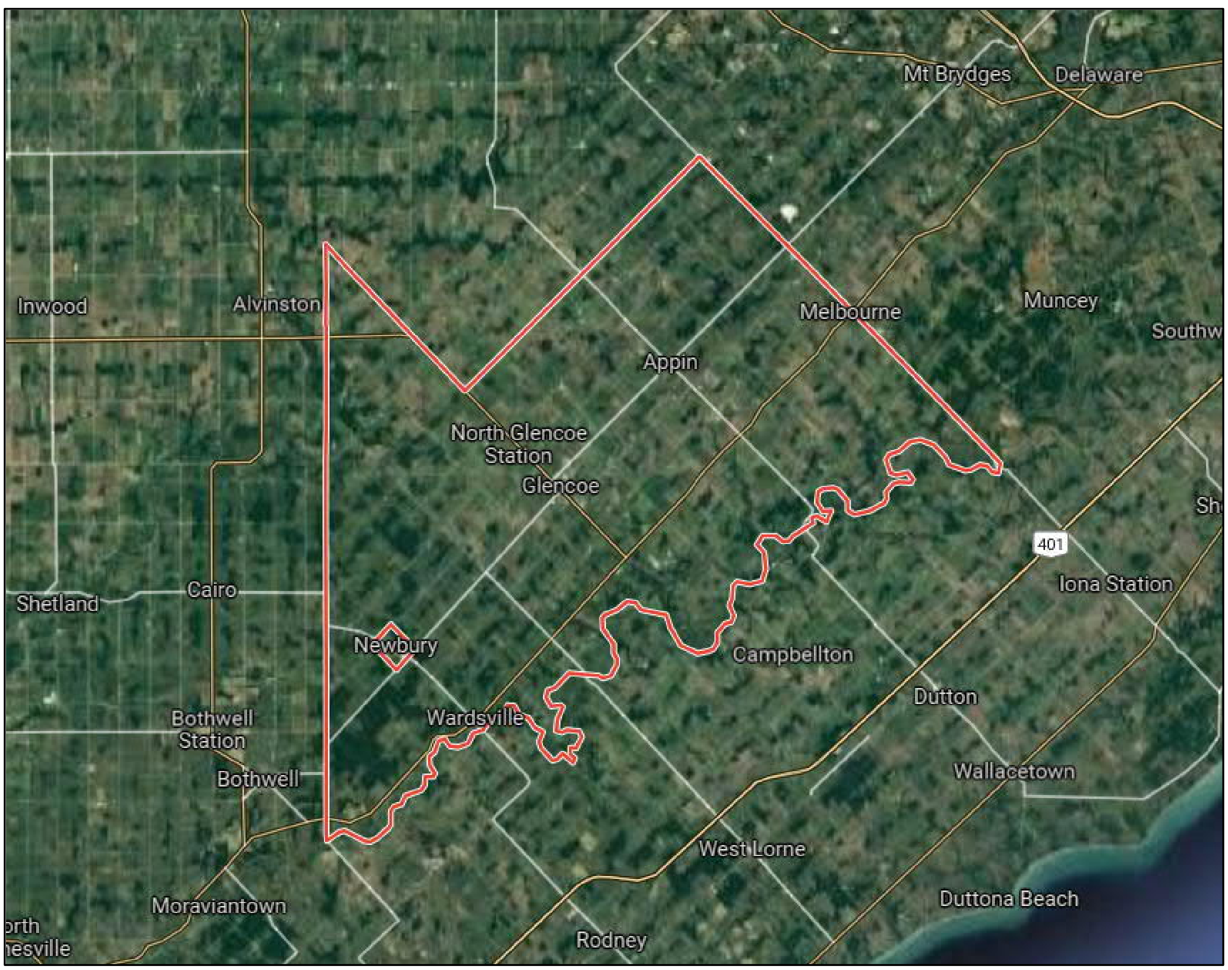
THE MUNICIPALITY OF SOUTHWEST MIDDLESEX

REPORT NUMBER: 211-06377-00

SOUTHWEST MIDDLESEX MASTER SERVICING PLAN

DRAFT WATER & WASTEWATER MODELING REPORT

JUNE 30, 2022



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DRAFT WATER & WASTEWATER MODELING REPORT

MUNICIPALITY OF SOUTHWEST MIDDLESEX

FINAL REPORT
CONFIDENTIAL

PROJECT NO.: 211-06377-00
DATE: JUNE 30, 2022

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June 30, 2022

WSP Ref: 211-06377-00

Subject: Southwest Middlesex Master Servicing Plan, Southwest Middlesex, Ontario – Draft Water & Wastewater Model Report

Dear Client:

WSP Canada Inc. (WSP) is pleased to provide this Master Servicing Report that contains a summary of the model simulations/results completed and the associated hydraulic results for the water and wastewater systems that service the Municipality of Southwest Middlesex (SWM), Ontario. Also included in this is a discussion of water quality considerations, while appended to this report are a Wastewater Treatment Plant capacity assessment and a Planning Memo to support the study.

In this analysis, WSP built water (WaterGEMs) and wastewater (SewerGEMS) hydraulic models of the Municipality's system and simulated the existing and future expected hydraulic conditions. The existing (2021) models are based on known system information and demands (based on existing populations and SCADA data), while the future (2041) model was updated to reflect the existing and projected demands/loads with suggested system growth to accommodate the new development.

As part of this, the 2021 and 2041 planning horizons were considered, and results were studied to identify servicing constraints in the system. As a result, WSP suggested infrastructure and operational recommendations to be considered that can help overcome capacity limitations in the existing systems and to accommodate future growth in the Municipality.

The results and recommendations are based on the Municipality of Southwest Middlesex design guidelines and recommended criteria by WSP. These conform to the requirements set forth by the Ministry of Environment Conservation and Parks (MECP).

Should you wish to discuss any aspect of this report, please do not hesitate to contact me.

Yours sincerely,

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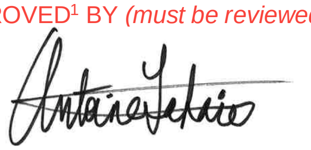

2022/06/30

Antoine Lahaie, B.Eng, P.Eng, P.M.P

2022/06/30

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EXECUTIVE SUMMARY

WSP is providing civil engineering consultation relating to Water and Wastewater Servicing for the Municipality of Southwest Middlesex, Ontario. The objective of this study is to review the existing and future water and wastewater servicing capacity, to identify potential servicing constraints for each system, and provide high-level development servicing plans and recommendations for infrastructure within the study area. To support this study, WSP completed a review of the Municipalities planning projections and plan and completed a study of water quality consideration and wastewater treatment capacity.

This report summarizes the model build process and contains the results of WSP's Water & Wastewater Servicing analysis in WaterGEMs and SewerGEMs. In this analysis, WSP built the hydraulic models to reflect the expected system operation for the Municipality and loaded the calculated water demands and sewer loads. The models were calibrated based on the available SCADA data, hydrant flow tests, and Wastewater Treatment Plant Annual Reports. WSP also provided recommendations, including upgrades to the existing infrastructures and construction of new infrastructures within the Municipality. Costs and timelines for the recommended projects were also provided based on the level of urgency.

WATER

The watermain analysis was completed using the WaterGEMs model built by WSP, considering the existing (2021) and future (2041) planning horizon. As part of this, WSP simulated and evaluated pressure and available fire flow in the study area under Average Day Demand (ADD), Maximum Day Demand (MDD), and Peak Hour Demand (PHD).

Based on the simulation, WSP concluded that most of the area within the Municipality is expected to have service pressure above the minimum pressure requirement of 275 kPa, with the exception of a few junctions on Melbourne Rd. and Parkhouse Dr. closed to the Village of Melbourne were simulated with pressure below 275kPa under PHD condition, which was majorly caused by the drop of elevation. The simulated fire flows in the model were consistent with the hydrant classification maps provided by the Municipality and were generally above 1000 gpm (63 L/s); however, the Village of Appin was a dead-end neighbourhood and was simulated with fire flow below 500 gpm (32 L/s) on all hydrants due to pressure limitation. Also, the existing watermains within the Municipality can operate with headlosses below 2km/m, except for the existing 250mm watermain along Victoria St. and the 150mm watermain on Parkhouse Dr. and Deane St. was simulated with headlosses over 2km/m.

Based on the hydraulic performance of the water distribution system simulated in WaterGEMs, WSP provided the following recommendations:

1. Installing an in-line Booster Pump Station close to the intersection of Parkhouse Dr. and Thames Rd. to increase system pressure and fire flow availability within the Village of Appin and Melbourne.
2. Adding a second connection to the Village of Glencoe along Main St. between Industrial Rd. and Parkhouse Dr., parallel to the existing 300mm major supply main east of Main St. This will provide two supply mains into Glencoe rather than one and reduce headloss along the existing 250mm watermain on Victoria St.

Water Quality

The Municipality presented two water quality challenges that it faced with regards to the low free chlorine residual at the outlet of the Melbourne standpipe in the summer months and the soluble manganese present in the distribution system. In order to investigate these issues, WSP performed an analysis of the available water quality data from 2018 to 2021.

For the Melbourne standpipe, the data confirmed that the onset of warmer summer temperatures results in outlet free chlorine residuals decreasing from the target 1.0 mg/L to a range of 0.5-0.8 mg/L for most of the summer period until late September. WSP recommended two improvements:

- The chlorine dosing point needs to be relocated from the inlet of the standpipe to its outlet, which WSP was informed had already been done during the summer of 2021. Therefore, the Municipality needs to closely monitor free chlorine residuals for at least one year until the end of September 2022 to confirm the successful implementation of this system change.
- The installation of an active mixer in the standpipe to mitigate the stored water thermal stratification in the summer months and the resulting free chlorine residuals losses through off-gassing.

For the presence of soluble manganese in the distribution system, the data confirmed that detection of soluble manganese at the outlet of the water treatment plant is tied to severe weather events. While it appears that the existing water treatment plant process is able to adequately remove manganese during heavy rain events to below the Aesthetic Objective (AO) of 0.02 mg/L, it is not able to do so when heavy rains result in major floods. Indeed, as a result of major floods in September 2021, dissolved manganese in concentrations above the Maximum Allowable Concentration (MAC) of 0.1 mg/L was able to pass through the existing water treatment plant process and enter the distribution system. WSP recommended that a feasibility study be performed to assess which dissolved manganese removal technology would be most appropriate for this application and for implementation into the existing water treatment plant.

WASTEWATER

WSP analyzed the wastewater network using the SewerGEMs model built by WSP in the context of this servicing study to assess the hydraulic performance on the wastewater network considering the existing (2021) and future (2041) planning horizon. As part of this analysis, WSP built the SewerGEMs model for the Village of Wardsville and Village of Glencoe, and it was calibrated based on the recorded raw flow from the WWTP Annual Report. WSP simulated the Dry Weather Flow (DWF), Wet Weather Flow (WWF), and Wet Weather Peak Flow (WWF_Peak) conditions to assess the conveyance capacity of the sewage system in the study area.

Based on the simulations, the existing sewers within the Village of Glencoe were expected to have enough capacity to convey wastewater loads with no surcharge to ground under both planning horizons, and the q/Q ratio can be maintained below 80%; however, the existing Victoria Sanitary Pump Station (SPS) cannot accommodate the required flow under WWF_Peak condition.

In the Village of Wardsville, the existing Small-Bore Sewer (SBS) System was simulated to have sufficient capacity to convey sewage to the Main Sanitary Pump Station without surcharge to ground under DWF for both planning horizons; however, under WWF_Peak condition, a few sewers along Run 'D' surcharged with q/Q ratio greater than 100%. It should be noted that the Wardsville SBS system was simplified ad set up as a steady state model which does not consider the retention time in each individual septic tank before it discharges to the collection system.

Based on the hydraulic performance of the wastewater distribution system simulated in SewerGEMs, WSP provided the following recommendations:

1. Adding a twinned forcemain in addition to the existing 200mm forcemain connecting the Victoria SPS to the Glencoe WWTP to accommodate more outflow from the station.
2. Upsizing the existing wet well to accommodate future flow and reduce pump cycle.

WASTEWATER CAPACITY REVIEW

WSP conducted a capacity review of the treatment process at the Glencoe WWTP and Wardsville WWTP and concluded that:

- Current average flow to the Glencoe WWTP for the last three years was 672 m^3/d. This is approximately 39% of the current rated capacity (1,723 m^3/d) of the plant. The capacity review of the treatment process indicated that all the process units have sufficient capacity for a projected future average flow of 1,113 m^3/d.
- Current average flow to the Wardsville WWTP for the last three years was 98 m^3/d. This is approximately 33% of the current rated capacity of the plant. The capacity review has shown that all the process components have sufficient capacity to meet the projected future average flow of 239 m^3/d.

According to the feedback received from the OCWA, the average alum dosing for each plant was 100 mg/L which is lower than the MOE Design Guidelines recommended values (110 mg/L to 225 mg/L). The theoretical dosing rates were estimated to be 176.6 mg/L for Glencoe WWTP and 265 mg/L for Wardsville WWTP. These values are higher than the historical dosing rate. Review of the Annual Reports for both plants also showed that there were some exceedances of the effluent TP objective which required have required adjustments to the alum dosing at both plants. Based on the theoretical calculations, it is recommended that consideration should be given to increasing the alum dosing at each plant. Bench testing of the coagulant and polymer from a variety of chemical suppliers should be undertaken to ensure the chemicals that provide optimum performance is selected.

1 INTRODUCTION

WSP Canada Inc. (WSP) has been retained by the Municipality of Southwest Middlesex (SWM) to assemble Water and Wastewater (W&WW) hydraulic models and to examine the W&WW servicing capacity of the existing and future growth planning horizons. The objective of this was to identify the potential servicing constraints for each system, and to provide engineering consultation for infrastructure upgrades to accommodate future growth in the Municipality. The primary Study Area encompasses approximately 42,788 hectares of agricultural, rural, and heritage lands with a population of 5,723 based on 2016 Canada Census, and it is located in the southwest corner of Middlesex County. Figure 1 illustrates the site location for the SWM study area.

This report summarizes the findings of the W&WW modeling work and system recommendations. To achieve this scope, WSP built separate hydraulic models of the W&WW systems for the Municipality in WaterGEMS and SewerGEMs respectively. These models were based on the GIS data and as-built drawings provided by the Client. In addition, WSP completed a review of water quality consideration for the distribution system and provides recommendation to maintain and improve water quality as appropriate. For the wastewater treatment, WSP completed a review of the treatment train at the Glencoe treatment plant quantifying the current treatment capacity and recommending solutions to expand the system to support the Municipalities forecasted growth.

In this report, WSP provides an overview of the model build process, the hydraulic analysis for both existing and future condition and provides high-level development servicing plans and recommendations including upgrades to existing infrastructure and suggested infrastructure projects within the study area to meet the existing and growing demands in the systems.

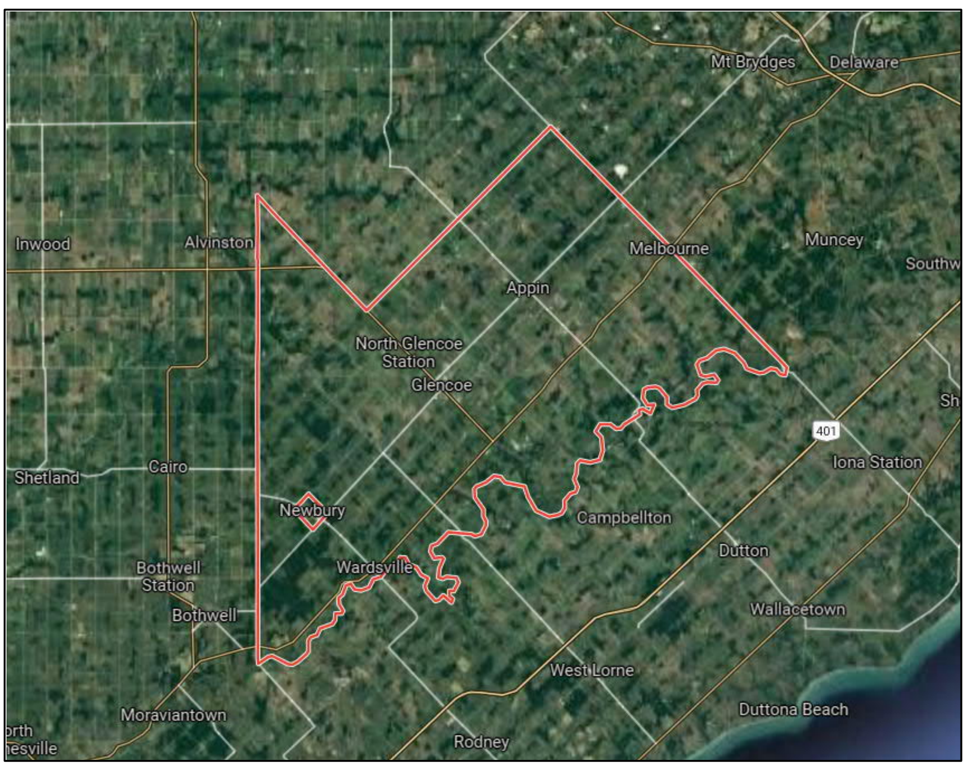


Figure 1: Location of the Municipality of Southwest Middlesex – Study Area

2 WATER NETWORK

The Municipality of Southwest Middlesex's (SWM) Water network receives its supply from the Tri-County Drinking Water System and services the communities of Wardsville, Glencoe, Appin, and Melbourne through the Southwest Middlesex (SWM) Reservoir and Booster Pumps Station. Figure 2 shows an overview of the water model layout developed by WSP. The core infrastructures within the system includes: SWM Booster Pump Station and Reservoir, Glencoe Water Tower, and Melbourne Standpipe. Detail maps to show the system layout for each community were also provided in Appendix A.

In addition, the SWM water system also connects and supplies water to the Newbury water distribution network and the Bothwell Distribution system, "place-holder" demands for these two systems were included in the context of this analysis. In this study, WSP built a hydraulic model of the network in Bentley's WaterGEMs based on GIS information, operation/performance reports and as-built or tender drawings of recently constructed infrastructure like the 200mm watermain along Main Street, in Glencoe, that is based on tender drawings.

