TECHNICAL MEMORANDUM

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From:	**Sam Ziemann, P.Eng** c. 519-404-4529	Project Ref. #:	**75-41-191370**
		Date:	**January 3, 2023**
Subject:	**TM3A Existing & Future Population, Employment & Land Use, and Servicing Implications**		

City of Guelph

Water and Wastewater Servicing Master Plan

TM3A Existing & Future Population, Employment & Land Use, and Servicing Implications

C3 WATER INC. / Stantec Consulting

January 3, 2023

VERSION	DATE	DESCRIPTION OF REVISIONS	REVISED BY	REVIEWED BY
1	May 17, 2022	Draft #1	Water: Michelle Scott Wastewater: Marty Anderson	Sam Ziemann, Marc Telmosse, Colleen Gammie, Scott Cousins, Adam Geldart
2	September 15, 2022	Draft # 2	Water: Michelle Scott Wastewater: Marty Anderson	Sam Ziemann, Marc Telmosse, Colleen Gammie, Scott Cousins, Adam Geldart, Emily Stahl
3	January 3, 2023	Final	Water: Michelle Scott Wastewater: Marty Anderson	Water: Sam Ziemann Wastewater: Mac Telmosse

SIGN OFF

This document, entitled **"TM3A Existing & Future Population, Employment & Land Use, and Servicing Implications"**, was prepared by C3 Water Inc. (C3W) and Stantec Consulting Ltd. (Stantec) for the **City of Guelph**.

C3W and Stantec certify that the information contained in this report is accurate, complete and in accordance to the terms of our engagement. This assessment is based, in part, on information provided by others. Unless specifically noted, C3W and Stantec have assumed that this information is correct, and has relied on it in the development of conclusions.

The material herein reflects C3W and Stantec's best judgement based upon the information available at the time of preparation. Any use which a third party makes of this report or any reliance on or decisions made based on it, are the responsibilities of such third parties. C3W and Stantec accept no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based upon this report.

DATE: January 3, 2023

Reviewed by: **Sam Ziemann, P.Eng, President**

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1.0 INTRODUCTION AND BACKGROUND

C3 Water Inc (C3W) and Stantec Consulting (Stantec) where retained by the City of Guelph (City) to complete the Water and Wastewater Servicing Master Plan (WWSMP). The purpose of this TM is to assess the existing water and wastewater systems under existing and future demand conditions and identify any constraints.

1.1 Existing Water System

The Guelph water distribution system consists of approximately 600 km of watermains throughout three pressure zones. The primary water source is the Arkell wells and the Glen Collector system which feed into the F.M. Woods Water Treatment Plant (Woods WTP) via the Arkell Aqueduct. The Woods WTP and pump station (PS) supplies approximately 60-80% of the City's drinking water into Zone 1. There are also a number of groundwater supply wells throughout the City. The Paisley, Robertson and Clythe PSs boost water from Zone 1 into Zone 2. The Clair PS boosts water from Zone 1 into Zone 3. The system has three elevated tanks (ETs), Verney and Clair ET located in Zone 1 and the Speedvale ET in Zone 2. There are four (4) in-ground storage reservoirs, Woods and University in Zone 1 and Paisley and Clythe in Zone 2. An overview of the existing water distribution system is presented in Figure 1-1 below.

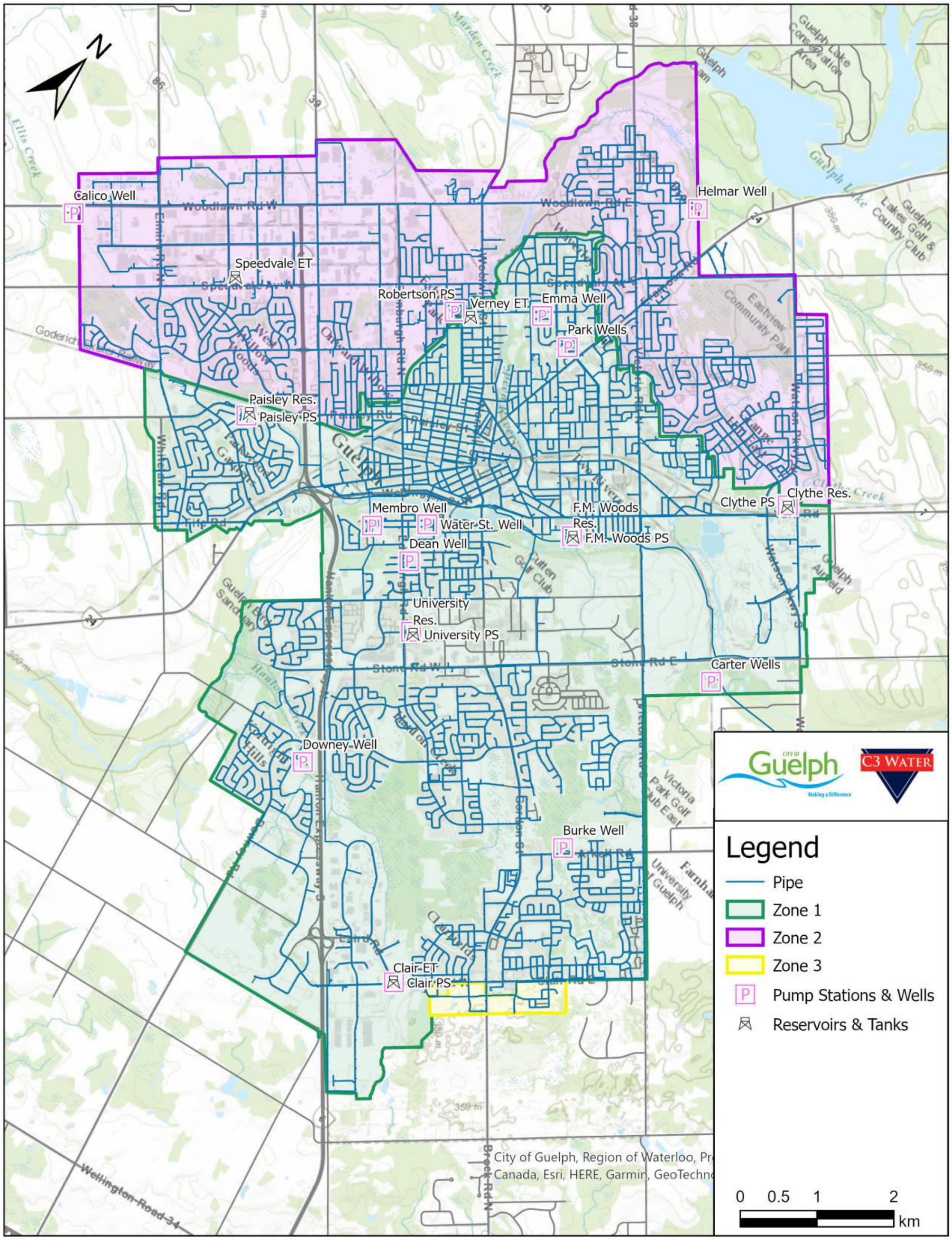


Figure 1-1 Existing Water System Overview

1.1.1 Water Demand

Existing average day demand (ADD) and maximum day demand (MDD) were estimated using 2019 billing meter records and production data and are summarized in Table 1-1 below. This is discussed further in the *Water and Wastewater Model Update, Field Testing and Calibration TM.* It should be noted that the 2021 demands in 2021 WSMP were based on a projected 2021 population, and therefore, vary slightly from the existing demands used in this WWSMP.

Table 1-1 Existing Demands Summary (L/s)

Scenario	Total Demand	Billed Consumed	NRW
ADD 2019	544	452	92
MDD 2019	729	637	92

1.1.2 Storage

The existing available storage is summarized in Table 1-2 below. Overall, the system has a floating storage volume through ETs of approximately 11 ML and a total storage volume of 57 ML.

Table 1-2 Existing Storage Summary

Facility	Type	Volume (ML)
Zone 1/3		
FM Woods Reservoir	Pumped	29
Verney Elevated Tank	Floating	4
Clair Elevated Tank	Floating	5
University Reservoir	Pumped	2
Zone 2		
Speedvale Elevated Tank	Floating	2
Clythe Reservoir	Pumped	1
Paisley Reservoir	Pumped	13
Summary - Floating Storage		
Zone 1 & 3 Floating		9
Zone 2 Floating		2
Total Available Floating Storage		**11**
Summary - Total Storage		
Zone 1 & 3 Total		41
Zone 2 Total		16
Total Available Storage		**57**

1.1.3 Supply

The existing available supply is summarized in Table 1-3 below. The existing capacity of each source was based on Table 4-2 within the Water Supply Master Plan completed by AECOM Canada Ltd. (AECOM) in 2021 (2021 WSMP). The largest existing source is the Arkell Wellfield which supplies the Woods WTP via the Arkell Aqueduct along with the Glen Collector and the Carter Wells. In total, the system has an available supply capacity of 903 L/s, 878 L/s of which is located in Zone 1.

Table 1-3 Existing Supply Summary (2021 WSMP Table 4-2)

Facility	Capacity (m3/day)	Capacity (L/s)
Zone 1		
Arkell Well 1	2,000	23
Arkell Wells 6, 7, 8, 14, 15	28,800	333
Glen Collector	5,100	59
Carter Wells (1, 2)	5,184	60
Burke Well	6,500	75
Dean Well	1,500	17
Downey Well	5,237	61
Emma Well	2,800	32
Membro Well	5,200	60
Park Wells (1, 2)	8,000	93
Queensdale Well	1,100	13
University Well	2,500	29
Water Well	1,901	22
Zone 2		
Calico Well	1,400	16
Helmar Well	800	9
Paisley Well	1,400	16
Summary		
Zone 1	75,822	878
Zone 2	3,582	41
Total System	**79,422**	**919**

1.1.4 Pumping

The existing system pump stations are summarized in Table 1-4 below. The pump information was sourced from the City's 2020 Drinking Water Works Permit (2020 DWWP). The firm capacity was calculated as the total capacity minus the rated flow of the largest pump at each facility. At well pump stations, the firm capacity was based on the supply capacity values from the 2021 WSMP.

Table 1-4 Existing Pump Summary

Facility	Existing Pumps				Firm Capacity (L/s)
	# of Pumps	Rated Flow (L/s)	Rated Head (m)	Total Capacity (L/s)	
Zone 1					
F.M. Woods Pump Station	2	284	70	1325	1050
	1	347	81		
	1	259	70		
	1	151	85		
Burke Pump Station*	2	76	58	152	75
Dean Pump Station*	1	20	64	20	17
Downey Pump Station*	1	61	70	61	61
Emma Well*	1	33	99	33	32
Membro Pump Station*	1	76	71	76	60
Park Pump Station	2	70	54	140	70
Queensdale Pump Station	1	30	66	30	13
University Pump Station	1	27	52	76	27
	1	49	51		
Water St Well*	1	30	145	30	22
Zone 2					
Calico Pump Station*	1	61	67	61	0
Helmar Pump Station*	1	38	53	38	9
Paisley - Vertical Turbine Pump Station	3	53	82	287	212
	1	75	82		
	1	53	62		
Paisley - Horizontal In-Line Pump Station	2	53	37	182	106
	1	76	37		
Robertson Pump Station	2	44	24	167	88
	1	79	23		
Clythe Pump Station	3	63	76	189	126
Zone 3					
Clair Pump Station	2	35	35	545	470
	1	75	35		
	2	200	35		
Summary					
Zone 1					1,428
Zone 2					541
Zone 3					470

Existing pump data sourced from 2020 DWWP
**Firm Capacity based on Well Supply Capacity (2021 WSMP)*

1.2 Existing Wastewater System

The City's sanitary sewer system is primarily gravity-based. There are approximately 520 km of gravity sanitary sewers within the study area, with pipe diameters ranging from 200 mm to 1650 mm. Over 85% of the sanitary system has pipe diameters of 375 mm or less. The sanitary sewer system eventually discharges into the Guelph Water Resource Recovery Center (WRRC) located in the central west-end of the City on the banks of the Speed River.

The York Trunk is the main trunk of the sanitary sewer system centrally located along the Speed and Eramosa Rivers, that flows east to west to the treatment plant. Several collectors discharge into the main trunk:

- The south of the City is serviced in its majority by a 900 - 1200 mm collector on the western limit of the City that ultimately crosses the Speed River through a triple barrel siphon (300 mm, 600 mm, 750 mm) to connect to the WRRC. Two other smaller 675 mm and 750 mm collectors service the South-East Side of the City. They connect to the York Trunk after crossing Eramosa River through two other siphons.
- The North Side of the City is serviced by four main collectors. The North-West is serviced by a 1050 mm pipe on Deerpath Dr with a 1200 mm rail crossing and a reduction to 600 mm crossing Wellington St W. This reduction in sewer size is irregular, however the capacities of the sewers are similar with an increased slope on the smaller 600 mm sewer. The North-Centre of the City is serviced by a 900 mm pipe that runs southernly on Dawson Rd and Alma St, with a reduction to 600 mm and ultimately connects to the main trunk at the intersection of Waterloo Rd and Wellington St. The North-East is serviced by two collectors: a 825 mm along the east shore of Speed River, and a 750 mm on York Rd. The Rockwood community is also serviced by Guelph's wastewater collection system. Rockwood flows have been included as a constant flow, connected to the 300 mm sewer on the eastern edge of the City on York Rd.

Over 45% of the sanitary collection system is composed of polyvinyl chloride (PVC) pipe. The remainder is split amongst several pipe materials, including asbestos cement, reinforced concrete, non-reinforced concrete, vitrified clay, iron, clay tile, etc. The sanitary sewers range from 1 to 100+ years old, with the older infrastructure located in the downtown core.

1.2.1 Pumping Stations

There are several pumping stations within the City that lift wastewater and convey it downstream and ultimately to the WRRC. This assessment includes the pump stations understood to be operational. The assessment does not consider private pump stations. The following pump stations were considered:

1. Barton Estates
2. Gazer Mooney
3. Kortright
4. Northern Heights
5. Terraview
6. Nima Trails
7. Landfill Eastview

It should be noted that the Gazer Mooney SPS is owned by the Guelph-Eramosa Township, but is operated by the City of Guelph. The community of Rockwood is also serviced by a sanitary pumping station however it is not included in the model and the contribution has been included as a constant flow at the receiving MH in the City system.

2.0 PLANNED FUTURE GROWTH

Future infrastructure requirements are largely driven by population growth and total water consumption. The areas of planned development were established in close collaboration with the City based on information from the planning department and ongoing secondary studies. Population and jobs growth for greenfield areas such as the Guelph Innovation District (GID) and Clair Maltby Secondary Plan (CMSP) were provided by the City to align with external studies. GIS was used to consolidate growth data and distribute population and job estimates at specified locations throughout the City.

The City's ongoing Municipal Comprehensive Review (Shaping Guelph) project outlines projected growth to 203,000 people and 116,000 jobs by 2051. However, when assessing underground infrastructure and its life expectancy, it is important to consider that new infrastructure will be in use past 2051 and thus must be sized to service growth that occurs after 2051. As such, the City has projected the maximum allowable growth that could be supported in each of the Strategic Growth Areas to create a 2051+ Ultimate Buildout population distribution for the purpose of this study. The 2051+ Ultimate Buildout scenario includes 239,770 people and 126,198 jobs and was established by applying the maximum densities across land uses for strategic growth areas and incorporating established populations for greenfield development (i.e. Clair-Maltby) within the existing urban boundary. This maximum growth scenario was used for the servicing master plan to evaluate the largest impact on water and wastewater linear infrastructure. Table 2-1 below shows summary of future population projections.

Table 2-1 Future Population Projections

Scenario	Total Population	Total Jobs
2051 (WSMP)	203,000	116,000
2051+ Ultimate Buildout	**239,770**	**126,198**

2.1 Growth Water Demand

This study builds from the work completed as part of the 2021 WSMP, where per capita consumption rates were analyzed to determine future supply requirements. The residential, ICI and NRW demands in liters per capita per day (Lcd) established through the 2021 WSMP are summarized in Table 2-2 below and were applied in this WWSMP to determine future drinking water requirements of planned growth.

Table 2-2 Growth Per Capita Water Usage

Category	Per capita water usage (Lcd)
Residential	167
Employment	191
Non-revenue water (NRW)	61

Residential and NRW per capita usage was multiplied by population growth, and employment Lcd was multiplied by estimated jobs. Table 2-3 below summarizes the existing and future population and employment values as well as the expected total water demand. As discussed in Section 2.0, the 2051+ growth scenario was used for this study and results in a higher demand than under the 2051 scenario used in the 2021 WSMP.

Table 2-3 Growth Water Demand

Scenario	Total Population	Total Jobs	Total Water Demand (m^3/d)	ADD (L/s)	MDD (L/s)
2051 (WSMP)	203,000	116,000	68,306	791	1,059
2051+ Ultimate Buildout	**239,770**	**126,198**	**77,346**	**895**	**1,199**

2.2 Growth Wastewater Generation

To establish future scenarios in the wastewater models, the intensification and growth areas provided to the project team were added to the models. Growth area flow contributions were established as follows:

- Residential growth flow generation – 227 L/c/d. The residential rate of 227 L/cap/day is the average per capita flow generated rate established during the calibration process. See Table 5-1 in TM2b.
- ICI growth flow generation – 191 L/employee/d. During the initial model development and calibration, employment population was not available, so area-based flow generation rates were used for ICI land use areas. The ICI growth data was provided as projected employee counts and as a result, an employee-based rate is required. The water consumption rate of 191 L/employee/d that is being used in the potable water system assessment was adopted for the wastewater analysis.
- RDII growth flow generation – The growth data included both infill development and greenfield development. For infill development, it is assumed that minimal additional sewers are introduced and as a result, no additional RDII considerations are included. For the greenfield areas, additional consideration for RDII is warranted. As the layout of the greenfield developments are unknown, estimates for the proportion of area that would contribute to RDII based on the future roadways and wastewater network expansion are required. Existing residential and commercial development areas were assessed to calculate the proportion of roadway area that contributes to RDII, and these values were used to assign additional area to the model for the greenfield developments. Table 2-4 and Table 2-5 show the proportion of different land uses in two existing residential and commercial developments used to calculate the areas for additional RDII.

Table 2-4 Representative Residential Landuse

Land Use	Area (ha)	Area (%)
Low Density Residential	68.6	73.0%
Neighbourhood Commercial Centre	0.5	0.6%
Open Space and Park	0.4	0.4%
Road	**21.7**	**23.1%**
Significant Natural Areas and Natural Areas	2.7	2.9%

Table 2-5 Representative ICI Landuse

Land Use	Area (ha)	Area (%)
Industrial	137.6	78.3%
Road	**25.6**	**14.6%**
Service Commercial	7.7	4.4%
Significant Natural Areas and Natural Areas	4.8	2.8%

3.0 WATER SYSTEM PERFORMANCE

3.1 Water System Criteria

The performance of the existing water distribution system was analyzed using the criteria discussed below.

The water system pressure criteria are discussed in the *Design Criteria, LOS and Sensitivity Analysis TM (C3/Stantec, December 2020)* and summarized in Table 3-1 below.

Table 3-1 Water System Performance Criteria

System Performance Criteria	
Pressure	• Preferred range of **50 - 80 psi** (345 - 550 kPa)
	• **40 - 100 psi** (275 - 690 kPa) where preferred range cannot reasonably be achieved
	• Minimum pressure of **20 psi** (140 kPa) under MDD + Fire Flow conditions
Headloss	• Maximum of **2 m/km** in watermains 300mm or greater under typical operating conditions
Velocity	• Maximum of **5 m/s** under typical operating conditions
	• Maximum of **3 m/s** in watermains 300mm or greater under MDD + Fire Flow conditions

The criteria for water infrastructure are discussed in the *Design Criteria, LOS and Sensitivity Analysis TM (C3/Stantec, December 2020)* and summarized in Table 3-2 below.

Table 3-2 Water System Infrastructure Criteria

Infrastructure Criteria	
Pumping	• Firm pumping capacity of each pressure zone meets MDD + Fire Flow
Storage	• Required storage for each pressure zone calculated using MECP guidelines. • A+B+C ○ A = Fire Storage ○ B = Equalization Storage (25% of maximum day demand) ○ C = Emergency Storage (25% of A+B)

The suggested fire flow criteria based on development type are summarized in Table 3-3 below. These fire flow guidelines were established through the *Review of Fire Flow Calculation Practices Study* completed by C3W in 2020 and are recommended guidelines for proposed future developments. For master planning purposes, existing areas with an available fire flow of less than 30 L/s were flagged as a concern.

Table 3-3 Fire Flow Suggested Criteria

Fire Flow Criteria	
Residential – low density	80 L/s for 2 hours
Residential – medium density	150 L/s for 2 hours
Residential – high density	200 L/s for 2.5 hours
Commercial – small	200 L/s for 2.5 hours
Commercial – medium	267 L/s for 3.5 hours
Commercial – large	367 L/s for 5 hours
Institutional – small	150 L/s for 2 hours
Institutional – large	250 L/s for 3.5 hours
Industrial	250 L/s for 3.5 hours

3.2 Planned Water System Development

Planned future water infrastructure projects identified through other Environmental Assessments were considered throughout this analysis and are summarized in the following sections.

3.2.1 Storage

Planned future and existing water storage is summarized in Table 3-4 below. Future storage facilities include a new Clythe Reservoir and a Clair Maltby ET. The existing Clythe PS and Reservoir is planned to be retired and replaced with a new facility in combination with the new Clythe Well and WTP. The pre-design of the new facility is expected to start in 2022 and the facility is expected to be online by approximately 2025. The volume for the future Clythe Reservoir is expected to be approximately 6.5 ML.

The future Clair Maltby ET is planned for Zone 3. A preliminary volume for the Clair Maltby ET of 5ML was used based on the Clair Maltby Master Environmental Servicing Plan completed by Wood Canada Limited (Wood) in 2022 (2019 CM MESP).

Upgrades are being completed at Woods which will involve reducing the Woods reservoir volume by 3,000 m^3. These upgrades are expected to be completed prior to 2031.

Based on the planned future storage facilities, the system is expected to have a total available storage volume of approximately 69 ML by 2051.

Table 3-4 Planned Future Storage Summary

Facility	Type	Available Volume (ML) Existing	2031	2041	2051
Zone 1/3					
FM Woods Reservoir	Pumped	29	26	26	26
Verney Elevated Tank	Floating	4	4	4	4
Clair Elevated Tank	Floating	5	5	5	5
University Reservoir	Pumped	2	2	2	2
Clair Maltby Elevated Tank	Floating			5	5
Zone 2					
Speedvale Elevated Tank	Floating	2	2	2	2
Clythe Reservoir	Pumped	1	7	7	7
Paisley Reservoir	Pumped	13	13	13	13
Summary - Floating Storage					
Zone 1 & 3 Floating		9	9	9	9
Zone 2 Floating		2	2	2	2
Total Available Floating Storage		**11**	**11**	**11**	**11**
Summary - Total Storage					
Zone 1 & 3 Total		41	38	43	43
Zone 2 Total		16	22	22	22
Total Available Storage		**57**	**59**	**64**	**64**

Red = future storage

3.2.2 Supply

Planned future and existing supply sources are summarized in Table 3-5 below. The values used for the future sources are from Table 8-5 within the 2021 WSMP and were based on the minimum potential capacity for each source. The minimum potential capacity for each future source is the conservative hypothetical capacity, modelled for each alternative scenario in the WSMP. Future sources are hypothetical in nature at this time.

It should be noted that the preferred alternative in the 2021 WSMP had a timing horizon for the Guelph Lake and Guelph Southeast sources of beyond 2051. However, the MSP uses a higher growth population and demand than the WSMP (2051+ Ultimate Build Out). For the purpose of sizing infrastructure that will be needed prior to 2051, but that will provide capacity for growth past 2051, these sources were considered to be active under the 2051+ Ultimate Build Out scenario for the purposes of hydraulic modelling. Both the Guelph Lake source and the Guelph North Well were utilized in the 2051+ Ultimate Build Out scenario. These modelling assumptions, along with population and supply assumptions, will be reviewed and updated on a 5 year basis when the WWSMP is updated.

Table 3-5 Planned Future Supply Summary

Facility	Capacity (m3/day)	Capacity (L/s)			
		Existing	2031	2041	2051+
Zone 1					
Arkell Well 1	2,000	23	23	23	23
Arkell Wells 6, 7, 8, 14, 15	28,800	333	333	333	333
Glen Collector	5,100	59	59	59	59
Carter Wells (1, 2)	5,184	60	60	60	60
Burke Well	6,500	75	75	75	75
Dean Well	1,500	17	17	17	17
Downey Well	5,237	61	61	61	61
Emma Well	2,800	32	32	32	32
Membro Well	5,200	60	60	60	60
Park Wells (1, 2)	8,000	93	93	93	93
Queensdale Well	1,100	13	13	13	13
University Well	2,500	29	29	29	29
Water Well	1,901	22	22	22	22
Ironwood Well	2250		26	26	26
Guelph South Well	2250		26	26	26
Dolime Quarry	3000		35	35	35
Lower Collector	4000			46	46
Arkell Collector ASR	1170			14	14
Hauser Well	425				5
Guelph Southeast Well	1600				19
Zone 2					
Calico Well	1,400	16	16	16	16
Helmar Well	800	9	9	9	9
Paisley Well	1,400	16	16	16	16
Clythe Well	1180		14	14	14
Logan/Flemming Well	4180			48	48
Guelph North Well	2935				34
Guelph Lake	12312				143
Summary					
Zone 1	88,517	878	964	1,024	1,048
Zone 2	25,057	41	55	104	280
Total System	**113,574**	**919**	**1,020**	**1,128**	**1,328**

Red = future supply

3.2.3 Pumping

The planned future pumping capacity is summarized in Table 3-6 below. Upgrades are planned at the Paisley pump station and the Robertson pump station is planned to be replaced with a new Verney pump station to increase the available capacity. A new pump station is planned for the Clythe facility. Additionally, pumping will be required for each of the future supply sources listed in Table 3-5 above. For future supply sources, such as the future wells, the Guelph Lake and the Dolime Quarry, the pump station capacity was assumed to be equivalent to the supply capacity presented in Table 3-5 above. The Lower Collector and Arkell ASR will supply the Woods WTP via the Arkell Aqueduct and therefore, were not considered as additional pump capacity. Pump station upgrades are planned at the Woods PS to improve redundancy, but the overall firm pump capacity is not expected to change from existing conditions. The existing Woods PS is typically controlled based on the Verney ET level. The future Woods PS is expected to operate based on the Clair ET level.

Table 3-6 Planned Future Pump Station Summary

Facility	Firm Capacity (L/s) Existing	2031	2041	2051+
Zone 1				
F.M. Woods Pump Station	1050	1050	1050	1050
Burke Pump Station*	75	75	75	75
Dean Pump Station*	17	17	17	17
Downey Pump Station*	61	61	61	61
Emma Well*	32	32	32	32
Membro Pump Station*	60	60	60	60
Park Pump Station	70	70	70	70
Queensdale Pump Station	13	13	13	13
University Pump Station	27	27	27	27
Water St Well*	22	22	22	22
Ironwood Well*		26	26	26
Guelph South Well*		26	26	26
Dolime Quarry*				35
Hauser Well*				5
Guelph Southeast Well*				19
Guelph Lake*				143
Zone 2				
Calico Pump Station *	16	16	16	16
Helmar Pump Station*	9	9	9	9
Paisley - Vertical Turbine Pump Station	212	300	300	300
Paisley - Horizontal In-Line Pump Station	106	180	180	180
Robertson Pump Station	88			
Verney Pump Station		240	300	300
Clythe Pump Station	126	255	255	255
Logan/Flemming Well*		48	48	48
Guelph North Well*				34
Zone 3				
Clair Pump Station	470	470	470	470
Summary				
Zone 1	1,428	1,480	1,480	1,680
Zone 2	557	1,049	1,109	1,143
Zone 3	470	470	470	470

Existing pump data sourced from 2019 DWWP
**Firm Capacity based on Well Supply Capacity (2021 WSMP)*
Red = future pump station

3.2.4 Linear Development

To assess the impact of future growth on existing infrastructure, the future growth was applied to the system's existing watermain network, with a few exceptions. New watermains were added to connect future well supply points or new pumping stations discussed above. New watermains were also added to the model to connect future growth areas where needed, such as greenfield sites including the GID, CMSP. While there are many linear infrastructure upgrades in the planning horizon from previous studies (such as the Wellington-Clair feedermain and the Downtown Servicing Study) these potential projects were not included in the model for this analysis and will be tested as part of the next phase of the master plan.

3.3 Water System Capacity – Desktop Analysis

The purpose of completing a desktop analysis of the water system is to identify potential concerns in storage, supply and pumping capacity of the system. In Section 3.4, the hydraulic performance of the water system will be evaluated utilizing the City's hydraulic model to better understand how the water system functions both under existing and future demand conditions. The hydraulic model will demonstrate how the various aspects of the system work together as a unit. Storage, supply, and pump station projects identified through other Environmental Assessments have been carried forward in the desktop analysis.

3.3.1 Water Storage Required

The required water storage for each pressure zone was calculated using the MECP guidelines and is summarized in Table 3-7 below. The large commercial fire flow of 367 L/s for 5 hours was used for Zone 1 based on the Stone Road Mall. A fire flow of 267 L/s for 3.5 hours was used for Zone 2. It should be noted that storage can be transferred between pressure zones if there is sufficient pumping and watermain capacity.

Table 3-7 Required Storage

	Required Storage (ML)	
Storage Requirements	**Existing**	**2051+**
Zone 1/3		
A - Fire Storage	7	7
B - Equalization Storage (25% of MDD)	11	19
C - Emergency Storage (25% of A+B)	4	7
Zone 2		
A - Fire Storage	3	3
B - Equalization Storage (25% of MDD)	5	6
C - Emergency Storage (25% of A+B)	2	2
Summary		
Zone 1/3 Required Storage	22	33
Zone 2 Required Storage	10	12
Total Required Storage (Zone 1/3 + 2)	32	45

The required storage for existing and 2051+ conditions from Table 3-7 is compared to the available storage from Table 3-4 in Figure 3-1 and Table 3-8 below. The 2051+ available storage includes the proposed future storage at the Clythe Reservoir and the Clair Maltby ET.

It can be seen that the available storage for each pressure zone is expected to exceed the required storage under 2051+ conditions.

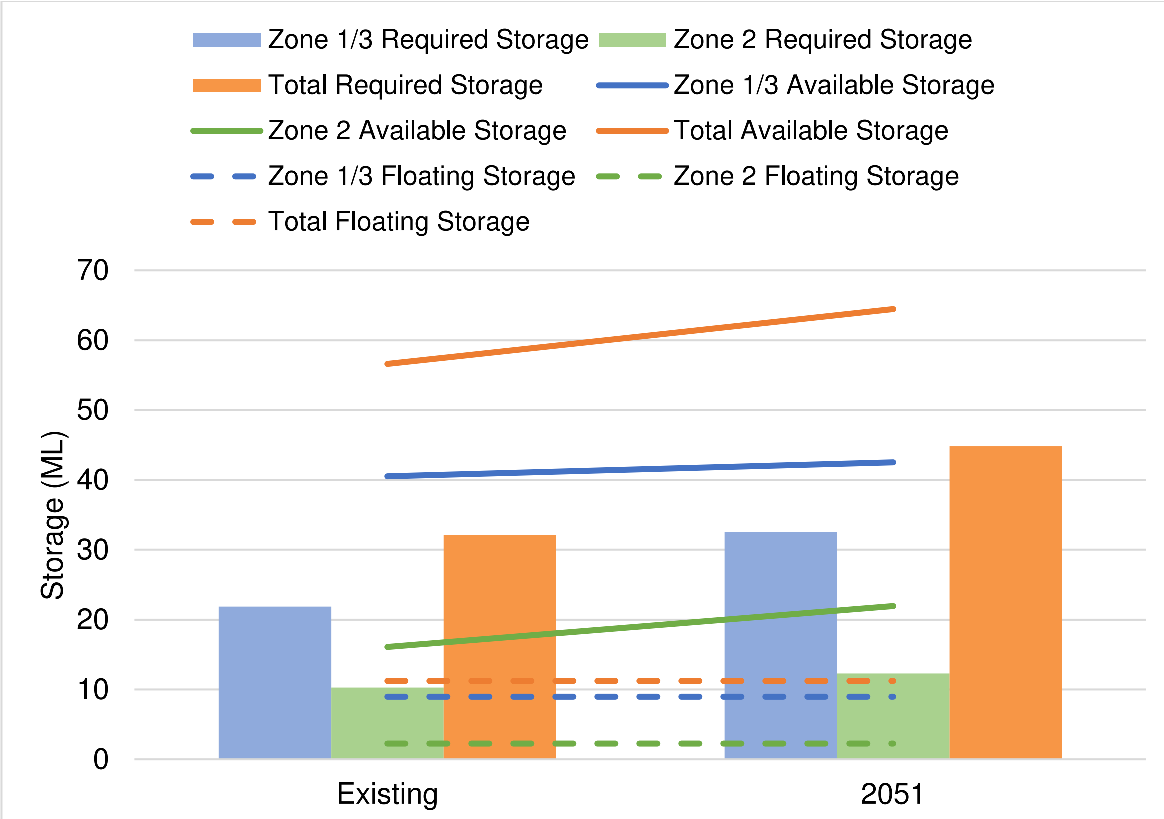


Figure 3-1 Existing Versus Available Storage

Table 3-8 Available Storage Surplus

	Storage Surplus (ML)*	
Zone	**Existing**	**2051+**
Zone 1 & 3	19	10
Zone 2	6	10
Total System	24	20

**Available - Required*

3.3.2 Water Supply

The available water supply (see Table 3-5) is compared to the existing and 2051+ MDD in Figure 3-2 and Table 3-9 below. The 2051+ available supply includes future supply sources identified in the 2021 WSMP. The available supply exceeded the MDD under existing and 2051+conditions. The MDD was 79% of the available supply under existing conditions and 90% of the available supply under 2051+ conditions.

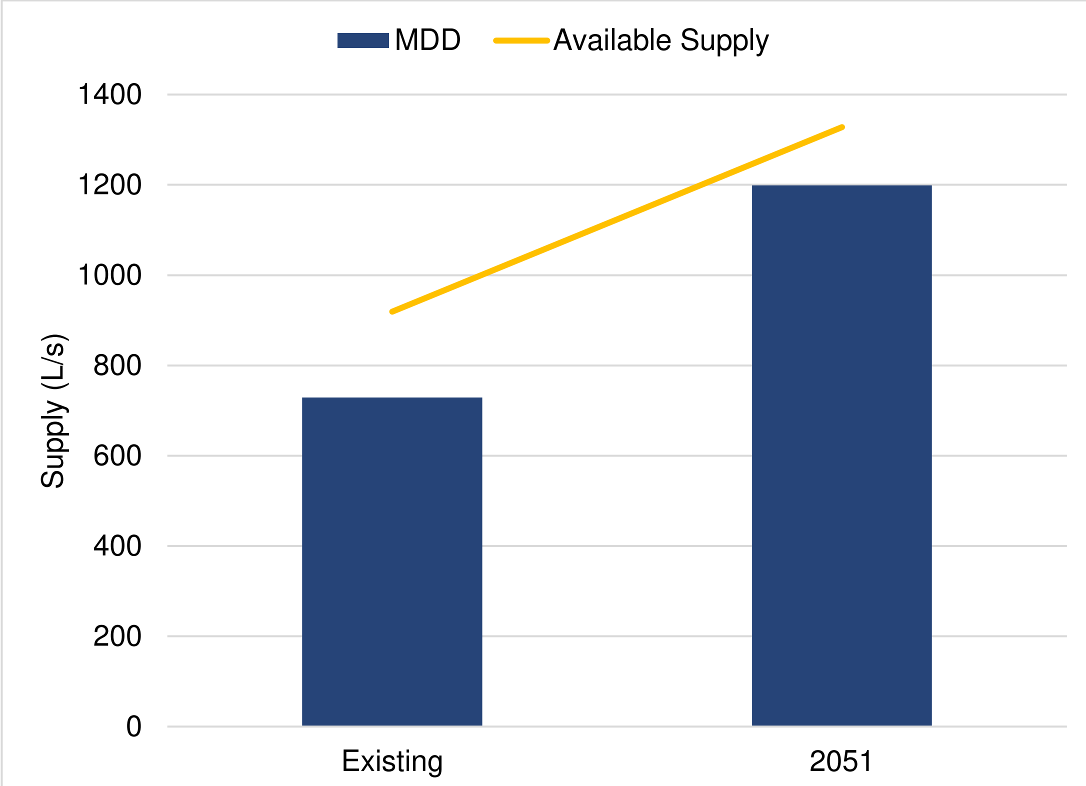


Figure 3-2 Available and Required Supply

Table 3-9 Available Supply Surplus (L/s)

	Available Supply Surplus (L/s)*	
	Existing	**2051+**
Available Supply	919	1328
MDD	729	1199
Supply Surplus*	**190**	**129**
MDD % of Supply	**79%**	**90%**

*Available Supply – MDD
**MDD/Available Supply

3.3.3 Pumping Capacity

The required pumping capacity based on the MDD and fire flow for each Zone is summarized in Table 3-10 below. The large commercial fire flow of 367 L/s for 5 hours was used for Zone 1 based on the Stone Road Mall. A fire flow of 267 L/s for 3.5 hours was used for Zone 2.

Table 3-10 Required Pumping

	Required Pumping (L/s)	
Zone	**Existing**	**2051+**
Zone 1		
MDD	498	792
Fire Flow	367	367
Zone 2		
MDD	225	300
Fire Flow	267	267
Zone 3		
MDD	6	106
Fire Flow	267	267
Summary		
Zone 1	865	1159
Zone 2	492	567
Zone 3	273	373

The required pump capacity for existing and 2051+ conditions from Table 3-10 is compared to the available firm pump capacity from Table 3-6 in Figure 3-3 and Table 3-11 below. It can be seen that the available pump capacity in each pressure zone is expected to be sufficient under existing and 2051+ conditions.

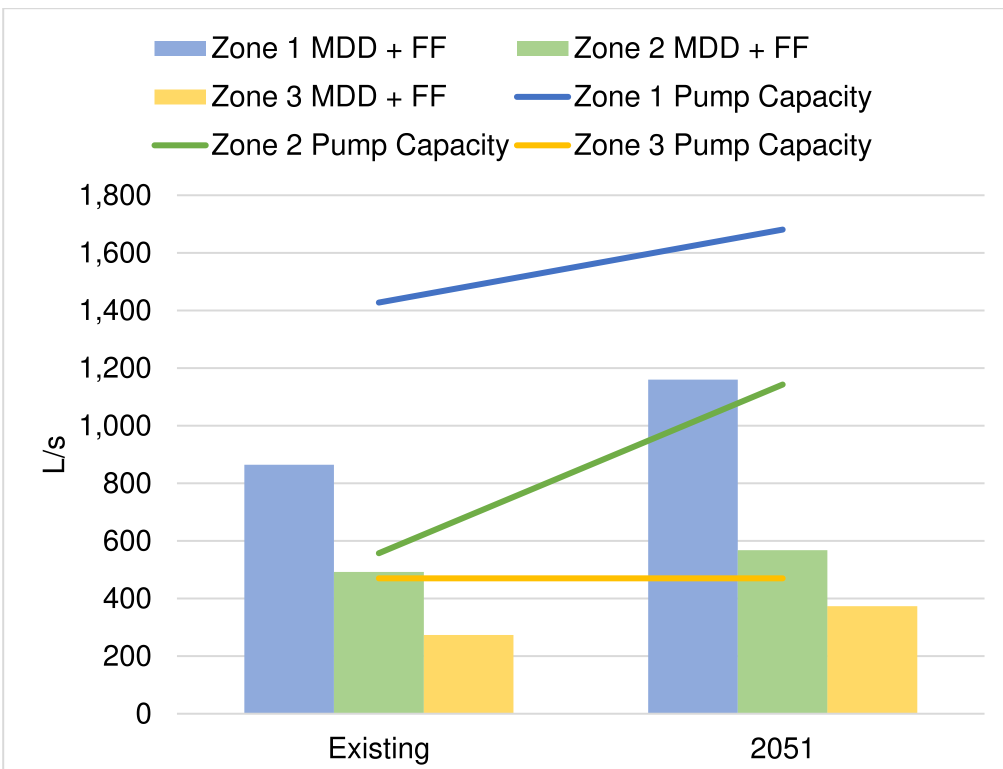


Figure 3-3 Available and Required Pumping

Table 3-11 Pump Capacity Surplus

	Pump Capacity Surplus (L/s)*	
Zone	**Existing**	**2051+**
Zone 1	563	521
Zone 2	65	575
Zone 3	197	97

**Available - Required*

3.4 Water System Assessment

The City's InfoWater Pro model was used to assess the existing water distribution system's performance under existing and 2051+ demand conditions. Planned future wells, pump stations and storage facilities that were identified in other EA projects were included in the 2051+ model scenarios. The future infrastructure that was included in this scenario is summarized in Section 3.2. There are many linear infrastructure upgrades identified in previous studies that were not included in the model for this analysis. These will be tested as part of the next phase of the master plan.

3.4.1 Water System Failure Analysis

A failure analysis was completed under existing conditions. The purpose of this analysis was to flag critical infrastructure for consideration when recommending proposed future upgrades in the next phase of this project. Twenty-two (22) infrastructure failure scenarios were modelled to assess the criticality of each location. The failure test locations are summarized in Table 3-12 and shown in Figure 3-4 below. The failure locations were selected based on their importance to the water system.

Each failure scenario was modelled under MDD conditions, unless it was assumed that the failure event would warrant a water use advisory under which case the scenarios was modelled under ADD conditions. The duration of the model simulation was based on the expected duration to resolve the failure at each location. The majority of watermain shutdowns were expected to be resolved within 24-hours unless the watermain is in a challenging location such as a railway or river crossing. Pump station and reservoir failures were assumed to be resolved within 24-hours. The Arkell Aqueduct shutdown scenario had the longest duration of 1-week due to the challenges associated with accessing and repairing the pipe.

All failure scenarios were modelled under existing conditions, with the exception of the Clair Rd BPS. As the existing Zone 3 demand is relatively small, the Clair BPS was not expected to be critical under existing conditions. As there is expected to be a significant amount of future growth in Zone 3, the Clair BPS was assessed under 2051+ conditions.

Table 3-12 Water System Failure Scenarios

Pressure Zone	Failure Scenario	Infrastructure Shutdown	Facility Type/Size	Rationale	Demand Scenario	Duration of Shutdown
Zone 1	1	FM Woods PS	Pumping Station/Supply	Largest source in the City	ADD	24-hours
	2	Arkell Aqueduct Shutdown	Pumping Station/Supply	Largest supply in the City, Woods Reservoir is available	ADD	1-week
	3	FM Woods Reservoir (2 cells offline)	Pumping Station/Supply	Largest supply in the City, less storage available	ADD	24-hours
	4	Park St Wells	Pumping Station/Supply	Second largest supply in the City and located near downtown	MDD	24-hours
	5	Stevenson Feedermain	600mm	Connection to the Verney ET. Emma Street.	MDD	24-hours
	6	Metcalfe Watermain (Railway Crossing)	300mm	Extended shutdown duration due to railway. Near downtown.	MDD	72-hours
	7	Downtown River Crossing (Wyndham)	350mm	Extended shutdown duration due to river. Key downtown connection.	MDD	72-hours
	8	University Watermain (River Crossing)	400mm	Extended shutdown duration due to river. Key Woods connection.	MDD	72-hours
	9	Victoria Rd Watermain	400mm	Connection to Clair ET	MDD	24-hours
	10	Stone Rd Watermain	250mm	Key east-west Zone 1 connection. Near Stone Road Mall.	MDD	24-hours
	11	Wellington Watermain (Crossing at Hanlon)	500mm	Paisley Reservoir fill.	MDD	24-hours
	12	Hanlon Watermain (South of Downey)	400mm	Connection to Clair ET.	MDD	24-hours
	13	York Feedermain	600mm	Clythe Reservoir fill.	MDD	24-hours

Pressure Zone	Failure Scenario	Infrastructure Shutdown	Facility Type/Size	Rationale	Demand Scenario	Duration of Shutdown
Zone 2	14	Paisley PS and Reservoir	Pumping Station/Supply	Zone 2 Supply and Storage. Largest Zone 2 Storage.	ADD	24-hours
	15	Clythe PS	Pumping Station/Supply	Zone 2 Supply and Storage. Main supply on east side of Zone 2.	ADD	24-hours
	16	Speedvale ET	Storage	Zone 2 floating storage.	ADD	24-hours
	17	Paisley Feedermain	500mm	Key connection between Paisley PS and Speedvale ET.	MDD	24-hours
	18	Speedvale Feedermain	400mm	Key connection between Robertson PS and Speedvale ET.	MDD	24-hours
	19	Woodlawn Watermain	300mm	Key east-west Zone 2 connection.	MDD	24-hours
	20	Watson Pkwy Watermain	400mm	Key Clythe connection.	MDD	24-hours
Zone 3	21	Clair Rd BPS	Pumping Station/Supply	Only Zone 3 supply.	2051+ ADD	24-hours
	22	Poppy Rd Watermain	400mm	Key Clair PS connection.	MDD	24-hours

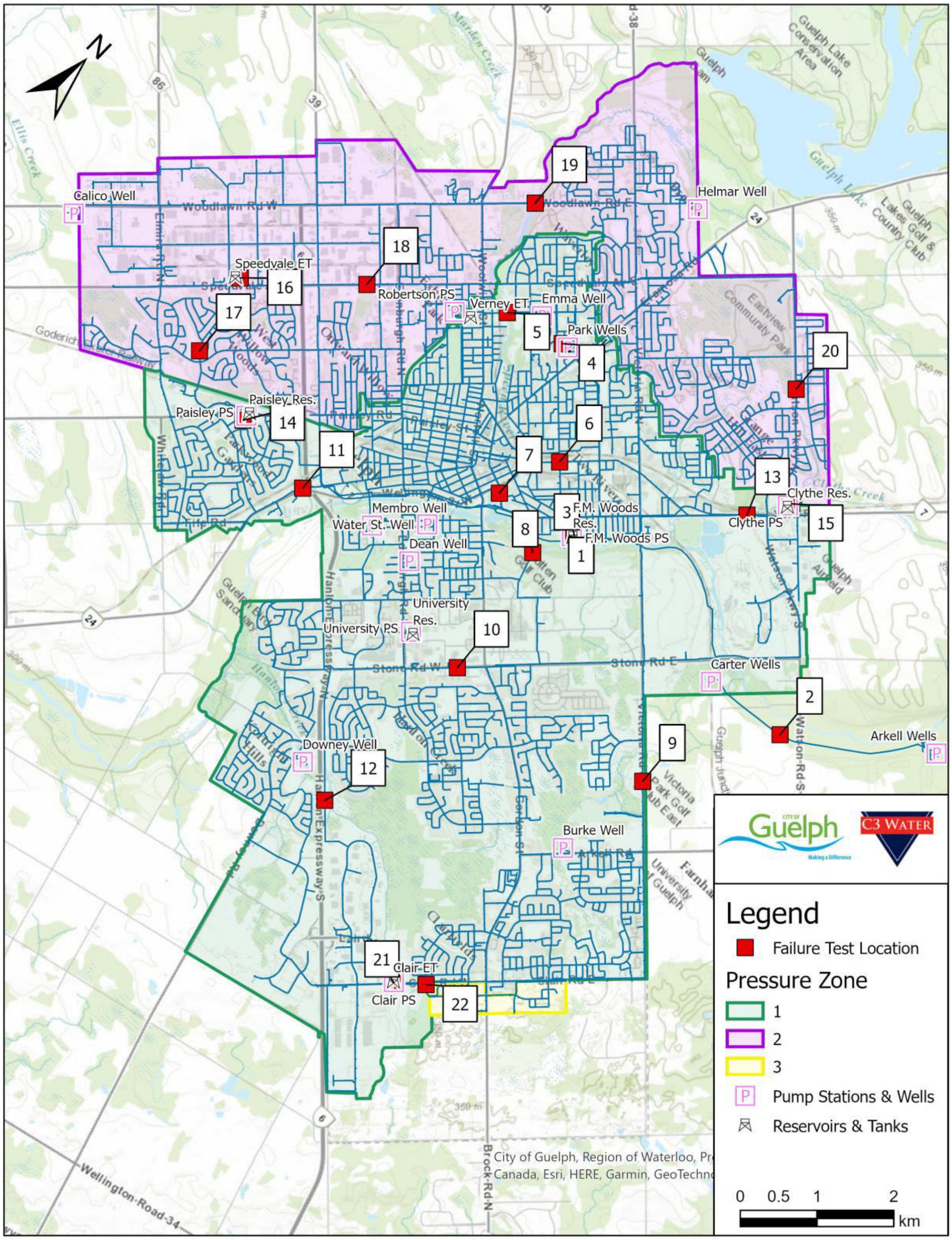


Figure 3-4 Failure Test Locations

The criticality of each infrastructure scenario was assessed based on the impact that the shutdown had on pressure, fire flow and pipe velocity. The impact on pressure was assessed based on the portion of the system that fell below 40 psi as a result of the failure. This was quantified based on the demand of the area below 40 psi.

Approximately one hundred (100) hydrant locations were selected across the system for fire flow analysis. The same hydrant locations were used for each failure scenario and the results were compared to the available fire flow results under typical operating conditions. The average available fire flow at the hydrant locations was compared the average fire flow under typical operating conditions to determine the percent reduction. Any locations where the available fire flow fell below either 30 L/s or 100 L/s where it was above these thresholds under typical operating conditions, was flagged.

The distribution system velocity was checked for each failure scenario to determine if the failure caused any watermains to exceed 3 m/s.

Three (3) critical customers were identified, the Guelph General Hospital, St Joseph's Retirement Home and the Southgate Data Center. The impact on pressure and fire flow for each of these customers was also considered when assessing the criticality of each infrastructure scenario.

The criteria used for assessing the impact of each failure scenario is summarized in Table 3-13 below. The criticality of each failure scenario was then classified as either high, medium or low. Definitions of the criticality ratings are summarized in Table 3-14 below.

Table 3-13 Water Failure Scenario Criteria

Criteria	Pressure	Fire Flow	Velocity	Storage	Critical Locations**
Impact	>1% of system fell below 40 psi*	Decreased by > 10% compared to normal operating conditions	Exceeded 3m/s	Reservoir or ET fell below 50% full	
Critical Impact	>10% of system fell below 40 psi*	< 30 L/s in residential areas or < 100 L/s in ICI areas***		Reservoir or ET fell below 25% full	Fire Flow below 100 L/s or Pressure below 40 psi at any Critical Location

**Excluding areas that fell below 40 psi under typical operating conditions*
***Guelph General Hospital, St Joseph's Retirement Home & Southgate Data Center*
****Where fire flow was above these limits under typical operating conditions*

Table 3-14 Water Failure Scenario Scoring

Criticality Score	Criteria
Low	No Impact
Medium	1-2 Impacts
High	> 3 Impacts or any Critical Impacts

Of the failure locations modelled, three (3) were given a score of high criticality and one (1) was given a score of medium criticality. This is summarized in Table 3-15 below. Full results of the failure analysis can be found in Appendix A.

Table 3-15 Critical Infrastructure Results Summary

Failure Scenario	Infrastructure Shutdown	Criticality Score	Rationale
1	Woods PS	High	System demand unable to be met and ETs are drained after 12 hours. Critical impact on system pressure and fire flow and critical impact at all three critical locations.
2	Arkell Aqueduct	High	System demand unable to be met once Woods Reservoir drained after 22 hours. Critical impact on system pressure and fire flow and critical impact at all three critical locations.
8	University Watermain (River Crossing)	Medium	Impact on system pressure in high elevation areas south of Speed River.
14	Paisley PS and Reservoir	Medium	Speedvale ET fell below 50% full.
15	Clythe PS	Medium	Impact on system pressure and fire flow in east end of Zone 2
21	Clair Rd BPS (2051+)	Medium	Under 2051+ conditions, impact on Zone 3 pressure and CM ET emptied.

Based on the findings of the failure analysis, the existing critical infrastructure will be taken into consideration when developing proposed upgrades in the next phase of this WWSMP.

3.4.2 System Pressure and Zone Analysis

A pressure zone boundary is defined by the elevations in a service area and the City's defined level of service. Locations with higher elevations are operated at a higher hydraulic grade line (HGL) to maintain target pressures from the Design Criteria. The City of Guelph is currently serviced by three (3) pressure zones. The target HGL for each zone as well as the existing ground elevation is summarized in Table 3-16 below. The ground elevations for the existing water system are presented in Figure 3-5 below.

At the target HGLs, Zone 1 is expected to have pressures above and below of the allowable operating range of 40 – 100 psi. Zone 2 has areas that are is expected to exceed the maximum allowable operating pressure of 100 psi but is not expected to fall below the minimum preferred pressure of 40 psi. Zone 3 is not expected to have pressures below the preferred minimum of 40 psi but the maximum pressure is expected to exceed the preferred maximum of 80 psi. This analysis is based on target HGL only. Actual system HGL can vary throughout each pressure zone due to losses throughout the distribution system and fluctuating ET levels.

Table 3-16 Pressure Zone HGLs and Required Ground Elevations

Pressure Zone	Target HGL (m)	HGL Based On	Existing Ground Elevation (m)	Static Pressure at Target HGL (psi)
1	375	Verney and Clair ETs	304.7 – 348.5	33.7 - 100.0
2	393	Speedvale ET	321.8 – 362.4	43.5 - 101.2
3	400*	Clair PS Discharge	335.0 – 353.2	66.5 - 92.4

**this is a long term HGL when Zone 3 is built out. Currently the Clair BPS operates at approximately 393m.*

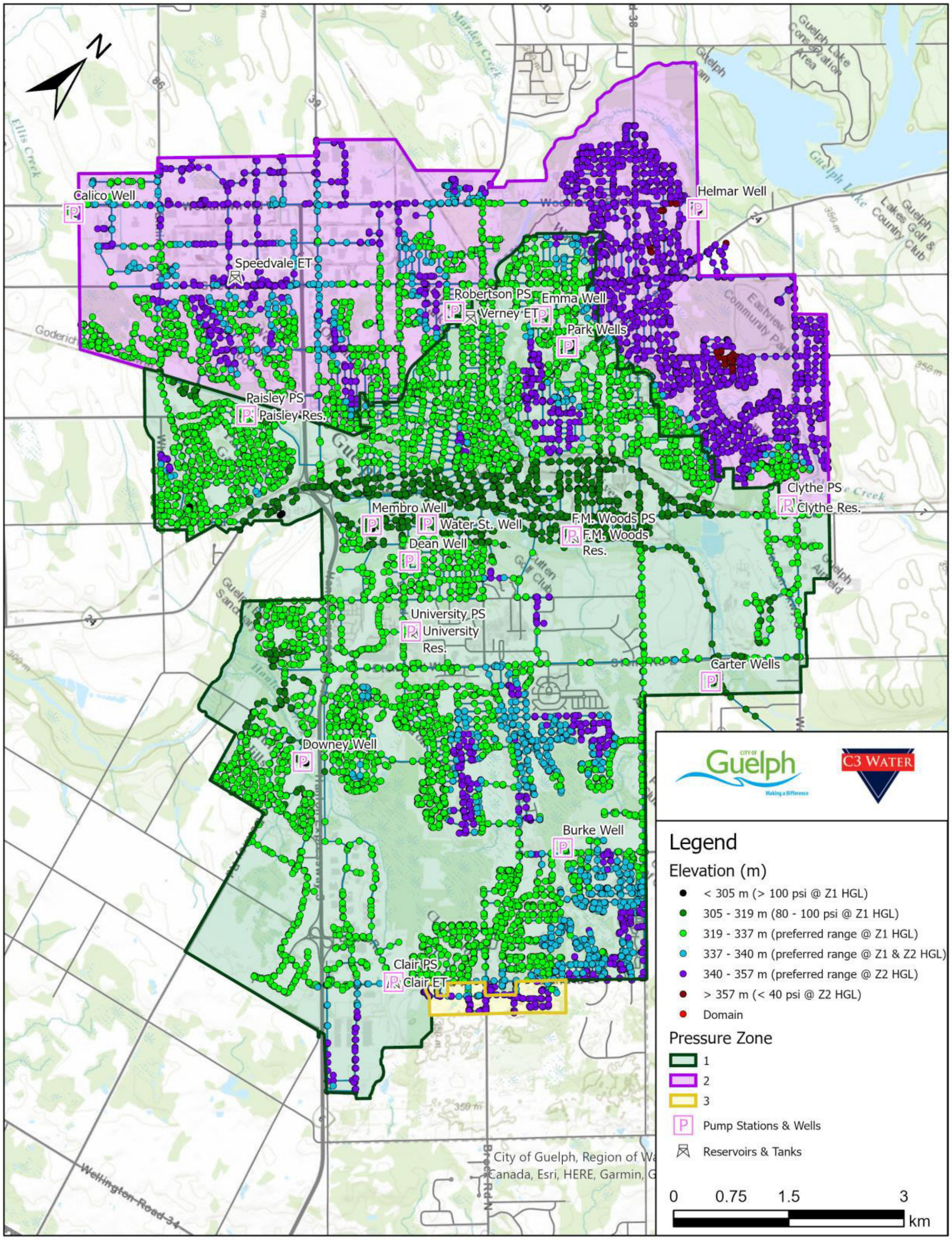


Figure 3-5 Existing System Ground Elevation

The minimum pressure results under existing MDD conditions are presented in Figure 3-6 below. The maximum pressure results under existing MDD conditions are presented in Figure 3-7 below. 2051+ minimum and maximum pressure results are presented in Figure 3-8 and Figure 3-9 below, respectively. Areas P-1 to P-6 were identified to fall outside of the allowable operating range of 40 – 100 psi and are summarized in Table 3-17.

Table 3-17 Pressure Areas of Concern

Area	Location	Pressure Zone	Elevation (m)	Pressure Concern	
				Existing	2051+
PLo-1	Stuart St from Hillcrest Dr to Spring St	1	346.1 - 348.5	Min Pressure of 36 psi. Pressure fluctuates with Woods pump operation.	
PLo-2	Shoemaker Cr	1	343.5 - 346.0	Min Pressure of 36 psi. Pressure fluctuates with Woods pump operation.	
PLo-3	Eastview Drive and Summit Ridge Dr	2	362.0 - 362.2	Min Pressure briefly below 40 psi during peak demand.	Pressure maintained above 40 psi with additional future supply in area (Logan Well, Guelph Lake)
PLo-4	Rickson Ave south of Darnell Rd	1	344.0 - 345.9	Min pressure of 37 psi during peak demand when Clair ET level low	Larger area of pressure below 40 psi compared to existing conditions when Clair ET level low.
PLo-5	Crawley Rd and Southgate Dr South of Clair Rd	1	344.2 - 346.3	Min pressure of 37 psi during peak demand when Clair ET level low	Min pressure of 35 psi during peak demand when Clair ET level low. Larger area compared to existing conditions.
PHi-6	North side of Speed River east of Mccrae Bv	1	305.1 - 308.7	Max Pressure of 104 psi. Pressure fluctuates with Woods pump operation.	Max Pressure of 116 psi when Verney ET full. Larger area above 100 psi (areas below 320m elevation) compared to existing conditions.

Under existing conditions, the pressure fell below 40 psi in high elevation areas of Zone 1. On the south end of Zone 1, the pressure was lowest during peak hour when the Clair ET level was lowest. In the north end of Zone 1, the pressure was lowest when only one (1) pump was running at the Woods PS, in response to the Verney ET level. The pressure briefly fell below 40 psi at the highest elevation location in Zone 2 during peak hour.

Under 2051+ conditions, the pressure fell below 40 psi in areas of the south end of Zone 1The pressure was not found to fall below 40 psi in Zone 2 under 2051+ conditions due to the sources on the east side of Zone 2 including the Guelph Lake and the Logan Well.

Under existing conditions, the maximum pressure was found to exceed 100 psi in low elevation areas along the Speed River when multiple pumps were running at the Woods PS. This high pressure area was found to expand under 2051+ conditions when the Woods PS was running at a higher pressure and flowrate to meet future demands and attempting to fill the Clair ET.

