipality set Pump 3 as the pump designated for fire flow scenarios. As a result, the boundary conditions that simulated a comprehensive fire flow summary was Pump 3 ON, with the elevated storage levels set to 70%.

Under these conditions, the existing water system can satisfy the majority of nodes in providing adequate fire flows in both the 2021 and 2036 planning scenarios. However, there remains a portion of nodes servicing residential and commercial buildings that fall under the required fire flows in the existing watermain infrastructure in Strathroy. The upgrades recommended for the proposed system target these areas, ensuring the achievability of required fire flows for all nodes in the system.

Simulations for ultimate buildout were also conducted by estimating demands from an increase in population up until 2046. Under this scenario, and with Pump 3 ON and elevated storage levels to 70%, the proposed system will be able to also satisfy the majority of the required fire flows for all nodes in the system.

2.4.4 Mount Brydges Baseline Condition Results

Modeled service pressures for the existing Mount Brydges system are summarized in **Table 10**. Detail pipe and node results tables are found in **Appendix D**.

The modeling indicates that under the existing watermain network, the expected service pressures range between 207 kPa and 675 kPa for the existing planning horizon and between 191 kPa to 673 kPa for the future planning horizon.

Table 10: Simulated Pressure Range Under Existing Mount Brydges Watermain Network

DEMAND SCENARIOS	AVERAGE DAY (KPA)	MAXIMUM DAY (KPA)[1]	PEAK HOUR (KPA)
2021	363-675	363-664	207-642
2036	363-673	360-657	191-626

Note 1: The Peak Hour Demand scenarios require firm capacity of Pumps 2, 3 and 4 ON to run, as InfoWater Pro software indicated Pump 3 and 4 ON is inadequate to service the system

During the Ultimate/Future buildout (2046) Peak Hour (PHD) demand scenario, junctions *J252, V123, V253, J-292, V141 and J408* fall under the minimum pressure ranging between 165 – 275kPa

Given that the model calibration identified that the simulated pressures are higher than the tested static pressures, WSP expect that the anticipated pressures shown in the tables above are also higher than what can be expected.

2.4.5 Recommended Mount Brydges Projects

In support of the increasing growth expected for the Mount Brydges community, upgrades and extensions to the existing water distribution network will help in satisfying the necessary fire flows and pressures in order to sustain the increasing demands on the network in the next 25 years. The following section will break down the recommendations based on their priority and phasing, which will be subject to the timing of the development projects across Mount Brydges.

The following section presents projects that can be considered to help support growth. These are presented based on their priority and phasing, which will be subject to the timing of the development projects across Mount Brydges. Projects should also be considered on an opportunity basis – if/when road reconstruction are expected, consider reviewing the size of watermains. Projects presented below should be considered, evaluated, and validated by a formal Water Master Plan and coordinated with the Transportation Master Plan to identify opportunities for upgrades. Once validated, these projects can be considered and included in future Development Charges (DC) studies based on needs and budget considerations.

Priority 1

Priority 1 encompasses the recommended upgrades in the north end of Mount Brydges, namely upsizing the Adelaide Road watermain from Woods Edge Road to Falconbridge Drive, as well as a 300mm watermain extension on Falconbridge Drive. The objective is to allow for the larger main to extend to the north-end of Mount Brydges water network to service the new developments, allowing for stronger and more robust servicing strategies for these new developments, and also provide support for any future developments that may be considered at the north end of the network. With the increased watermain size, we effectively reduce the total head-loss along the Adelaide Road watermain which allows for a higher static pressure at existing and proposed services. It also provides a larger fire flow capacity for the future developments. The 300mm main along Adelaide Road, as shown in **Figure 9**, is hydraulically possible given the combination of the

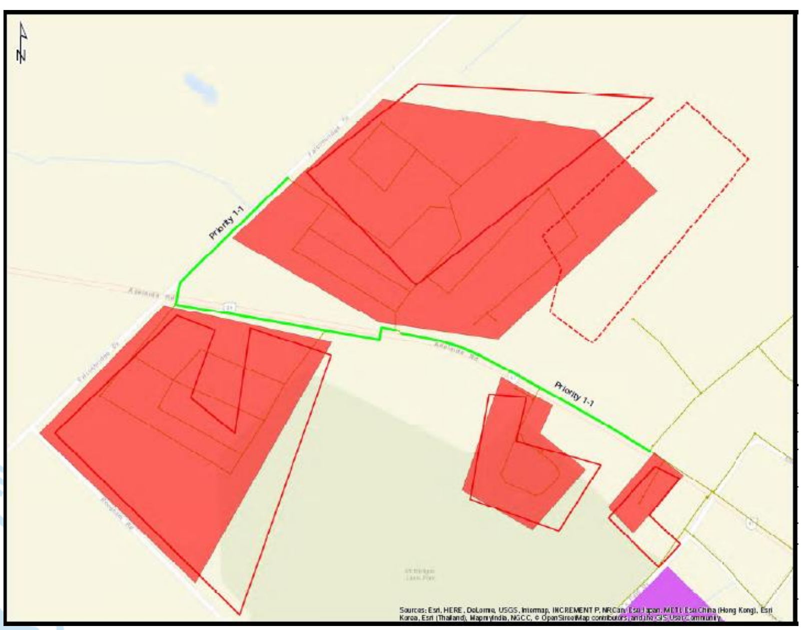

Figure 9: Scope of Work for Mount Brydges Priority 1. Red outline and fill shows development lands considered.

existing 150mm main along Adelaide Road (south of Woods Edge Road) and the 250mm watermain along Woods Edge Road.

Figure 10: Scope of Work for Mount Brydges Priority 1

Project No.	Street	Proposed Solution
Priority-1-1	Adelaide Road	Upgrade watermain from 150mm and 200mm to 300mm for the length of 1142.71m from Woods Edge Road to the end of Falconbridge Drive at Adelaide Road

Priority 2

Priority 2 speaks to the recommended upgrades suggested for the watermain along Gibson Road, stretching from the road's intersection with Adelaide Road to approximately 768m south of the connection point. This watermain, located in the south of the Mount Brydges community, would be upsized from the existing 150mm to a 250mm watermain. The purpose of this upgrade would be to remove the bottleneck by matching the size of watermain on Adelaide Road and downstream of Gibson Road, and thus improving fire flows capacity in the area.

Priority 2 also includes a watermain upgrade along Glendon Drive between Adelaide Road and Veterans Drive. This upgrade involves upsizing the current 150mm diameter pipe into a 250mm pipe in order to connect the 250mm backbone watermain along Adelaide Road to the 250mm watermain that goes along Glendon Drive, Bond Street, Radisson Lane, Lucas Avenue and Woods Edge Road, which ultimately connects into the upgraded watermains in Priority 1. This watermain upgrade is proposed to support a large water used (Greenhouse) and the expected impact on water age is not significant. Note however water age was not part of the WSP scope and WSP did not have a model setup to simulate water age. This upgrade will prevent bottlenecking near the pump station and allow for the larger flows to be uninterrupted when provided to the northern portion of Mount Brydges.

In Mount Brydges, WSP recommends completing further pump station investigations. At this time, as reflected in the model, operating the station at firm capacity is sufficient to meet the pressure requirements of projected domestic demand conditions. During emergency fire flow conditions however, simulations reflected the need of operating with the largest pump ON in parallel with two smaller pumps ON. This operation goes beyond the firm capacity definition of the station. WSP recommends investigating the benefits of upsizing Pump 4 to be as large or larger than Pump 1. This would allow for the operation of Pump 1 while maintaining firm capacity at this station.

Figure 11: Scope of Work for Mount Brydges Priority 2-1

Figure 12: Scope of Work for Mount Brydges Priority 2-2

Project No.	Street	Proposed Solution
Priority-2-1	Gibson Road	Upgrade watermain from 150mm to 250mm for the length of 768.57m from Adelaide Road to junction J118 at Gibson Road

Priority-2-2	Glendon Drive	Upgrade watermain from 150mm to 250mm for the length of 86.55m from Adelaide Road to Emerson Street at Glendon Drive

Priority 3

Priority 3 is based on creating a watermain loop to strengthen flows in the core of Mount Brydges. This priority recommendation covers an upgrade of the 150mm watermain along Rougham Road, Pamela Drive, Seburn Drive, Wellington Street, and Parkhouse Drive, connecting into the 300mm watermain extending from Lions Park Drive on the north point and the 250mm watermain along Adelaide Road on the south point. Through this upgrade, the existing and proposed developments situated in the core of the community can be supported through encircling with watermains large enough to create a looping effect, thus facilitating larger flows.

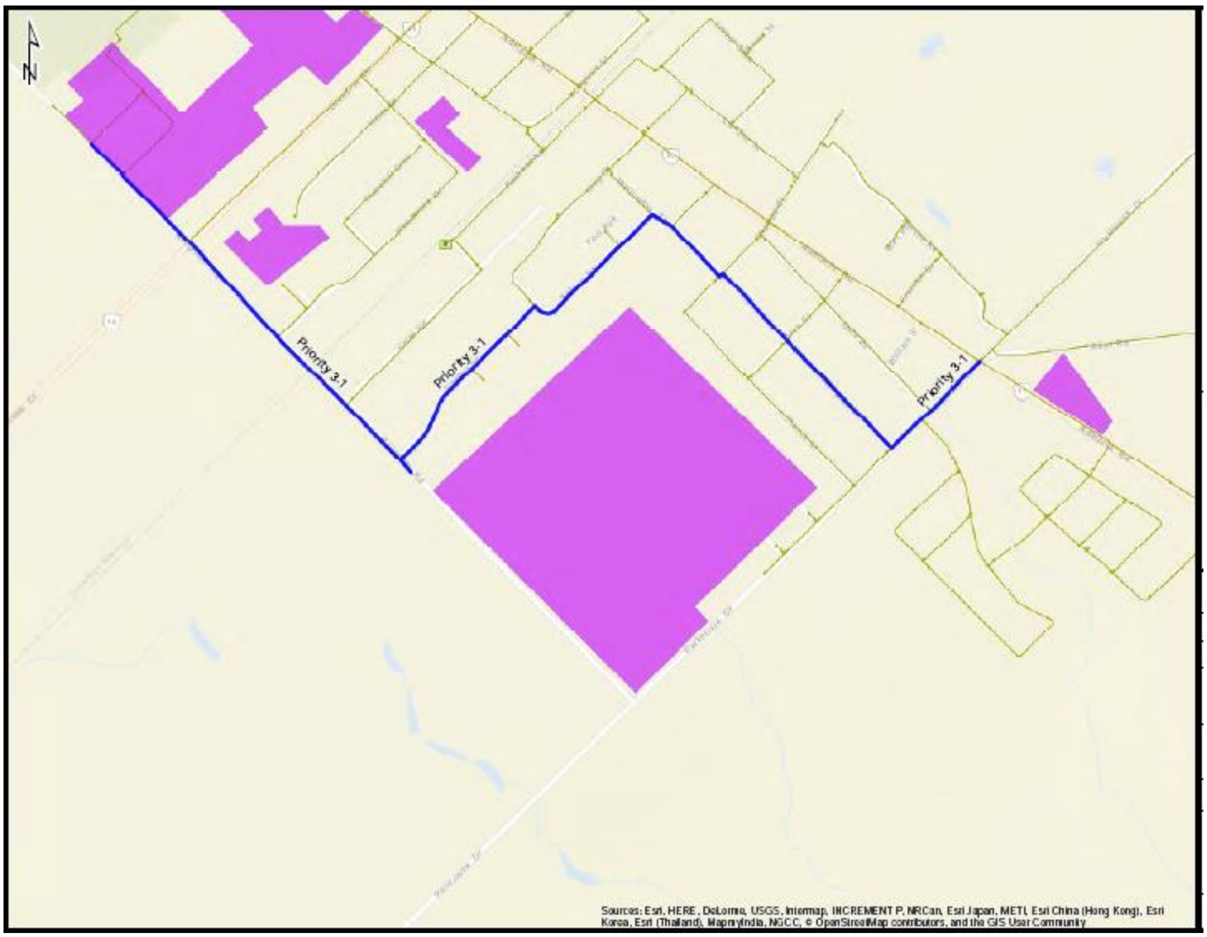

Figure 13: Scope of Work for Mount Brydges Priority 3

Project No.	Street	Proposed Solution
Priority-3-1	Rougham Road, Pamela Drive, Seburn Drive, Wellington Street, Parkhouse Drive	Upgrade watermain from 150mm to 300mm for the length of 2798.45m from Lions Park Drive to Pamela Drive at Rougham Road, Rougham Road to Seburn Drive at Pamela Drive, Pamela Drive to Wellington Street at Seburn Drive, Wellington Street to Parkhouse Drive at Wellington Street, and Wellington Street to Adelaide Road at Parkhouse Drive.

2.4.6 Mount Brydges Proposed Condition Results

2.4.6.1 Simulated Pressure Results

For the watermain network that incorporates the priority upgrade recommendations, the service pressures expected during the existing planning horizon range between 296 kPa to 685 kPa and, while it ranges from 285 kPa and 683 kPa for the future planning horizon.

Table 11: Simulated Pressure Range Under Mount Brydges Watermain Network with Priority Upgrades

DEMAND SCENARIOS	AVERAGE DAY (KPA)	MAXIMUM DAY (KPA)	PEAK HOUR (KPA)[1]
2021	364-685	363-677	296-661[1]
2036	364-683	363-672	285-650[1]

Note 1: The Peak Hour Demand scenarios require firm capacity of Pumps 2, 3 and 4 ON to run, as InfoWater Pro software indicated Pump 3 and 4 ON is inadequate to service the system

Given that the model calibration identified that the simulated pressures are higher than the tested static pressures, WSP expect that the anticipated pressures shown in the tables above are also higher than what can be expected.

Further PHD simulations were run in anticipation for ultimate buildout conditions, with demands estimated for population growth up until 2046. Junction *J252* falls under the minimum pressure by 6 kPa. This junction is at the dead-end of a 50mm watermain on Mill Road, and the proposed system would otherwise have a pressure range of 292 to 634 kPa when this junction is not considered. A detailed figure of this junction can be found in **Appendix D**.

2.4.6.2 Fire Flow Results

The minimum allowable pressure under Maximum Day Demand plus Fire Flow is 20 psi (140 kPa) at the location of the fire or anywhere else in the pressure district. As part of the fire flow analysis, WSP simulated the available fire flow (AFF) using the automated fire flow solver built in the InfoWaters software. The fire flow scenarios were simulated under Maximum Day Demand conditions for the existing and future planning horizons for both Alternative 1 and Alternative 2.

The hydraulic water model provided by the Municipality does not simulate a complete fire flow report during underperforming conditions. As a result, the boundary conditions that simulate a comprehensive fire flow summary were Pumps 1, 2 & 3 ON for Alternative 1 and Alternative 2, as well as elevated storage levels set to 72%. The conditions in which the fire flow solver is simulated may update as a result of further investigation of boundary conditions during the Official Plan Review.

Under these conditions, the modelling indicated that the existing system proves insufficient in supplying the necessary fire flows required for multiple areas of Mount Brydges during the existing and future planning horizons. When fire flows were simulated for the applied recommended upgrades to the network system, the majority of nodes were able to provide adequate fire flows in both the existing and future planning scenarios.

Given the uncertainty with the model calibration, WSP recommends that hydrant flow tests be conducted near or at the connection of a new development when a site plan application is submitted. This will allow the actual fire flow capacity and the tested capacity to be compared to the required fire flow calculations or specific fire flow policies.

A detailed analysis of fire flow availability at all nodes along the proposed municipal watermain is included in **Appendix F**.

2.5 Analysis and Recommendations

Based on the analysis completed in this study and summarized in previous sections, the simulations show that under the full buildout condition, pressure during PHD are expected to be below 275 kPa. In Strathroy, pressures were simulated down to 274 kPa and in Mt-Brydges pressure were simulated down to 191 kPa. These results reflect the existing system pumps and elevated storages. Consequently, challenges for growth are that sources of hydraulic head do not appear to be sufficient for the full buildout condition.

WSP conducted an analysis to investigate the dynamics of each zone that make up the Strathroy water system under PHD conditions (2036) to identify additional infrastructure upgrades necessary to adequately service each pressure district. The PHD simulation under 2036 conditions was simulated with the zone boundary valves opened. Under this condition, Zone 1 can support Zone 2 through internal looping which is not uncommon for a PHD scenario. This indicates that Zone 1 is operating at a higher head than Zone 2 because of the fire pump that was turned on. Note that this is not the case for ADD based on our simulations. For the ADD simulation under 2036 conditions, each zone operates independently with the zone boundaries closed.

A total of 6 Scenarios were run with boundary conditions summarized in Table 12:

- Scenario 1: Simulation results from the previous report under PHD conditions
- Scenario 2: Pumps 2, 4, and 5 ON with both PRV opens
- Scenario 3: All Pumps ON expect for the fire pump (Pump 3) with both PRVs open
- Scenario 4: All Pumps ON expect for the fire pump (Pump 3) with both PRVs closed
- Scenario 5: All Pumps ON including the fire pump with both PRVs closed
- Scenario 6: All Pumps ON expect for the fire pump (Pump 3) with both PRVs closed with a storage tank active in Zone 1

Note that WSP does not recommend operating the fire pump for everyday domestic demand supply, nor do we suggest operating pump stations beyond firm capacity. The study considers these pumps to evaluate the impact of having a larger pump ON in the various scenarios. A pump station focused scenario should be completed to properly size the pump station and determine what are the best options for expansion.

Table 12: Boundary Conditions for Scenarios in Zones 1 and 2

	ZONE 1			**ZONE 2**		
Scenario	Pump 3	Pump 4	Pump 5	Pump 1	Pump 2	PRV Settings
1	ON	OFF	OFF	OFF	ON	Open
2[1]	OFF	ON	ON	ON	OFF	Open
3[2]	OFF	ON	ON	ON	ON	Open
4[1]	OFF	ON	ON	OFF	ON	Closed
5[2]	ON	ON	ON	ON	ON	Closed
6[1]	OFF	ON	ON	OFF	ON	Closed

1 – Pumping stations for Zone 1 and 2 @ firm capacity

2 – Pumping station for Zone 1 and 2 exceeds firm capacity

As outlined in the table above, Zone 1 is controlled by three pumps and Zone 2 is controlled by two. Based on the pump curves defined for each pump in the model, the size of each pump in order from largest to smallest is Pump 3 (fire pump), Pump 2, Pump 1, Pump 4 and Pump 5 (Pump 4 and 5 are the same size). It should be noted that each set of pumps responsible for delivering flow to each zone can be viewed as an individual pumping station. Thus, firm capacity for Zone 1 and 2 can be defined as Pump 4 and 5 ON with Pump 3 OFF and Pump 1 and 2 ON, respectively.

Each pump is supplied by the Second Street Reservoir which is supplied by the Lake Huron Primary Water Supply System. The reservoir has a maximum capacity of 11,250 m^3 and is divided into three cells each with a capacity of 3,750 m^3 (Source: 2019 Summary Report for Strathroy-Caradoc Water Distribution System). Additionally, Zone 2 is also supplied by an elevated storage tank which has a capacity of approximately 1,900 m^3. The elevated storage has a maximum water level of 15.24 m based on the predetermined level set in the model. It should be noted for the analysis, a tank volume of 75% was set for the elevated storage under PHD conditions. As there was no SCADA data available for reference, a volume of 75% was assumed based on systems from other jurisdictions.

The simulation results for each scenario are outlined in **Table 13**. **Appendix I**, attached, contains the screenshots of the pressures at each junction for zone 1 and 2.