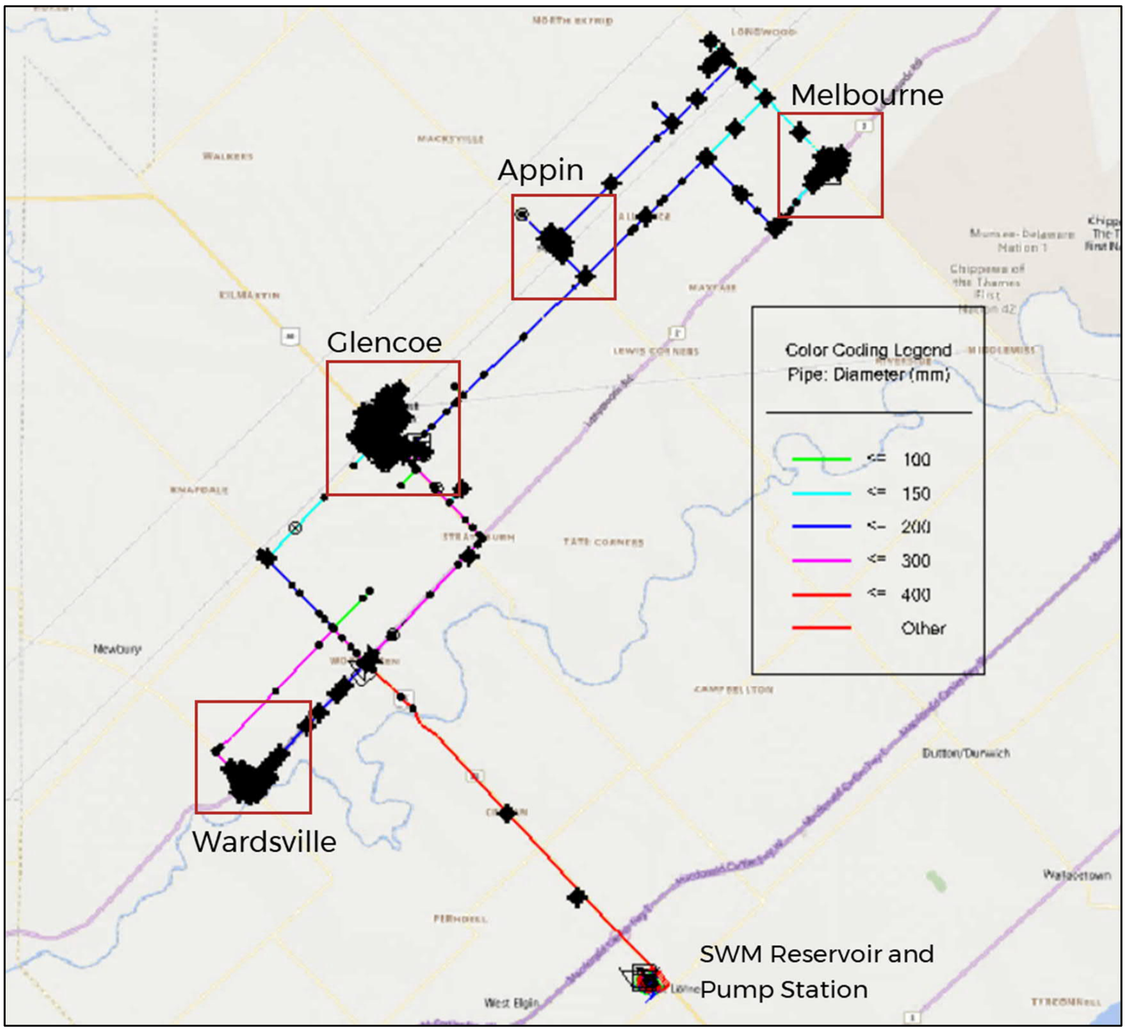


Figure 2: Southwest Middlesex Water System Overview

2.1 DOMESTIC DEMAND

Water demands are a key input to hydraulic models. Once the GIS layers were assembled in the water model space and connectivity was achieved between relevant infrastructure, WSP estimated the required water consumption for the “existing” scenario and loaded the hydraulic model with domestic demands that reflect the current population estimates.

Since no water meter data or user consumption reports were available for individual user within the Municipality for this exercise, demands for the study area that included Wardsville, Glencoe, Appin, and Melbourne were estimated by approximately counting the number of homes in each community using Google Earth. Neighborhoods were divided into multiple groups/parcels with approximately the same surface area and demands were calculated by first estimating the population in each of these groups/parcels and then calculating the estimated water consumption based on consumption rates and peaking factors. Figure 3 shows the map layout for the Village of Glencoe as an example with divided polygons, while Table 1 lists the factors used to estimate population and calculate demands. Peaking factors and water consumption rates were obtained from the Southwest Middlesex Municipal Design and Construction Standards (2021) or based on common/best practise values consistent with many Cities/Towns/Municipalities in Southern Ontario when not available through the Design and Construction Standards.

The exercise of calculating the approximate current population and resulting water demands was followed by completing the same tasks and updating the model with demands reflecting the approved and planned growth/development areas that would make up the “future condition” or “full buildout” planning horizon.

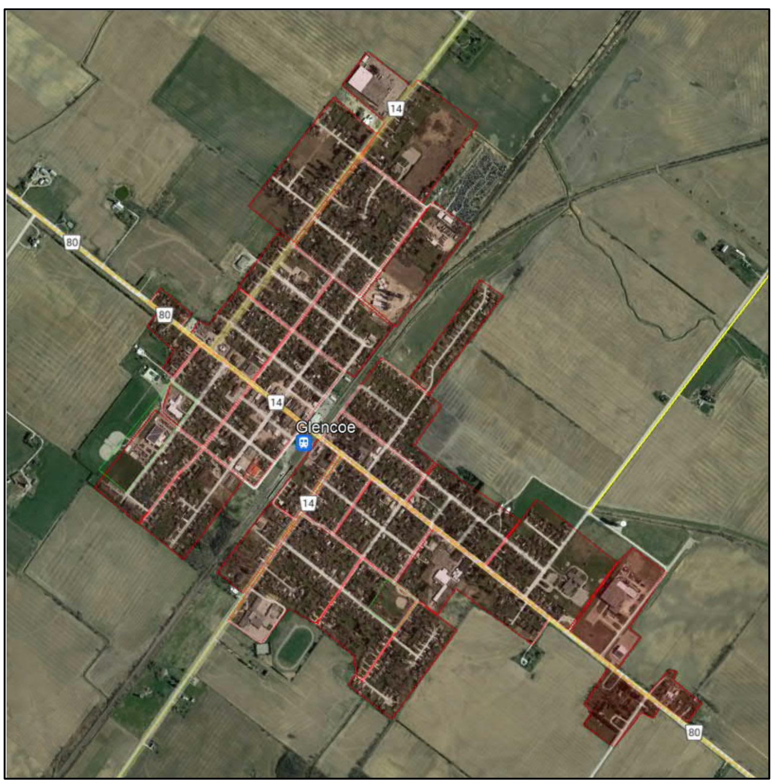


Figure 3: Southwest Middlesex Glencoe with Divided Population Polygons

Table 1 - Demand Factors and Inputs

DEMAND FACTORS AND INPUTS	VALUE
Single Family*	3.39 ppu
Townhouses*	2.45 ppu
Apartments*	1.76 ppu
Commercial	80 persons/ha
Industrial/Institutional	115 persons/ha
Average Day Demand	350 L/cap/day
Maximum Day Peaking Factor	3.0
Peak Hour Factor	4.1

* Note: The values shown in this table are consistent with the Southwest Middlesex Municipal Design and Construction Standards (2021) or taken as commonly accepted values from other Southern Ontario Cities & Municipalities if not available in the Design & Construction Guidelines.

The calculated demands for the entire study area were determined to be roughly 20.6 L/s under Average Day Demand (ADD), and the demand for various parcels were distributed and assigned to the closest node in the water model. The water demands for the SWM were then verified by comparing the calculated results to the water consumption data (at storages/supply) from 2018 to 2020 provided by the Municipality. This water consumption information, which is the total outflow from key storages and supply points in the system, also allowed WSP to add a "place holder" demand for the communities of Bothwell and Newbury along the flow path to these areas. Table 2 summarizes the yearly water consumption for the SWM, Newbury, and Bothwell from the meter data between 2018 and 2020 as provided by the Client.

Table 2 - Water Meter Consumption Data from 2018 to 2020

	SWM		NEWBURY		BOTHWELL	
	m^3/yr	L/s	m^3/yr	L/s	m^3/yr	L/s
2018	282577	8.96	40852	1.30	90849	2.88
2019	284043	9.01	47136	1.50	79653	2.53
2020	294916	9.35	49369	1.57	86238	2.73

Table 2 shows that the water demands for Newbury and Bothwell are approximately 15% and 30% of the total SWM consumption, respectively. When determining the placeholder demands for Newbury and Bothwell, WSP took 15% and 30% of the SWM domestic demands determined previously and assigned them at the boundary of the network to represent system outflow to the Newbury and Bothwell distribution system.

Figure 4 shows the location for the future developments in SWM, and the associated demand calculation for each development is also provided in Appendix A. Table 3 summarizes the calculated future domestic demand for the proposed development using the design rates described in Table 1.

Figure 4: Southwest Middlesex Future Developments

Table 3 - Summary of the Future Water Demand

	AVERAGE DAY DEMAND (L/S)	MAXIMUM DAY DEMAND (L/S)	PEAK HOUR DEMAND (L/S)
Future Water Demands	5.91	17.72	24.22

Domestic demands for future developments in SWM were allocated and assigned to the closest junction in the 2041 planning horizon.

2.2 MODEL CALIBRATION

Ahead of using the model to predict performance and identify challenges and solutions, the model had to be calibrated to available field data. The macro and micro calibration were both completed under the 2021 Average Day Demand scenario and carried into the future/ultimate buildout planning horizon.

The macro calibration was based on the available SCADA data, including 2018 to 2020 inflow/outflow data from the network storages: the SWM Reservoir, Glencoe Elevated Water Tower, and Melbourne Standpipe. More details on the SCADA data were provided in the plots for year 2018, 2019, and 2020 which are provided in Appendix D. In reviewing the SCADA data, WSP identified some data gap in the 2019 and 2020 data; therefore, WSP used the 2018 SCADA data for the model calibration given that it had the most consistent data and provided a full data set. That being said, we supplemented the 2018 data set with the 2019 and 2020 data as required to get the best understanding and calibration. Table 4 summarizes the average flow at the three water facilities and the HGL/water level set at the corresponding reservoir/tank levels – the strategy is to have the reservoir/tank level and the generated outflow match the SCADA data.

Table 4 - Summary of the SCADA Data from 2018 to 2020

	AVERAGE	MINIMUM	MAXIMUM
Glencoe Reservoir Outflow	16.67 L/s	0.05 L/s	31.45 L/s
Glencoe Elevated Tank Level	8.27m	5.90m	10.01m
Glencoe Elevated Tank Outflow	7.59 L/s	0.66 L/s	14.89 L/s
Glencoe Elevated Tank Inflow (Calculated)	8.00 L/s	N/A	N/A
Melbourne Standpipe Inflow*	0.15 L/s	0.06 L/s	0.55 L/s
Melbourne Standpipe Outflow	1.36 L/s	0.12 L/s	4.72 L/s
Melbourne Standpipe Level	32.21m	30.09m	35.76m

*Note: Summarized SCADA data for Melbourne standpipe inflow was taken from 2019 data instead of 2018 data. The 2018 Melbourne standpipe inflow SCADA data showed the same number over the entire measurement period, and therefore, WSP considered the 2018 Melbourne standpipe inflow data not reliable and used the 2019 SCADA data instead.

As part of the calibration, WSP completed a water balance to make sure that the "water entering" the system from storages was equal to the "water leaving" the system from demands. A water balance was applied using the following equation:

$$\sum Outflow - \sum Inflow - \sum Demands = 0$$

The "Sum of Outflow" includes outflows from the SWM Reservoir, the Glencoe Elevated Tower, and the Melbourne Standpipe obtained from the SCADA Data inputs; the sum of inflow includes inflow to the Glencoe Elevated Tower and the Melbourne Standpipe; and the sum of demands reflects only domestic water demands for SWM (including Wardsville, Glencoe, Appin, and Melbourne), Newbury, and Bothwell. No data was available to quantify leakage/pressure dependent demands, and therefore they were captures in the "domestic water demands".

When comparing the calculated water demands, using the population method, to SCADA outflow data, we recognized that the calculated value was higher than the measured outflow. The SCADA outflow data ("measured")_was 11.89 L/s instead of the population-based calculation of 20.6 L/s. As a result, the water demand loadings in the model were reduced globally by a factor of 1.73 so that the sum of demands in the model equalled to 11.89 L/s, and the place holder demands for Newbury and Bothwell was also adjusted to 1.78 L/s and 3.57 L/s respectively, which was 15% and 30% of the SWM total demand loading. Table 5 summarizes the sum of the reduced demand loading in the model under existing condition.

Table 5 - Summary of the Existing Domestic Water Demand Loading in Model After Calibration

	AVERAGE DAY DEMAND (L/S)	MAXIMUM DAY DEMAND (L/S)	PEAK HOUR DEMAND (L/S)
SWM	11.89	35.67	48.75
NEWBURY	1.78	10.70	14.62
BOTHWELL	3.57	5.35	7.31
TOTAL - EXISTING	17.24	51.72	70.68

2.2.1 VALIDATION OF MODEL CALIBRATION

Once we validated that the model was calibrated to SCADA data, a micro calibration of the model was completed by adjusting the pipe C-Factors in order for static and residual pressures throughout the study area to match with the 2018 hydrant flow test results provided by the Municipality. WSP used the following test data to validate the model calibration. Table 6 summarizes the hydrant flow tests used to validate the model calibration.

Table 6 - Hydrant Flow Tests for Model Validation

HYDRANT ID	PRESSURE ZONE	ADDRESS	DATE OF TEST
29	Wardsville	21878 Hagerty Rd	2018/9/19 6:46 AM
47	Wardsville	1948 Longwoods Rd.	2018/9/17 2:15 PM
33	Wardsville	22051 Talbot St.	2018/9/18 5:45 PM
19	Wardsville	187 Queen St.	2018/9/18 4:05 PM
62	Glencoe	203 Reycraft St.	2018/9/6 9:30 AM
5	Glencoe	266 Appin Rd.	2018/9/12 6:09 PM
97	Glencoe	154 North St.	2018/9/12 10:51 AM
15	Glencoe	3578 Concession Dr.	2018/9/10 11:41 AM
36	Glencoe	181 Main St.	2018/9/11 9:27 AM
3	Melbourne	6463 Longwoods Rd.	2018/9/18 9:46 AM
11	Melbourne	1907 Archer St.	2018/9/18 8:08 AM
18	Melbourne	21985 Melbourne Rd.	2018/9/18 12:06 PM
4	Melbourne	6507 Longwoods Rd.	2018/9/18 10:07 AM
6E	Rural	22189 Melbourne Rd.	2018/9/19 4:44 PM
20M	Rural	2351 Longwoods Rd.	2018/9/17 12:02 PM
17M*	Rural	2730 Longwoods Rd.	2018/9/17 9:56 AM

Comparisons between the hydrant flow test results and the modelled hydrant flow curve were done at each of the test locations. It was found that the difference between modelled static pressures and hydrant flow test static pressures were within 10% for most of the tests, except for Hydrant W029 in southern Wardsville that was simulated with a static pressure roughly 14% higher than the hydrant test.

When further comparing the simulated hydrant flow curves to the tested flow curves, we can say that both the residual pressures and extrapolated flows at 140 kPa (20 psi) are conservative in that the flow estimated by the model is lower than that calculated from the hydrant test for hydrants in Glencoe and Melbourne. Two hydrants, in the southern Wardsville with lower elevation, were simulated with flow at 20 psi higher than the extrapolated flow from the test, and one of the rural Hydrant R017M, located at the intersection of Longwoods Rd. and Pratt Siding Rd., was simulated with fire flow slightly higher than the test data. Appendix D provides detailed results of the model verification using the hydrant flow tests.

2.3 DESIGN CRITERIA

2.3.1 SYSTEM PRESSURE REQUIREMENTS

The Ministry of the Environment Conservation and Parks (MECP) pressure criterion stipulates a minimum of 40 psi (275 kPa) and maximum pressure of 100 psi (690 kPa) under domestic demand conditions. Under fire flow conditions, pressures above 20 psi (140 kPa) must be maintained.

It is important to note that the Ontario Building Code (OBC) requires individual pressure regulating valves if pressures are above 80 psi (550 kPa).

2.3.2 FIRE FLOW REQUIREMENTS

In this planning study, WSP is presenting the Available Fire Flow (AFF) for information. Specific fire flow calculations will have to be completed at the time future Site Plan are available and applications are submitted in this Study Area.

2.4 BOUNDARY CONDITIONS

The SWM Reservoir and Booster Pump Station are located in the Village of West Lorne and receives water supply from the Tri-County Drinking Water System. The SWM BPS consists of two fixed speed pumps rated at 2943 m^3/d and two emergency backup pumps rated at 1226 m^3/d. This BPS has a total firm capacity of 5395 m^3/d and serves as the principal supply to the SWM Water Distribution System. Table 7 summarizes the pump status considered in this analysis for the SWM BPS under all conditions.

Table 7 - Operational Status for the SW Middlesex BPS

	AVERAGE DAY DEMAND	MAXIMUM DAY DEMAND	MAXIMUM DAY DEMAND PLUS FIRE FLOW	PEAK HOUR DEMAND
PUMP 1	ON	ON	ON	ON
PUMP 2	OFF	OFF	OFF	OFF
PUMP 3 (BACKUP)	OFF	OFF	OFF	OFF
PUMP 4 (BACKUP)	OFF	OFF	OFF	OFF

No Pump On/Off SCADA data or station SOP was available to validate the pump status used in this analysis. From WSP experience however, this is the only pump station in the system, and it is reasonable that pumping is often On, particularly during MDD and PHD scenarios.

The Glencoe Water Tower has a total capacity of 3600 m^3 and receives supply from the SWM BPS to serve as the primary supply for the Village of Glencoe. By analyzing the SCADA data as shown in Table 4, the tank water level for the Tower was set to 264.27m under all scenarios, which is approximately 59% of the total tank level. From WSP's experience, 60%-85% Top Water Level (TWL) is a typical operation range for elevated storages. Simulation with an approximately 60% TWL is therefore deemed acceptable and conservative.

Similarly, the Melbourne Standpipe receives supply from the SWM BPS and serves as the primary supply for the Village of Melbourne. The tank water level for the Standpipe was set based on the SCADA data summarized in Table 4, and hence the initial TWL in the standpipe was set to 251.52m (30m high) under Average Day Demand (ADD), which is close to the recorded minimum standpipe level, and 253.7m (32.19m high) under Maximum Day Demand (MDD) and Peak Hour Demand (PHD), which is close to the recorded average standpipe level.

An existing Pressure Reducing Valve (PRV) is located at the intersection of Longwoods Rd. and Pratt Siding Rd. to reduce pressure in the Village of Wardsville. The setting of the PRV was adjusted to 340 kPa based on the Static pressure measured through the Hydrant Flow Test for rural Hydrant R017M located downstream of the PRV.

3 WATER HYDRAULIC ANALYSIS

Included in this analysis are the simulation of hydraulic conditions for both the existing and future demand conditions. The simulations were completed for both 2021 and 2041 planning horizons and included the following scenarios:

1. Average Day Demands (ADD);
2. Maximum Day Demands (MDD);
3. Maximum Day Demands + Fire Flows (MDD+FF); and,
4. Peak Hour Demands (PHD)

WSP simulated and mapped results for the worst-case scenario (PHD and MDD+FF) as to identify existing network challenges that create pressure deficient areas, excessive head loss through pipes and fire flow constraints, if any. This exercise was completed a second time after adding new demands for residential and non-residential land uses for the 2041 planning horizon.

3.1 SYSTEM PRESSURE

WSP simulated the existing conditions for the study area that reflect demands established in Section 2 and summarized in Table 5. These existing condition results will be used as a baseline to compare and quantify the impacts of intensification. Table 8 summarizes the simulated pressures in SWM under all scenarios.

Table 8 - Simulated Service Pressure for the SWM – Existing Conditions

	AVERAGE DAY DEMAND (KPA)	MAXIMUM DAY DEMAND (KPA)	PEAK HOUR DEMAND (KPA)
2021	259 – 659	264 – 654	259 – 645
2041	259 – 658	264 – 652	258 – 641

Note:

- Junction J-0629 is a local low elevation point along the 350mm Graham Rd. transmission main & was simulated with pressures above 690 kPa in all conditions. Upon evaluation, it was deemed to not be a concern since no service connection are attached along the transmission line, and hence Junction J-0629 was excluded from the pressure summary.
- Melbourne Standpipe was simulated with a higher water level under MDD and PHD, resulting an increase at the lower end of the pressure range.

Complete tables of node and pipe data for the simulated results are included in Appendix B. In addition, maps presenting the pressure at all junctions and headloss in all pipes within the study area are provided in the Appendix A.

Table 8 shows that the simulated pressures, within the study area, are expected to range from 258 kPa to 659kPa with the existing and planned domestic demands and watermains. The simulations show that most junctions within the study area can maintain pressures above the minimum service pressure requirement, with the exception of six (6) junctions on Parkhouse Dr. and Melbourne Rd. that were simulated below 275 kPa. These junctions are all located along the transmission main with no service connection; however, this needs to be confirmed with the Municipality. Figure 5 shows the locations of junctions with low pressure in Appin and Melbourne during the 2041 PHD scenario. To analyze the cause of pressure loss along the 200mm watermain on Parkhouse Dr. and the 150mm watermain on Melbourne Rd., WSP generated profiles along the highlighted mains shown in Figure 5.