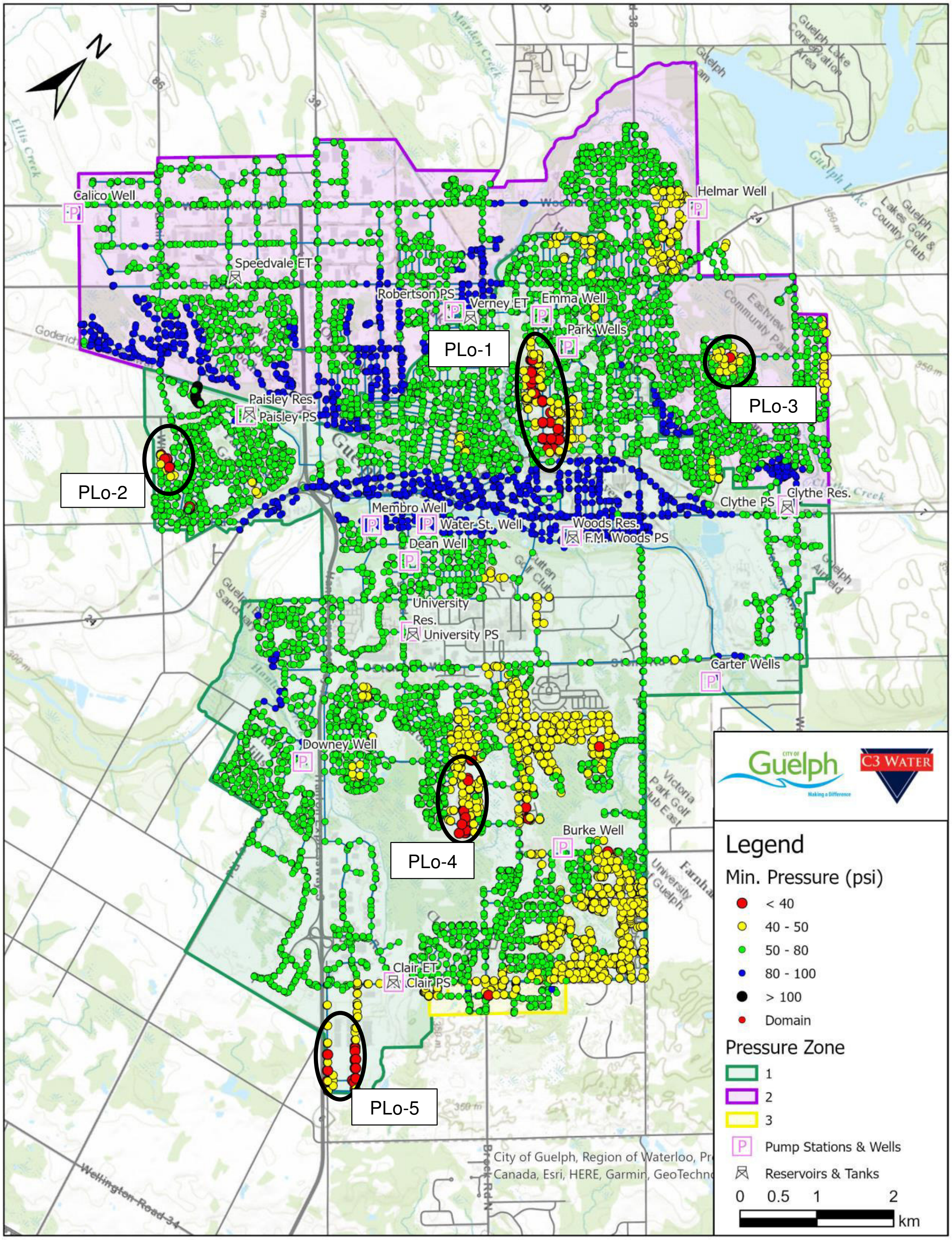


Figure 3-6 Existing MDD – Minimum Pressure

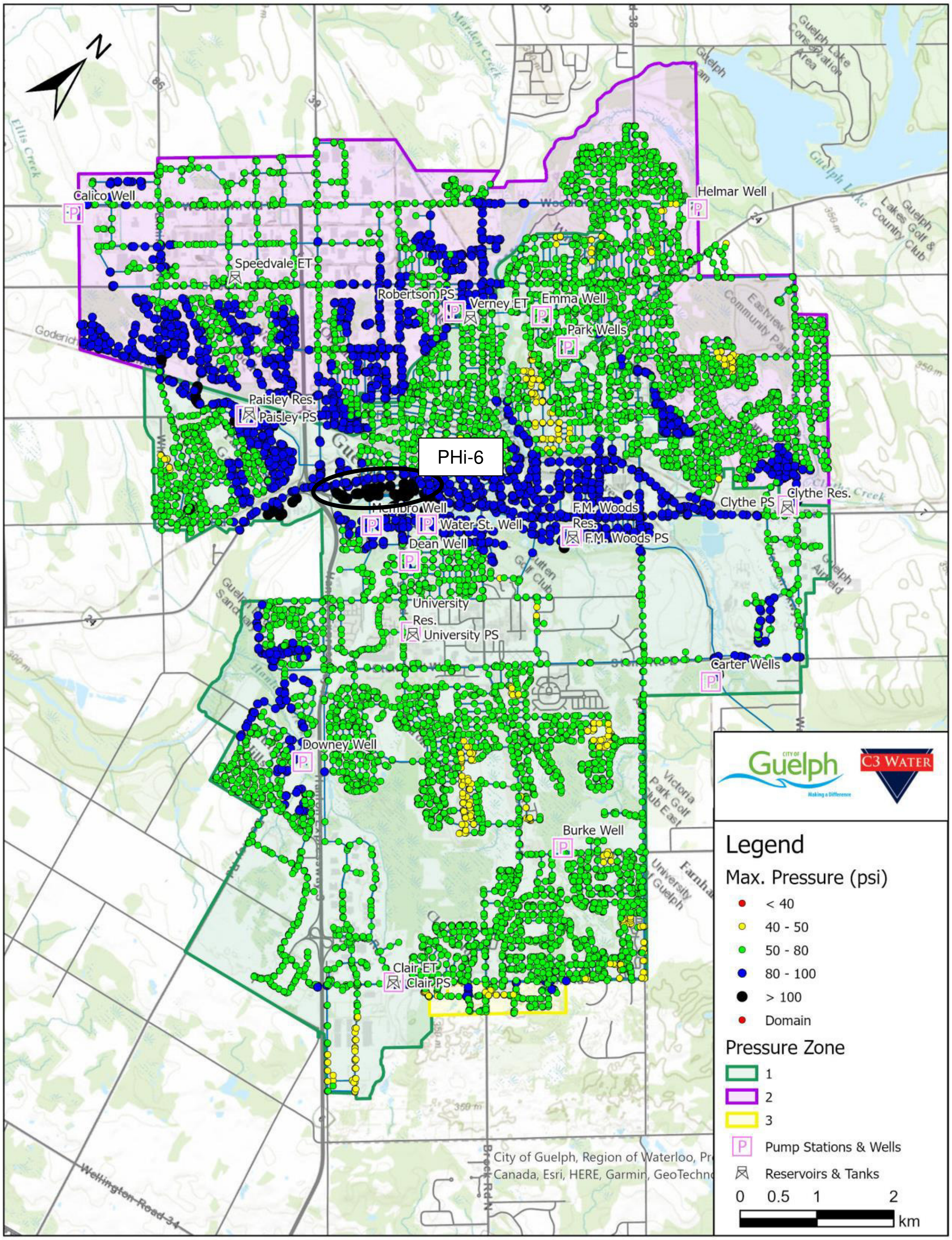


Figure 3-7 Existing ADD – Maximum Pressure

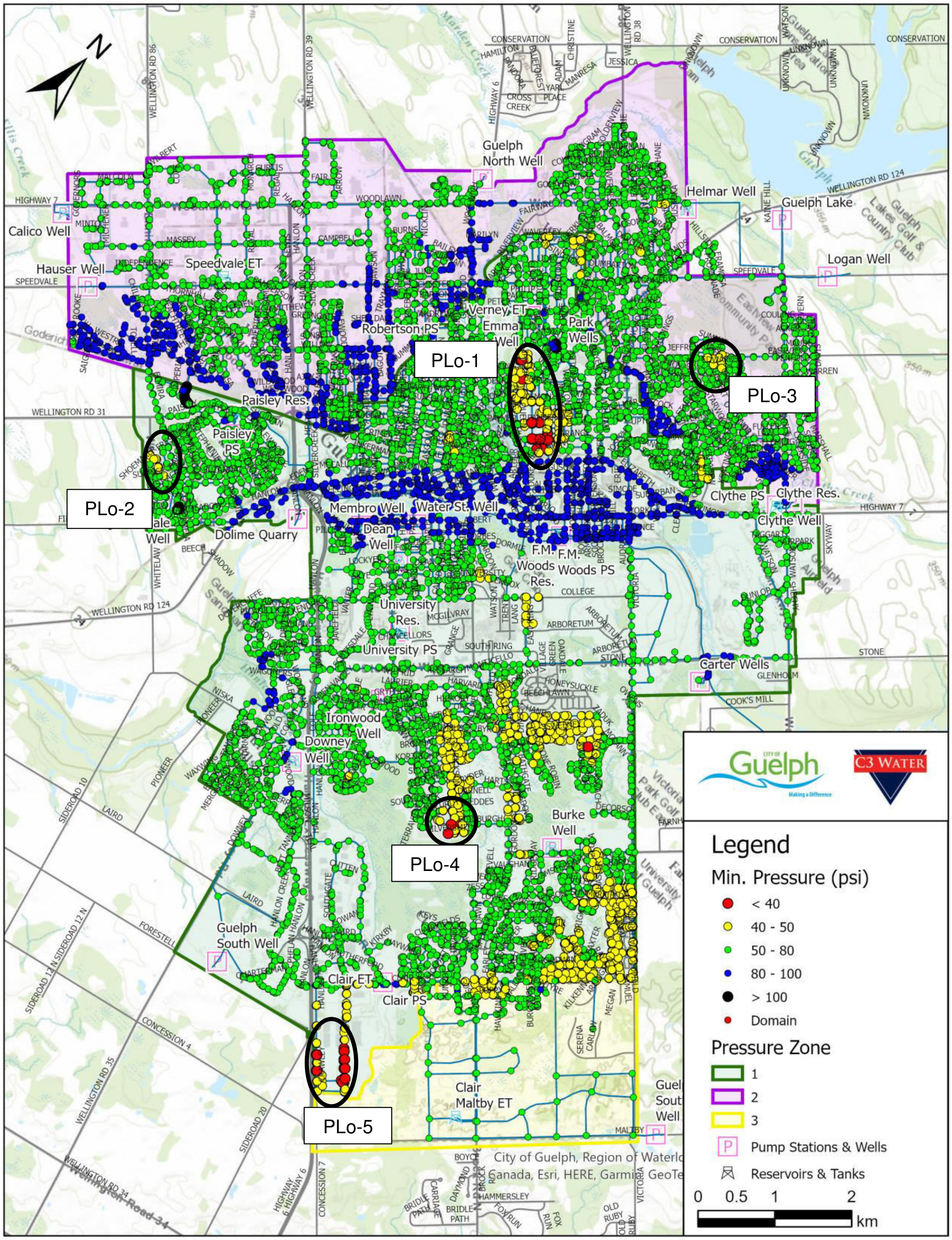


Figure 3-8 2051+ MDD – Minimum Pressure

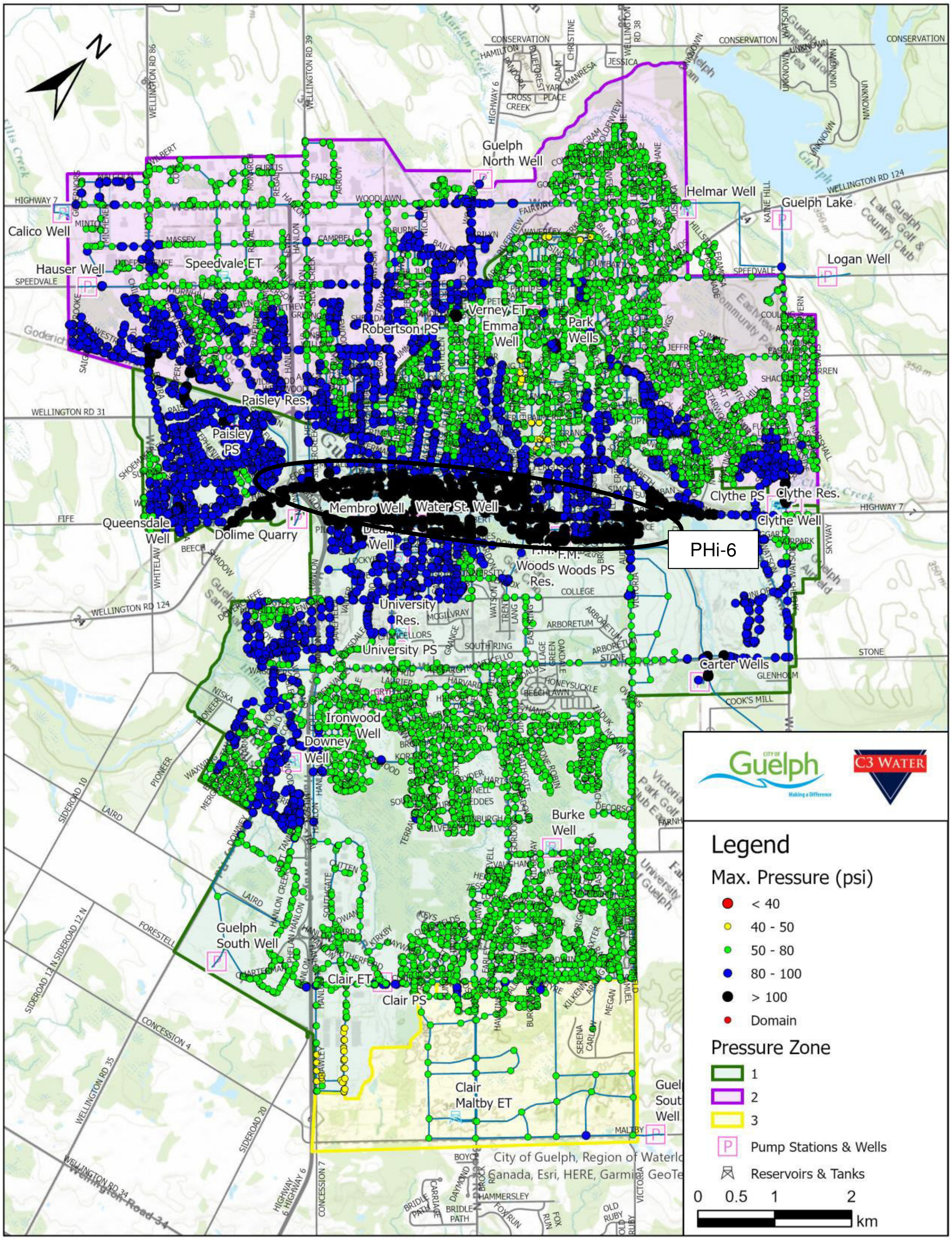


Figure 3-9 2051+ ADD – Maximum Pressure

3.4.3 Watermain Capacity Analysis

The existing watermain diameters are presented in Figure 3-10 below.

The maximum headloss gradient results under existing MDD conditions for watermains with a diameter of 300mm or greater are presented in Figure 3-11 and Figure 3-12 for existing and 2051+ conditions, respectively. The watermains that exceeded the maximum criteria of 2m/km are summarized in Table 3-18 below.

Under 2051+ conditions, a number of pipes exceeded 2m/km headloss in the downtown area. Additionally, a number of key feedermains from Woods PS to the Clair ET exceeded 2m/km including the College Street river crossing, Victoria Street and Hanlon Parkway. Limited watermain capacity towards the South end of Zone 1 resulted in the Clair ET being unable to fill under 2051+ peak hour conditions, as was discussed in Section 3.4.2.

The maximum velocity results under existing and 2051+ MDD conditions are presented in Figure 3-13 and Figure 3-14 below, respectively. The velocity did not exceed the criteria of 3m/s under either existing or 2051+ conditions. As expected, the water system is stressed attempting to deliver 2051+ MDD due to the limitations of the existing watermains.

Table 3-18 Watermains Exceeding Headloss Criteria

#	Location	Diameter (mm)	Material	Year of Installation	Exceeded HL Criteria?	
					Existing	**2051+**
H-1	Dunlop	300	Ductile Iron	1992	✓	✓
H-2	Watson from York to Taggart	300	Ductile Iron	1987	✓	
H-3(1)	Metcalfe from Park Wells to Eramosa	300	Cast Iron	1951	✓	✓
H-4	Exhibition from London to Verney	300	Cast Iron	1954		✓
H-5	Huron from York to Winston	300	Cast Iron	1902		✓
H-6	Membro Well to College and Hanlon	300	Cast Iron/Concrete	1968		✓
H-7	Eramosa River Crossing to College	400	Ductile Iron	1970		✓
H-8	Neeve and Arthur from Woods PS to Speed River	300	Cast Iron	1902		✓
H-9	Norfolk from Paisley to Norwich	300	PVC	2009		✓
H-10	Wyndham from Speed River to Quebec	300	Cast Iron/PVC	1903/2009		✓
H-11	Hanlon from Kortright to Southgate	400	Ductile Iron	1978		✓
H-12	Edinburgh from University PS to Stone	300	Cast Iron	1970		✓
H-13	Stone from Gordon to Village Green	400	Concrete	1971		✓
H-14	Victoria from Stone to Macalister	400	PVC	2000		✓
H-15	Clair from Clair PS to Gordon	400	Ductile Iron	1986		✓
H-16	Woodslawn from Woolwich to Country Club	300	Cast Iron	1967		✓

(1) City staff have advised that upgrades are currently underway On Metcalfe from Terry to Eramosa. These upgrades are not yet reflected in the water model.

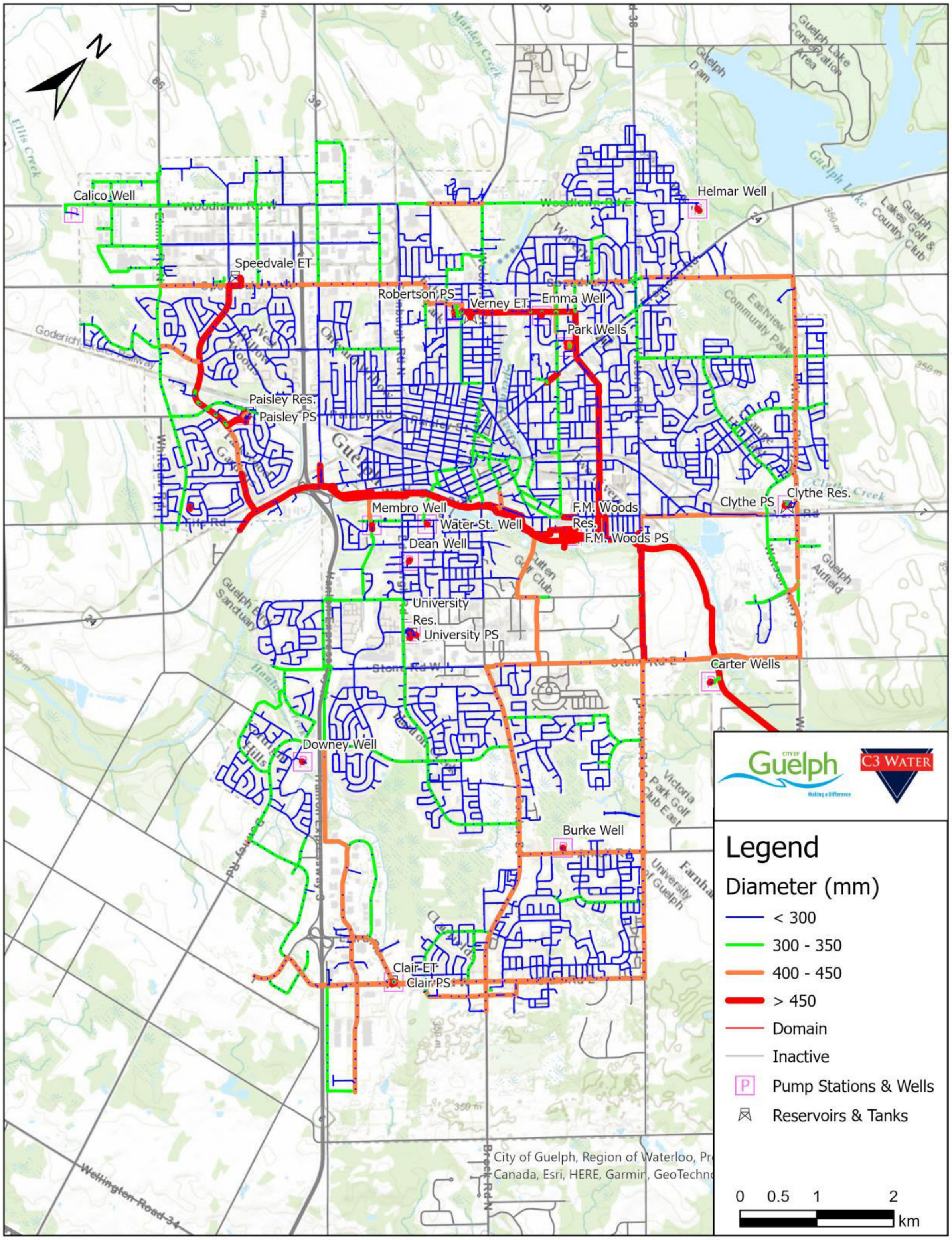


Figure 3-10 Existing Pipe Diameters

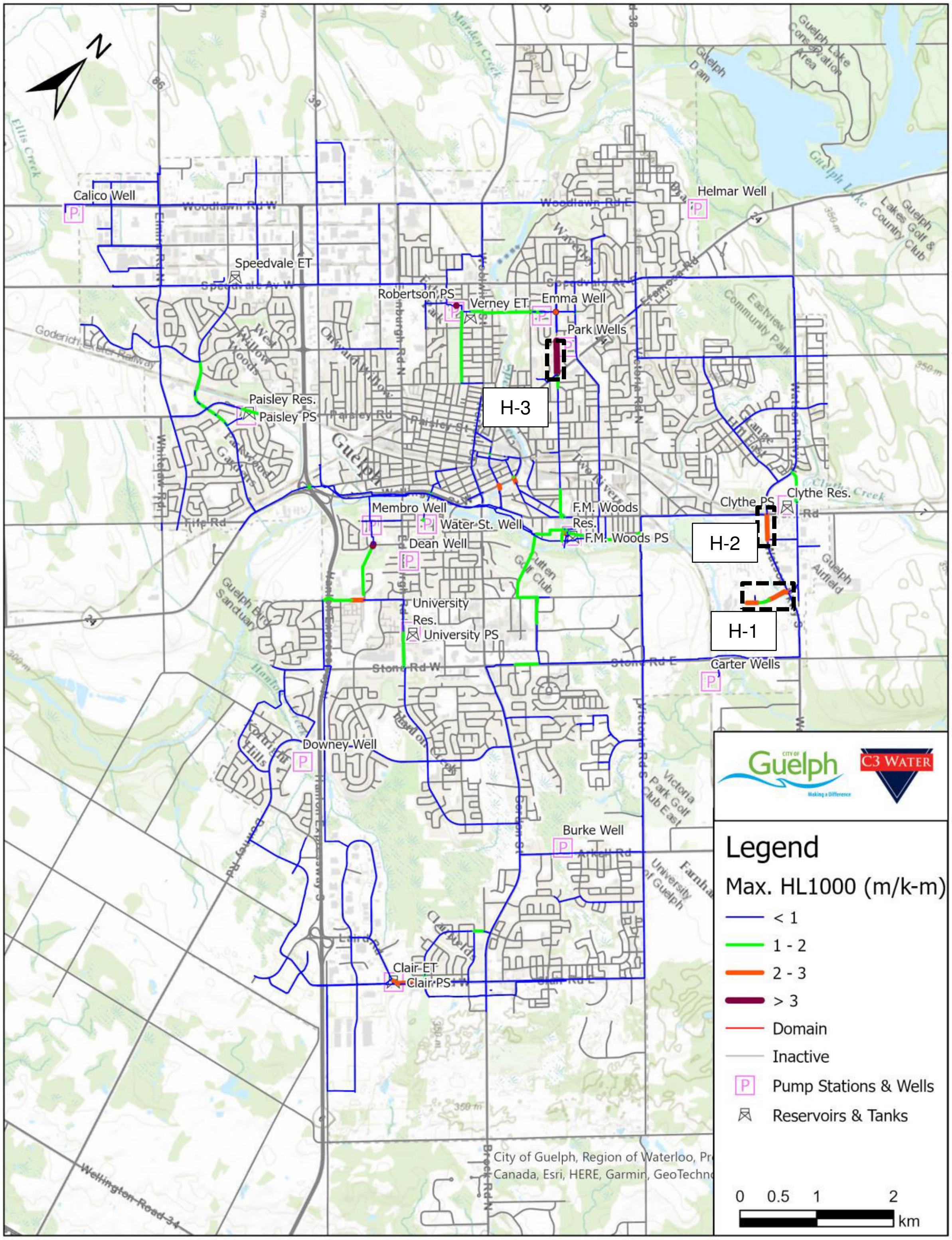


Figure 3-11 Existing MDD – Maximum Headloss (300mm and greater)

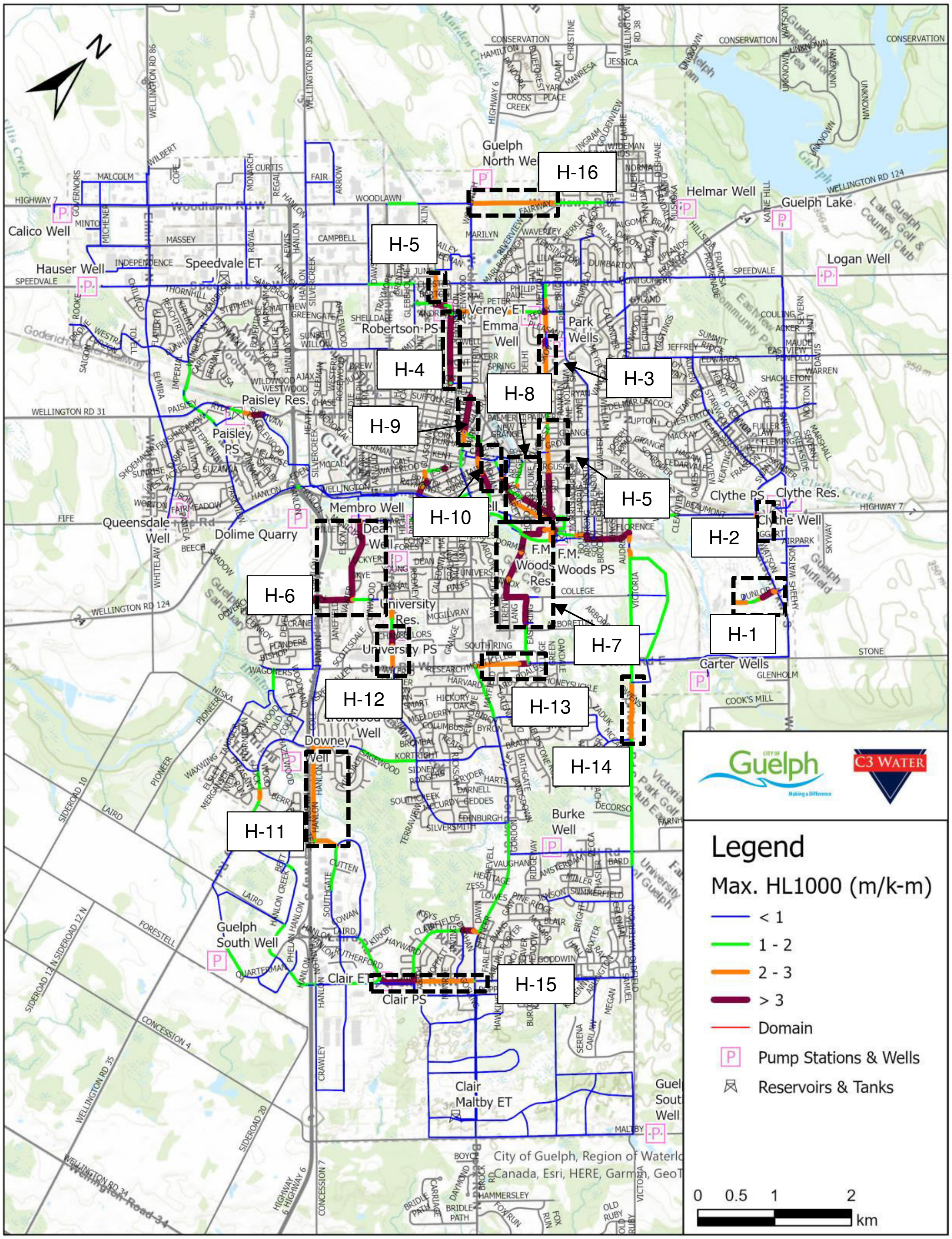


Figure 3-12 2051+ MDD – Maximum Headloss (300mm and greater)

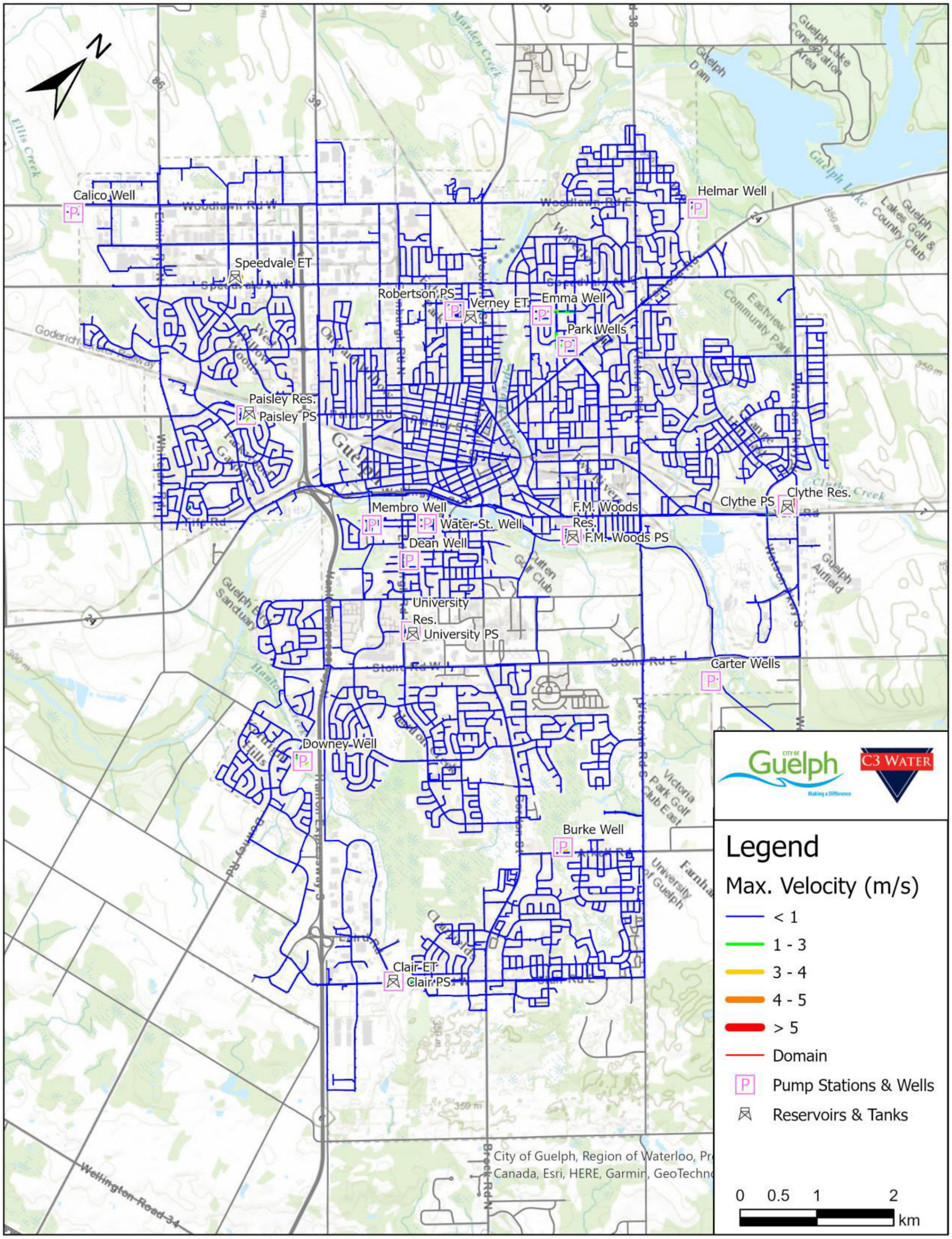


Figure 3-13 Existing MDD – Maximum Velocity

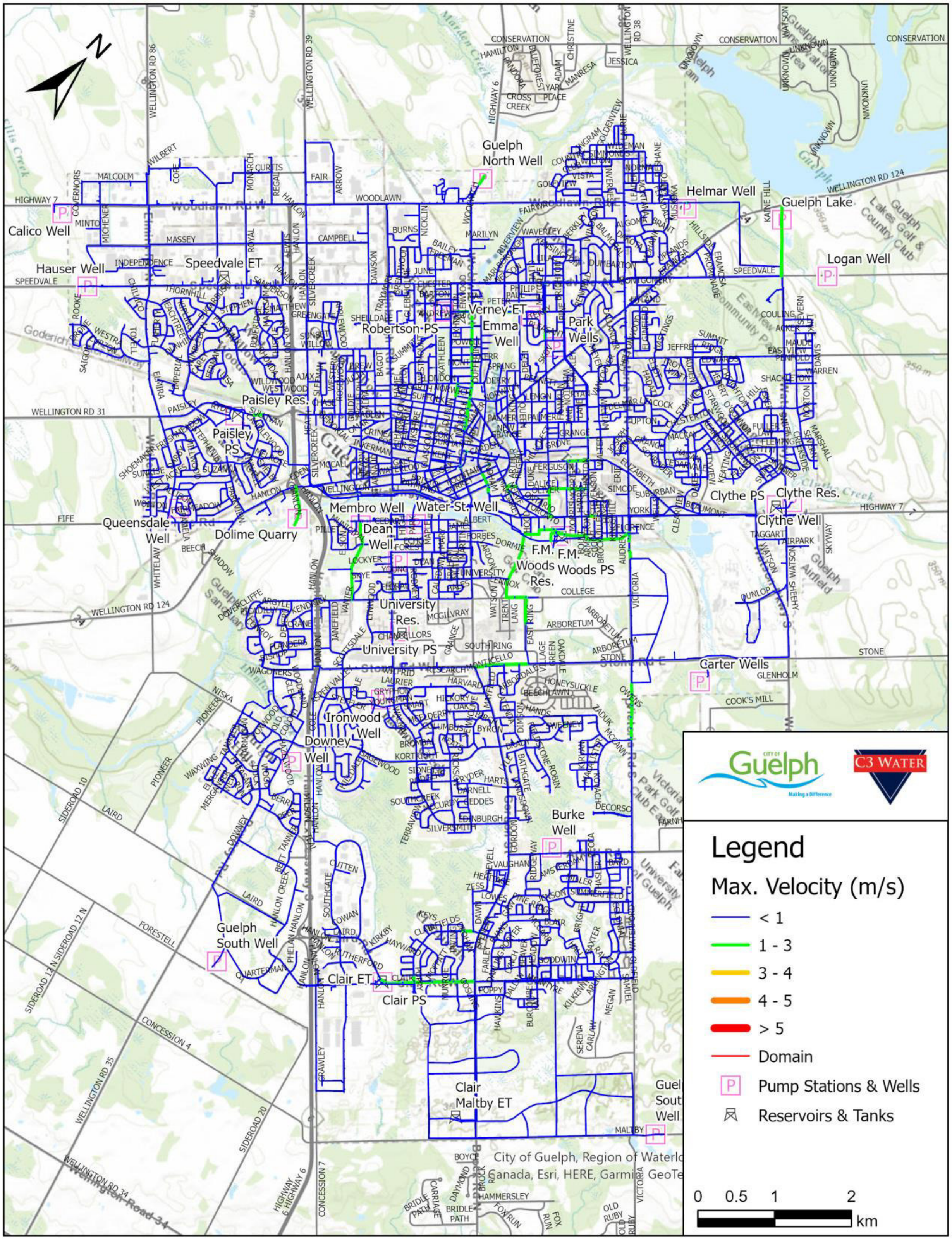


Figure 3-14 2051+ MDD – Maximum Velocity

3.4.4 Fire Flow Capacity Analysis

The system-wide fire flow results under MDD conditions are presented in Figure 3-15 and Figure 3-16 below for existing and 2051+ conditions, respectively. The fire flow results predicted by the model are representative of the amount of water available in a watermain and not the extent of flow available from a hydrant. Several hydrants may need to be operated to provide the desired fire flows. Areas with available fire flow of less than 30 L/s were flagged and are summarized in Table 3-19 below. Areas of low fire flow were generally a result of localized constraints due to old, small diameter, cast iron watermains with high roughness. There was not found to be a significant difference in available fire flow under existing and 2051+ conditions.

Table 3-19 Fire Flow Areas of Concern

#	Location	Notes
F-1	South of Speed River, north of College Ave.	Low fire flow capacity on 100mm pipes of cast iron material > 50 years old.
F-2	East of Speed River, west of Metcalfe St, north of Grange Street, south of Waverley Dr.	Low fire flow capacity on 100-150mm pipes of cast iron material > 50 years old.
F-3	West of Speed River, east of Edinburgh Rd, north of London St, south of Speedvale Ave.	Low fire flow capacity on 100-150mm pipes of cast iron material > 50 years old.
F-4	North of Waterloo Ave., south of Willow Rd, east of Silvercreek Pkwy, west of Edinburgh R	Low fire flow capacity on 100-150mm pipes of cast iron material > 50 years old.

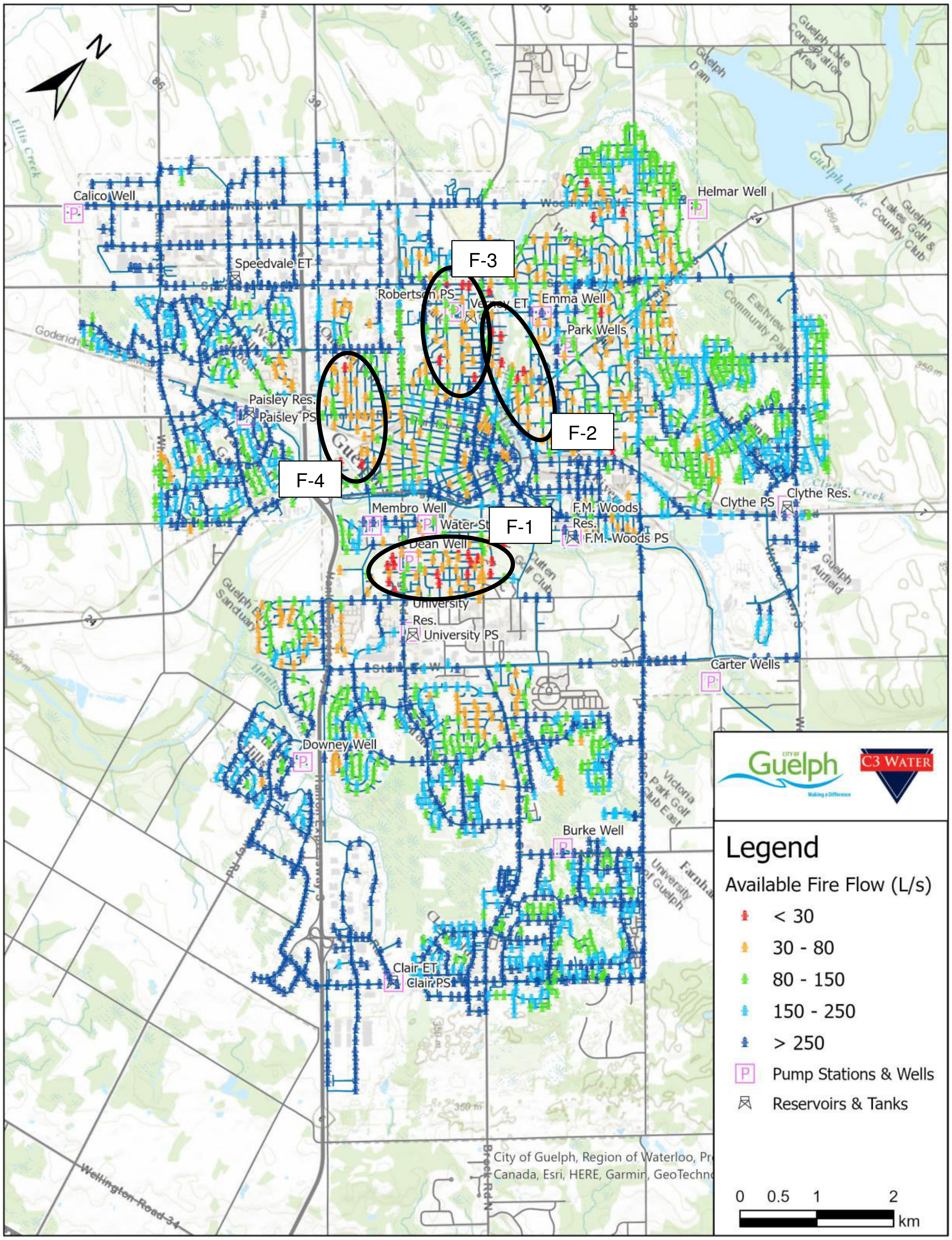


Figure 3-15 Existing MDD – Available Fire Flow

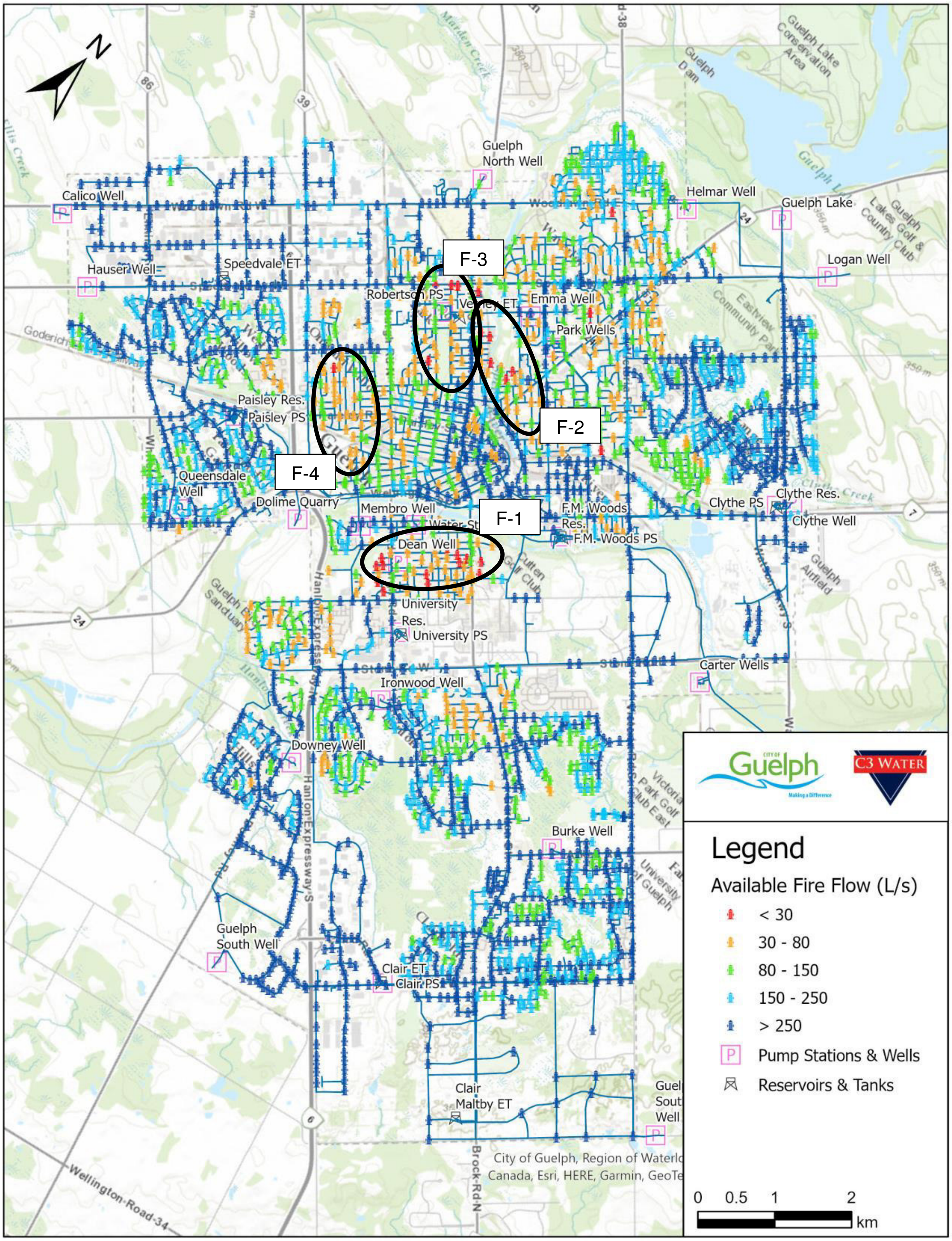


Figure 3-16 2051+ MDD – Available Fire Flow

3.4.5 Storage Hydraulic Analysis

3.4.5.1 Storage Hydraulic Modeling Results Existing Conditions Analysis

Modelling results for the ETs and in-ground reservoirs under existing MDD conditions are presented in Figure 3-17 and Figure 3-18 below. In Zone 1, the Verney ET primarily remained above 80% full while the Clair ET dropped to a minimum of approximately 45% full during the evening peak demand. Under existing conditions, the Woods pump controls were based on the Verney ET level. It can be seen that while Woods PS maintained the level at Verney ET, the Clair ET was draining during high demand periods to supply the south end of Zone 1. The Woods Reservoir level dropped in the afternoon as the pump station ran at a higher flow than what was supplied via the Arkell Aqueduct but remained above 60% full. The University Reservoir generally remained above 90% full.

In Zone 2, the Speedvale ET remained above 85% full. The Paisley reservoir remained above 75% full. The Clythe Reservoir was found to drain below 50% full in the afternoon as the Clythe PS ran at a higher flow to supply the peak demands on the east end of Zone 2. During the low night time demands, the Clythe Reservoir began to refill.

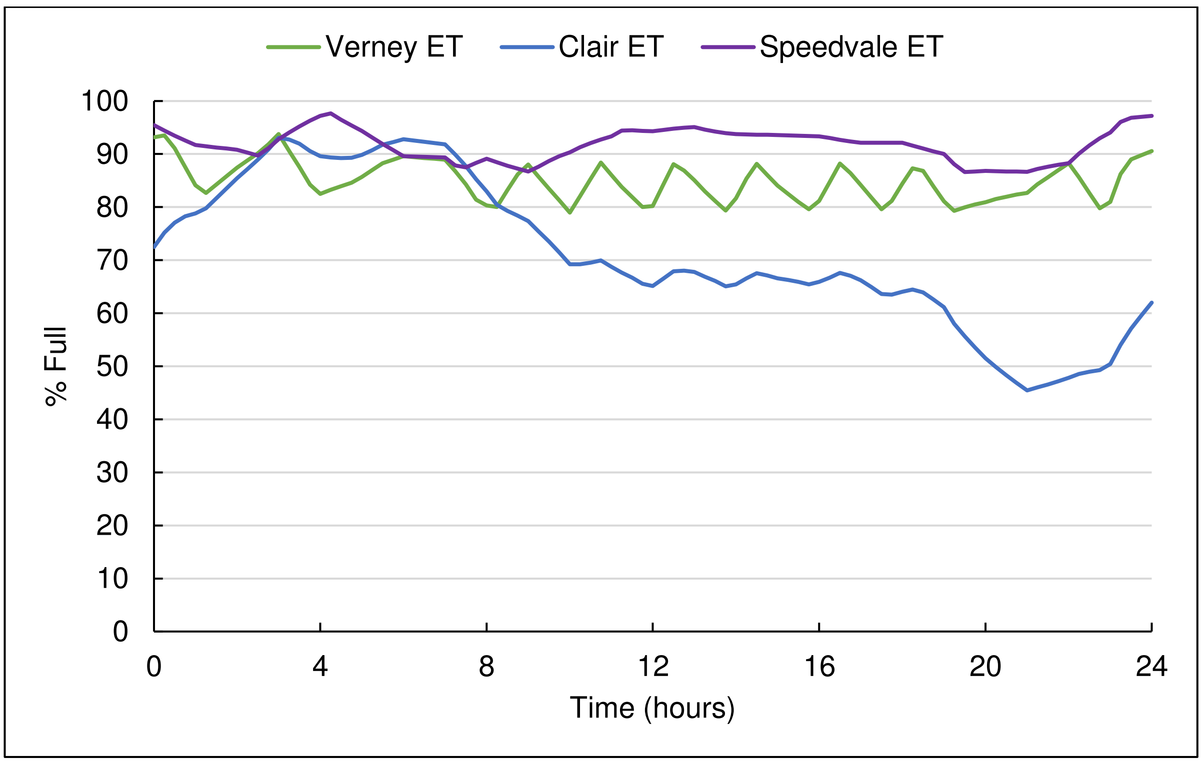


Figure 3-17 Existing MDD – ET Levels

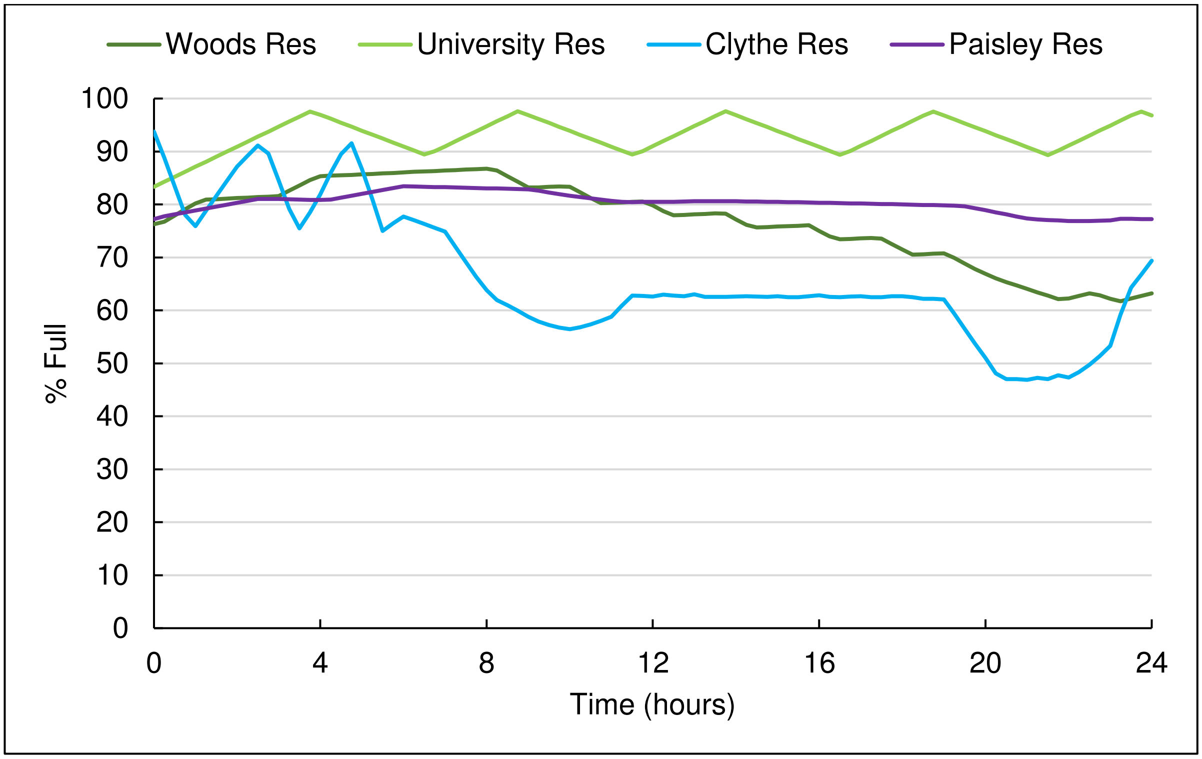


Figure 3-18 Existing MDD – Reservoir Levels

3.4.5.2 Storage Hydraulic Modeling Results 2051+ Conditions Analysis

Modelling results for the ETs and in-ground reservoirs under 2051+ MDD conditions are presented in Figure 3-19 and Figure 3-20 below. In Zone 1, the Verney ET overflowed during periods of the day, while the Clair ET dropped to a minimum level of approximately 45% full during the evening peak demand. Under 2051+ conditions, the Woods pump controls were based on the Clair ET level. It can be seen that Woods PS struggled to push waster into the south end of Zone 1 to maintain the Clair ET level. The Woods Reservoir drained to 30% full throughout the day as the PS ramped up to try to fill the Clair ET and the Arkell Aqueduct sources were not able to keep up. The University Reservoir generally remained above 90% full. Although it was discussed in Section 3.3 that there is expected to be sufficient supply and storage under 2051+ conditions, limitations were noted in moving water appropriately around the system. The north end of Zone 1 was being oversupplied, causing the Verney ET to overflow creating water loss, while the south end of Zone 1 was undersupplied, causing the Clair ET to drain.

In Zone 2, the Speedvale ET remained above 85% full. The Paisley reservoir remained above 75% full. The Clythe Reservoir remained above 90% full. Although the demands on the east side of Zone 2 were higher compared to existing conditions, the Clythe Reservoir was maintained at a higher level due to the increased reservoir capacity, inflow from the Clythe Well and the additional supply sources on the east side of Zone 2 such as the Guelph Lake and the Logan well, which reduced the reliance on the Clythe PS.

In Zone 3, the Clair PS was able to maintain the Clair Maltby ET level above 80% full.

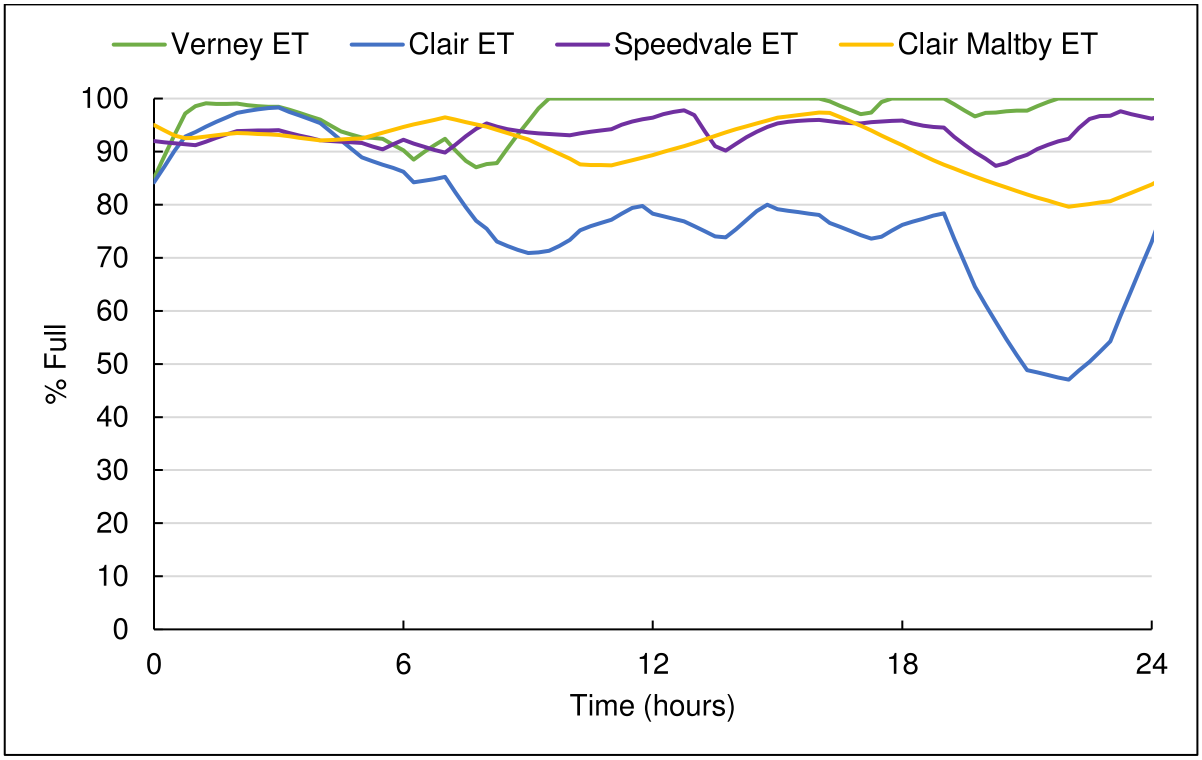


Figure 3-19 2051+ MDD – ET Levels

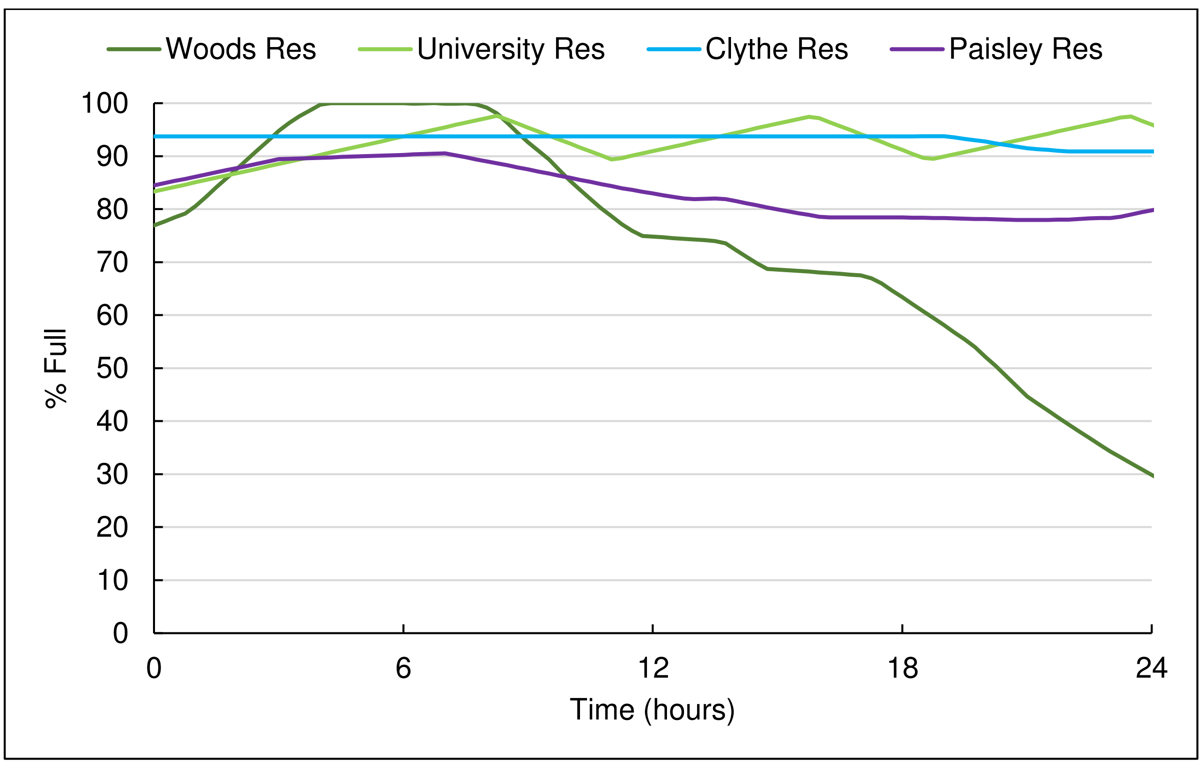


Figure 3-20 2051+ MDD – Reservoir Levels

3.4.6 Pumping Station Hydraulic Analysis

3.4.6.1 Pump Station Hydraulic Modeling Results Existing Conditions Analysis

The discharge flow results at the major PSs under existing MDD conditions are presented in Figure 3-21 and Figure 3-22 below. Woods PS provided the majority of the system's supply. Woods PS ran at a higher flow in the afternoon as demands increased, to maintain the Verney ET level but was well below the firm capacity of 1050 L/s.

In Zone 2, the Robertson PS ran at a consistent flow with Paisley and Clythe PSs ramping up as needed to supply peak demands and maintain the Speedvale ET. During peak hour, Clythe PS ran at a maximum flow of 100 L/s, to maintain the discharge pressure setpoint. This maximum flow at Clythe PS was only 25 L/s below the existing firm capacity. Paisley PS operated significantly below its firm capacity. Only one of the three (3) pumps operated at Robertson PS.

To supply the Zone 3 demand, Clair PS operated significantly below its firm capacity, at a maximum flow of 7 L/s.

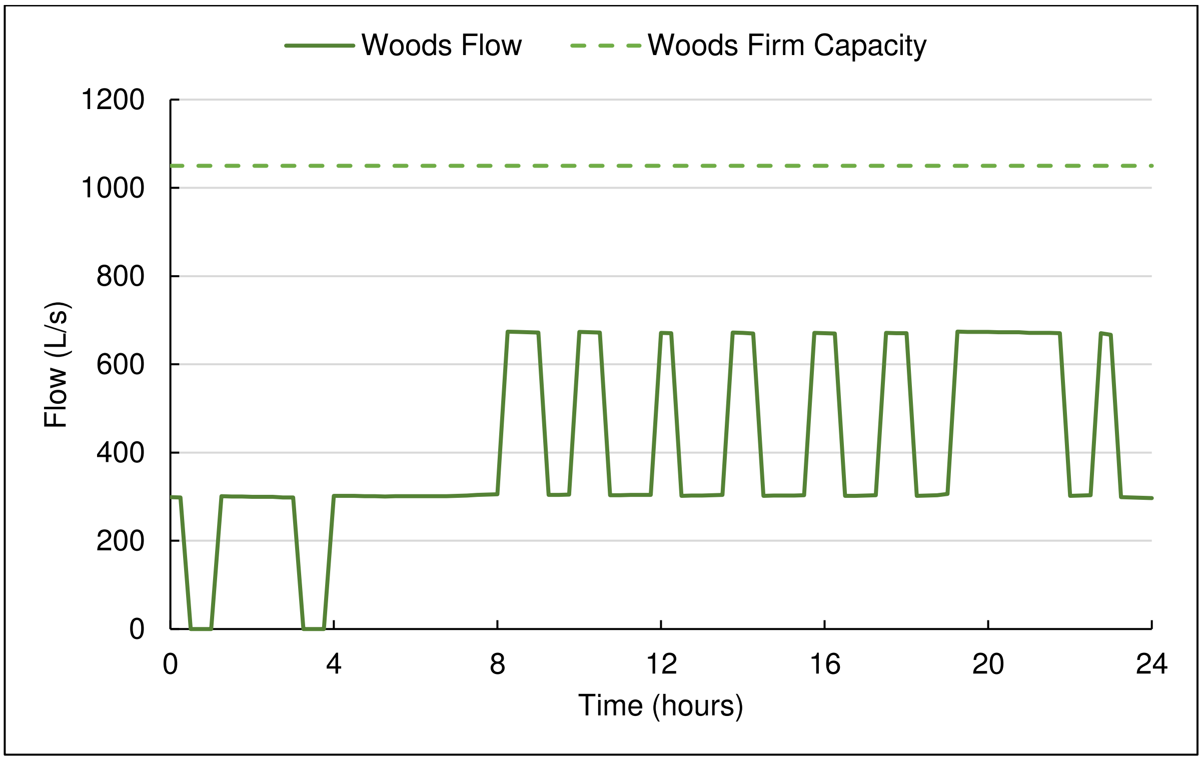


Figure 3-21 Existing MDD – Woods Pump Station Flow

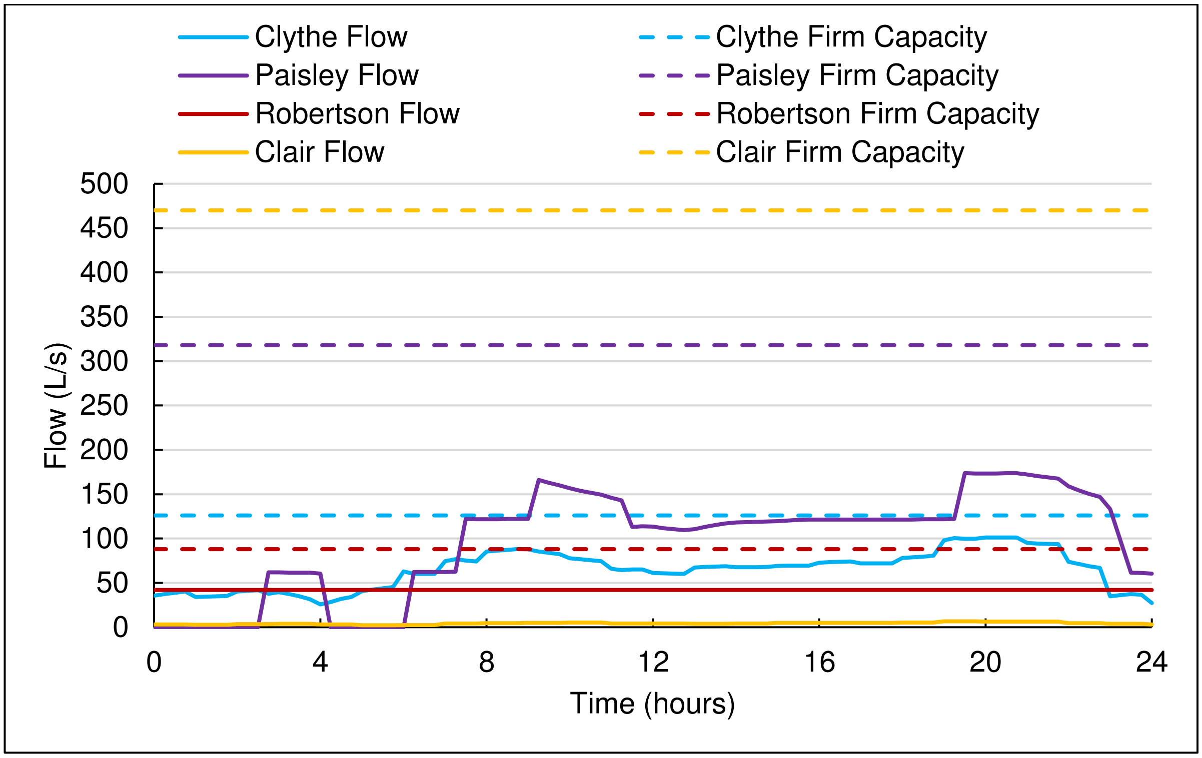


Figure 3-22 Existing MDD – Zone 2&3 Pump Stations Flow

3.4.6.2 Pump Station Hydraulic Modeling Results 2051+ Conditions Analysis

The discharge flow results at the major PSs under 2051+ MDD conditions are presented in Figure 3-23 and Figure 3-24 below. Woods PS again provided the majority of the system's supply. The Woods PS discharge flow slightly exceeded the firm capacity during high demands when trying to maintain the Clair ET level. The average discharge flow at the Woods PS was 574 L/s which exceeds the total available supply into the Arkell Aqueduct (Arkell Wells, Glen Collector, Lower Collector and Carter Wells).