Table 13: Simulation Results for Each Scenario

SCENARIO	PRESSURE (KPA)	ELEVATED STORAGE OUTFLOW (L/S)
1	274 – 513	307
2	183 – 433	389
3	253 – 451	277
4	176 – 433	388
5	293 – 760	266
6	233 – 433	388

Starting with Scenario 2, both pumping stations (i.e., the set of pumps for Zone 1 and 2) were set so that each station was operating below firm capacity. With both PRVs open, pressures across the entire system drops below the required pressures of 275 kPa; however, the junctions that fail are more concentrated in Zone 1 (see Figure 1 in **Appendix I**). For Scenario 3, both pumping stations were set to firm capacity (i.e., all pumps on; no fire pump). The result is an increase in pressure with a few junctions falling below the target pressure of 275 kPa in Zone 1 (see Figure 2 in **Appendix I**). Under Scenario 4, both PRVs were forced closed resulting in inadequate pressures across Zone 1 even at firm capacity due to the lack of internal looping between pressures zones (see Figure 3 in the **Appendix I**). It is only until the fire pump is switched on in Scenario 5 that the pressures across the entire system pass (see Figure 4 in the **Appendix I**). However, this is not ideal as both stations are operating above firm capacity.

To increase pressures across the system, a fixed head reservoir in Scenario 6 was add at the highest elevation point in Zone 1 within the town's boundaries. This is highlighted in Figure 5 in **Appendix I**. It was determined that to increase pressures above 275 kPa without exceeding firm capacity, an HGL of 274.5 m (or an elevated storage tank approximately 30 m tall) is required. Under this scenario, all junctions in Zone 1 pass with a couple junctions failing in Zone 2 (see Figure 5 in the **Appendix I**). However, the junctions that fail in Zone 2 are located around the pumping station with the exception of one junction located at the far end of the network. The pressure at this junction is approximately 273 kPa, which falls short of the required pressure target of 275 kPa by 2 kPa (or 0.7%).

Additionally, an analysis of the existing storage tank in Zone 2 was conducted under PHD and is outlined in the table below:

Table 14: Analysis of Existing Storage Tank in Zone 2 under PHD

SCENARIO	ELEVATED STORAGE OUTFLOW (L/S)	ELEVATED TANK STORAGE 100% FULL (L)	ELEVATED TANK STORAGE 75% FULL (L)	TIME TO DRAIN TO A VOLUME OF 50% (25% DROP IN VOLUME) (MIN)	TIME TO DRAIN TO A VOLUME OF 60% (15% DROP IN VOLUME) (MIN)
1	307	1,900,000	1,425,000	26	15
2	388	1,900,000	1,425,000	20	12
3	277	1,900,000	1,425,000	29	17
4	388	1,900,000	1,425,000	20	12
5	266	1,900,000	1,425,000	30	18
6	388	1,900,000	1,425,000	20	12

Based on the outflow from the tank, the time for the elevated storage tank to drain is dependent on the number of pumps that are on for Zone 2 and whether the PRVs are open or closed. When one of the two pumps supplying flow to Zone 2 is OFF, the time for the elevated storage to drain to 50% of its volume is shorter for Scenarios 1 and 2. However, with the additional pump turned on (Scenarios 3, 4, 5, and 6), more flow can be supplied by the pump station with the system being less reliant on the elevated storage for supply. However, the time for the storage tank to drain is relatively the same across all the scenarios suggesting that a larger or second elevated storage tank may be required.

Based on the results of this analysis, increasing the hydraulic head of the distribution system is required to achieve adequate pressures in order for each zone to operate independently of each other (i.e., valves closed at each zone boundary). Adequate pressures can be achieved across the distribution system when all pumps are ON; however, this is not ideal as each pump station would be operating beyond firm capacity. To improve pressures, an elevated storage needs be added to Zone 1 to increase the hydraulic head and water supply volume across the zone. This is ideal as it would create an additional source of supply into the system and can be recharged at night. Other methods to increase head include increasing the impeller size of each pump at the pump station or adding additional pumps to increase firm capacity. Alternatively, pump replacement can be considered.

Similarly, the simulations completed for the Mount Brydges system indicate that in the planned full buildout scenario, the pump station needs to operate at or above firm capacity to deliver the required head needed to maintain the minimum pressure target throughout the zone. To achieve this, similar pump station strategies should be consider: either increasing the impeller size of existing pumps (effectively giving them a larger capacity) or adding a pump to the pump station increasing its firm capacity overall.

2.6 Conclusions

The proposed municipal watermain system for the Municipality of Strathroy-Caradoc can achieve hydraulic requirements as prescribed by the Ministry of the Environment, Conservations and Parks and the Municipality of Strathroy-Caradoc watermain design criteria as summarized below.

Strathroy

The current challenge that growth faces in the Strathroy system is the availability of storage volume needed to meet consumption demands as well as the supply of hydraulic head from the pump station. In its current state Zone 2 is able to support some additional growth, but not up to the full buildout (2046) presented herein assuming that the PRV between Zone 1 and Zone 2 opens during Peak Hour Conditions.

Prior to developments being built, WSP recommends that individual development watermain analyses be completed using the hydraulic model to validate that an adequate amount of pressure, fire flow and water supply is available. Furthermore, WSP recommends that the hydraulic models be updated to simulate as an extended period simulation to better determine the impact of demands on storage volume.

1 The service pressures under existing conditions, and future conditions are expected to range between 274 kPa and 521 kPa for the proposed watermain network system, which is within standards established by the MECP and Strathroy-Caradoc Servicing Standards with the exclusion of junction J54;

   a Junction J54 is connected to a trunk watermain adjacent to a pressure reducing valve, and without accounting for this junction, the minimum pressure simulated would be 276 kPa, meeting the minimum requirement;

2 While some available fire flows in the existing system are not able to meet the required fire flows, all required fire flows can be achieved under Maximum Day Demand conditions for the proposed development under existing and future conditions for the proposed network;

   a WSP recommends the implementation of the priority upgrades in order to target nodes in the existing system that fail to reach required fire flows;

3 Under Maximum Day plus Fire Flow during existing and future conditions for the proposed system, the Strathroy distribution system is able to maintain pressure above 140 kPa (20psi) at ground level at all modeled nodes in the community;

4 When installing AWWA C900-compliant PVC pipe with a pressure rating of 150 psi (or greater), the proposed watermains can withstand the transient pressure created by stopping a water column moving at 0.6 m/s plus maximum operating pressure.

5 To improve pressures, an elevated storage needs be added to Zone 1 to increase the hydraulic head and water supply volume across the zone. This is ideal as it would create an additional source of supply into the system and can be recharged at night. Other methods to increase head include increasing the impeller size of each pump at the pump station or adding additional pumps to increase firm capacity. Alternatively, pump replacement can be considered.

Mount Brydges

The current challenge that growth faces in the Mount Brydges system is the supply of hydraulic head from the pump station. In its current state, the system is able to support some additional growth but not the full buildout conditions (2046) considered in this study.

Prior to developments being built, WSP recommends that individual development watermain analyses be completed using the hydraulic model to validate that an adequate amount of pressure, fire flow and water supply is available. Furthermore, WSP recommends that the hydraulic models be updated to simulate as an extended period simulation to better determine the impact of demands on storage volume.

1 Based on the validation of the model's calibration, WSP expects that the results presented herein are higher than what would be expected in the field. The hydrant tests show that the model over-predicts results by roughly 20 kPa. WSP recommends that hydrant flow test(s) near or at the connection of the site's watermain/service line to determine the actual simulated pressure and fire flow capacity.

  a A full model calibration was not part of the scope of this study – in order for the model to match hydrant flow test more closely, a model calibration (including field work to support) would be required.

2 The service pressures under existing conditions, and future conditions are expected to range between 191 kPa and 675 kPa for the existing system and 285 kPa and 685 kPa for the proposed system;

3 Most required fire flows can be achieved under Maximum Day Demand conditions for the proposed development under existing and future conditions for the proposed system;

4 Under Maximum Day plus Fire Flow during existing and future conditions for the proposed system, the distribution system is able to maintain pressure above 140 kPa (20psi) at ground level at all modeled nodes in the district, and;

5 When installing AWWA C900-compliant PVC pipe with a pressure rating of 150 psi (or greater), the proposed watermains can withstand the transient pressure created by stopping a water column moving at 0.6 m/s plus maximum operating pressure.

6 To achieve this, similar pump station strategies should be consider: either increasing the impeller size of existing pumps (effectively giving them a larger capacity) or adding a pump to the pump station increasing its firm capacity overall.

These conclusions remain valid as long as the proposed water distribution system and the Municipality's network configuration remain as described herein. If significant changes are contemplated, this analysis should be updated.

3 Wastewater Conveyance

3.1 Introduction

3.1.1 Objective

Considering the expected growth of Mount Brydges and Strathroy, the Municipality of Strathroy-Caradoc is anticipating multiple residential developments spread throughout the service area up to the year 2046. In order to facilitate this expansion, WSP was retained to evaluate the capacity of the Mount Brydges and Strathroy sanitary sewer network and identify any improvements, if required. The report presents a brief description of the sanitary sewer system, spreadsheet model setup, modeling scenarios, and the results of the analysis for the existing and future planning horizons.

3.1.2 Study Area

On January 1st, 2001, the Town of Strathroy and the Township of Caradoc amalgamated to form the Municipality of Strathroy-Caradoc. The Mount Brydges community is located in the eastern portion and the community of Strathroy is located on the northern limit of the Municipality of Strathroy-Caradoc. Both the communities contain full municipal water supply and sewage services. Within Mount Brydges, there are areas of existing development where a partial servicing condition exists, whereby only municipal water supply is provided, while sewage is provided by private systems.

3.1.2.1 Mount Brydges

Mount Brydges is a small community in the Municipality of Strathroy-Caradoc. This community is serviced by a sanitary sewer network that includes gravity sewers, maintenance holes, two (2) sewage pumping stations (SPS), one (1) sewage treatment plant. The existing sanitary sewer network including the pumping stations and the sewage treatment plant are shown in **Figure 13**.

The Northwest SPS is located on Lions Park Drive. This SPS receives flows from the portion of Mount Brydges north and west of the Canadian National Railway (CNR) which runs through the middle of the community. Sewage from the Northwest SPS discharges via 200mm diameter forcemain. The forcemain follows Lions Park Drive northeasterly to Adelaide Road and then southwesterly to discharge to a gravity sewer at the first manhole south of the CNR, which flows to the Main SPS.

The main SPS is located at the intersection of Adelaide Road and Mill Road. The Main SPS receives sewage from the south area of the community plus the flows transmitted from the Northwest SPS. The main SPS then discharges the entire flow to the sewage treatment plant via a 250mm diameter forcemain.

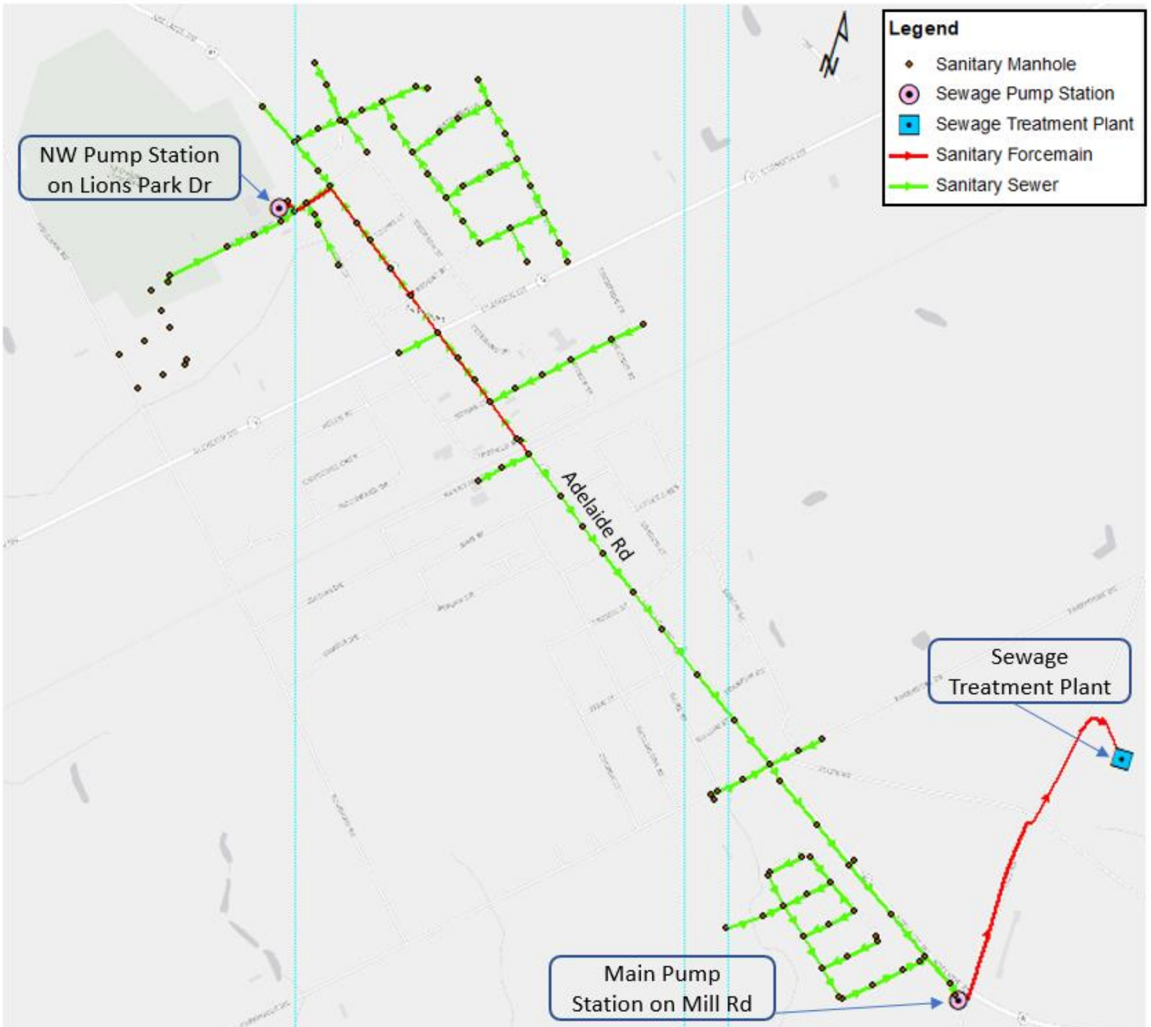

Figure 14: Overview of the Existing Mount Brydges Sanitary Sewer Network

3.1.2.2 Strathroy

Strathroy is a residential community in the Municipality of Strathroy-Caradoc, serving commuters who work in London and Sarnia, and a commercial center servicing the surrounding agricultural community. The community also has a diverse industrial base with an active manufacturing sector. The community of Strathroy is serviced by a sanitary sewer network that includes gravity sewers, maintenance holes, nine (9) sewage pumping stations (SPS), and one (1) sewage treatment plant. **Figure 14** show the existing sanitary sewer collection system including pumping stations and sewage treatment plant.

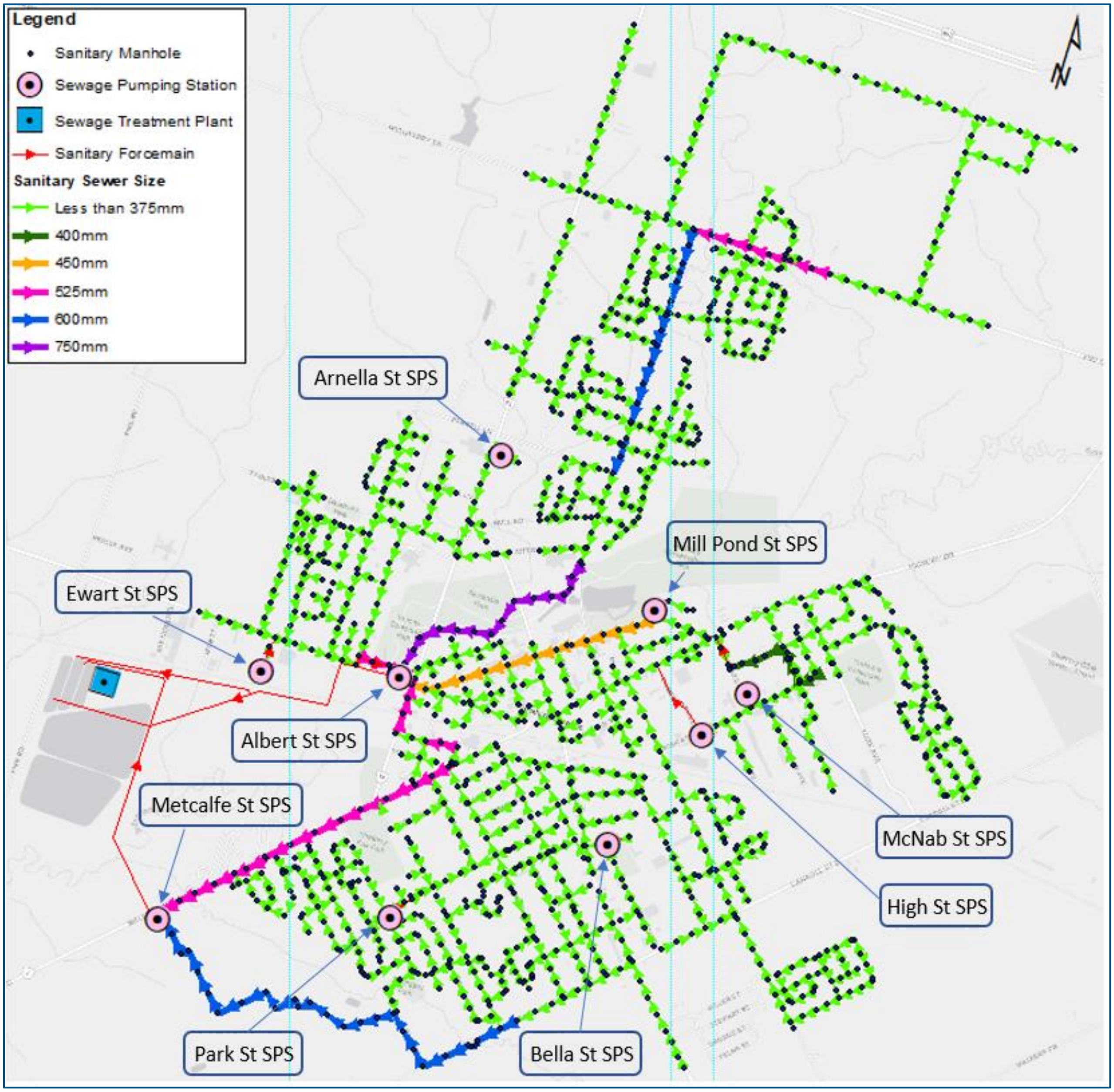

Figure 15: Overview of the Existing Strathroy Sanitary Sewer Network

As shown in **Figure 14**, the Strathroy sewer system has nine sewage pumping stations. Out of nine, two pumping stations service the majority of the drainage areas. Albert St SPS and Metcalfe St SPS pump all of the wastewater from the Strathroy collection system to the sewage treatment plant. The other SPS's are all secondary, discharging to the existing sanitary sewers. Details of the available pumping stations are provided in **Table 15**.

Table 15: Strathroy - Sewage Pumping Station Information

STATION NAME	NO. OF PUMPS	RATED FLOW (L/S)	RATED HEAD (M)	PUMP POWER (HP)
Albert Street PS	3	258	39	140
Arnella Street PS	1	-	-	-
Bella Street PS	2	-	-	-
Ewart Street PS	2	193	12	-
High Street PS	1	-	-	-
McNab Street PS	2	41.4	9.1	7.5
Metcalfe Street PS	2	75	30.8	-
Mill Pond PS	2	-	-	2.4
Park Street PS	2	22.1	-	9.4

Note: "-" indicates information not available.