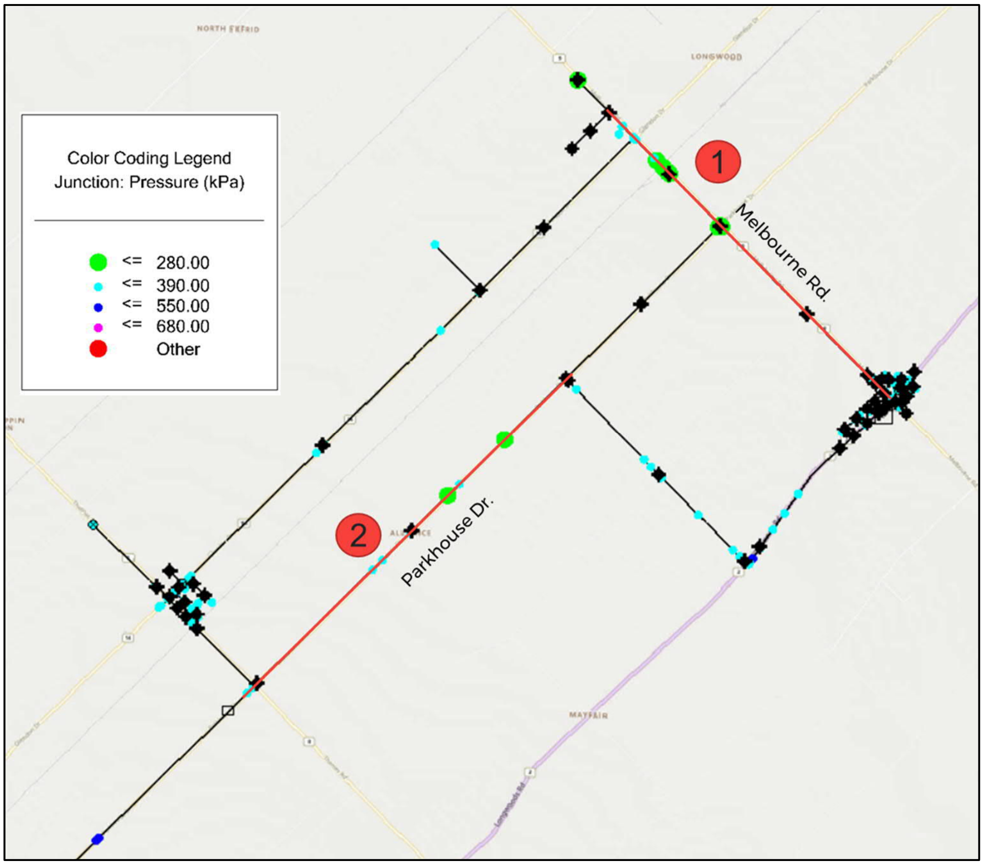


Figure 5: Junction with Simulated Pressure below 275 kPa under 2041 PHD

Figure 6 and Figure 7 show the simulated HGL, elevation inputs, and simulated pressure profile for the existing watermains along Parkhouse Dr. and Melbourne Rd. respectively. As shown in Figure 6, an elevation drop of 6.55m can be observed along the 150mm Melbourne main from south to north, which resulted in a significant pressure drop of approximately 60 kPa with a headloss gradient of 0.33 m/km. The junction with the lowest pressure was simulated at 258 kPa which was approximately 6% lower than the minimum requirement of 275 kPa. Similarly, Figure 7 shows an elevation drop along Parkhouse Dr., between Thames Rd. and Springfield Rd., causing the pressure to drop below 275 kPa at the highest point.

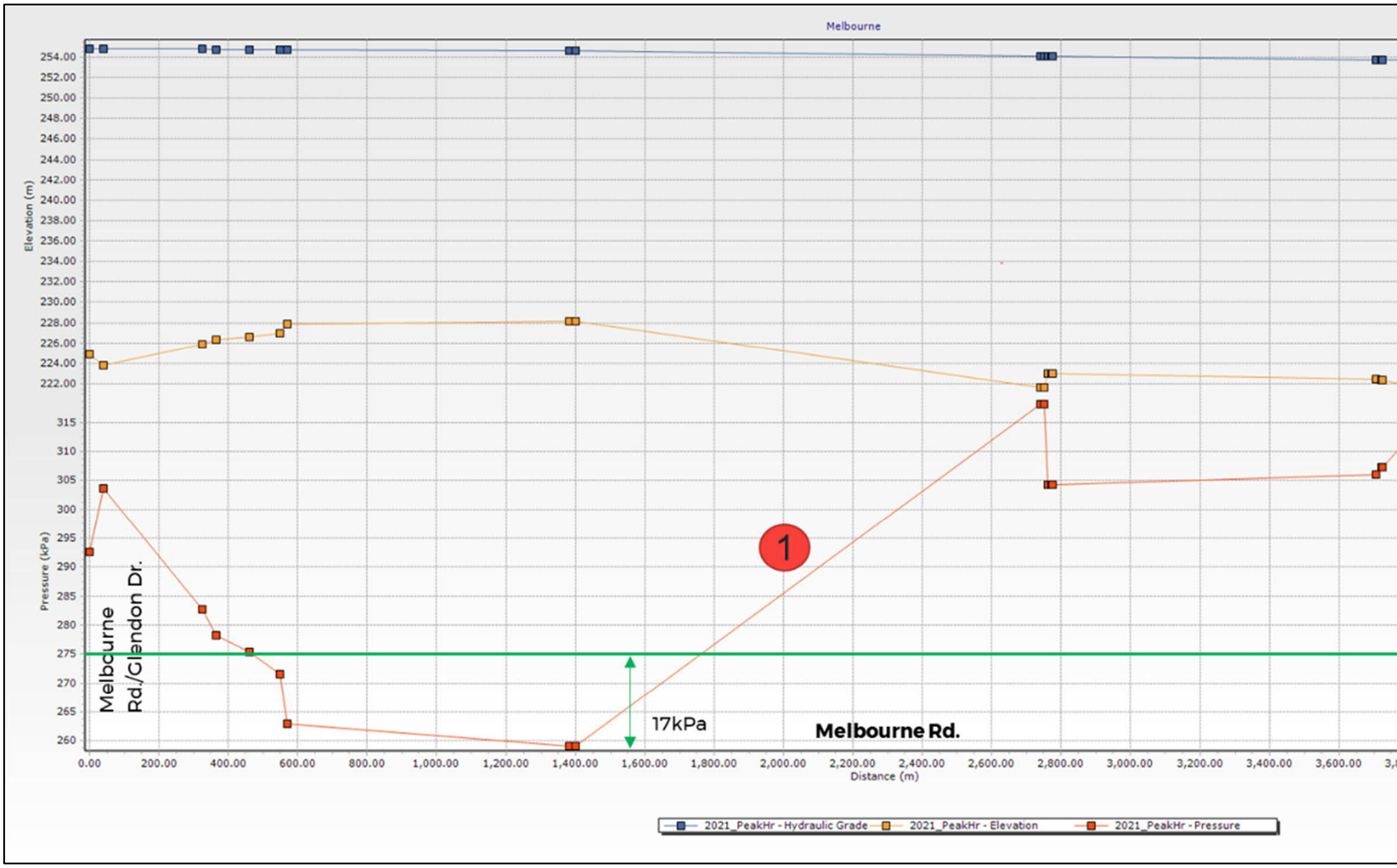


Figure 6: HGL, Elevation, and Pressure Profile along the 150mm Melbourne Rd. Watermain

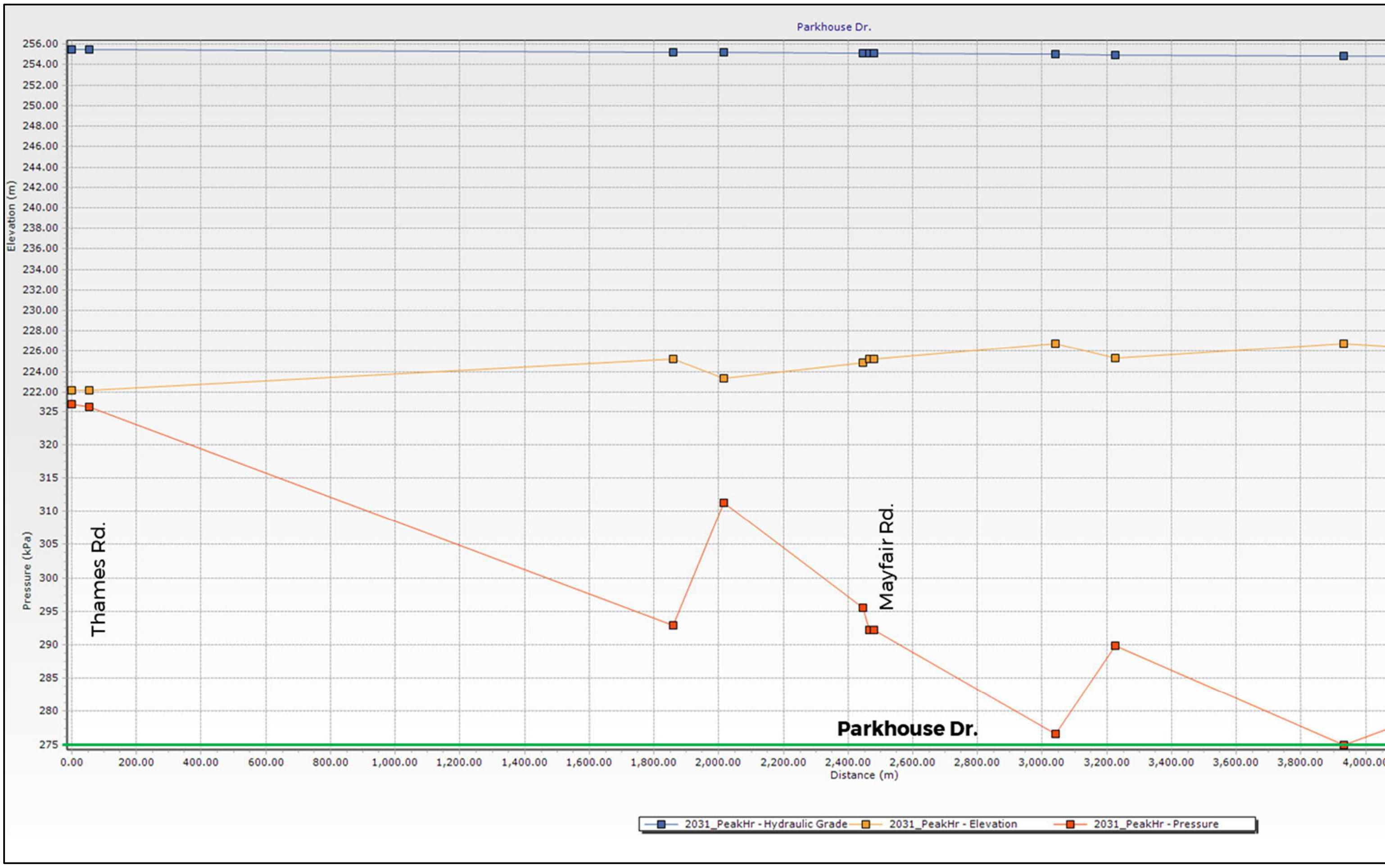


Figure 7: HGL, Elevation, and Pressure Profile along the 200mm Parkhouse Dr. Watermain

Based on simulations, that were validated in conversation with the Municipality, the Village of Wardsville was simulated to operate at a high-pressure range due to the fact that it's located at a lower elevation than the remaining communities and supplies. The existing PRV located at the intersection of Longwoods Rd. and Pratt Siding Rd. was included in the model to reduce pressure in the Village of Wardsville. The setting of the PRV was set to 340 kPa based on the Hydrant Flow Test results for hydrant R017M located downstream of the PRV as described in Section 2. All junctions in Wardsville were simulated within the allowable range as indicated in Section 2.3.1.

In addition, the 2041 PHD pressure and headloss map shows that the Municipality's existing watermains generally have head losses lower than 2m/km – this is an acceptable result for water distribution systems. Figure 8 shows the simulated headloss along existing watermains in the Village of Glencoe under 2041 PHD condition. The Village of Glencoe received its primary supply through a single 300mm connection at the intersection of Parkhouse Dr. and Victoria St., supplying a total flow of approximately 59 L/s under 2041 PHD. As a result, the existing 250mm watermain along Victoria St. between Parkhouse Dr. and Prince William St. was simulated with headloss greater than 2m/km – which is higher than what is deemed reasonable. Also, the existing 150mm watermain on Parkhouse Dr. and Deane St., connecting the 250mm Victoria St. Watermain to the 200mm Main St. watermain, resulted in a bottleneck and were simulated with excessive headloss over 5m/km.

As a result, WSP recommends adding a second connection into Glencoe along Main St., and this will provide two supply mains into Glencoe rather than one and help reduce headloss along the existing watermains along Victoria St.

Figure 9 shows the simulated headloss on pipes in the Village of Glencoe under 2041 PHD condition, with an addition of a 250mm watermain on Main St. between Industrial Rd. and Parkhouse Dr., serving as a second connection to convey flow from the SW Middlesex BPS to Glencoe. As shown in Figure 9, the simulated headloss on the existing 250mm watermain on Victoria St. between Parkhouse Dr. and Deane St. was reduced below 5km/m with the second connection on Main St.

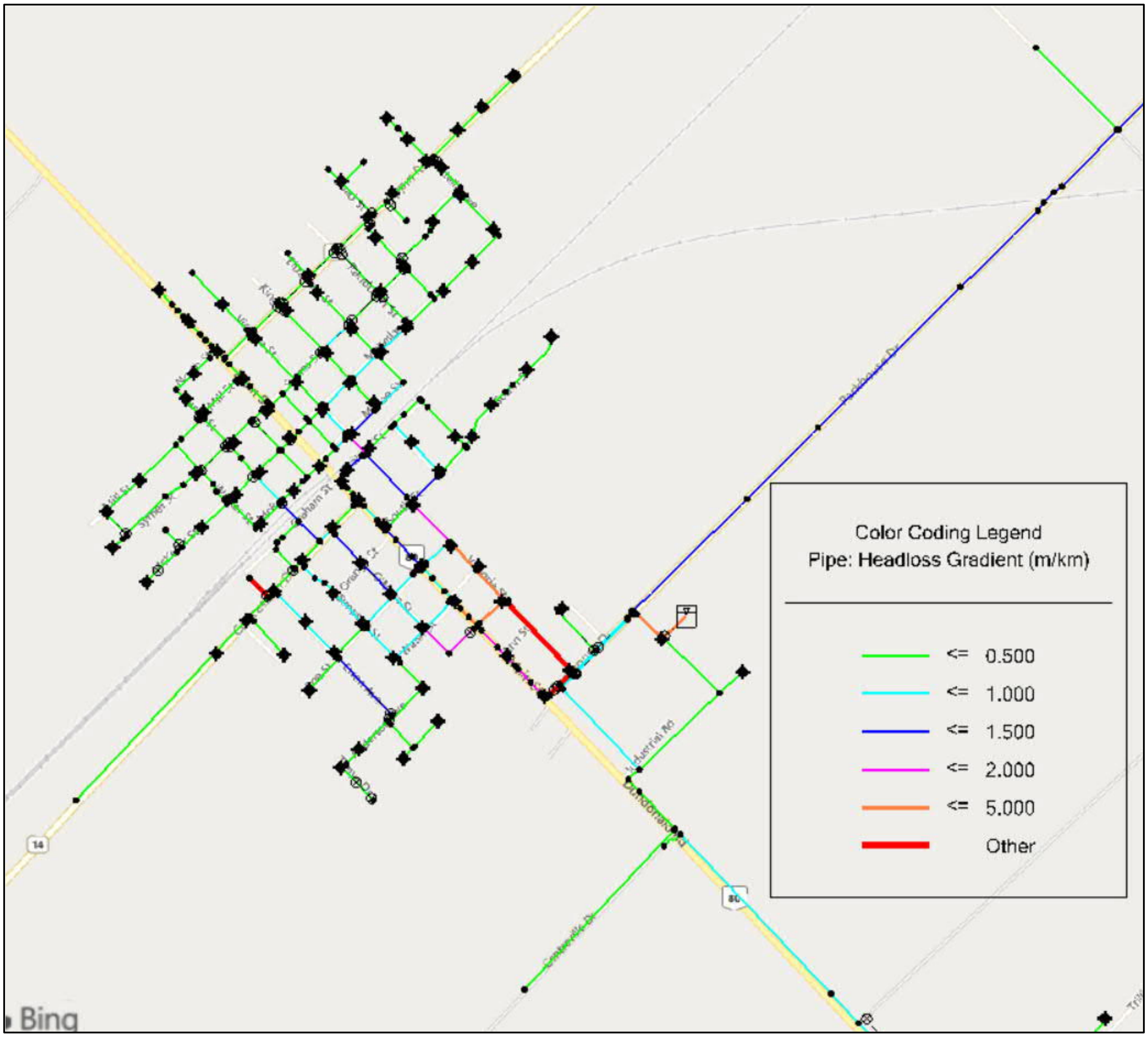


Figure 8: Headloss Gradient on Pipes in Glencoe under 2041 PHD

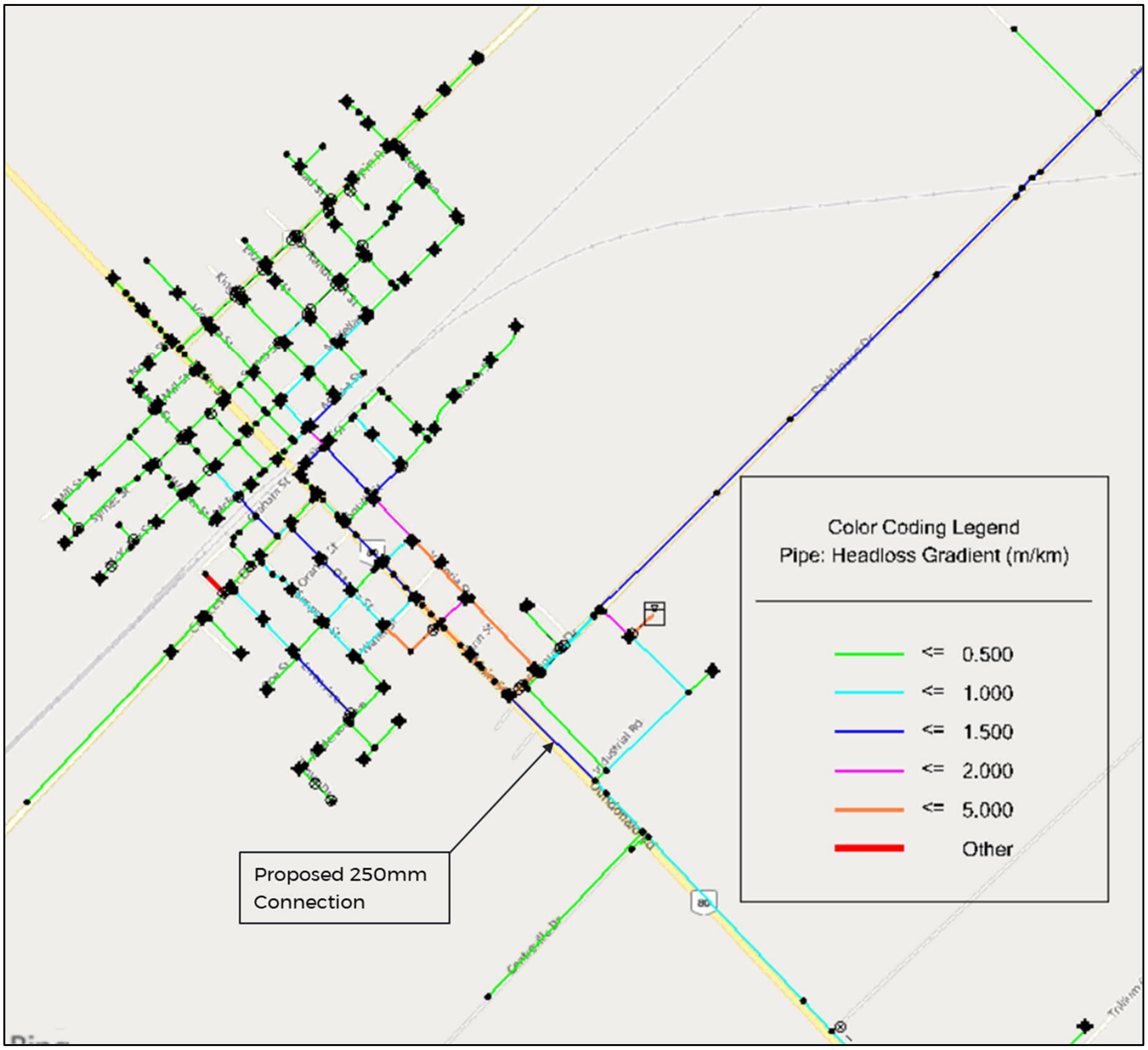


Figure 9: Headloss Gradient on Pipes in Glencoe under 2041 PHD with a Second Connection on Main St.