In Zone 2, Verney and Paisley PSs ran at similar flow rates, each averaging approximately 60 L/s. Clythe ran at a lower average flow of 16 L/s and had a maximum flow of 38 L/s. Clythe PS ran at a lower flow rate under 2051+ conditions than what was seen under existing conditions due to the additional supply sources on the east side of Zone 2 such as the Guelph Lake and the Logan Well. All Zone 2 PSs ran well below their future firm capacities.

In Zone 3, the Clair BPS operated at an average flow of approximately 80 L/s to maintain the Clair Maltby ET level.

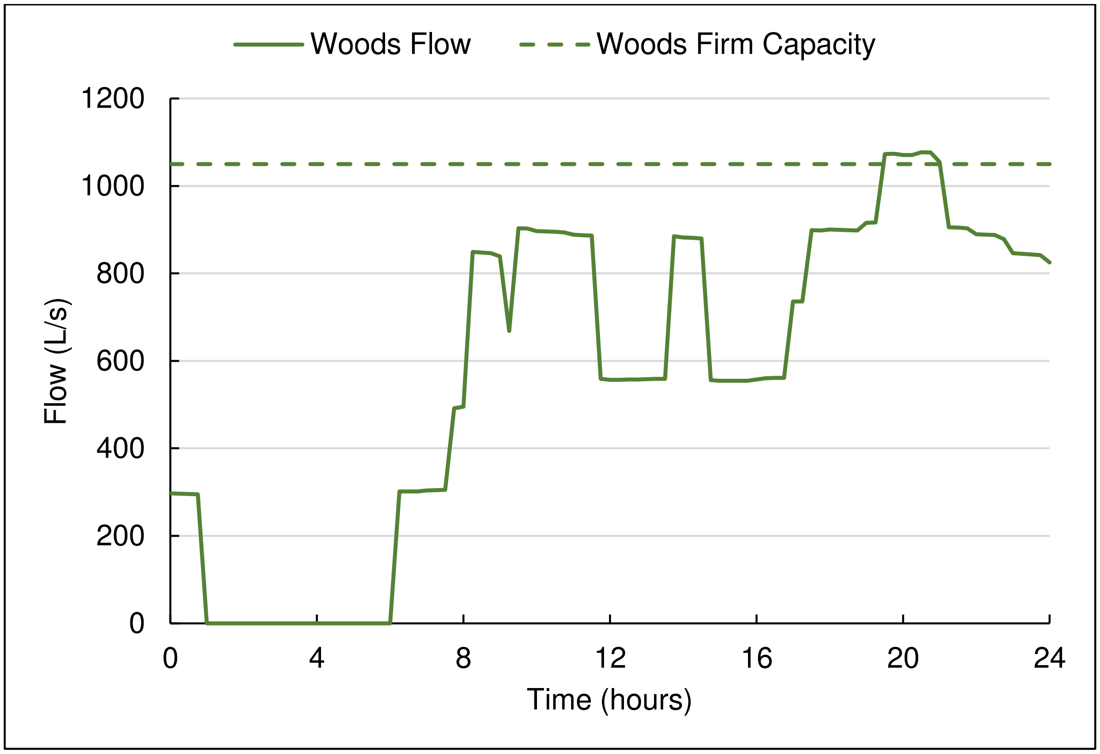


Figure 3-23 2051+ MDD – Woods Pump Station Flow

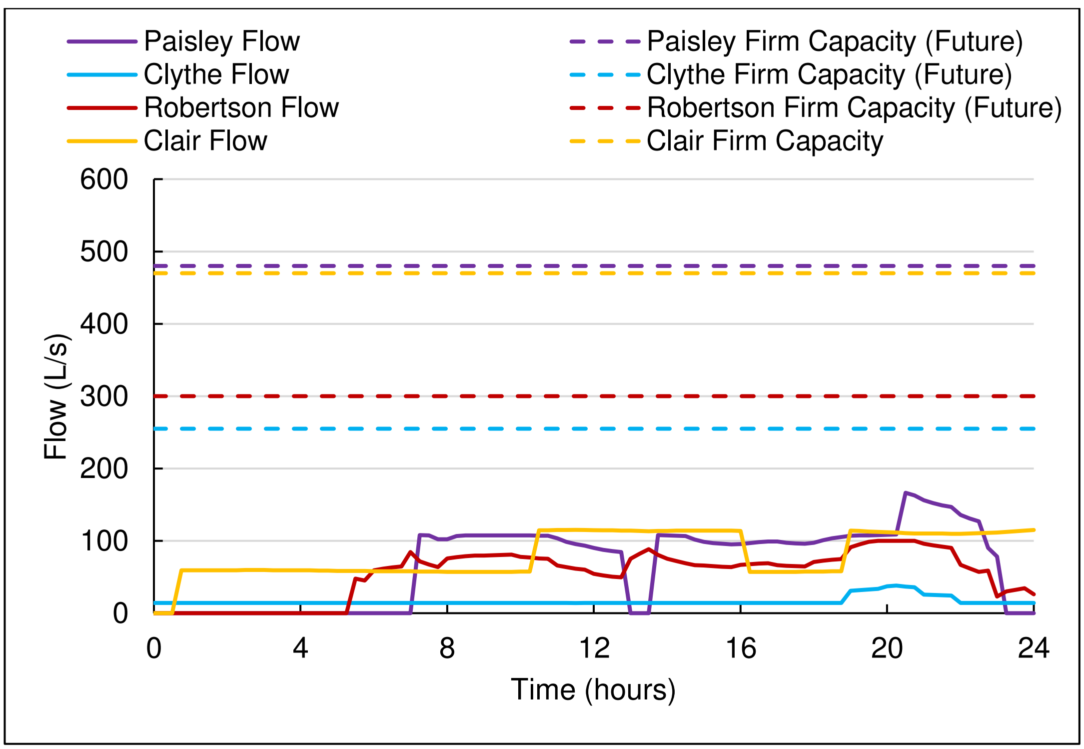


Figure 3-24 Existing MDD – Zone 2&3 Pump Stations Flow

4.0 WASTEWATER SYSTEM PERFORMANCE

4.1 Wastewater System Criteria

The wastewater system was analyzed for both the existing and future conditions in consideration of the findings and recommendations presented in the *Design Criteria, LOS and Sensitivity Analysis Technical Memorandum (C3/Stantec, December 2020).* This document includes a review of the City's existing guidelines in consideration of other regional and provincial documents and provides recommendations for the analysis presented in this document. In general, the recommendation is to align with the MECP and then Regional guidelines. The reader is encouraged to review this document to aid in the interpretation of the findings presented herein. A selection of some of the most pertinent recommendations are summarized in Table 4-1 below.

Table 4-1 Wastewater System Criteria

Wastewater System Criteria	
Flow Generation & Sewer Sizing	• Maintain 80% full capacity approach for new sewers.
	• Calculate design flows using population or maximum capacity.
HGL Considerations	• HGLs should be below basement elevations (assumed to be 1.8m (~6ft) below TOG elevations).
Dry Weather Performance	• The HGL should be within the sewer obvert.
	• Diurnal patterns to be considered.
Wet Weather Performance	• Shorter (3hr or 6hr) events to be used for peak flow assessment
	• Longer (24hr) events to be used for assessments where volumes are of interest.
	• A design event distribution is to be selected and applied.
	• Recommended to use the 25-year event for the initial assessment.
	• Target HGLs (1.8m) to be used in result interpretation.
Pumping Stations	• Establish if the modelled flow coming into the PS surpasses the existing firm capacity. Complete an upstream HGL assessment if warranted. Complete an overall wastewater collection system analysis while conveying the peak WWF downstream.
Siphons	• Establish the minimum velocity under DWF conditions
Water Resource Recovery Center (WRRC)	• The instantaneous peak WWF should be considered.
	• The range of operating levels at the WRRC require consideration.

4.2 Wastewater System Assessment

4.2.1 Approach

The wastewater system assessment was completed for the dry weather flow (DWF) and wet weather flow (WWF) conditions using the hydraulic model developed. For the WWF condition, the 25-year 3-hour Chicago event is used. Both the existing and future scenarios were considered.

For the existing scenario assessments completed, the calibrated model was used as is for DWF. For the WWF condition, the model was loaded with the targeted design event (25-year 3-hour Chicago).

For the future scenario assessments completed, the model was used as described in Section 2.2.

In addition to the wastewater system assessment, the wastewater pumping stations, siphons, and WRRC were also assessed for performance. A failure analysis was also completed. The results of the wastewater system assessment are presented in the following sections.

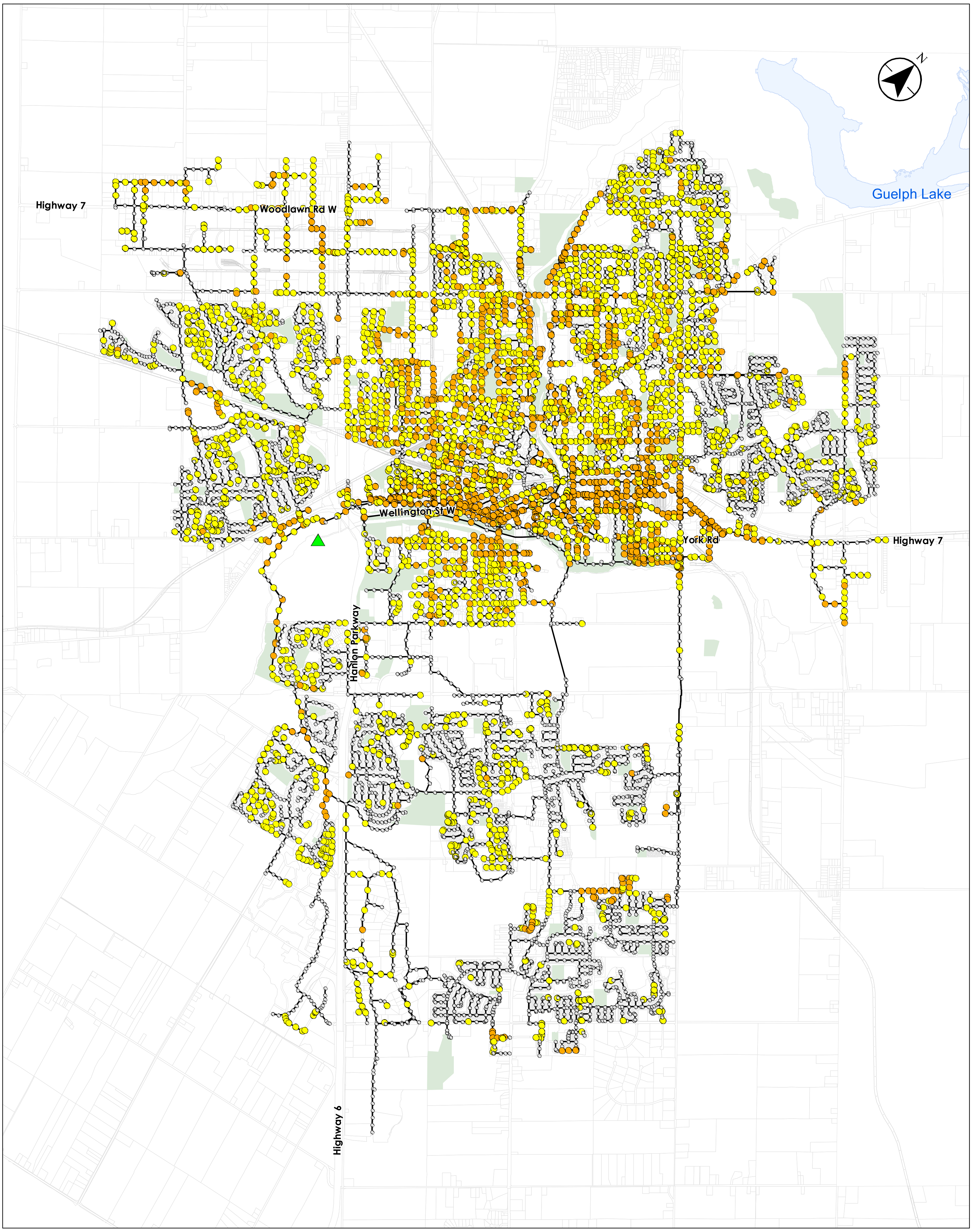
4.2.1.1 Consideration of Existing Secondary Plans

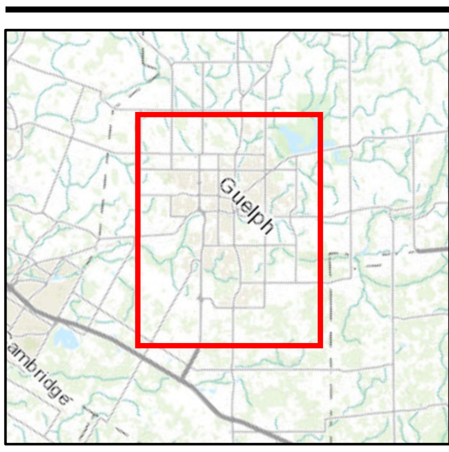
It should be noted that there are existing secondary plans for the Clair Maltby Secondary Plan (CMSP) area and the Guelph Innovation District (GID) area that carried out detailed wastewater analysis, including population estimates, DWF and WWF flow generation rates, and connection points to the existing wastewater network. These secondary plans were deemed to supersede the population growth shapefile and growth in these areas have been excluded, with the published DWF and WWF rates and connection points from the secondary plans adopted instead.

4.2.1.2 Observations - Maintenance Hole Elevations

Currently in the model used for the assessment, there are 3,047 (36.8%) maintenance holes (MHs) with between 2.8m and 1.8 m of cover, and 1,263 (15.3%) MHs with less than 1.8m of cover. Figure 4-1 shows the distribution of MHs with less than 2.8m of cover and less than 1.8m of cover. We recommend that the accuracy of this distribution be confirmed. The results presented in this document will illustrate that the relative “shallowness” of the City’s wastewater collection system is problematic from a performance perspective.

\\Ca0207-ppfss01\workgroup\1656\active\165640298\preliminary\3_Report\TM3a - existing system assessment\figures\fig_4.1_Existing_Conditions_minimum_cover_103.mxd Revised: 2022-09-06 By: MaAnderson





Legend

Minimum Cover Above Obvert

- < 1.8 m (1,263)
- 1.8 m - 2.8 m (3,047)
- > 2.8 m (3,965)
- Sanitary Network
- Wastewater Treatment
- Railway
- Property

0 1,000 2,000 Meters

1:50,000 (At Original document size of 11x17)

Project Location 165640298

City of Guelph

Client/Project

CITY OF GUELPH WATER AND WASTEWATER SERVICING MASTER PLAN – TM1 BACKGROUND REVIEW

Figure No.

4.1

Title

Wastewater Network - Minimum Cover Above Pipe Obvert

Notes

1. Coordinate System: NAD 1983 UTM Zone 17N
2. Base features produced under license with the Ontario Ministry of Natural Resources and Forestry © Queen's Printer for Ontario, 2018.

Sources: Esri, HERE, Garmin, Intermap, increment P Corp., GEBCO, USGS, FAO, NPS, NRCAN, GeoBase, IGN, Kadaster NL, Ordnance Survey, Esri Japan, METI, Esri China (Hong Kong), swisstopo, © OpenStreetMap contributors, and the GIS User Community

4.2.1.3 Implementing Recent Capital Works Upgrades

A list of recent capital work upgrades and the associated drawings were provided by the City during the model calibration process. The upgrades that would have influenced the calibration were included, and those upgrades that have been or will be constructed outside the calibration period are considered in the existing and future conditions assessment. Since the model calibration was completed, there are an additional 4 capital projects that have been added including:

- Project WW-I-19: York Trunk and Paisley-Clythe Watermain Phase 2A (Contract 2-1606)
- Project WW-I-4: Paisley Feedermain (Contract 2-1812)
- Project WW-I-2: Stevenson: Eramosa to Bennet (Contract 2-2006)
- York Road Phase 2 Reconstruction (Contract 20-004)

The drawings were reviewed and cross-referenced with the model and updated accordingly.

4.2.1.4 WWF RTK Parameter Updates

As noted in section 5.2 of the Model Update, Field Testing and Calibration TM2b, the wet weather response was modeled using the RTK method, with each flow monitoring site being assigned a unit hydrograph composed of 3 sets of 3 parameters. Each set of parameters represents a type of wet weather response: short-term, medium-term, and long-term response. The results of the calibration indicated that the use of the short-term RDII response parameters (R1, T1, K1) was adequate for calibrating the flow monitor recorded response as the calibration was limited by the magnitude of available rainfall events. This resulted in all flow monitors being calibrated for the Short-Term response, five flow monitors being calibrated for the Medium-Term Response, and only 3 flow monitors being calibrated for the Long-Term response. In addition to the flow monitors calibrated by Stantec, there were an additional 11 flow monitors calibrated as part of the other input models, that also lacked some of the RTK values for some of each of the Short-Term, Medium-Term, and Long-Term responses. The calibrated parameters for each flow monitoring site, including those calibrated for models / assessments used for model development are shown in Table 4-2.

Table 4-2 WWF Calibrated RTK parameters (unedited)

Flow Monitor	Total R	Short Term			Medium Term			Long Term		
		R	T	K	R	T	K	R	T	K
FM01	0.001 = 0.1%	0.001	0.10	0.5	-	0.60	2.0	-	1.00	2.0
FM02	0.0 = 0%	-	0.10	2.0	-	0.50	2.0	-	1.00	2.0
FM03	0.003 = 0.3%	0.001	0.10	1.0	0.001	0.80	2.0	0.001	5.00	5.0
FM04	0.004 = 0.4%	0.002	0.10	2.0	0.001	1.00	2.0	0.001	4.00	2.0
FM05	0.002 = 0.2%	0.001	0.40	0.4	0.001	1.00	4.0	-	2.00	2.0
FM06	0.011 = 1.1%	0.008	0.15	1.0	0.003	0.70	4.0	-	-	-

Flow Monitor	Total R	Short Term			Medium Term			Long Term		
		R	T	K	R	T	K	R	T	K
FM06a	0.0036 = 0.36%	0.002	0.15	1.0	0.002	0.50	4.0	-	-	-
FM07	0.012 = 1.2%	0.010	0.15	1.0	0.002	0.50	3.0	-	-	-
FM07a	0.008 = 0.8%	0.005	0.18	1.0	0.003	0.50	3.0	-	-	-
FM08	0.006 = 0.6%	0.004	0.18	0.9	0.002	0.25	1.7	-	-	-
FM09	0.002 = 0.2%	0.001	0.40	0.4	0.001	1.00	4.0	-	2.00	2.0
FM10	0.020 = 2.0%	0.007	0.05	0.4	0.006	0.50	1.5	0.007	3.00	2.5
FM11	0.005 = 0.5%	0.005	0.10	0.4	-	4.00	2.5	-	12.50	5.0
FM12	0.006 = 0.6%	0.003	0.05	1.5	0.003	5.00	2.5	-	12.50	5.0
FM13	0.002 = 0.2%	0.001	0.10	0.4	0.001	1.25	1.5	-	12.50	5.0
FM14a	0.033 = 3.3%	0.033	1.25	1.5	-	4.00	2.5	-	12.50	5.0
FM14b	0.004 = 0.4%	0.004	1.25	1.5	-	4.00	2.5	-	12.50	5.0
FM15	0.002 = 0.2%	0.002	1.25	1.5	-	4.00	2.5	-	12.50	5.0
FM16	0.017 = 1.7%	0.004	0.20	1.5	0.006	4.00	2.5	0.007	12.50	5.0
FM17	0.004 = 0.4%	0.004	0.50	1.5	-	4.00	2.5	-	12.50	5.0
FM18	0.001 = 0.1%	0.001	1.25	1.5	-	4.00	2.5	-	12.50	5.0
FM19	0.007 = 0.7%	0.004	0.50	1.5	0.002	4.00	2.5	0.001	12.50	5.0
FM20	0.004 = 0.4%	0.004	0.50	1.5	-	4.00	2.5	-	12.50	5.0
FM21	0.003 = 0.3%	0.003	0.50	1.5	-	4.00	2.5	-	12.50	5.0

Flow Monitor	Total R	Short Term			Medium Term			Long Term		
		R	T	K	R	T	K	R	T	K
Average	***0.0106 = 1.06%***	***0.005***	***0.40***	***1.1***	***0.002***	***2.25***	***2.6***	***0.003***	***8.84***	***4.1***

The concern with leaving these parameters with no values, particularly when assessing future conditions, is that a zero R value will result in no RDII response and therefore no difference between the DWF and WWF conditions for those sites. It is assumed that over time the collection system becomes "leakier" and the RDII rates will increase. To provide a more robust assessment of the WWF characteristics and ensure a WWF response across the system, the RTK parameters were updated to replace all missing values/0's with the average values for each R, T, and K values presented in Table 4-2. The updated RTK values for each flow monitor location in the model are shown on Table 4-3.

Table 4-3 WWF Calibrated RTK parameters (edited)

Flow Monitor	Total R (original)	Total R	Short Term			Medium Term			Long Term		
			R	T	K	R	T	K	R	T	K
FM01	0.002 = 0.2%	0.0068 = 0.68%	0.001	0.10	0.5	0.002	0.60	2.0	0.003	1.00	2.0
FM02	0.011 = 1.1%	0.0106 = 1.06%	0.005	0.10	2.0	0.002	0.50	2.0	0.003	1.00	2.0
FM03	0.0036 = 0.36%	0.003 = 0.3%	0.001	0.10	1.0	0.001	0.80	2.0	0.001	5.00	5.0
FM04	0.012 = 1.2%	0.004 = 0.4%	0.002	0.10	2.0	0.001	1.00	2.0	0.001	4.00	2.0
FM05	0.008 = 0.8%	0.0054 = 0.54%	0.001	0.40	0.4	0.001	1.00	4.0	0.003	2.00	2.0
FM06	0.006 = 0.6%	0.0144 = 1.44%	0.008	0.15	1.0	0.003	0.70	4.0	0.003	8.84	4.1
FM06a	0.002 = 0.2%	0.007 = 0.7%	0.002	0.15	1.0	0.002	0.50	4.0	0.003	8.84	4.1
FM07	0.020 = 2.0%	0.0154 = 1.54%	0.010	0.15	1.0	0.002	0.50	3.0	0.003	8.84	4.1

Flow Monitor	Total R (original)	Total R	Short Term			Medium Term			Long Term		
			R	T	K	R	T	K	R	T	K
FM07a	0.005 = 0.5%	0.0114 = 1.14%	0.005	0.18	1.0	0.003	0.50	3.0	0.003	8.84	4.1
FM08	0.006 = 0.6%	0.0094 = 0.94%	0.004	0.18	0.9	0.002	0.25	1.7	0.003	8.84	4.1
FM09	0.002 = 0.2%	0.0054 = 0.54%	0.001	0.40	0.4	0.001	1.00	4.0	0.003	2.00	2.0
FM10	0.033 = 3.3%	0.02 = 2%	0.007	0.05	0.4	0.006	0.50	1.5	0.007	3.00	2.5
FM11	0.004 = 0.4%	0.0108 = 1.08%	0.005	0.10	0.4	0.002	4.00	2.5	0.003	12.50	5.0
FM12	0.002 = 0.2%	0.0094 = 0.94%	0.003	0.05	1.5	0.003	5.00	2.5	0.003	12.50	5.0
FM13	0.017 = 1.7%	0.0054 = 0.54%	0.001	0.10	0.4	0.001	1.25	1.5	0.003	12.50	5.0
FM14a	0.004 = 0.4%	0.0388 = 3.88%	0.033	1.25	1.5	0.002	4.00	2.5	0.003	12.50	5.0
FM14b	0.001 = 0.1%	0.0098 = 0.98%	0.004	1.25	1.5	0.002	4.00	2.5	0.003	12.50	5.0
FM15	0.007 = 0.7%	0.0078 = 0.78%	0.002	1.25	1.5	0.002	4.00	2.5	0.003	12.50	5.0
FM16	0.004 = 0.4%	0.017 = 1.7%	0.004	0.20	1.5	0.006	4.00	2.5	0.007	12.50	5.0
FM17	0.003 = 0.3%	0.0098 = 0.98%	0.004	0.50	1.5	0.002	4.00	2.5	0.003	12.50	5.0

Flow Monitor	Total R (original)	Total R	Short Term			Medium Term			Long Term		
			R	T	K	R	T	K	R	T	K
FM18	0.002 = 0.2%	0.0068 = 0.68%	0.001	1.25	1.5	0.002	4.00	2.5	0.003	12.50	5.0
FM19	0.011 = 1.1%	0.007 = 0.7%	0.004	0.50	1.5	0.002	4.00	2.5	0.001	12.50	5.0
FM20	0.0036 = 0.36%	0.0098 = 0.98%	0.004	0.50	1.5	0.002	4.00	2.5	0.003	12.50	5.0
FM21	0.012 = 1.2%	0.0088 = 0.88%	0.003	0.50	1.5	0.002	4.00	2.5	0.003	12.50	5.0

The unit hydrographs were updated in the model using the values presented in Table 4-3. With the preceding revisions complete, the model was ready for the existing conditions assessment. The results of the assessment are described in the following section.

4.2.2 Wastewater System Failure Analysis

The intent of the failure analysis was to simulate “breaks” at 20 locations throughout the City’s wastewater collection system. These locations were chosen based on multiple factors such as, risk to adjacent property, proximity to essential infrastructure, trunk diameter, and size of upstream catchment. The analysis was completed using the developed model under the 25-year 3-hour distribution WWF event. Sewer failure was simulated by limiting the flow in the selected sewer to 0 L/s. This approach allows an HGL analysis on the implications of these failures on the upstream collection system.

The analysis considers both the increased risk of basement and surface flooding. The results for each failure location are compared to the “non-failure” WWF scenario (as shown on the series of figures provided), and are discussed in further detail below.

4.2.2.1 Site 1

Site 1 is located in the northwestern part of the city, near Lisa Lane in Margaret Greene Park, see Figure 4-3. The chosen gravity main sewer, Pipe ID 5176, has a pipe diameter of 900 mm, with an upstream catchment area of 313.9 ha. This location was chosen due to the sewer diameter and its upstream service area.

Figure 4-3 demonstrates the location of the surcharged MHs and sewers operating over capacity. Under existing conditions, there is no surcharging observed upstream of Pipe ID 5176. In the event of a sewer failure, the upstream influence is surcharging is observed in 28 sewers and 21 MHs.

4.2.2.2 Site 2

Site 2 is located in the northeastern part of the city, on Arthur St N, see Figure 4-4. The chosen gravity main sewer, Pipe ID 6869, has a pipe diameter of 825 mm, with an upstream catchment area of 685.2 ha. This location was chosen due to the sewer diameter and its upstream service area.

Figure 4-4 demonstrates the location of the surcharged MHs and sewers operating over capacity. Under existing conditions, there is no surcharging observed upstream of Pipe ID 6869. In the event of a sewer failure, the upstream influence is surcharging is observed in 26 sewers and 11 MHs.

4.2.2.3 Site 3

Site 3 is located in the western part of the city, on Wellington St S, near the WRRC, see Figure 4-5. The chosen gravity main sewer, Pipe ID 9608, has a pipe diameter of 1200 mm, with an upstream catchment area of 3475.8 ha. This location was chosen due to the sewer diameter and its upstream service area.

Figure 4-5 demonstrates the location of the surcharged MHs and sewers operating over capacity. Under existing conditions, there is no surcharging observed upstream of Pipe ID 9608. In the event of a sewer failure, the upstream influence is surcharging is observed in 10 sewers and 11 MHs.

4.2.2.4 Site 4

Site 4 is located in the central part of the city, along the Hanlon Expressway, see Figure 4-6. The chosen siphon, Pipe ID 109413, has a pipe diameter of 1650 mm, with an upstream catchment area of 4028.0 ha. This location was chosen due to the sewer diameter and its upstream service area.

Figure 4-6 demonstrates the location of the surcharged MHs and sewers operating over capacity. Under existing conditions, there is no surcharging observed upstream of Pipe ID 109413. In the event of a sewer failure, the upstream influence is surcharging is observed in 115 sewers and 63 MHs.

4.2.2.5 Site 5

Site 5 is located in the central part of the city, near Silvercreek Pkwy S and Wellington St W, see Figure 4-7. The chosen gravity main sewer, Pipe ID 5915, has a pipe diameter of 1050 mm, with an upstream catchment area of 1294.3 ha. This location was chosen due to the sewer diameter and its upstream service area.

Figure 4-7 demonstrates the location of the surcharged MHs and sewers operating over capacity. Under existing conditions, there is no surcharging observed upstream of Pipe ID 5915. In the event of a sewer failure, the upstream influence is surcharging is observed in 40 sewers and 53 MHs.

4.2.2.6 Site 6

Site 6 is located in the central part of the city, near Waterloo Ave and Wellington St W, see Figure 4-8. The chosen gravity main sewer, Pipe ID 5923, has a pipe diameter of 1350 mm, with an upstream catchment area of 2734.1 ha. This location was chosen due to the sewer diameter, its upstream service area, and is a new trunk with known surcharging.

Figure 4-8 demonstrates the location of the surcharged MHs and sewers operating over capacity. Under existing conditions, there no surcharging observed upstream of Pipe ID 5923. In the event of a sewer failure, the upstream influence is surcharging is observed in 66 sewers and 26 MHs.

4.2.2.7 Site 7

Site 7 is located in the central part of the city, on Wellington St W, near Edinburgh Rd, see Figure 4-9. The chosen gravity main sewer, Pipe ID 9096, has a pipe diameter of 1200 mm, with an upstream catchment area of 2588.8 ha. This location was chosen because it isolates the new 1200 mm trunk, Kortright SPS flows into this trunk at Victoria Rd, significant elevation changes and likely an overflow location.

Figure 4-9 demonstrates the location of the surcharged MHs and sewers operating over capacity. Under existing conditions, there is no surcharging observed upstream of Pipe ID 9096. In the event of a sewer failure, the upstream influence is surcharging is observed in 74 sewers and 32 MHs.

4.2.2.8 Site 8

Site 8 is located in the northeastern part of the city, on Cross St, near Arthur St, see Figure 4-10. The chosen gravity main sewer, Pipe ID 9361, has a pipe diameter of 825 mm, with an upstream catchment area of 797.1 ha. This location was chosen due to the sewer diameter, its upstream service area and is known for surcharging.

Figure 4-10 demonstrates the location of the surcharged MHs and sewers operating over capacity. Under existing conditions, surcharging is observed in 6 sewers and 7 MHs upstream of Pipe ID 9361. In the event of a sewer failure, the upstream influence is surcharging is observed in 29 sewers and 44 MHs.

4.2.2.9 Site 9

Site 9 is located in the central part of the city, crosses the Eramosa River, near Eramosa River Park, see Figure 4-11. The chosen siphon, Pipe ID 109409, has a pipe diameter of 500 mm, with an upstream catchment area of 202.2 ha. This location was chosen because it is a siphon and its proximity to the University.

Figure 4-11 demonstrates the location of the surcharged MHs and sewers operating over capacity. Under existing conditions, there is no surcharging observed upstream of Pipe ID 109409. In the event of a sewer failure, the upstream influence is surcharging is observed in 4 sewers and 5 MHs.

4.2.2.10 Site 10

Site 10 is located is the eastern part of the city, on Morris St, near York Rd, see Figure 4-12. The chosen gravity main sewer, Pipe ID 9009, has a pipe diameter of 300 mm, with an upstream catchment area of 21.8 ha. This location was chosen due to the number of MHs with lower rim elevations upstream.

Figure 4-12 demonstrates the location of the surcharged MHs and sewers operating over capacity. Under existing conditions, there is no surcharging observed upstream of Pipe ID 9009. In the event of a sewer failure, the upstream influence is surcharging is observed in 40 sewers and 35 MHs.

4.2.2.11 Site 11

Site 11 is located in the eastern part of the city, on Stevenson St S near Beverley St, see Figure 4-13. The chosen gravity main sewer, Pipe ID 8406, has a pipe diameter of 600 mm, with an upstream catchment area of 434.1 ha. This location was chosen due to the sewer diameter and 48 upstream MHs with lower rim elevations.

Figure 4-13 demonstrates the location of the surcharged MHs and sewers operating over capacity. Under existing conditions, there is no surcharging observed upstream of Pipe ID 8406. In the event of a sewer failure, the upstream influence is surcharging is observed in 42 sewers and 40 MHs.

4.2.2.12 Site 12

Site 12 is located in the western part of the city, parallel to Speed River, near Dovercliffe Rd, see Figure 4-14. The chosen gravity main sewer, Pipe ID 2903, has a pipe diameter of 1200 mm, with an upstream catchment area of 2542.9 ha. This location was chosen due to the sewer diameter and its upstream service area.

Figure 4-14 demonstrates the location of the surcharged MHs and sewers operating over capacity. Under existing conditions, there is no surcharging observed upstream of Pipe ID 2902. In the event of a sewer failure, the upstream influence is surcharging is observed in 11 sewers and 10 MHs.

4.2.2.13 Site 13

Site 13 is located in the western part of the city, near College Ave W, see Figure 4-15. The chosen gravity main sewer, Pipe ID 1685, has a pipe diameter of 900 mm, with an upstream catchment area of 2081.7 ha. This location was chosen due to the sewer diameter and its upstream service area.

Figure 4-15 demonstrates the location of the surcharged MHs and sewers operating over capacity. Under existing conditions, there no observed surcharging upstream of Pipe ID 1685. In the event of a sewer failure, the upstream influence is surcharging is observed in 17 sewers and 7 MHs.

4.2.2.14 Site 14

Site 14 is located in the western part of the city, in Kortright Waterfowl Park, see Figure 4-16. The chosen gravity main sewer, Pipe ID 1700, has a pipe diameter of 250 mm, with an upstream catchment area of 51.3 ha. This location was chosen due to being the downstream sewer of a historical failure on a siphon.

Figure 4-16 demonstrates the location of the surcharged MHs and sewers operating over capacity. Under existing conditions, there is no surcharging observed upstream of Pipe ID 1700. In the event of a sewer failure, the upstream influence is surcharging is observed in 8 sewers and 6 MHs.

4.2.2.15 Site 15

Site 15 is located in the western part of the city, near Flanders Rd and Hanlon Rd, see Figure 4-17. The chosen gravity main sewer, Pipe ID 1807, has a pipe diameter of 600mm, with an upstream catchment area of 401.8 ha. This location was chosen due to the sewer diameter and its upstream service area.

Figure 4-17 demonstrates the location of the surcharged MHs and sewers operating over capacity. Under existing conditions, there is no surcharging observed upstream of Pipe ID 1807. In the event of a sewer failure, the upstream influence is surcharging is observed in 61 sewers and 32 MHs.

4.2.2.16 Site 16

Site 16 is located in the southern part of the city, on Jean Anderson Cres, see Figure 4-18. The chosen gravity main sewer, Pipe ID PWOPRMHD0005455.1, has a pipe diameter of 600 mm, with an upstream catchment area of 778.4 ha. This location was chosen due to the sewer diameter and its upstream service area.

Figure 4-18 demonstrates the location of the surcharged MHs and sewers operating over capacity. Under existing conditions, there is no surcharging observed upstream of Pipe ID PWOPRMHD0005455.1. In the event of a sewer failure, the upstream influence is surcharging is observed in 232 sewers and 53 MHs.

4.2.2.17 Site 17

Site 17 is located in the eastern part of the city, near Victoria Rd S and Boult Ave, see Figure 4-19. The chosen gravity main sewer, Pipe ID 4662, has a pipe diameter of 900 mm, with an upstream catchment area of 652.3 ha. This location was chosen due to the sewer diameter and its upstream service area.

Figure 4-19 demonstrates the location of the surcharged MHs and sewers operating over capacity. Under existing conditions, there is no surcharging observed upstream of Pipe ID 4662. In the event of a sewer failure, the upstream influence is surcharging is observed in 30 sewers and 23 MHs.

4.2.2.18 Site 18

Site 18 is located in the southwestern part of the city, at Woodland Glen Dr and Downey Rd, see Figure 4-20. The chosen gravity main sewer, Pipe ID 2553, has a pipe diameter of 900 mm, with an upstream catchment area of 1993.7 ha. This location was chosen due to the wanting to see the impact of a failure to Kortright Hills and Preservation Park area.

Figure 4-20 demonstrates the location of the surcharged MHs and sewers operating over capacity. Under existing conditions, there is no surcharging observed upstream of Pipe ID 2553. In the event of a sewer failure, the upstream influence is surcharging is observed in 25 sewers and 13 MHs.

4.2.2.19 Site 19

Site 19 is located in the northwestern part of the city, at the rail corridor crossing of Alma St N and Alma St S, see Figure 4-21. The chosen gravity main sewer, Pipe ID 3393, has a pipe diameter of 900 mm, with an upstream catchment area of 659.0 ha. This location was chosen due to the sewer diameter and the crossing of the rail corridor.

Figure 4-21 demonstrates the location of the surcharged MHs and sewers operating over capacity. Under existing conditions, there is no surcharging observed upstream of Pipe ID 3393. In the event of a sewer failure, the upstream influence is surcharging is observed in 108 sewers and 106 MHs.

4.2.2.20 Site 20

Site 20 is located is the western part of the city, at the WRRC, see Figure 4-22. The chosen gravity main sewer, Pipe ID 9614, has a pipe diameter of 1200 mm, with an upstream catchment area of 7423.8 ha. This location was chosen to analyze the potential impacts of the WRRC operating at very high levels (simulating an overall failure condition).

Figure 4-22 demonstrates the location of the surcharged MHs and sewers operating over capacity. Under existing conditions, there is no surcharging observed upstream of Pipe ID 3393. In the event of a WRRC failure, surcharging is observed in 14 sewers and 14 MHs.

Table 4-4 below is a summary of the system failure analysis results.

Table 4-4 Failure Analysis Results Summary

System Failure Location	Flood Volume (m^3)	Upstream Service Area (ha)	Site Selection Rationale
Site 1	3,872	314	900 mm diameter sewer with multiple upstream shallow MHs
Site 2	11,638	685	825 mm diameter sewer near hospital and along Speed River
Site 3	59,958	3,476	1200 mm diameter sewer near WWTP and Speed River
Site 4	100,074	4,028	1650 mm diameter sewer near Hanlon Expressway
Site 5	31,080	1,294	1050 mm diameter sewer, multiple upstream shallow MHs
Site 6	65,667	2,734	1350 mm diameter sewer, multiple upstream shallow MHs
Site 7	62,015	2,589	1200 mm diameter sewer, near Speed River. Downstream of Kortright SPS
Site 8	14,888	797	825 mm sewer near Speed River. Known surcharging upstream.
Site 9	11,869	202	Siphon downstram of University
Site 10	23	22	300 mm diameter sewer, multiple upstream shallow MHs
Site 11	3,725	434	600 mm diameter sewer, multiple upstream shallow MHs
Site 12	36,366	2,543	1200 mm diameter sewer, upstream of triple siphon.
Site 13	25,607	2,082	900 mm diameter sewer with multiple upstream shallow MHs
Site 14	872	51	250 mm siphon, historical failure location
Site 15	6,826	402	600 mm diameter sewer, multiple upstream shallow MHs
Site 16	9,550	778	600 mm diameter sewer, multiple upstream shallow MHs
Site 17	18,932	652	900 mm diameter sewer, upstream industrial/private pumping stations
Site 18	24,207	1,994	900 mm diameter sewer, downstram of Kortright Hills / Preservation Park Area
Site 19	11,420	659	900 mm diameter sewer, crossing rail corridor
Site 20	314,707	-	WWTP, fixed water level

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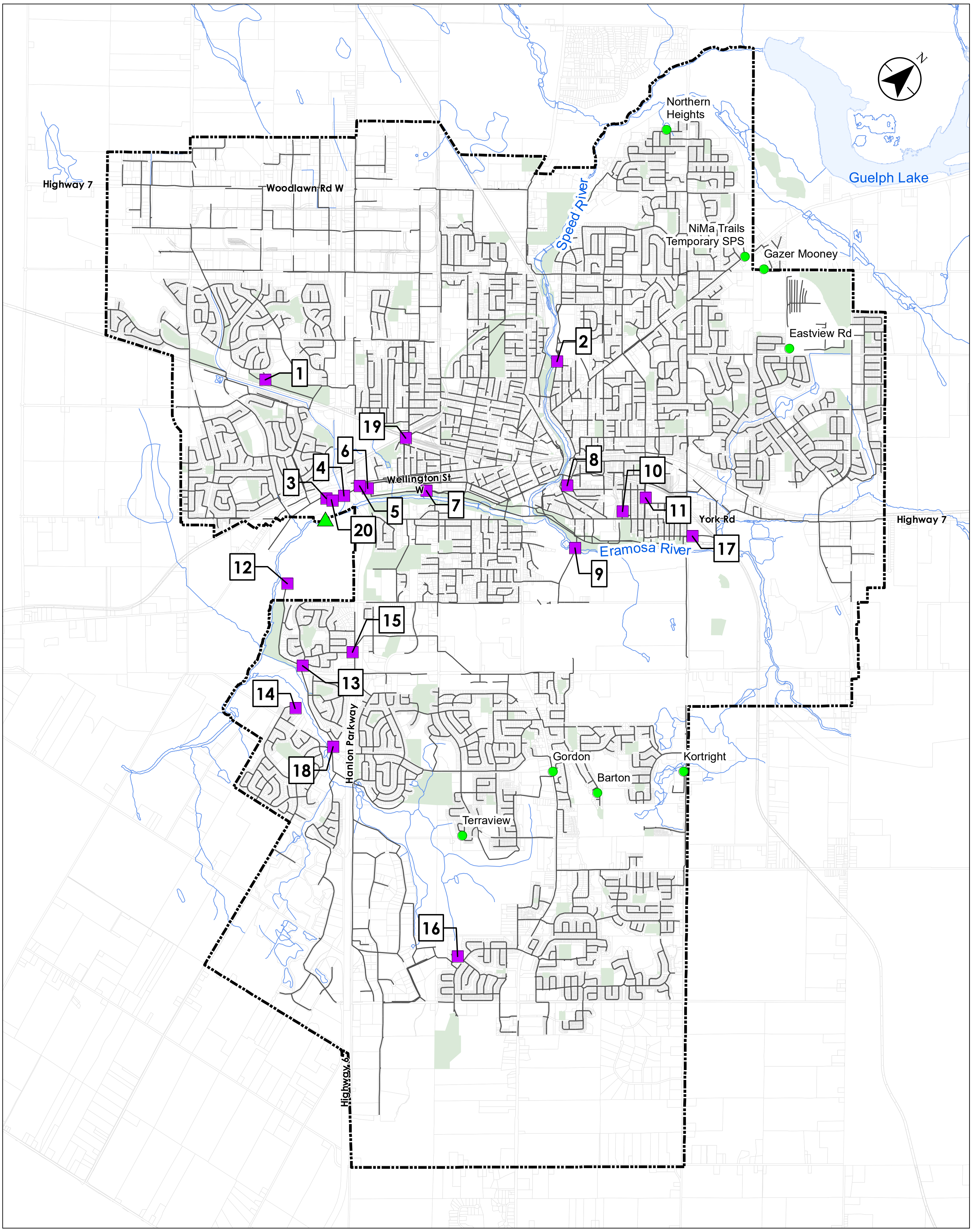

Legend

- Study Area
- Parks
- Watercourse
- Railway
- Property
- Forcemain
- Siphon
- Sanitary Sewer
- Failure Test Location
- Pump Station
- Wastewater Treatment Plant

0 1 2 Km

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Notes

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CITY OF GUELPH WATER AND WASTEWATER SERVICING MASTER PLAN – TM1 BACKGROUND REVIEW

Figure No.
4.2

Title
Test Failure Locations Overview

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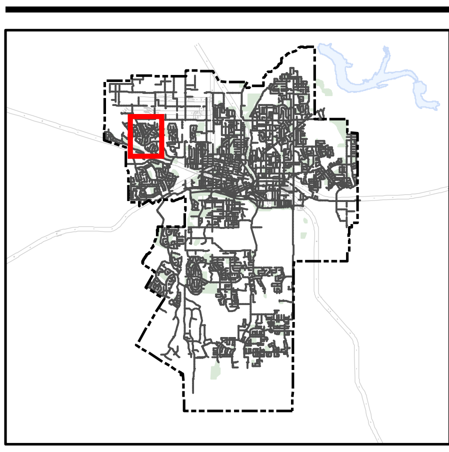

Legend

- Sanitary Sewer
- Railway
- Watercourse
- Property
- Parks

Site 1 Tributary Sewers

- No Surcharge
- Surcharging (d / D >= 1)

Failure Location #1

MH Surcharging

- Existing MH FB >= 1.8m, Failure MH FB >= 1.8m
- Existing MH FB >= 1.8m, Failure MH FB < 1.8m
- Existing MH FB >= 1.8m, Failure MH Flooding
- Existing MH FB < 1.8m, Failure MH FB < 1.8m
- Existing MH FB < 1.8m, Failure MH Flooding
- Existing MH Flooding, Failure MH Flooding

0 100 200 Meters

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Figure No.

4.3

Title

Pipe Failure Analysis Results Location #1

Notes

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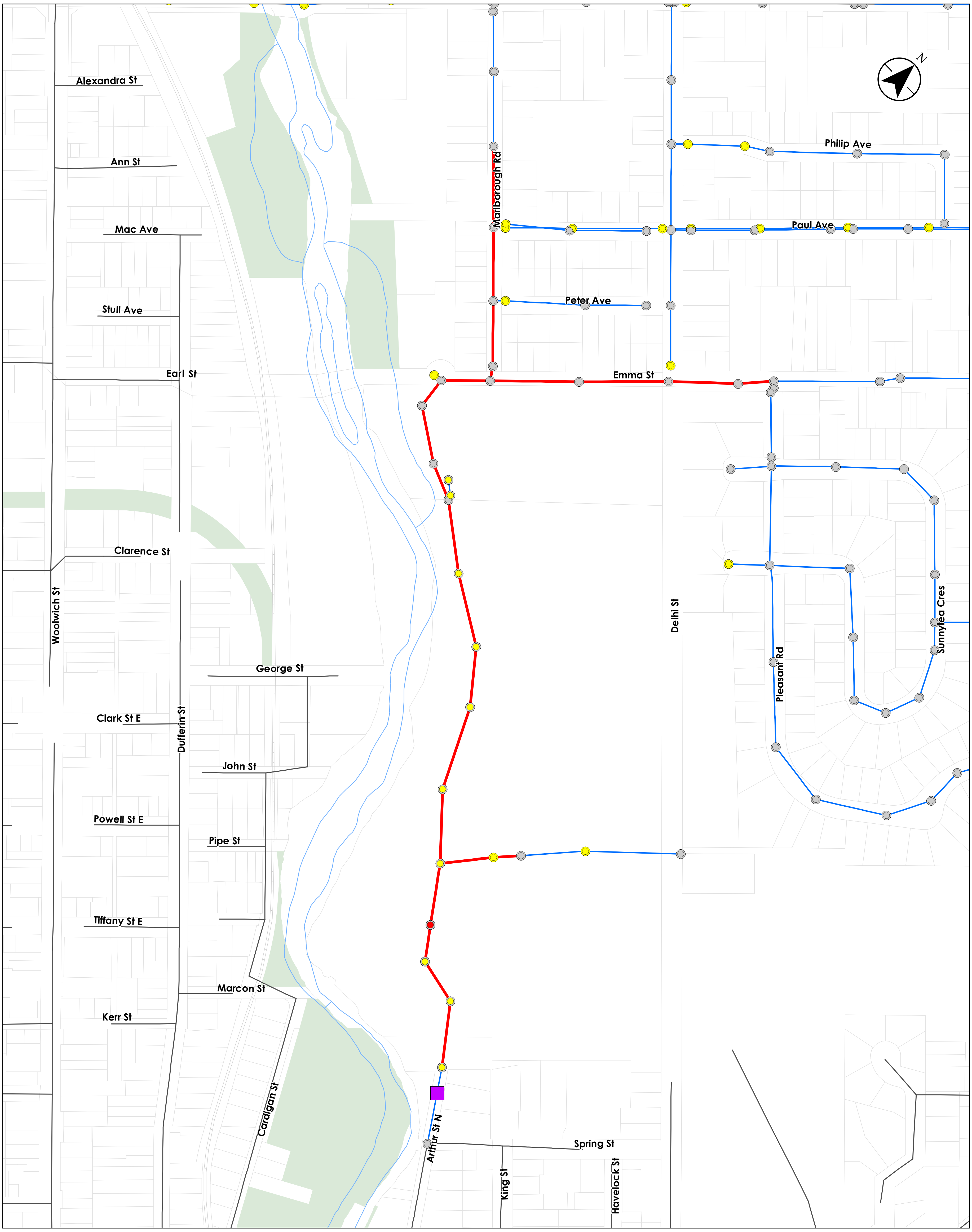

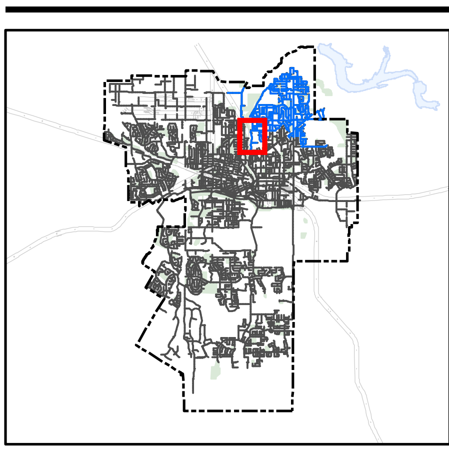

Legend

- Sanitary Sewer
- Railway
- Watercourse
- Property
- Parks

Site 2 Tributary Sewers

- No Surcharge
- Surcharging (d / D >= 1)

- Failure Location #2

MH Surcharging

- Existing MH FB >= 1.8m, Failure MH FB >= 1.8m
- Existing MH FB >= 1.8m, Failure MH FB < 1.8m
- Existing MH FB >= 1.8m, Failure MH Flooding
- Existing MH FB < 1.8m, Failure MH FB < 1.8m
- Existing MH FB < 1.8m, Failure MH Flooding
- Existing MH Flooding, Failure MH Flooding

0 100 200 Meters

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Figure No.

4.4

Title

Pipe Failure Analysis Results Location #2

Notes

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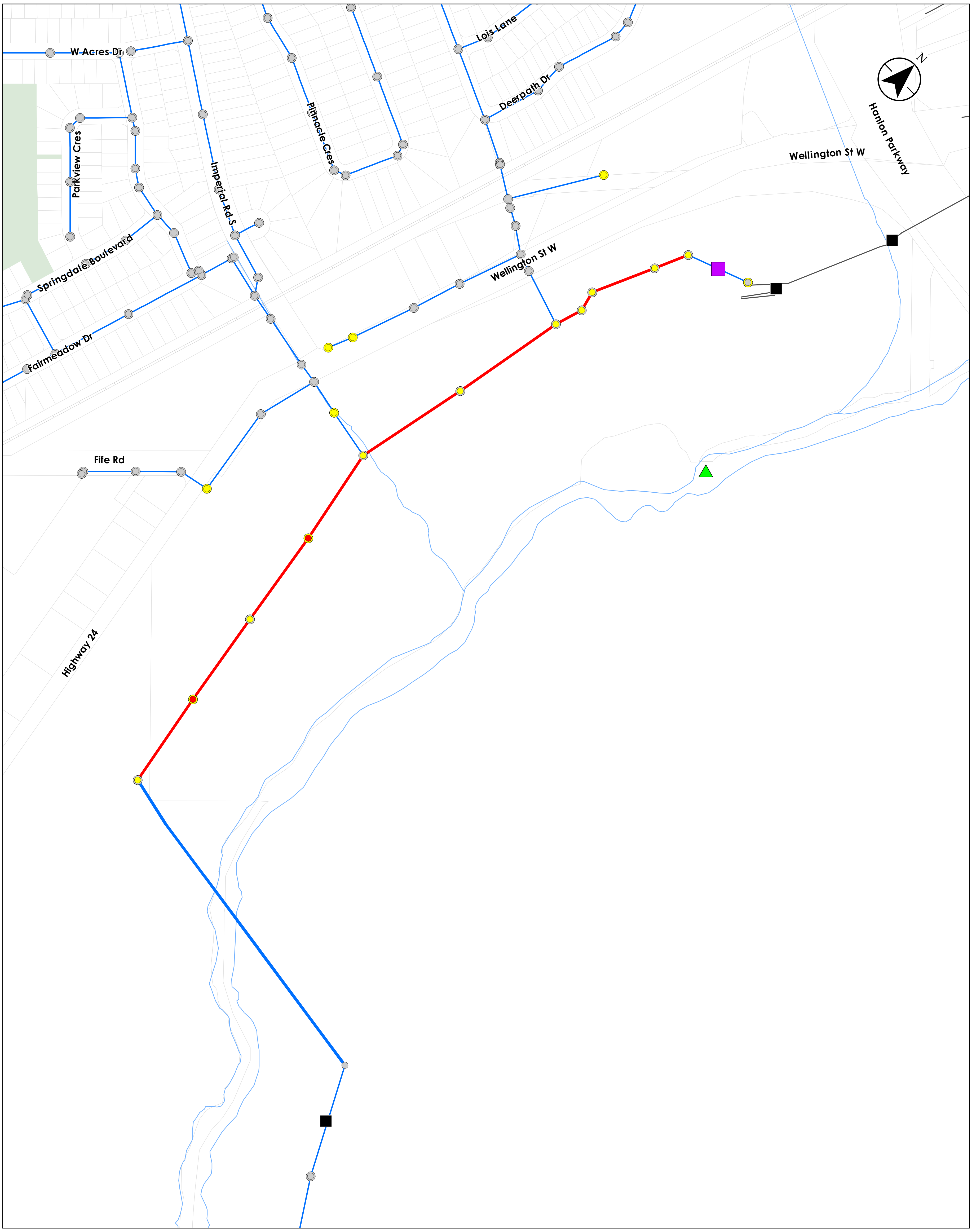

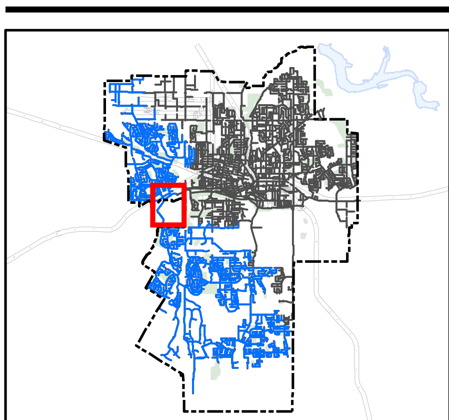

Legend

- Wastewater Treatment Plant
- Sanitary Sewer
- Railway
- Watercourse
- Property
- Parks

Site 3 Tributary Sewers

- No Surcharge
- Surcharging (d / D >= 1)

- Failure Location #3
- Other Failure Site

MH Surcharging

- Existing MH FB >= 1.8m, Failure MH FB >= 1.8m
- Existing MH FB >= 1.8m, Failure MH FB < 1.8m
- Existing MH FB >= 1.8m, Failure MH Flooding
- Existing MH FB < 1.8m, Failure MH FB < 1.8m
- Existing MH FB < 1.8m, Failure MH Flooding
- Existing MH Flooding, Failure MH Flooding

0 100 200 Meters

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Figure No.
4.5

Title
Pipe Failure Analysis Results Location #3

Notes

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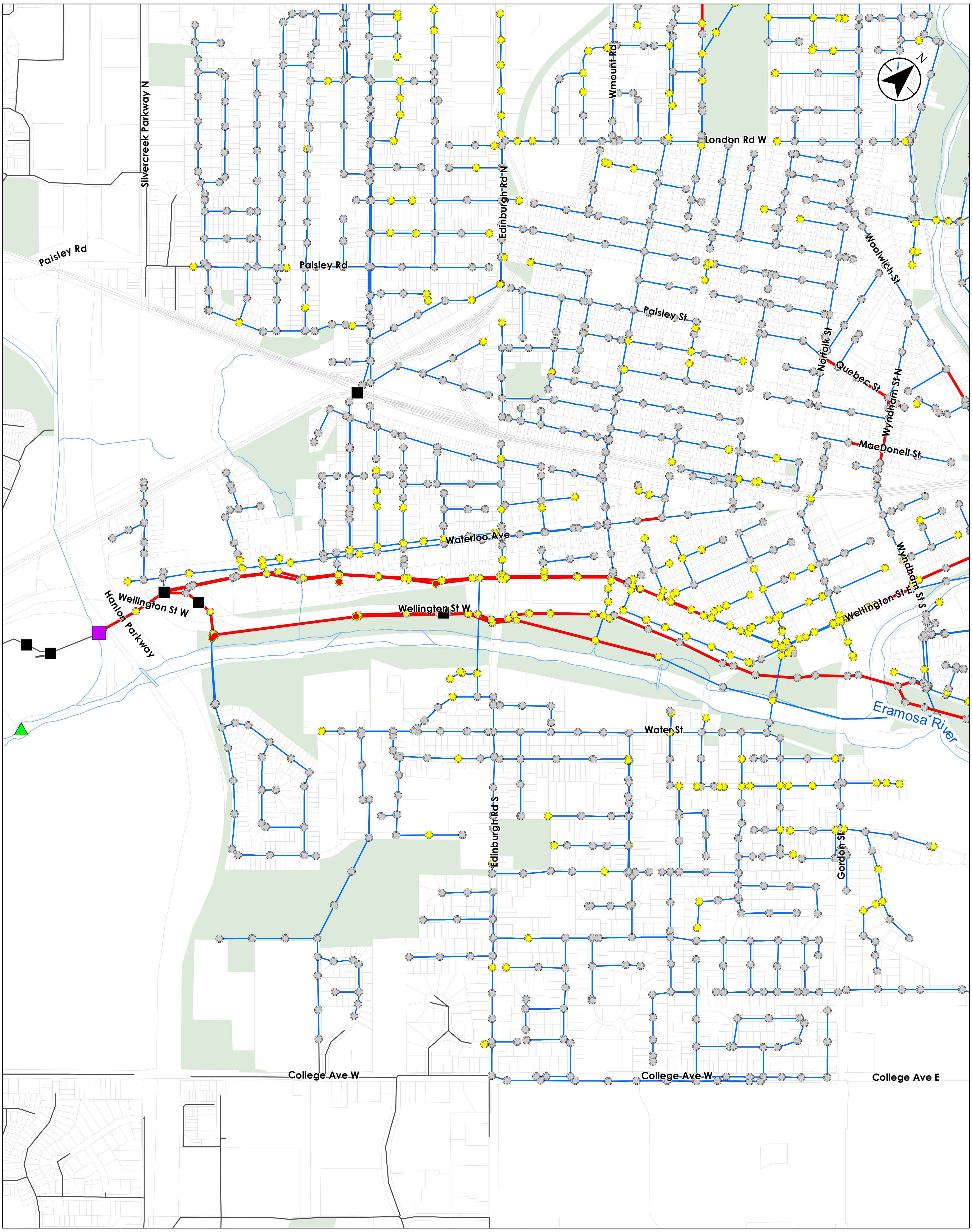

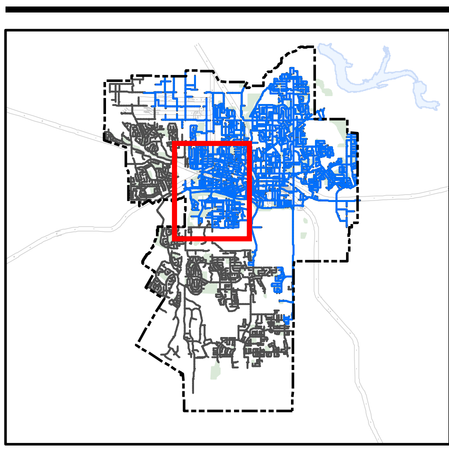

Legend

- Wastewater Treatment Plant
- Sanitary Sewer
- Railway
- Watercourse
- Property
- Parks

Site 4 Tributary Sewers

- No Surcharge
- Surcharging (d / D >= 1)

- Failure Location #4
- Other Failure Site

MH Surcharging

- Existing MH FB >= 1.8m, Failure MH FB >= 1.8m
- Existing MH FB >= 1.8m, Failure MH FB < 1.8m
- Existing MH FB >= 1.8m, Failure MH Flooding
- Existing MH FB < 1.8m, Failure MH FB < 1.8m
- Existing MH FB < 1.8m, Failure MH Flooding
- Existing MH Flooding, Failure MH Flooding

0 200 400 Meters

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CITY OF GUELPH WATER AND WASTEWATER SERVICING MASTER PLAN – TM1 BACKGROUND REVIEW

Figure No.