3.2 Sanitary Sewer Capacity Analysis

Sanitary Sewer Capacity Analysis was performed using the spreadsheet model that WSP developed for this study. The spreadsheet model was limited to evaluate the capacity of the trunk mains only; no local sanitary sewer mains were included in the analysis. The Mount Brydges sewer segments analyzed in this study under the existing and future planned developments are presented in **Figure A-1** and **Figure A-2** in **Appendix A**. The Strathroy sewer segments analyzed in this study under the existing and future planned developments are presented in **Figure A-3** in **Appendix A**.

3.2.1 Spreadsheet Model Setup

The GIS database of the Mount Brydges and Strathroy sewer network, as provided by the Municipality of Strathroy-Caradoc, was utilized for this study. The sewer network data included sewer mains, maintenance holes, and pumping stations. All the sanitary sewers that service the Mount Brydges and Strathroy communities were reviewed and the main sewer segments that collect local sanitary flow from the study areas were identified. The identification of the main sewer segments was based on the location of the main sewers, which collect sanitary flows from local residential and non-residential areas and convey the flows to the sewage pumping stations. The sewer segments analyzed in this study are shown in **Figure A-1** and **Figure A-3** in **Appendix A**.

3.2.1.1 Mount Brydges

The Mount Brydges spreadsheet model is divided into two parts - first part includes sewer segments upstream of Northwest SPS as shown in **Figure A-1a**. This part of the sewer network collects the flows from north of CNR and discharges them to the Northwest SPS Wetwell. Based on the *"Mount Brydges – Main Sewage Pumping Station and Northwest Sewage Pumping Station – Operation and Maintenance Manual"* the Northwest SPS is equipped with pumps to meet the 20-year design flows. As per the O & M manual, the 20-year peak design flow for Northwest SPS is 31.3 L/s.

The second part of the spreadsheet includes sewer segments downstream of Northwest SPS as shown in **Figure A-1b**. This part of the sewer network receives flows from the south of CNR in addition to the 31.3 L/s flow transmitted from Northwest SPS, and discharges to the Main SPS Wetwell.

3.2.1.2 Strathroy

The Strathroy spreadsheet model is divided into four sewer legs as shown in **Figure 16** below. Albert St PS and Metcalfe St PS currently pump all of the wastewater from Strathroy directly to the sewage treatment plant. Sewer Leg#1, 2, and 3 includes sewer segments upstream of Albert St PS and Sewer Leg#4 includes sewer segments upstream of Metcalfe St PS.

Sewer Leg #1 - This part of the sewer network collects the flows along Second St and Head St N and discharges them to the Albert St PS.

Sewer Leg #2 - This part of the sewer network collects the flows from Ewart St PS along Albert St and discharges them to the Albert St PS.

Sewer Leg #3 - This part of the sewer network collects the flows from McNab St PS, Mill Pond PS, High St PS, Front St E, and Victoria St and discharges them to the Albert St PS.

Sewer Leg #4 - This part of the sewer network collects the flows from Park St PS, Park St, and Metcalfe St W and discharges them to the Metcalfe St PS.

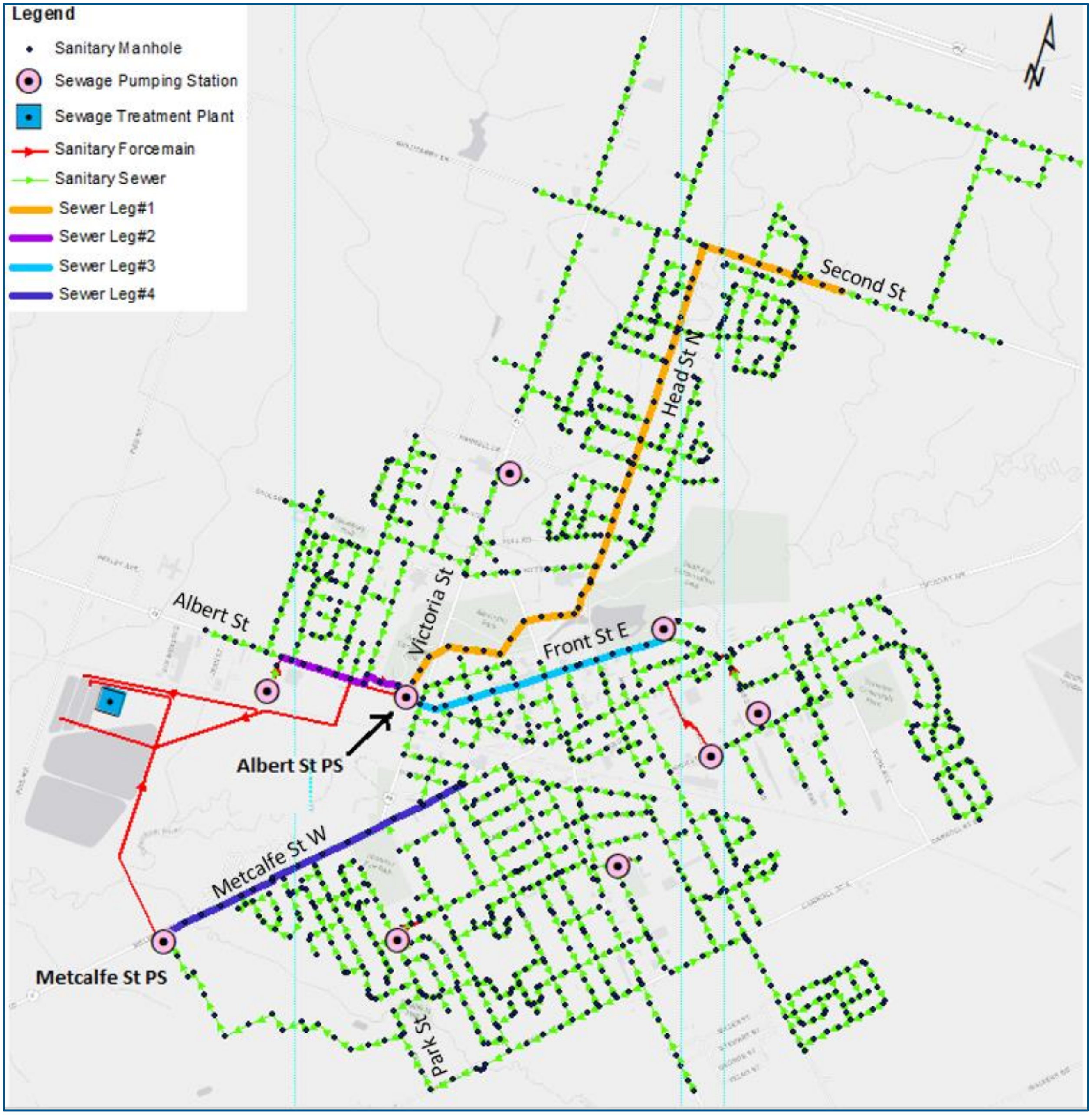

Figure 16: Strathroy Sanitary Sewer Network with Analyzed Sewer Legs

3.2.1.3 Sub-catchment Delineation and Wastewater Flows

To calculate the theoretical flows for the study area, sanitary sub-catchments were delineated for the local residential and non-residential areas which convey flows to the main sewers. A total of 9 sub-catchments for Mount Brydges and 71 sub-catchments for Strathroy were delineated based on the parcel layer to allocate the flows to the appropriate sewer segments. The sub-catchment boundaries are generally aligned with the parcel lot boundaries. The gross area of the sub-catchment included the area of the residential/ non-residential parcel, green areas, and roads.

Since no land-use information per parcel was available for the study area, the base wastewater flow (generated from customers) was estimated based on the water demand data obtained from the Mount Brydges and Strathroy water distribution network hydraulic models (updated for this study). It was assumed that 80% of the water customer water consumption within a sub-catchment converts to wastewater. The Mount Brydges sanitary sub-catchments (ID: 1 to 9) are shown in **Figure A-1** and Strathroy sanitary sub-catchments (ID: 0 to 70) are shown in **Figure A-3**. A peaking factor of 3.5 was applied to the average wastewater flows estimated for each sub-catchments.

Additionally, the rainfall-dependent inflow and infiltration rate was applied to estimate the peak wastewater flow generation from each sub-catchment. The rainfall-dependent inflow and infiltration was calculated from the contributing area of each sub-catchment. Contributing area was calculated by subtracting approximate green areas from the gross area of each sub-catchment. Green areas in each sub-catchment were estimated based on the aerial base map for the study area. The contributing area of the sub-catchment included the area of the residential/ non-residential parcel, and the roads if any. For example, in **Figure 17**, the gross area of sub-catchment 8 in the Strathroy network was 30 ha, however, the contributing area estimated based on the review of the aerial map, after deducting green area, was 0.75 ha.

Figure 17: Sample Sub-catchment with Contributing Area

3.2.2 Study Assumptions

The following assumptions were made in developing the spreadsheet model for performing the sewer capacity analysis:

- Gravity sewers Manning's minor loss coefficient = 0.013
- Base wastewater flow generated is 80% of the water demand
- A peaking factor of 3.5 was used to calculate the peak flow generated within each sub-catchments
- Pipe capacity calculation does not include the local sewers; the flows generated from any land-use type are added to the nearest sewer segment identified.

3.2.3 Model Scenarios

The servicing capacity of the study area sewer network was determined under the following scenarios:

1 Existing Condition, and

2 Future Condition

3.2.3.1 Mount Brydges

Scenario 1 - Existing Condition - This scenario evaluates the theoretical sewer capacity under the existing flow conditions generated from the delineated sub-catchments as presented in **Figure A-1**.

Scenario 2 – Future Condition - This scenario evaluates the theoretical sewer capacity under 2046 flow conditions. WSP utilized the statistics of developments proposed for 2036 and vacant lands out to 2046 as provided by the municipality. The anticipated developments in Mount Brydges are of two types: proposed developments and vacant residential lands that are designated for potential developments. The proposed and planned developments (ID: 10 to 18) are presented in **Figure A-2**. For consistency, the water demand data for these future developments were obtained from the Mount Brydges water distribution network hydraulic model. The theoretical sanitary flows for each future development were estimated as 80% of the water consumption in respective developments.

3.2.3.2 Strathroy

Scenario 1 - Existing Condition - This scenario evaluates the theoretical sewer capacity under the existing flow conditions generated from the delineated sub-catchments as presented in **Figure A-3**.

Scenario 2 – Future Condition - This scenario evaluates the theoretical sewer capacity under 2046 flow conditions. The statistics for proposed developments in Strathroy were not available hence the water demand data forecasted in the Strathroy water distribution network hydraulic model was used. The theoretical sanitary flows for each future development were estimated as 80% of the water consumption in respective developments.

3.3 Results and Conclusions

Appendix B presents the sanitary sewer design sheets for all the analyzed sewer segments under the scenarios mentioned in Section 2.3. In addition, the pumping capacities of the sewage pumping stations were evaluated to determine their adequacy to handle the current and future flows. The results of each scenario are summarized below.

3.3.1 Mount Brydges

GRAVITY SYSTEM

Scenario 1 - Existing Condition

The spreadsheet model presented in **Table B-1** in **Appendix B** confirmed that the analyzed sewer segments have adequate capacity to serve the current peak flows. Under the current peak flow conditions, the sewer capacity utilization was identified in the range of 6% to 22%. Therefore, the remaining capacity of the analyzed sewer segments ranges from 78% to 94%.

Scenario 2 – Future Condition

The spreadsheet model presented in **Table B-2** in **Appendix B** confirmed that the analyzed sewer segments have adequate capacity to serve the future flow conditions. Under the future development flow conditions, the sewer capacity utilization will be in the range of 8% to 56%. Therefore, the remaining capacity in analyzed sewer segments will be in the range of 44% to 92%. This range includes a variety of collector sewers from upstream (higher remaining capacity) and downstream (lower remaining capacity) parts of the system. The range presented herein is not abnormal.

NORTHWEST SEWAGE PUMPING STATION

This is a duplex pumping station (1 working + 1 standby). The capacity of each installed pump is 31.3 L/s at 15.78m TDH as mentioned in *"Mount Brydges – Main Sewage Pumping Station and Northwest Sewage Pumping Station – Operation and Maintenance Manual".* It is assumed that only one pump will be in operation during peak flow conditions. As shown in **Table B-1** in **Appendix B**, the current peak inflow to the Northwest SPS is 9.22 L/s. Therefore, it can be concluded that the Northwest SPS capacity is adequate to handle current flows while maintaining the current configuration and forcemain sizing and connectivity.

As shown in **Table B-2** in **Appendix B**, the future (2046) peak inflow to the Northwest SPS is 37.36 L/s. Considering the future peak inflow (37.36 L/s) is marginally higher than the design flow (31.3 L/s), which indicates that this pump station is expected to be undersized for the future conditions. Further review of the pump station performance is needed to determine any capacity increase. The following two scenarios may occur while the system experience peak inflow.

- Pump discharge flow is equal to or slightly less than the design flow. In this situation, the difference in the peak inflow and pump discharge flow will start accumulating inside the wet well and subsequently pumped out soon after the inflow is reduced.
- Pump discharge flow is more than the design flow. This situation may occur due to the lower system head requirement (anticipated based on preliminary review) than the design head

calculated during pump station design. In this situation, the existing pump may be adequate to handle the future peak flow.

In summary, WSP recommends pumps test be performed to investigate the actual pumping capacity of the installed pumps. Accordingly, an upgrade to the existing pumps can be determined.

MAIN SEWAGE PUMPING STATION

This is a duplex pumping station (1 working + 1 standby). The capacity of each installed pump is 53 L/s at 15.63m TDH as included in *"Mount Brydges – Main Sewage Pumping Station and Northwest Sewage Pumping Station – Operation and Maintenance Manual".* It is assumed that only one pump will be in operation during peak flow conditions. As shown in **Table B-1** in **Appendix B**, the existing peak inflow to the Main SPS is 35.97 L/s. Therefore, it can be concluded that the Main SPS capacity is adequate to handle existing peak flows while maintaining the existing configuration and forcemain sizing and connectivity.

As shown in **Table B-2** in **Appendix B**, the future (2046) peak inflow to the Main SPS is 51.29 L/s. Considering the design flow of 53 L/s, it can be concluded that the Main SPS capacity is adequate to handle future peak flows.

3.3.2 Strathroy

GRAVITY SYSTEM

Scenario 1 - Existing Condition

The spreadsheet model presented in **Table B-3** in **Appendix B** confirmed that the analyzed sewer segments have adequate capacity to serve the existing flows. Under the existing flow conditions, the sewer capacity utilization was identified in the range of 6% to 76%. Therefore, the remaining capacity of the analyzed sewer segments ranges from 24% to 94%. This range includes a variety of sewers from upstream (higher remaining capacity) and downstream (lower remaining capacity) parts of the system. The range presented herein is not abnormal.

Scenario 2 – Future Condition

The spreadsheet model presented in **Table B-4** in **Appendix B** confirmed that the analyzed sewer segments have adequate capacity to serve the future flow conditions. Under the future development flow conditions, the sewer capacity utilization will be in the range of 6% to 82%. Therefore, the remaining capacity in analyzed sewer segments will be in the range of 18% to 94%. This range appears to overlap the existing range, which is not abnormal given that: 1) not all sewers will have new loads apply to them, and 2) small loads added to steeper sewers will have a non significant impact on the sewer capacity.

Albert St SPS and Metcalfe St SPS pump all of the wastewater from Strathroy directly to the sewage treatment plant. The other SPS's are all secondary, discharging to the existing sanitary sewers hence capacity assessment is carried out for Albert St SPS and Metcalfe St SPS only.

ALBERT STREET SEWAGE PUMPING STATION

This is a triplex pumping station (2 working + 1 standby). It is assumed that two pumps will be in operation during peak flow conditions. As shown in **Table B-3** in **Appendix B**, the current total peak inflow to the Albert St SPS is 225.95 L/s however, the total capacity of the pump station is 258 L/s at 39m TDH as mentioned in *"Albert Street Sewage Pumping Station Pumps 1 & 2 Replacement – Operation and Maintenance Manuals".* Therefore, it can be concluded that the Albert St SPS is adequate to handle existing peak flows while maintaining the existing configuration and forcemain sizing and connectivity.
As shown in **Table B-4** in **Appendix B**, the future (2046) peak inflow to the Albert St SPS is 282.2 L/s. Considering the future peak inflow (282.2 L/s) is marginally higher than the design flow (258 L/s), further review of the pump station performance is needed to determine any capacity increase. The following two scenarios may occur while the system experience peak inflow.

- Pump discharge flow is equal to or slightly less than the design flow. In this situation, the difference in the peak inflow and pump discharge flow will start accumulating inside the wet well and subsequently pumped out soon after the inflow is reduced.
- Pump discharge flow is more than the design flow. This situation may occur due to the lower system head requirement (anticipated based on preliminary review) than the design head calculated during pump station design. In this situation, the existing pump may be adequate to handle the future peak flow.

In summary, WSP recommends pumps test be performed to investigate the actual pumping capacity of the installed pumps. Accordingly, an upgrade to the existing pumps can be determined.

METCALFE STREET SEWAGE PUMPING STATION

This is a duplex pumping station (1 working + 1 standby). It is assumed that one pump will be in operation during peak flow conditions. As shown in **Table B-3** in **Appendix B**, the existing total peak inflow to the Metcalfe St SPS is 65.21 L/s however, the capacity of the existing pumps is 75 L/s at 30.8m TDH as mentioned in *"Metcalfe Street Sewage Pumping Station – Operation and Maintenance Manuals".* Therefore, it can be concluded that the Metcalfe St SPS is adequate to handle existing peak flows while maintaining the existing configuration and forcemain sizing and connectivity.

As shown in **Table B-4** in **Appendix B**, the future (2046) total peak inflow to the Metcalfe St SPS is 84.44 L/s. Considering the future peak inflow (84.44 L/s) is marginally higher than the design flow (75 L/s), further review of the pump station performance is needed to determine any capacity increase. The following two scenarios may occur while the system experience peak inflow.

- Pump discharge flow is equal to or slightly less than the design flow. In this situation, the difference in the peak inflow and pump discharge flow will start accumulating inside the wet well and subsequently pumped out soon after the inflow is reduced.
- Pump discharge flow is more than the design flow. This situation may occur due to the lower system head requirement (anticipated based on preliminary review) than the design head

calculated during pump station design. In this situation, the existing pump may be adequate to handle the future peak flow.

In summary, WSP recommends pumps test be performed to investigate the actual pumping capacity of the installed pumps. Accordingly, an upgrade to the existing pumps can be determined.

3.3.3 Conclusions

A spreadsheet model was developed for the Mount Brydges and Strathroy sanitary sewer system. The capacity analysis was performed on the existing condition scenario and future development scenario. Based on the capacity analysis for Mount Brydges and Strathroy, the analyzed existing sanitary sewer segments have sufficient capacity to service the existing condition and the planned development.

The capacity analysis for Mount Brydges suggests that the Northwest SPS has adequate capacity to handle current flows. The analysis further indicated that the Northwest SPS does not have adequate capacity to handle future flows (2046). However, the actual pump performance via pump test is recommended to determine whether any upgrades are required. Main SPS is identified with sufficient capacity to handle existing and future flows (2046).

When considering the available wastewater conveyance capacity within Mount Brydges and Strathroy and the Municipality’s intentions to further study and consideration system upgrades, it is recommended that Official Plan policies be considered which limit the future use of partial servicing within the settlement area. The policies should reflect the improved land use efficiency of developing on full municipal services (e.g., smaller lot sizes) and future connections to municipal wastewater collection more broadly.