3.2 AVAILABLE FIRE FLOW

The MDD+FF was modelled considering boundary conditions described in Section 2.4 under both 2021 and 2041 planning horizon and were captured at all hydrants in the model and compared the hydrant classifications for an extra step in validation. Table 9 summarizes simulated available fire flow on all hydrants within the Municipality under 2021 and 2041 planning horizon.

Table 9 - Simulated Service Pressure for the SWM – Existing Conditions

	WARDSVILLE (L/s)	GLENCOE (L/s)	APPIN (L/S)	MELBOURNE (L/S)	RURAL (L/S)
2021 MDD+FF	40 – 77	34 – 430	22 – 27	41 – 173	13 – 121
2041 MDD+FF	40 – 77	32 – 421	22 – 27	40 – 173	13 – 120

Complete tables of hydrant data for the simulated fire flow results are included in the appended material. In addition, maps comparing the Available Fire Flow at all hydrants and the hydrant classifications within the study area are provided in the Appendix C.

WSP compared the simulated AFF in the model to the hydrant classification maps provided by the Municipality and concluded that most of the hydrants in the model were classified with the same color coding with a few along dead-end watermains simulated with lower fire flow; however, it can be inferred that the model was conservative. As shown in Error! Reference source not found., all hydrants in the Village of Appin were simulated with fire flow lower than 500 gpm (32 L/s), which was a known challenge in the Municipality. Under existing condition, none of the hydrants within the Appin network can be used for fire events as reported by the Municipality's staff. Figure 10 and Figure 11 presents all the existing hydrants with simulated fire flow lower than 50 L/s in the Village of Wardsville, Glencoe, Appin, and Melbourne.

Figure 10: Simulated Fire Flow Lower than 50 L/s in Wardsville and Glencoe under 2041 MDD

(Note: Hydrants highlighted in red represent hydrants with simulated AFF lower than 50 L/s)

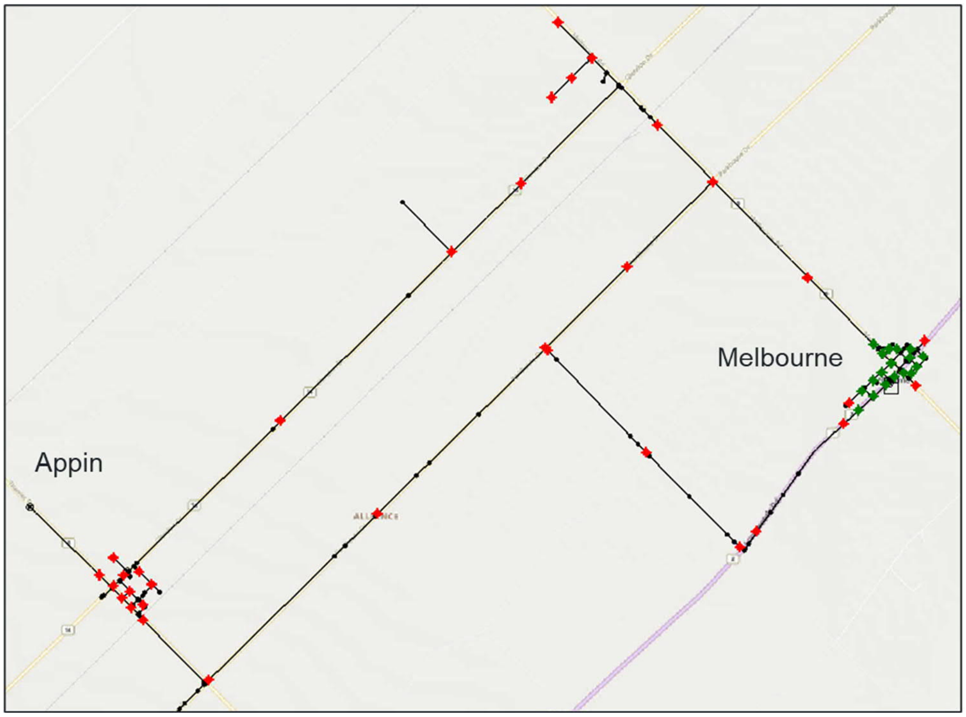


Figure 11: Simulated Fire Flow Lower than 50 L/s in Appin and Melbourne under 2041 MDD

(Note: Hydrants highlighted in red represent hydrants with simulated AFF lower than 50 L/s)

As shown in Figure 10, most of the hydrants within Wardsville and Glencoe were simulated with AFF greater than 50 L/s, with a few dead-end hydrants simulated with fire flow lower than 50 L/s. In Figure 11, it can be seen that hydrants in Appin as well as rural hydrants in the northeast SWM were simulated with AFF lower than 50 L/s. This can be attributed to the fact that its a dead-end system. In conversation with the Municipality, WSP understands that due to the fire flow limitations in the Village of Appin, no future developments or population growth are planned for this area until the fire flow limitations are resolved.

To increase the fire flow availability in this area, an in-line booster pump station is recommended to boost pressure and fire flow in the Village of Appin. WSP ran additional simulation by adding two booster pumps with a design flow and head of 20 L/s and 40 m and 40 L/s and 80m, respectively, at the northwest corner of Parkhouse Dr. and Thames Rd. In addition to the proposed booster pump station, a 200mm supply watermain along the easement west of Thames Rd., crossing the railway and connected to the dead-end junction on Dugald St., was recommended and added in the model to improve water resilience for the Appin network. With the smaller pump on, the simulated fire flow in Appin under 2041 MDD+FF ranged between 32 L/s and 40 L/s with a 50% increase in average compared to the baseline. While with the bigger pump on, the simulated fire flow in Appin ranged between 38 L/s and 53 L/s with approximately 92% increased in average; however, junctions along Parkhouse Dr., located on the suction side of the proposed pump station, were impacted, and simulated with pressure lower than 275 kPa. No service connection was added in the model along the 200mm Parkhouse watermain, but this needs to be validated in a further EA study.

4 WATER QUALITY

4.1 BACKGROUND & OBJECTIVES

The Municipality of Southwest Middlesex's distribution system obtains its water from the Tri-County water supply system which is operated by the Ontario Clean Water Agency (OCWA). The distribution system services the towns of Wardsville, Glencoe, Appin, and Melbourne. It consists of a reservoir, a high lift pumping station, a re-chlorination facility, a standpipe, and a tower. The reservoir services these two water storage facilities: the Glencoe tower and the Melbourne standpipe. The Municipality has identified the following challenges in its water system:

- Maintaining adequate free chlorine residuals within the Melbourne standpipe during summer months.
- Manganese in soluble form is suspected to be present in the distribution system. In 2012, manganese was identified in the raw water in soluble form and thus the Municipality concluded that manganese was able to pass through the water treatment plant membrane filtration system.

The objective of this section is to investigate these challenges and propose recommendations on a solution or adequate next steps in finding a solution at the storage/treatment facility. Solutions that would involve the piping in the distribution system are not considered.

4.2 MELBOURNE STANDPIPE – CHLORINE RESIDUAL

The Melbourne standpipe system as shown in Figure 12 below has a storage capacity of 1,589 m^3 and its water level is maintained through a flow control valve operated by the SCADA system. It also includes a re-chlorination facility which consists of two (2) chemical metering pumps and one (1) 200L sodium hypochlorite storage tank in a prefabricated enclosure. Before summer 2021, re-chlorination was performed based on a fixed steady dose (i.e., not flow-paced) at the inlet of the standpipe. Since then, re-chlorination has been switched to the outlet of the standpipe and is flow-paced based on a set-point of 1.0 mg/L.

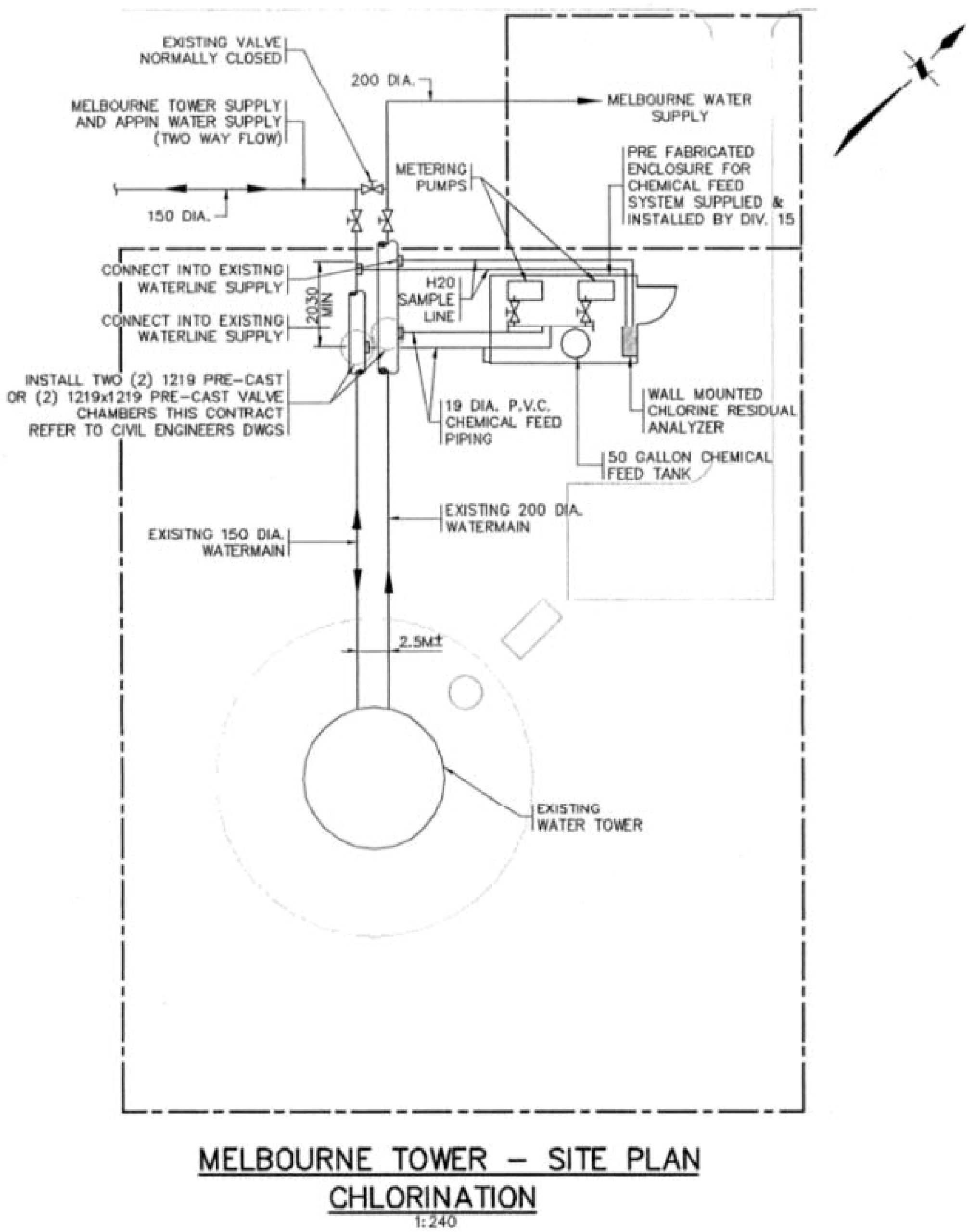


Figure 12 - Melbourne Standpipe and Re-Chlorination Systems Site Plan Drawing

Due to standpipes requiring a fixed storage volume in order to provide adequate pressure in the downstream distribution system, the usable capacity is typically limited. Indeed, the Melbourne standpipe can only use approximately 31% (i.e., 494 m^3) of its total capacity. This leads to high average water age in standpipes, which in summertime can be an issue for maintaining chlorine residuals due to solar exposure which leads to stored water undergoing thermal stratification (i.e., temperature gradient from low to high along the standpipe height) and thus off-gassing its chlorine residual. Canadian Drinking Water Guidelines require a minimum free chlorine residual of 0.2 mg/L at the tap, and in most distribution systems, a free chlorine residual between 1.0-2.0 mg/L at the outlet of a given storage facility is sufficient to meet the guidelines.

Available SCADA data on the daily average and minimum free chlorine levels (in mg/L) at the outlet of the Melbourne standpipe was plotted for the years 2018, 2019, and 2020 in Figure 13, Figure 14 and Figure 15 below, respectively. 2021 SCADA data was not used as it was deemed not reliable for the purposes of this analysis due to continuity gaps. Analysis of the available data reveals the following:

- Figure 13 shows that in March, the outlet chlorine residual is fairly constant at around 0.95 mg/L. However, by mid-April the outlet chlorine residual sharply drops. This coincides with a temperature high of 20°C in the region on April 12th, a sharp increase from the previous seasonal high of 4°C on April 2nd. Following this, with the onset of spring and summer that comes with warmer temperatures, one can observe that the outlet chlorine residual struggles to reach close to 1.0 mg/L and mostly remains between 0.5-0.8 mg/L with the occasional overshoot as the system attempts to overcompensate for the low residuals.

- Figure 14 shows a similar pattern as in Figure 13 in the seasonal transition months of September and October as summer ends and autumn starts. The outlet chlorine residuals mainly remain between 0.5-0.8 mg/L in September and as the temperatures start going down in October, they increase and appear to stabilize around 1.0 mg/L.

- Figure 15 shows outlet chlorine residuals only in the early summer months, but the same trend persists as in Figure 13. The outlet chlorine residuals mainly remain between 0.5-0.8 mg/L.

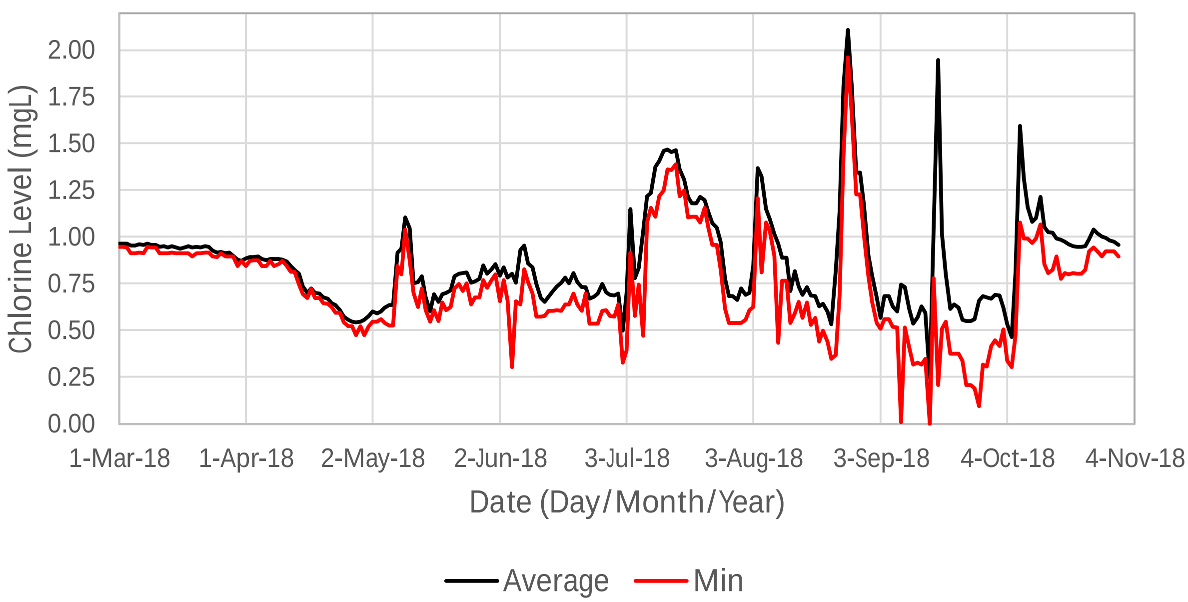


Figure 13 - Daily Free Chlorine Concentrations (Average and Minimum) at the Outlet of the Melbourne Standpipe in 2018 (March 1st – Oct 31st)

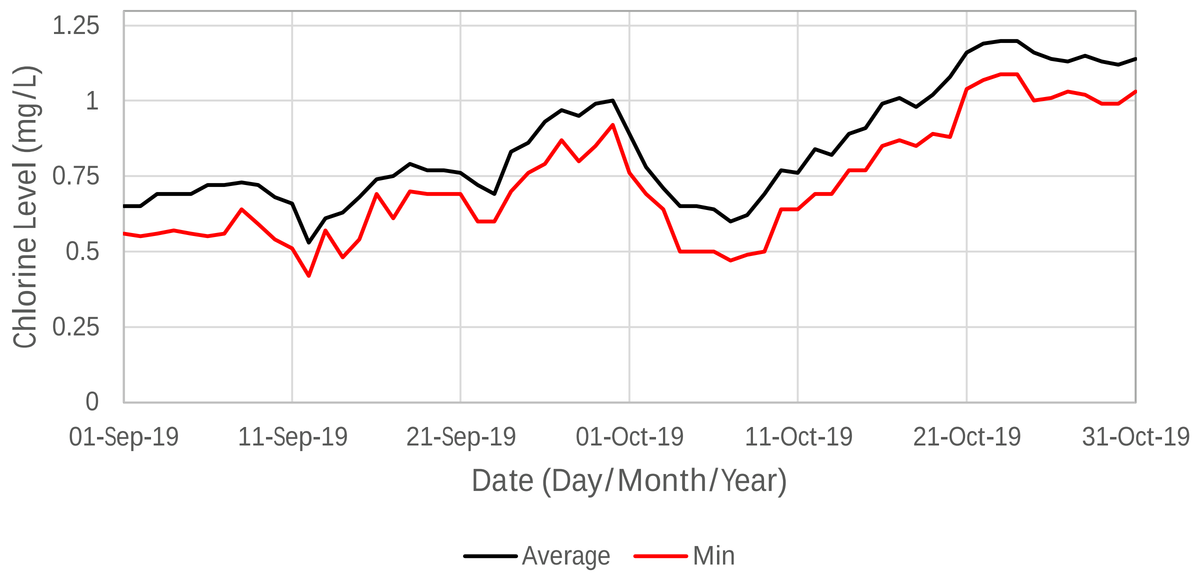


Figure 14 - Daily Free Chlorine Concentrations (Average and Minimum) at the Outlet of the Melbourne Standpipe in 2019 (Sept. 1st – Oct 31st)

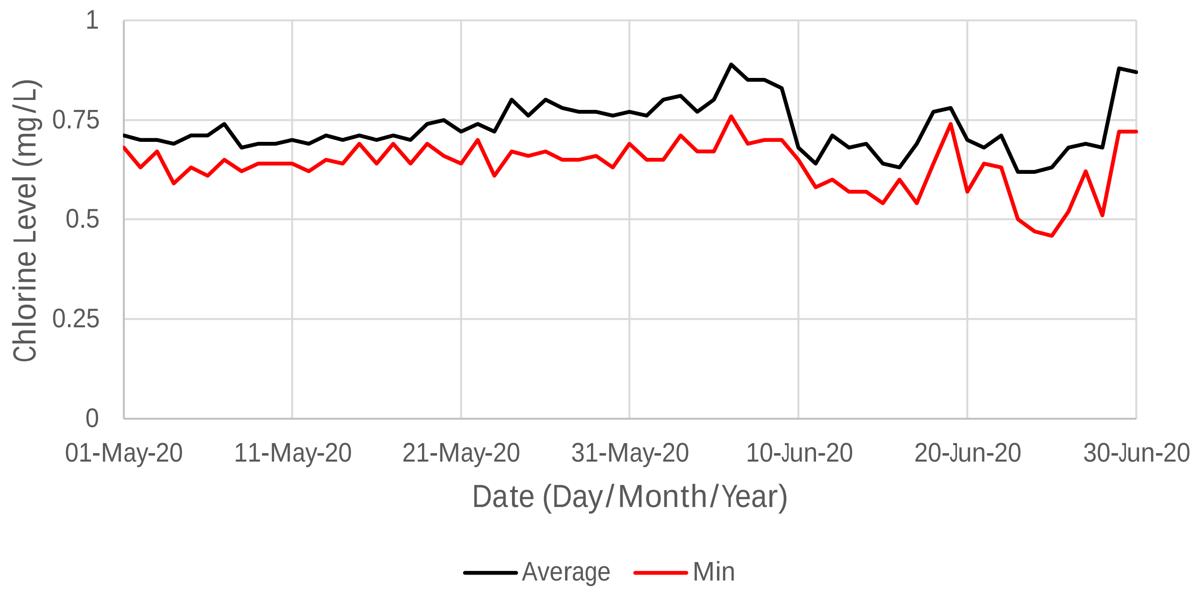


Figure 15 - Daily Free Chlorine Concentrations (Average and Minimum) at the Outlet of the Melbourne Standpipe in 2020 (May 1st – June 30th)

Based on the above analysis of the available data, in order to maintain the outlet residual chlorine at 1.0 mg/L and mitigate its fluctuations in the summer months, the following is recommended in order of priority:

1. Relocate the chlorine dosing point from the standpipe inlet to its outlet and change the dosing control from fixed volume to flow-paced. This will allow for better control of residual chlorine in the distribution system downstream of the standpipe, ensuring that the appropriate amount is dosed for the volume entering the distribution system and that the bulk of it is not lost during storage due to off-gassing. Since the Municipality already made this adjustment in the summer of 2021, it is recommended that the outlet chlorine residual be closely monitored for at least one (1) year (in order to observe all seasonal changes) to confirm the success of the implementation.