4.6

Title

Pipe Failure Analysis Results Location #4

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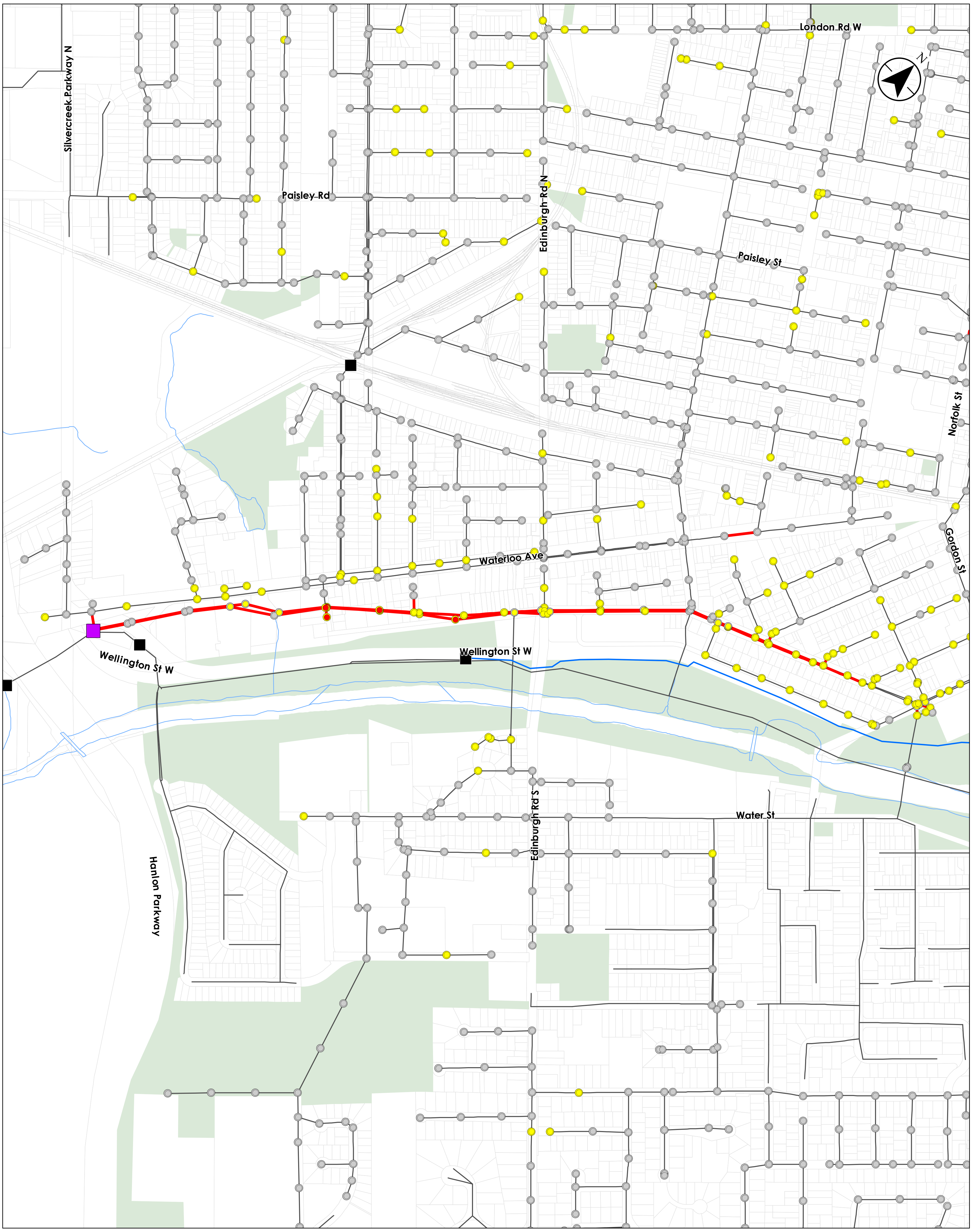

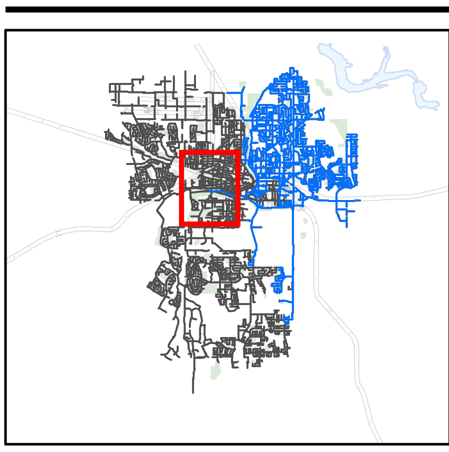

Legend

- Sanitary Sewer
- Railway
- Watercourse
- Property
- Parks

Site 5 Tributary Sewers

- No Surcharge
- Surcharging (d / D >= 1)

- Failure Location #5
- Other Failure Site

MH Surcharging

- Existing MH FB >= 1.8m, Failure MH FB >= 1.8m
- Existing MH FB >= 1.8m, Failure MH FB < 1.8m
- Existing MH FB >= 1.8m, Failure MH Flooding
- Existing MH FB < 1.8m, Failure MH FB < 1.8m
- Existing MH FB < 1.8m, Failure MH Flooding
- Existing MH Flooding, Failure MH Flooding

0 200 400 Meters

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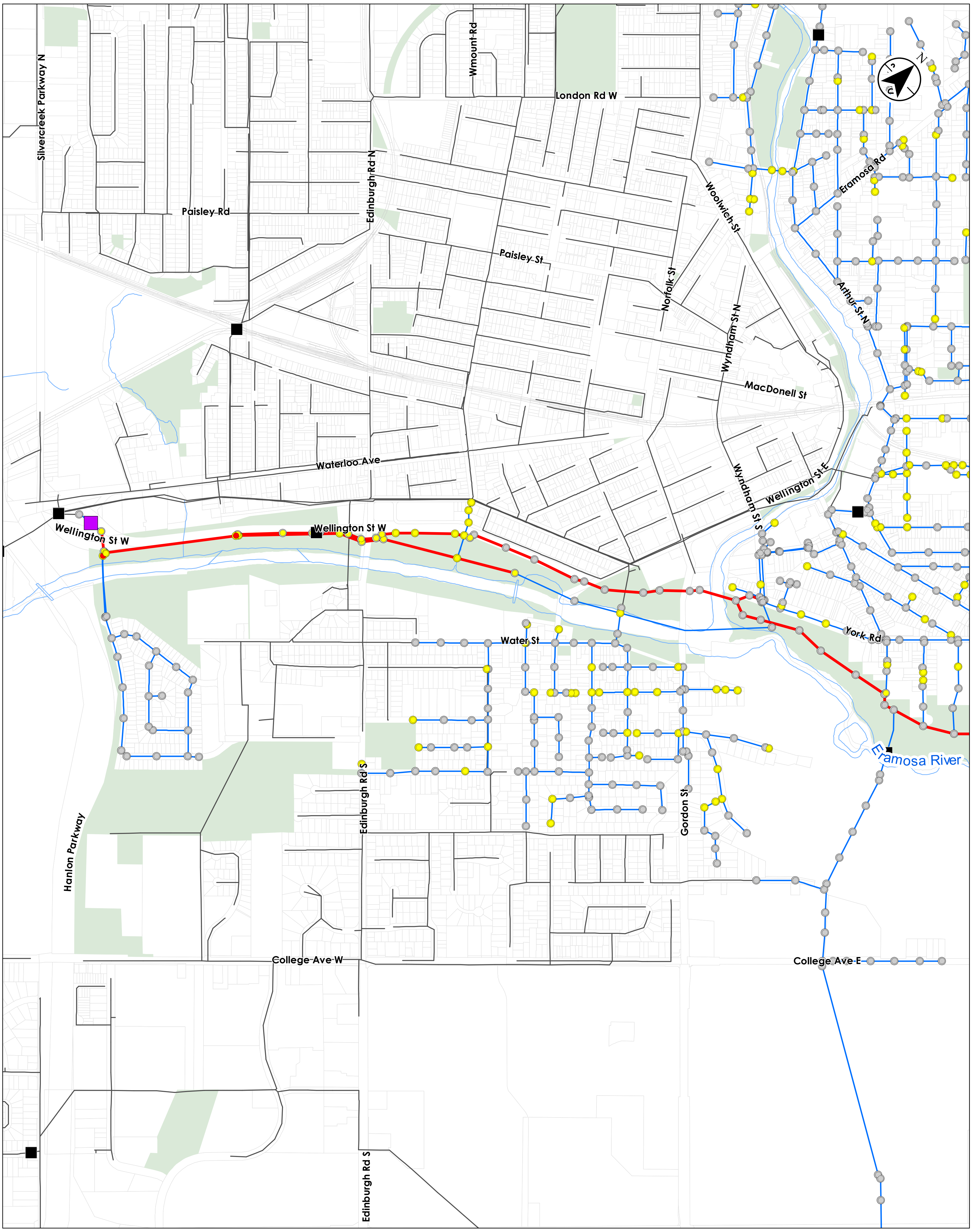
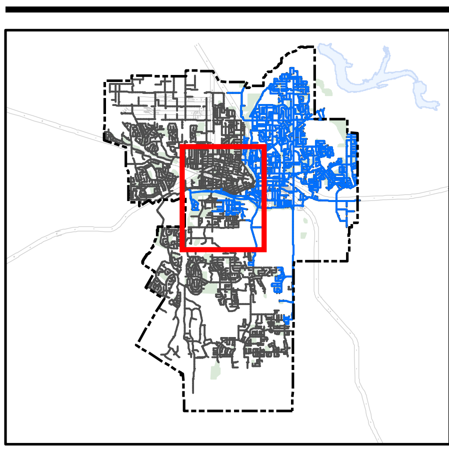
Figure No.

4.7

Title

Pipe Failure Analysis Results Location #5





Legend

- Sanitary Sewer
- Railway
- Watercourse
- Property
- Parks

Site 6 Tributary Sewers

- No Surcharge
- Surcharging (d / D >= 1)

- Failure Location #6
- Other Failure Site

MH Surcharging

- Existing MH FB >= 1.8m, Failure MH FB >= 1.8m
- Existing MH FB >= 1.8m, Failure MH FB < 1.8m
- Existing MH FB >= 1.8m, Failure MH Flooding
- Existing MH FB < 1.8m, Failure MH FB < 1.8m
- Existing MH FB < 1.8m, Failure MH Flooding
- Existing MH Flooding, Failure MH Flooding

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Figure No.
4.8

Title
Pipe Failure Analysis Results Location #6

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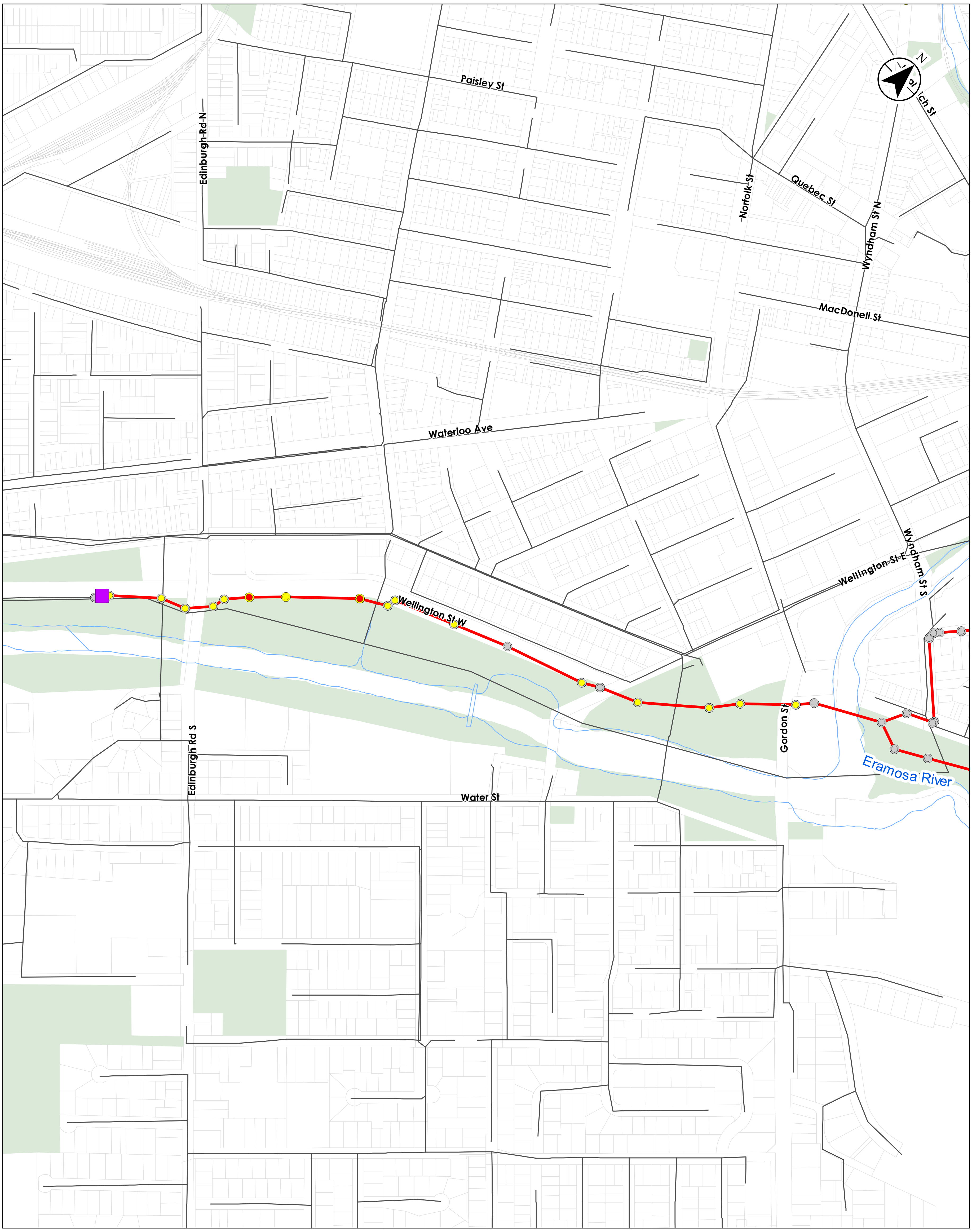

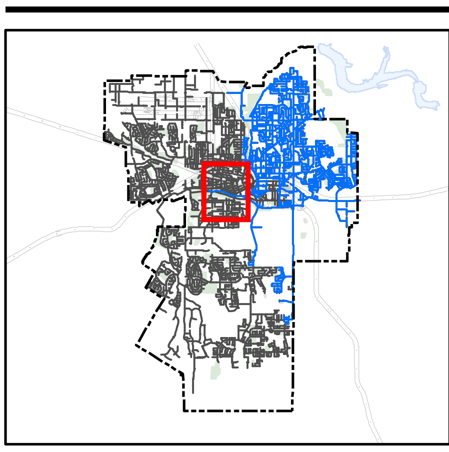

Legend

- Sanitary Sewer
- Railway
- Watercourse
- Property
- Parks

Site 7 Tributary Sewers

- No Surcharge
- Surcharging (d / D >= 1)

- Failure Location #7
- Other Failure Site

MH Surcharging

- Existing MH FB >= 1.8m, Failure MH FB >= 1.8m
- Existing MH FB >= 1.8m, Failure MH FB < 1.8m
- Existing MH FB >= 1.8m, Failure MH Flooding
- Existing MH FB < 1.8m, Failure MH FB < 1.8m
- Existing MH FB < 1.8m, Failure MH Flooding
- Existing MH Flooding, Failure MH Flooding

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Figure No.

4.9

Title

Pipe Failure Analysis Results Location #7

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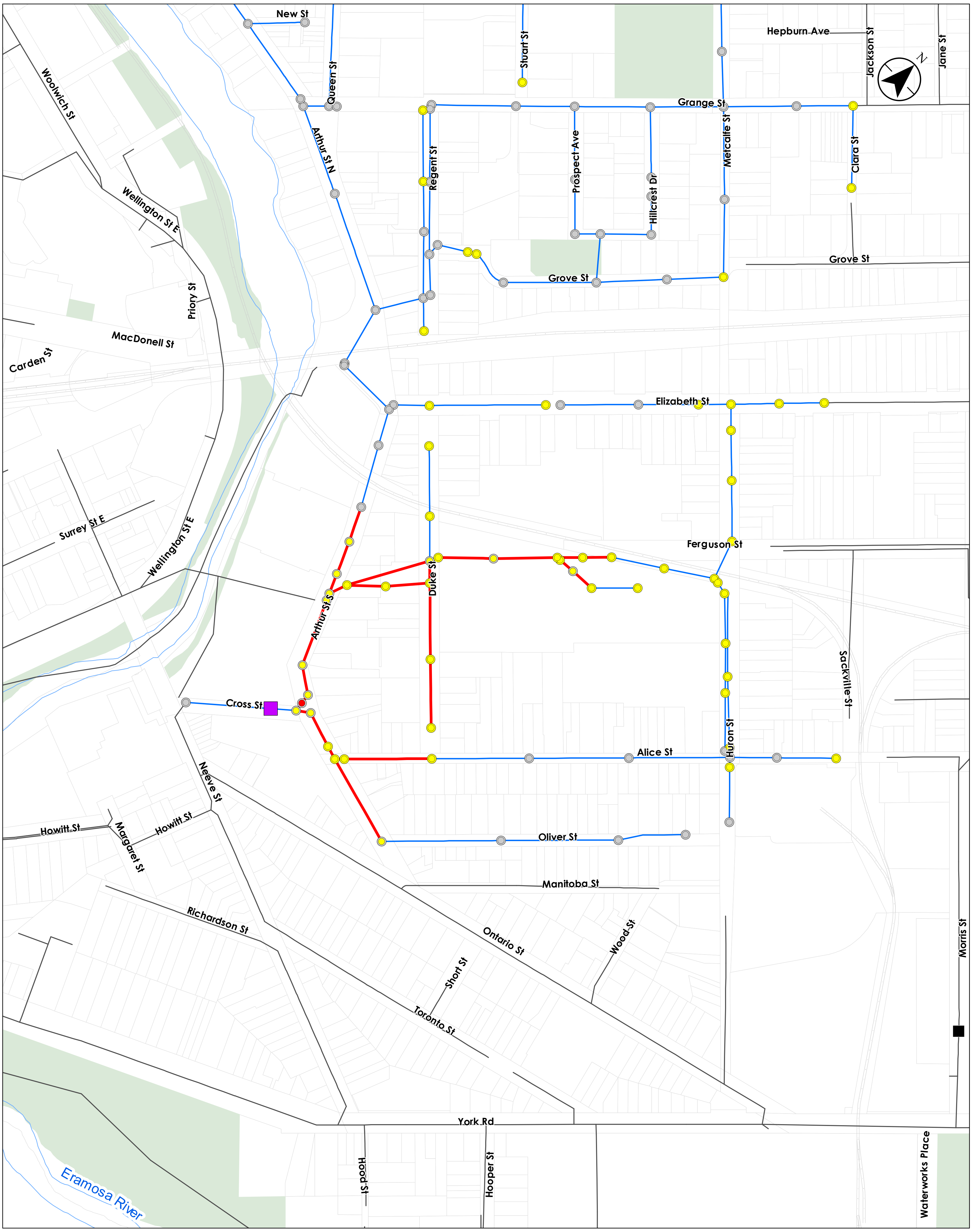

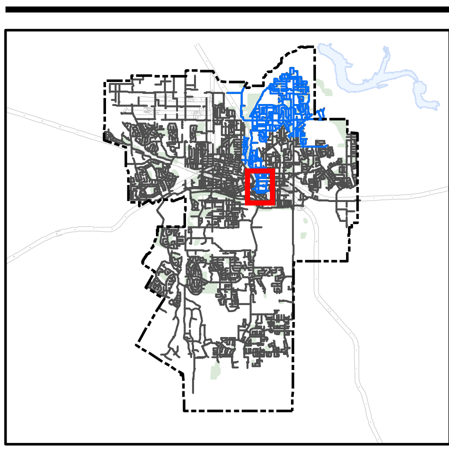

Legend

- Sanitary Sewer
- Railway
- Watercourse
- Property
- Parks

Site 8 Tributary Sewers

- No Surcharge
- Surcharging (d / D >= 1)

- Failure Location #8
- Other Failure Site

MH Surcharging

- Existing MH FB >= 1.8m, Failure MH FB >= 1.8m
- Existing MH FB >= 1.8m, Failure MH FB < 1.8m
- Existing MH FB >= 1.8m, Failure MH Flooding
- Existing MH FB < 1.8m, Failure MH FB < 1.8m
- Existing MH FB < 1.8m, Failure MH Flooding
- Existing MH Flooding, Failure MH Flooding

0 100 200 Meters

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Figure No.
4.10

Title
Pipe Failure Analysis Results Location #8

Notes
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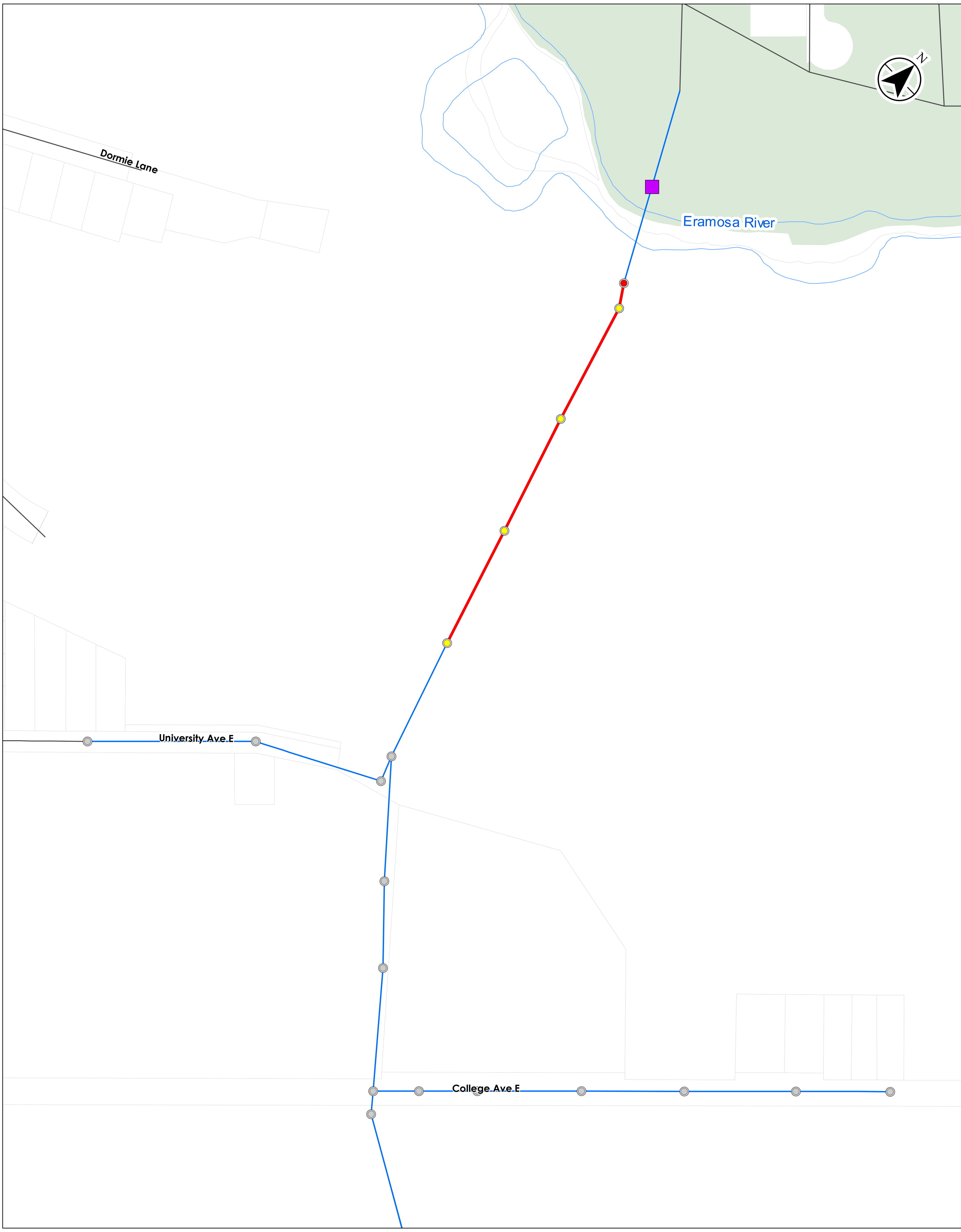

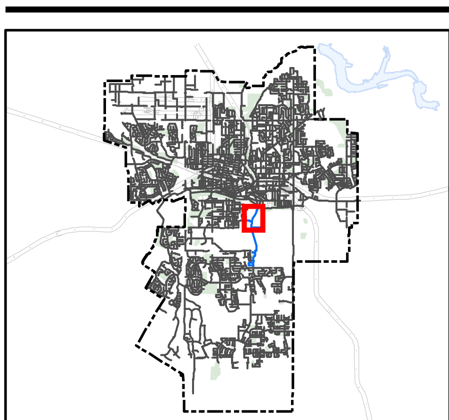

Legend

- Sanitary Sewer
- Watercourse
- Property
- Parks

Site 9 Tributary Sewers

- No Surcharge
- Surcharging (d / D >= 1)

- Failure Location #9

MH Surcharging

- Existing MH FB >= 1.8m, Failure MH FB >= 1.8m
- Existing MH FB >= 1.8m, Failure MH FB < 1.8m
- Existing MH FB >= 1.8m, Failure MH Flooding
- Existing MH FB < 1.8m, Failure MH FB < 1.8m
- Existing MH FB < 1.8m, Failure MH Flooding
- Existing MH Flooding, Failure MH Flooding

0 50 100 Meters
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CITY OF GUELPH WATER AND WASTEWATER SERVICING MASTER PLAN – TM1 BACKGROUND REVIEW

Figure No.
4.11

Title
Pipe Failure Analysis Results Location #9

Notes

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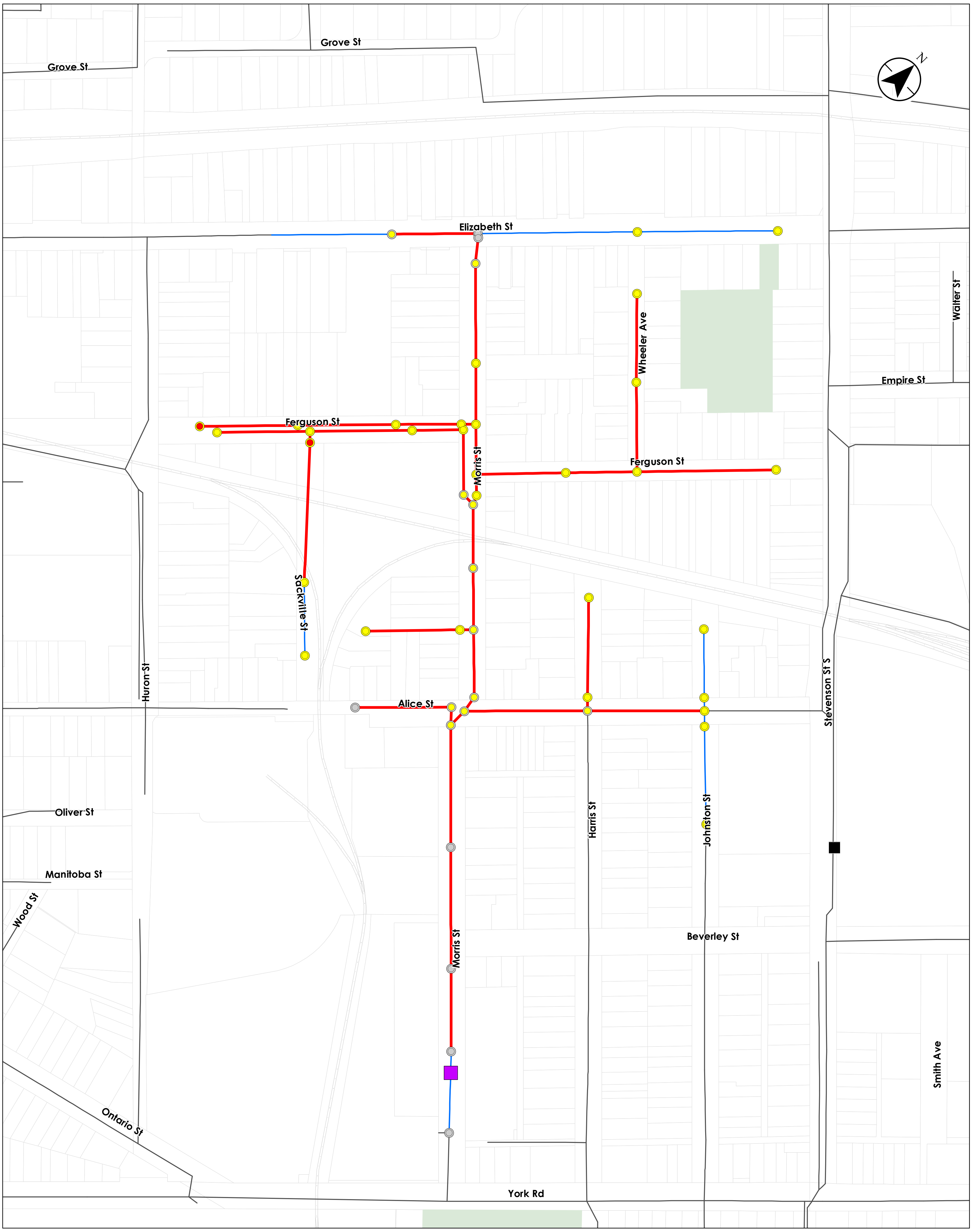

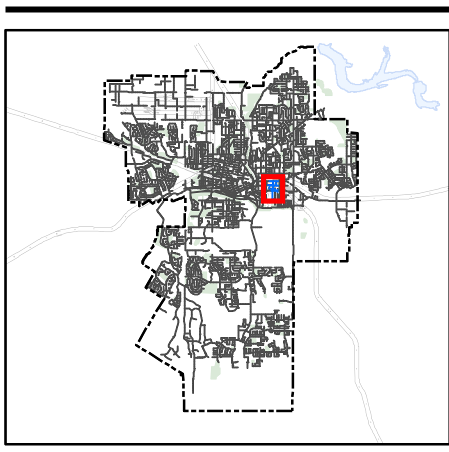

Legend

- Sanitary Sewer
- Railway
- Property
- Parks

Site 10 Tributary Sewers

- No Surcharge
- Surcharging (d / D >= 1)

- Failure Location #10
- Other Failure Site

MH Surcharging

- Existing MH FB >= 1.8m, Failure MH FB >= 1.8m
- Existing MH FB >= 1.8m, Failure MH FB < 1.8m
- Existing MH FB >= 1.8m, Failure MH Flooding
- Existing MH FB < 1.8m, Failure MH FB < 1.8m
- Existing MH FB < 1.8m, Failure MH Flooding
- Existing MH Flooding, Failure MH Flooding

0 50 100 Meters

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Figure No.
4.12

Title
Pipe Failure Analysis Results Location #10

Notes

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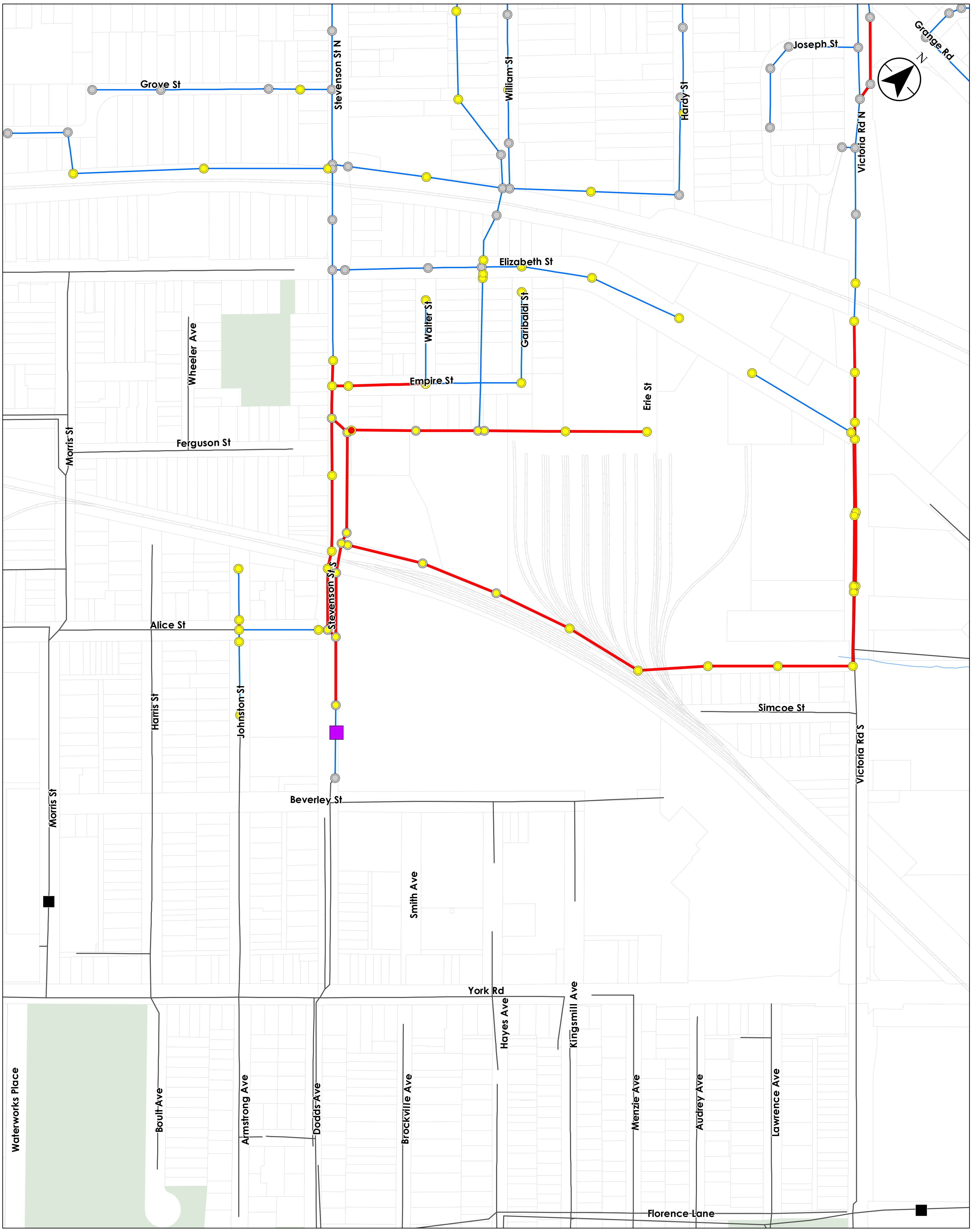

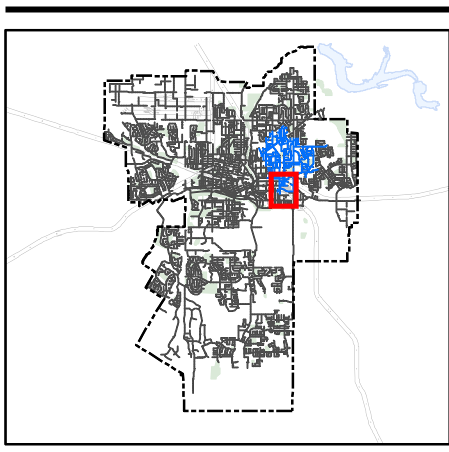

Legend

- Sanitary Sewer
- Railway
- Watercourse
- Property
- Parks

Site 11 Tributary Sewers

- No Surcharge
- Surcharging (d / D >= 1)

- Failure Location #11
- Other Failure Site

MH Surcharging

- Existing MH FB >= 1.8m, Failure MH FB >= 1.8m
- Existing MH FB >= 1.8m, Failure MH FB < 1.8m
- Existing MH FB >= 1.8m, Failure MH Flooding
- Existing MH FB < 1.8m, Failure MH FB < 1.8m
- Existing MH FB < 1.8m, Failure MH Flooding
- Existing MH Flooding, Failure MH Flooding

0 50 100 Meters
1:4,000 (At Original document size of 11x17)

Notes

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C3 WATER

Project Location 165640298
City of Guelph

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CITY OF GUELPH WATER AND WASTEWATER SERVICING MASTER PLAN – TM1 BACKGROUND REVIEW

Figure No.
4.13

Title
Pipe Failure Analysis Results Location #11

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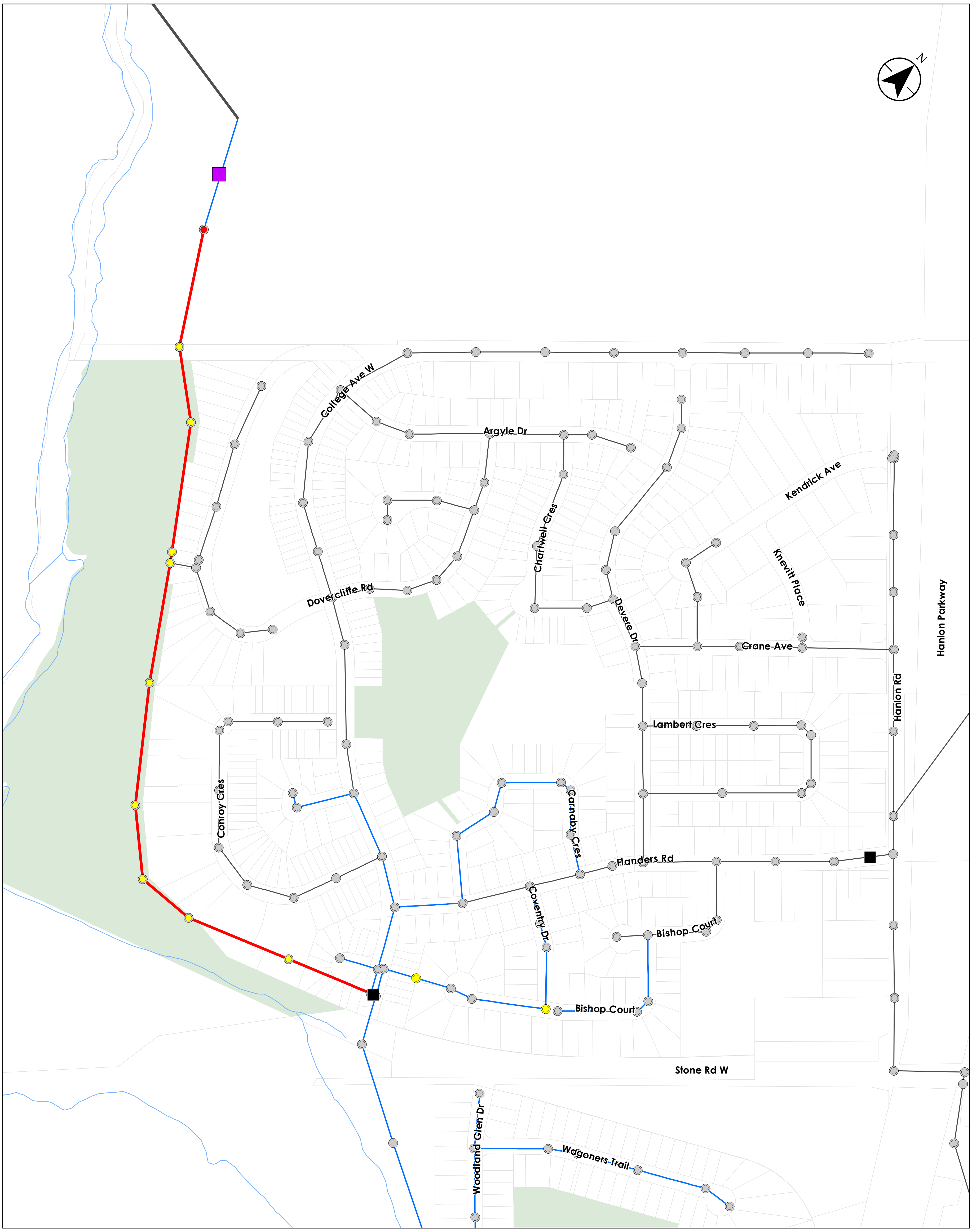

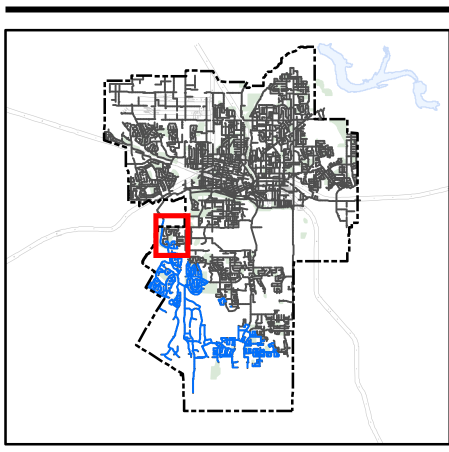

Legend

- Sanitary Sewer
- Watercourse
- Property
- Parks

Site 12 Tributary Sewers

- No Surcharge
- Surcharging (d / D >= 1)

- Failure Location #12
- Other Failure Site

MH Surcharging

- Existing MH FB >= 1.8m, Failure MH FB >= 1.8m
- Existing MH FB >= 1.8m, Failure MH FB < 1.8m
- Existing MH FB >= 1.8m, Failure MH Flooding
- Existing MH FB < 1.8m, Failure MH FB < 1.8m
- Existing MH FB < 1.8m, Failure MH Flooding
- Existing MH Flooding, Failure MH Flooding

0 100 200 Meters

1:5,000 (At Original document size of 11x17)

Notes

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Project Location
City of Guelph

165640298

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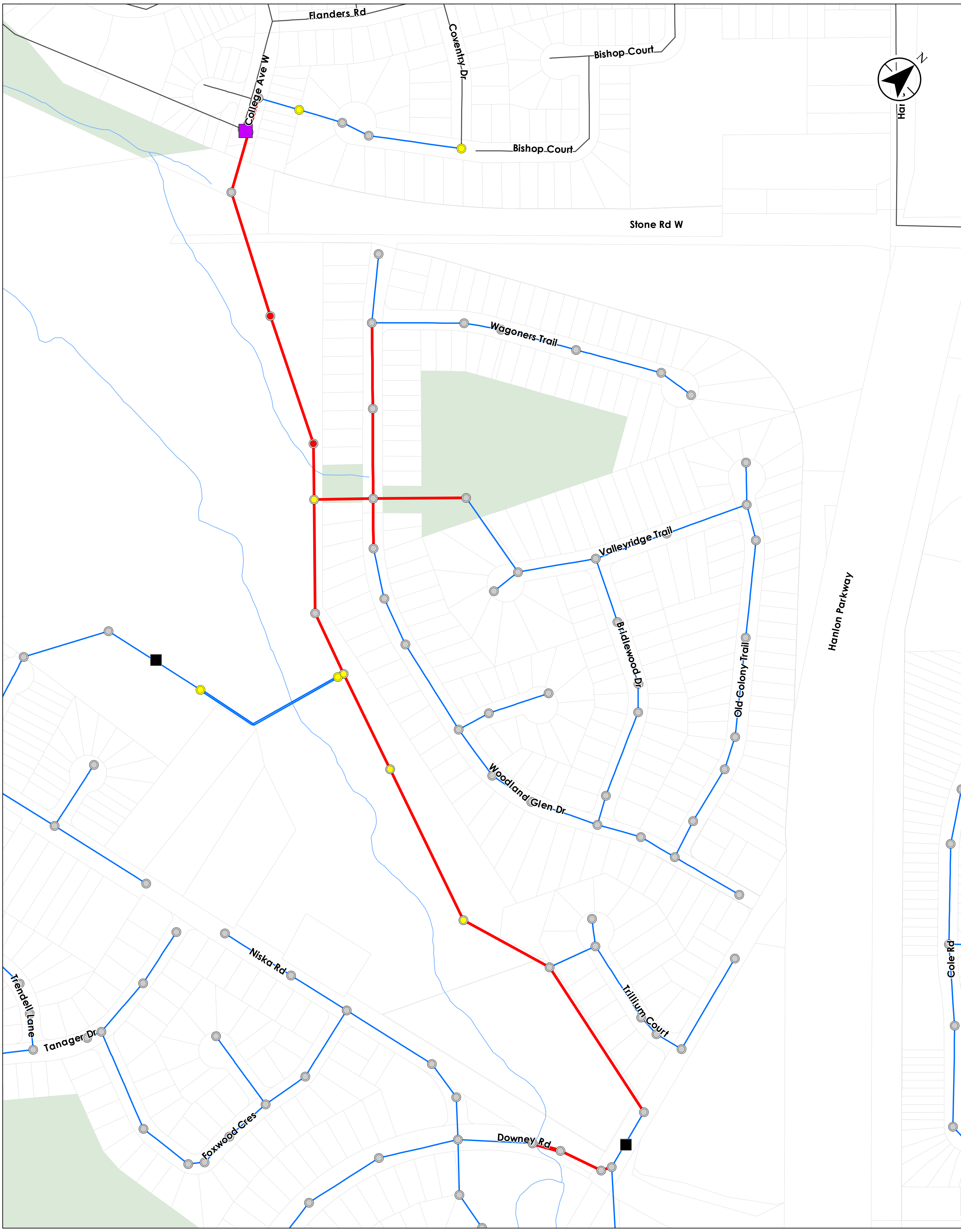
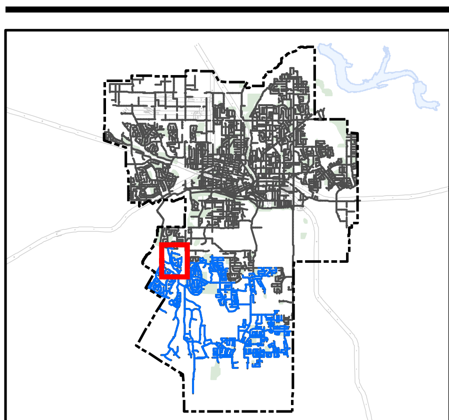
Figure No.
4.14

Title
Pipe Failure Analysis Results Location #12





Legend

- Sanitary Sewer
- Watercourse
- Property
- Parks

Site 13 Tributary Sewers

- No Surcharge
- Surcharging (d / D >= 1)

- Failure Location #13
- Other Failure Site

MH Surcharging

- Existing MH FB >= 1.8m, Failure MH FB >= 1.8m
- Existing MH FB >= 1.8m, Failure MH FB < 1.8m
- Existing MH FB >= 1.8m, Failure MH Flooding
- Existing MH FB < 1.8m, Failure MH FB < 1.8m
- Existing MH FB < 1.8m, Failure MH Flooding
- Existing MH Flooding, Failure MH Flooding

0 100 200 Meters

1:4,000 (At Original document size of 11x17)

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C3 WATER

Project Location 165640298
City of Guelph

Client/Project
CITY OF GUELPH WATER AND WASTEWATER SERVICING MASTER PLAN – TM1 BACKGROUND REVIEW

Figure No.
4.15

Title
Pipe Failure Analysis Results Location #13

Notes

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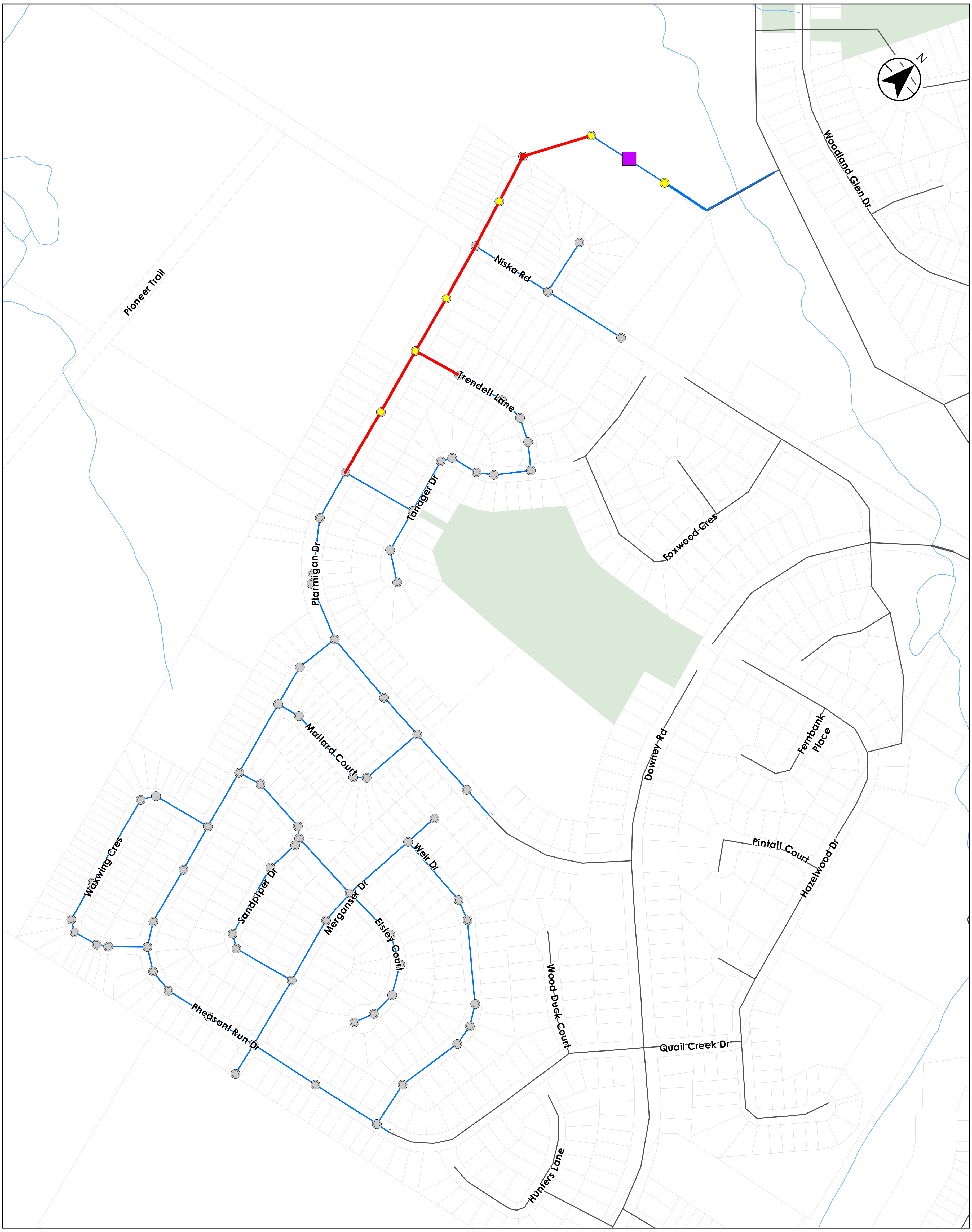

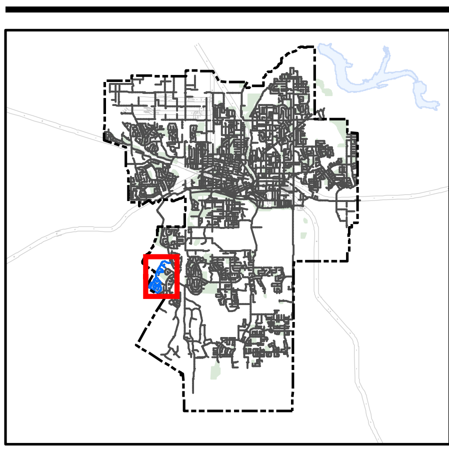

Legend

- Sanitary Sewer
- Watercourse
- Property
- Parks

Site 14 Tributary Sewers

- No Surcharge
- Surcharging (d / D >= 1)

Failure Location #14

MH Surcharging

- Existing MH FB >= 1.8m, Failure MH FB >= 1.8m
- Existing MH FB >= 1.8m, Failure MH FB < 1.8m
- Existing MH FB >= 1.8m, Failure MH Flooding
- Existing MH FB < 1.8m, Failure MH FB < 1.8m
- Existing MH FB < 1.8m, Failure MH Flooding
- Existing MH Flooding, Failure MH Flooding

0 100 200 Meters
1:5,000 (At Original document size of 11x17)

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C3 WATER

Project Location 165640298
City of Guelph

Client/Project
CITY OF GUELPH WATER AND WASTEWATER SERVICING MASTER PLAN – TM1 BACKGROUND REVIEW

Figure No.
4.16

Title
Pipe Failure Analysis Results Location #14

Notes

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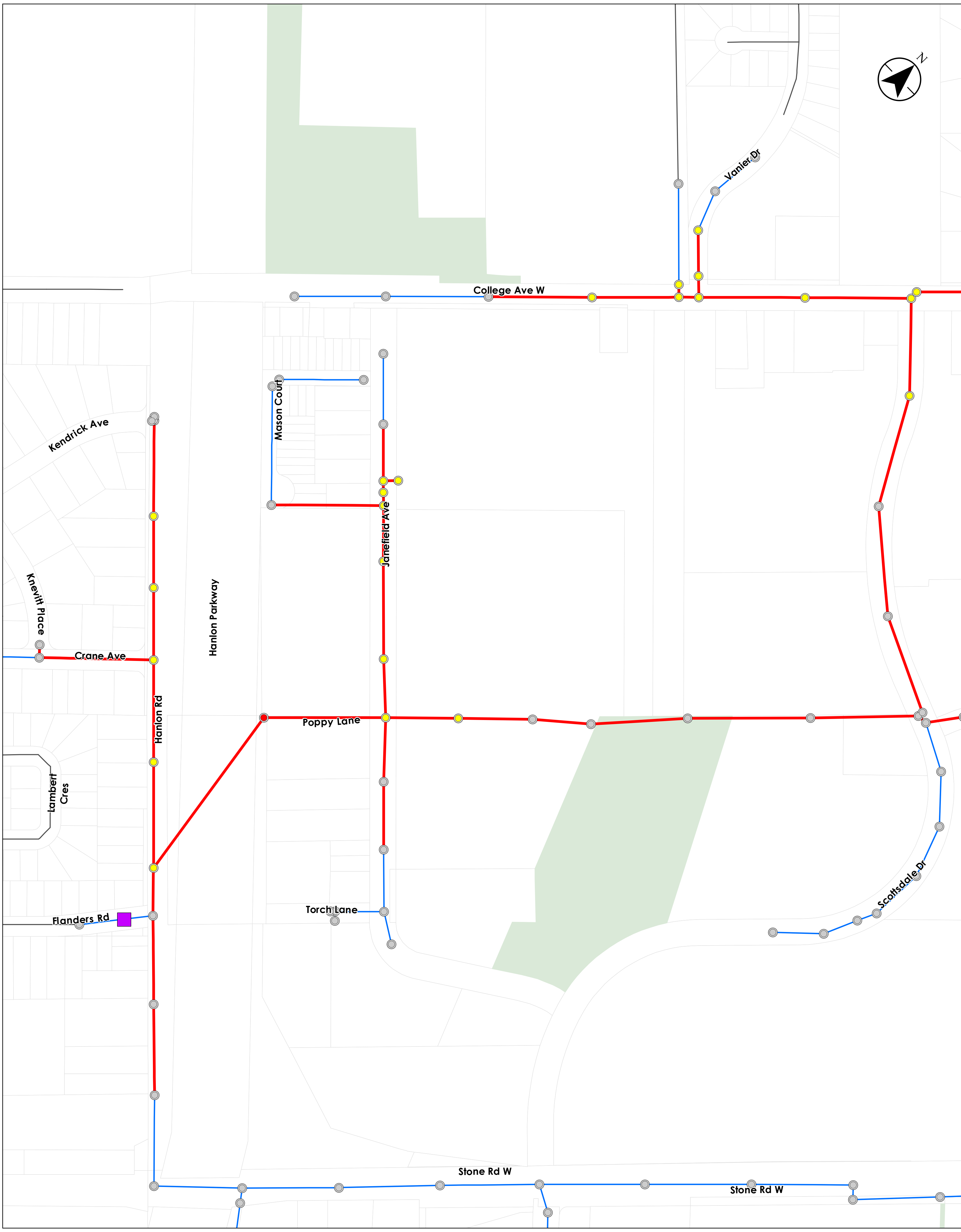

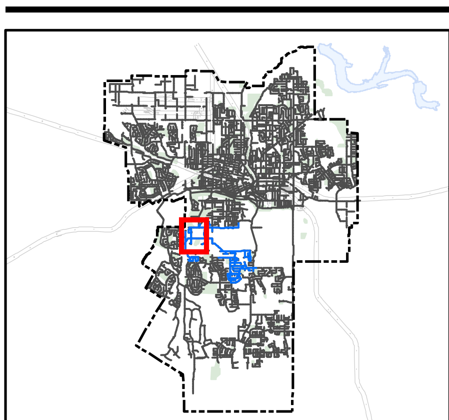

Legend

Sanitary Sewer

Property

Parks

Site 15 Tributary Sewers

No Surcharge

Surcharging (d / D >= 1)

Failure Location #15

MH Surcharging

Existing MH FB >= 1.8m, Failure MH FB >= 1.8m

Existing MH FB >= 1.8m, Failure MH FB < 1.8m

Existing MH FB >= 1.8m, Failure MH Flooding

Existing MH FB < 1.8m, Failure MH FB < 1.8m

Existing MH FB < 1.8m, Failure MH Flooding

Existing MH Flooding, Failure MH Flooding

0 50 100 Meters

1:4,000 (At Original document size of 11x17)

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C3 WATER

Project Location 165640298

City of Guelph

Client/Project

CITY OF GUELPH WATER AND WASTEWATER SERVICING MASTER PLAN – TM1 BACKGROUND REVIEW

Figure No.

4.17

Title

Pipe Failure Analysis Results Location #15

Notes

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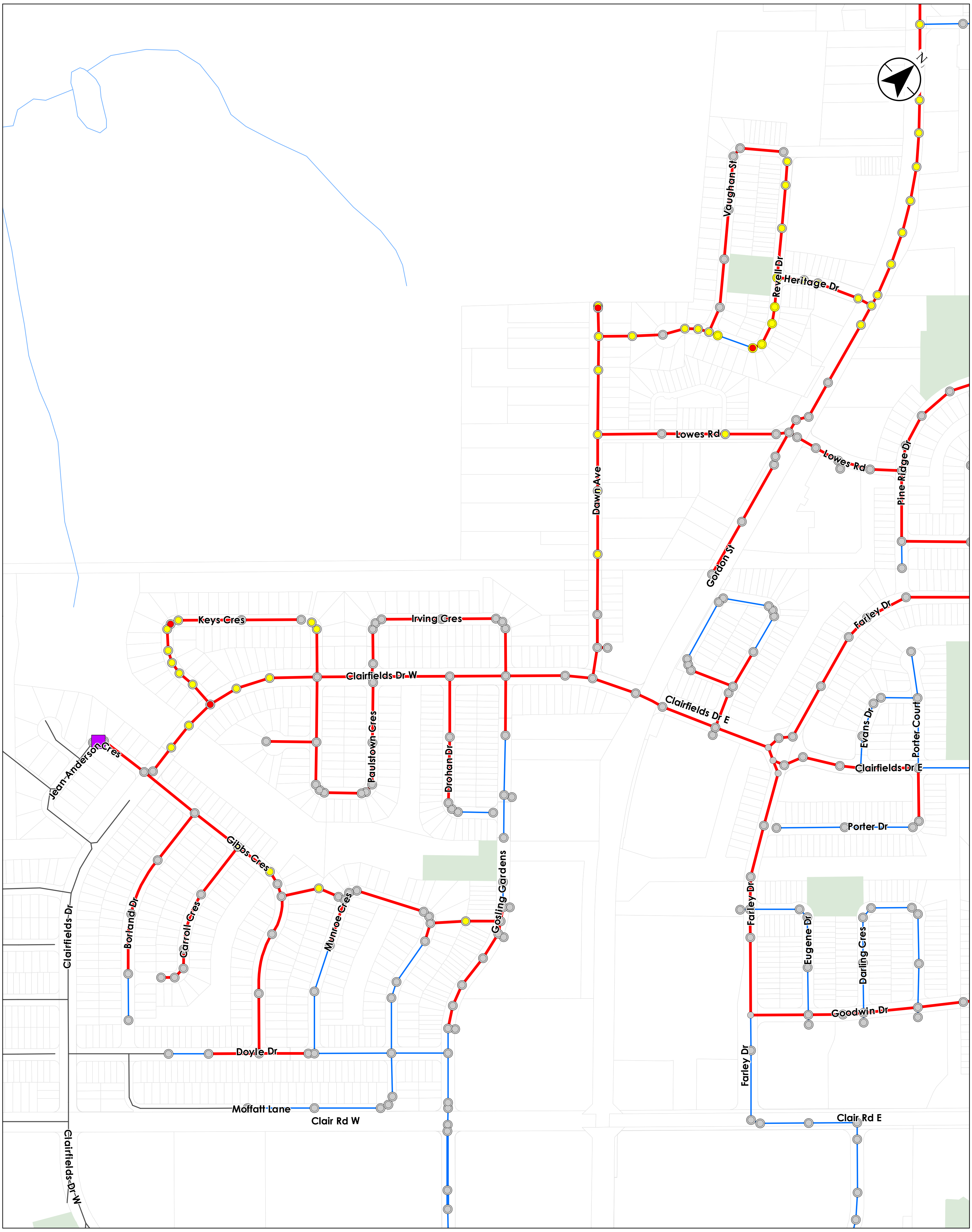

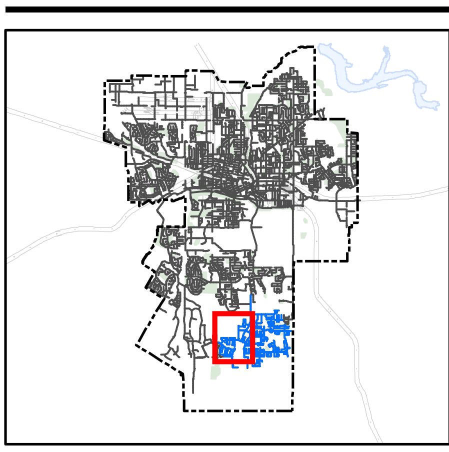

Legend

- Sanitary Sewer
- Watercourse
- Property
- Parks

Site 16 Tributary Sewers

- No Surcharge
- Surcharging (d / D >= 1)

- Failure Location #16

MH Surcharging

- Existing MH FB >= 1.8m, Failure MH FB >= 1.8m
- Existing MH FB >= 1.8m, Failure MH FB < 1.8m
- Existing MH FB >= 1.8m, Failure MH Flooding
- Existing MH FB < 1.8m, Failure MH FB < 1.8m
- Existing MH FB < 1.8m, Failure MH Flooding
- Existing MH Flooding, Failure MH Flooding

0 100 200 Meters

1:6,000 (At Original document size of 11x17)

Notes

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C3 WATER

Project Location 165640298

City of Guelph

Client/Project

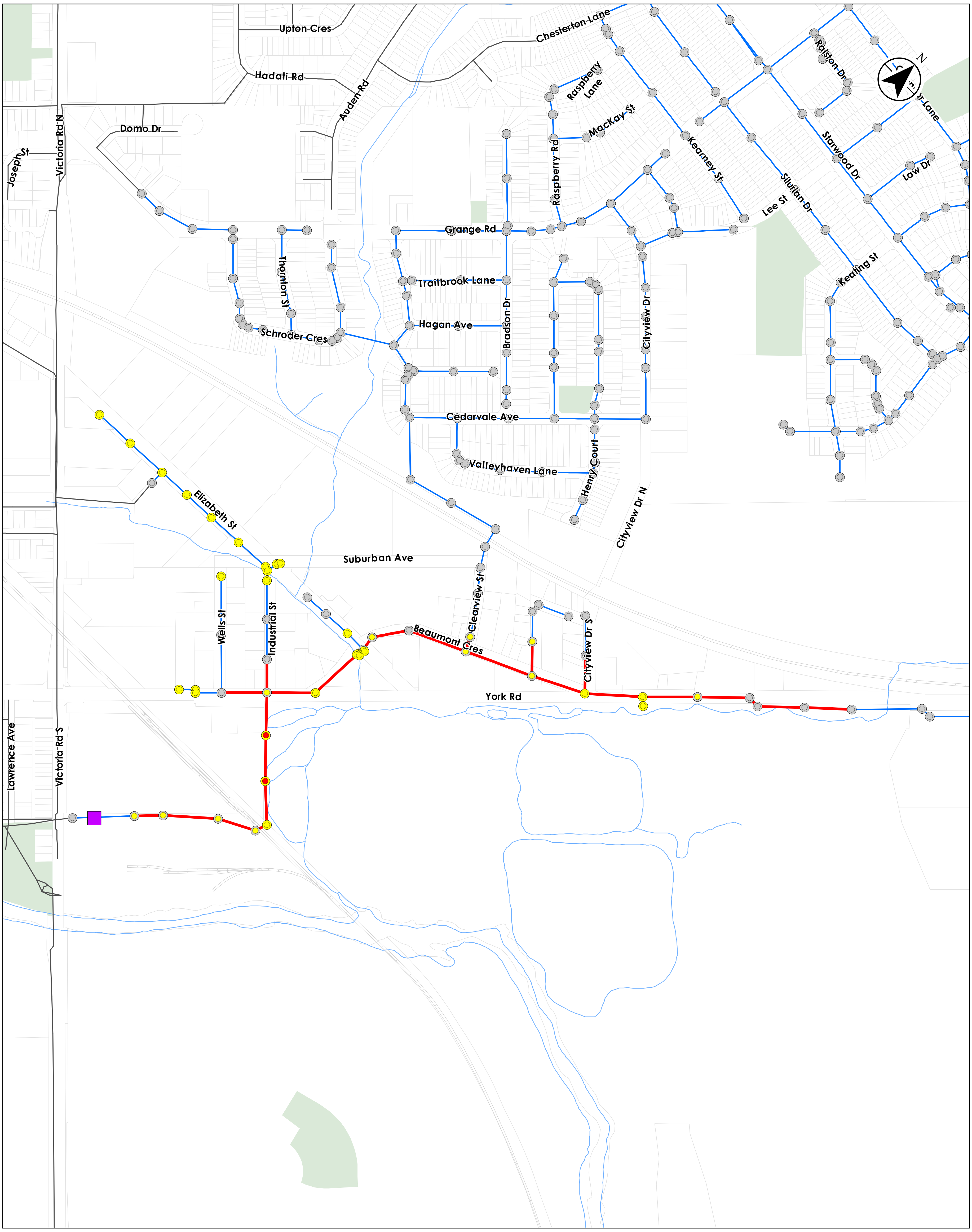
CITY OF GUELPH WATER AND WASTEWATER SERVICING MASTER PLAN – TM1 BACKGROUND REVIEW

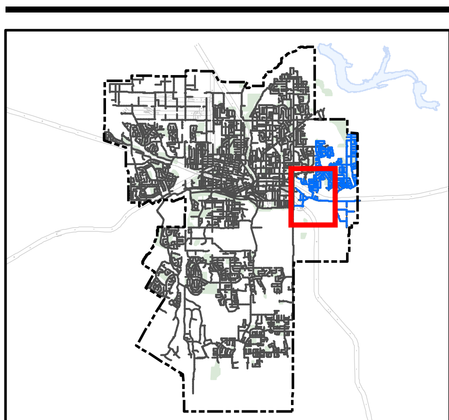
Figure No.