The capacity analysis for Strathroy identifies that the Albert St SPS and Metcalfe St SPS have adequate capacity to handle existing flows. Based on the future peak inflows it seems these pump stations do not have adequate capacity, however the actual pump performance via pump test is recommended to determine whether any upgrades are required.

4 Wastewater Treatment

4.1 Introduction

This component of the Servicing Capacity and Constraints Study report specifically addresses the system review of the Strathroy Wastewater Treatment Plant (WWTF) and the Mount Brydges Sewage Treatment Plant (WWTF) to ensure there is capacity to service the needs to 2046.

4.2 Background and Objectives

The Municipality of Strathroy-Caradoc is a lower-tier Municipality within the County of Middlesex with residential, industrial and commercial land uses. With an overall population of approximately 22,000, the Municipality is serviced by two sewage treatment plants – Strathroy WWTF (WWTF) and Mount Brydges WWTF (WWTF). The Strathroy WWTF has a rated capacity of 10,000 m^3/d and is operated in accordance with the Amended ECA No. 5933-C37KWJ issued on June 24, 2021. The WWTF consists of inlet works, earthen aeration basin with a fine bubble aeration system, two secondary clarifiers, RAS/WAS pump station, cloth disk media filter, UV disinfection, sludge storage and standby power diesel generator.

The Mount Brydges WWTF has a rated capacity of 825 m^3/d and is operated in accordance with the C of A No. 7788-8BJRL8 issued on January 26, 2011. The WWTF includes rotating biological contactors (RBCs), final clarifiers, filter pump chamber, an effluent filtration system, UV disinfection system, chemical storage and feed system for alum and soda ash, effluent outfall and standby power diesel generator.

The objective of this report is to:

1. Provide a summary of the major maintenance needs and proposed rehabilitation strategies
2. Provide summary of capacity to meet development needs to 2046.
3. Provide recommendations for appropriate system upgrades.
4. Review expansion considerations for Mount Brydges WWTF with options for expanding or replacing the RBC technologies .
5. Review expansion considerations for Strathroy WWTF.
6. Evaluate sludge handling needs and options for future decommissioning of the historical sewage lagoons.

4.3 Sources of Data

The following sources of data were used for the capacity review:

1. Amended Environmental Compliance Approval No. 2228-9XXQKQ dated July 20, 2015 (for the Strathroy WWTF)
2. Certificate of Approval No. 7788-8BJRL8 dated January 26, 2011 (for Mount Brydges WWTF)
3. 2016 – 2021 Annual Reports for Strathroy WWTF and Mount Brydges WWTF

4. Daily SCADA Summary Report for Strathroy WWTF for 2018 – November 2020
5. Daily In-House Lab Results for Strathroy WWTF for 2018 – November 2020
6. Daily Wastewater Summary Report for Mount Brydges WWTF for 2018 – September 2020
7. Operation Manual for the Mount Brydges WWTF
8. Strathroy WWTF Operational Improvement Action Plan
9. Mount Brydges WWTF Treatment Upgrades Study
10. Strathroy WWTF Upgrades – Aerated Lagoon Upgrade Report

4.4 Strathroy Sewage Treatment Plant

4.4.1 Treatment Process

According to the Amended ECA No. 5933-C37KWJ, the Strathroy WWTF has a rated capacity of 10,000 m^3/d with a peak flow rate of 23,280 m^3/d. The plant is comprised of inlet works, earthen aeration basin with a fine bubble aeration system, two secondary clarifiers, RAS/WAS pump station, cloth disk media filter, UV disinfection, sludge storage and standby power diesel generator. According to the 2016 Annual Performance Report, the plant was upgraded in 2000 and converted from an aerated lagoon treatment process to a mechanical sewage treatment plant with a capacity of 8,560 m^3/d. The plant was re-rated in 2010 with a capacity of 10,000 m^3/d.

Wastewater (raw sewage) is directed to an inlet works which contains a flow splitter box to divert the wastewater flow to the screening channel. Grit material or other large debris is removed using a mechanically cleaned bar screen that is located in the screening channel. The screened wastewater is then directed to the aeration basin for secondary treatment which operates as an extended aeration process. The aeration basin is constructed with a membrane liner and aeration is provided using a fine bubble aeration system. Three air blowers are used to provide the air requirements for the lagoon. A 150-kW diesel generator is used to provide emergency standby power. The wastewater from the aeration basin is directed to two circular secondary clarifiers. Cloth disk media filters are used for post-secondary treatment of the effluent from the secondary clarifier. Return activated sludge (RAS) from the secondary clarifier flows to the RAS/WAS wet well which is equipped with two (2) RAS pumps for returning the RAS to the aeration basin. Secondary scum from the clarifiers flows to a scum pit which is equipped with a scum transfer pump. The treated effluent leaving the secondary clarifier is subjected to UV disinfection using a system comprised of two channels equipped six UV lamps per channel. This provides a total of fourteen (14) modules.

Waste activated sludge (WAS) is directed to one sludge storage pond (lagoon) that is equipped with a surface aerator. A supernatant pump is used to transfer the clarified effluent from the pond to the aeration section for the treatment.

4.4.2 Process Design Summary

The process design summary for the Strathroy WWTF is shown in **Table 16**.

Table 16: Strathroy Wastewater Treatment Plant Process Design Summary

UNIT PROCESS	DESIGN PARAMETER
Inlet Works	
Splitter box	
Number	1
Screening Channel	
Type	Equipped with a mechanically cleaned bar screen
Number	1
Dimension	0.76 m x 6 m
Capacity	Peak flow of 23,280 m^3/d
Aeration Basin	
Aeration Basin	
Type	Earthen aeration basin equipped with fine bubble aeration system
Number	1
Volume	8,560 m^3
Aeration Blowers	
Number	2
Capacity (each)	4,750 m^3/h
Number	1
Capacity	9,500 m^3/h
Standby Diesel Generator	
Number	1
Capacity	150 kW
Fuel Tank Capacity	1,135 L
Secondary Clarifier	
Secondary Clarifier	
Type	Circular
Number	2
SWD	4 m
Diameter	22.5 m
RAS/WAS wet well	
Number	1
RAS Pump	
Number	2 (1 duty and 1 spare)
Pump Capacity (each)	72 L/s (6,221 m^3/d) @ 7m TDH
WAS Pump	
Number	1
Pump Capacity (each)	17 L/s (1,469 m^3/d) @ 13m TDH
Scum Pit	
Number	1
Volume	3.5 m^3
Scum Transfer Pump	
Number	1
Pump Capacity (each)	17 L/s (1,469 m^3/d) @ 13m TDH
Post-Secondary Treatment System	
Cloth Media Disk Filter Units	Packaged cloth media disk filter units arranged in parallel
Number	2 (1 duty and 1 spare)
Capacity (each)	Peak hourly flow of 1,104 m^3/h

UNIT PROCESS	DESIGN PARAMETER
Supplementary Treatment System	
Alum Pumps No. of Pumps Capacity (each)	 2 100 (L/h)
Disinfection	
UV Disinfection System No. of UV channels No. of Modules No. of lamps per channel	 2 14 6
Outfall Sewer	
Re-aeration chamber and outfall sewer	1
Sludge System	
Sludge Storage Type Number	 Pond 1
Sludge Pond Aerator Type Number	 Surface Aerator 1
Supernatant Pump Number Capacity	 1 12 L/s (1,037 m^3/d) @ 11 m TDH

4.4.3 Effluent Quality Requirements

According to the Amended ECA No. 5933-C37KWJ, the effluent criteria for the Strathroy WWTF are shown in **Table 17** and **18**. The effluent loading limits are shown in **Table 19**.

Table 17: Effluent Objectives for the Strathroy Wastewater Treatment Plant

PARAMETERS	AVERAGING CALCULATOR	EFFLUENT OBJECTIVES
$cBOD_5$	Monthly Average Effluent Concentration	5 mg/L (April 1 – October 31) 10 mg/L (November 1 – March 31)
Total Suspended Solids (TSS)	Monthly Average Effluent Concentration	5 mg/L (April 1 – October 31) 10 mg/L (November 1 – March 31)
Total phosphorous (TP)	Monthly Average Effluent Concentration	0.3 mg/L (April 1 – October 31) 0.5 mg/L (November 1– March 31)
Total Ammonia Nitrogen (TAN)	Monthly Average Effluent Concentration	1.0 mg/L (April 1 – October 31) 3.0 mg/L (November 1 – March 31)
E. Coli	Monthly Geometric Mean Density	150 CFU/100 mL (1)
pH	Single Sample Result	6.5 – 8.5 inclusive
Dissolved Oxygen	Single Sample Result	>4.0 mg/L

(1) If the MPN method is utilized for E. coli analysis the objective shall be 150 MPN/100 ml

Table 18: Effluent Limits for the Strathroy Wastewater Treatment Plant

PARAMETERS	AVERAGING CALCULATOR	EFFLUENT LIMITS
$CBOD_5$	Monthly Average Effluent Concentration	10 mg/L (April 1 – October 31) 15 mg/L (November 1 – March 31)
Total Suspended Solids (TSS)	Monthly Average Effluent Concentration	10 mg/L (April 1 – October 31) 15 mg/L (November 1 – March 31)
Total phosphorous (TP)	Monthly Average Effluent Concentration	0.5 mg/L (April 1 – October 31) 1.0 mg/L (November 1 – March 31)
Total Ammonia Nitrogen (TAN)	Monthly Average Effluent Concentration	2 mg/L (April 1 – October 31) 5 mg/L (November 1 – March 31)
E. Coli	Monthly Geometric Mean Density	200 CFU/100 mL (1)
pH	Single Sample Result	Between 6.0 – 9.5 inclusive
Dissolved Oxygen	Single Sample Result	minimum 4.0

(1) If the MPN method is utilized for E. coli analysis the objective shall be 200 MPN/100 ml

Table 19: Effluent Loading Limits for the Strathroy Wastewater Treatment Plant

PARAMETER	AVERAGING CALCULATOR	EFFLUENT LOADING LIMITS (KG/D)
cBOD5	Annual Average Daily Effluent Loading	103.4
Total Suspended Solids	Annual Average Daily Effluent Loading	103.4
Total Phosphorous	Annual Average Daily Effluent Loading	6.1
Total Ammonia Nitrogen	Annual Average Daily Effluent Loading	27.8

4.4.4 Plant Design Parameters

In 2010 the Strathroy WWTF was re-rated to a capacity of 10,000 m^3/d. The design parameters for the plant at this re-rated capacity are summarized in **Table 20**.

Table 20: Design Parameters for the Strathroy Wastewater Treatment Plant

PARAMETER	VALUE
Rated Average Daily Flow, m3/d	10,000
Rated Peak Flow, m3/d	23,820
Design Peak Flow Factor	2.38
Effluent Quality Requirements	See Table 2 and 3 above for the ECA requirements

4.4.5 Historical Review

Annual Performance Reports for 2017 to 2021 for Strathroy Wastewater Treatment Plant were used to review the wastewater flow and effluent characteristics for the plant.

4.4.6 Wastewater Flow

The average day (ADF) and maximum daily flows (MDF) for the period 2017 to 2021, along with the 3-year average for the same period are shown in **Table 21**.

Table 21: Historical Wastewater Flows at the Strathroy Wastewater Treatment Plant

YEAR	ADF (M^3/D)	MDF (M^3/D)	PEAK FACTOR (MDF/ADF)
2017	5,276	6,830	1.3
2018	5,439	7,906	1.5
2019	5,121	6,835	1.3
2020	4,982,6	6,702	1.3
2021	4,652.4	6,637	1.4
5-year average	**5,094**	**6,982**	**1.4**

For the five-year period from 2017 to 2021, the average flow accounts for 51% of the rated capacity of the plant. This is also illustrated in **Figure 18**. Peak flow information was not provided for the plant, but in lieu of this data, the maximum day flows are also shown on **Figure 18.** As shown in **Figure 18**, over this five (5) year period, there were only six instances where the max day flow exceeded the rated capacity, however, it should be noted these max day flows were lower than the peak capacity of the plant (23,820 m^3/d).

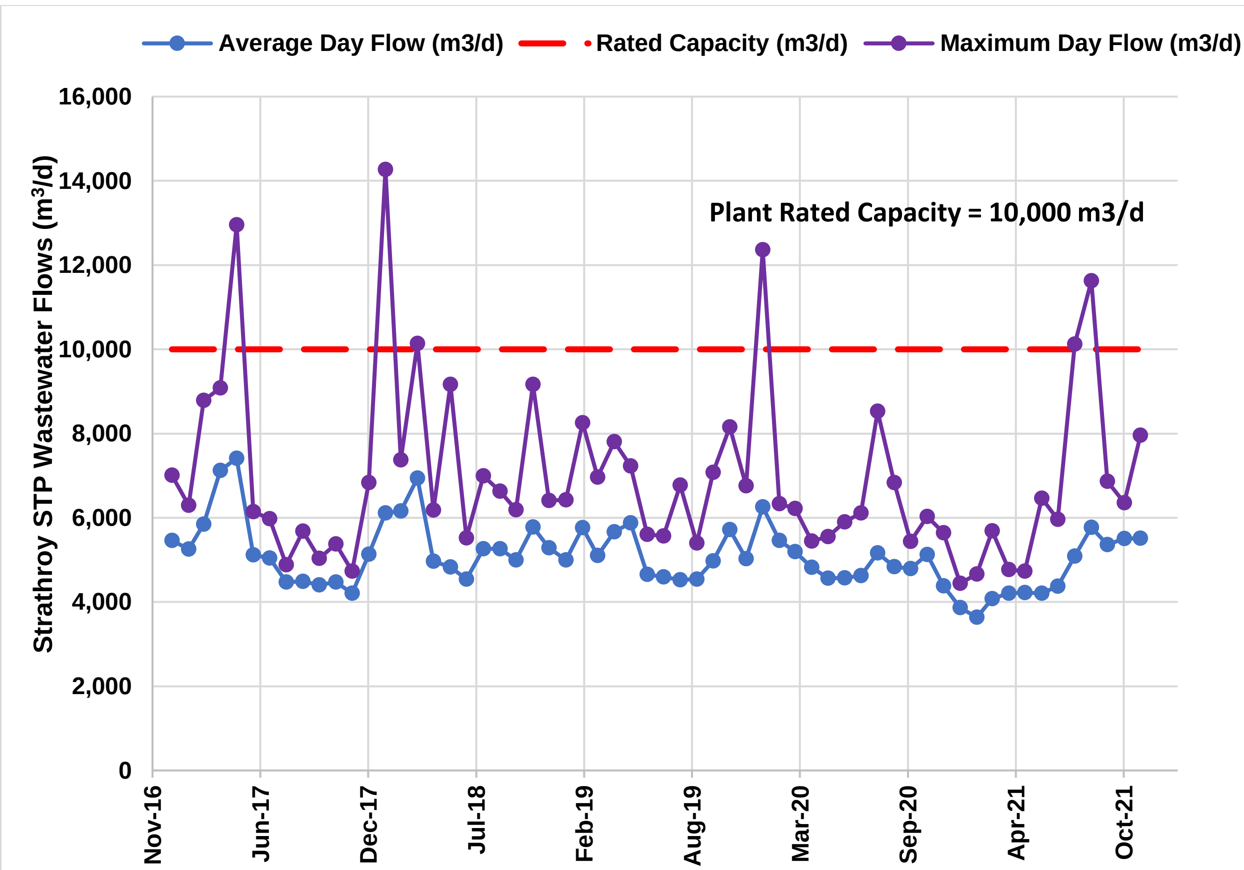

Figure 18: Average Day and Maximum Day Flows for the Strathroy WWTF for 2017 to 2021

4.4.7 Effluent Wastewater Characteristics

The effluent characteristics (concentrations and loadings) for the period 2017 to 2021 are outlined in **Table 22** and **Table 23**. **Figure 19** to **Figure 22** graphically illustrate the monthly averages for the effluent parameters (cBOD5, TSS, TP and Ammonia-N). The performance of the plant was assessed by comparing the concentrations and loadings to the effluent objectives and limits specified in the ECA for the Strathroy WWTF. It should be noted that the Annual Performance Reports appear to use the term total nitrogen (TN) and total ammonia-nitrogen interchangeably. Given that the ECA objectives and limits are based on the Total Ammonia-Nitrogen, this report assumes that the data provided in the annual reports are total ammonia-nitrogen.

Table 22: Final Effluent Concentration at the Strathroy WWTF for 2017 to 2021.

YEAR	CBOD5 (MG/L)		TSS (MG/L)		TP (MG/L)		NH_3-N (MG/L)	
	Apr-Oct	Nov-Mar	Apr-Oct	Nov-Mar	Apr-Oct	Nov-Mar	Apr-Oct	Nov-Mar
2017	3.91	6.68	5.31	7.72	0.30	0.43	0.25	2.58
2018	9.24	5.30	7.20	4.32	0.34	0.25	0.30	0.72
2019	9.07	11.68	6.0	11.88	0.28	0.37	0.25	1.37
2020	2.4	5.5	5.61	8.08	0.32	0.33	0.28	0.48
2021	2.19	2.90	4.6	6.92	0.28	0.38	0.32	0.3
5-year Average	5.36	6.41	5.75	7.78	0.3	0.35	0.28	1.09
Effluent Objective	**5.0**	**10**	**5.0**	**10**	**0.3**	**0.5**	**1.0**	**3.0**
Effluent Limit	**10**	**15**	**10**	**15**	**0.5**	**1.0**	**2.0**	**5.0**

Table 23: Final Effluent Loadings at the Strathroy WWTF for 2017 to 2021

YEAR	CBOD5 (KG/D)		TSS (KG/D)		TP (KG/D)		NH_3-N (KG/D)	
	Apr-Oct	Nov-Mar	Apr-Oct	Nov-Mar	Apr-Oct	Nov-Mar	Apr-Oct	Nov-Mar
2017	20.0	31.5	27.7	37.4	1.6	2.1	1.2	11.17
2018	47.6	29.7	37.4	24.2	1.8	1.4	1.5	3.8
2019	45.1	61.6	29.8	62.0	1.4	2.0	0.8	7.0
2020	11.43	30.58	26.62	43.01	1.54	1.79	1.33	2.67
2021	10.27	13.49	22.20	33.63	1.34	1.61	1.47	1.33
5-year Average	**26.88**	**33.38**	**28.74**	**40.04**	**1.55**	**1.79**	**1.27**	**5.17**
Effluent Load Limit	**103.4**	**103.4**	**103.4**	**103.4**	**6.1**	**6.1**	**27.8**	**27.8**

Review of the historical data shows that Ammonia-N concentrations are typically in compliance with the ECA objectives and limits. However, the data also showed that from 2018 to date, there have been exceedances in the final effluent for $cBOD_5$ TSS and TP effluent concentrations during both summer and winter periods. This is particularly evident in 2021.