2. Provide active mixing in the standpipe to avoid thermal stratification of the stored water which would help mitigate chlorine off-gassing in the hot summer months. In turn, this could also provide savings in sodium hypochlorite usage through reduced dosing at the standpipe outlet. The cost estimate for retrofitting an active mixer in this standpipe is $50,000.

4.3 DISTRIBUTION SYSTEM – SOLUBLE MANGANESE

The water treatment plant servicing the Southwest Middlesex distribution system sources its raw water from Lake Erie. As such, manganese levels are not typically a major concern as in the case of a groundwater source, and the water treatment plant does not have a dedicated manganese removal treatment system. While the water treatment process train includes filters which are capable of removing any particulate manganese, the Municipality has concerns with soluble manganese passing through the process into the distribution system.

Manganese which has accumulated in a distribution system is referred to as “legacy manganese”. It can accumulate through various physico-chemical (i.e., precipitation and sorption), physical (i.e., physical deposition of particulates), and biological (i.e., catalyzed oxidation onto pipe surfaces) mechanisms. Once accumulated, manganese can be released through physical/hydraulic disturbances (i.e., flow velocity increase, flow reversal, hydraulic pressure transient) and bulk water chemical instability (i.e., pH, ORP, dissolved organic carbon concentration, and phosphate concentration changes). Manganese concentration criteria for drinking water are set by the Canadian Drinking Water Guidelines such that the Maximum Allowable Concentration (MAC), a target to protect consumer health, is 0.1 mg/L, while the Aesthetic Objective (AO), a target to ensure consumer acceptance and minimize impact on distribution systems, is 0.02 mg/L. It is typically recommended that drinking water should have no more than 0.01 mg/L total manganese to minimize manganese accumulation in the distribution system.

Available monthly total manganese concentrations obtained from grab samples in the raw and treated water were plotted. Figure 16 shows the average and maximum monthly total manganese concentrations in the raw water from Lake Erie from 2018 to 2021, while Figure 17 shows the same data for the same time period but for treated water at the outlet of the water treatment plant:

- In Figure 16, a total of three significant spikes in total manganese concentrations can be observed, with values reaching maximums between 0.4-1.15 mg/L, well above the MAC. Each of these instances can be tied to weather events that are likely to be the source of the significant increase in total manganese concentrations in Lake Erie at those times. The months of August 2018, October 2019, and September 2021 all recorded unusually heavy rainfalls. In particular, September 2021 stands out with a maximum total manganese concentration in the raw water of 1.15 mg/L. This is due to the rainfall in this month causing severe flooding in Middlesex County which contributed to higher amounts of manganese leaching and being carried into Lake Erie compared to the other heavy rainfall events that had no flooding.

- In Figure 17, it can be observed that most of the time the treated water total manganese concentrations are below detection levels. There are only three (3) instances in which total manganese concentrations were detectable: in August 2018 the average is 0.002 mg/L while the maximum is 0.01 mg/L, well below the AO; in September 2020 the average and maximum are 0.01 mg/L, well below the AO; in September 2021 the average is 0.12 mg/L while the maximum is 0.35 mg/L, well above both the AO and MAC. The August 2018 and September 2021 instances coincide with the ones for raw water that are tied to weather events as discussed above. It can be concluded that while the plant is able to remove total manganese to below the AO level in most weather circumstances, it is unable to do so during extreme weather events such as flooding which result in high amounts of dissolved manganese reaching the raw water source. On the other hand, the September 2020 treated water instance is tied to only a slight uptick in raw water total manganese concentrations (0.15 mg/L) while the October 2019 raw water instance (see Figure 16) is not tied to any uptick in treated water total manganese concentrations. It is likely that this is due to natural variations in the particulate to dissolved mass ratios of manganese in the raw water source.

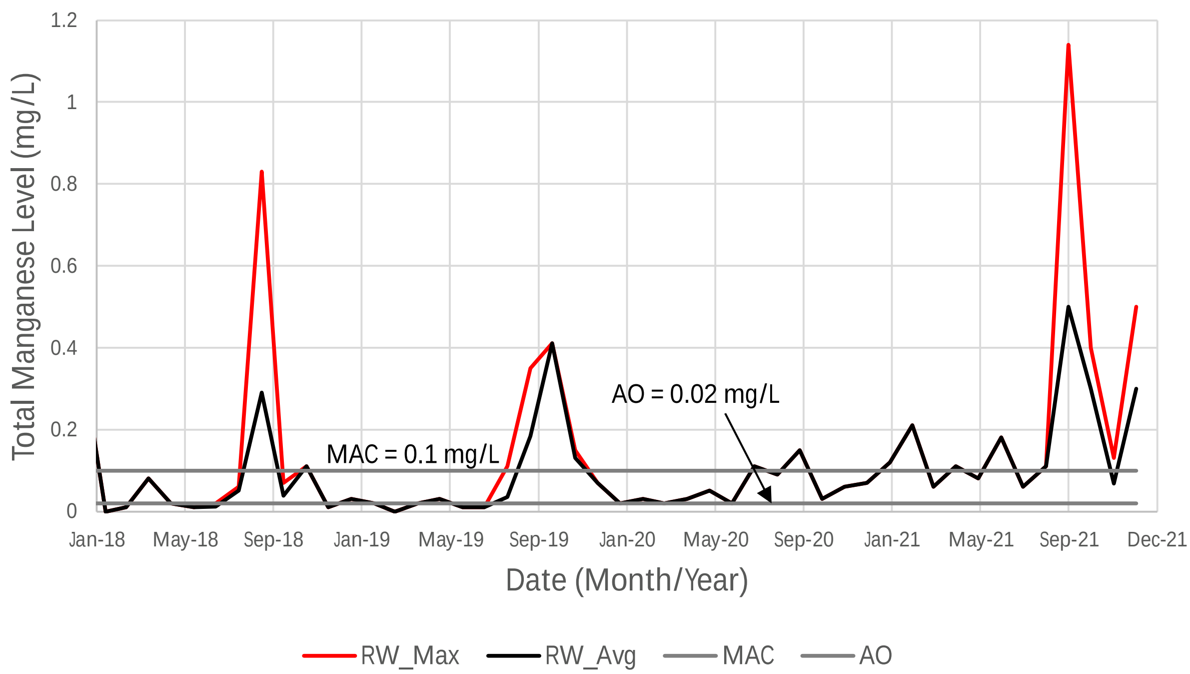


Figure 16 - Monthly Total Manganese Concentrations (Average and Maximum) in the Raw Water at the Inlet of the Water Treatment Plant (2018-2021)

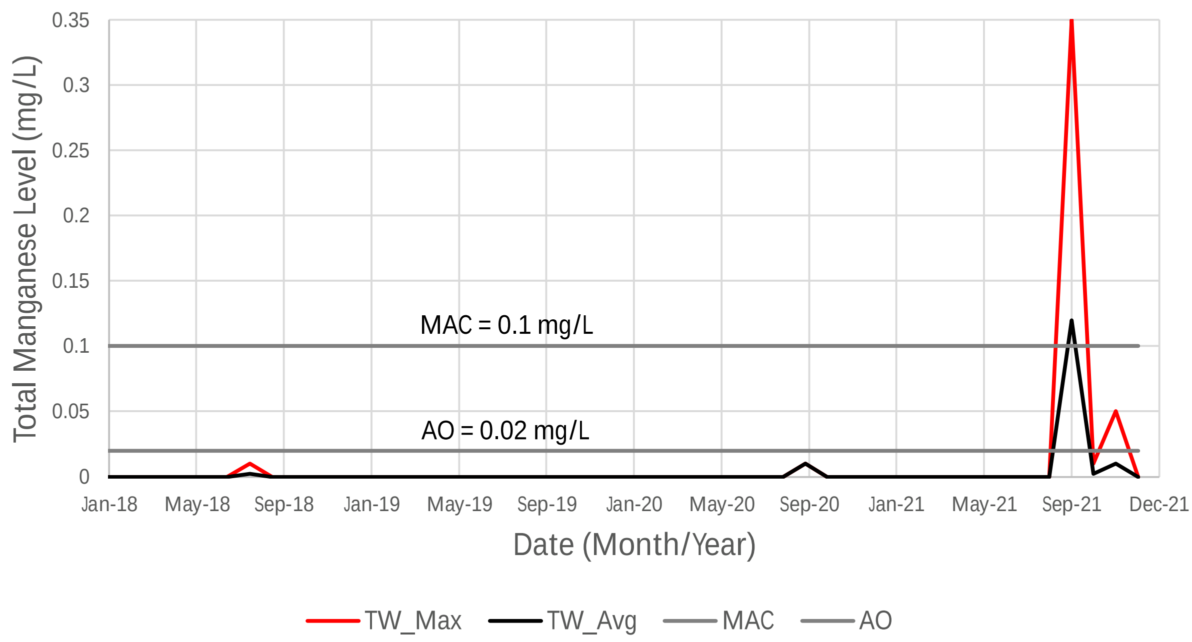


Figure 17 - Monthly Total Manganese Concentrations (Average and Maximum) in the Treated Water at the Outlet of the Water Treatment Plant (2018-2021)

Based on the analysis of the above available data, it can be concluded that the Municipality has dissolved manganese issues in the distribution system during extreme weather events, but otherwise the existing water treatment system can adequately meet the AO levels and thus mitigate manganese levels at the consumer's tap and its impact on the distribution system. Implementing a treatment system that can quickly be brought online during such extreme weather event to meet AO levels and then shut down once it has subsided would be the recommended solution. Appropriate dissolved manganese removal technologies for this objective could include:

- Chemical oxidation through the dosing of chlorine, permanganate, ozone, or chlorine dioxide. Oxidation of dissolved manganese would transform it into particulate form, which can then be removed through physical separation (i.e., filtration, sedimentation, or dissolved air flotation).

- Filters with Oxide-Coated Media. Suitable media could include greensand and pyrolusite. The manganese removal mechanism consists in adsorption of the dissolved manganese to the media surface followed by its oxidation to particulate form which can then be removed through physical separation.

- Nanofiltration membranes which can remove the dissolved manganese without requiring the addition of chemicals to precipitate it.

A feasibility study would be required to evaluate in further details the existing treatment system and which manganese removal technology would be most appropriate. The cost estimate for this study is $30,000.

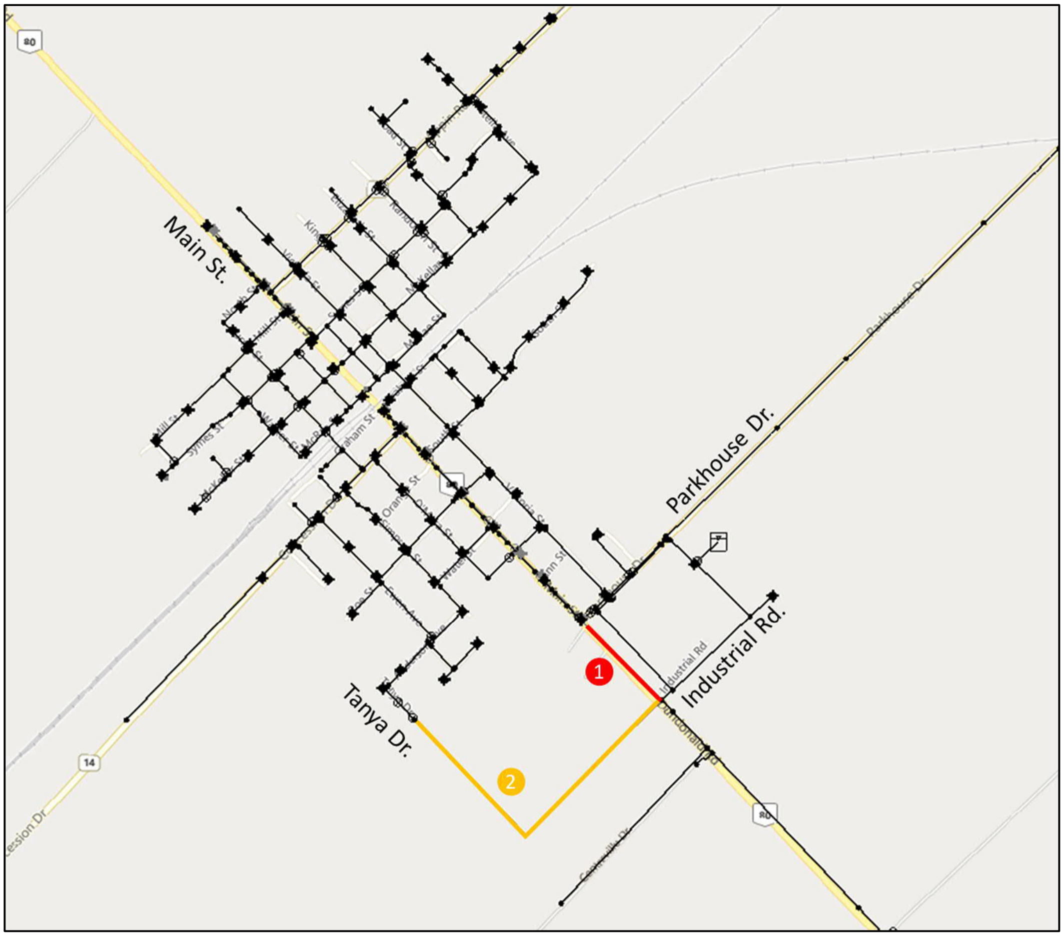
5 CONCLUSIONS & RECOMMENDATIONS - WATER NETWORK

Based on the hydraulic simulations conducted using the WaterGEMs model built by WSP and water quality review, the hydraulic performance of the distribution system in and around the Municipality is as follows:

1. The service pressure is expected range between 258 and 653 kPa with a few junctions on Melbourne Rd. and Parkhouse Dr. closed to the Village of Melbourne were simulated below the minimum pressure requirement, which was caused by a change in elevation.
2. Available fire flows simulated in the model were generally consistent with the hydrant classification maps under 2021 planning horizon are generally above 1000 gpm (63 L/s). The Village of Appin was simulated with fire flow below 500 gpm (32 L/s), and hence, WSP recommends installing an in-line Booster Pumps Station and a second supply main to help increase pressure as well as fire flow in Appin and Melbourne. Junction that has a simulated fire flow below 100 L/s is located on a short dead-end main. With these simulated fire flows, the expected fire flow targets for residential and ICI developments, that range from 75 L/s to 250 L/s, can generally be met.
3. Pipe results for the network indicated that most of the existing watermains within the Municipality can operate with a headlosses below 2m/km, with the exception of the existing 250mm watermain along Victoria St. and the 150mm watermain on Parkhouse Dr. and Deane St., were simulated with headloss over 2m/km. WSP recommends adding a second connection into Glencoe along Main St. parallel to the existing 300mm major supply main east of Main St. This will provide two mains into Glencoe rather than one and reduce headloss along the existing 250mm watermain on Victoria St. at the connection. Alternatively, a second connection could be added through a future development. Adding a second connecting into Glencoe also increasing the security of supply – with the redundant supply, Glencoe would remain in service if the first connection along Main St. is temporarily closed for maintenance or in and emergency condition.

These conclusions remain valid as long as the proposed water distribution system and the Municipalities' network configuration remain as described herein. If significant changes are contemplated, this analysis should be updated. In addition, WSP provided the following recommendations to improve the system performance based on the simulation.

1. Consider adding a second connection to Glencoe along Main St. This will provide two mains into Glencoe rather than one and reduce headloss along the existing 250mm watermain on Victoria St. at the connection. Figure 18 shows the recommended watermain connection as highlighted at the Village of Glencoe to reduce headloss along the existing mains along Victoria St. The second connection can be made along Main St. between Industrial Rd. and Parkhouse Dr., which will connect to the existing 200mm watermain at the intersection of Main St. and Parkhouse Dr. Alternatively, the connection can be made by extending the existing 300mm on Industrial Dr. to the south of Main St. and connecting to the existing dead-ended 150mm watermain on Tanya Dr.



Note: The Red and Yellow Line represent two possible options for a secondary main

Figure 18: Recommended Projects in the Village of Glencoe.

2. To increase fire flow availability within the Village of Appin, WSP recommends installing an in-line Booster Pump Station and adding a secondary supply watermain to the Appin network. Figure 19 shows the recommended location (highlighted in red) for the In-Line Booster Pump for consideration and the proposed secondary supply connection. This would effectively create a third pressure zone and supply additional head to the Appin network that would help in delivering more fire flow capacity to that community; however, the addition of the booster pump station can reduce pressure along Parkhouse Dr. on the suction side of the station, and a detailed EA study needs to be conducted in the future to examine the impact of adding a booster pump station.

   The booster station considered in this analysis had two booster pumps with a design flow and head of 20 L/s and 40 m and 40 L/s and 80m, respectively, at the northwest corner of Parkhouse Dr. and Thames Rd.

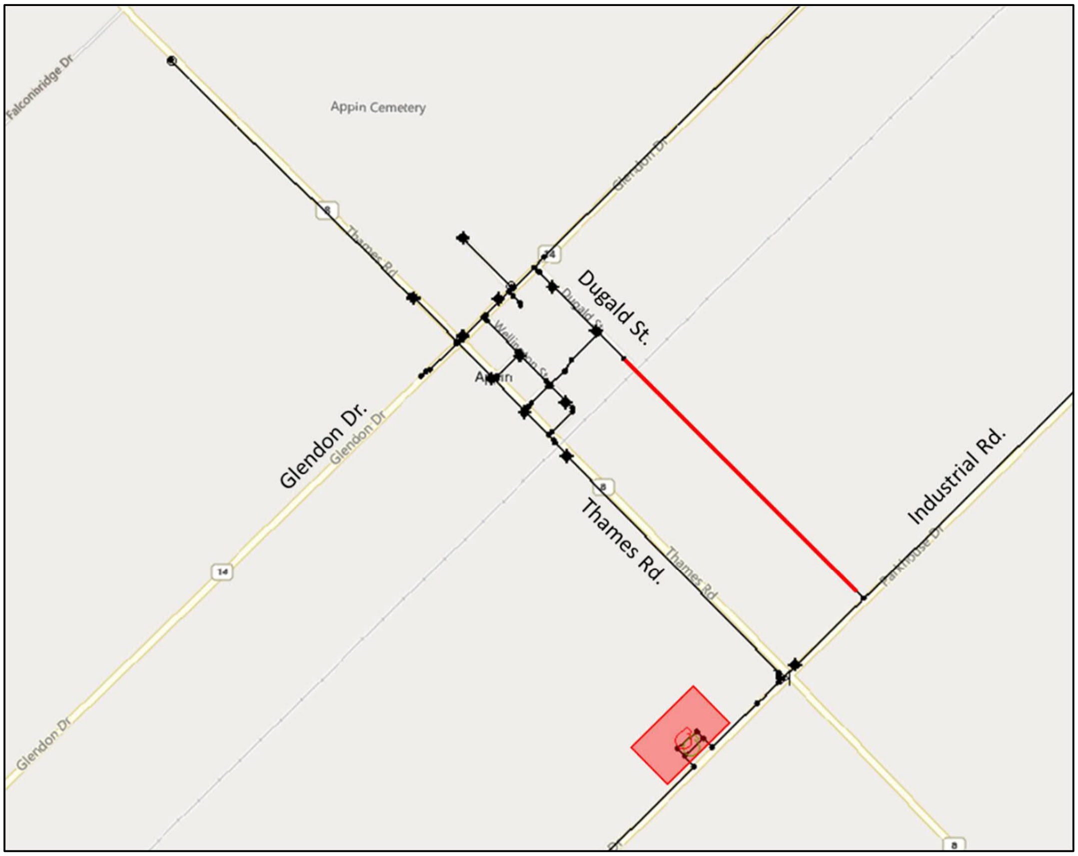


Figure 19: Recommended Project at the Village of Appin

6 WASTEWATER NETWORK

In this study, WSP built a hydraulic model and completed a study for two separate sewer systems in SWM: the systems in the Village of Glencoe and the system in the Village of Wardsville respectively.