4.18

Title

Pipe Failure Analysis Results Location #16





Legend

- Sanitary Sewer
- Railway
- Watercourse
- Property
- Parks

Site 17 Tributary Sewers

- No Surcharge
- Surcharging (d / D >= 1)

- Failure Location #17

MH Surcharging

- Existing MH FB >= 1.8m, Failure MH FB >= 1.8m
- Existing MH FB >= 1.8m, Failure MH FB < 1.8m
- Existing MH FB >= 1.8m, Failure MH Flooding
- Existing MH FB < 1.8m, Failure MH FB < 1.8m
- Existing MH FB < 1.8m, Failure MH Flooding
- Existing MH Flooding, Failure MH Flooding

0 100 200 Meters

1:7,000 (At Original document size of 11x17)

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C3 WATER

Project Location
City of Guelph

165640298

Client/Project
CITY OF GUELPH WATER AND WASTEWATER SERVICING MASTER PLAN – TM1 BACKGROUND REVIEW

Figure No.
4.19

Title
Pipe Failure Analysis Results Location #17

Notes

1. Coordinate System: NAD 1983 UTM Zone 17N
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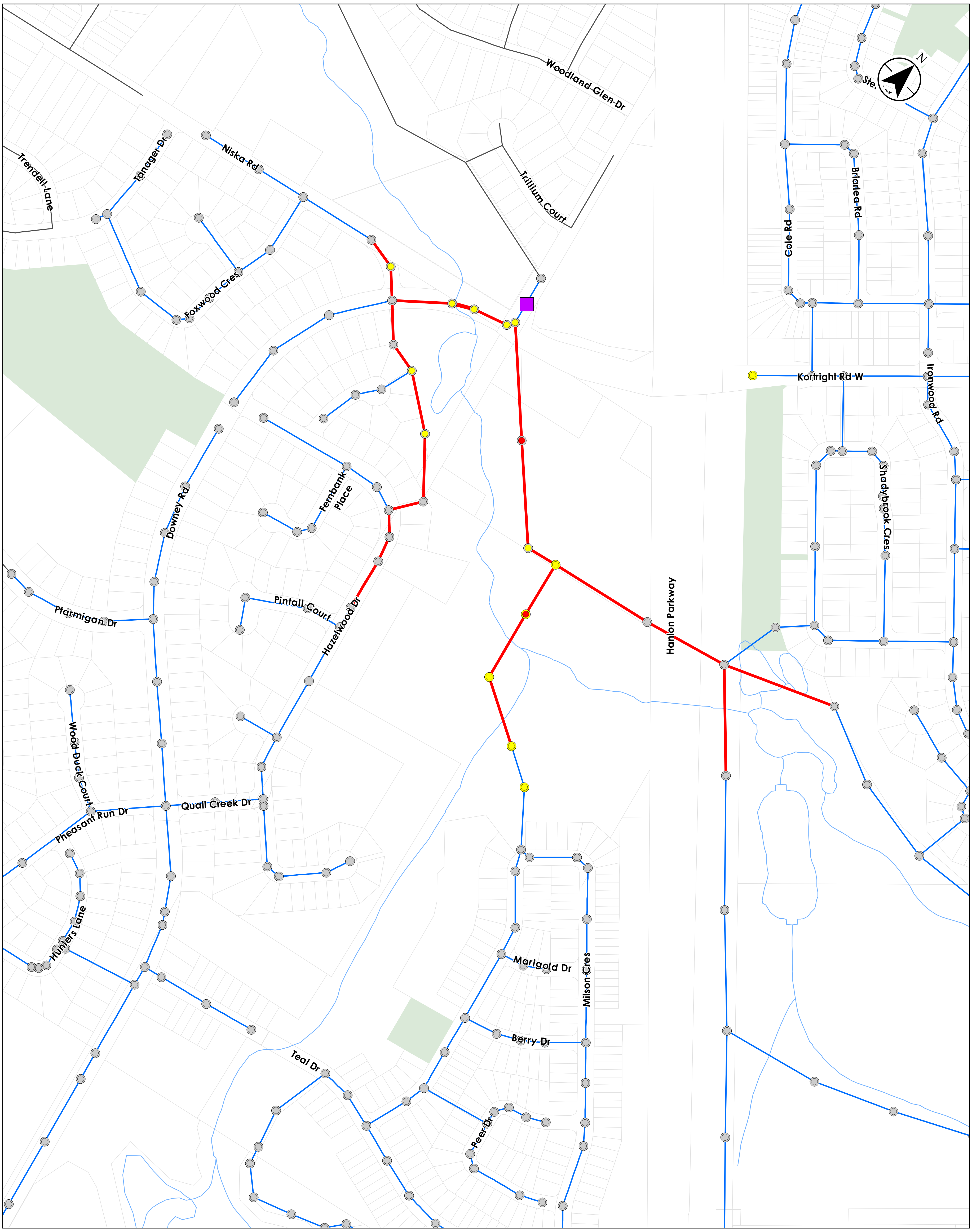

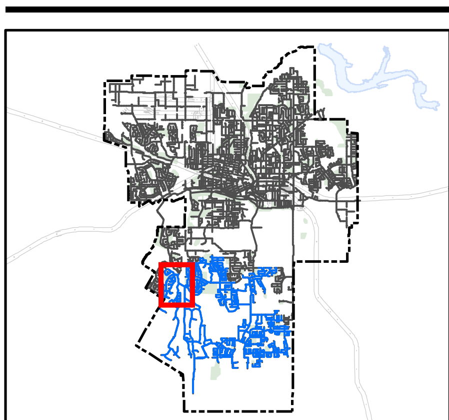

Legend

- Sanitary Sewer
- Watercourse
- Property
- Parks

Site 18 Tributary Sewers

- No Surcharge
- Surcharging (d / D >= 1)

Failure Location #18

MH Surcharging

- Existing MH FB >= 1.8m, Failure MH FB >= 1.8m
- Existing MH FB >= 1.8m, Failure MH FB < 1.8m
- Existing MH FB >= 1.8m, Failure MH Flooding
- Existing MH FB < 1.8m, Failure MH FB < 1.8m
- Existing MH FB < 1.8m, Failure MH Flooding
- Existing MH Flooding, Failure MH Flooding

0 100 200 Meters

1:5,000 (At Original document size of 11x17)

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C3 WATER

Project Location 165640298

City of Guelph

Client/Project

CITY OF GUELPH WATER AND WASTEWATER SERVICING MASTER PLAN – TM1 BACKGROUND REVIEW

Figure No.

4.20

Title

Pipe Failure Analysis Results Location #18

Notes

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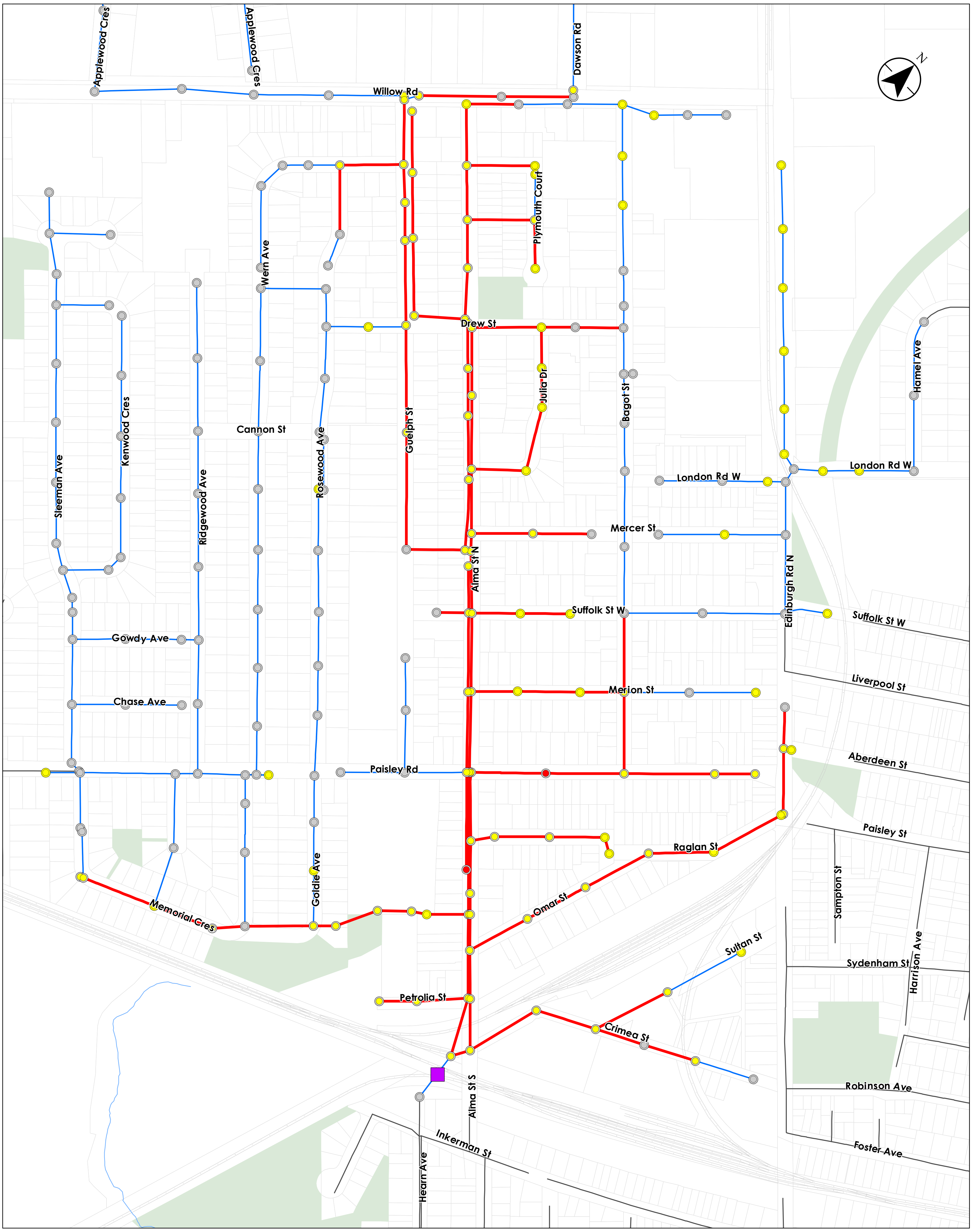

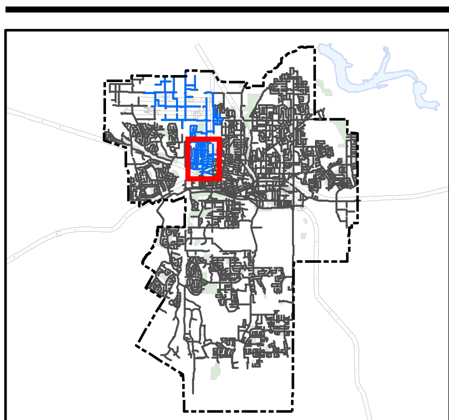

Legend

- Sanitary Sewer
- Railway
- Watercourse
- Property
- Parks

Site 19 Tributary Sewers

- No Surcharge
- Surcharging (d / D >= 1)

Failure Location #19

MH Surcharging

- Existing MH FB >= 1.8m, Failure MH FB >= 1.8m
- Existing MH FB >= 1.8m, Failure MH FB < 1.8m
- Existing MH FB >= 1.8m, Failure MH Flooding
- Existing MH FB < 1.8m, Failure MH FB < 1.8m
- Existing MH FB < 1.8m, Failure MH Flooding
- Existing MH Flooding, Failure MH Flooding

0 100 200 Meters

1:5,000 (At Original document size of 11x17)

Stantec C3 WATER

Project Location 165640298

City of Guelph

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CITY OF GUELPH WATER AND WASTEWATER SERVICING MASTER PLAN – TM1 BACKGROUND REVIEW

Figure No.

4.21

Title

Pipe Failure Analysis Results Location #19

Notes

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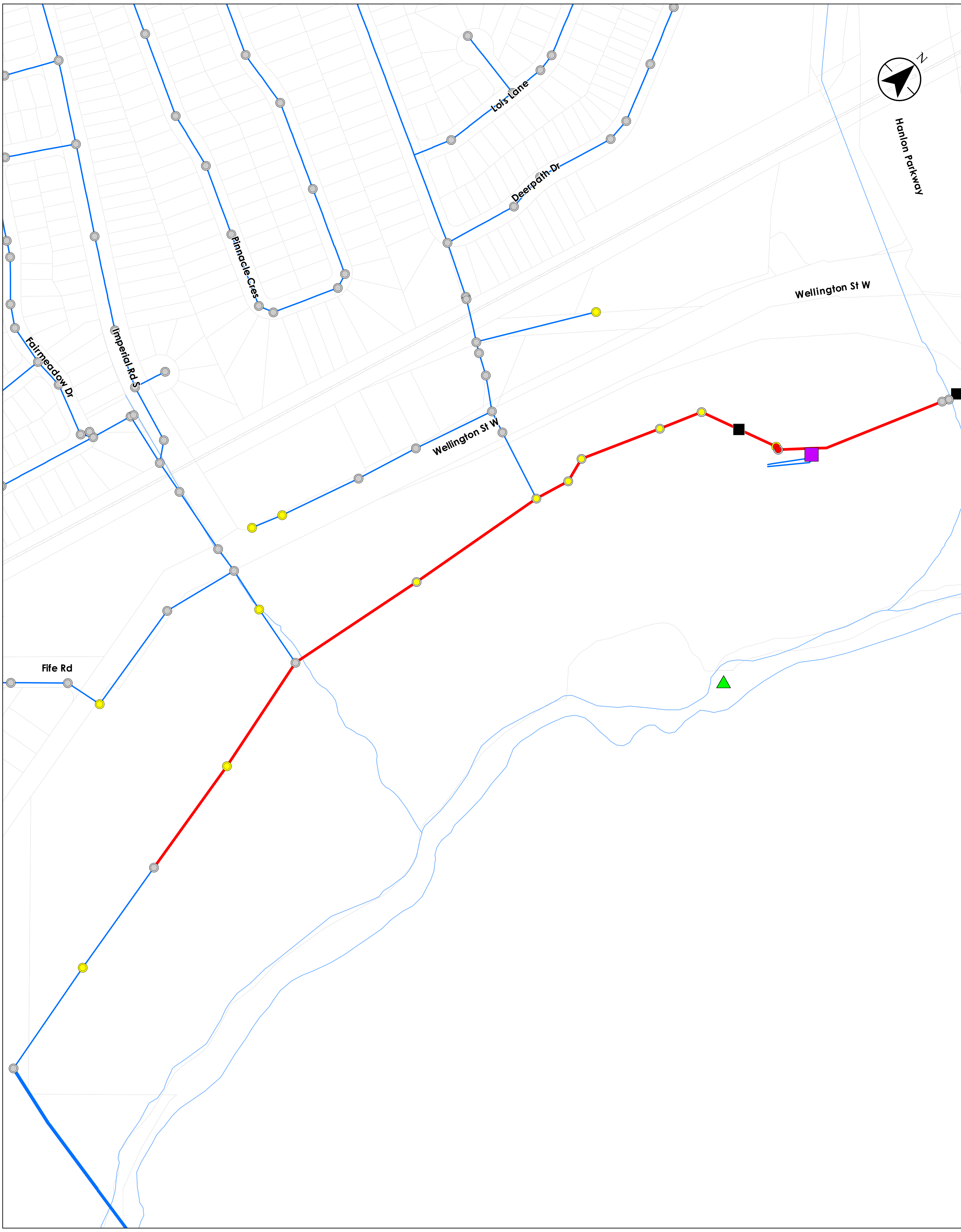

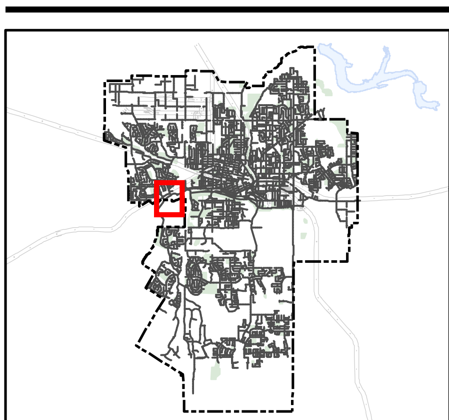

Legend

- Wastewater Treatment Plant
- Sanitary Sewer
- Railway
- Watercourse
- Property

Site 20 Tributary Sewers

- No Surcharge
- Surcharging (d / D >= 1)

- Failure Location #20
- Other Failure Site

MH Surcharging

- Existing MH FB >= 1.8m, Failure MH FB >= 1.8m
- Existing MH FB >= 1.8m, Failure MH FB < 1.8m
- Existing MH FB >= 1.8m, Failure MH Flooding
- Existing MH FB < 1.8m, Failure MH FB < 1.8m
- Existing MH FB < 1.8m, Failure MH Flooding
- Existing MH Flooding, Failure MH Flooding

0 100 200 Meters

1:4,000 (At Original document size of 11x17)

Notes

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C3 WATER

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CITY OF GUELPH WATER AND WASTEWATER SERVICING MASTER PLAN – TM1 BACKGROUND REVIEW

Figure No.

4.22

Title

Pipe Failure Analysis Results Location #20

4.2.3 DWF Analysis

The DWF scenario considers wastewater generated from the residential population and institutional, commercial, and industrial (ICI) areas, as well as baseflow as calibrated.

4.2.3.1 DWF Existing Conditions Analysis

The attached Figure 4-23 shows the overall results for the entire wastewater system under existing DWF conditions. The LOS assessment focused on sewers operating above the Manning's Full Pipe Capacity (FPC), and any MHs that show surcharge above the pipe obvert. As shown on Figure 4-23, there are 2 locations with multiple sewers operating at or near FPC.

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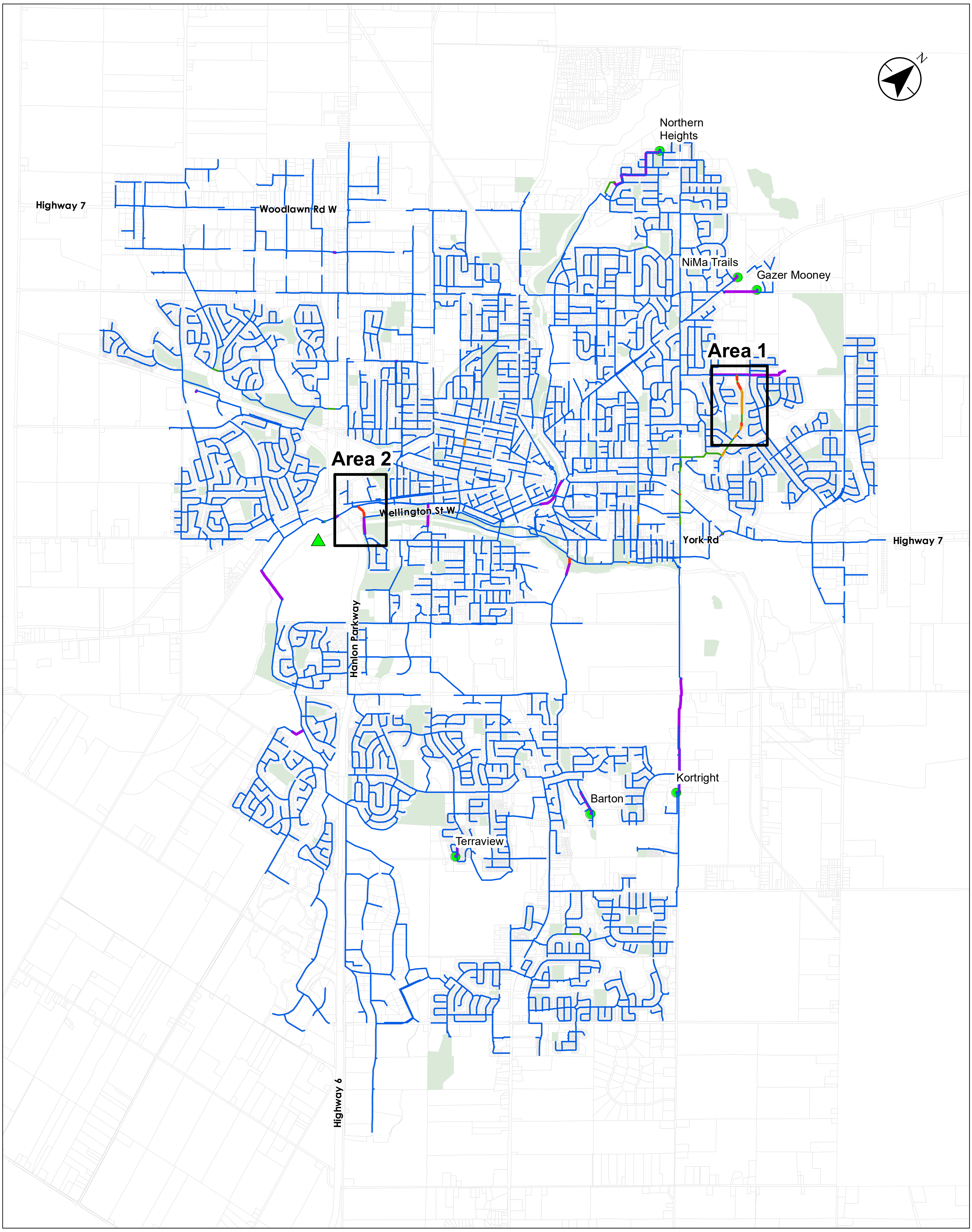

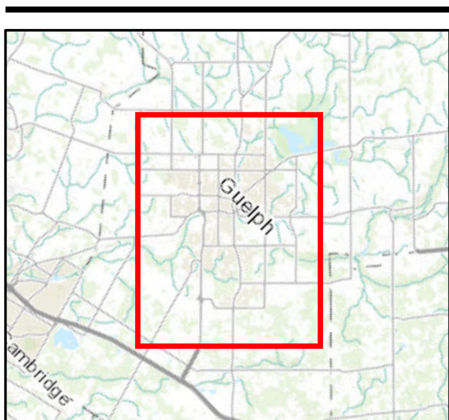

Legend

- Active Pump Station
- Wastewater Treatment Plant
- Property
- Forcemain / Siphon

q / Q

- <= 50%
- > 50% - 80%
- > 80% - 100%
- > 100%

0 1,000 2,000 Meters
1:50,000 (At Original document size of 11x17)

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C3 WATER

Project Location
City of Guelph

165640298

Client/Project
CITY OF GUELPH WATER AND WASTEWATER SERVICING MASTER PLAN – TM3A EXISTING AND FUTURE SYSTEM CAPACITY

Figure No.
4.23

Title
Existing System DWF Results

Notes

1. Coordinate System: NAD 1983 UTM Zone 17N
2. Base features produced under license with the Ontario Ministry of Natural Resources and Forestry © Queen's Printer for Ontario, 2018. Sources: Esri, HERE, Garmin, Intermap, increment P Corp., GEBCO, USGS, FAO, NPS, NRCAN, GeoBase, IGN, Kadaster NL, Ordnance Survey, Esri Japan, METI, Esri China (Hong Kong), (c) OpenStreetMap contributors, and the GIS User Community

The first location, noted as Area 1 on Figure 4-23 is a reach of 200 mm diameter sewers along Auden Road south east of Eastview Road, as shown in more detail below.

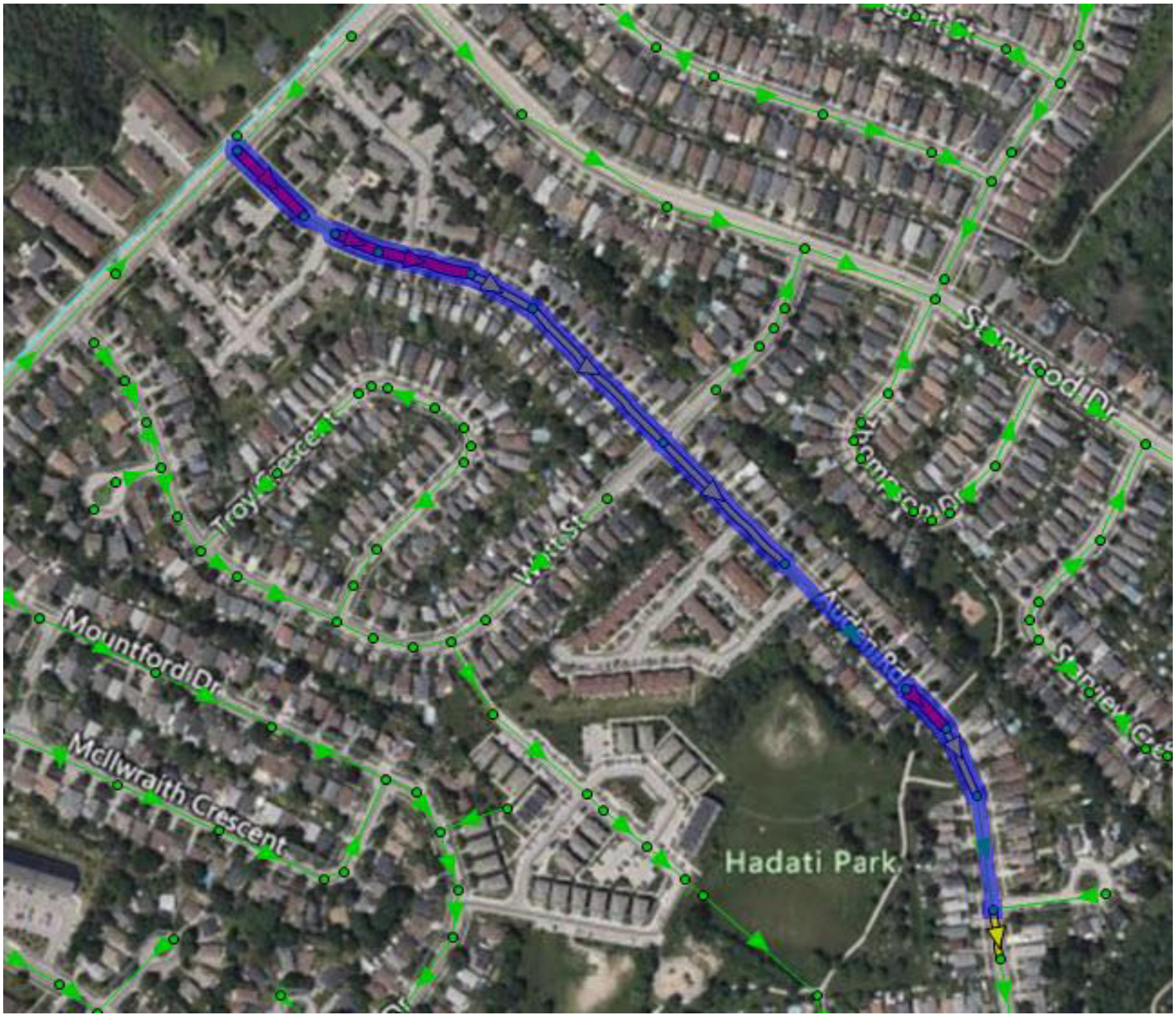


Figure 4-24 Existing Conditions DWF Results - Area 1 Site Plan

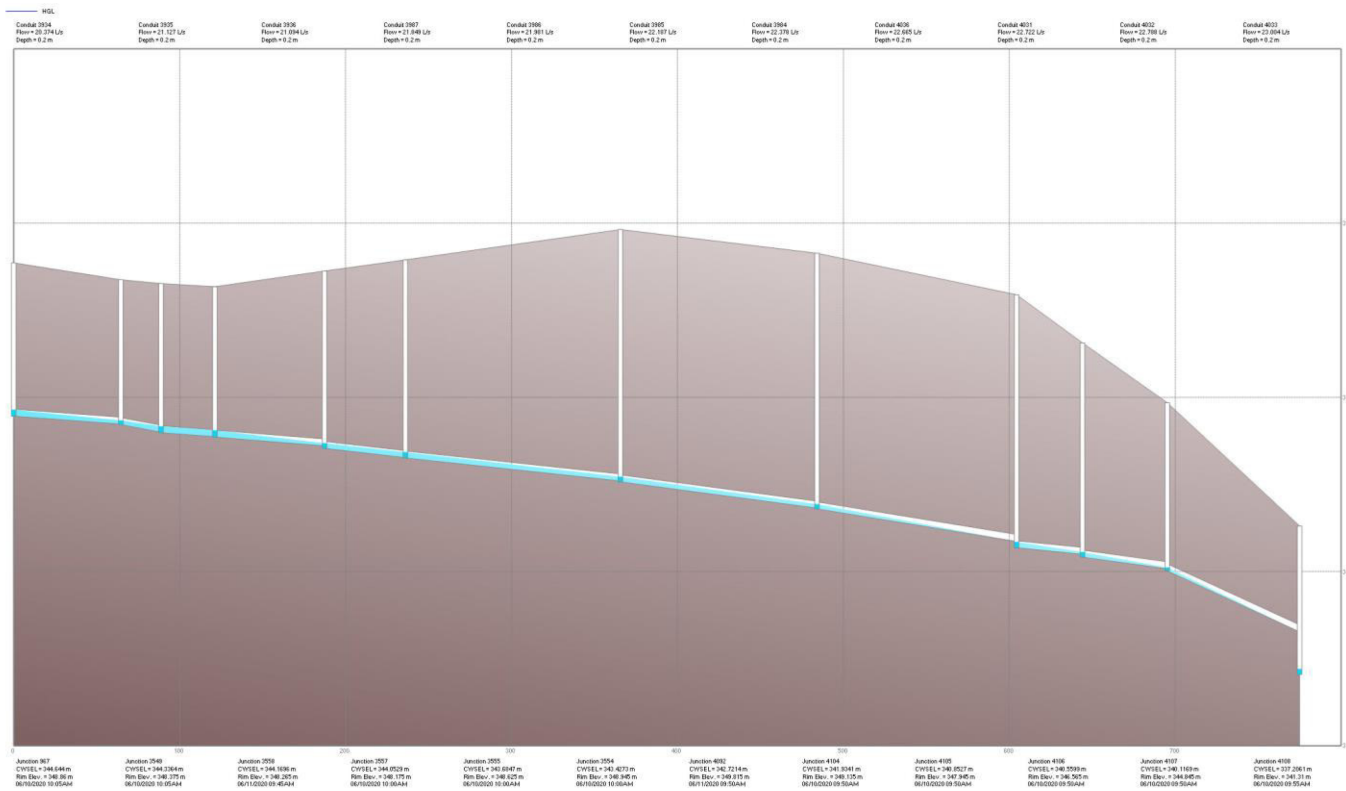


Figure 4-25 Existing Conditions DWF Results - Area 1 HGL Profile

As shown above on Figure 4-24, there are 4 sewers in close proximity with flows above the FPC. The model has shown these flowing slightly above capacity and results in only limited surcharge.

The second location noted as Area 2, is a short reach 1350 mm diameter sewers crossing Wellington Street W near the Hanlon Expressway, as shown in more detail below.

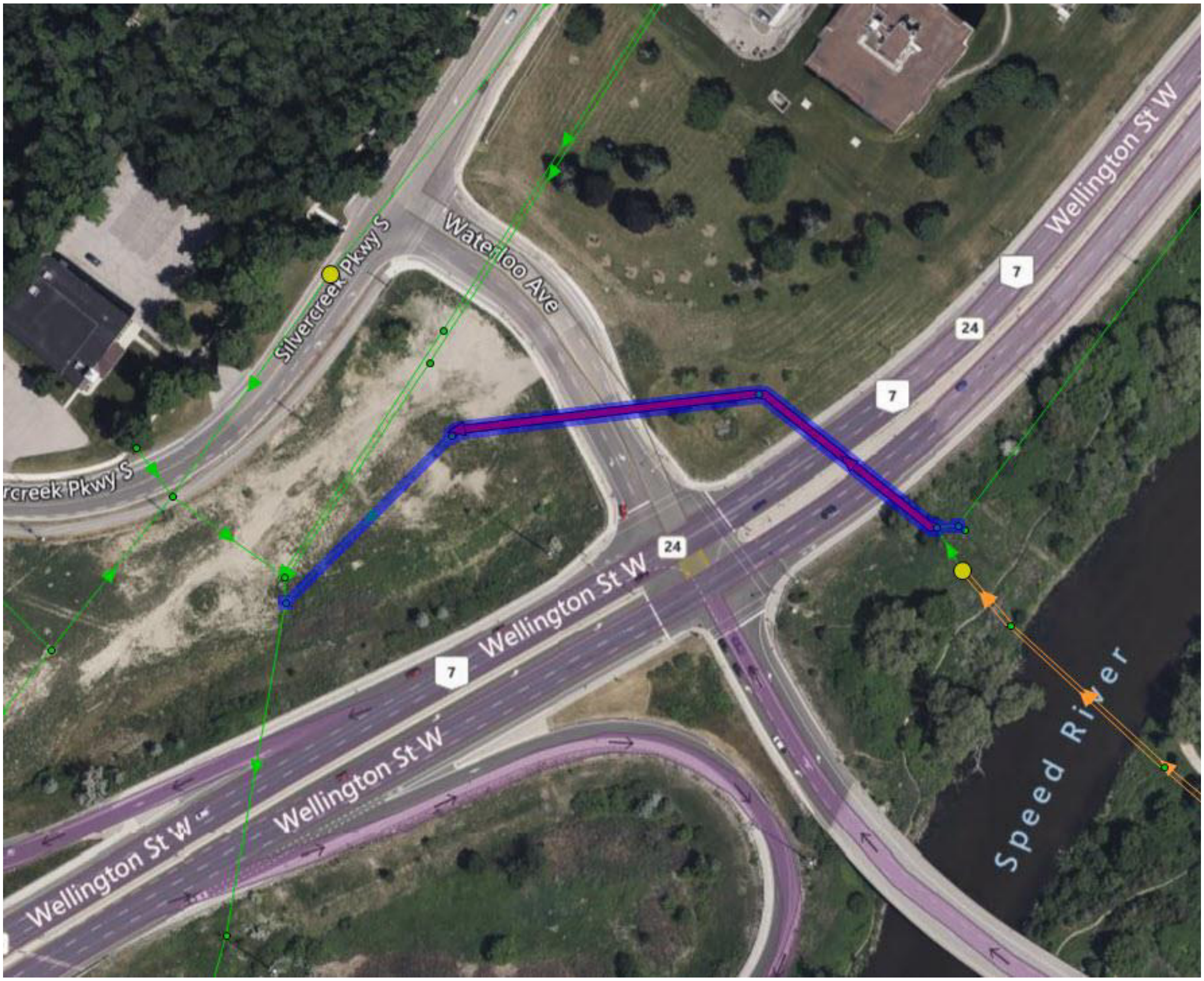


Figure 4-26 Existing Conditions DWF Results - Area 2 Site Plan

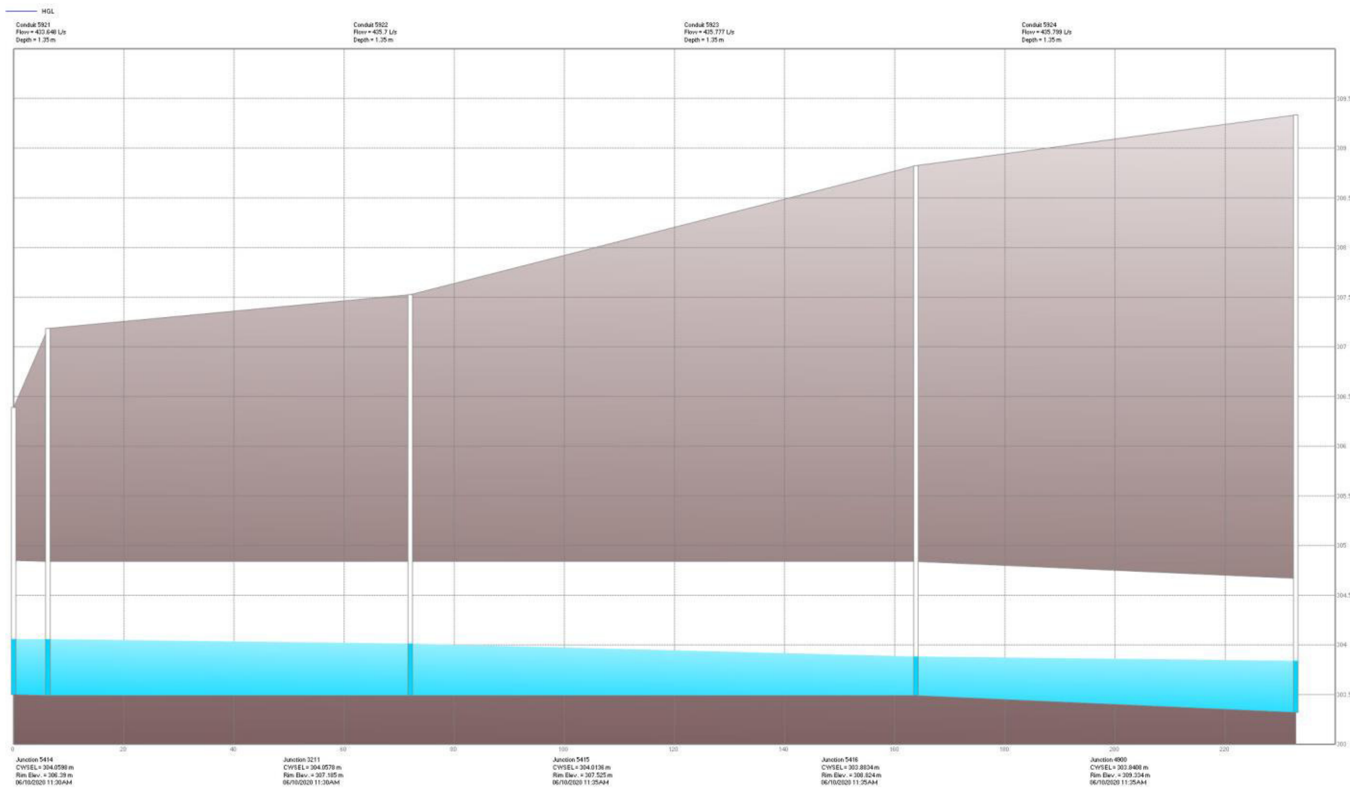


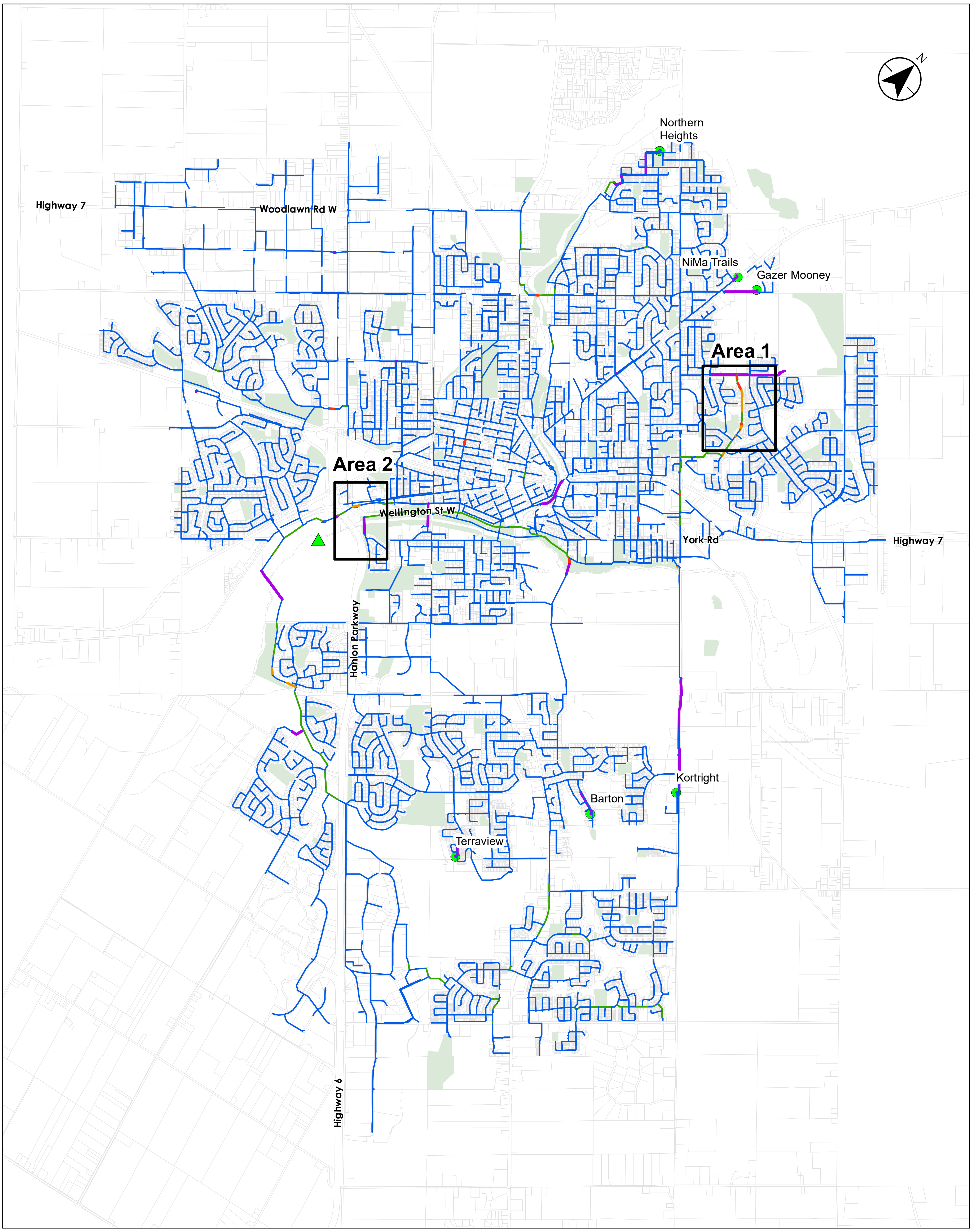
Figure 4-27 Existing Conditions DWF Results - Area 2 HGL Profile

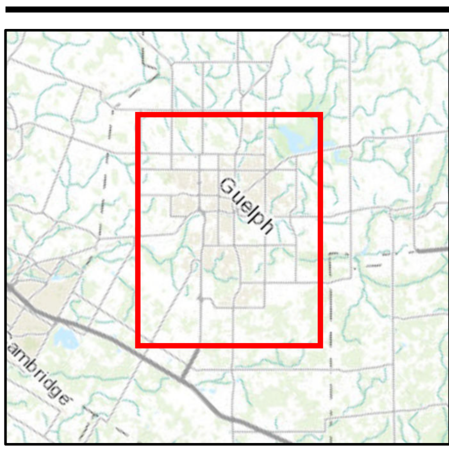
At this location, there are 2 sewers with 0% slope, and the slope of the sewer dictates the Manning's Full Pipe Capacity. Therefore, these sewers will always appear to be over capacity. We recommend confirming the inverts along this reach.

No additional sewers show any capacity constraints under DWF conditions.

4.2.3.2 DWF Future Conditions Analysis

The attached Figure 4-28 shows the overall results for the entire wastewater system under future DWF conditions. The LOS assessment focused on sewers operating above the Manning's Full Pipe Capacity (FPC), and any MHs that show surcharge above the pipe obvert. As shown on Figure 4-28, there are 2 locations with multiple sewers operating at or near FPC. These locations correspond with those identified in the Existing Conditions Assessment and are being reviewed to confirm any change to the results.





Legend

- Active Pump Station
- Wastewater Treatment Plant
- Property
- Forcemain / Siphon

q / Q

- <= 50%
- > 50% - 80%
- > 80% - 100%
- > 100%

0 1,000 2,000 Meters

1:50,000 (At Original document size of 11x17)

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C3 WATER

Project Location
City of Guelph

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Client/Project
CITY OF GUELPH WATER AND WASTEWATER SERVICING MASTER PLAN – TM3A EXISTING AND FUTURE SYSTEM CAPACITY

Figure No.
4.28

Title
Future System DWF Results

Notes

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The first location, noted as Area 1 on Figure 4-28 is a reach of 200 mm diameter sewers along Auden Road southeast of Eastview Road, as shown in more detail below.

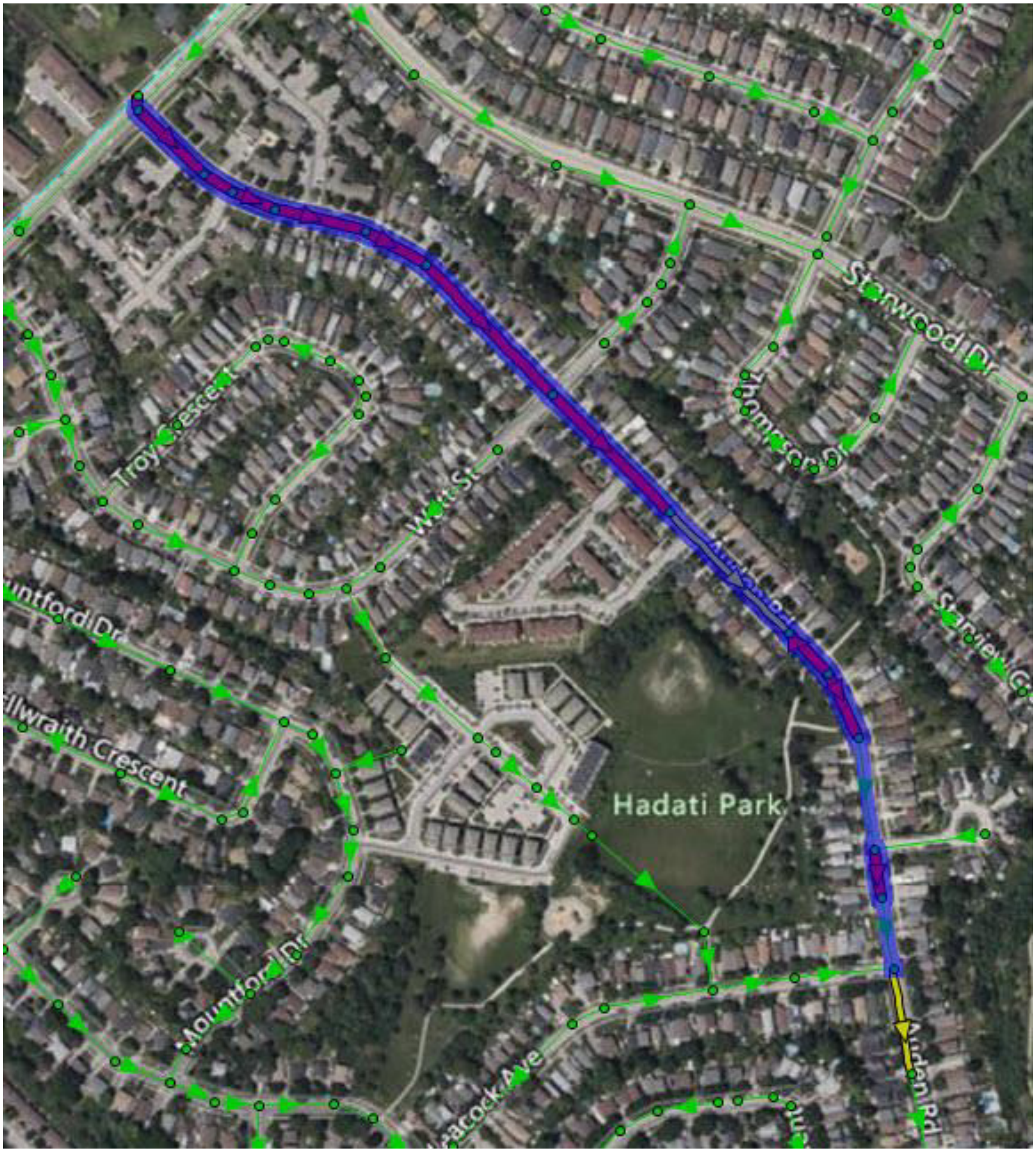


Figure 4-29 Future Conditions DWF Results - Area 1 Site Plan

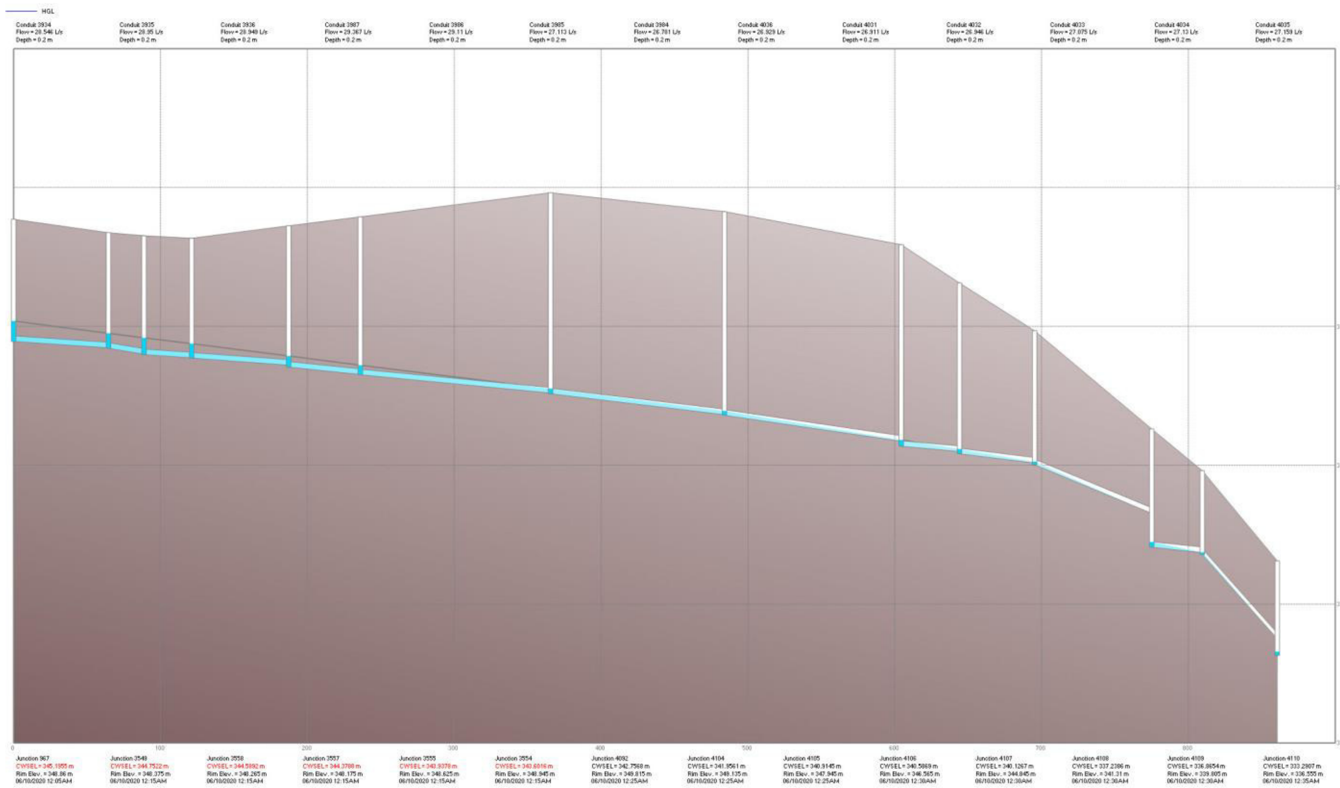


Figure 4-30 Future Conditions DWF Results - Area 1 HGL Profile

This location was identified in the existing conditions assessment, with sewers flowing above FPC, but no surcharging was observed. As shown above under future conditions, there are 9 sewers flowing above FPC, with surcharging observed in 6 MHs and HGLs reaching 0.54 m above the sewer obverts.

The second location noted as Area 2, is a short reach 1350 mm diameter sewers crossing Wellington Street W near the Hanlon Expressway, as shown in more detail below.

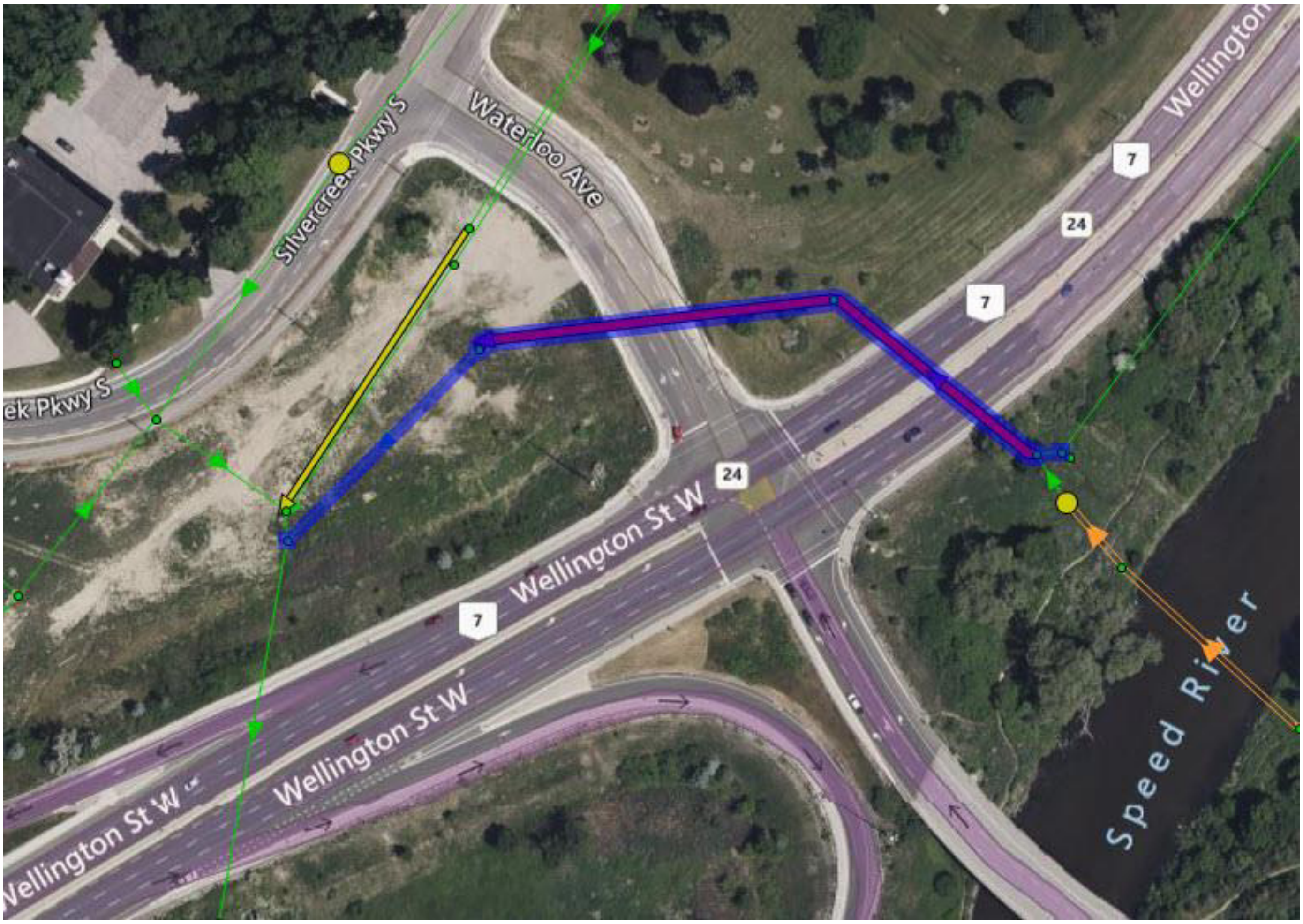


Figure 4-31 Future Conditions DWF Results - Area 2 Site Plan

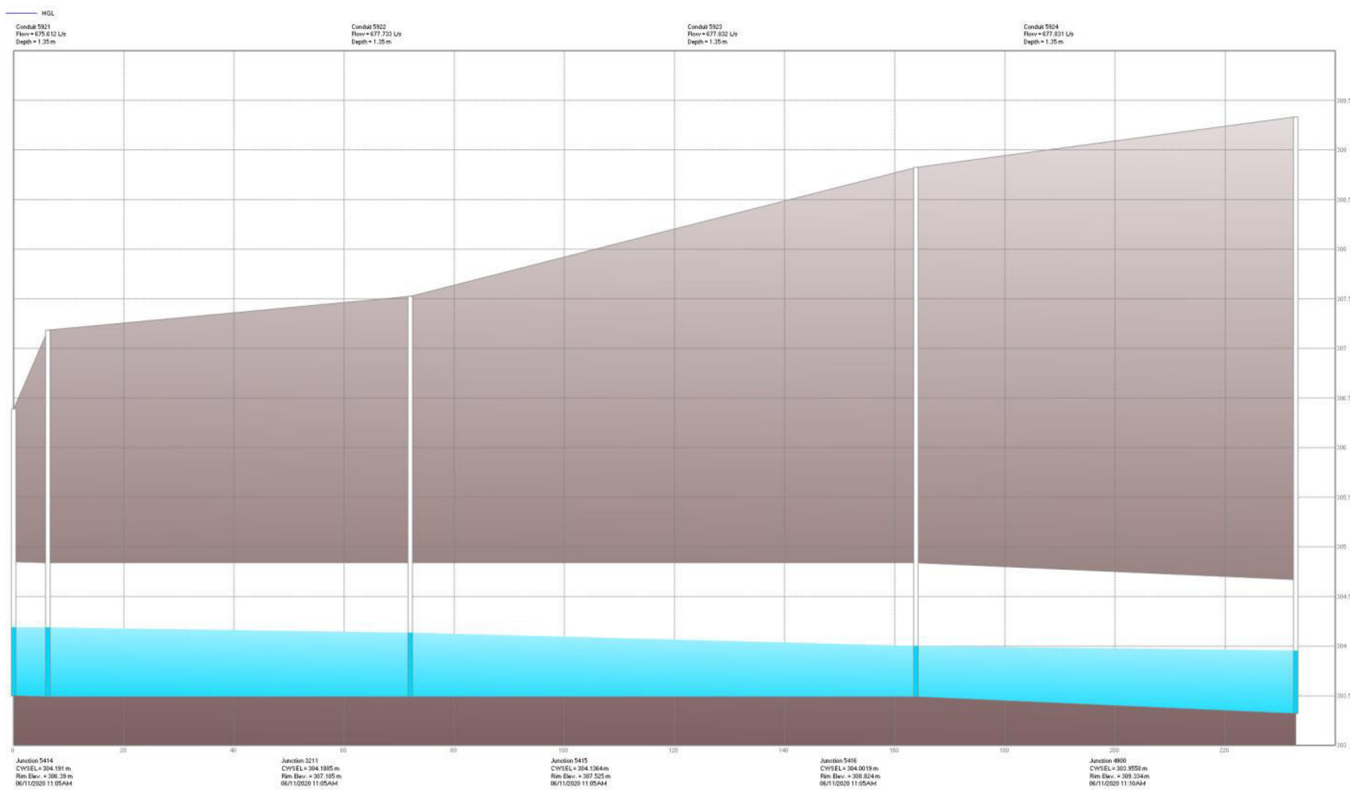


Figure 4-32 Future Conditions DWF Results - Area 2 HGL Profile

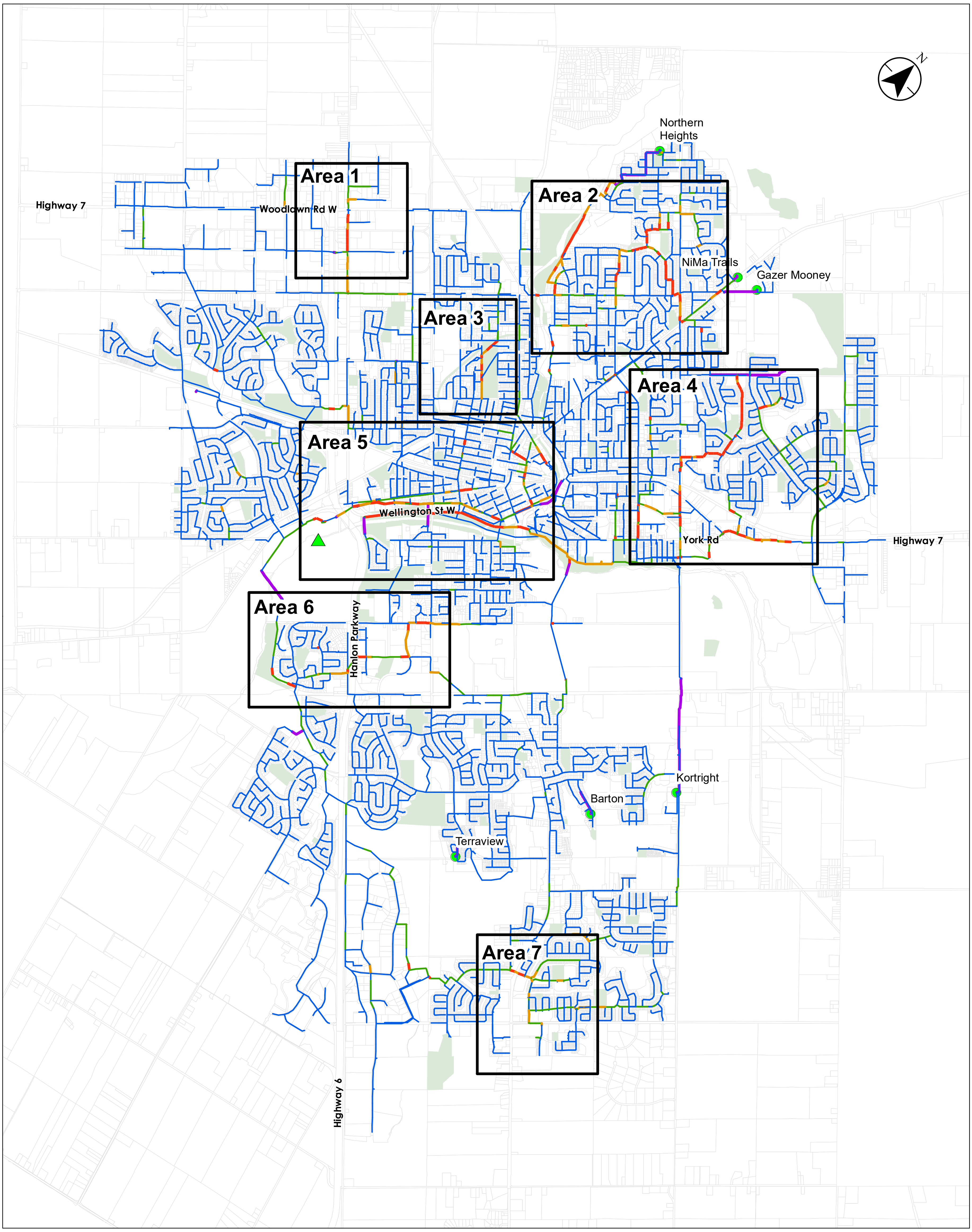
The results at this location are similar to those in the existing conditions assessment. At this location, there are 2 sewers with 0% slope, and the slope of the sewer dictates the Manning's Full Pipe Capacity. Therefore, these sewers will always appear to be over capacity. We recommend confirming the inverts along this reach.