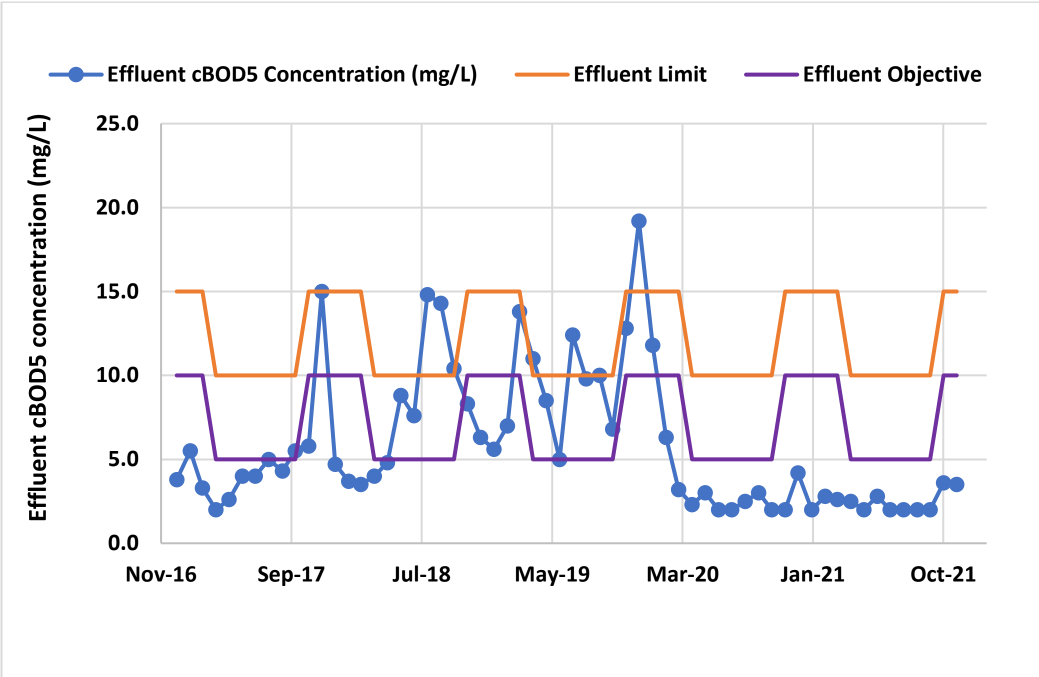

Figure 19: Monthly cBOD5 Effluent Concentration for the Strathroy WWTF for 2017 to 2021

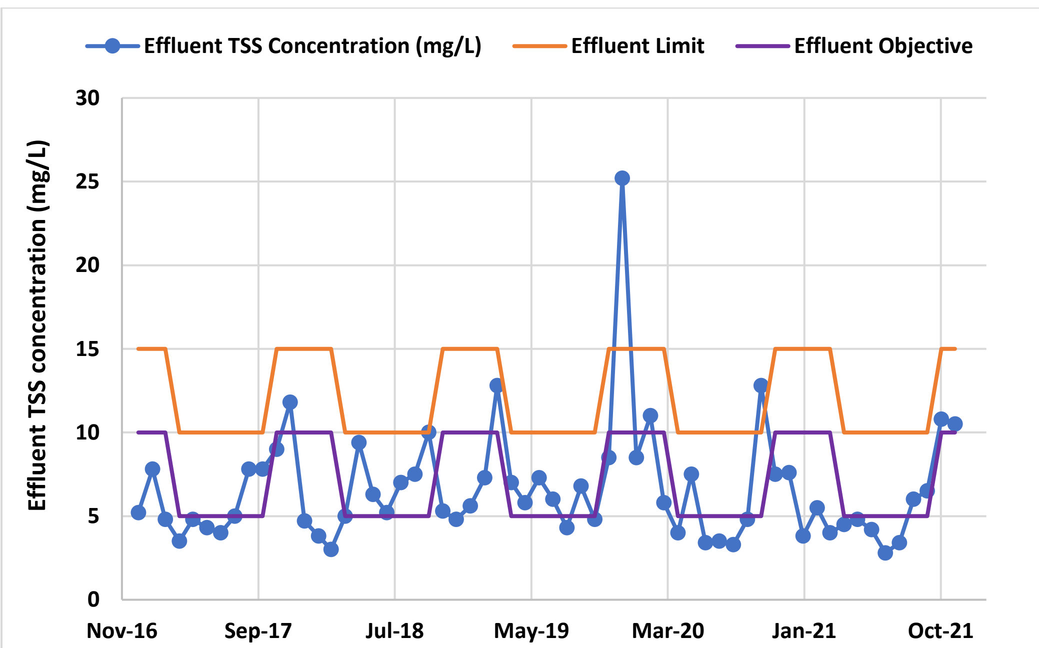

Figure 20: Monthly TSS Effluent Concentrations for the Strathroy WWTF for 2017 to 2021

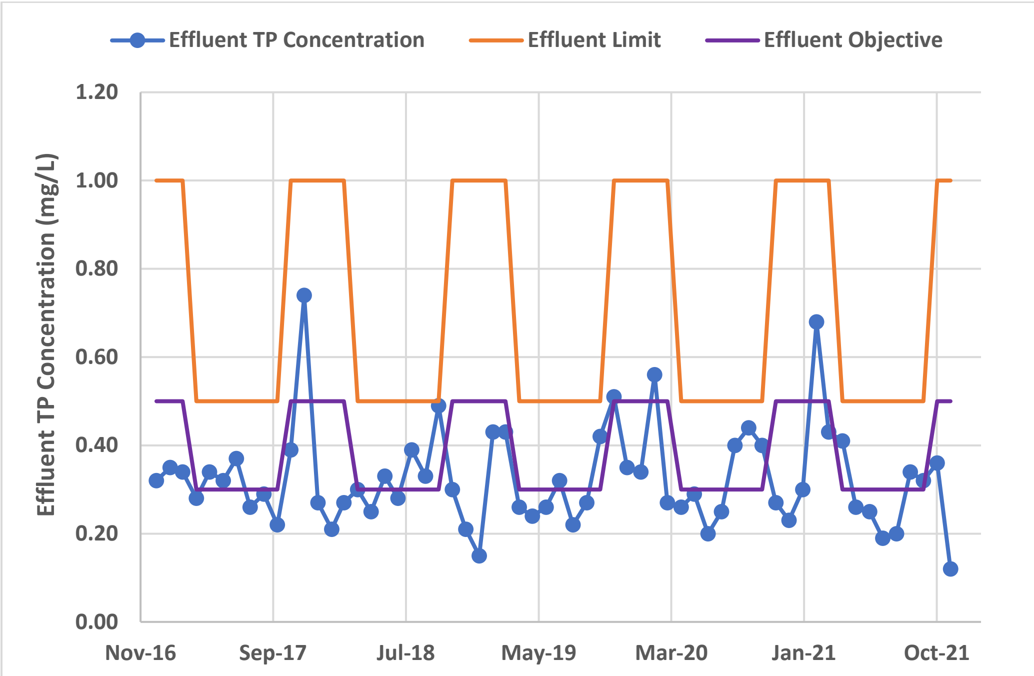

Figure 21: Monthly Effluent TP Concentration for Strathroy WWTF for 2017 to 2021

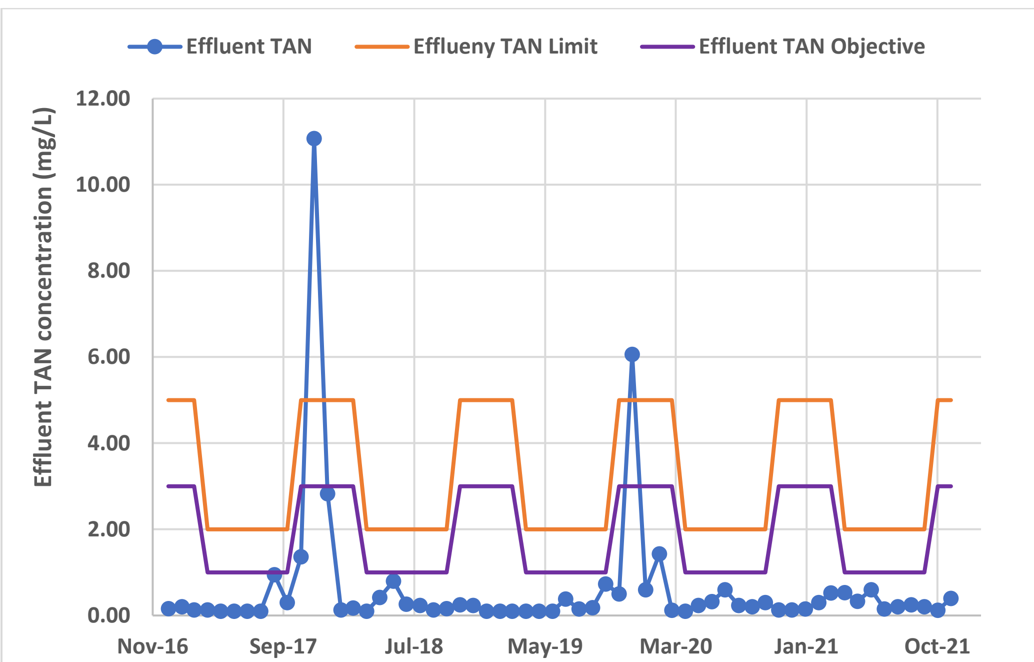

Figure 22: Monthly Effluent TAN Concentrations at Strathroy WWTF for 2017 to 2021

4.4.8 Sludge Generation

The waste activated sludge (WAS) generated from the wastewater treatment process is directed to the sludge storage pond (lagoon) which is sized to accommodate both WAS and any precipitation. According to the 2016 – 2019 Annual Performance Reports, this lagoon also received sludge from the Mount Brydges WWTF during this period. As of 2019, sludge from Mount Brydges is no longer accepted at the Strathroy WWTF. Sludge hauling and biosolids application is only done when required. **Table 24** below summarises the historical sludge volumes reported for the Strathroy WWTF.

Table 24: Reported Sludge Volumes Generated at the Mount Brydges WWTF and Strathroy WWTF

YEAR	SLUDGE VOLUMES (M^3)	
	From Mount Brydges to Strathroy	**At Strathroy**
2016	130	n.d.
2017	266	n.d.
2018	484	105,106
2019	204	95,819
2020	-	73,280

n.d. – no data available

4.5 Mount Brydges Sewage Treatment Plant

4.5.1 Treatment Process

The Mount Brydges Sewage Treatment Plant has a rated capacity of 825 m^3/d with a peak flow of 1,650 m^3/d, and services a population of approximately 1,950 persons. The plant has provision for future expansion to 1,180 m^3/d. Treatment at the plant includes primary and secondary treatment, tertiary treatment and final polishing before the treated effluent is discharged to a dry drainage ditch that drains into the Vermeersch Drain.

Incoming wastewater to the plant is first screened at the Main and Northwest W Sewage Pumping Station. The screened effluent flows to a four-chamber inlet flow splitter box. Two of these four chambers direct the incoming flow to the two rotating biological contactor (RBC) units. The remaining two are available for future expansion to two additional RBC units.

Each RBC unit is equipped with four stages. The first two stages are aerobic while the last two are anaerobic. Wastewater from the splitter box flows through the four stages providing reduction solids and nutrient concentrations. The mixed liquor from the RBC units then flows by gravity to two rectangular mechanical secondary clarifiers. Each clarifier is equipped with a chain and flight sludge scraper to remove the settled sludge. Alum addition at the inlet to the secondary clarifier is used for phosphorous removal. Sludge is removed from the bottom of the secondary clarifiers using submersible transfer pumps and pumped to the primary tanks of the RBC units. The clarified effluent from the secondary clarifiers flows by gravity through an effluent weir/trough to the filter feed basin.

Three tertiary continuous backwash filters are used for tertiary treatment of the effluent from the secondary clarifier. The filters are single media, continuous backwash, up-flow tertiary filters and the clarified water from the filter feed basin is pumped into the lower section of the sand media bed. The water flows upward through the filter bed and the filtered water exits the top of the filter over an effluent weir plate. Alum addition into the pumped flow to the lower section of sand filter is used for phosphorous removal. Rejected water from the filters flow by gravity to the reject water basin. This basin is equipped with duty/standby pumps to pump the water to an injection point upstream of the inlet flow splitter box.

From the tertiary filters, the filtered effluent flows by gravity through a flowmeter to the UV disinfection system. The UV system is comprised of two (2) banks with a total of 16 lamps per bank. The UV system automatically cycles between the two banks providing a duty/standby operation when flows are less than 2,100 m^3/d. When the flow exceeds 2,100 m^3/d, both banks are operated simultaneously.

The Mount Brydges WWTF is also equipped with an internal bypass system that comprises a 7,000 m^3/d portable submersible pump and a manhole located along the feed pipe to the filters. This arrangement allows wastewater to temporarily bypass the filter feed pumps and the upstream process to be redirected to filter flow splitter. Wastewater can also bypass the UV chamber by opening a gate valve just upstream of the strainer and the filter flow splitter. This water would be directed to the outfall manhole.

4.5.2 Process Design Summary

Table 25: Mount Brydges Wastewater Treatment Plant Process Design Summary

UNIT PROCESS	DESIGN PARAMETER
Rotating Biological Contactors	
Rotating Biological Contactors	
Number	2
Capacity (total)	825 m3/d
Media Surface Area (total)	27,282 m2
No. of stages (each RBC)	4
Diameter	3.7 m
Type of media	HDPE
No. of cover (each RBC)	1
Type of cover	Fiber-glass
Future Rotating Biological Contactor Tank	
Number	1
Type	Concrete
Dimension	9.8 m (L) x 4.7 m (W) x 4.5 m (H)
Capacity	207.3 m3
Final Clarifier	
Final Clarifier	
Number	2
Capacity	Peak flow of 3,260 m3/d
SWD	2.6 m
Length x Width	12 m x 3.75 m
No. of Sludge Collector (each clarifier)	1
Type of Sludge Collector	Chain and flight
Filter Pump Chamber	
Number	1
Filter Feed Pump	

UNIT PROCESS	DESIGN PARAMETER
Number	2
Capacity	65 L/s @ 9.2 m TDH
Control	Variable frequency drive (VFD)
Effluent Filtration System	
Flow Splitter box	
Capacity	Peak flow of 5,595 m3/d
No. of Overflow Weirs	3
Width of Weir (each)	400 mm
Effluent Filtration System	
Number	1
Continuous Backwash Filter	
Type	Single media, continuous backwash, up-flow tertiary filter
Media	Sand
Number	3 (2 duty and 1 standby)
Diameter	3100 mm
Depth	6200 mm
Capacity	Peak flow of 3,260 m^3/d
Filtration Surface Area (each)	7.54 m2
Filter Reject Pump	
Number	2 (1 duty and 1 standby)
Pump Capacity	8.3 L/s @ 9.2 m TDH
UV Disinfection System	
UV Disinfection Unit	
Number	1
dimensions (LxW)	3.8 m (L) x 0.6 m (W)
No. of UV banks	2
No. UV Modules (for each UV bank)	4
No. UV Lamp (for each module)	8
Type of Lamp	Low pressure, high Intensity
Target UV Dose	30 mJ/cm^2 at a minimum 65% UVT254 nm
Chemical Storage and Feed System	
Soda Ash (CaCO3) Mixing Tank	
Number	1
Capacity	500 L
Soda Ash Solution Storage Tank	
Type	Polyethylene
Number	1
Capacity	5,600 L
Soda Ash Feed Pump	
Type	ProMinent Sigma/2 (S2Ca) Motor Driven Pump
Number	2
Pump Capacity	2 – 264 L/hr
Alum Storage Tank	
Type	Polyethylene
Number	1
Volume	3,600 L
Chemical metering Pump	
Type	ProMinent gamma/L Solenoid Dosing Pump
Number	2

UNIT PROCESS	DESIGN PARAMETER
Capacity	18.43 L/h
Effluent Outfall	
Effluent Outfall Sewer	
Number	1
Length	7 m
Diameter	250 mm
Effluent Discharge Point	
Number	1
Length	50 m
Diameter	300 mm
Final discharge point	Dry ditch that drains to Vermeersch Drain
Standby Power	
Standby Power Diesel Generator	
Number	1
Capacity	250 kW

4.5.3 Effluent Quality Requirements

The effluent objectives and the effluent limits as stated in the Certificate of Approval (CoA) Number 7788-8BJRL8 issued on January 26, 2011, are shown in **Table 26** and **Table 27** respectively.

Table 26: Mount Brydges WWTF Effluent Quality Objectives

EFFLUENT PARAMETER	CONCENTRATION OBJECTIVES	
	[1]Freezing	[2]Non-Freezing
cBOD5	10	5.0
Total suspended solids (TSS)	10	5.0
Total phosphorous (TP)	0.8	0.3
Total ammonia nitrogen	3.0	1.0
Chlorine residual	0.0	0.0
E. coli	---	150 E. Coli /100 mL
pH	pH 6.5 to 8.5	

[1]Freezing refers to temperatures =< 5 deg. C

[2]Non-freezing refers to temperatures > 5 deg. C

Table 27: Mount Brydges WWTF Effluent Quality Limits

EFFLUENT PARAMETER	CONCENTRATION OBJECTIVES	
	[1]Freezing	[2]Non-Freezing
cBOD5	15.0	10.0
Total suspended solids (TSS)	15.0	10.0
Total phosphorous (TP)	1.0	0.5
Total ammonia nitrogen	5.0	3.0
Dissolved oxygen	5.0	5.0
E. coli	-----	200 E. Coli/100 mL
pH	pH 6.0 to 9.5 inclusive	

[1]Freezing refers to temperatures =< 5 deg. C

[2]Non-freezing refers to temperatures > 5 deg. C

4.5.4 Plant Design Parameters

According to the Mount Brydges Operation Manual, the design parameters for the WWTF are shown in **Table 28** below.

Table 28: Plant Design Parameters for Mount Brydges WWTF

PARAMETER	VALUE
Rated Average Daily Flow, m^3/d	825
Rated Peak Flow, m^3/d	1,650
Effluent Quality Requirements	See Table 11 and Table 12 above for the ECA requirements

4.5.5 Historical Review

Data from the Annual Performance Reports for 2017 to 2021 for Mount Brydges WWTF were used to provide a review of the wastewater flow and effluent characteristics for the plant.

4.5.6 Wastewater Flow

The average day (ADF) and maximum daily flows (MDF) for the period 2017 to 2021, and the 5-year average for the same period are shown in **Table 29**.

Table 29: Historical Wastewater Flows at the Mount Brydges WWTF

YEAR	ADF (M^3/D)	MDF (M^3/D)	PEAK FACTOR (MDF:ADF)
2017	124	150	1.21
2018	167	211	1.27
2019	201	259	1.29
2020	207	261	1.26
2021	273	347	1.27
5-year average	**194**	**246**	**1.26**

The data shows the flows to the plant have been gradually increasing over the five years from 2017 to 2021. Based on the average flow for 2021, the ADF flows account for 33% of the rated capacity of the plant. **Figure 23** shows the both the average day and max day flows which are much lower than the rated capacity. Peak hourly and peak instantaneous flow information was not provided for the plant, as such information on historical peak flow events could not be presented.