The Village of Glencoe is serviced by a gravity sewer system that conveys raw sewage to the Victoria Pump Station and then discharges to the Glencoe Wastewater Treatment Plant, while the Village of Wardsville is serviced by a Small-Bore Sewer (SBS) System which consists of on-site septic tanks to retain sewage flow from each individual connection and only release overflow into the collection system. The SBS system collects sewage flow and conveys it to the Main Street Pump Station where it discharges to the Wardsville Wastewater Treatment Plant.

WSP built steady-state models for the each of these systems using SewerGEMs. The infrastructure layout and details were based on the GIS information and as-built drawings provided by the Client. For the Glencoe system, information for certain conduits were not available in GIS; Start/Stop invert elevations for these conduits were verified against As-built drawings when possible. When not possible, WSP assumed invert elevations on the basis that sewers have a minimum slope of 0.2%.

Please also note that for the steady state model, the SBS network in Wardsville is simplified and includes the collection system only. The sanitary loadings determined for the analysis were loaded on manholes, and retention time in individual septic tank cannot be captured using a steady-state model. To accurately simulate the change in sewage level for individual septic tanks, an Extended-Period-Simulation (EPS) model is required. Figure 20 and Figure 21 show the sewer model for the Village of Glencoe and Wardsville respectively.

Figure 20 – Sewer Model Layout for the Village of Glencoe

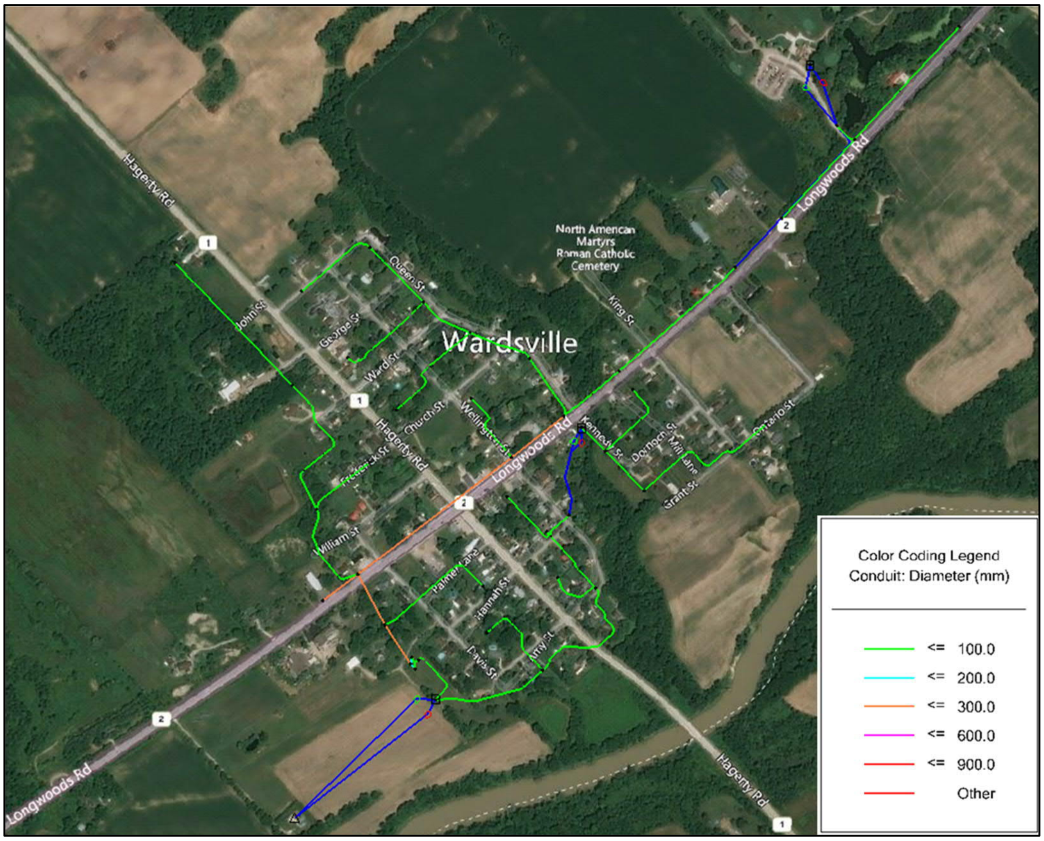


Figure 21 – Sewer Model Layout for the Village of Wardsville

6.1 WASTEWATER DESIGN FLOW GENERATION

The proposed design sewage flow that is expected to be generated by the Study Area consists of the Dry Weather Flow (DWF) multiplied by a Peaking Factor (PF) and supplemented by a rain-induced Infiltration/Inflow (I/I). I/I is assumed to be similar for both existing and future scenarios. Table 10 lists the factors used to calculate wastewater loading, including wastewater design rates and I/I rate obtained from the Southwest Middlesex Municipal Design and Construction Standards 2021.

The wastewater model build started by assembling available GIS layers and estimating the sanitary loadings for the Southwest Middlesex study area including Wardsville and Glencoe. Similar to the domestic water demand calculation, the wastewater calculation was completed by estimating the existing population by counting the number of units for each parcel determined previously in Section 2.1 and multiplying the estimated existing population by the wastewater design rate obtained from the Southwest Middlesex Municipal Design and Construction Standards 2021.

Table 10 - Design Criteria for Sewage Design Flow Calculation

DESIGN CRITERIA	RATE
Residential	275 L/persons/ha*
Non-Residential	22500 L/Ha/day
I/I	0.1 L/Ha/s

*Note: the wastewater design rate was found to be 350 L/Person/Day according to the SWM design guidelines; however, the design rate used in this study was reduced to 275 L/Person/Day assuming a safety factor of 25% applied on the guideline. This ratio also generate wastewater flows that are consistent with WWTP data received.

The total sewage flow for residential areas in each parcel was allocated to the closest manhole within the polygon, while the calculated sewage flow for non-residential areas was also assigned to the closest manhole based on its location. Flow is captured at Maintenance Holes (MH) and routed along sewers in the direction of lower elevations, i.e., gravity flow. Detailed calculation for the existing DWF for each parcel in 2021 planning horizon and planned wastewater flow in 2041 horizon was provided in Appendix E. Table 11 summarizes the existing wastewater flow calculation for the Village of Glencoe.

Table 11 - Summary of Calculated Sewage Generation Rate for the Village of Glencoe

	FLOW (L/S)	I/I (L/S) SEE ERROR! REFERENCE SOURCE NOT FOUND.	DESIGN FLOW = PEAK FLOW + I/I (L/S)	I/I RATE (L/S/HA)
DWF	7.30	0.47	7.78	0.0025
WWF	17.92	1.16	19.09	0.0061
Peak Flow	63.23	4.10	67.33	0.0214

Note:

- When determining the Dry Weather Flow (DWF), the sewage design rate of 350 L/Person/Day was reduced to 275 L/Person/Day assuming a safety factor of 25% applied on the design rate.
- The Harmon Peaking factor was calculated to be 3.52 by assuming that all of Glencoe was one catchment, and thus, the Harmon Peaking factor was determined using the "total area" of Glencoe.
- The WWF was determined by peaking the DWF by a factor of 2.45, which was determined by finding the % difference between DWF and WWF based on the Glencoe WWTP Annual Report.

The calculated wastewater flow was then compared to the Raw Flow recorded in the WWTP Annual Report from 2018 to 2020 provided by the SWM. Table 12 summarizes the annual average and maximum monthly wastewater raw flow into the Glencoe wastewater treatment plant from 2018 to 2020. Dry Weather Flow (DWF) was assumed to be equal to the average of the average monthly flow recorded at the WWTP, while the Wet Weather Flow (WWF) referred to the average of the maximum monthly flow to the WWTP, considering both the sanitary flow and infiltration. The Peak Flow of the study area was determined by multiplying the WWF and the Harmon Peaking Factor determined using the total area of Glencoe/Wardsville assuming it as one catchment.

Table 12 - Glencoe Wastewater Treatment Plant Raw Flow Summary

	AVERAGE OF MONTHLY AVERAGE RAW FLOW (M3/DAY)	AVERAGE OF MONTHLY AVERAGE RAW FLOW (L/S)	AVERAGE OF MONTHLY MAXIMUM RAW FLOW (M3/DAY)	AVERAGE OF MONTHLY MAXIMUM RAW FLOW (L/S)
2018	689.70	7.98	1634.08	18.91
2019	692.02	8.01	1918.25	22.20
2020	633.64	7.33	1395.33	16.15
Average	671.79	7.78	1649.22	19.09

The total I/I under DWF, WWF, and Peak Flow conditions was derived by comparing the recorded raw flow data from the WWTP and the estimated design flow calculated by WSP. The differences in flow would be the total I/I within the study area by assuming the calculated DWF was accurate. Then, by dividing the total I/I by the total sewer length in the area, the I/I rate was determined in L/s/km and assigned on all conduits in the model. Table 13 summarizes the total I/I in Glencoe under the three flow conditions respectively.

Table 13 - Summary of Calculated Infiltration and Inflow Rate for the Village of Glencoe

	AREA (HA)	TOTAL SEWER LENGTH IN AREA (M)	TOTAL I/I (L/S)	TOTAL I/I (L/S/KM)
DWF	191.94	17,155.61	0.47	0.028
WWF	191.94	17,155.61	1.16	0.068
Peak Flow	191.94	17,155.61	4.10	0.239

Note: The split between "base sewer flow" and "I/I" is based on assumptions. However, the total flow simulated is consistent with the total flows monitored and reported in the Wastewater Treatment Plant Annual Reports.

As shown in Table 13, the total I/I for the Village of Glencoe was converted to L/s/km and assigned on all the gravity sewers within the model.

Table 14 summarizes the existing flow calculation for the Village of Wardsville, while Table 15 summarizes the total I/I in Wardsville under three conditions respectively.

Table 14 - Summary of Calculated Sewage Generation Rate for the Village of Wardsville

	FLOW (L/S) (L/S)	I/I (L/S) SEE TABLE 16	DESIGN FLOW = PEAK FLOW + I/I (L/S)	I/I RATE (L/S/HA)	I/I % DIFFERENCE (COMPARED TO GLENCOE
DWF	1.00	0.11	1.11	0.0025	0.59
WWF	1.47	0.16	1.63	0.0037	N/A
Peak Flow	5.88	0.66	6.54	0.0147	N/A

Note:

- When determining the Dry Weather Flow (DWF), the sewage design rate of 350 L/Person/Day was reduced to 210 L/Person/Day (40% reduction).
- The Harmon Peaking factor was calculated to be 4 by assuming that all of Wardsville was one catchment, and thus, the Harmon Peaking factor was determined using the "total area" of Wardsville.
- The WWF was determined by peaking the DWF by a factor of 1.48, which was determined by finding the % difference between DWF and WWF based on the Wardsville WWTP Annual Report.

Table 14 shows the design rate used for the Wardsville sewage calculation. It was adjusted until the calculated I/I rate in L/s/Ha for Wardsville was equal or close to the Glencoe I/I rate under DWF condition as shown in Table 11. Table 15 summarizes the annual average and maximum monthly wastewater raw flow into the Wardsville wastewater treatment plant from 2018 to 2020.

Table 15 - Wardsville Wastewater Treatment Plant Raw Flow Summary

	AVERAGE OF MONTHLY AVERAGE RAW FLOW (M^3/DAY)	AVERAGE OF MONTHLY AVERAGE RAW FLOW (L/S)	AVERAGE OF MONTHLY MAXIMUM RAW FLOW (M^3/DAY)	AVERAGE OF MONTHLY MAXIMUM RAW FLOW (L/S)
2018	96.99	1.12	146.29	1.69
2019	97.77	1.13	147.28	1.70
2020	92.16	1.07	130.13	1.51
Average	95.64	1.11	141.24	1.63

The calculated design flow for Wardsville shown in Table 14 is consistent with the monitored data from the WWTP Annual Report summarized in Table 15. Similarly, the calculated wastewater flow was then compared to the Raw Flow recorded in the WWTP Annual Report from 2018 to 2020 provided by the SWM.

Table 16 - Summary of Calculated Infiltration and Inflow Rate for the Village of Wardsville

	AREA (HA)	TOTAL SEWER LENGTH IN AREA (M)	TOTAL I/I (L/S)	TOTAL I/I (L/S/KM)
DWF	44.86	5614.717	0.11	0.020
WWF	44.86	5614.717	0.16	0.029
Peak Flow	44.86	5614.717	0.66	0.117

Note: The split between "base sewer flow" and "I/I" is based on assumptions. However, the total flow simulated is consistent with the total flows monitored and reported in the Wastewater Treatment Plant Annual Reports

Similarly, the total I/I for in L/s the Village of Wardsville was converted to L/s/km and assigned on all the gravity sewers within the model as shown in Table 16.

6.2 WASTEWATER MODEL CRITERIA

The study area was assessed for sewage capacity, bottlenecks, and surcharge for the 2041 planning horizon in terms of Sewer Flow vs. Theoretical Sewer Capacity (q/Q) criteria and Depth of Flow vs. Sewer Diameter (d/D) under existing and post-intensification conditions.

Based on Manning's equation, the q/Q ratio is a commonly used indicator of the allocated or 'in-use' capacity of the sewers. This ratio is a number ranging between 0 and 1 (or 0-100%), calculated as the relationship between actual flow in the sewer to its maximum allowed flow capacity. A number close to 1 denotes a pipe flowing full, a non-ideal situation for a gravity sewer because sewers can transition to/from full-pipe flow in 'bottleneck' or pressurizes flow regimes that can limit conveyance capacity or cause sewage surcharges to ground.

Based on the benchmark for q/Q ratios from other Southern Ontario Cities and Municipalities, the sewer sections should be no more than 80% full; otherwise, it may be triggered for upsizing. In modeling the site, WSP noticed that when "backflow" was simulated in downstream sewers, the q/Q ratio did not always recognize this, and capacity would be underestimated. For this reason, WSP also calculated the d/D and considered this when sizing proposed sewers. A ratio of 80% full was also considered for the d/D. When calculating the d/D the actual depth of flow, accounting for backflow, was considered.

At each MH, the simulated Hydraulic Grade Line (HGL) is provided to illustrate flow conditions. These levels should be checked against the Municipality's standard in terms of the margin required between the HGL and basements that could be flooded if the Municipality's sewer surcharges – in this analysis WSP considered basement depths of 2m, any HGLs that were simulated within 2m of the ground elevation was deemed problematic and addressed. The unfilled MH depth represents the space available for sewage to rise to the Ground Level (GL) from the liquid level in the MH (GL-HGL), where zero means the manhole is full and will flood to the surface.

By contrast, the surcharge depth is the difference between the HGL and the top (or "crown") of the highest elevation connecting conduit. A positive surcharge depth means the node water surface elevation is above the highest pipe crown, while a negative depth means the node depth is below the highest pipe crown.

7 WASTEWATER ANALYSIS

Model simulations were completed for the Dry Weather Flow (DWF), Wet Weather Flow (WWF), and Wet Weather Peak Flow (WWF_Peak) for the 2021 and 2041 horizon to assess the conveyance capacity of the sewage system in the study area.

7.1 VILLAGE OF WARDSVILLE

WSP simulated the 2021 and 2041 planning horizon, including the calculated design flow determined in Section 5.1, to review the available carrying capacity (q/Q) of the existing pipes and unfilled depths of manholes.

Table 17 and Table 18 summarizes the simulated results for sewers and MHs respectively under DWF conditions in both 2021 and 2041 planning horizons for the Village of Wardsville. Complete tables of manhole and gravity main data for the simulated results are included in the appended material. In addition, map presenting the q/Q ratio at all pipes within the study area are provided in the Appendix F.

Table 17 - Pipe Summary Table for the Village of Wardsville under DWF Condition

PLANNING HORIZON	TOTAL FLOW (L/S)	VELOCITY (M/S)	D/D (%)	Q/Q (%)	# SEWERS WITH HIGH Q/Q
2021	0 – 1.55	0.04 - 0.86	0.4 – 70.6	0 – 84.5	1
2041	0 - 1.59	0.04 - 0.87	0.4 – 70.6	0 – 84.5	1

As shown in Table 17 and the q/Q Map from the Appendix F, most sewers in Wardsville were simulated with a q/Q ratio lower than 0.8 (80%), except for a small section along Run 'D' that was simulated at an approximate ratio of 0.845 (84.5%) full. Run "D" is identified in Figure 22, connecting the Kennedy SPS to the Main SPS from easements located east of Hagerty Rd. and south of Amy St. via the 75mm sanitary sewer.

Table 18 - MH Summary Table for the Village of Wardsville under DWF Condition

PLANNING HORIZON	TOTAL FLOW (L/S)	GRADE (M)	UNFILLED DEPTH (M)*	# OF MANHOLES WITH SURCHARGE
2021	0 – 1.55	193.55 – 211.01	0.53 – 12.7	0
2041	0 – 1.59	193.55 – 211.01	0.53 – 12.7	0

*Note: the unfilled depth of the manholes was calculated by subtracting the total depth manhole by the depth of water simulated in the model.

The unfilled depth of the manhole is governed by the invert of downstream manhole and sewer, the model indicated that no manhole surcharged with unfilled depth ranged between 0.53m and 12.7m.

Table 19 compares the simulated flow to the Main SPS to the recorded WWTP raw flows from the Wardsville WWP Report. Based on the simulation, the station inflow was found to be higher than the WWTP recorded flow because the steady state model did not reflect the retention time at the individual septic tanks; however, the model was deemed to be conservative.

Table 19 - Simulated Station Inflow for the Village of Wardsville

	SIMULATED INFLOW UNDER 2021 DWF (L/S)	AVERAGE OF MONTHLY AVERAGE RAW FLOW (L/S)	SIMULATED INFLOW UNDER 2021 WWF (L/S)	AVERAGE OF MONTHLY MAXIMUM RAW FLOW (L/S)
Average	2.72	1.11	3.17	1.63

Note: the simulated inflow to the Wardsville WWTP in the model was conservative comparing to the WWTP flows recorded in the Wardsville WWTP report because the retention time on individual septic tanks were not considered in a steady-state model.

Figure 22 shows the profile of the Wardsville SBS system along Run 'D' under 2031 DWF. From Figure 22, the hydraulic profile shows that the existing 75mm sewer along Run 'D' was simulated with a q/Q ratio closed to 80% in 2041 DWF condition.