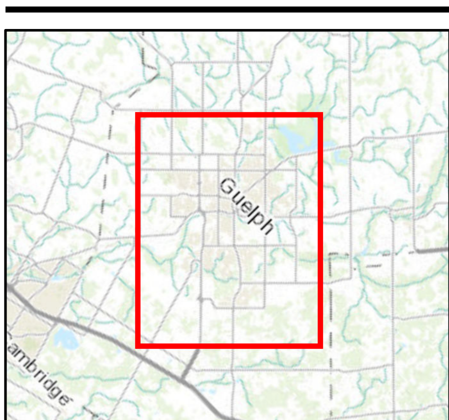
4.2.4 WWF Analysis

The WWF scenario considers wastewater generated from the residential population and institutional, commercial and industrial (ICI) areas, as well as baseflow as calibrated.

4.2.4.1 WWF Existing Conditions Analysis

The attached Figure 4-33 shows the overall results for the entire wastewater system. The assessment focused on sewers operating above the Manning's Full Pipe Capacity (FPC), and any MHs that show surcharge above the pipe obvert. As shown on Figure 4-33, there are 7 locations with multiple sewers operating at or near FPC.





Legend

- Active Pump Station
- Wastewater Treatment Plant
- Property
- Forcemain / Siphon

q / Q

- <= 50%
- > 50% - 80%
- > 80% - 100%
- > 100%

0 1,000 2,000 Meters
1:50,000 (At Original document size of 11x17)

Project Location
City of Guelph

165640298

Client/Project
CITY OF GUELPH WATER AND WASTEWATER SERVICING MASTER PLAN – TM3A EXISTING AND FUTURE SYSTEM CAPACITY

Figure No.
4.33

Title
Existing System WWF Results

Notes

1. Coordinate System: NAD 1983 UTM Zone 17N
2. Base features produced under license with the Ontario Ministry of Natural Resources and Forestry © Queen's Printer for Ontario, 2018. Sources: Esri, HERE, Garmin, Intermap, increment P Corp., GEBCO, USGS, FAO, NPS, NRCAN, GeoBase, IGN, Kadaster NL, Ordnance Survey, Esri Japan, METI, Esri China (Hong Kong), (c) OpenStreetMap contributors, and the GIS User Community

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The first location, noted as Area 1 on Figure 4-33 is a reach of 300 mm and 525 mm diameter sewers along Silvercreek Parkway North between Woodlawn Road West and Speedvale Avenue W, as shown in more detail below.

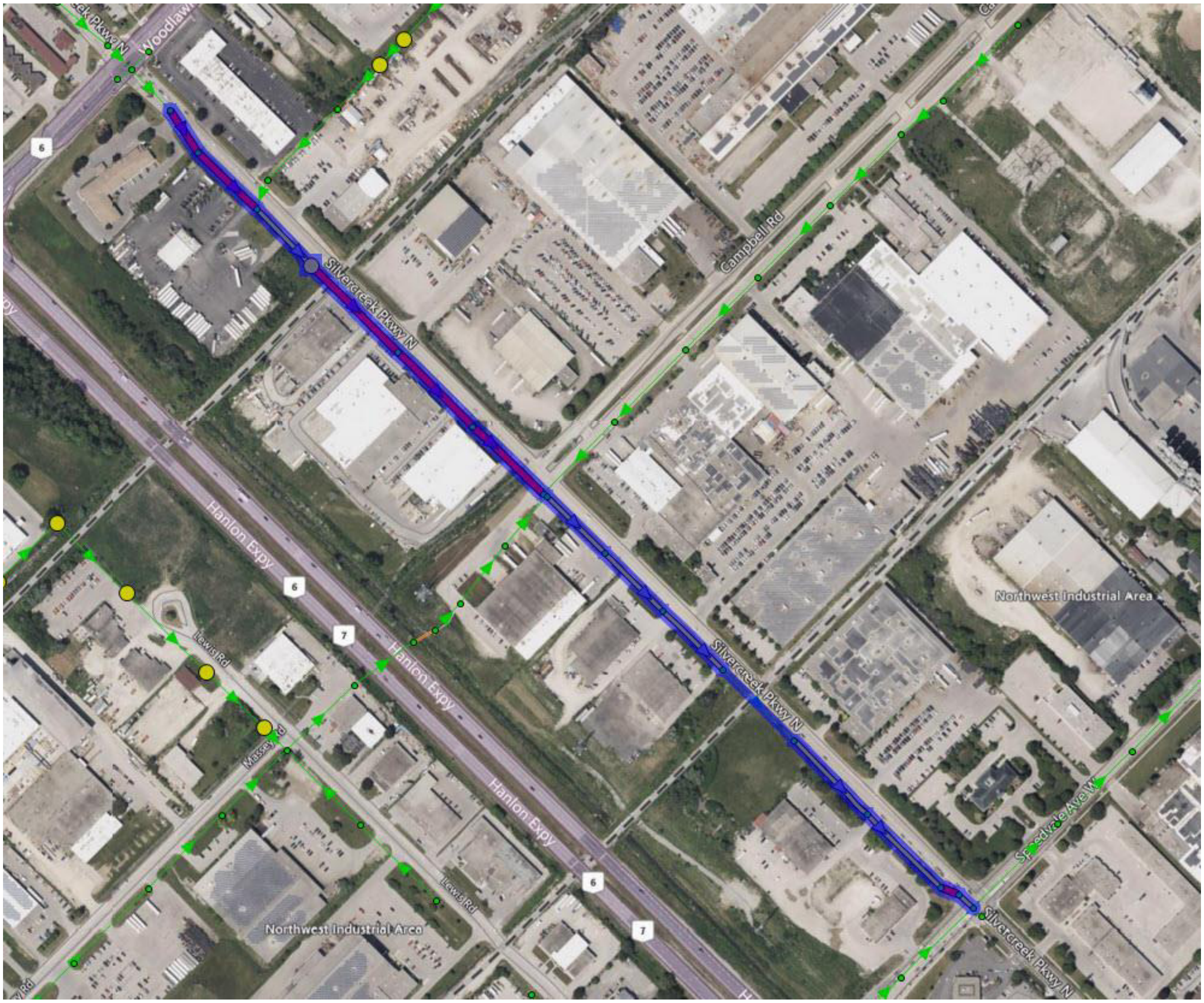


Figure 4-34 Existing Conditions WWF Results - Area 1 Site Plan

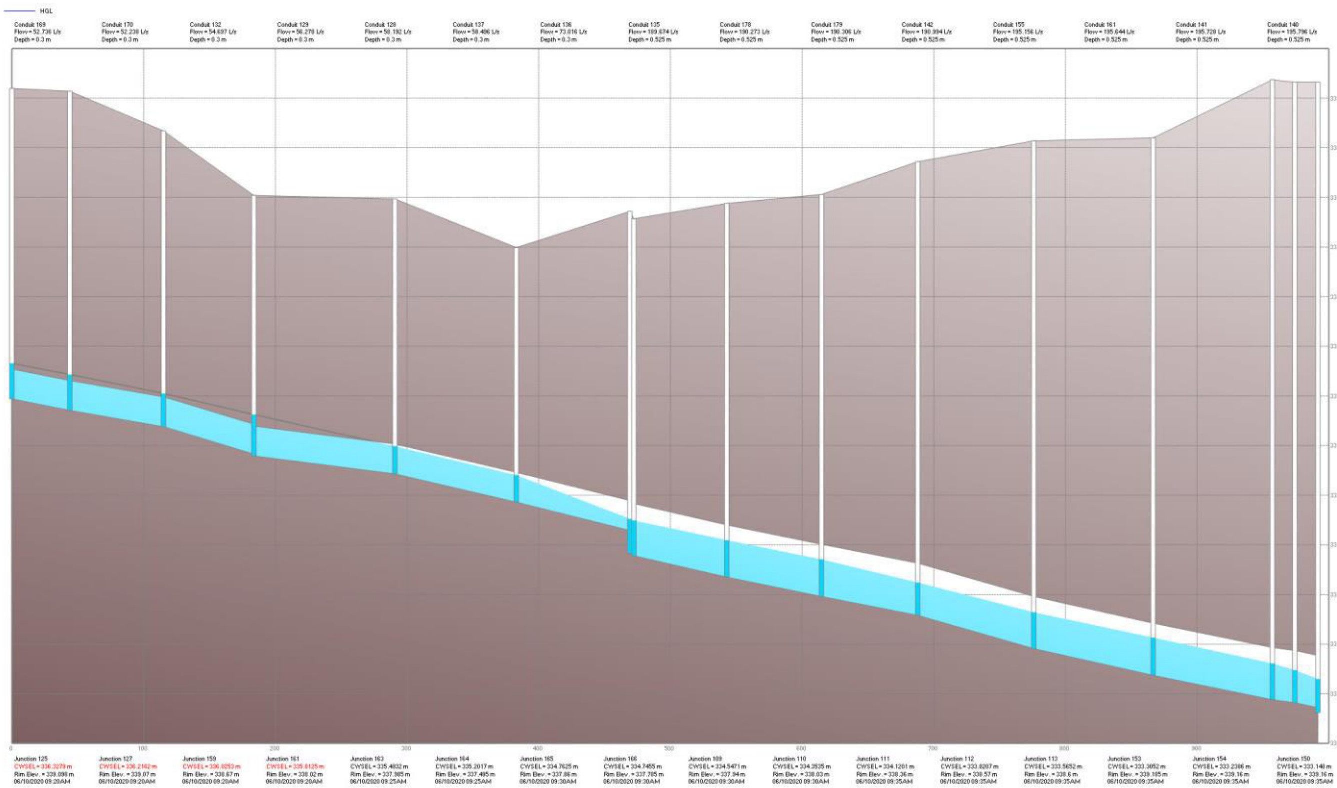


Figure 4-35 Existing Conditions WWF Results - Area 1 HGL Profile

As shown above, there are 6 sewers flowing above FPC, however surcharging is observed in only 4 MHs and the maximum surcharge above the sewer obverts is 0.12 m.

The second location, noted as Area 2 on Figure 4-33 includes multiple reaches in the northeast of the City. The first reach ranges from 225 mm to 375 mm travelling from Eramosa Road northwest along Victoria Road and Waverley Drive, as shown in more detail below.

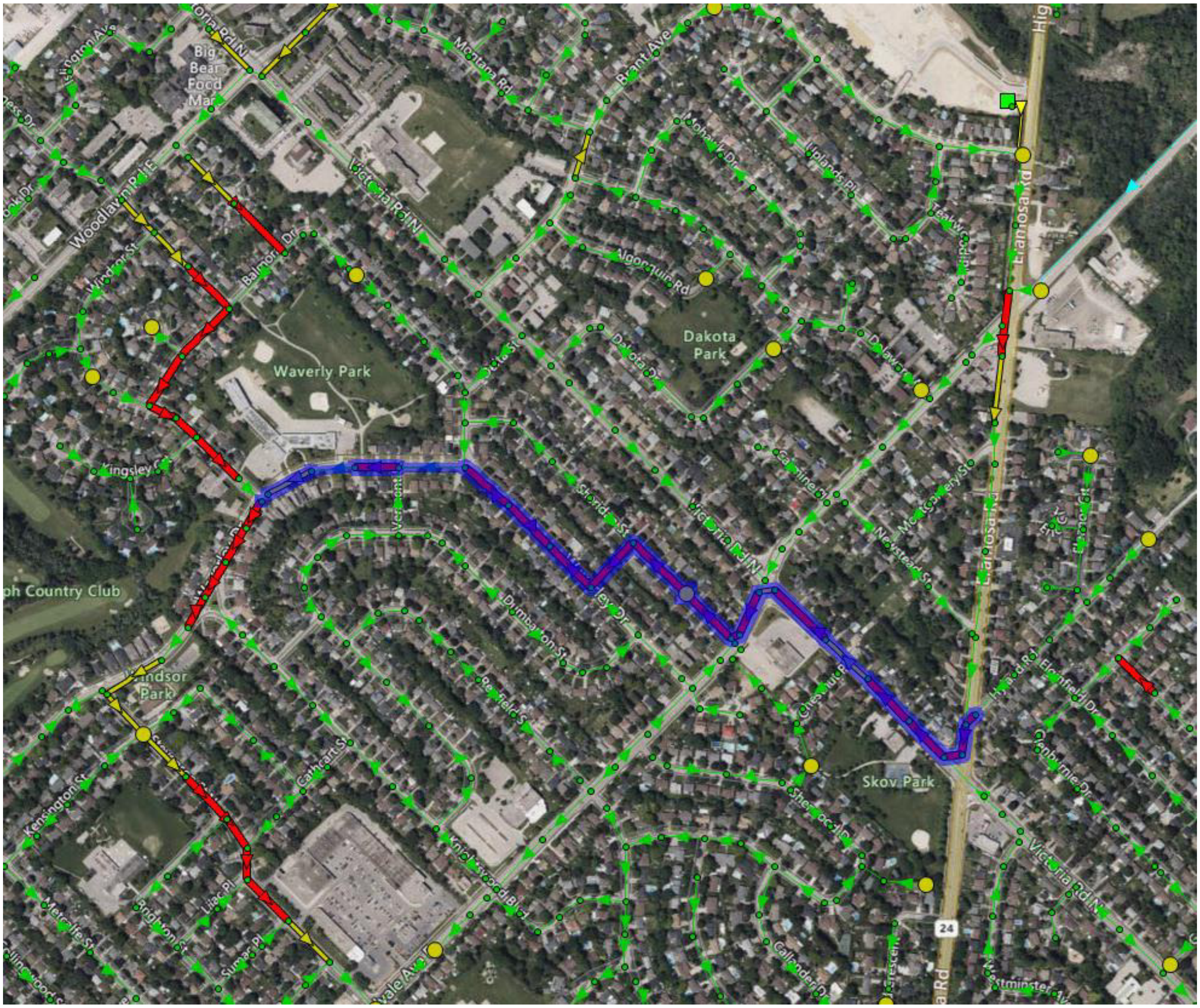


Figure 4-36 Existing Conditions WWF Results - Area 2 Site Plan 1

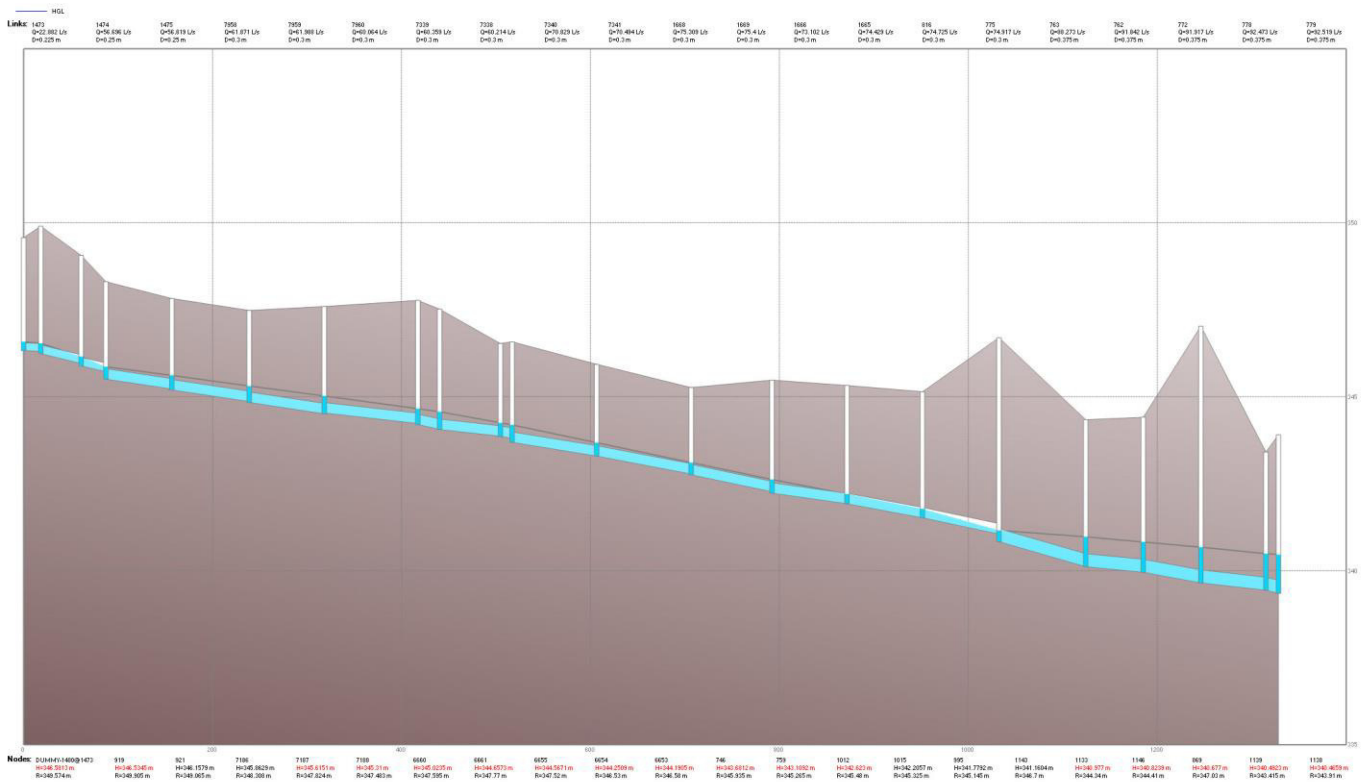


Figure 4-37 Existing Conditions WWF Results - Area 2 HGL Profile 1

As shown above, there are 15 sewers flowing above FPC, with minor surcharging observed in the upper portion of the reach, and HGL's reaching 0.21 m above the sewer obverts, and greater surcharging at the lower portion of the reach with HGL's reaching 0.75 m above the sewer obverts.

The second reach in Area 2 includes 300 mm and 375 mm diameter sewers travelling from Woodlawn Road East to Speedvale Avenue as shown in more detail below.

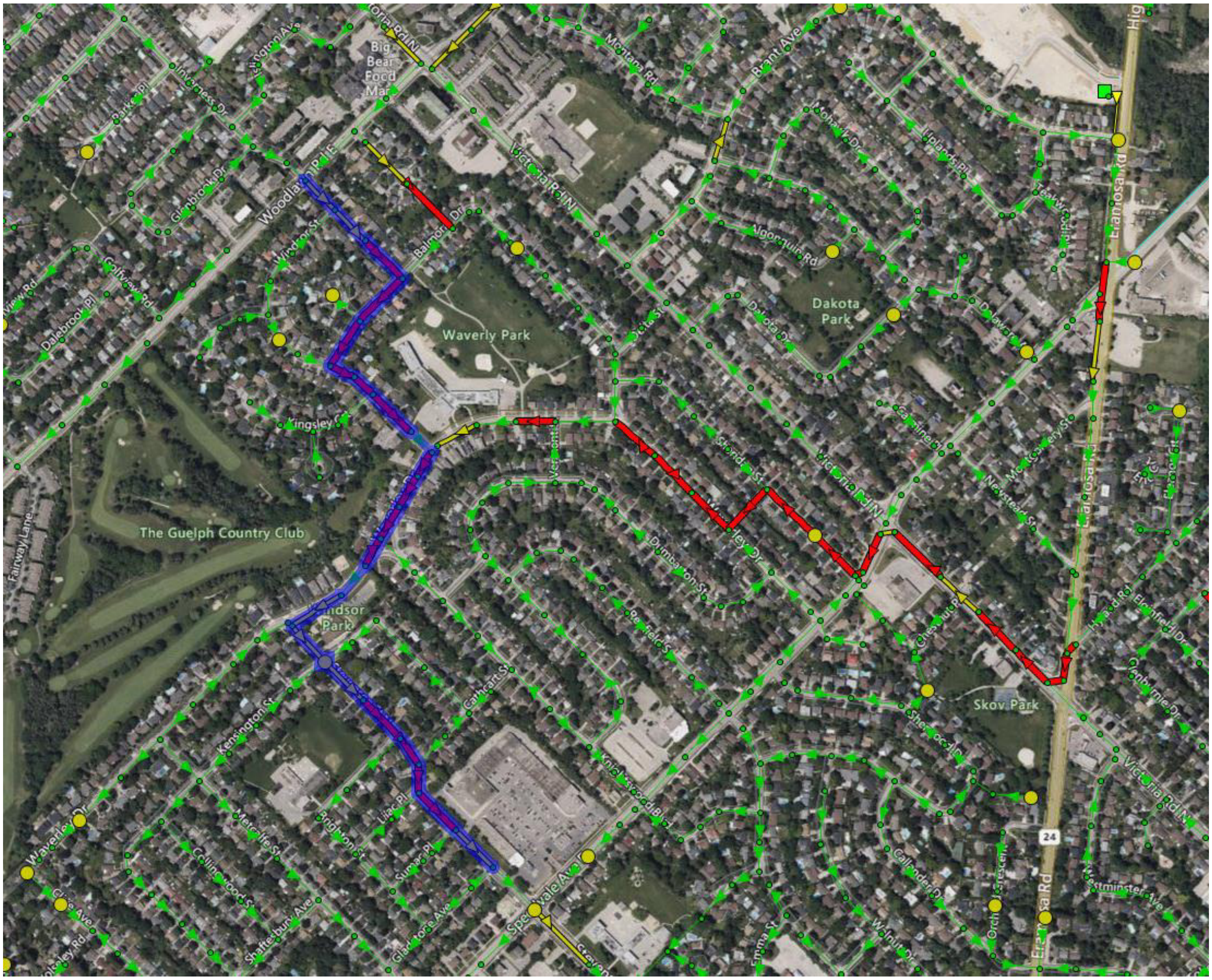


Figure 4-38 Existing Conditions WWF Results - Area 2 Site Plan 2

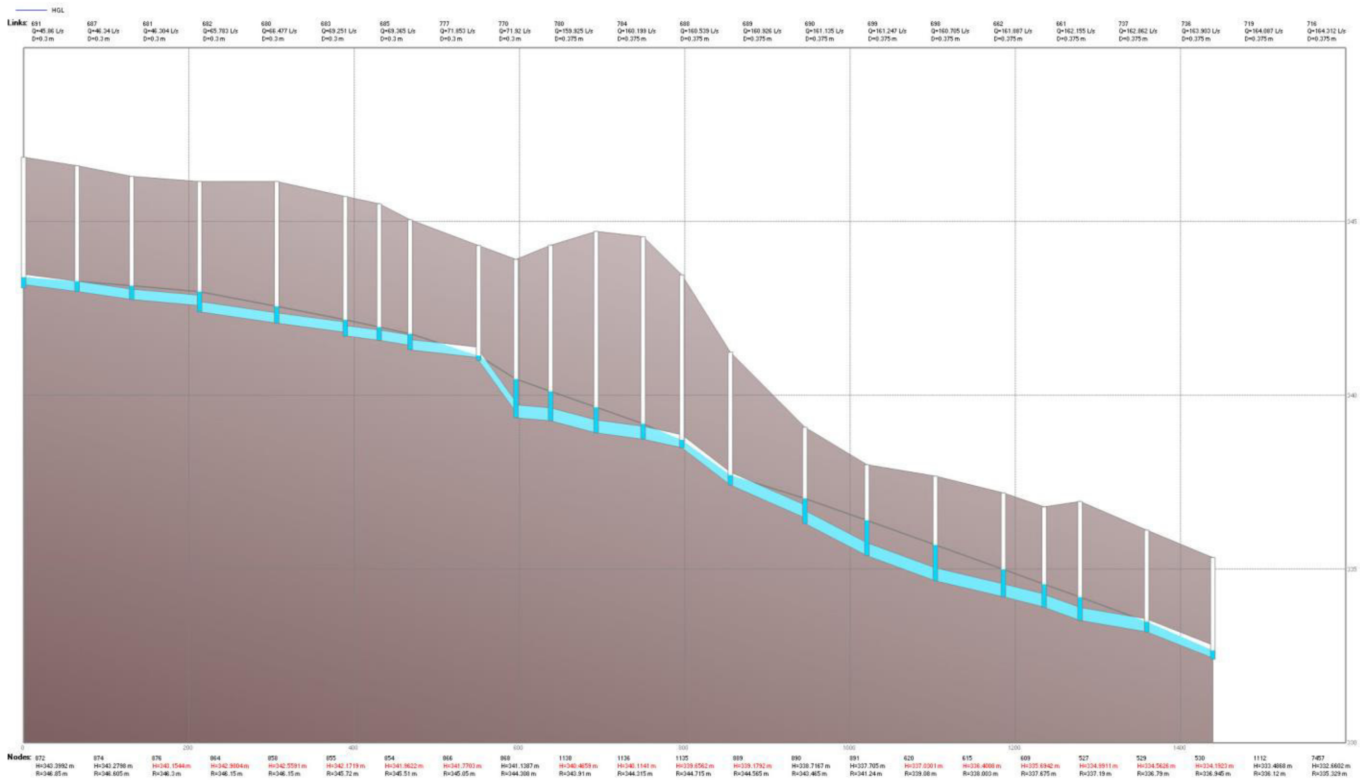


Figure 4-39 Existing Conditions WWF Results - Area 2 HGL Profile 2

As shown above, there are 14 sewers flowing above FPC, with minor surcharging observed in most MHs, with HGLs ranging from 0.05 m to 0.75 m above the sewer obverts. All but one surcharged MH along this reach have greater than 1.8 m of freeboard.

The third reach in Area 2 ranges in diameter from 225 mm to 300 mm sewers travelling from Pondview Crescent to the south across Woodlawn Road East and along Riverview Drive to Speedvale Avenue, shown in more detail below.

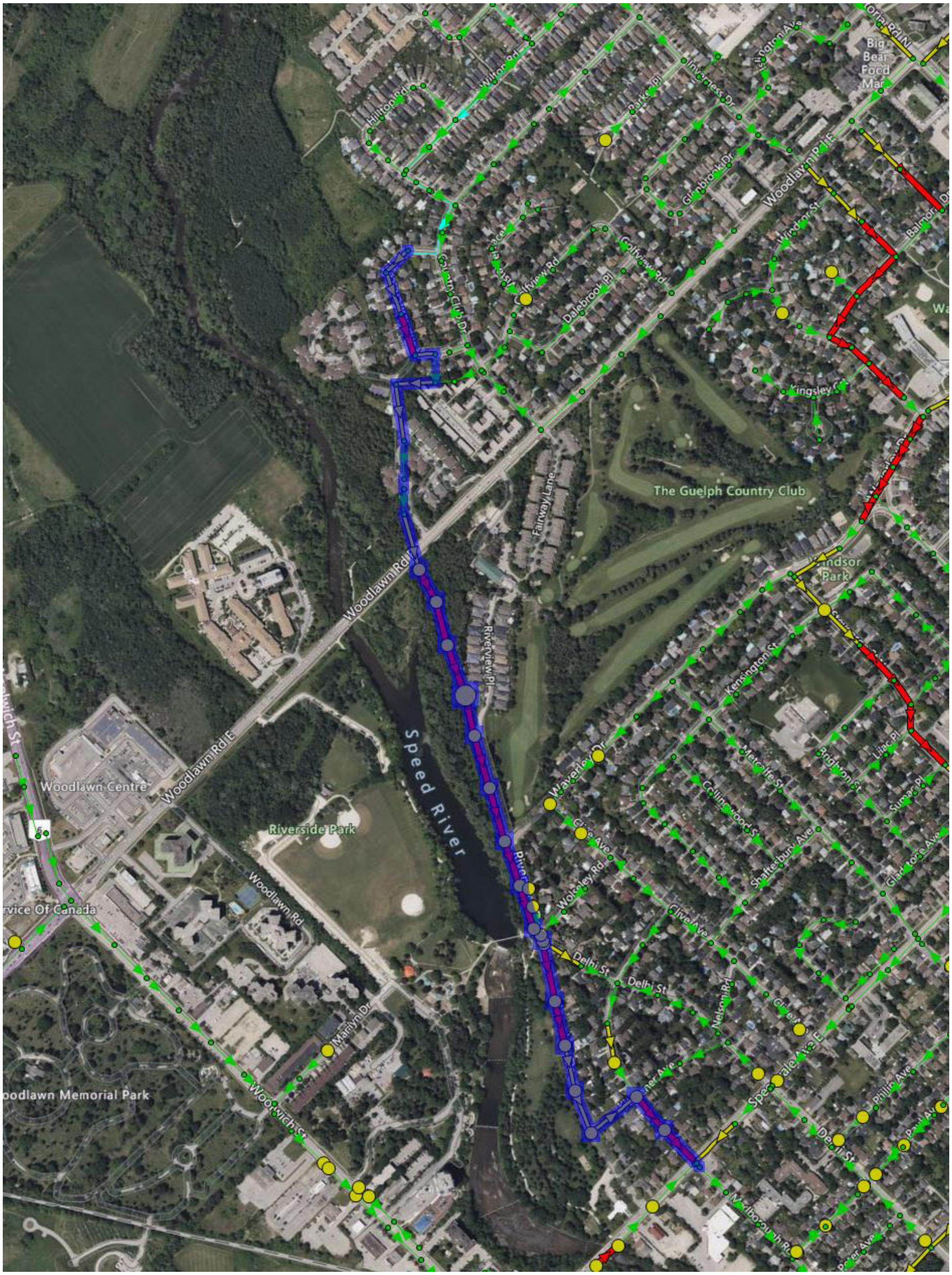


Figure 4-40 Existing Conditions WWF Results - Area 2 Site Plan 2

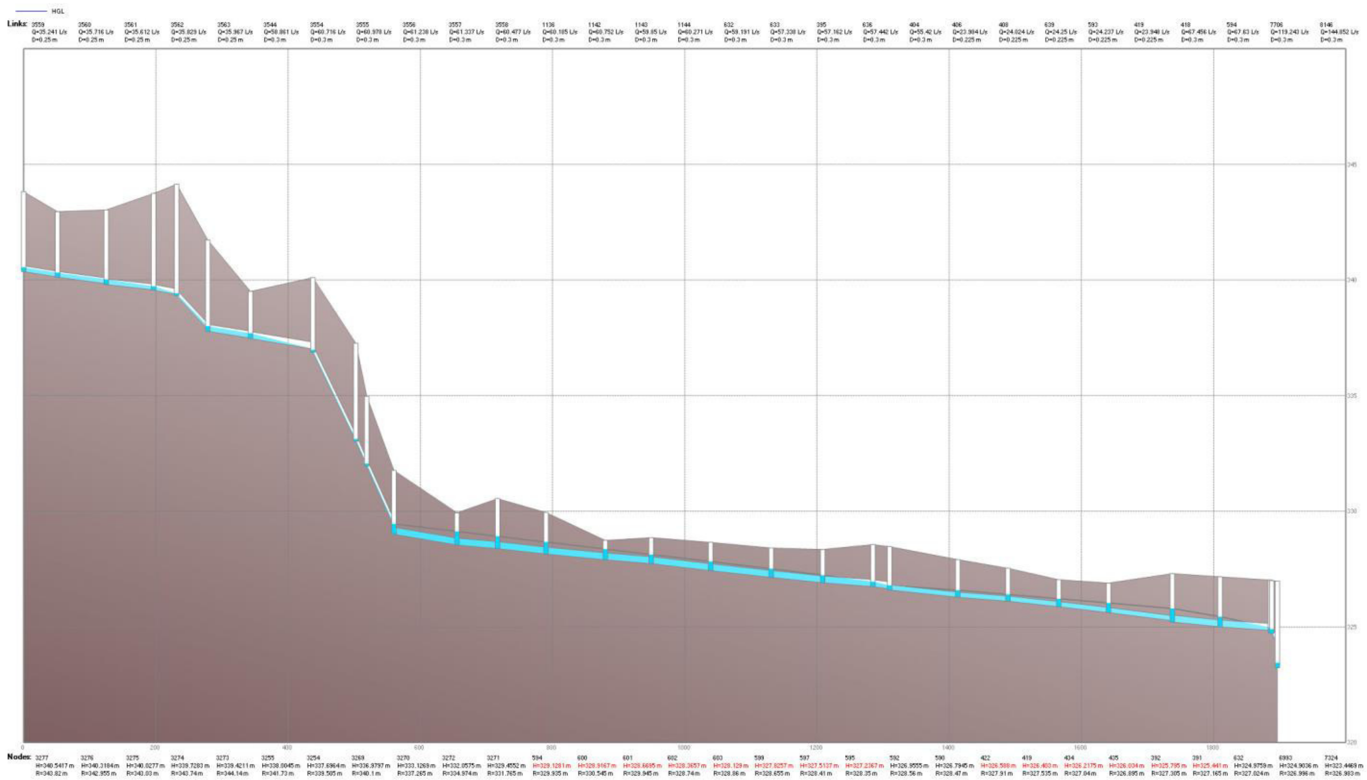


Figure 4-41 Existing Conditions WWF Results - Area 2 HGL Profile 3

As shown above, there are 13 sewers flowing above FPC, with minor surcharging observed in most MHs, with HGLs reaching 0.3 m above the sewer obverts. It should be noted that 15 of the MHs along this reach have less than 1.8 m of cover over the connected sewers, with multiple MHs having less than 1.0 m of cover.

The third location, noted as Area 3 on Figure 4-33 is a reach of 375 mm sewers travelling from Division Street to London Road W along Kathleen Street, as shown in more detail below.

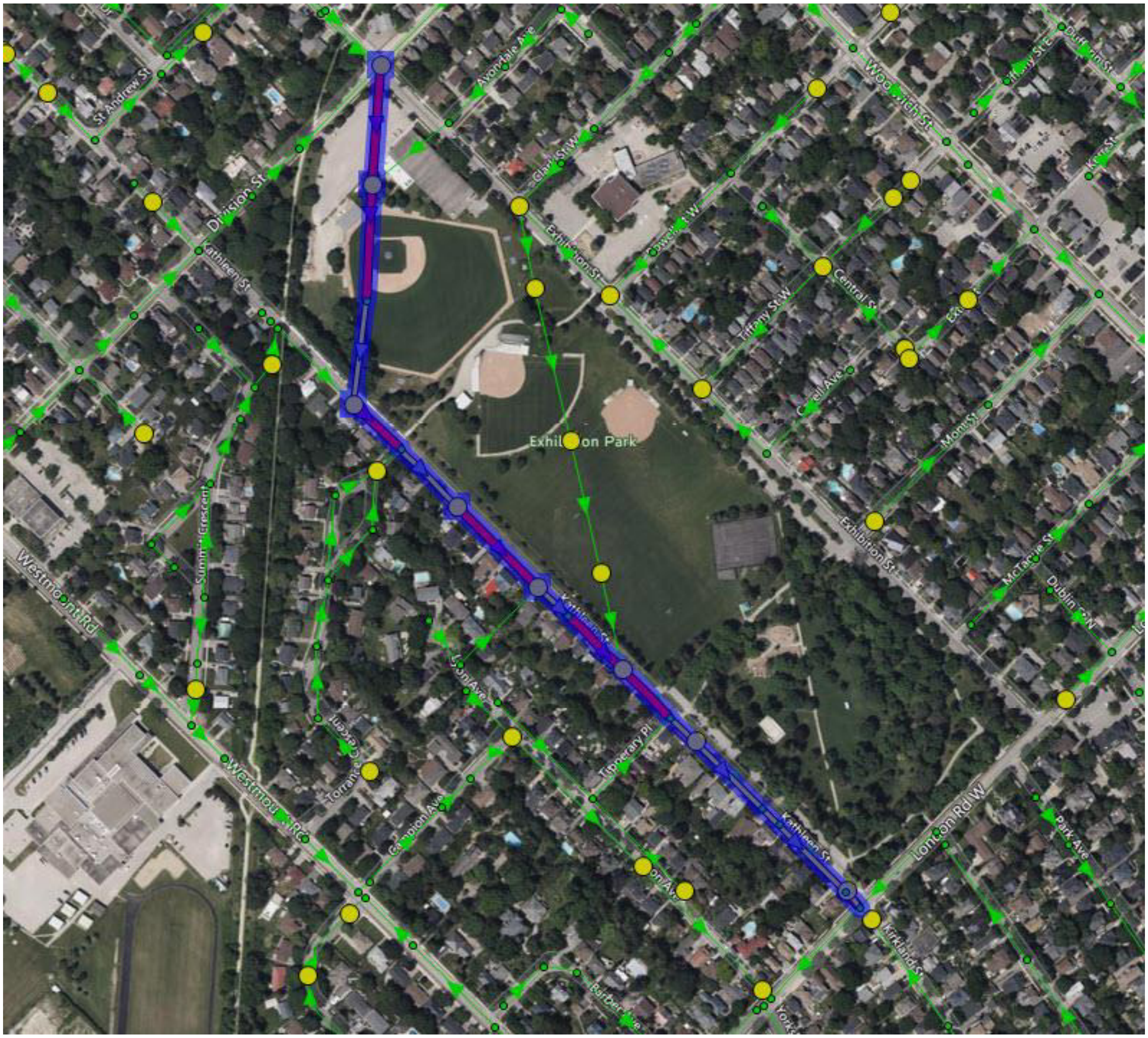


Figure 4-42 Existing Conditions WWF Results - Area 3 Site Plan

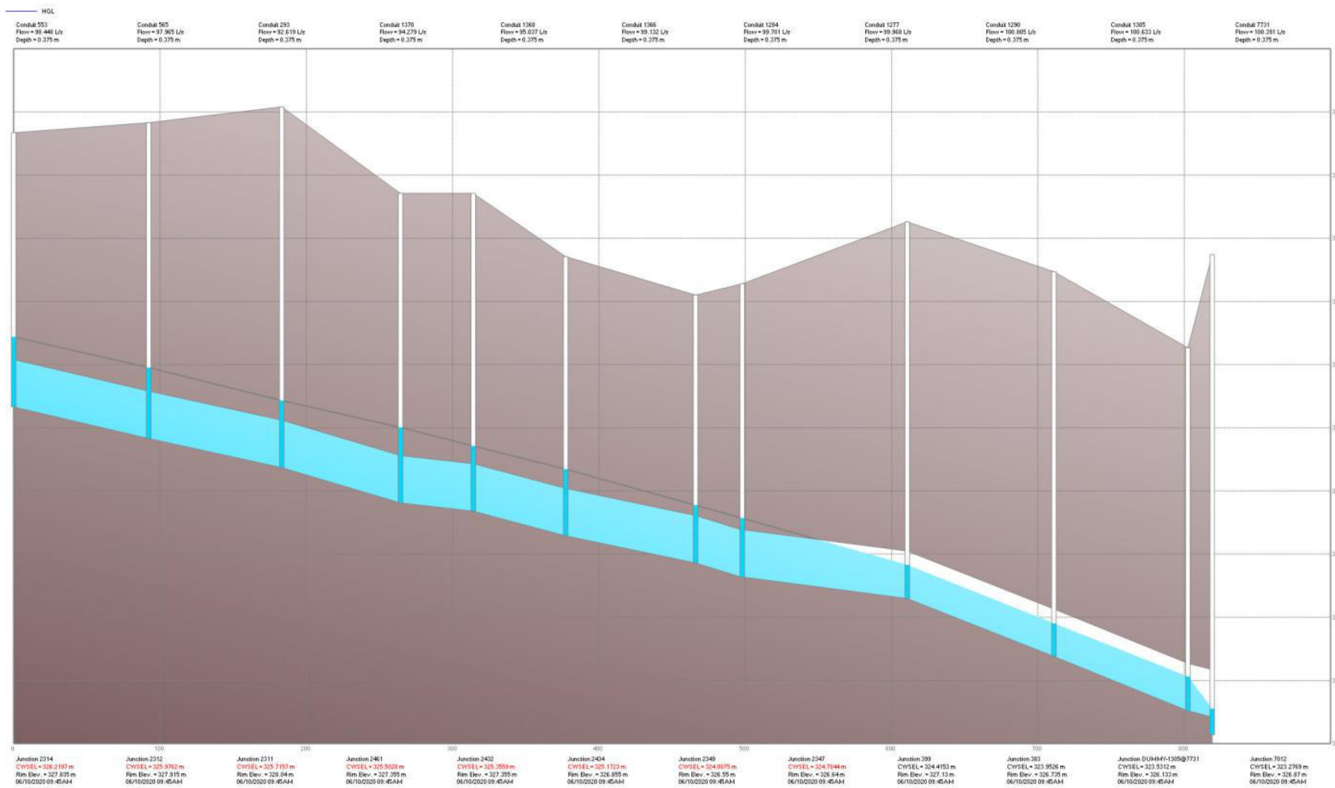


Figure 4-43 Existing Conditions WWF Results - Area 3 HGL Profile

As shown above, there are 6 sewers flowing above FPC, with surcharging observed in 8 MHs, and HGLs reaching 0.23 m above the sewer obverts.

The fourth location, noted as Area 4 on Figure 4-33 includes multiple reaches of sewers in the eastern part of the City. The first reach includes sewers with diameters ranging from 200 mm to 350 mm, travelling from Eastview Road to Victoria Road, as shown in more detail below.

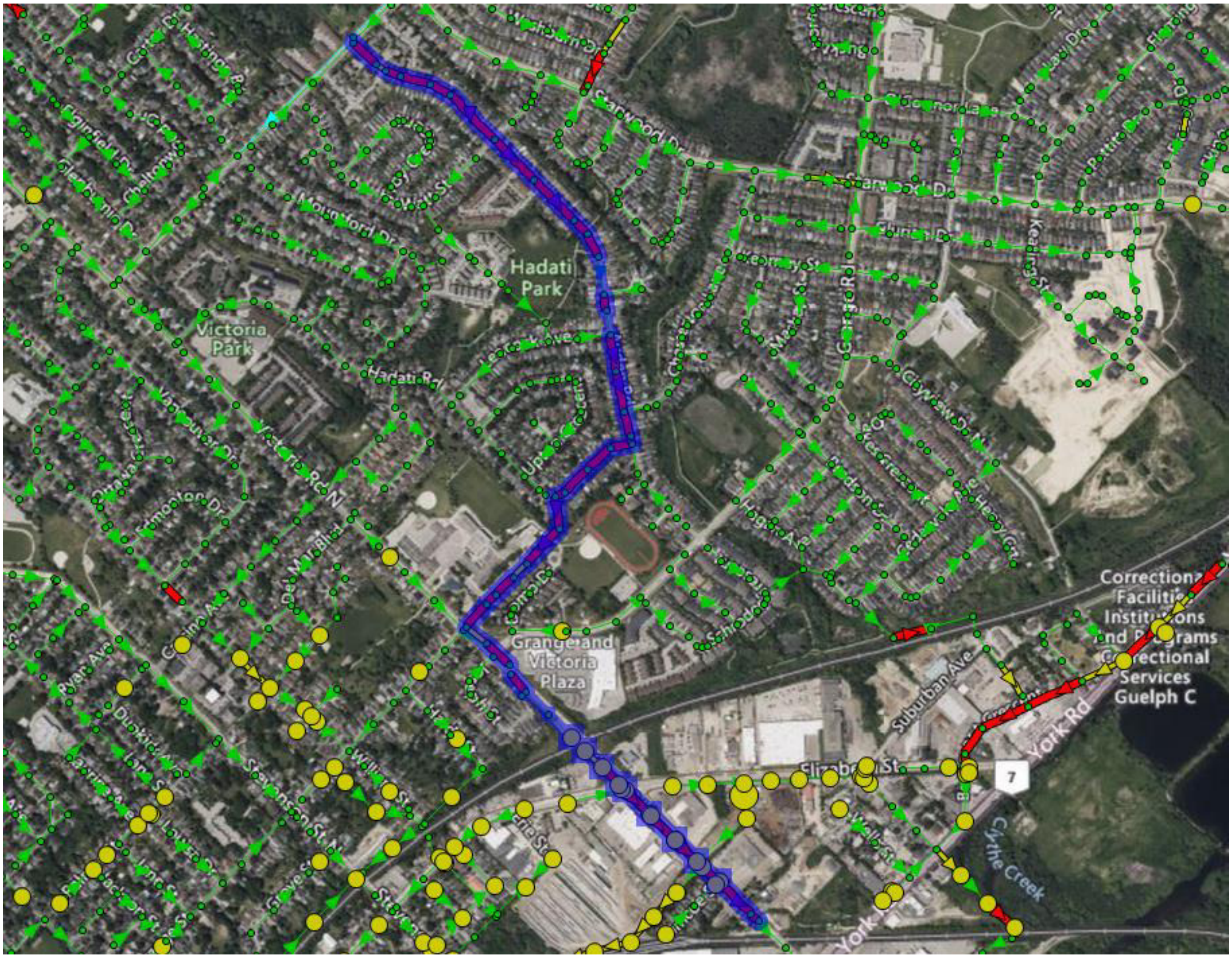


Figure 4-44 Existing Conditions WWF Results - Area 4 Site Plan 1

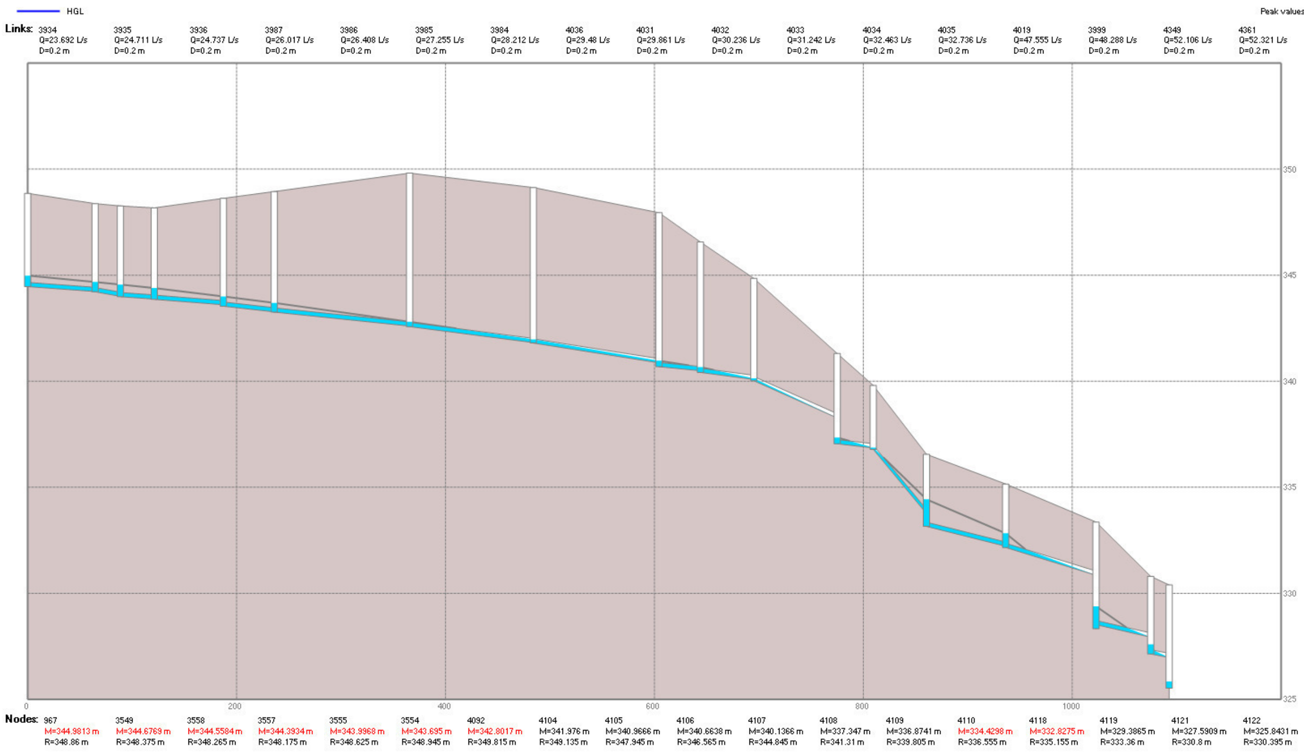


Figure 4-45 Existing Conditions WWF Results - Area 4 HGL Profile 1.1

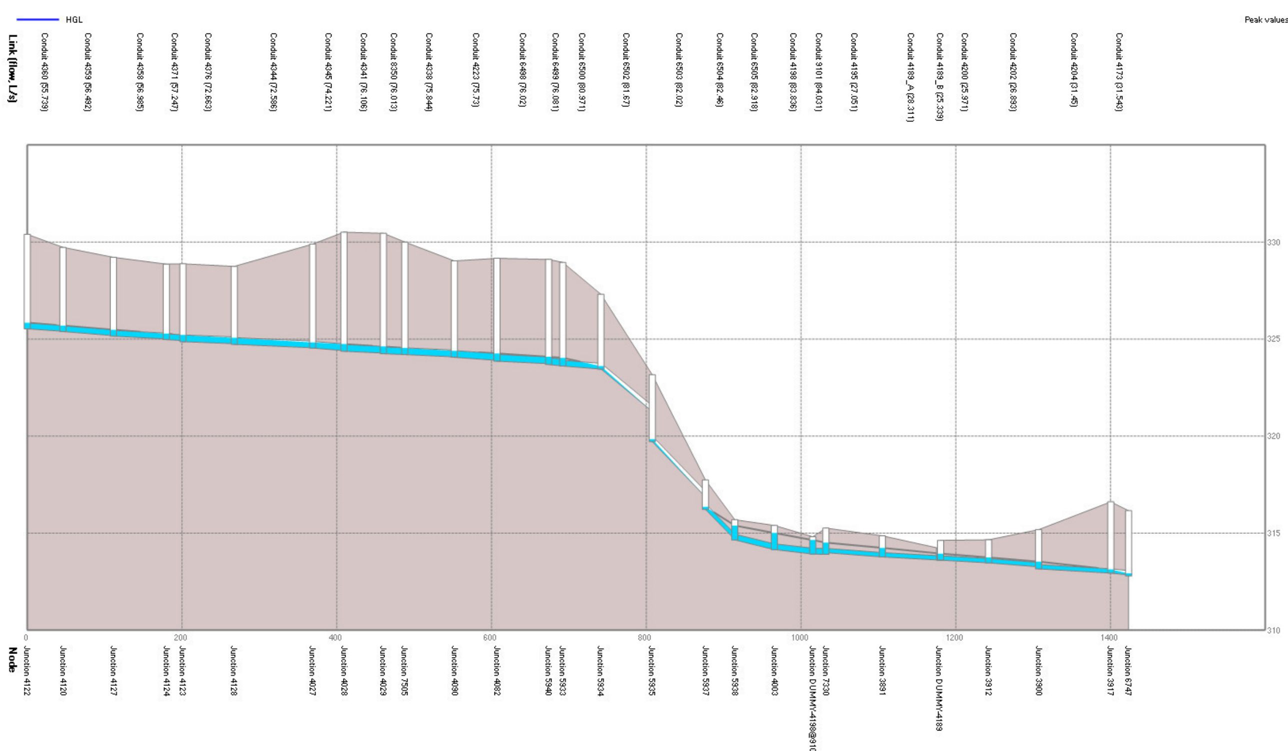


Figure 4-46 Existing Conditions WWF Results - Area 4 HGL Profile 1.2

As shown above, there are 33 sewers flowing above FPC, with surcharging observed in most MHs along the reach, and HGLs reaching 1.07 m above the sewer obverts. There are 9 MHs with less than 1.8 m of cover, and 6 MHs with less than 1.0 m of cover.

The second reach in Area 4 ranges includes 675 mm diameter sewers along York Road and Beaumont Crescent , shown in more detail below.

Figure 4-47 Existing Conditions WWF Results - Area 4 Site Plan 2

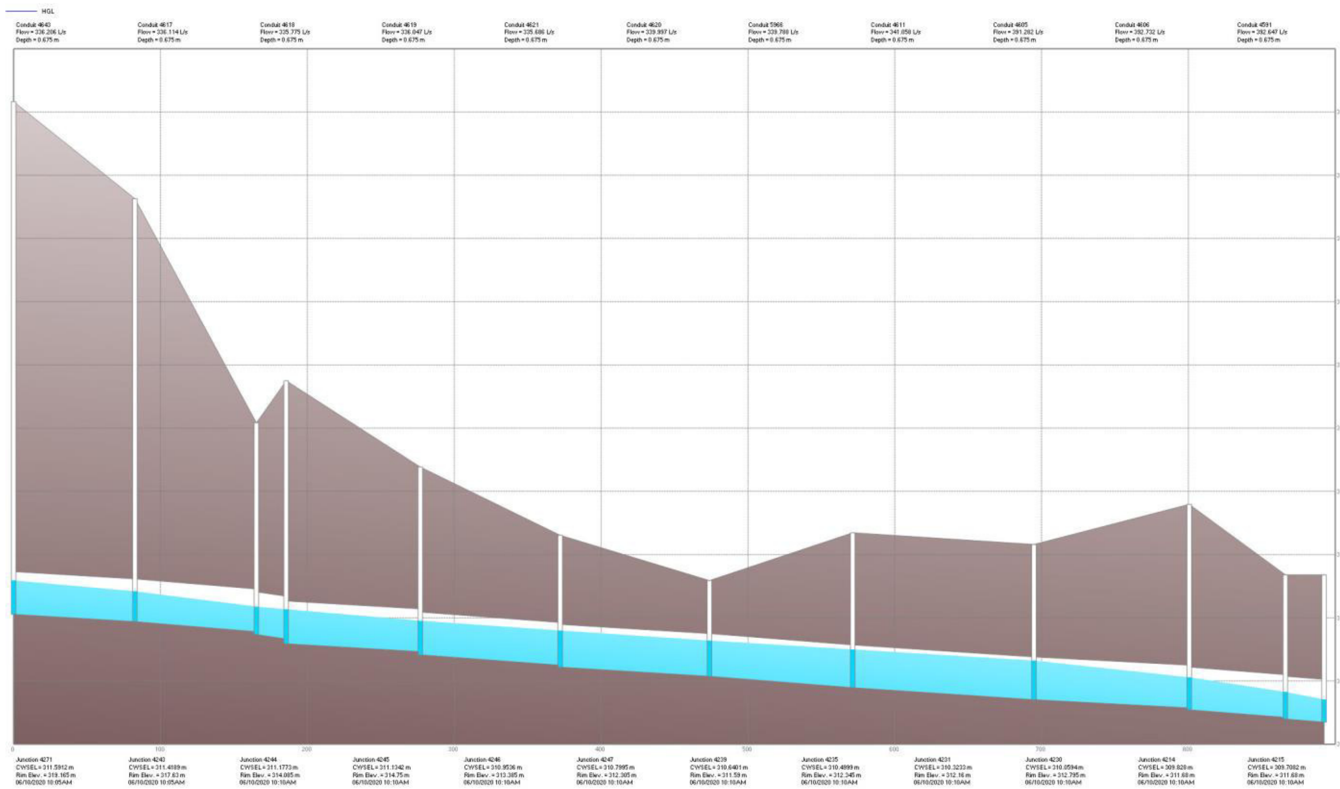


Figure 4-48 Existing Conditions WWF Results - Area 4 HGL Profile 2

As shown above, there are 7 sewers flowing above FPC, however no surcharging is observed in any MH. This is because these sewers are generally flowing only slightly above capacity and the constraint is not enough to cause surcharge before the capacity improves downstream. There are 6 MHs with less than 1.8 m of cover, and 1 MH with less than 1.0 m of cover.

The third reach in Area 4 ranges includes 600 mm diameter sewers along Stevenson Street between Ferguson Street and York Road, shown in more detail below.

Figure 4-49 Existing Conditions WWF Results - Area 4 Site Plan 3

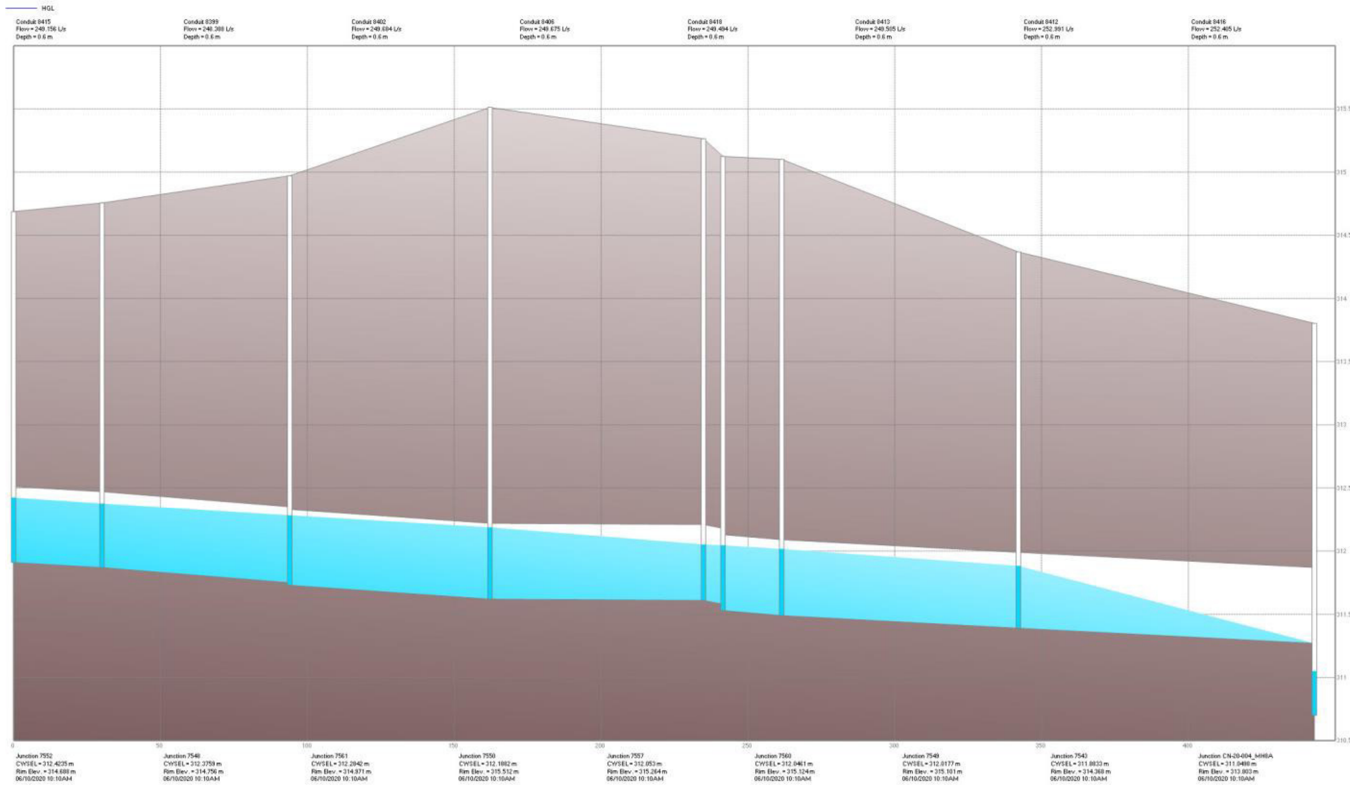


Figure 4-50 Existing Conditions WWF Results - Area 4 HGL Profile 3

As shown above, there are 5 sewers flowing above FPC, however no surcharging is observed in any MH. This is because these sewers are generally flowing only slightly above capacity and the constraint is not enough to cause surcharge before the capacity improves downstream. All MHs have greater than 1.8 m of cover.