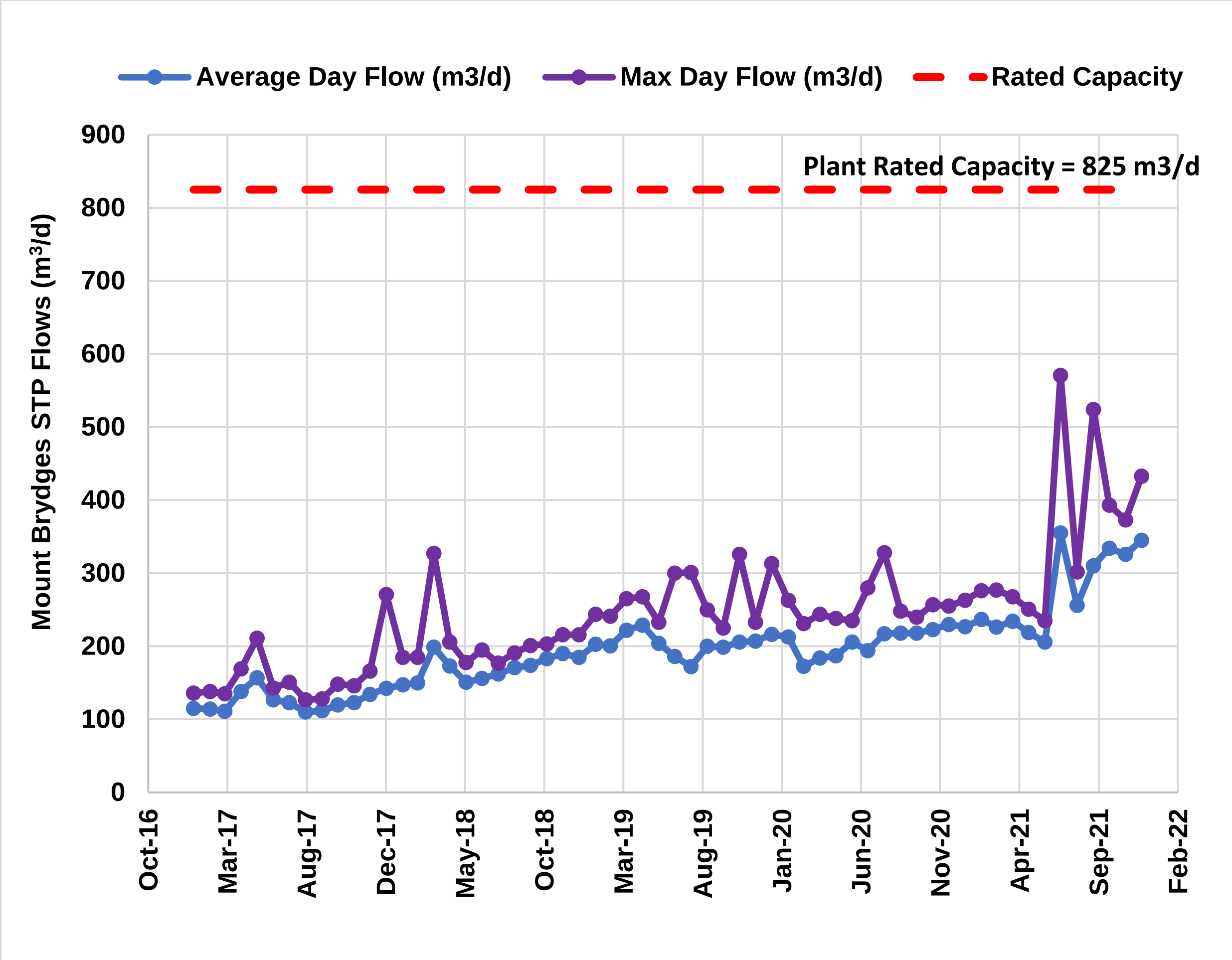

Figure 23: Historical Average Day and Maximum Day Flows at the Mount Brydges WWTF for 2017 to 2021

4.5.7 Effluent Wastewater Characteristics

The effluent characteristics for the period 2017 to 2021 are outlined in Table 30 with the average concentrations for the parameters of interest. The effluent loadings are shown in Table 31. The performance of the plant was assessed by comparing the concentrations and loadings to the effluent objectives and limits specified in the ECA for the Strathroy WWTF. It should be noted that the Annual Performance Reports appear to use the term total nitrogen (TN) and total ammonia-nitrogen interchangeably. Given that the ECA objectives and limits are based on Total Ammonia-Nitrogen, this report assumes that the data provided in the annual reports are total ammonia-nitrogen. The historical data shows that the effluent has been in compliance with the effluent discharge limits for all parameters except ammonia-nitrogen and cBOD5.

Table 30: Effluent Concentrations for the Mount Brydges WWTF for 2017 to 2021

YEAR	CBOD5 (MG/L)	TSS (MG/L)	TP (MG/L)	NH_3-N (MG/L)
2017	3.87	8.1	0.30	1.5
2018	5.97	7.4	0.27	3.9
2019	6.45	10.9	0.36	6.2

YEAR	CBOD5 (MG/L)	TSS (MG/L)	TP (MG/L)	NH_3-N (MG/L)
2020	5.96	12.8	0.32	4.4
2021	3.55	8.6	0.31	2.8
5-year Average	5.2	9.5	0.3	3.8
Effluent Limit	10 (April – Nov) 15 (Dec – Mar)	10 (April – Nov) 15 (Dec – Mar)	0.5 (Apr – Nov) 1.0 (Dec – Mar)	3.0 (Apr – Nov) 5.0 (Dec – Mar)
Effluent Objective	5 (April – Nov) 10 (Dec – Mar)	5 (Apr – Nov) 10 (Dec – Mar)	0.3 (Apr – Nov) 0.8 (Dec – Mar)	1.0 (Apr – Nov) 3.0 (Dec – Mar)

Table 31: Effluent Loadings at the Mount Brydges WWTF for 2017 to 2021

YEAR	CBOD5 (KG/D)	TSS (KG/D)	TP (KG/D)	NH_3-N (KG/D)
2017	0.46	1.02	0.02	0.20
2018	0.97	1.18	0.04	0.67
2019	1.31	2.21	0.07	1.26
2020	1.22	2.62	0.07	0.82
2021	0.94	2.21	0.08	0.75
5-year Average	0.98	1.85	0.06	0.74
Effluent Limit	10 (April – Nov) 15 (Dec – Mar)	10 (April – Nov) 15 (Dec – Mar)	0.5 (Apr – Nov) 1.0 (Dec – Mar)	3.0 (Apr – Nov) 5.0 (Dec – Mar)

4.5.8 Sludge Production

From 2016 to the beginning of May 2019, the sludge from the Mount Brydges WWTF was transported to the sludge lagoon located at the Strathroy WWTF. From May 2019 to the end of 2021, sludge was hauled to the City of London. Based on the Annual Performance Reports for the Mount Brydges WWTF, the volume of sludge generated and transferred to Strathroy WWTF and City of London is shown in **Table 32**.

Table 32: Sludge Production for Mount Brydges WWTF and Strathroy WWTF

YEAR	VOLUME OF SLUDGE TRANSFERRED TO CITY OF LONDON (M^3)	VOLUME OF SLUDGE TRANSFERRED TO STRATHROY WWTF (M^3)
2016	n/a	130
2017	n/a	266
2018	n/a	484
2019	708	204
2020	900	-

4.6 Future Flows

The Sanitary Sewer Analysis was used to inform the future average dry weather and peak wastewater flows to the Mount Brydges WWTF and Strathroy WWTF.

For the purpose of assessing the capacity of the process units, the maximum day flow was estimated using a peaking factor based on the historical MDF: ADF flows received for 2017 to 2021. The peak hour flow was estimated by applying a factor of 1.44 to the max day flow in accordance with WEF MOP 8. These values are shown in the **Table 33**. For this report, the Peak flow in Table 19 is equivalent to the Peak Instantaneous Flow.

Table 33: Future Wastewater Flow to the Mount Brydges WWTF

PARAMETER	STRATHROY WWTF	MOUNT BRYDGES WWTF
Current Rated Capacity, m3/d	10,000	825
Future Average Dry Weather Flow (ADWF), m^3/d	5,429	1,059
Future Max Day Peaking Factor	1.4	1.3
Future Max Day Flow (MDF), m^3/d	7,600	1,377
Future Peak Hourly Flow (PHF), m^3/d	10,945	1,983
Future Peak Flow, m3/d	31,678	7,659

4.7 Process Units Design Basis

In Ontario, wastewater treatment plants are typically designed in accordance with the Ministry of Environment Design Guidelines for Sewage Works. **Table 34** shows the recommended design basis for each of the major process units as applicable to the Mount Brydges WWTF and Strathroy WWTF.

Table 34: MOE Recommended Design Basis for Process Units

PROCESS	DESIGN BASIS
Screening	Peak instantaneous flow (PIF)
Grit Removal	Peak Hour flow
Primary Treatment	Peak Daily flow
Aeration with nitrification	Average BOD5 loading based on average day flow Peak daily TKN loading (based on design peak daily flow)
Secondary sedimentation	Peak hourly flow and Peak daily solids Loading
Disinfection	Peak Hourly flow
Effluent filtration	Peak Hourly flow
Sludge treatment	Maximum monthly mass loading and flows

4.8 Strathroy WWTF Process Capacity Assessment

4.8.1 Preliminary Treatment

Preliminary treatment at the Strathroy WWTF is provided in the inlet works which comprises a flow splitter box and a mechanically cleaned bar screen which is located in the screening channel. According to the ECA, the screen has been sized to accommodate the peak flow of 23,280 m^3/d.

Based on the projected future peak flow for the plant of 31,678 m^3/d, the screens may not have the capacity to handle a peak flow through the plant.

4.8.2 Secondary Treatment – Aeration

Secondary treatment is achieved using an extended aeration process in an aeration basin equipped with fine bubble diffusers, secondary clarification and a cloth disk media filter. The aeration basin provides an overall volume of 8,560 m^3 and the system is operated with a dissolved oxygen (DO) concentration in the range of 1.2 to 3.0 mg/L based on the SCADA Log reports. The mixed liquor in the aeration tanks is maintained in the range of 2,000 to 3,700 mg/L. This is in accordance with the typical design values for an extended aeration system of 2,000 – 5,000 mg/L for MLSS.

Return activated sludge from the aeration tanks is returned to the aeration basin. For the period 2018 to 2020, the RAS flow has ranged from 3000 m^3/d to 4500 m^3/d accounting for approximately 70 – 80% of the incoming wastewater flow to the plant. The RAS:ADF ratio is in accordance with the recommended design value of 50 – 150% of the incoming wastewater flow. A portion of the RAS stream is removed as waste activated sludge (WAS) with a WAS:RAS ratio of 11%. Influent load data was not available for the plant as such the organic loading rate (OLR) could not be reviewed.

4.8.3 Secondary Treatment – Clarification

The clarification system is comprised of the two secondary clarifiers followed by post-secondary filters. Each clarifier has a diameter of 22.5 m, a side water depth of 4m and a total surface area of 397 m^2. With two units installed, the clarifiers provide a total surface area 795 m^2. According to the MOE Design Guidelines for Sewage Works, with an extended aeration treatment process, the clarifier should not exceed the design surface overflow rate (SOR) of 40 m^3/m^2.d based on the peak hourly flow through the plant. Based on the projected future PHF of 10,945 m^3/d, the SOR for the clarifiers was estimated to be 13.8 m^3/m^2.d. This SOR is below the recommended design criteria which indicates the clarifiers will have sufficient capacity for the future flow.

The cloth disk media filters provide an overall capacity of 26,496 m^3/d. This capacity exceeds the future PHF of 10,945 m^3/d and is expected to have adequate capacity at the future flows.

4.8.4 UV Disinfection

The UV disinfection system which is comprised of 14 modules, each with six lamps, would have been sized to accommodate the design peak hourly flow in accordance with the MOE Design Guidelines. This design PHF of 20,000 m^3/d exceeds the projected future peak hourly flow of 10,945 m^3/d and as such it is expected that the existing UV system has adequate capacity to accommodate the future flows through the plant.

4.8.5 Sludge Treatment

Solids processing at the Strathroy WWTF involves pumping of the waste activated sludge from the aeration process to a sludge storage pond (lagoon) that is equipped with a surface aerator. The supernatant from the lagoon is pumped from the pond to the aeration tanks for treatment.

Information on the capacity of the pond, frequency of hauling on an as needed basis and quality of the sludge was not available.

4.8.6 Summary

In accordance with the MOE Design Guidelines, the capacity of the process units are based on different flow parameters such as average day flow, peak hourly or peak instantaneous flows (see Table 35). To compare the different units the equivalent average day flow was calculated using the peaking factors associated with the future flows. The summary of the capacity assessment is shown in **Table 35** while the process capacity chart is shown in **Figure 24**.

Table 35: Capacity Assessment Summary for Strathroy WWTF

TREATMENT UNIT	DESIGN BASIS	AVERAGE DAY FLOW (M^3/D)	MAXIMUM DAY FLOW (M^3/D)	PEAK FLOW (M^3/D)	EQUIVALENT AVERAGE DAY FLOW (M^3/D)
Inlet Works - Screening	PIF			23,280	3,990
Aeration Basin	ADF	10,000			10,000
Secondary Clarifiers	PHF			20,000	9,921
Cloth Media Disk Filter	PIF			26,496	13,143
UV Disinfection	PHF			20,000	9,921

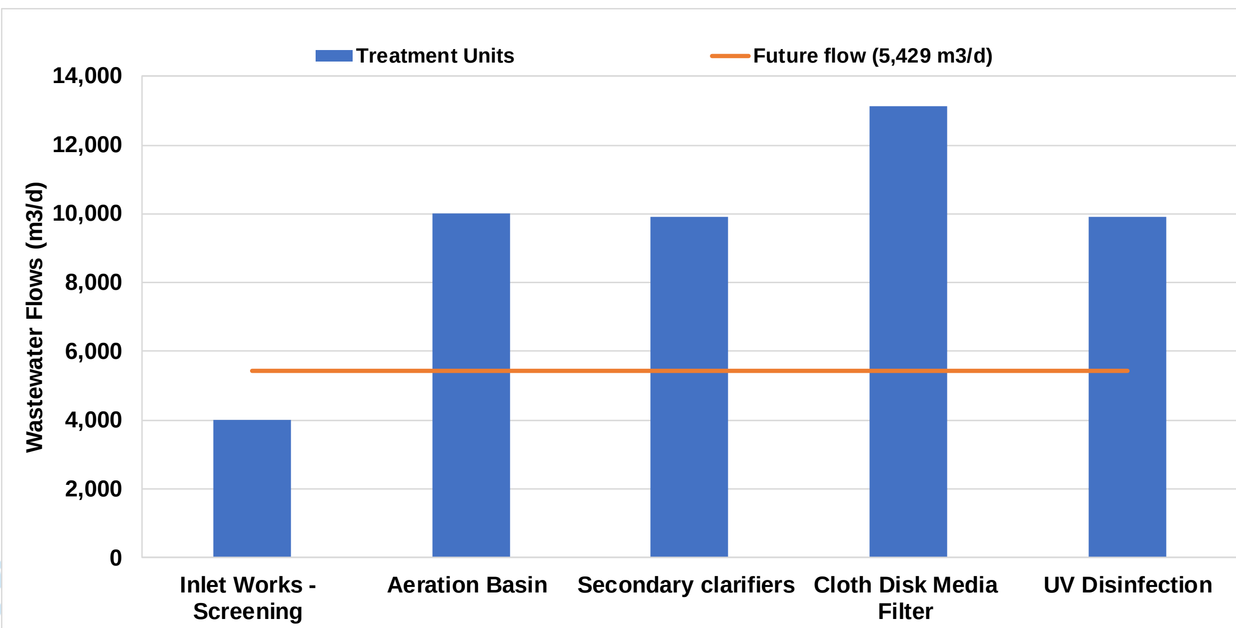

Figure 24: Process Capacity Chart for Strathroy WWTF

4.8.7 System Upgrade & Expansion Considerations

4.8.7.1 Operational Challenges

The Strathroy WWTF is currently operating at 53% of the rated capacity (10,000 m^3/d) of the plant. The plant experiences seasonal fluctuations in the influent flow and quality due to discharges from a food processing facility. Historically, this has resulted in exceedances in the concentrations of BOD_5, TSS and TP in the final effluent when compared with the ECA objectives and limits. In addition to these exceedances, the plant had a number of maintenance issues over the years inclusive of replacement of the blower air line, modifications to the clarifiers to address short circuiting and repairs of the clarifier control mechanism (the underwater sluice gate). There were also reports of a major plant failure in 2017, 2018 and 2019. Attempts have been made to adjust the approach used for maintenance activities at the plant by implementing a more proactive approach. However, there are some ongoing challenges:

1. Inability to clean and properly maintain the aeration lagoon due to lack of redundancy in the system. A single lagoon is used for aeration. Due to the membrane liner in the lagoon, cleaning of the system is also considered to be a high-risk activity.
2. The lack of maintenance causes the accumulation of solids along the side and bottom of the basin
3. There have been a number of exceedances of cBOD5, TSS and TP effluent limits in recent years (see Table 8 and Figure 1).
4. Poor energy efficiencies in the operation of the plant due to the power consumption requirements for the aeration lagoon
5. Flows to the plant are impacted by level of precipitation in the area and the period of the year during which the precipitation occurs. In response, the Municipality has undertaken a proactive approach to reduce extraneous flow by inspecting the sanitary sewer, conducting spot repairs or lining the sewers, and the installing sub-surface drainage that would redirect groundwater from the sanitary sewer to the storm sewer system.

Other than the challenges highlighted above, the Annual Performance Reports for the period 2016 to 2021 have only reported routine maintenance activities at the Strathroy WWTF.

4.8.7.2 Expansion and Upgrades

The plant is currently operating at 53% of its rated capacity. Additionally, the capacity review has indicated that all process components, except for the screens in the inlet works, have sufficient capacity to meet the future flow to 2046. As such, expansion of the inlet headworks is required to accommodate additional capacity is required. The ongoing operational, maintenance, and energy efficiencies issues can be addressed by upgrading specific components of the plant as follows:

1. Based on the capacity review, increase the capacity of the inlet screens such that they are sized to accommodate the future peak flow of 31,678 m^3/d.
2. Construction of an additional lagoon to provide redundancy and allow the existing aeration lagoon to taken offline for cleaning and maintenance.
3. Replacement of the existing blowers for more energy efficient options.

To improve overall plant operation, the Municipality may undertake additional measures that are not within the purview of the plant, some of which are already being considered by the Municipality. These include:

4. Ensure the implementation of the operational improvements at pre-treatment plant at the food processing facility to minimize potential upsets to the Strathroy treatment plant. These would include:
    a. Reduce the amount of soil on the produce before they are transported for processing at the food facility. This applies specifically to the carrots.
    b. Reduce and/or eliminate soil waste by redesigning the primary drain system
    c. Reduce water usage
    d. Reduce the infiltration of stormwater to the wastewater system
5. Implementation of sewer use by-laws for industrial discharges to the wastewater collection system
6. Possible implementation of flow monitoring and control system at the food processing facility to assist with the sewer by-laws and also to inform the plant in advance of potential high strength waste streams that could negatively impact operation of the plant.

4.9 Mount Brydges WWTF Process Capacity Assessment

4.9.1 Secondary Treatment

Wastewater flow into the Mount Brydges WWTF is directed to the rotating biological contactor (RBC) units for secondary treatment. The two existing units were sized for the current rated capacity of 825 m^3/d and provides a media surface area of 27,282 m^2. There is provision for a third RBC unit with a concrete tank having a capacity of 207.3 m^3. If included, this would provide an overall capacity of 1032 m^3. However, even with a third unit, the RBCs would not provide adequate capacity for the future flow of 1,059 m^3/d to 2046.

4.9.2 Secondary Clarifiers

Secondary clarification at the plant is achieved using two rectangular clarifier with the dimensions of 12m in length, 3.75 m wide and 2.6 m side water depth (SWD). Th clarifiers have been sized to accommodate a peak flow of 3,260 m^3/d. According to the MOE Design Guidelines, final clarifiers located after rotating biological contactors should not exceed a surface overflow rate (SOR) of 50 m^3/m^2.d and the solids loading rate of 240 kg/m^2.d. The future peak hourly flow for the Mount Brydges plant was estimated to be 1,983 m^3/d. This provides an SOR of 22 m^3/m^2.d which is lower than the recommended SOR level. As such, the secondary clarifiers will provide sufficient hydraulic capacity for the future flows. It is also noted that with one clarifier out of service, the SOR would 44 m^3/m^2.d. Since this SOR is also lower than the recommended SOR, it indicates that one clarifier is able to handle all the flows coming through the plant should one clarifier be taken off-line.

4.9.3 Effluent Filtration

The effluent filtration system is comprised of a flow splitter box and a three continuous backwash filters that operated with two duty and one standby configuration. Each filter has a surface area of 7.54 m^2 and has been designed for a peak flow of 3,260 m^3/d. According to Section 15.2.4 of the MOE Design Guidelines, the filters should be sized to accommodate the peak hourly flows. The filtration rate of deep bed filters (i.e., filters with a media of 1.2 to 1.8 m) should have a filtration rate that does not exceed 3.3 L/(m^2.s) based on a total available filter area with one unit out of service.