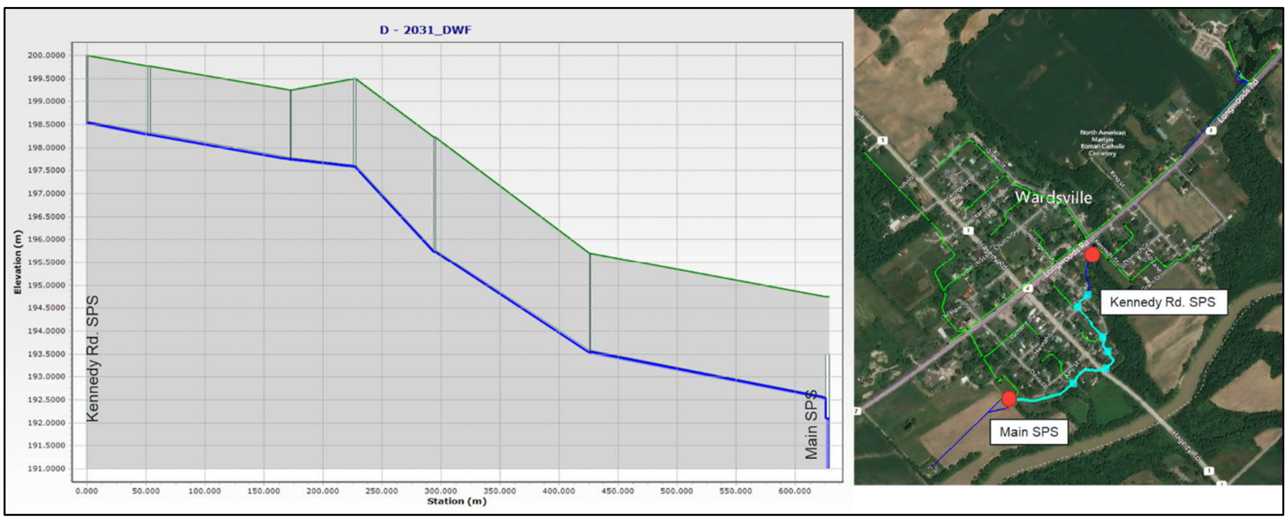


Figure 22 – HGL Profile for Run 'D' in Wardsville under 2041 DWF

(Note: Sewers highlighted in blue represent Run 'D' of the Wardsville SBS system)

Similarly, Table 20 and Table 21 summarizes the simulated results for Pipe and MH respectively under WWF_Peak in both 2021 and 2041 planning horizon for the Village of Wardsville. Complete tables of manhole and gravity main data for the simulated results are included in the appended material. In addition, map presenting the q/Q ratio at all pipes within the study area is provided in the Appendix F.

As previously discussed in Section 5, a steady-state model cannot fully represent the operation of the Wardsville SBS network, especially under WWF_Peak condition when a Harmon Peaking factor of 4 was applied to the wastewater loadings in the model; therefore, the following results are presented for consideration understanding that they represent a conservative simulation method.

Table 20 - Pipe Summary Table for the Village of Wardsville under 2041 WWF_Peak Condition

PLANNING HORIZON	TOTAL FLOW (L/S)	VELOCITY (M/S)	D/D (%)	Q/Q (%)	# SEWERS WITH HIGH Q/Q
2021	0.01 - 4.88	0.08 – 1.03	1.0 - 74.5	0 – 148.9	5
2041	0.01 – 5.05	0.08 – 1.04	1.0 - 74.5	0 – 148.9	5

As shown in Table 20 and the maps from the Appendix B, a total of five (5) sewers was simulated with q/Q ratio greater than 80%, and three of them surcharged with an q/Q ratio greater than 100%. As mentioned in Section 5, the steady state sewer model for Wardsville cannot capture the retention time at each individual septic tank, and hence, sewage flow was loaded on manhole, meaning sanitary flow travelled immediately downstream and was transferred to the Wardsville SBS collection system all at once, resulting in high q/Q ratio. To determine the performance of the Wardsville SBS system more accurately, an EPS model is recommended.

Table 21 - MH Summary Table for the Village of Wardsville under 2041 WWF_Peak Condition

PLANNING HORIZON	TOTAL FLOW (L/S)	GRADE (M)	UNFILLED DEPTH (M)*	# OF MANHOLES WITH SURCHARGE
2021	0 – 4.86	194.67 – 211.02	0.52 – 12.7	3
2041	0 – 5.04	194.67 – 211.02	0.52 – 12.7	3

*Note: the unfilled depth of the manholes was calculated by subtracting the total depth manhole by the depth of water simulated in the model.

The unfilled depth of the manhole is governed by the invert of downstream manhole and sewer, the model indicated that no manhole surcharged with unfilled depth ranged between 0.53m and 12.7m, and three manholes on Run 'D' were simulated to surcharge.

Figure 23 shows the simulated q/Q ratio on the collection sewers in the Village of Wardsville under 2041 WWF_Peak, and sewers with q/Q ratio greater than 80% were found along Run 'D' and highlighted in orange. A total of three manholes was simulated with surcharge along the highlighted sewers, meaning the manholes were filling up and acting as a temporary storage before the wastewater flow was conveyed to the sanitary pump station.

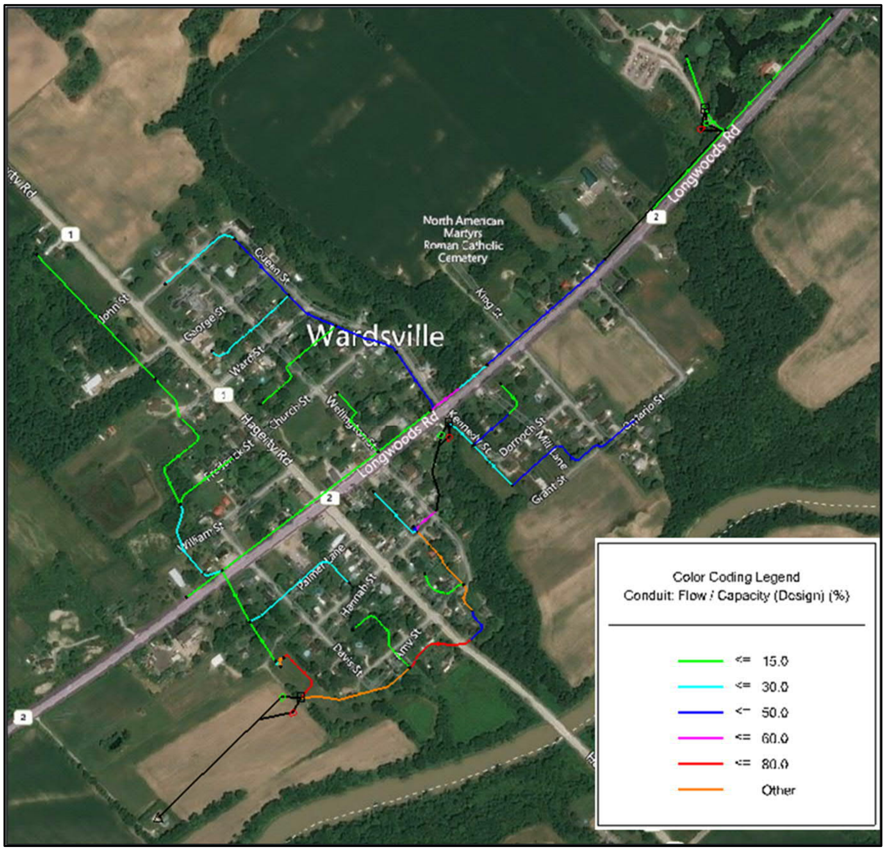


Figure 23 – Simulated q/Q for all Conduits in Wardsville under 2041 WWF_Peak

7.2 VILLAGE OF GLENCOE

WSP simulated the 2021 planning horizon, including the calculated design flow determined in Section 5.1, to review the available carrying capacity (q/Q) of the existing pipes and unfilled depths of manholes.

Table 22 and Table 23 summarizes the simulated results for Pipe and MH respectively under both 2021 and 2041 planning horizon. Complete tables of manhole and gravity main data for the simulated results are included in the appended material. In addition, map presenting the q/Q ratio at all pipes within the study area is provided in the Appendix F.

Table 22 - Pipe Summary Table for WWF_Peak Condition for the Village of Glencoe

PLANNING HORIZON	TOTAL FLOW (L/S)	VELOCITY (M/S)	D/D (%)	Q/Q (%)	# SEWERS WITH HIGH Q/Q
2021	0.01 - 64.67	0.08 - 0.86	1.0 -52.70	0 – 54.7	0
2041	0.01 – 85.57	0.08 - 0.88	1.0 – 67.2	0 – 79.2	0

Table 23 - MH Summary Table for WWF_Peak Condition for the Village of Glencoe

PLANNING HORIZON	TOTAL FLOW (L/S)	HYDRAULIC GRADE (M)	UNFILLED DEPTH (M)*	# OF MANHOLES WITH SURCHARGE
2021	0 – 64.66	215.50 – 220.43	0.58 – 6.14	0
2041	0 – 85.56	215.53 – 220.43	0.58 – 6.14	0

*Note: the unfilled depth of the manholes was calculated by subtracting the total depth manhole by the depth of water simulated in the model.

The unfilled depth of the manhole is governed by the invert of downstream manhole and sewer, the model indicated that no manholes are surcharged to ground; unfilled depth ranged between 0.58m and 6.15m.

As shown in the q/Q Map from the Appendix F, it can be seen all the sewer within Glencoe was simulated with a q/Q ratio lower than 0.8 under WWF_Peak for both planning horizons.

Table 24 compares the simulated flow to the Victoria SPS to the recorded WWTP raw flows from the Glencoe WWP Report. Based on the simulation, the station inflow was similar to the WWTP recorded flow, and the model was deemed to be representative of the actual flow into the Glencoe WWTP.

Table 24 - Simulated Station Inflow for the Village of Glencoe

	SIMULATED INFLOW UNDER 2021 DWF (L/S)	AVERAGE OF MONTHLY AVERAGE RAW FLOW (L/S)	SIMULATED INFLOW UNDER 2021 WWF (L/S)	AVERAGE OF MONTHLY MAXIMUM RAW FLOW (L/S)
Average	7.80	7.78	19.17	19.09

Figure 24 shows the profile of the major sewer running along Victoria St. between Parkhouse Dr. and Appin Rd. under WWF_Peak condition for 2041 planning horizon, which conveys sewage flow to the Victoria Sanitary Pump Station at the Prince William St.

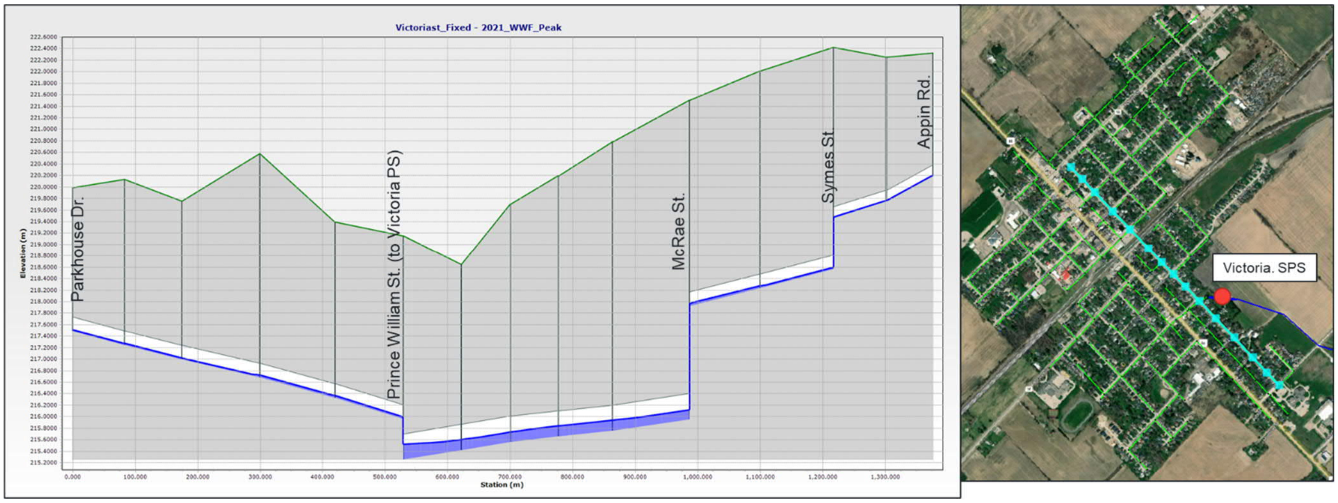


Figure 24 – HGL Profile for the Victoria St. Sewer under 2041 WWF_Peak

(Note: Profiles for Sewers highlighted in blue are shown)

Figure 25 shows the profile of the major sewer running along Roe St. from the easement north of Ewen Ave. to the Victoria Sanitary Pump Station. Figure 26 shows the profile of the major sewer running along Elizabeth St. and McRae which eventually connects to the Victoria St. sewer.

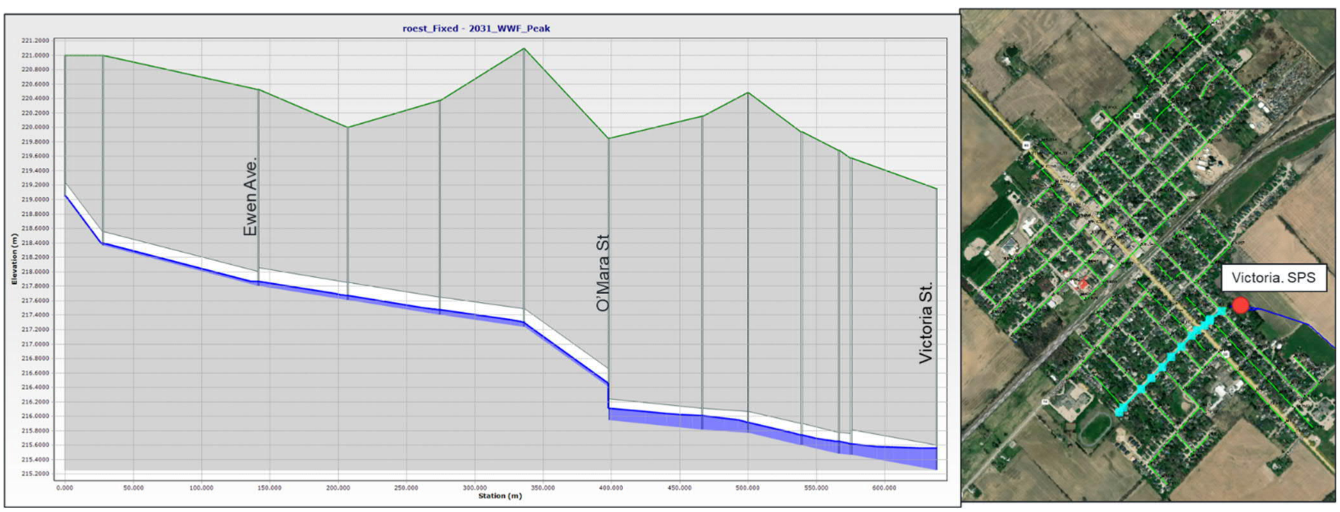


Figure 25 – HGL Profile for the Roe St. Sewer under 2041 WWF_Peak

(Note: Profiles for Sewers highlighted in blue are shown)

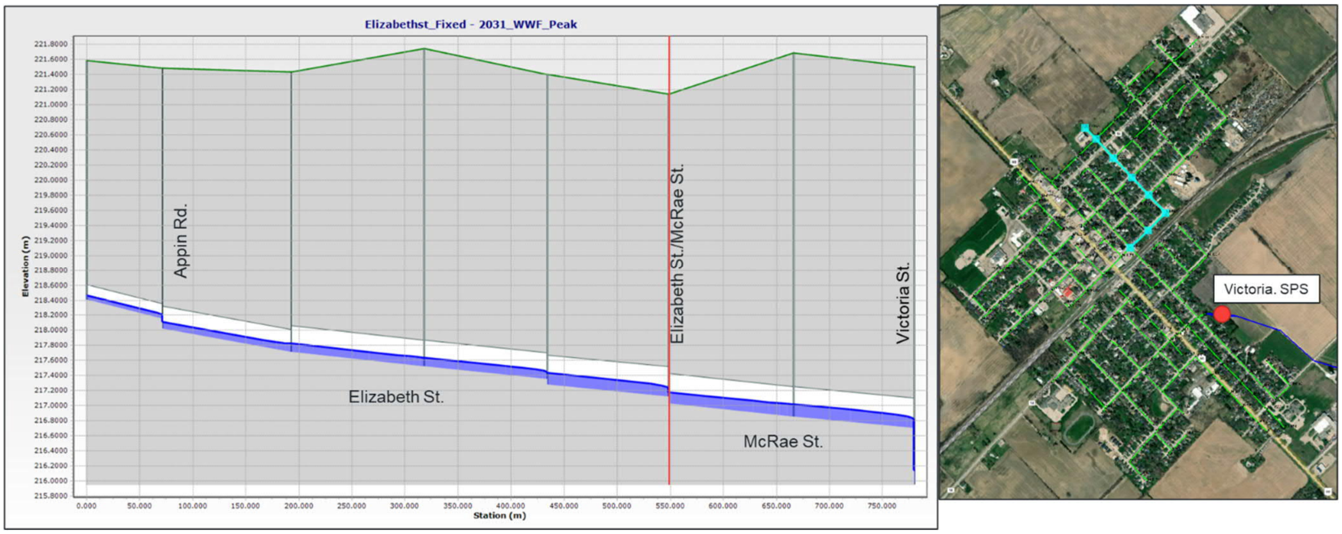


Figure 26 – HGL Profile for the Elizabeth St. and McRae St. Sewer under 2041 WWF_Peak

(Note: Profiles for Sewers highlighted in blue are shown)

From the above figures, it can be shown that the existing sewers within Glencoe can maintain a q/Q ratio below 80% and provide sufficient capacity to support future growth of the Village even under 2041 WWF_Peak condition.

7.2.1 VICTORIA SANITARY PUMP STATION CAPACITY ANALYSIS

The Victoria Sanitary Pump Station (SPS) is located at the southeast corner of William St. and Victoria St. It consists of two pumps each rated at 3182 m3/d with a design head of 26.5 m.

WSP simulated the WWF Peak condition for the 2021 planning horizon, and the simulated inflow to the Victoria SPS was found to be 64.5 L/s, while the simulated outflow from the Station was found to be 47.4 L/s. The existing wet well at the Station was determined to have a total volume of 24m^3 which cannot accommodate all the excess wastewater, and therefore, the wet well is expected to overflow during the WWF Peak condition. The required station outflow to prevent the wet well from overflowing was determined to be approximately 183.4 L/s; however, the existing 200mm forcemain would not have enough capacity to accommodate an outflow of 183.4 L/s. Figure 27 presents the plot summarizing the simulated inflow and outflow and the wet well volume from the Victoria SPS under 2021 WWF Peak condition.

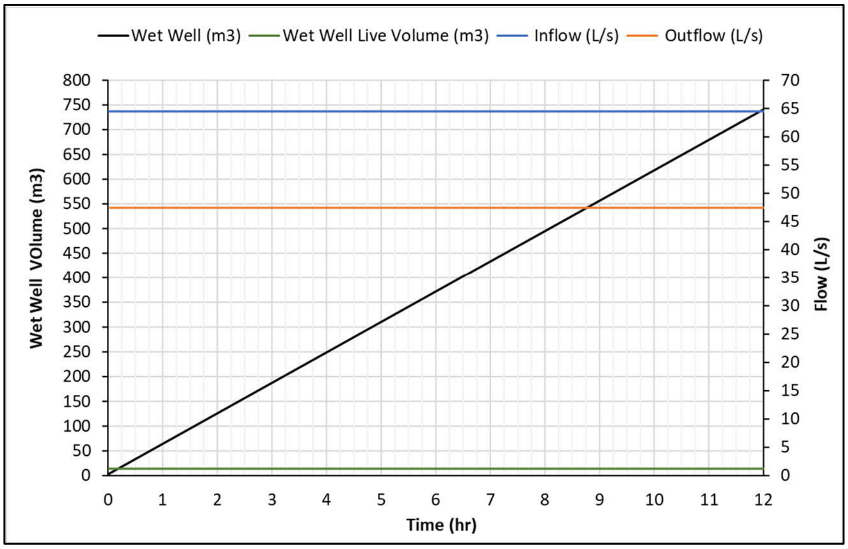


Figure 27 – Simulated Inflow and Outflow from the Victoria SPS under 2021 WWF_Peak

To analyze the performance of the existing pumps at the station, WSP re-ran the 2021 WWF_Peak Scenario by upsizing the existing forcemain to 500mm and generated operating pump curves at the Victoria SPS. Figure 28 presents the simulated pump curve at the SPS under 2021 WWF_Peak condition with two pumps ON. From Figure 28, the pumps were operating at about 22% efficiency during the simulation and provided an overall outflow of 130.7 L/s with two pumps were ON. It can be seen that even with the 500mm forcemain, the existing pumps cannot accommodate the required outflow of 183.4 L/s, and they should be upgraded to provide more flow.