The fifth location, noted as Area 5 on Figure 4-33 includes multiple reaches in the City centre. The first reach includes sewers ranging in diameter from 1050 mm to 1350 mm, travelling from York Road to the Hanlon Expressway along Wellington Street, as shown in more detail below.

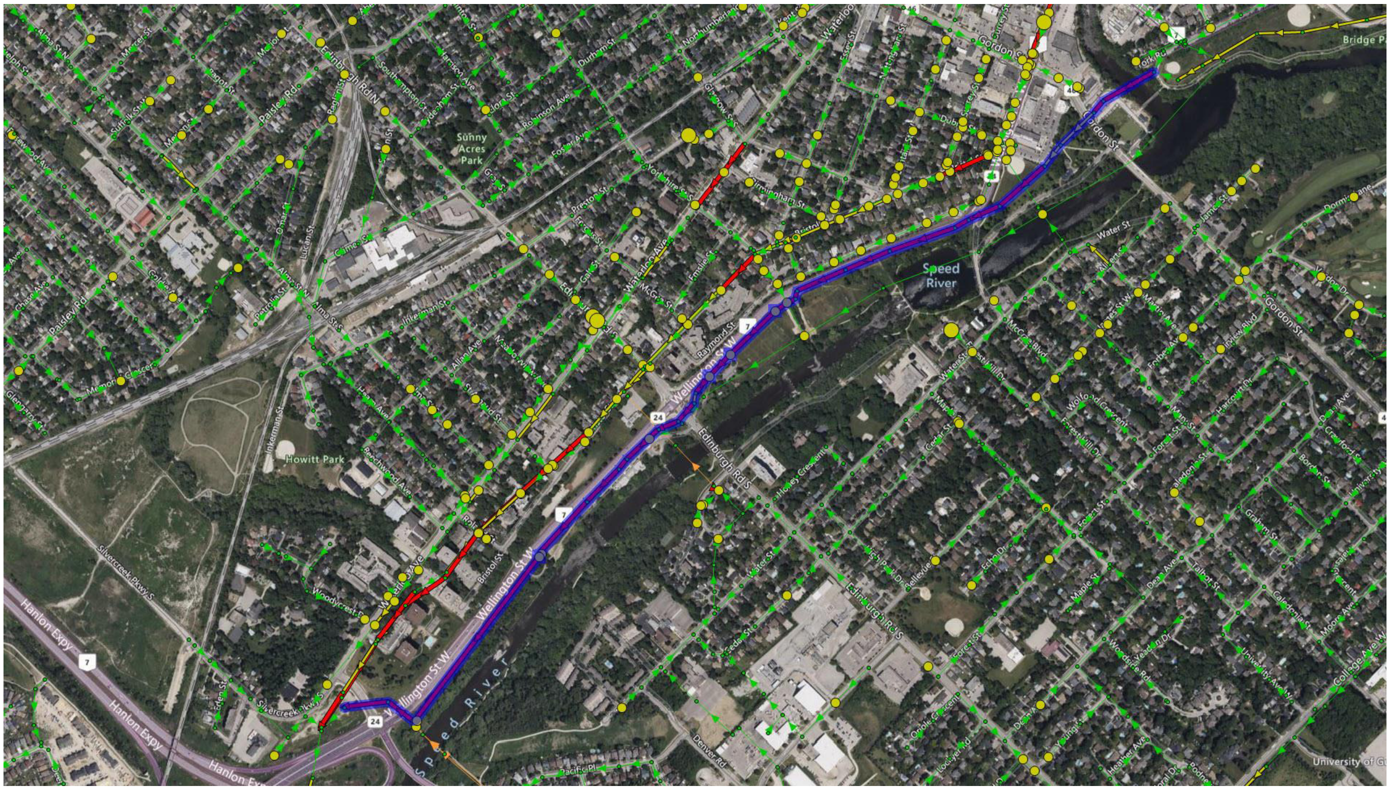


Figure 4-51 Existing Conditions WWF Results - Area 5 Site Plan 1

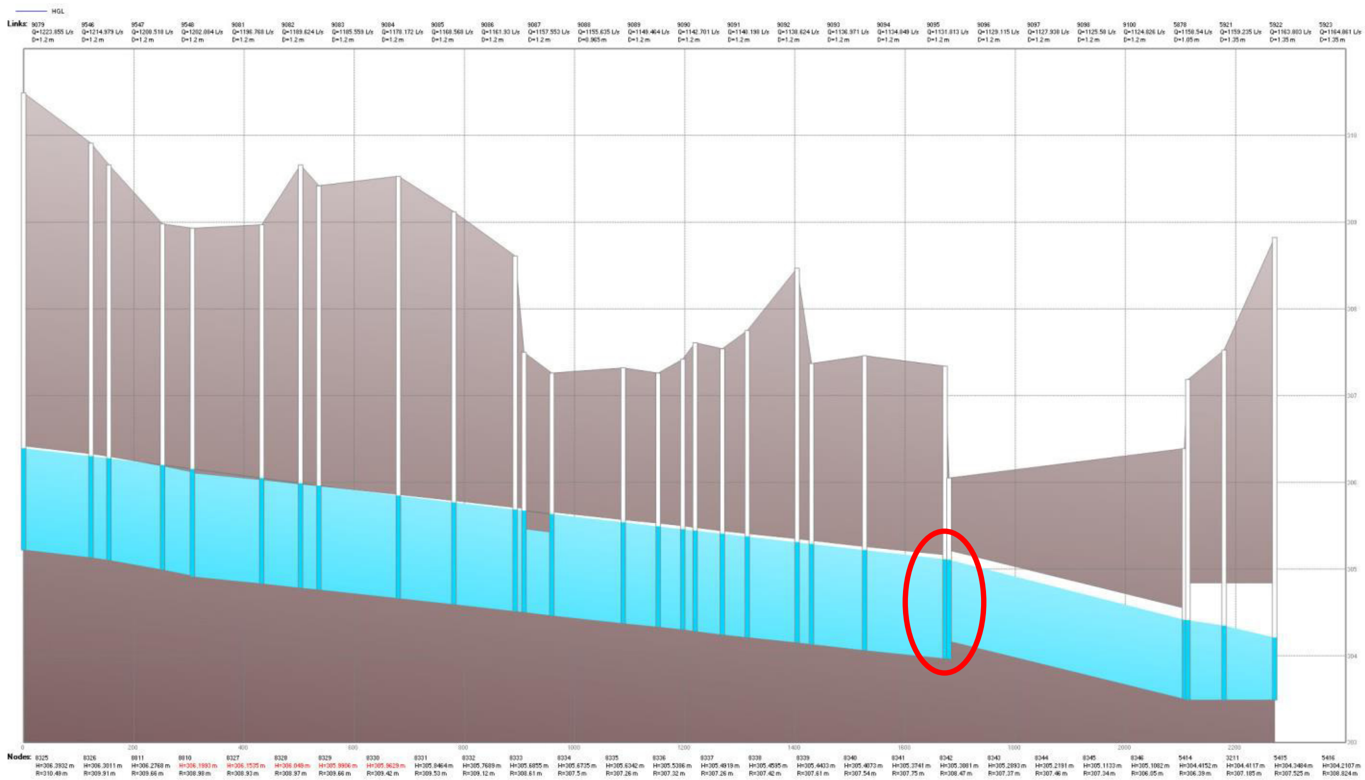


Figure 4-52 Existing Conditions WWF Results - Area 5 HGL Profile 1

As shown above, there are 22 sewers flowing above FPC, however only very minor surcharging is observed in 5 MHs, with HGLs reaching only 0.05 m above the sewer obverts. There is a horizontal elliptical sewer midway through the profile, and there also appears to be an incorrect invert, circled in red above. This should be confirmed through field investigation or as-built drawing review.

The second reach in Area 5 is primarily 750 mm sewers travelling from St Arnaud Street to Waterloo Avenue along Bristol Street, as shown in more detail below.

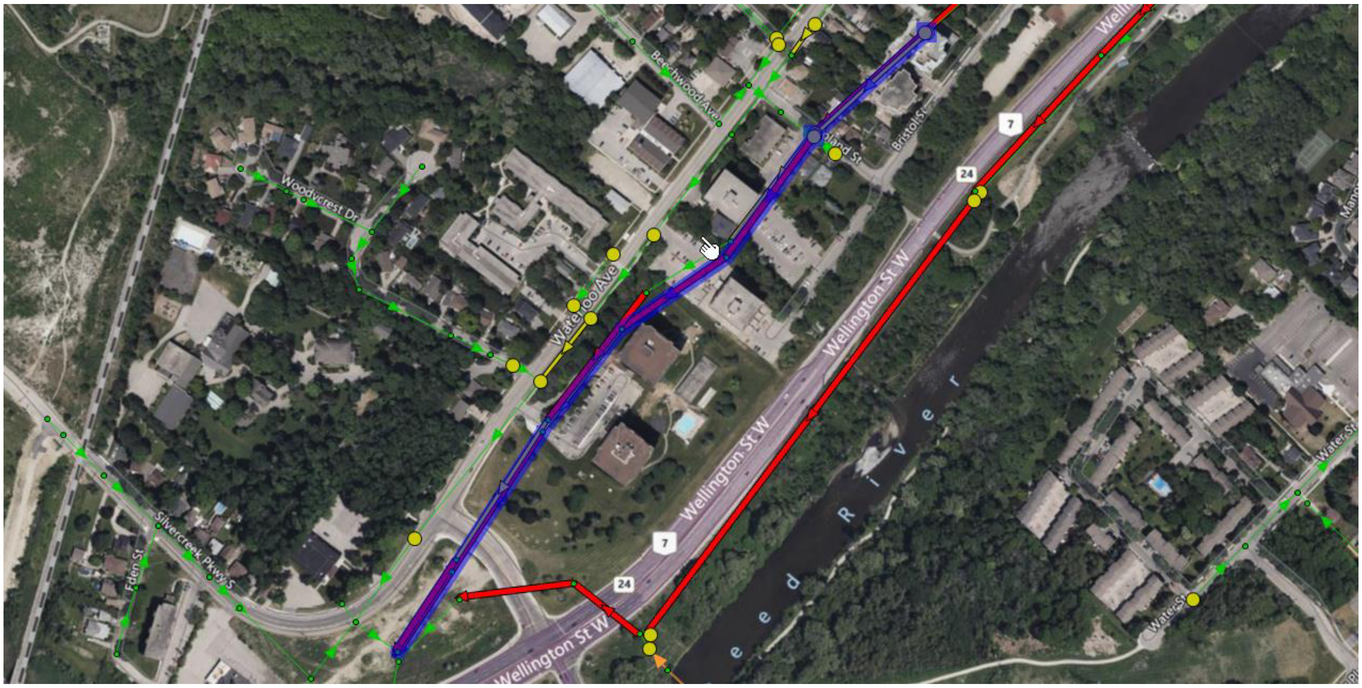


Figure 4-53 Existing Conditions WWF Results - Area 5 Site Plan 2

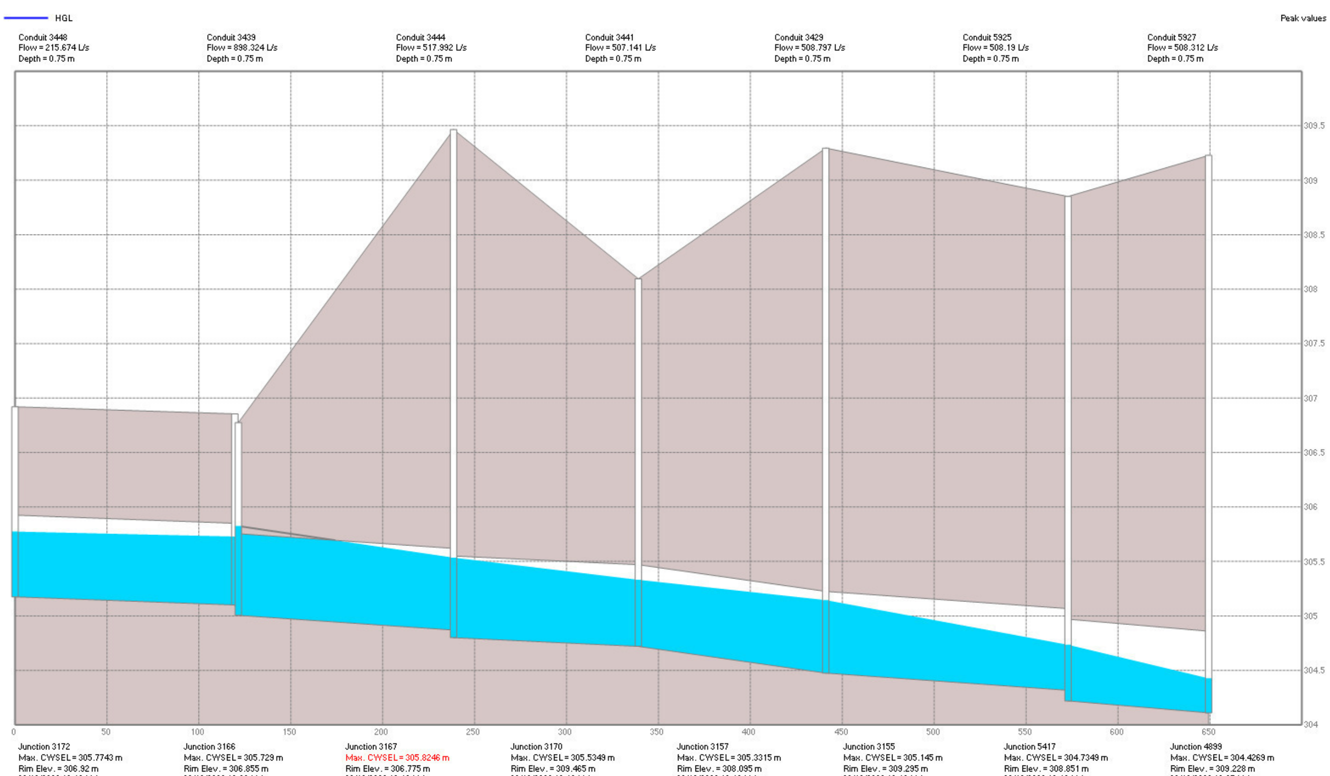


Figure 4-54 Existing Conditions WWF Results - Area 5 HGL Profile 2

As shown above, there are 4 sewers flowing above FPC however surcharging is only observed in 1 MH, with HGL reaching 0.08 m above the sewer obvert. This is because there are very flat sewers with slopes less than 0.001 %. There are 3 MH with less than 1.0 m of cover above the sewer obverts.

The third reach in Area 5 is parallel to the second reach, with 500 mm diameter sewers travelling from St Arnaud Street to Waterloo Avenue along Bristol Street, as shown in more detail below.

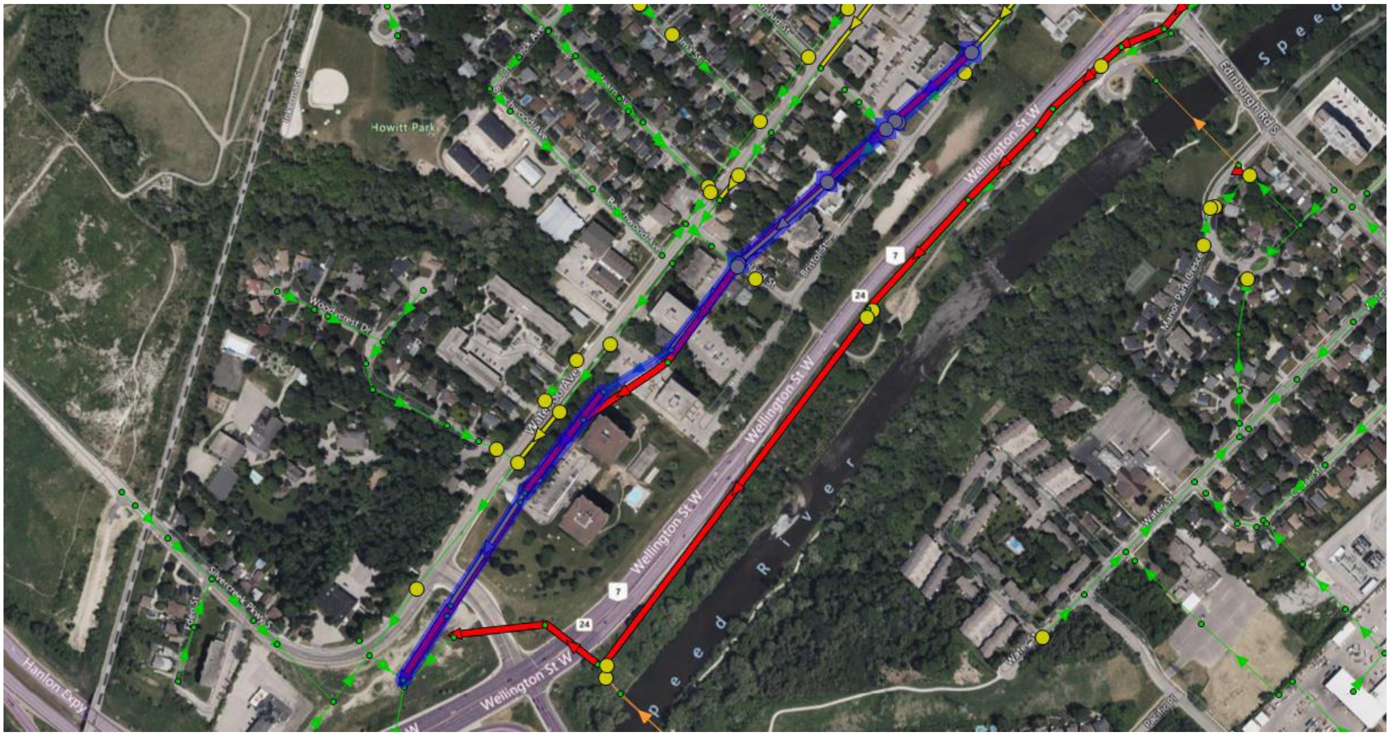


Figure 4-55 Existing Conditions WWF Results - Area 5 Site Plan 3

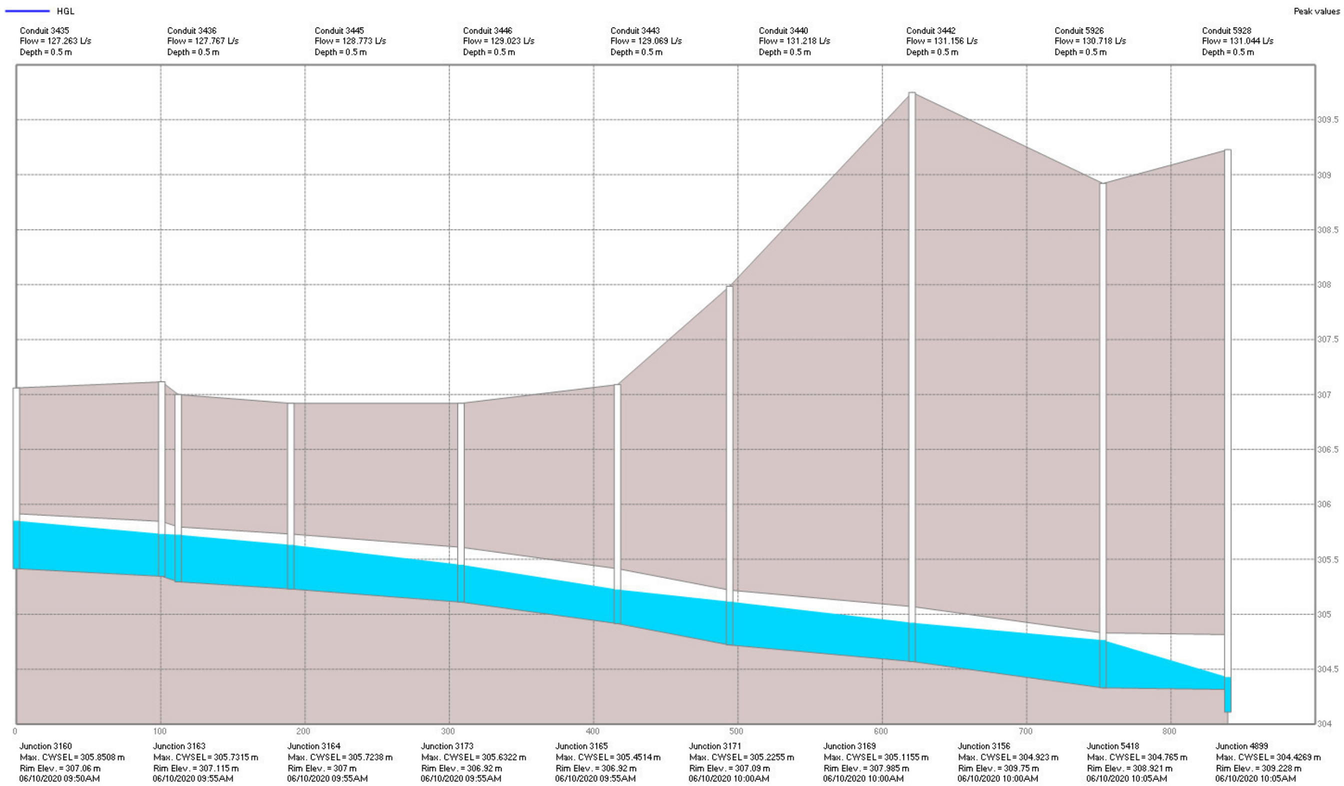


Figure 4-56 Existing Conditions WWF Results - Area 5 HGL Profile 3

As shown above, there are 4 sewers flowing above FPC however no surcharging is observed. This is because there are very flat sewers with slopes less than 0.001 %. There are 6 MH with less than 1.8 m of cover above the sewer obverts.

The fourth reach in Area 5 includes only 3 sewers with 1200 mm diameter near the WRRC, as shown in more detail below.

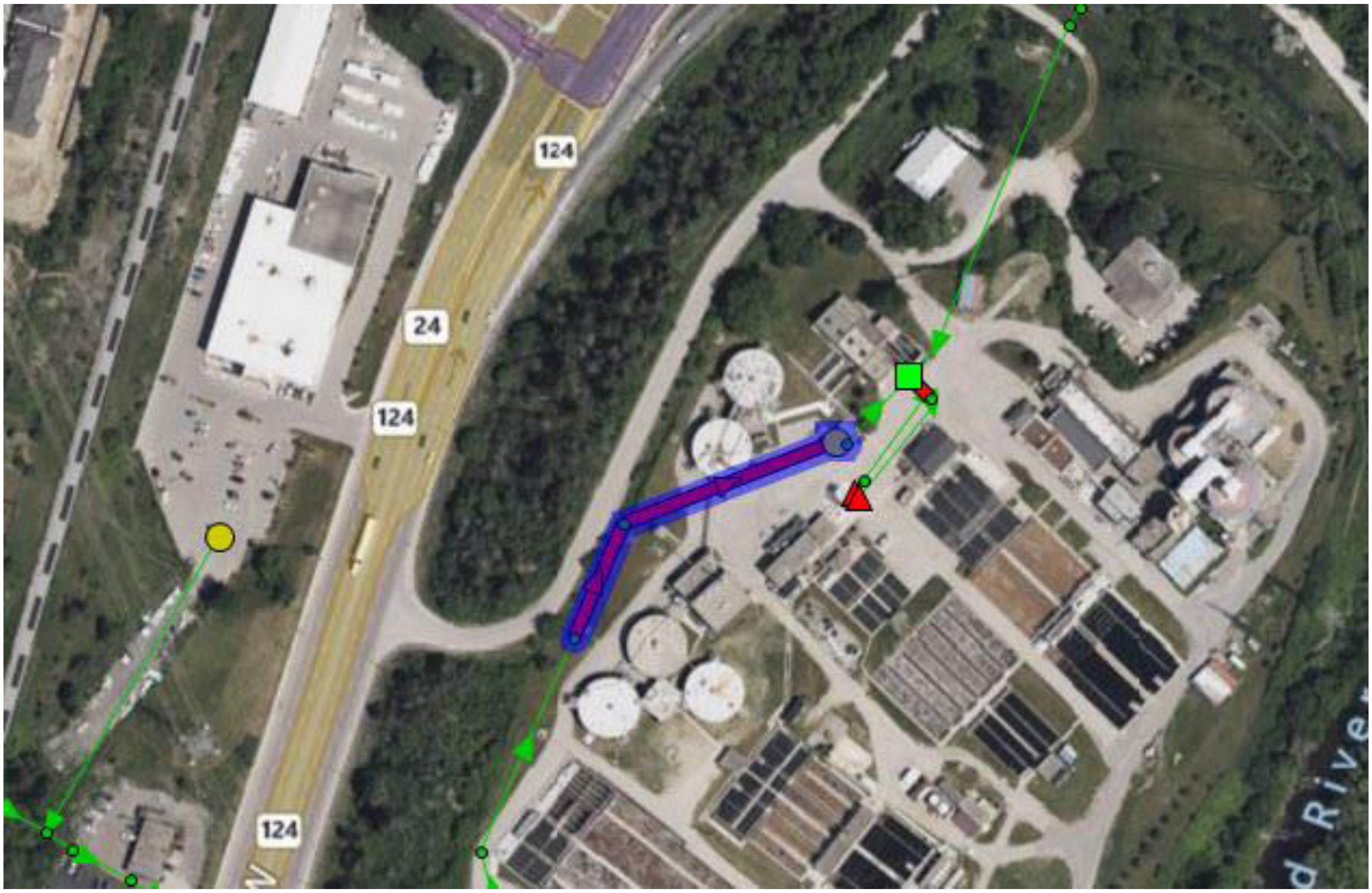


Figure 4-57 Existing Conditions WWF Results - Area 5 Site Plan 4

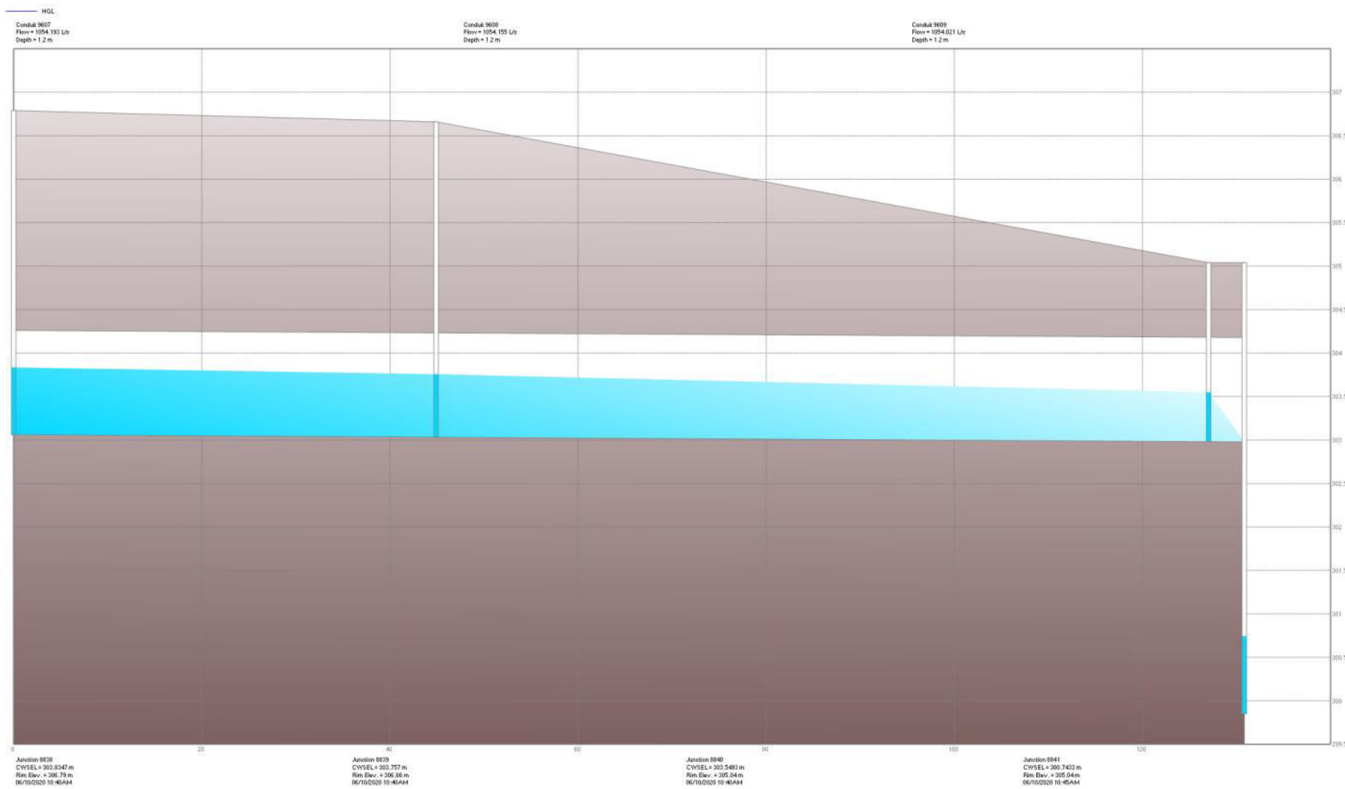


Figure 4-58 Existing Conditions WWF Results - Area 5 HGL Profile 4

As shown above, all three sewers are flowing above FPC however the HGL remains within the sewer obverts. This is because they are very flat sewers with slopes of approximately 0.05% - 0.06%. Two of the MHs have less than 1.8 m of cover above the sewer obverts.

The fifth reach in Area 5 includes sewers ranging from 300 mm to 450 mm diameter, traveling along Quebec Street, Wyndham Street North and Wellington Street East, as shown in more detail below.

Figure 4-59 Existing Conditions WWF Results - Area 5 Site Plan 5

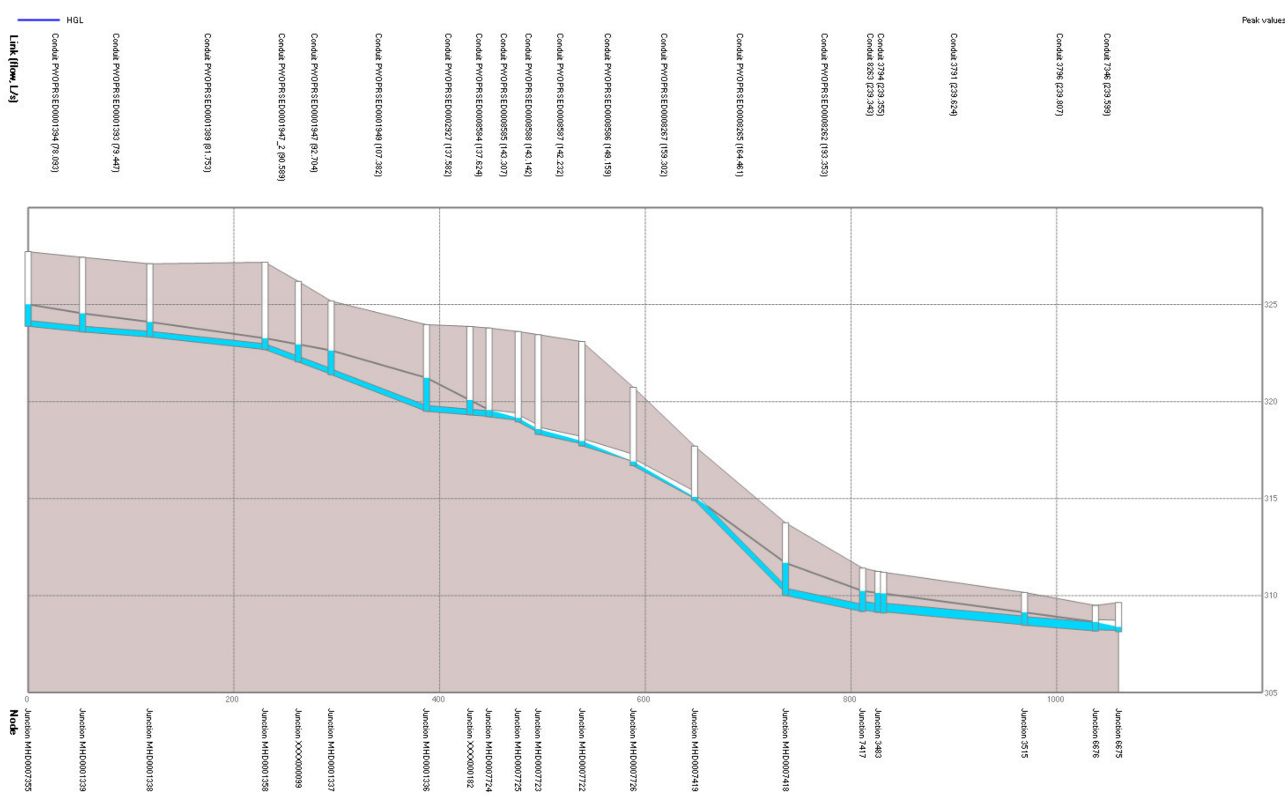


Figure 4-60 Existing Conditions WWF Results - Area 5 HGL Profile 5

As shown above, there are 11 sewers flowing above FPC, and surcharging observed in 13 MHs with HGLs reaching 1.43 m above the sewer obverts. There are other individual sewers flowing above FPC in Area 5 however there are no surcharging issues related to them identified.

The sixth location, noted as Area 6 on Figure 4-33 is a reach including sewers ranging in diameter from 250 mm to 675 mm, travelling along College Avenue and Scottsdale Drive, as shown in more detail below.

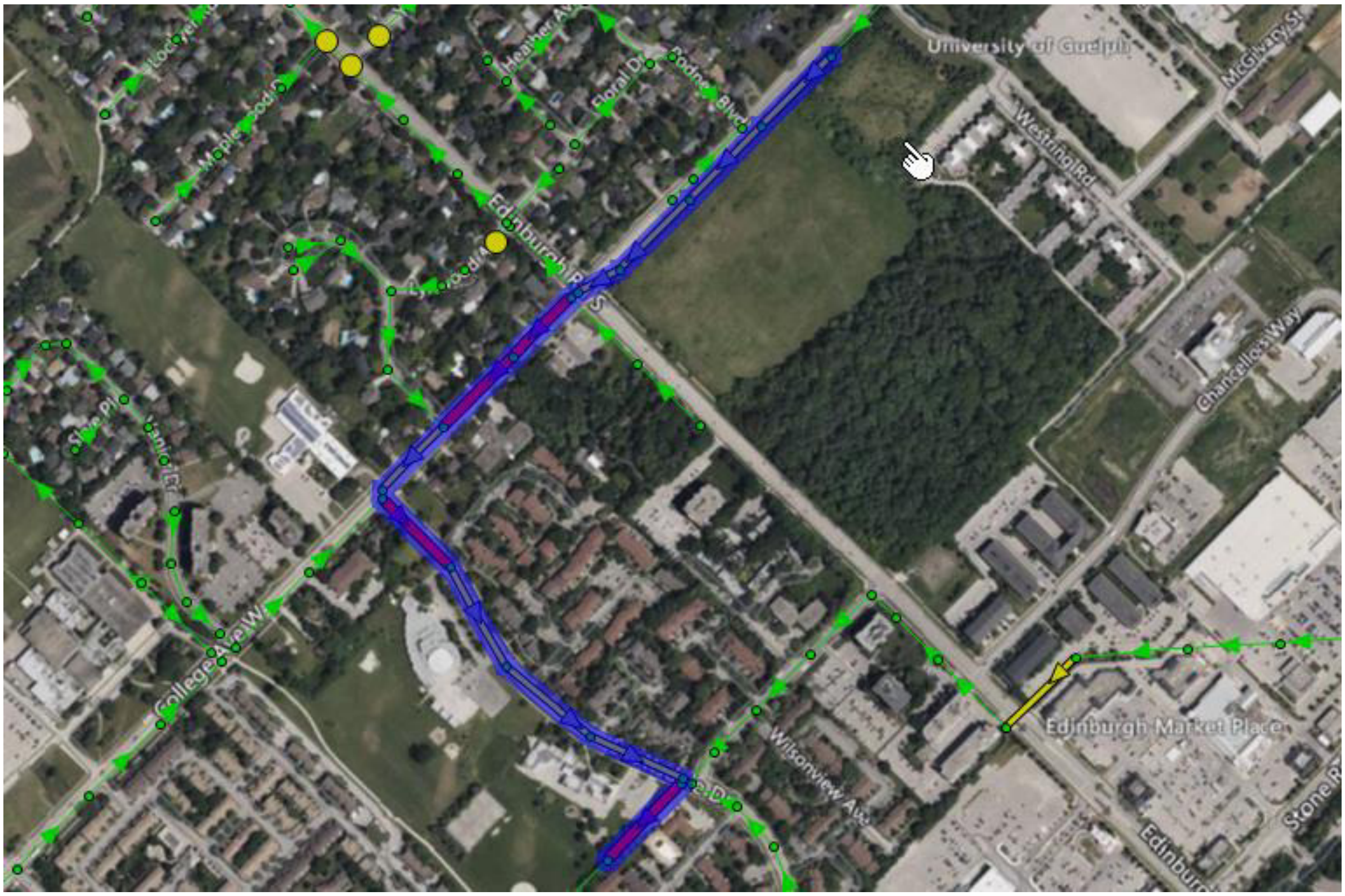


Figure 4-61 Existing Conditions WWF Results - Area 6 Site Plan

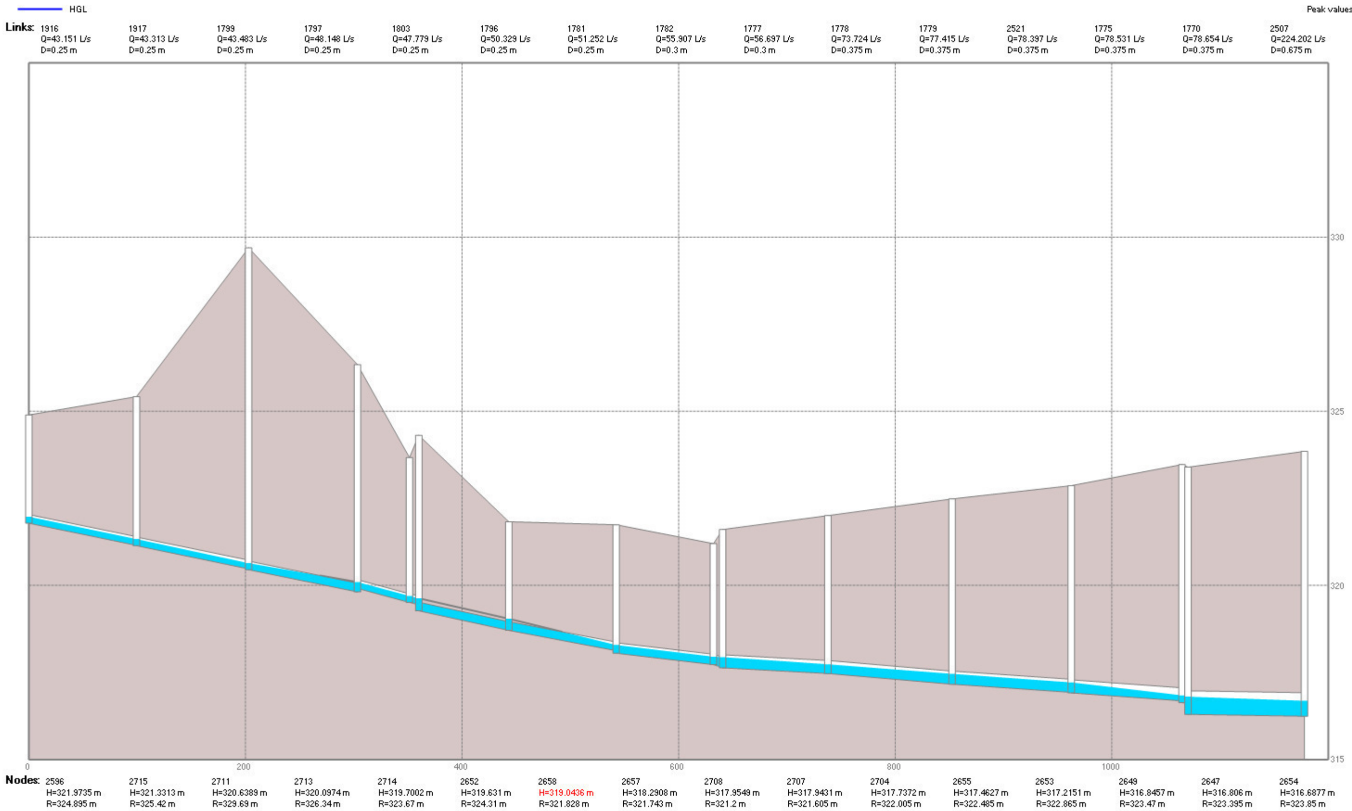


Figure 4-62 Existing Conditions WWF Results - Area 6 HGL Profile

As shown above, there are 4 sewers flowing above FPC, but only minor surcharging is observed in 3 MHs with HGLs reaching 0.11 m above the sewer obverts. There are other individual sewers flowing above FPC in Area 6 however there are no surcharging issues related to them identified.

The seventh and final location, noted as Area 7 on Figure 4-33 is a reach of 200 mm and 450 mm sewers travelling along Farley Drive and Clairfields Drive West, as shown in more detail below.

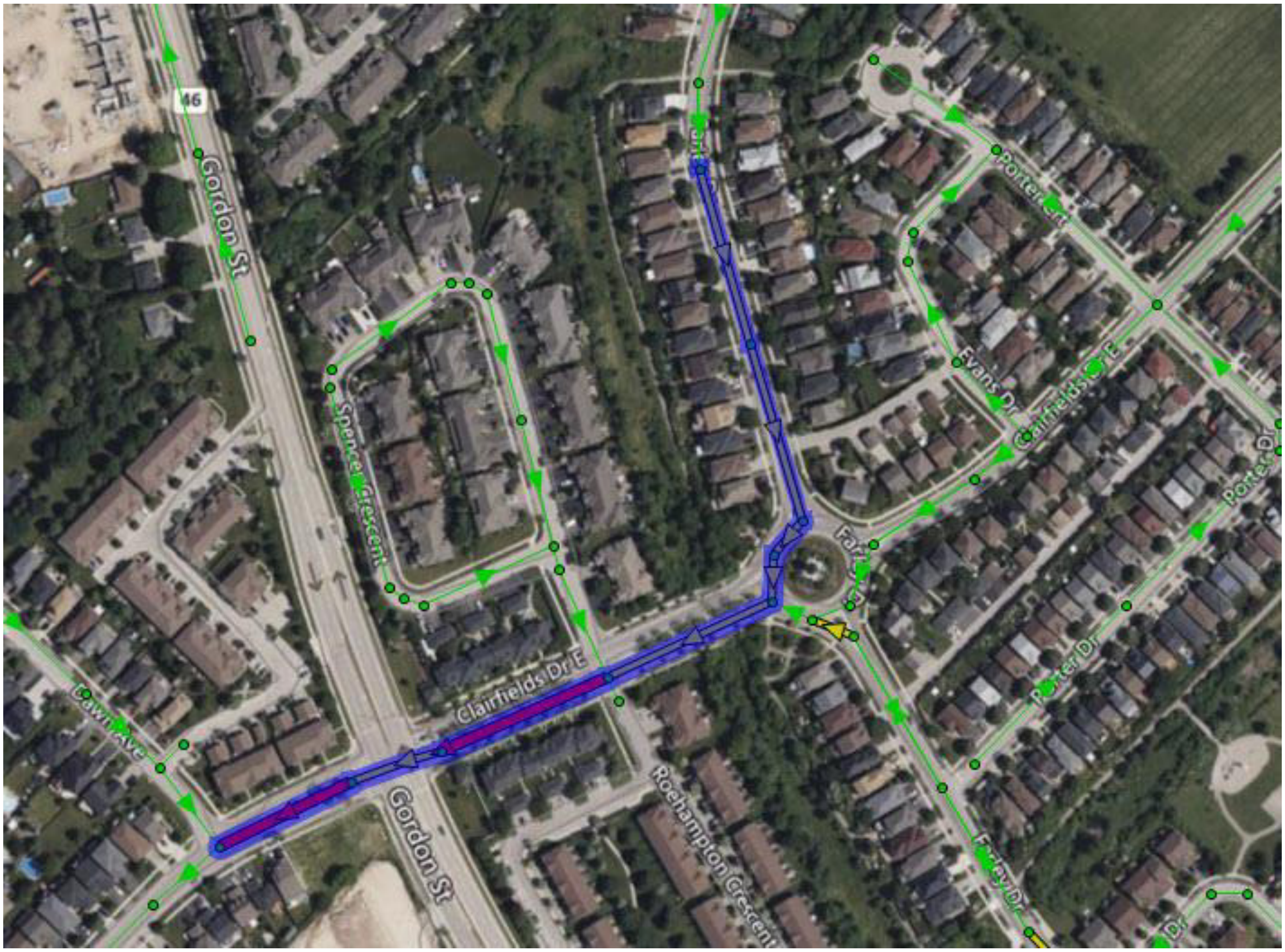


Figure 4-63 Existing Conditions WWF Results - Area 7 Site Plan

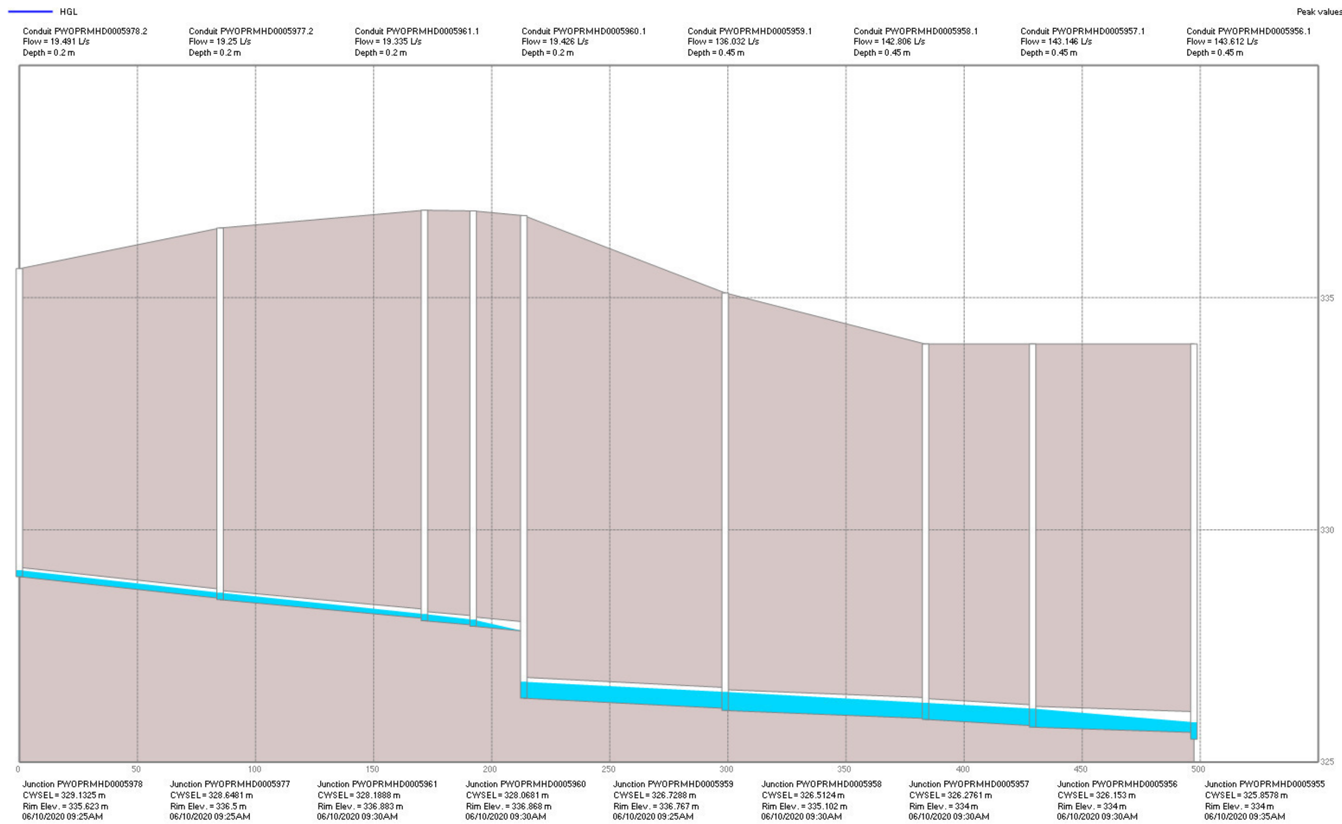


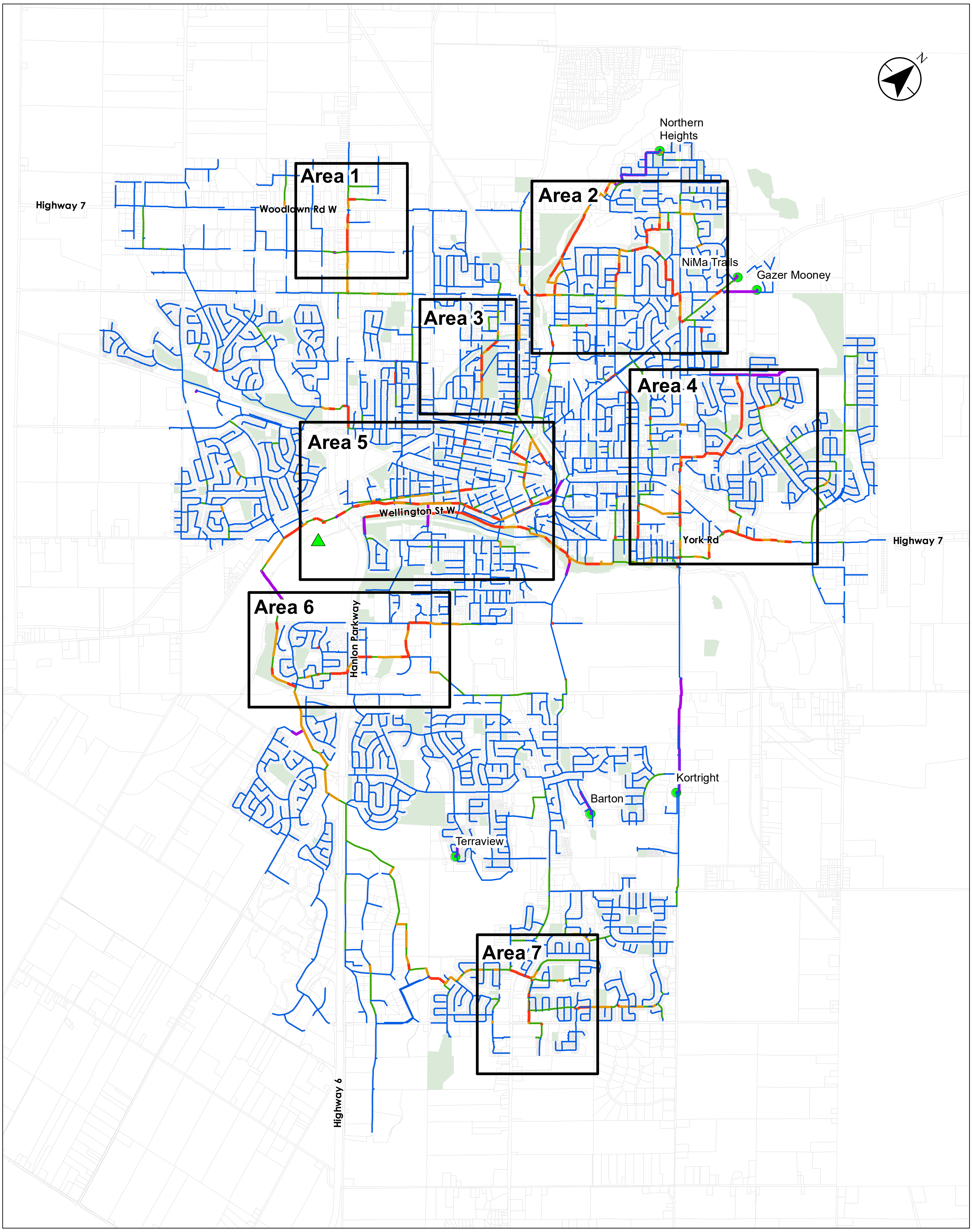
Figure 4-64 Existing Conditions WWF Results - Area 7 HGL Profile

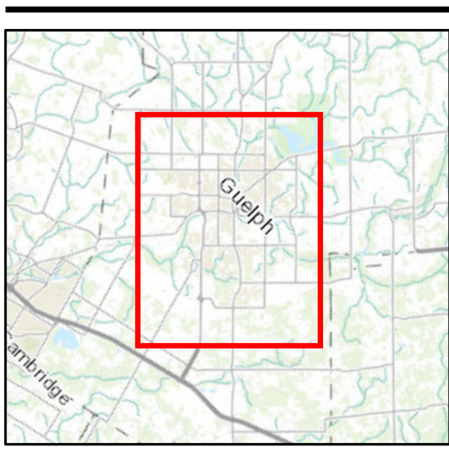
As shown above, there are 2 sewers flowing above FPC however no surcharging is observed as the HGL remains within the sewer obverts. It should be noted that there are other individual sewers flowing above FPC scattered around the City however a single sewer flowing above capacity is rarely a cause of surcharge and these have all been reviewed and it was determined that they did not pose a risk of surcharge or flooding.

4.2.4.2 WWF Future Conditions Analysis

The attached Figure 4-65 shows the overall future results for the entire wastewater system. The assessment focused on sewers operating above the Manning's Full Pipe Capacity (FPC), and any MHs that show surcharge above the pipe obvert. As shown on Figure 4-65, there are 7 locations with multiple sewers operating at or near FPC.

V:\1656\active\165640298\preliminary\3_Report\TM3a - existing system assessment\figures\fig_4.65_Future_WWF_results.mxd Revised: 2022-09-06 By: ehewko





Legend

- Active Pump Station
- Wastewater Treatment Plant
- Property
- Forcemain / Siphon

q / Q

- <= 50%
- > 50% - 80%
- > 80% - 100%
- > 100%

0 1,000 2,000 Meters

1:50,000 (At Original document size of 11x17)

Project Location 165640298

City of Guelph

Client/Project

CITY OF GUELPH WATER AND WASTEWATER SERVICING MASTER PLAN – TM3A EXISTING AND FUTURE SYSTEM CAPACITY

Figure No.

4.65

Title

Future System WWF Results

Notes

1. Coordinate System: NAD 1983 UTM Zone 17N
2. Base features produced under license with the Ontario Ministry of Natural Resources and Forestry © Queen's Printer for Ontario, 2018. Sources: Esri, HERE, Garmin, Intermap, increment P Corp., GEBCO, USGS, FAO, NPS, NRCAN, GeoBase, IGN, Kadaster NL, Ordnance Survey, Esri Japan, METI, Esri China (Hong Kong), (c) OpenStreetMap contributors, and the GIS User Community

The first location, noted as Area 1 on Figure 4-65 is a reach of 300 mm and 525 mm diameter sewers along Silvercreek Parkway North between Woodlawn Road West and Speedvale Avenue W, as shown in more detail below.

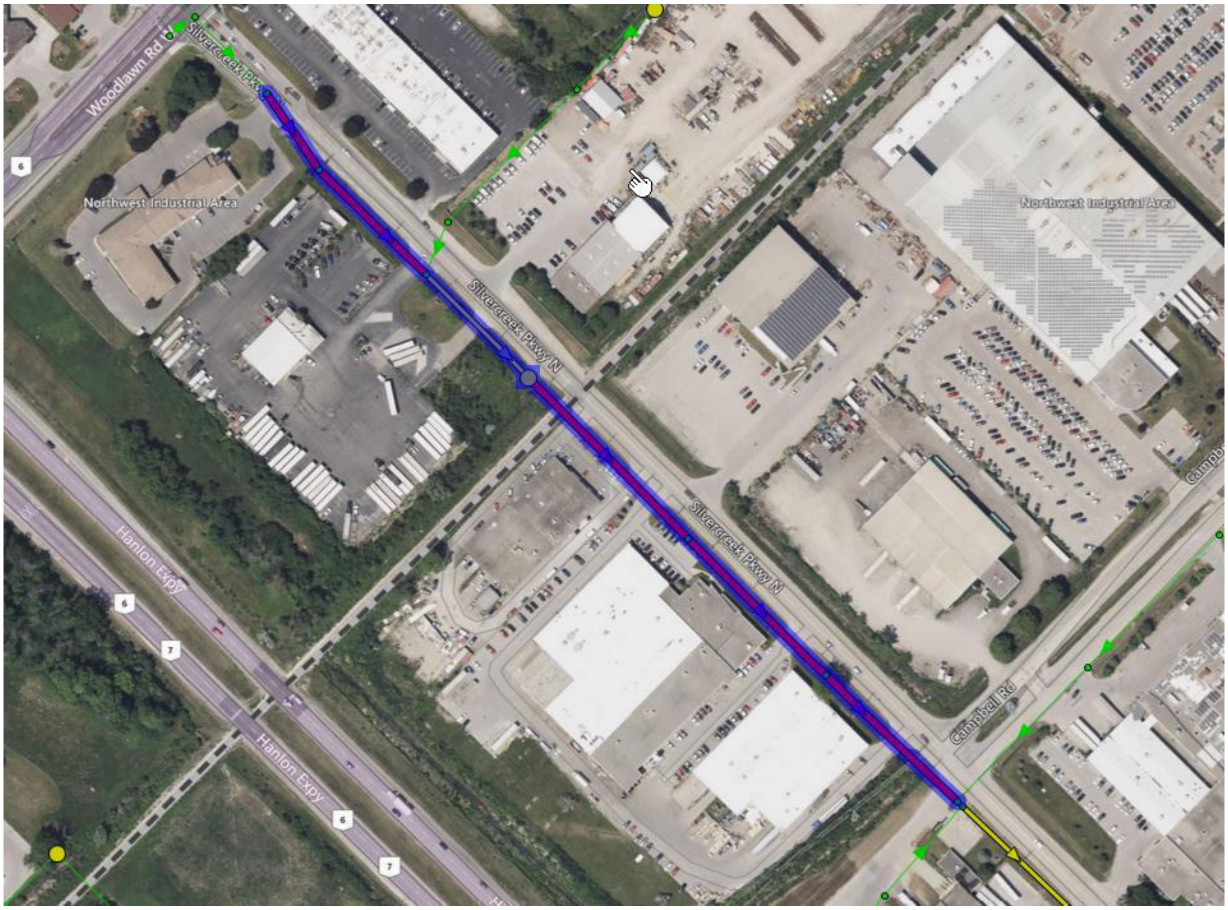


Figure 4-66 Future Conditions WWF Results - Area 1 Site Plan

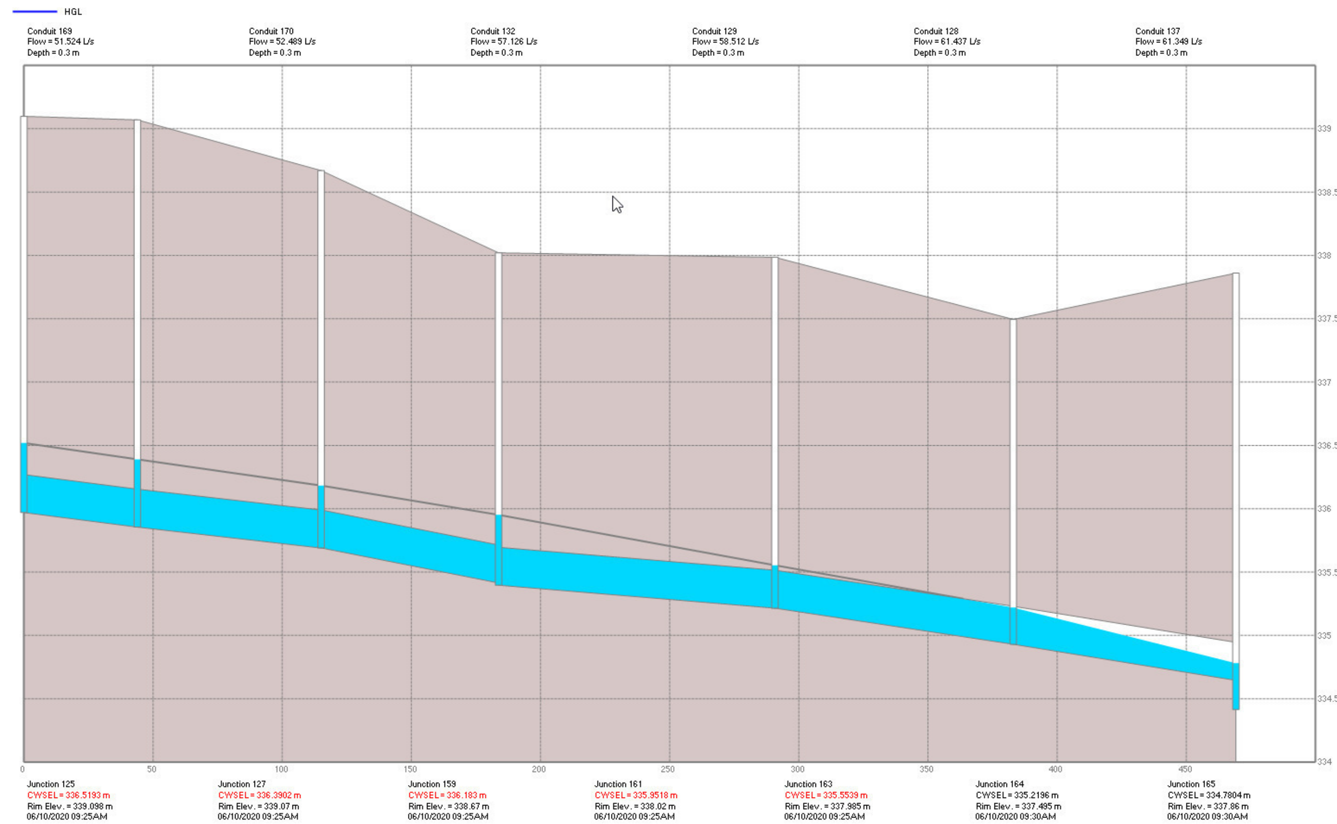


Figure 4-67 Future Conditions WWF Results - Area 1 HGL Profile

As shown above, there are 5 sewers flowing above FPC, with surcharging observed in 5 MHs, and HGLs reaching 0.25 m above the sewer obverts. This location was identified in the Existing Conditions assessment, and the surcharge has increased in the Future Conditions.