The two duty filters provide a total filter area of 22.62 m^2. With one unit out of service and using the future peak hour flow of 1,983 m^3/d, the filtration rate was calculated to 3.0 L/(m^2.s).This indicates the filtration system has adequate capacity for the future flows, even with one unit of service.

4.9.4 UV Disinfection

UV disinfection is achieved by a single unit located in a 0.6 m wide by 3.8m long channel. The unit has two banks of low pressure, high output UV lamps, with each having four modules. Since each module contains eight lamps, the unit has a total of 32 lamps. The unit has been sized for a peak flow of 4,200 m^3/d and will provide a 30 m/cm^2 of UV dose at a minimum UVT of 65%.

According to the MOE Design Guidelines, a UV system should be designed to accommodate the peak hourly flow in the plant. Given that the future peak hour flow of 1,983 m^3/d is lower than the design capacity of the unit, it was determined that the UV system will have sufficient capacity for the future flows.

4.9.5 Sludge Treatment

Sludge streams at the plant are generated from the RBC units and the final clarifiers. The sludge from the RBC units is removed using vacuum trucks and hauled offsite for treatment and disposal. A chain and flight sludge collector is used to remove the sludge from the bottom of the clarifier.

4.9.6 Summary

The process units at the plant would have been sized based on average day flow, peak hourly flow and peak instantaneous flow data in accordance with the MOE Design Guidelines. In assessing the capacity of the units to handle the future flows at the plant, the equivalent average day flow for each unit was calculated using the peaking factors associated with the future flows. The summary of the capacity assessment is shown in **Table 36** while the process capacity chart is shown in **Figure 25**. Based on the capacity review, the existing process units except the RBC units will have sufficient capacity for the future flows.

Table 36: Capacity Assessment Summary for the Mount Brydges WWTF

TREATMENT UNIT	DESIGN BASIS	AVERAGE DAY FLOW (M^3/D)	PEAK FLOW (M^3/D)	EQUIVALENT AVERAGE DAY FLOW (M^3/D)
Rotating Biological Contactors (RBCs)	ADF	825		825
Final clarifiers (SOR)	PHF		3,260	1,741
Filtration system - Tertiary filters	PHF		3,260	1,741
UV Disinfection	PHF		4,200	2,243

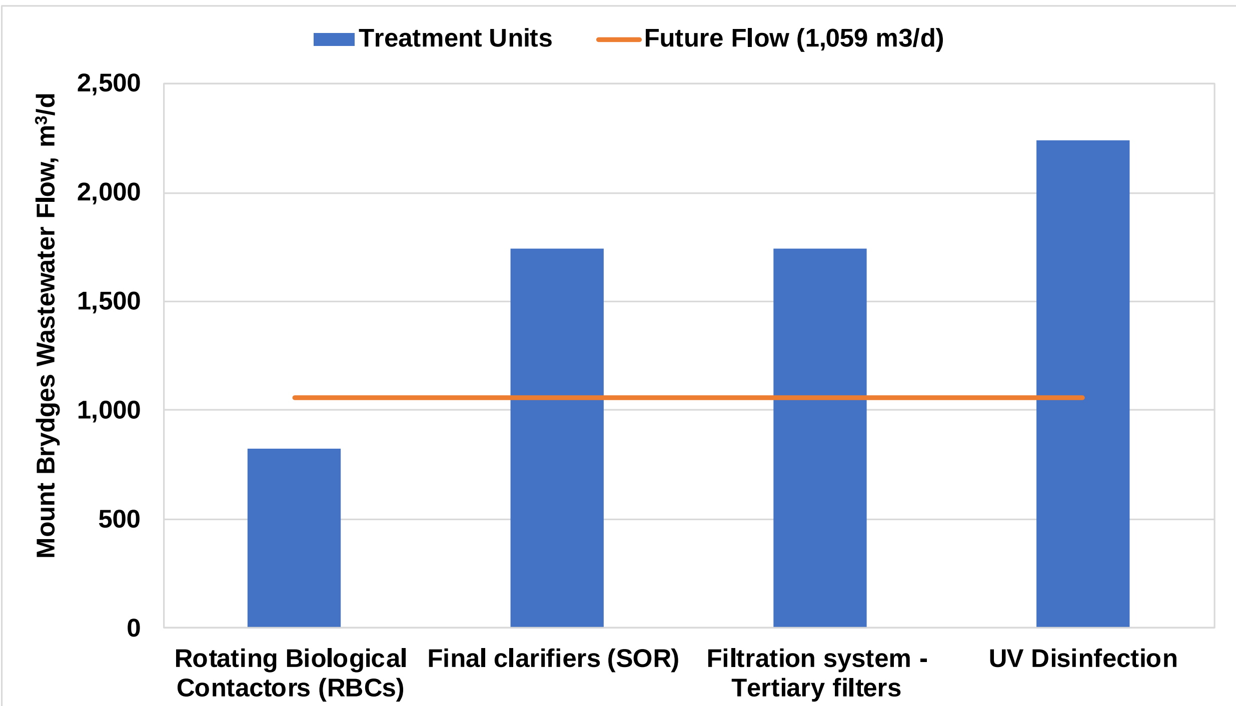

Figure 25: Process Capacity Chart for Mount Brydges WWTF

4.9.7 System Upgrade & Expansion Considerations

4.9.7.1 Challenges

According to the information provided for the Mount Brydges plant, operational challenges the plant has experienced over the years include:

1. Insufficient flushing of the sand filter which causes a reduction in the efficiency of the filtration process
2. Elevated suspended solids concentration from the RBC due to starvation of biological growth in the RBC units. This starvation occurs due to low flows to the plant.

3. Elevated ammonia levels due to a reduced nitrification in the RBC.
4. Cyclical influent flows to the plant due to the pump cycle of the Main Pumping Station that feeds the plant. This has resulted in uneven flow cycles causing an overloading and underloading the plant, which has caused a reduction in the effectiveness of the overall treatment process.
5. Mechanical failure of the RBC unit

A major plant failure was reported in February 2020 with failure of the drive gear box for the RBC units. This has since been addressed with the repair of the box, but it is reported that the plant still experiences elevated solids concentration that exceeds the effluent discharge limit.

In light of the foregoing challenges, the major maintenance requirements that have been identified, and in some cases committed to, for Mount Brydges would include:

1. Replacement of the mechanical gate control valve with a pinch valve at the Main Street SPS to reduce flow rates to the plant to assist in managing the cyclical flows to the plant. This was recommended as a temporary measure until a permanent solution is installed to address the cyclical flows. The pinch valve has since been implemented
2. Modification of the piping/flowmeter at the Main Pumping Station to improve flow distribution to the plant.
3. Ensuring adequate spare essential equipment are kept in stock for repairs.
4. Upgrade to the metal platforms around the filters to provide safer access

4.9.7.2 Expansion and Upgrades

The capacity review has shown that a future average day flow of 1,059 m^3/d, all the process units have sufficient capacity except the RBC units. Although provision is provided for a third unit, this will not provide the required capacity. As such, an expansion of the biological treatment process will be required.

The Mount Brydges WWTF Treatment Upgrades Study Conceptual Design Report completed in December 2021 recommended that an extended aeration process should be installed to replace the RBCs. According to the Design Report, in order to address a future capacity of 2,000 m^3/d, three sub-alternative were considered.

1. Expansion of the extended aeration process
2. IFAS retrofit of the system
3. MBR retrofit of the system.

Of all these alternatives, it was recommended that the RBCs should be replaced with an extended aeration process as this would address the mechanical issues related to the RBC units, address process issues and would enable the plant to meet the effluent discharge limits consistently. For this alternative, the existing RBC tanks could be used as the extended aeration tanks eliminating the need to construct additional tanks. This option also provided ease of constructability or phasing of the works and lower capital costs. Given that the current RBCs would not provide adequate capacity for the future flow, replacement of the RBCs with an alternative treatment approach is a suitable approach. The extended aeration process eliminates the need for primary clarification while capitalising on the using the existing RBC tanks. It is noted that Municipal Council has agreed to the design of these works as part of its 2022 Capital Budget.

The Municipality's current Official Plan (Section 4.4.3.2 and 3.4.3.3) requires that when 90% of the design capacity of the sewage treatment plant is reached within Mount Brydges and Strathroy, respectively, the process of expanding the sewage treatment plant to meet future needs will be initiated. Considering the rate of growth occurring in Strathroy-Caradoc and the need for future works to increase capacity and improve operations, it is recommended that this percentage (i.e., 90%) be lowered to initiate further studies at an earlier stage.

4.9.7.3 Sludge Handling

Given the size of the wastewater treatment plant with a current rated capacity of 825 m^3/d, consideration could be given to improving sludge handling at the facility with the installation of aerobic digesters along with sludge holding tanks(s). Aerobic digesters are typically installed in smaller plants and for treating secondary sludge from extended aeration treatment processes. The system can be operated at ambient temperatures in a conventional aerobic digestion process, provides a less complex process for stabilization of the sludge. The system can be operated in batch or continuous mode. In a continuous mode, a settling tank would be required after the aerobic digester to allow the separation of the clarified supernatant and digested sludge. The supernatant will be decanted and returned to the treatment process. Some advantages of this approach include lower capital costs, lower levels of organic in the supernatant that is returned to the plant, better ability handle any upset that may occur in the treatment process in terms of loading and pH, and simple operation and maintenance of the system. However, it must be noted that process will require aeration of the digester which can provide some additional operating costs for blowers and pumps.

4.10 Decommissioning of Sludge Lagoons

In preparing to decommission the sludge storage lagoons, the volume of sludge and the depth of sludge that has accumulated over time will need to be identified. Sampling and analysis of the sludge material is recommended to determine the quality of the sludge and the appropriate method for final disposal of the sludge material. Some options for decommissioning include:

Excavation/Dredging of the Lagoon – To facilitate the removal of the sludge from the bottom of the lagoon, the supernatant will have to be transferred to the biological treatment step in the treatment plant. Special care would have to be taken if mechanical equipment is used for the removal of the sludge is used as there is a high risk of damaging the membrane liner in the lagoon and has a high cost. Alternatively, prior to the transfer of the supernatant, floating dredge pumps can be used to remove the settled sludge on the bottom of the lagoon.

Use of Geotubes – Geotubes are porous tubular containers that are constructed of polyethylene and can be used for containment and dewatering. These tubes can achieve final solids concentrations of more than 30% TSS. In using the geotubes, the sludge is pumped from the lagoon to the tubes and once the solids have been dewatered, the geotubes can be used to contain the solids until the material can be land applied

Haulage to a Third Party – The sludge removed from the lagoon can be hauled to a third-party contractor for final treatment and processing.

5 Storm Water

5.1 Introduction

5.1.1 Objectives

WSP has examined the servicing capacity of the storm sewer systems of Strathroy and Mount Brydges as part of its Official Plan Review.

Growth forecasts indicate that the Municipality of Strathroy-Caradoc is expected to experience considerable growth in the coming years and multiple residential, commercial and industrial developments are anticipated throughout the two communities.

In order to facilitate the expected developments, WSP was retained to evaluate the capacity of the existing storm sewer network within the community of Strathroy and Mount Brydges.

The storm water section of this report presents a brief description of the storm sewer system, spreadsheet model setup, and the results of the capacity analysis. Full-size figures contained in this Report are also included in **Appendix A**.

5.1.2 Study Area

The Municipality of Strathroy-Caradoc encompasses roughly over 27,000 ha. and is located in the south-central portion of Middlesex County approximately 25 km west of the City of London in southwestern Ontario.

The Municipality of Strathroy-Caradoc is an urban-rural municipality, with two major settlement areas – Strathroy and Mount Brydges. Functioning as the administrative and business centre of the Municipality, Strathroy supports the largest resident population of the Middlesex County.

Figure 26 illustrates the municipal boundary along with current land uses and drainage features. As shown, majority of the lands are used for agricultural purpose.

5.1.2.1 Surface Drainage Watershed

In a watershed context, **Figure 27** illustrates the municipal boundary, two watershed basins to which the stormwater drainage from the Municipality is discharged, other natural drainage features and municipal drains.

East Sydenham River Drainage Basin (17,100 ha, 63.2%)

Approximately 17,100 ha or 63.2% of Municipality's lands drain to East Sydenham River to the north which is under the jurisdiction of the St. Clair Region Conservation Authority (SCRCA).

Strathroy, the largest settlement of the Municipality, and northwest portion of Mount Brydges is located in this drainage basin.

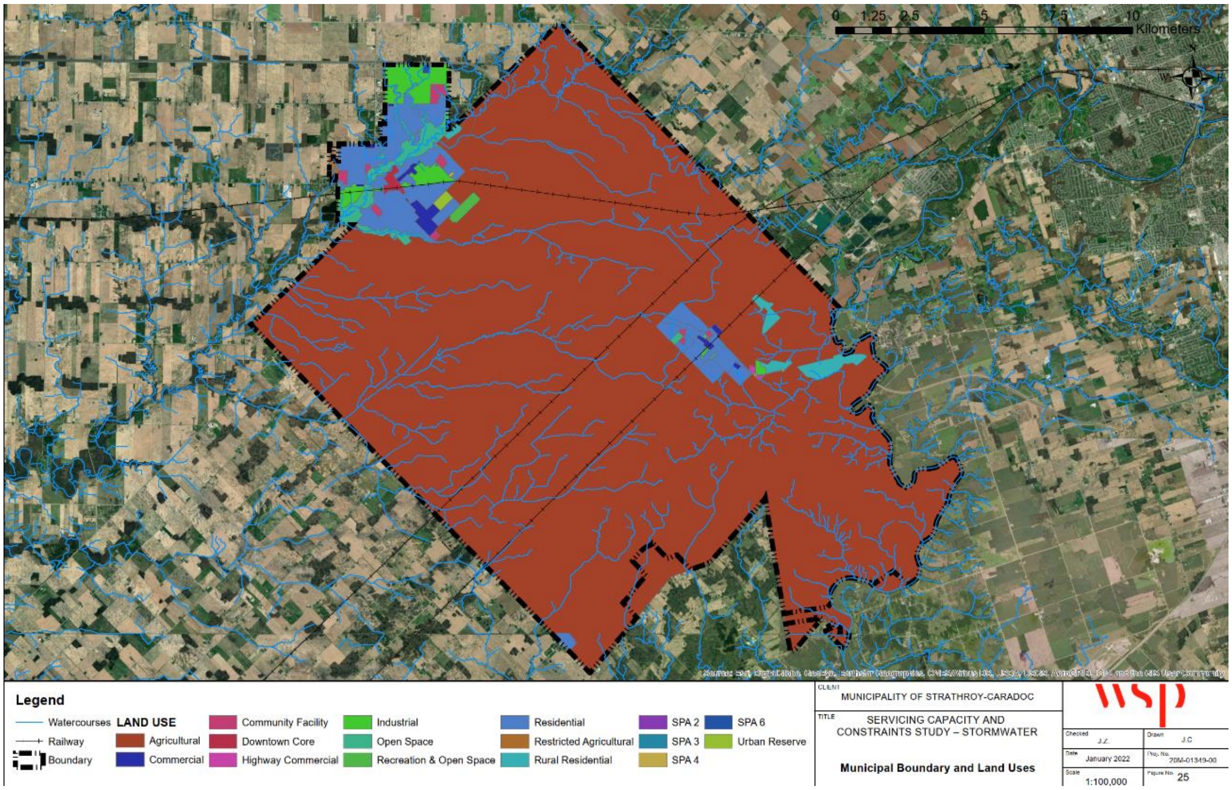

Figure 26: Municipal Boundary and Land Uses

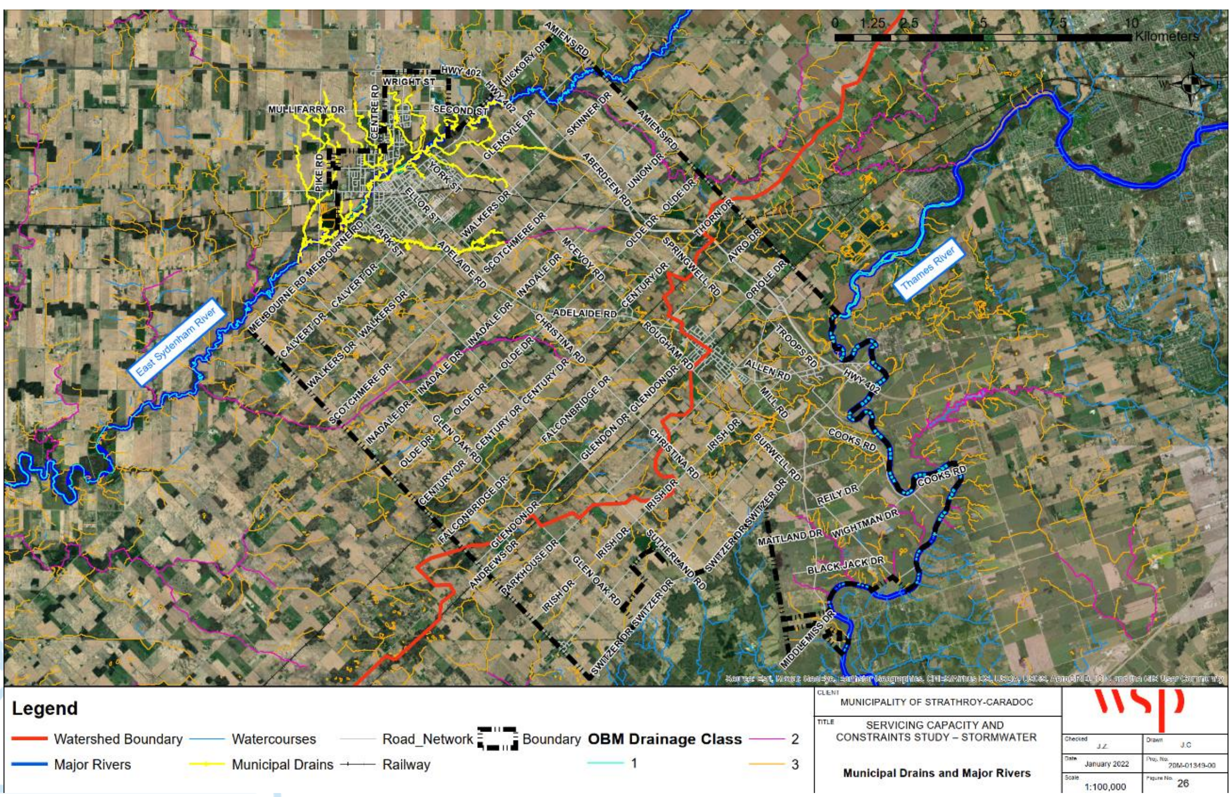

Figure 27: Municipal Drains and Major Rivers

Thames River Drainage Basin (9,970 ha, 36.8%)

Approximately 9,970 ha or 36.8% of the Municipality's lands drain to Thames River to the south, which is under the jurisdiction of the Lower Thames Valley Conservation Authority (LTVCA).

5.1.2.2 Minor System - Storm Sewers

The Municipality of Strathroy-Caradoc adopted a dual storm drainage system in its urbanized areas of Strathroy and Mount Brydges. As per the Servicing Standards of the Municipality (2021), the storm sewers are to be designed to convey the 5-year minor storm event to a sufficient outlet or a stormwater management facility for quantity control.

Figure 28 presents an overview of the existing storm sewer networks in both Strathroy and Mount Brydges, shown in yellow lines.