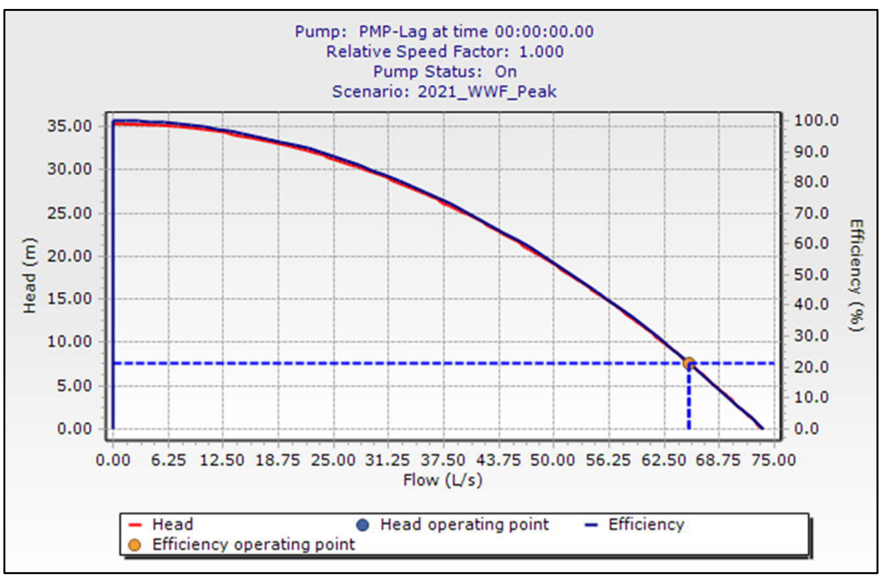


Figure 28 – Simulated Pump Curve at the Victoria SPS under 2021 WWF_Peak Condition

However, it was understood that the existing pumps in the Victoria SPS were installed not long ago, and sewage continued to surcharge to ground under severe rainfall event. Therefore, upgrading the existing pumps was not the most ideal option and WSP considered additional alternatives to help alleviate surcharging.

7.2.2 TWINNED FORCEMAIN

The existing 200mm forcemain does not have enough capacity to accommodate the flow of 183.4 l/s (i.e., which is required to prevent overflow at the wet well). Therefore, in addition to the pump upgrades, the forcemain should be upsized as well. This can be accomplished by installing a single larger size forcemain and decommissioning the existing forcemain. Another option would be twinning the existing forcemain, connecting the Victoria SPS to the Glencoe WWTP. This would also increase the flow capacity of the forcemain while keeping the existing 200mm forcemain and would accommodate an outflow of 183 L/s based on the WSP calculation.

WSP completed a desktop study to determine the appropriate sizing for the proposed twin forcemain, and the results of this analysis are summarized in Table 25, highlighting the different size combinations of the twin forcemain. As shown in Table 25, if the existing forcemain is twinned with another 200mm pipe, the velocity on both forcemains nearly achieve the maximum allowable velocity of 3m/s with the required flow of 183 L/s. Similarly, the maximum twinned forcemain size was determined to be 450mm, and in this case, the velocity within the existing 200mm forcemain was calculated to be 0.68 m/s, which was slightly higher than the required scouring velocity of 0.6 m/s.

Although the size of the twinned forcemain can range from 200mm to 450mm, the twin forcemain smaller than 400mm is not recommended. As shown in Table 25, the velocity in the smaller size twin forcemain (200mm to 350mm) ranges from 2.92m/s to 1.53m/s. These high velocities correspond to large friction headloss along the forcemain. To accommodate the large headloss associated with the friction losses in the twinned forcemain, a larger pump should be selected. Table 26 shows the design flow and head for the selected pump assuming twinned forcemain sizes of 200mm to 450mm. As shown, the smaller the twinned forcemain is the larger the required design head (i.e., the required pump size) will be. If the City prefers a twinned forcemain, WSP recommends a 400mm forcemain to be twinned with the existing forcemain. As shown in Table 25, if a 400mm forcemain is selected, the amount of flow passing through the 200mm existing forcemain is only 15% of the total flow. All the flow can be accommodated by a single 400mm or 500mm forcemain with velocities of 1.46 m/s and 0.93 m/s, respectively. Using a single forcemain as opposed to twinning with the existing forcemain, eliminates the significant maintenance costs of one forcemain (i.e., the 200mm existing). To be more cost effective in terms of maintenance costs, WSP recommends using a single 400mm or 500mm forcemain and decommissioning the existing forcemain.

Table 25 - Summary of Flow Capacity with Different Forcemain Combination

Twin Forcemain	Forcemain Dia. (in)	Forcemain Dia. (mm)	Forcemain Length (m)	Area (m^2)	$(D2/D1)^{5/2}$	Flow in each FM (L/s)	Flow (m^3/s)	Velocity (m/s)
1st	8	200	1443.75	0.031	1.00	92	0.09	2.92
2nd	8	200	1443.75	0.031		92	0.09	2.92
Total Maximum Outflow from the Wet Well (L/s)						183		
1st	8	200	1443.75	0.031	1.75	67	0.07	2.13
2nd	10	250	1443.75	0.049		117	0.12	2.38
Total Maximum Outflow from the Wet Well (L/s)						183		
1st	8	200	1443.75	0.031	2.76	49	0.05	1.55
2nd	12	300	1443.75	0.071		135	0.13	1.90
Total Maximum Outflow from the Wet Well (L/s)						183		
1st	8	200	1443.75	0.031	4.05	36	0.04	1.16
2nd	14	350	1443.75	0.096		147	0.15	1.53
Total Maximum Outflow from the Wet Well (L/s)						183		
1st	8	200	1443.75	0.031	5.66	28	0.03	0.88
2nd	16	400	1443.75	0.126		156	0.16	1.24
Total Maximum Outflow from the Wet Well (L/s)						183		
1st	8	200	1443.75	0.031	7.59	21	0.02	0.68
2nd	18	450	1443.75	0.159		162	0.16	1.02
Total Maximum Outflow from the 6m Wet Well (L/s)						183		

Table 26 - Required Pump Size (Design Flow and Head) with Different sizes of Twinned Forcemain

	Twinned FM Size											
	200mm-200mm		200mm-250mm		200mm-300mm		200mm-350mm		200mm-400mm		200mm-450mm	
	Flow (L/s)	Head (m)	Flow (L/s)	Head (m)	Flow (L/s)	Head (m)	Flow (L/s)	Head (m)	Flow (L/s)	Head (m)	Flow (L/s)	Head (m)
Shutoff	0	132.8	0	75.24	0	44.6	0	28.71	0	20.36	0	15.87
Design	92	99.6	92	56.43	92	33.45	92	21.53	92	15.27	92	11.9
Max Operating	184	0	184	0	184	0	184	0	184	0	184	0

7.2.3 UPGRADED PUMP CYCLES

The pump cycles vary with wet well size. Larger wet wells can maintain more sewage, and therefore, they decrease the number of pump cycles. The existing wet well in the Victoria SPS has two cells with a total volume of 24m³, and with the wet well size to be maintained, the pump cycle was determined to be 10 cycles/hr. Figure 29 shows the pump cycle in one hour period with the 24m³ wet well Size.

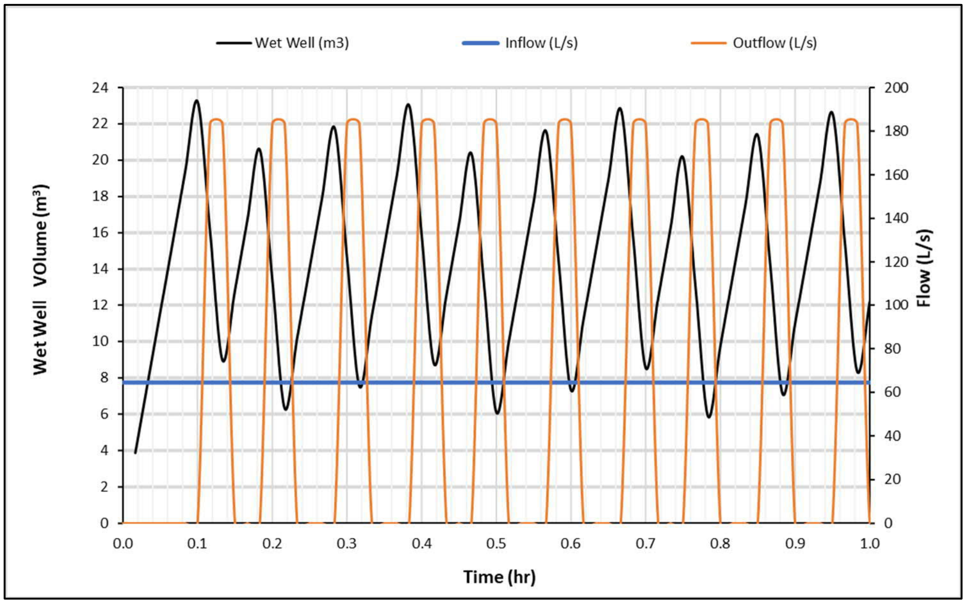


Figure 29 – Pump Cycle with 24m³ Wet Well Size

Under 2021 WWF_Peak condition, the pump cycles can be reduced to 4 cycles/hr if the wet well is upsized to 56m³. Under 2041 WWF_Peak Condition, the wet well was further upsized to 105m³, and the pump cycle was reduced to 2 cycle/hr in future condition. Figure 30 and Figure 31 presents the pump cycles in one hour with an upsized wet well of 56m³ and 105m³ respectively.

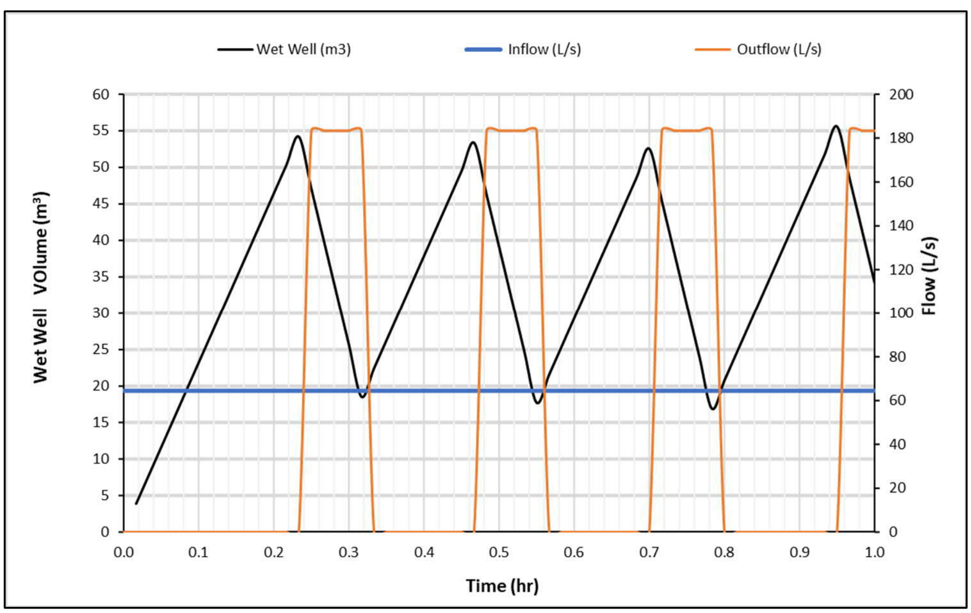


Figure 30 – Pump Cycle with 56m^3 Wet Well Size

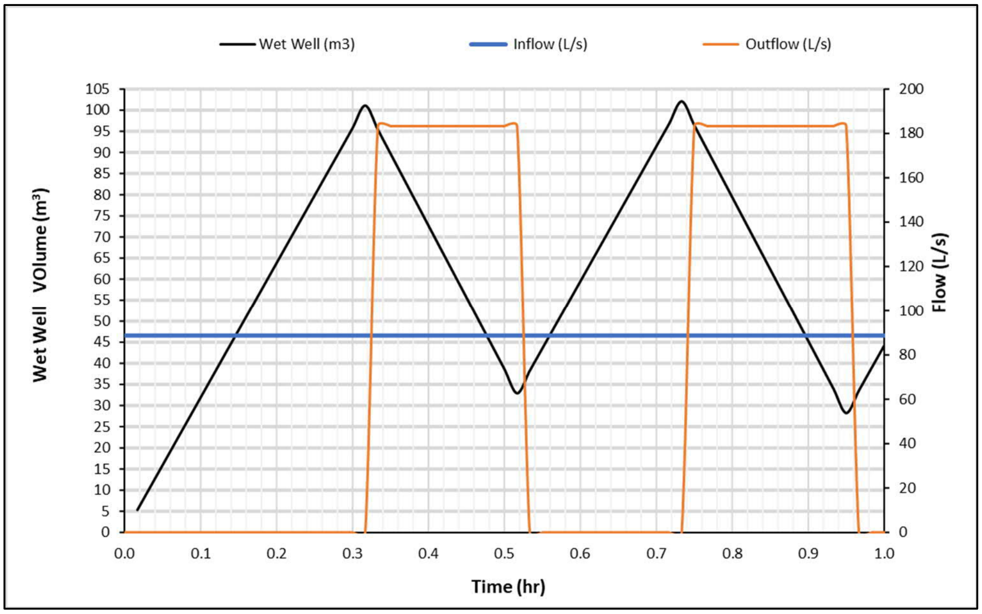


Figure 31 – Pump Cycle with 105m^3 Wet Well Size

7.2.4 VICTORIA SANITARY PUMP STATION UPGRADE COST ESTIMATE

WSP considered two alternatives for performing cost estimate. The first alternative is to upgrade the pumps only and the second alternative is to upgrade the wet well and the pump station including all the electrical equipment.

UPGRADING THE PUMPS

If the wet well has enough space to install upgraded pumps and it is not to be upsized and all the electrical equipment are sufficient for the new pumps, the required expense for upgrading the pump station is the cost of the pumps, the cost of bypassing the pump station during the pump upgrade, and the cost of upsizing the forcemain plus a contingency allowance.

If a 400mm forcemain is to be twinned with the existing 200mm forcemain, a pump with a design flow and head of 92 L/s and 15.27m is required (Table 26). The cost of the pump and 1500m-forcemain upgrades and bypassing as well as allowing 35% of contingency are shown in Table 27.

Table 27 - Cost Estimate for Upgrading Pumps in Victoria Sanitary Pump Station

	PS Details		Upgraded Forcemain	Cost (M$)					
	No of Pumps	Firm Capacity (l/s)	Diameter (mm)	Forcemain	Bypass Duration (Month)	Bypass	PS Upgrades	Contingency	Total Cost
Existing Wet well	2	183	400	2.7	12	1.875	1.725	1.26	7.56

UPGRADING THE PUMP STATION

If the wet well does not have sufficient space to accommodate the two upgraded pumps, all the pump station including the wet well, pumps, and electrical equipment should be upsized. As a rule of thumb, a new pump station with two pumps having a design flow and head of 92l/s and 15.27m would cost 13 M$.

8 CONCLUSIONS & RECOMMENDATIONS - SEWER NETWORK

In the Village of Glencoe, the existing sewers were simulated to have enough capacity to convey the wastewater loads downstream with no surcharge to ground under both existing and future condition. All of the Glencoe flow was conveyed to the Victoria SPS and eventually directed to the Glencoe WWTP via a 200mm forcemain. With the existing infrastructures, the Victoria SPS cannot accommodate the required flow under WWF_Peak condition. Therefore, WSP provided the following recommendations for the Municipality to consider:

1. Adding a twinned forcemain in addition to the existing 200mm forcemain to accommodate more outflow from the SPS. In this study WSP investigated decommissioning the existing forcemain to build a large diameter forcemain in its place and twinning the existing forcemain with a larger one. To be more cost effective in terms of maintenance costs, WSP recommends using a single 400mm or 500mm forcemain and decommissioning the existing forcemain. However, prior to designing and implementing a solution, WSP recommends completing an Environmental Assessment that focuses on this station.
2. Upsizing the existing wet well to accommodate future flow and reduce pump cycle. Upsizing the wet well from $24m^3$ to $56m^3$ could help reduce the pump cycle from 10 to 4 times per hour. The feasibility of upsizing the wet well should be studied in the same Environmental Assessment as was suggested for the forcemain upsizing/twinning.

In the Village of Wardsville, the existing SBS collection system was simulated to have sufficient capacity to convey sewage to the Main Pump Sanitary Pump Station without surcharge under DWF for both planning horizons. When simulating WWF_Peak condition, a few sewers along Run 'D' surcharged with q/Q ratio greater than 100% based on the simulation. The steady state model for the Wardsville SBS network is simplified and does not capture the retention time in each individual septic tank before it discharges to the collection system. Sanitary loads on MHs flowed immediately downstream in the model, resulting in high q/Q ratio. To represent the performance of the Wardsville SBS system more accurately, an EPS model is required.

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

9 RECOMMENDED PROJECTS AND COSTS

Table 28 and Table 29 summarize the recommended water and wastewater capital projects from WSP as well as the associated Class D cost estimates and level of urgency for the 2041 planning horizon.

Table 28 - WSP Water Project Recommendations and Class D Cost Estimates

PROJECT NO.	PROJECT DESCRIPTION	COST ESTIMATE	LEVEL OF URGENCY (1 – MOST IMPORTANT)	REASON
SWM_W1	In-Line Booster Pump Station for Appin	$ 2,500,000.00	1	Low fire flow availability is a significant limitation to development in Appin. An inline booster station would create an Appin Pressure Zone and would help increase the fire flow across the Appin network
SWM_W2	Adding a 200mm watermain in Appin on the easement west of Thames Rd., approximately 1000m in length.	$ 1,100,000.00	2	Create resiliency to the Appin water network and provide a second supply point in the system to supplement fire flow
SWM_W3	Adding a 250mm watermain (approximately 350m in length) on Main St. between Industrial Rd. and Parkhouse Dr., serving as a second connection to the Village of Glencoe.	$ 400,000.00	3	Increase supply resiliency into Glencoe and reduce the overall headloss along the Main St. main to help lower operational costs. Two supplies into the Glencoe system

Note:

- Estimates for linear infrastructures were completed based on historical unit rates from the Basement Flooding Protection Program in Toronto. Costs will vary for other municipalities.
- For Project SWM_W1, the cost estimate includes the construction cost for the Station. The design cost and the linear components are not included in the estimate.

A feasibility study/EA should be completed to understand the full impact of the In-Line Booster Pump Station for Appin. In this study, WSP provides preliminary sizing for the booster pumps in this station. A detailed study should be completed to assess sizing options and what impact this could have on the suction side of the station.

Table 29 - WSP Wastewater Project Recommendations and Class D Cost Estimates

PROJECT NO.	PROJECT DESCRIPTION	COST ESTIMATE	LEVEL OF URGENCY (1 – MOST IMPORTANT)	REASON
SWM_WW1	Adding a second forcemain approximately 1450m in length.	$ 2,000,000.00 - $ 2,500,000.00 (Depending on sizes)	1	Increase the outflow capacity of the Glencoe sewage pump station to alleviate surcharging during high flow events.
SWM_WW2	Victoria SPS upgrade, including wet well and pump upgrades, to accommodate future growth	$ 5,000,000.00	2	Increase the outflow capacity of the Glencoe sewage pump station to alleviate surcharging during high flow events.

For best cost-effectiveness, detailed EA studies should be completed before the final designs to ascertain the optimum solutions based on cost and hydraulic effectiveness. The total Class D capital cost estimate for water project is approximately $4M including contingency, and the total Class D capital cost estimate for wastewater project is approximately $7.5M including contingency; however, they are subject to change before detailed EA studies are carried out.