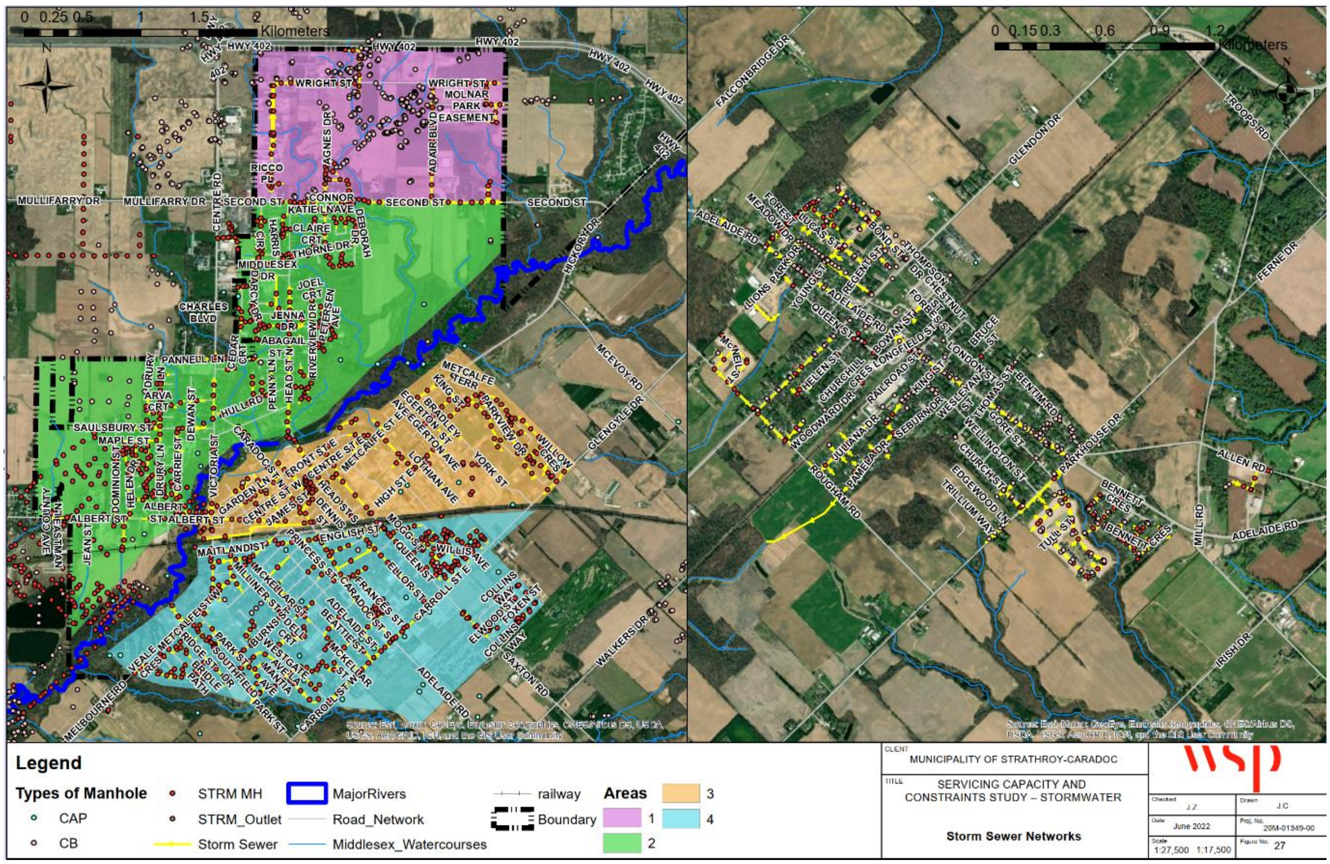

Figure 28: Overview of the Existing Storm Sewer Networks

5.1.2.3 Major System – Overland Flow Route

As per the Servicing Standards of the Municipality, overland flow routes should be designed to convey major storm flows in excess the minor system up to the 100-year and Regional storm. The overland flow route shall be either the roadway right-of-way (ROW) or by other lands such as flow easements under the control of the Municipality.

5.1.2.4 Municipal Drains

As a rural municipality, municipal drains play significant roles in the storm drainage system. Municipal drains consist of roadside ditches and natural watercourses, and covey the stormwater runoff from the Strathroy and Mount Brydges, and rural area to major watercourses being the East Sydenham River to the north and Thames River to the south. Further, the Municipality is establishing some municipal drains in urban areas to manage flows which may be converted to storm sewers at a later date.

5.1.2.5 Stormwater Management Facilities

In Ontario, prior to the 1980's, stormwater runoff from urban development areas was typically discharged from storm sewers or municipal drains directly to the receiving stream or river. Much of the existing urban development within the Municipality took place prior to the implementation of modern stormwater management. As such, there are large areas within the Municipality where uncontrolled and untreated stormwater runoff is discharged directly to the receiving water bodies such as East Sydenham River and Thames River.

Table 37 summarizes the location and characteristics of the existing stormwater management facilities within the Municipality of Strathroy-Caradoc. This summary does not include unassumed stormwater management facilities.

Table 37: Existing SWM Facilities within the Municipality of Strathroy-Caradoc

SWM FACILITY NO.	LOCATION	WATERSHED	TYPE OF FACILITY	TYPE OF CONTROLS	DRAINAGE AREA (HA)	IN-SERVICE DATE
E1	Pinetree Ln., Strathroy	East Sydenham	Wet Pond	N/A	N/A	1993
E2	Parkview Dr., Strathroy	East Sydenham	Wet Pond	N/A	N/A	1994
E3	Parkview Dr., Strathroy	East Sydenham	Wet Pond	N/A	N/A	1994
E4	Second Str. & Adair Blvd. Strathroy	East Sydenham	Wet Pond	N/A	N/A	2001
E5	Head St. N. & Thorne Dr. Strathroy	East Sydenham	Wet Pond	N/A	N/A	2004
E6	Off of Wright St., Strathroy	East Sydenham	Wet Pond	N/A	N/A	2010
E7	Bennett Cres., Mt. Brydges	Thames River	Wet Pond	N/A	N/A	2013
E8	North and South of Pond-view Ln., Mt. Brydges	East Sydenham	Wetland	Quality, Erosion, and Quantity	27.0	2013

5.2 Storm Sewer Capacity Analysis

The storm sewer capacity analysis was carried out using a spreadsheet model developed for this study. The spreadsheet model was used to evaluate the capacity of the minor storm system as per the Municipality's current Servicing Standards (2021). The capacity analysis consists of a hydrologic analysis of the upstream contributing drainage area and hydraulic analysis of the storm sewer systems.

The overland flow routes and open portions of municipal drains were not included in the analysis.

5.2.1 Spreadsheet Model Setup

The GIS database of the Strathroy and Mount Brydges storm sewer networks, provided by the Municipality of Strathroy-Caradoc, were used for the hydrology and hydraulic analysis of the storm sewer system. The storm sewer network data include storm pipes, maintenance holes (MHs), catch-basin and maintenance holes (CBMHs), catch-basins (CBs), double catch-basins (DCBs), stormwater management (SWM) ponds, storm headwalls / outlets.

All the storm sewers servicing the communities of Strathroy and Mount Brydges were reviewed. Refer to **Figure 28** for the storm sewer networks for Strathroy and Mount Brydges.

5.2.1.1 Strathroy

For the study purpose, the Strathroy is divided into four sub-areas as shown on **Figure 28**.

Area #1 includes the Molnar Industrial Park located north of Second Street and south of Highway 402. Stormwater runoff from Area #1 is either conveyed by the storm sewers or directly discharge into various tributaries running southernly, and ultimately drains to the East Sydenham River. The storm sewers on Wright Street and Second Street are generally installed during 2000 to 2010.

Area #2 is a predominantly residential area located south of Second Street and north of the East Sydenham River. Stormwater runoff from this area is conveyed by the storm sewers to various tributaries running southernly, or directly discharged into the East Sydenham River. The storm sewers in this area are generally installed since 1990s with portion installed as early as the 1970s.

Area #3 is located south of the East Sydenham River and north of the Canadian National Railway (CNR). This area consists of Downtown Strathroy, industrial/commercial areas, and residential areas. Stormwater runoff from this area is generally conveyed by the storm sewers northernly to East Sydenham River. The storm sewers in this area are installed prior to 1990s and as early as in 1950s.

Area #4 is located south of the CNR. This area represents the older residential areas with newer developments east of Queen Street and south of Carroll St. Stormwater runoff from this area generally drains westerly to Humphrey Drain and then to East Sydenham River. Storm sewers in this area are installed as early as in 1950s.

5.2.1.2 Mount Brydges

Mount Brydges is a predominantly residential community and is divided into two sub-areas. The first area, north of the CNR, generally flows westerly and ultimately discharges into the East

Sydenham River. The area south of the CNR drains through various municipal drains and ultimately discharges into the Thames River. The Lipsit Drain is the municipal drain in Mount Brydges with the largest drainage area. The storm sewers on Adelaide Road were installed as early as in 1978.

5.2.2 Sub-catchment Delineation

To calculate the design peak flow rates (5-year) for the storm sewer systems, sub-catchments were delineated for the entire study area. The catchment delineation was carried out based on the storm sewer network or location of CBs and CBMHs, parcel layer, topographic information or contours, and engineering judgement. Refer to **Appendix C** for storm drainage plans.

5.2.3 Hydrologic Analysis

The Rational Formula is used to determine the quantity of stormwater runoff.

$$Q = 0.002778CIA$$

Where, Q is the design peak flow rates (m^3/s)
C is the runoff coefficient
I is the average rainfall intensity (mm/hour)
A is the drainage area tributary to the point under design (hectares).

Table 38 presents the typical runoff coefficients to be used based on different land use categories. A lumped runoff coefficient is calculated for the catchment where mixed land uses are found.

Table 38: Runoff Coefficients

LAND USE	RUNOFF COEFFICIENT
High Density Residential - Towns	0.75
Low Density Residential - Single	0.45
Commercial / Industrial	0.85
School	0.75
Roadway	0.60
Parks / Farmlands / Open Space	0.25

The initial time of concentration (Tc) for all types of development is ten (10) minutes. In case that detailed storm sewer information for upstream development area is not available, the Tc value at the study point is estimated as the initial time plus the travel time for estimated length of storm sewers. The Tc value for a large rural area is estimated using airport method, and then adjusted to produce reasonable design peak flow rates.

$$T_C = \frac{3.26(1.1 - C)L^{0.50}}{S_w^{\ 0.33}}$$

Where, Tc is the time of concentration (minutes)
C is the runoff coefficient
L is catchment length (m)
S_W is catchment slope (%)

As per the Servicing Standards of the Municipality (2021), storm sewer design is to be based on the Rainfall-Intensity-Duration curve on Drawing No. SCSD-14. Therefore, the capacity analysis of the existing storm sewer system shall be undertaken for the 5-year design storm.

The constant parameters (A, B, and C) for the 5-year storm events are presented in **Table 39**.

Table 39: Rainfall Parameters

RETURN PERIODS(YEARS)	A	B	C
5	1137.257	7.184	0.830

The SWM wet pond / wetlands for the Woodward Subdivision in Mount Brydges discharges into existing Lipsit Drain. It is assumed that, during the 5-year design storm, the first segment of the outlet sewer is 50% full. This assumption might not be valid but only has impact to the results for the 110 m storm sewer, where it is joined with another leg of storm sewer which contributes much larger flows.

Note that on-site quantity control such as rooftop controls, etc., for the existing industrial / commercial developments are not included in the analysis.

5.2.4 Hydraulic Analysis

Sewers are to be considered as open channels in the selection of hydraulic formulae for design purposes and the Manning's Formula shall be used to design/analyze the gravity storm sewers.

The Manning Equation is expressed as:

$$Q = \frac{1}{n} A R^{0.67} S^{0.5}$$

Where, Q is the design flow rates (m3/s)
R is the hydraulic radius (m)
S is the slope of conduit (m/m)
n is the roughness coefficient (0.013 for all pipes)

Note that there are missing data for the pipe slope in the GIS database. In this case, the pipe slope is either determined by referring the available municipal drain records (plan and/or profiles) or referring the slope of adjacent storm sewers. This might result in inaccurate result for the local storm sewer but has minimum impact to downstream sewers.

5.3 Results and Conclusion

Appendix B presents the storm sewer design sheets for all analyzed storm segments. The values in the last column "Capacity Check" are ratios of the 5-year design peak flow rate to the full capacity of the storm sewer. The values less than 100% are shown in green, the values greater than 100% but less than 200% are shown in yellow, and the values greater than 200% are shown in red.

The results of this analysis are summarized below and have informed the recommendations presented subsequently in this Report.

5.3.1 Strathroy

5.3.1.1 Strathroy – Area #1

Area #1 covers the Molnar Industrial Park located north of Second Street and south of Highway 402. The storm sewers on Wright Street and Second Street have adequate capacity to convey the 5-year design peak flows from the right-of-way (ROW) and un-developed parcels contributing flows under existing conditions.

The storm sewers do not have capacity to convey uncontrolled flows from vacant development blocks.

5.3.1.2 Strathroy – Area #2

Area #2 is residential area located south of Second Street and north of East Sydenham River. The storm sewers at the following locations do not have adequate capacity to the convey the 5-year peak flows:

- Storm sewers on Head Street North, near the outlet, from MH756 to Outlet to East Sydenham River.
- Storm sewers on Darcy Drive, between Acton Street and Abagail Street, from MH23 to MH10.
- Storm sewers on Hull Road.
- Storm sewers on Drury Lane, between Parcreek Place and Saulsbury Street, from MH565 to MH553.
- Storm sewers on Brennan Drive.
- Storm sewers on Albert Street, between Helen Driver and Carrie Street, from MH771 to MH69.
- Storm sewers on Carrie Street / Easement, between Peach Avenue and Albert Street, from MH73 to MH70, from MH69 to MH70, and from MH70 to Outlet to East Sydenham River.
- Storms sewers on Helen Drive.
- Dominion Street, 460 m long, 5.0 ha area without storm sewers.

The above noted segments of storm sewer were generally installed in 1970s and do not have capacity to convey the uncontrolled flows from developments since then.

5.3.1.3 Strathroy – Area #3

Area #3 is located south of East Sydenham River and north of the Canadian National Railway (CNR). This area consists of Downtown Strathroy, industrial/commercial areas, and residential areas. Other than the Parkview Subdivision, the majority of the storm sewer systems in the downtown and industrial areas were installed prior to 1990's and do not have adequate capacity to convey the 5-year design flows from its contributing areas. Refer to **Appendix B** for more details.

5.3.1.4 Strathroy – Area #4

Area #4 is located south of CNR. This area represents the old residential areas with new developments east of Queen Street and south of Carroll St. The storm sewers in this area are installed as early as in 1950s.

The storm sewers at the following locations do not have adequate capacity to the convey the 5-year peak flows:

- Storm sewers all along Queen Street (from MH210 to MH200 and from MH200 to MH 216), English Street (from MH219 to MH224), Maitland Terrace (from MH224 to MH419), Adelaide Street (from MH419 to MH420), Metcalfe Street West (from MH420 to MH422), and Duke Street (from MH422 to Outlet to East Sydenham River).
- Storm sewers on Carroll Street East and Carroll Street West, between Ellor Street and Pearson Avenue, from MH231 to Outlet to Humphrey Drain.
- Storm sewers on Caradoc Street, between Ontario Street and English Street, from MH734 to MH224.
- Storm sewers on Ridge Street, between Bridle Path and Metcalfe Street West, from MH147 to MH114.
- Storm sewers on Oak Avenue, between Heritage Court and Metcalfe Street West, from MH521 to MH485 and from MH491 to MH485.
- Storm sewers on McKellar Street, between St. Vincent De Paul Elementary School and Carroll Street West, from MH698 to MH697.

There are also large undeveloped areas without storm sewer connections. Stormwater runoff from these areas drains overland until it reaches the first catch-basin. The downstream sewers do not have adequate capacity to capture and convey these flows.

5.3.2 Mount Brydges

The underground storm sewers, or closed portion of municipal drains, were installed dating back to the 1970s in Mount Brydges. These municipal drains generally do not have adequate capacity to convey the uncontrolled flows from large developments since then.

The storm sewers at the following locations do not have adequate capacity to the convey the 5-year peak flows:

- Storm sewers on Adelaide Road (Lipsit Drain, Branch A), between Woods Edge Road and Yonge Street, from MH17 to MH62 and then Outlet to Lipsit Drain south of Queen Street.
- Storm sewers on Emerson Street (Lipsit Drain), between Glendon Drive and Yonge Street, from MH65 to MH39.

- Storm sewers on Yonge Street (Lipsit Drain), between Emerson Street and Adelaide Road, from MH39 to MH63.
- Storm sewers on Adelaide Road (Gillam Drain), between Thomas Street and Parkhouse Drive, from MH81 to CBMH41 and then Outlet to Lipsit Drain south of Queen Street.
- Other municipal drains, such as Helen Street Drain, Applewood Acres Drain – Crow Road Branch, Pamela Drive Drain, do not have adequate capacity to convey the 5-year design flows form contributing areas.

5.3.3 Conclusions and Recommendations

The results of the above capacity analysis indicate that the storm sewer systems of the Strathroy and Mount Brydges generally do not have adequate capacity to convey the 5-year design flows from the contributing areas. This is resulted from lower level of servicing standard at the time of installation and/or lack of stormwater quantity controls for the developments since then.

Urban flooding may occur when the capacities of both the minor and major system are exceeded during a storm event. It is recommended that a Master Drainage Plan (MDP) including comprehensive hydrologic and hydraulic analysis of the storm drainage system (both minor and major) be undertaken to:

1. Identify the location and frequency of the urban flooding.
2. Inform the required upgrades on both minor and major systems to mitigate the urban flooding issue.
3. An implementation plan for the existing infrastructure upgrades shall be developed as part of MDP.

For future residential, commercial and industrial developments in both Strathroy and Mount Brydges, it is recommended that more stringent stormwater management policies be implemented for the future developments to minimize the impacts on downstream flooding and overflows of sewer systems. The following guidelines should be adopted in the Municipality's Servicing Standards.

1. For new development sites with runoff directly discharging into the East Sydenham River, Thames River, and their tributaries, the required level of peak flow control shall be determined through evaluation of the downstream impacts to the satisfactory to the Conservation Authorities.

   These types of developments include but not limited to,
   - Industrial development sites within Strathroy Area #1 (Molnar Industrial Park);
   - North Meadows in Strathroy Area #2;
   - Community development southeast of Carroll Street West and southwest of Adelaide Road in Strathroy Area #4;
   - Community developments at Falconbridge East and Falconbridge West in Mount Brydges.
2. For new development sites with runoff discharging into existing storm sewer systems, the required level of peak flow control shall be determined site by site in considering the capacity of the receiving storm sewers. If no positive overland route exists, runoff from all

storm events should be controlled to the allowable release rate to the storm sewer system as per current Servicing Standards (2021).

These types of developments include the residential and commercial development south of Carroll Street West and Carroll Street East.

3. Quantity control levels for the intensification/infill site should be determined site-by-site and overland control may be required for developments, of which either the receiving sewer has limited capacity and/or no positive overland routes exists.

Section 10.4.6 of the Municipality's Servicing Standards includes brief descriptions and guidelines regarding Low Impact Development (LID) measures. It is recommended that this section be updated to require the implementation of LID measures for all new development and re-development sites. A list of long list and short list of LID measures endorsed by the Municipality should be included to provide direction to applicants.

Further, the Municipality's Servicing Standards should address the effect of climate change on design storm events by adopting Intensity-Duration-Frequency (IDF) curves – a graphical tool to describe the likelihood of a range of extreme rainfall events, which are adapted to account for an increase in rainfall intensity and volume.