The second location, noted as Area 2 on Figure 4-65 includes multiple reaches in the northeast of the City. The first reach ranges from 225 mm to 375 mm travelling from Eramosa Road northwest along Victoria Road and Waverley Drive, as shown in more detail below.

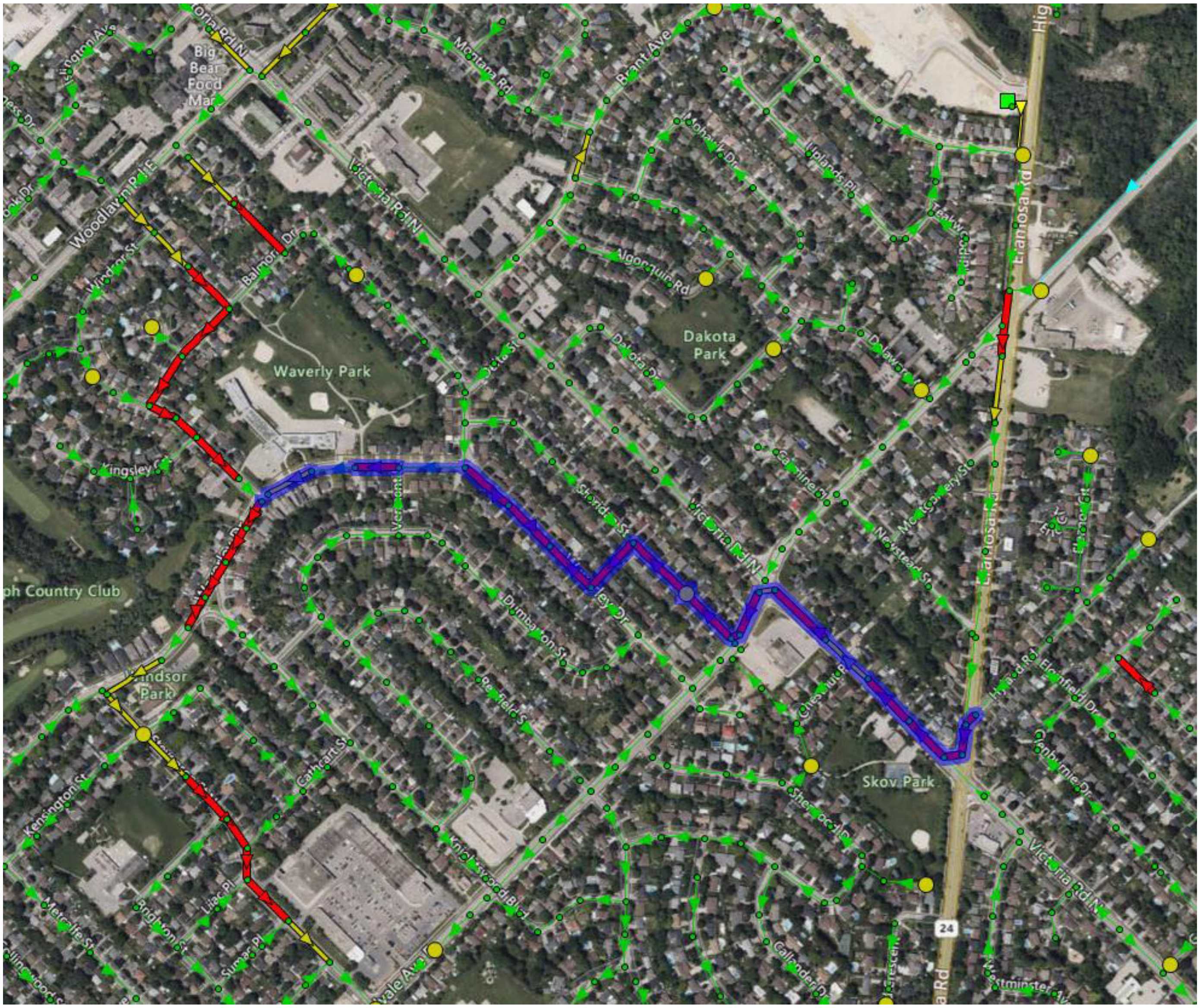


Figure 4-68   Future Conditions WWF Results - Area 2 Site Plan 1

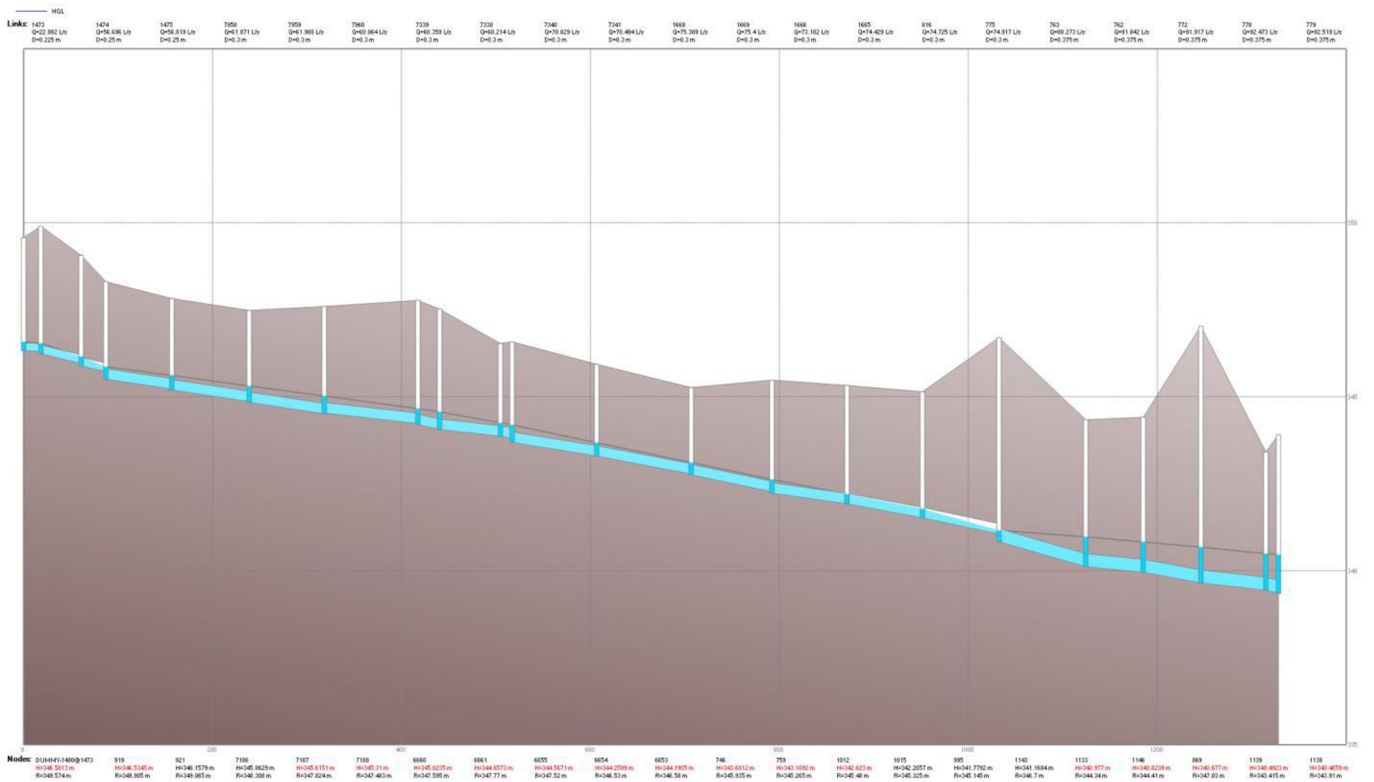


Figure 4-69 Future Conditions WWF Results - Area 2 HGL Profile 1

As shown above, there are 17 sewers flowing above FPC, with minor surcharging observed in the upper portion of the reach, and HGL's reaching 0.24 m above the sewer obverts, and greater surcharging at the lower portion of the reach with HGL's reaching 0.76 m above the sewer obverts. This location was identified in the Existing Conditions assessment, and the results are similar in the Future System Assessment.

The second reach in Area 2 includes 300 mm and 375 mm diameter sewers travelling from Woodlawn Road East to Speedvale Avenue as shown in more detail below.

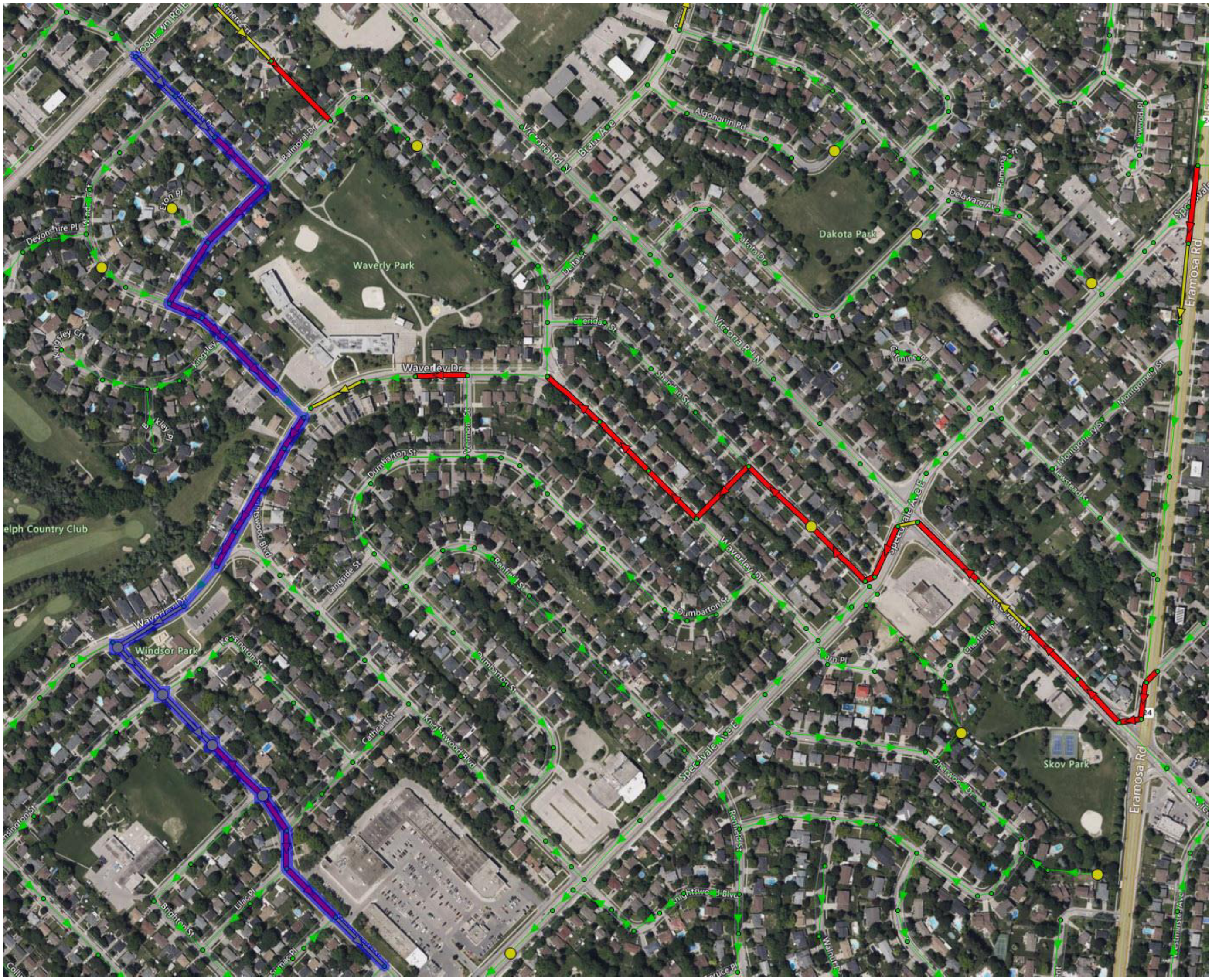


Figure 4-70 Future Conditions WWF Results - Area 2 Site Plan 2

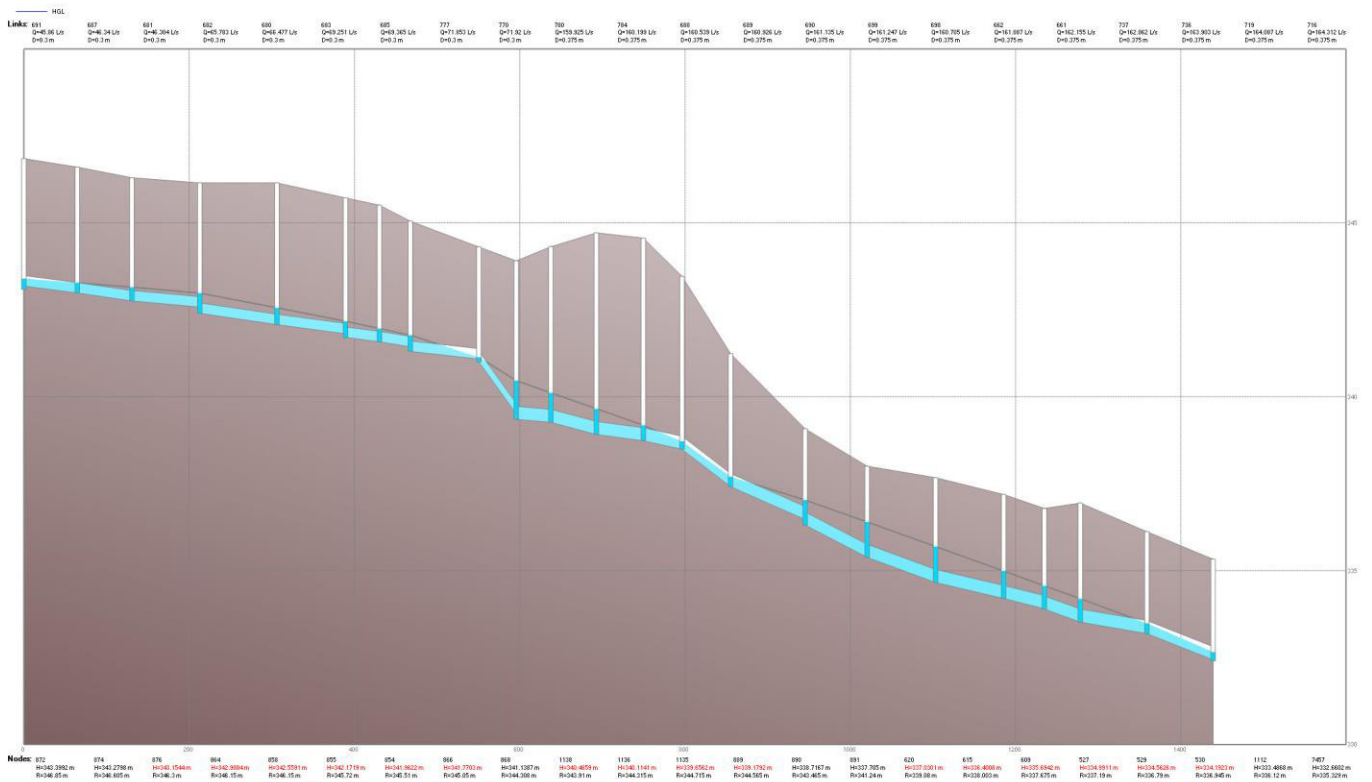


Figure 4-71 Future Conditions WWF Results - Area 2 HGL Profile 2

As shown above, there are 14 sewers flowing above FPC, with minor surcharging observed in most MHs, with HGLs ranging from 0.04 m to 0.94 m above the sewer obverts. All but 3 surcharged MH along this reach have greater than 1.8 m of freeboard. This location was identified in the Existing Conditions assessment, and the surcharge has increased in the Future Conditions.

The third reach in Area 2 ranges in diameter from 225 mm to 300 mm sewers travelling from Pondview Crescent to the south across Woodlawn Road East and along Riverview Drive to Speedvale Avenue, shown in more detail below.

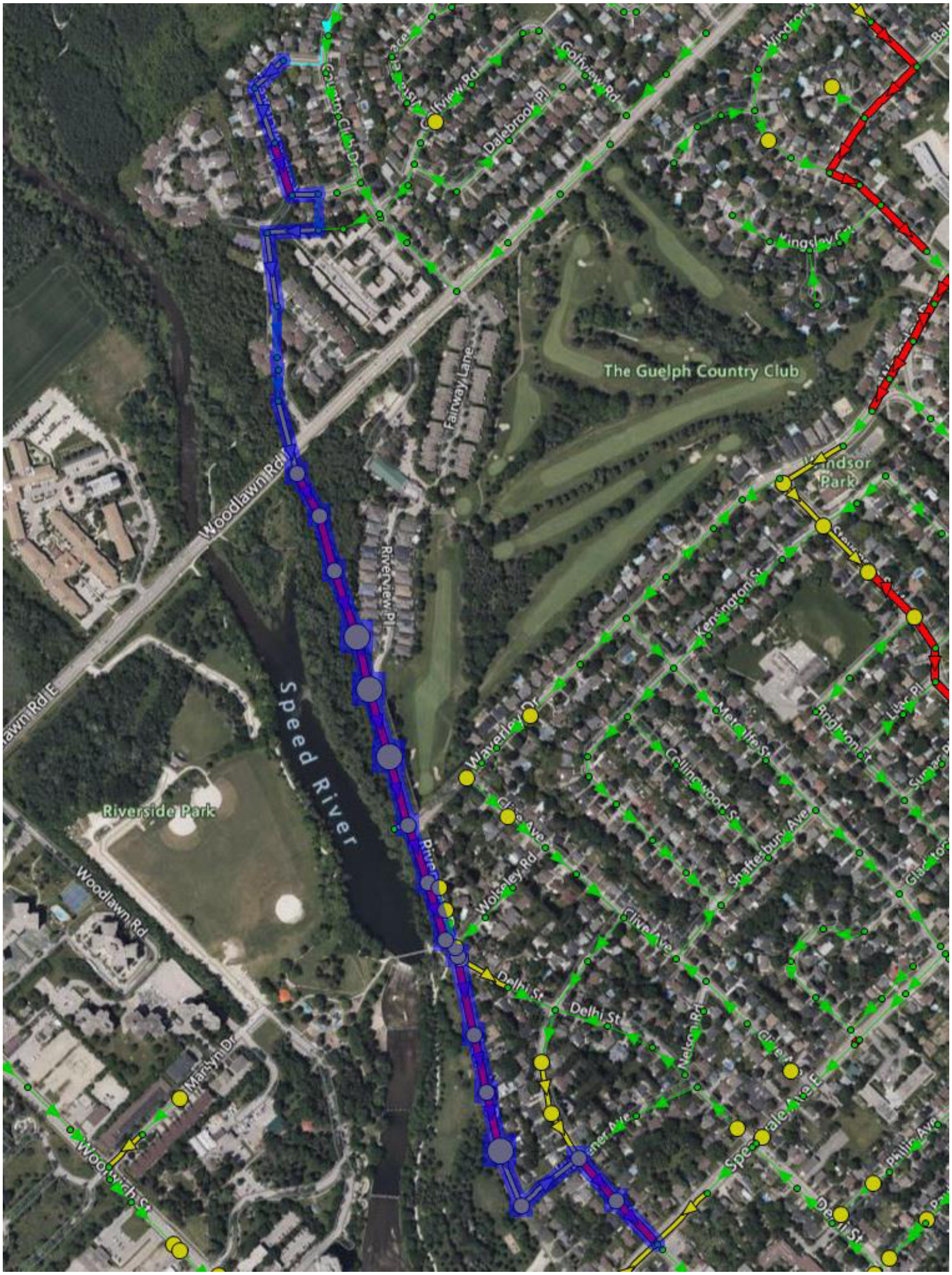


Figure 4-72 Future Conditions WWF Results - Area 2 Site Plan 2

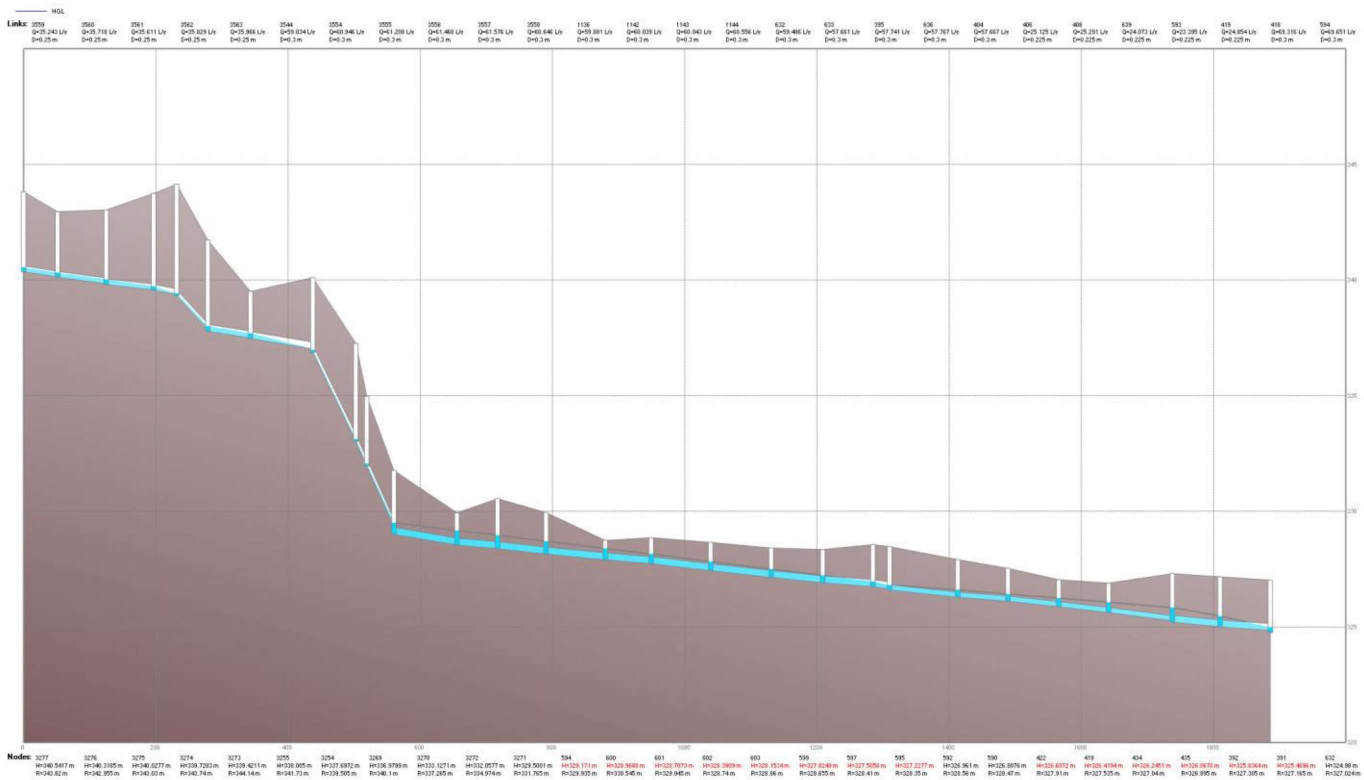


Figure 4-73 Future Conditions WWF Results - Area 2 HGL Profile 3

As shown above, there are 14 sewers flowing above FPC, with minor surcharging observed in most MHs, with HGLs reaching 0.34 m above the sewer obverts. It should be noted that 15 of the MHs along this reach have less than 1.8 m of cover over the connected sewers, with multiple MHs having less than 1.0 m of cover. This location was identified in the Existing Conditions assessment, and the surcharge has increased in the Future Conditions.

The third location, noted as Area 3 on Figure 4-65 is a reach of 375 mm sewers travelling from Division Street to London Road W along Kathleen Street, as shown in more detail below.

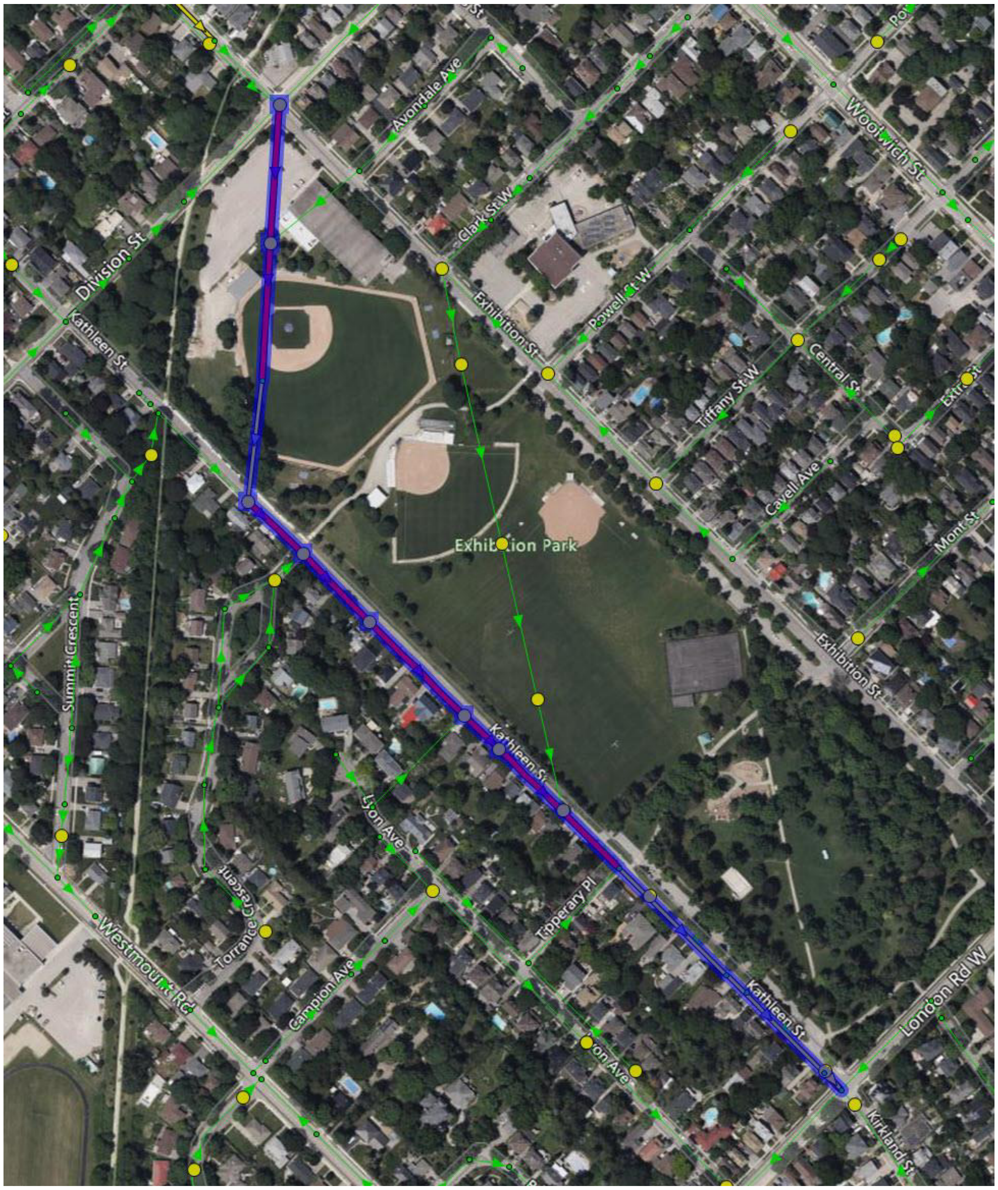


Figure 4-74 Future Conditions WWF Results - Area 3 Site Plan

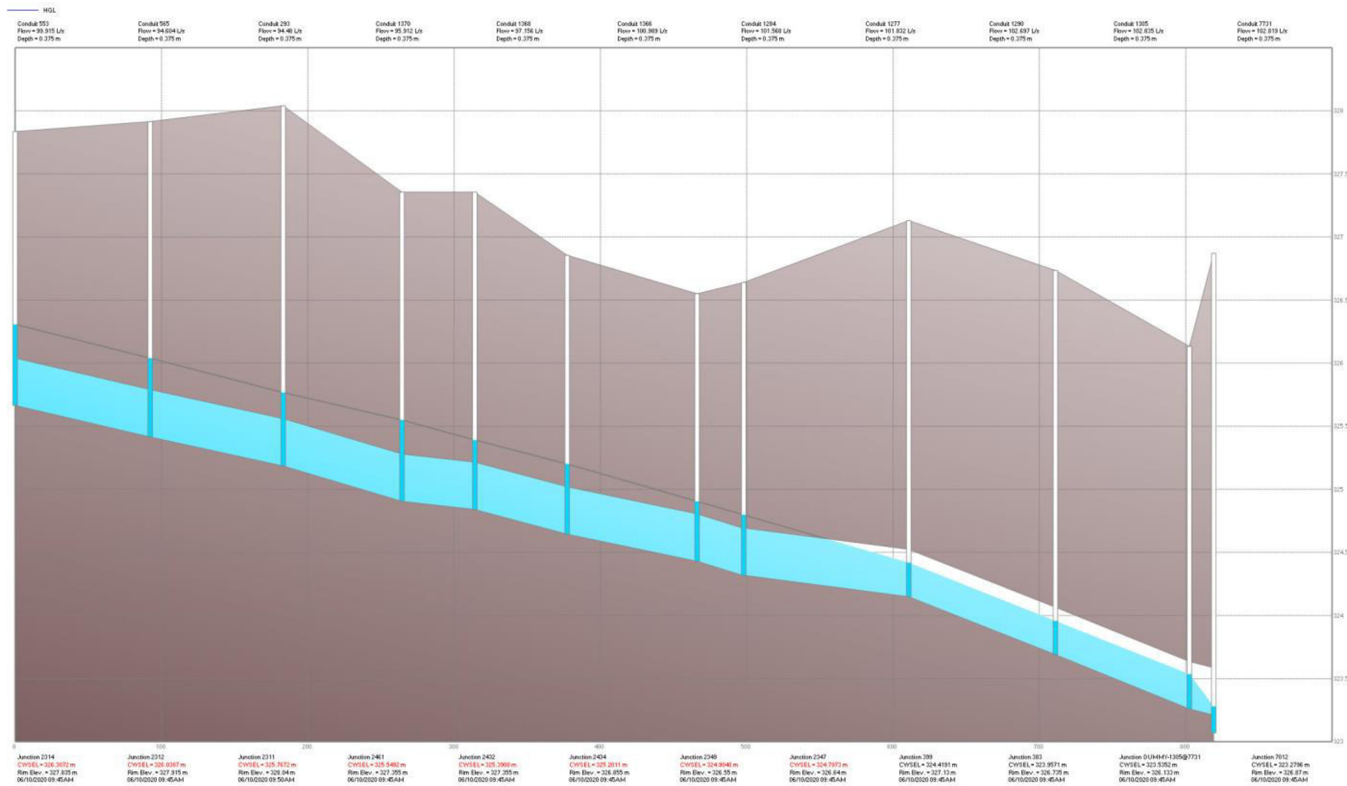


Figure 4-75 Future Conditions WWF Results - Area 3 HGL Profile

As shown above, there are 8 sewers flowing above FPC, with surcharging observed in 8 MHs, and HGLs reaching 0.38 m above the sewer obverts. This location was identified in the Existing Conditions assessment, and the surcharge has increased in the Future Conditions.

The fourth location, noted as Area 4 on Figure 4-65 includes multiple reaches of sewers in the eastern part of the City. The first reach includes sewers with diameters ranging from 200 mm to 350 mm, travelling from Eastview Road to Victoria Road, as shown in more detail below.

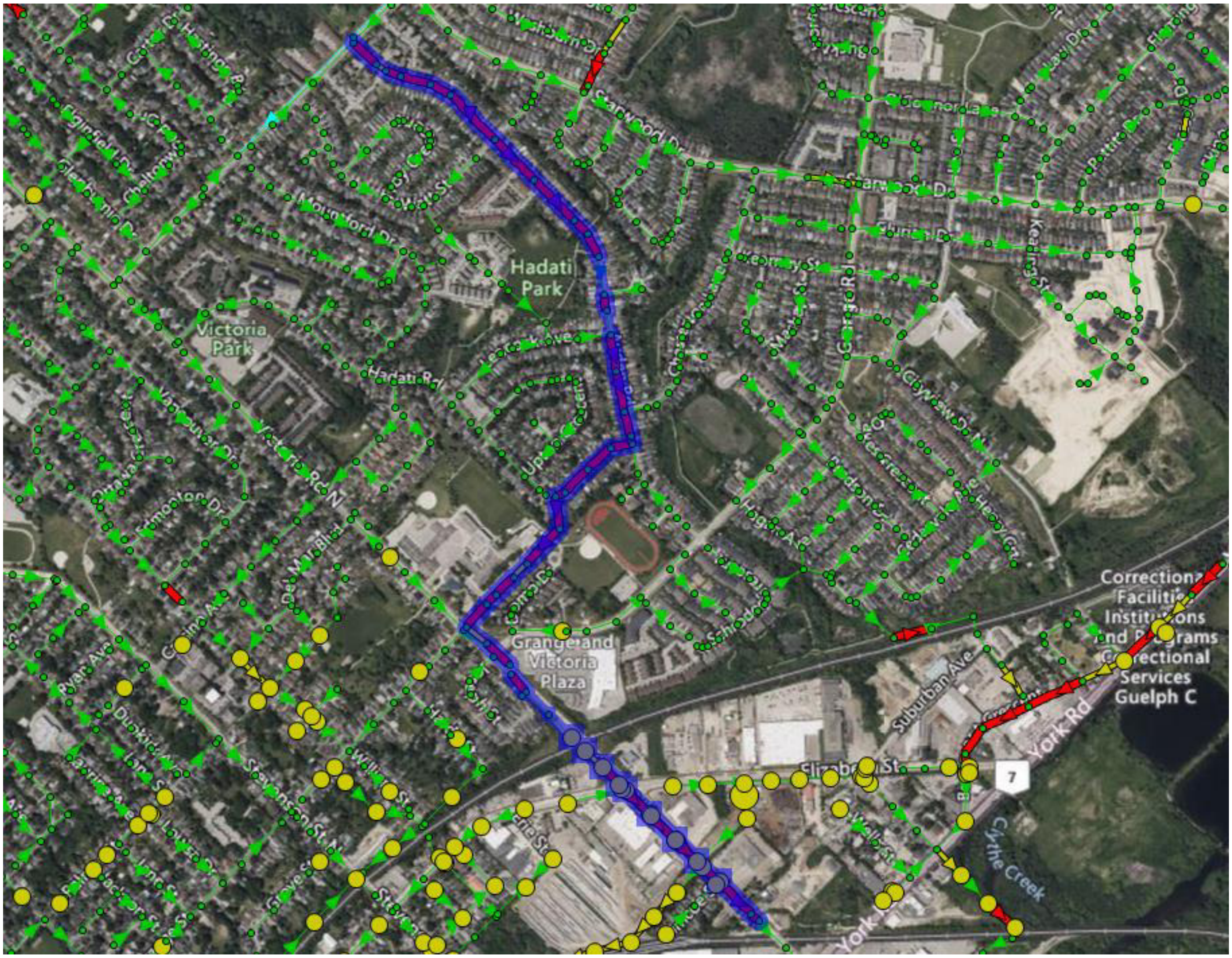


Figure 4-76 Future Conditions WWF Results - Area 4 Site Plan 1

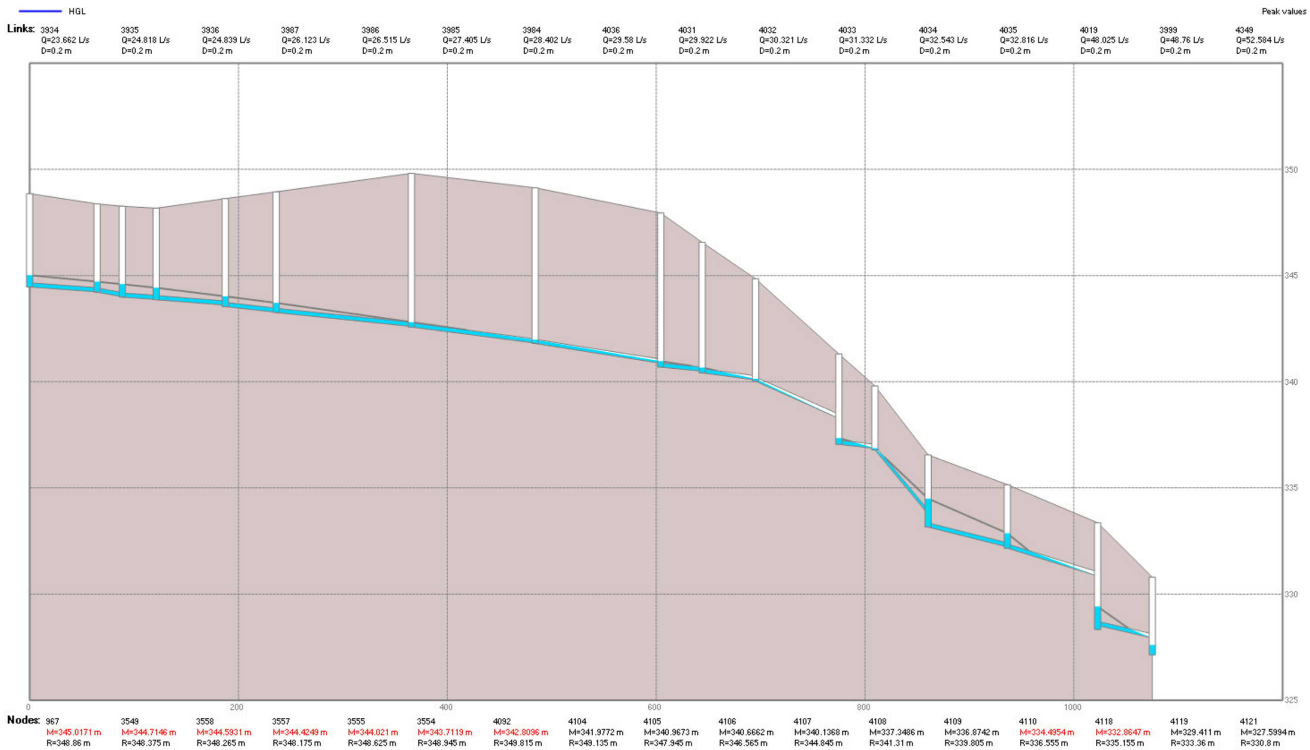


Figure 4-77 Future Conditions WWF Results - Area 4 HGL Profile 1.1

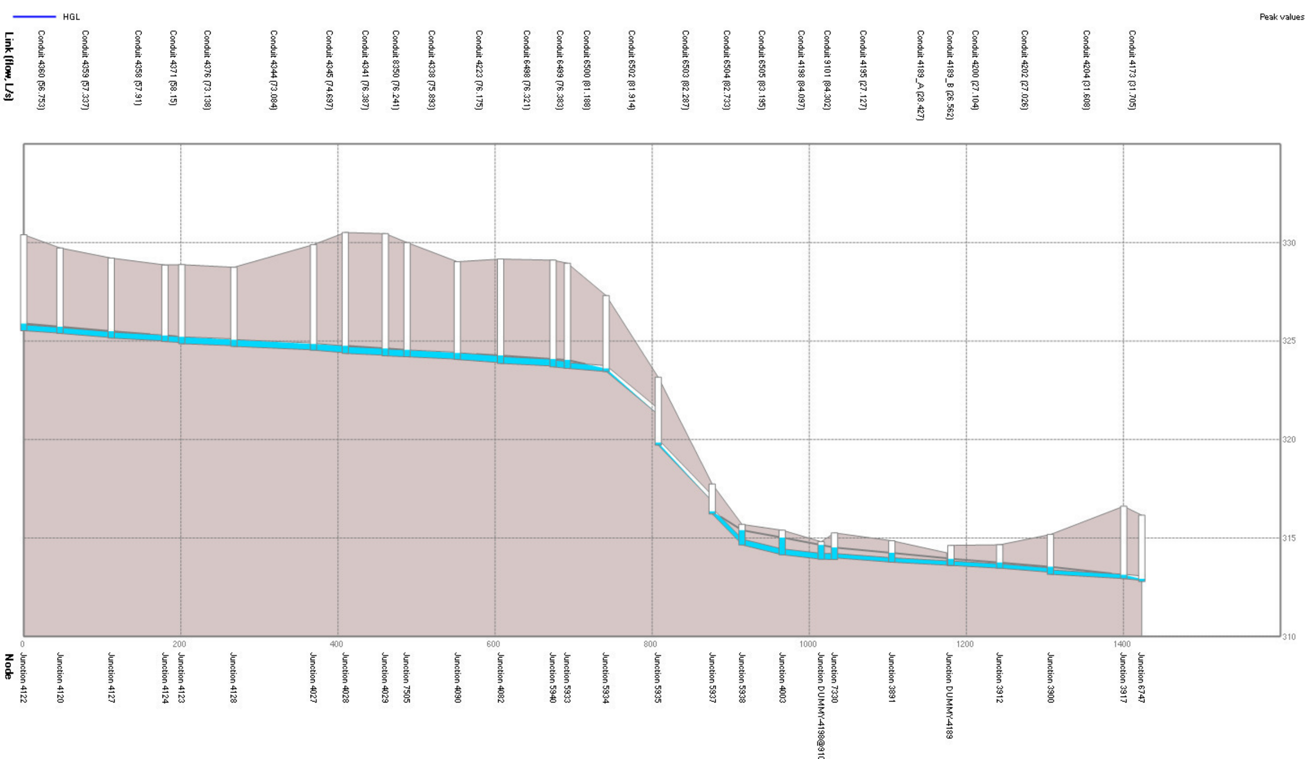


Figure 4-78 Future Conditions WWF Results - Area 4 HGL Profile 1.2

As shown above, there are 34 sewers flowing above FPC, with surcharging observed in most MHs along the reach, and HGLs reaching 1.14 m above the sewer obverts. There are 9 MHs with less than 1.8 m of cover, and 6 MHs with less than 1.0 m of cover. This location was identified in the Existing Conditions assessment, and the extent and degree of surcharge has increased slightly in the Future Conditions assessment.

The second reach in Area 4 ranges includes 675 mm diameter sewers along York Road and Beaumont Crescent , shown in more detail below.

Figure 4-79 Future Conditions WWF Results - Area 4 Site Plan 2

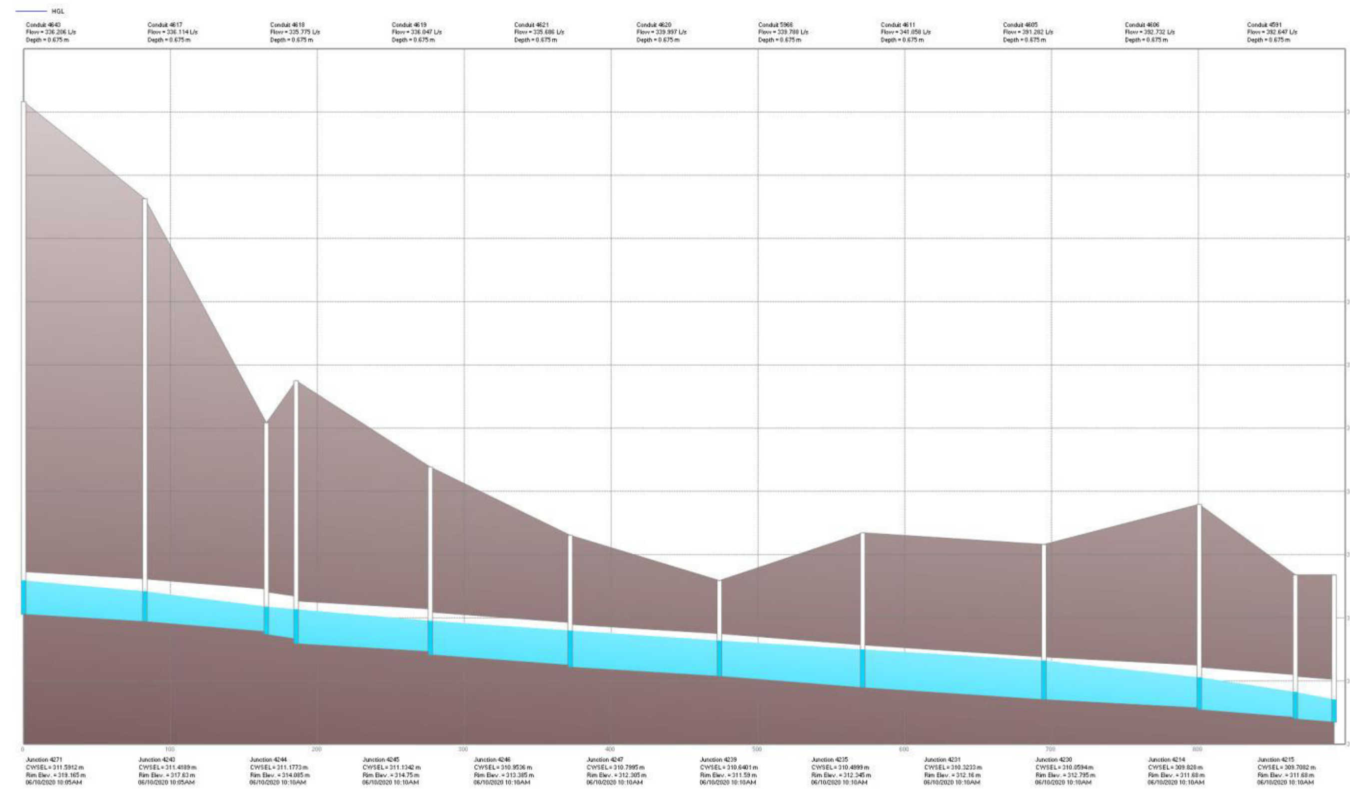


Figure 4-80 Future Conditions WWF Results - Area 4 HGL Profile 2

This location was identified in the Existing Conditions assessment. As shown above, there are 7 sewers flowing above FPC, however no surcharging is observed in any MH. This is because these sewers are generally flowing only slightly above capacity and the constraint is not enough to cause surcharge before the capacity improves downstream. There are 6 MHs with less than 1.8 m of cover, and 1 MH with less than 1.0 m of cover.

The third reach in Area 4 ranges includes 600 mm diameter sewers along Stevenson Street between Ferguson Street and York Road, shown in more detail below.

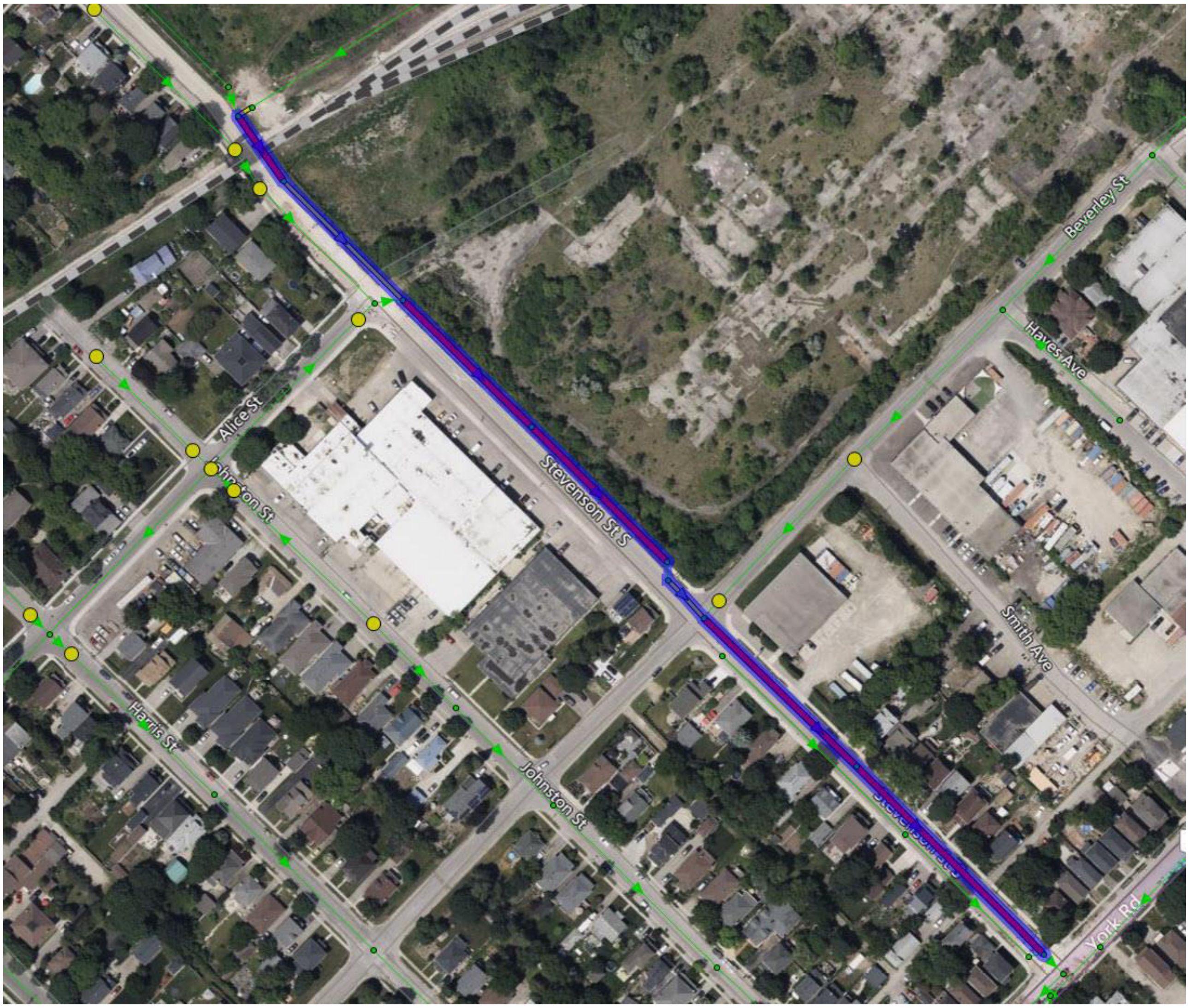


Figure 4-81 Future Conditions WWF Results - Area 4 Site Plan 3

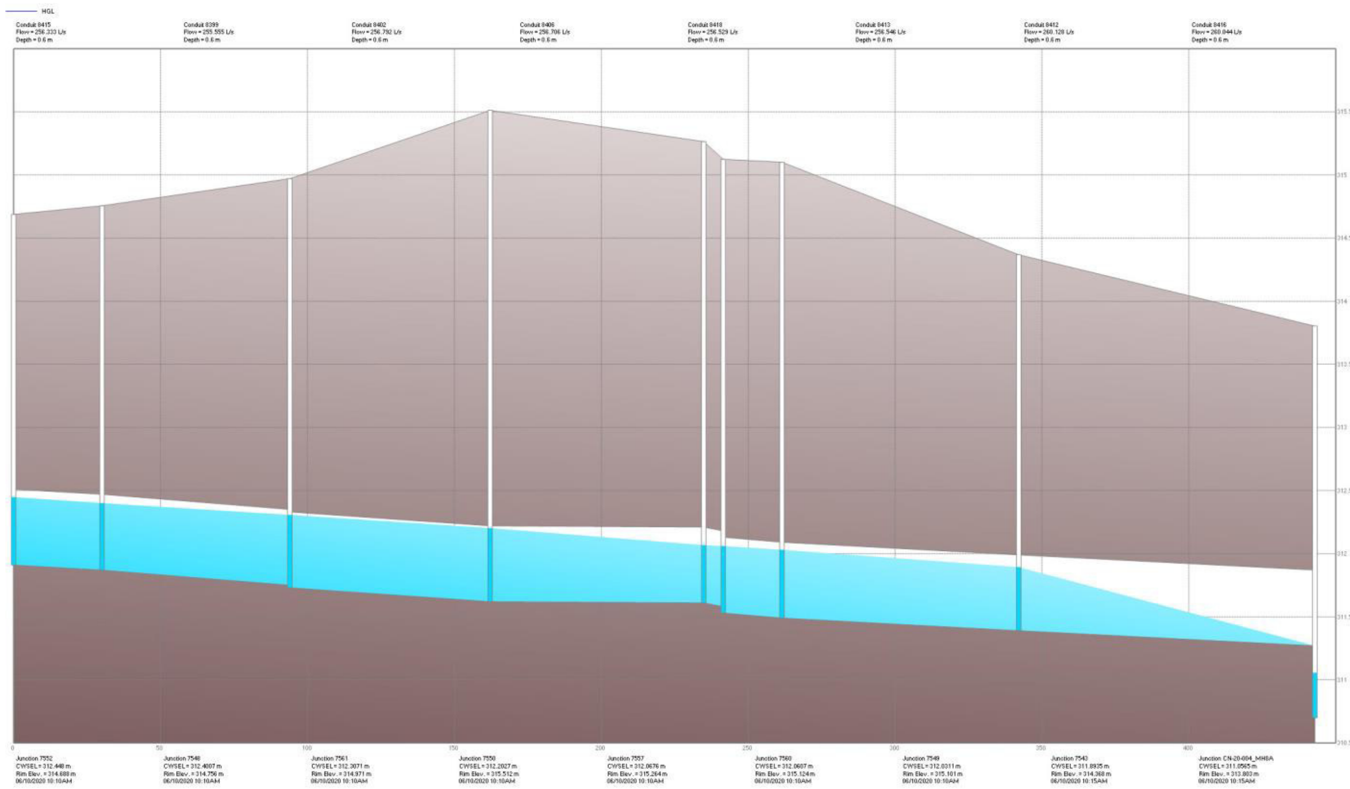


Figure 4-82 Future Conditions WWF Results - Area 4 HGL Profile 3

As shown above, there are 5 sewers flowing above FPC, however no surcharging is observed in any MH. This is because these sewers are generally flowing only slightly above capacity and the constraint is not enough to cause surcharge before the capacity improves downstream. This location was identified in the Existing Conditions assessment, and the extent and degree of surcharge has increased slightly in the Future Conditions assessment. All MHs have greater than 1.8 m of cover.

The fifth location, noted as Area 5 on Figure 4-65 includes multiple reaches in the City centre. The first reach includes sewers ranging in diameter from 1050 mm to 1350 mm, travelling from York Road to the Hanlon Expressway along Wellington Street, as shown in more detail below.

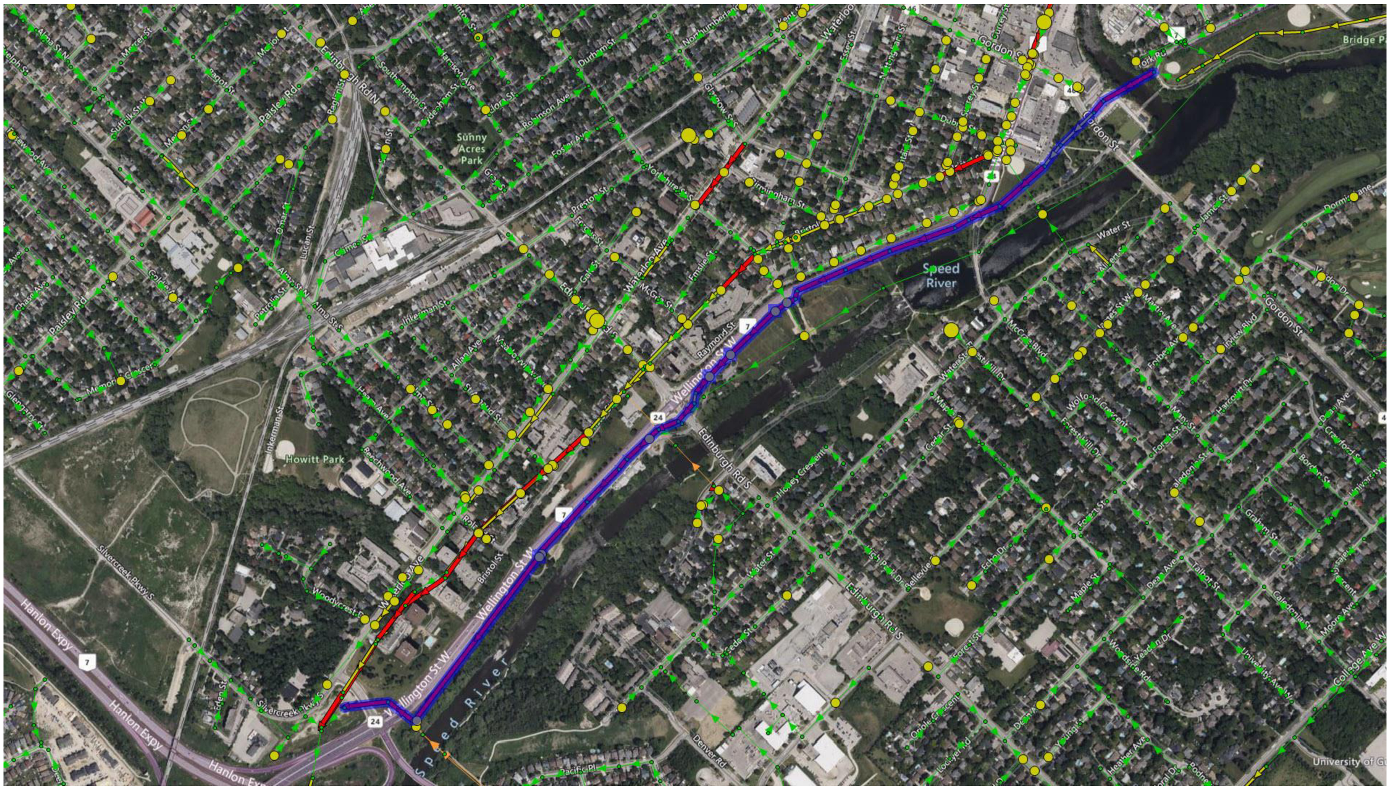


Figure 4-83 Future Conditions WWF Results - Area 5 Site Plan 1

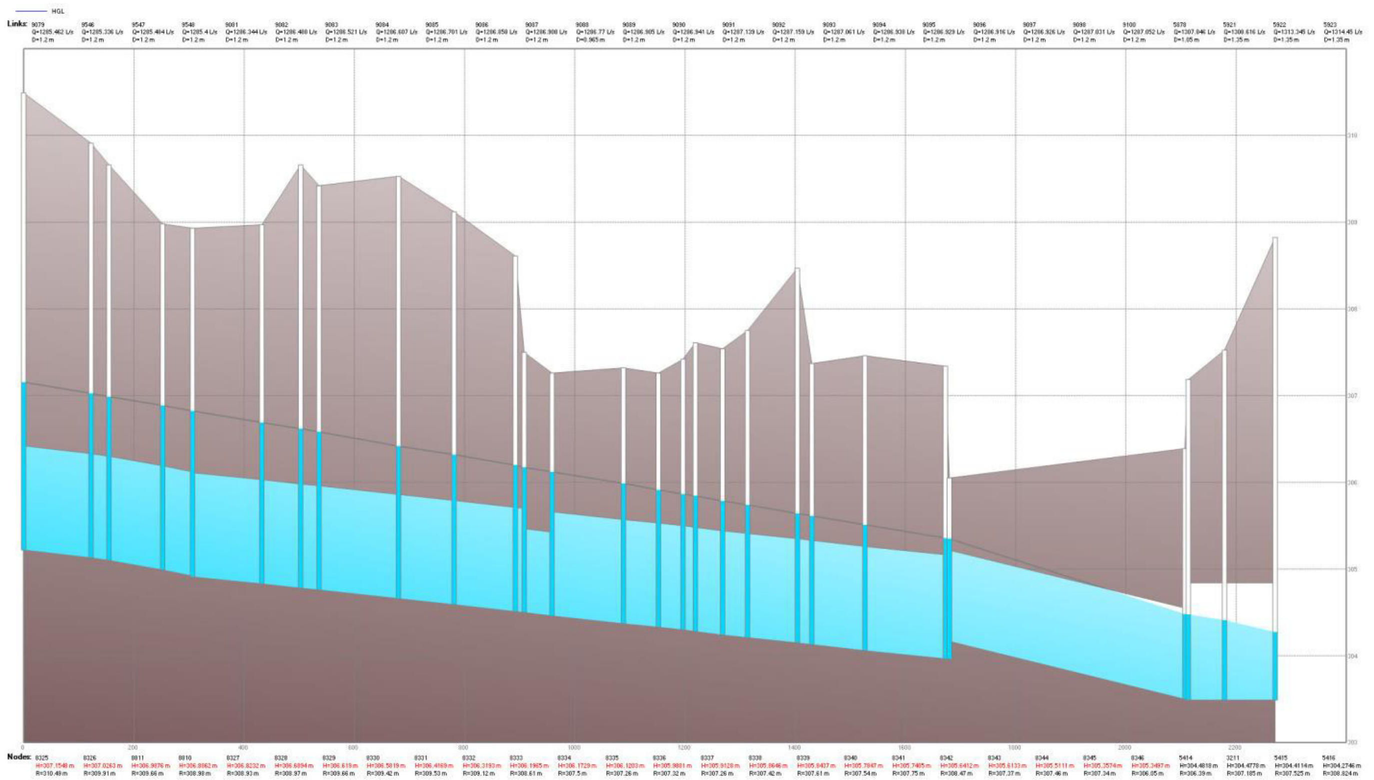


Figure 4-84 Future Conditions WWF Results - Area 5 HGL Profile 1

As shown above, there are 24 sewers flowing above FPC, and there is surcharging is observed in 24 MHs, with HGLs reaching 0.85 m above the sewer obverts. This location was identified in the Existing Conditions assessment, and the extent and degree of surcharge has increased in the Future Conditions assessment.

The second reach in Area 5 is primarily 750 mm sewers travelling from St Arnaud Street to Waterloo Avenue along Bristol Street, as shown in more detail below.

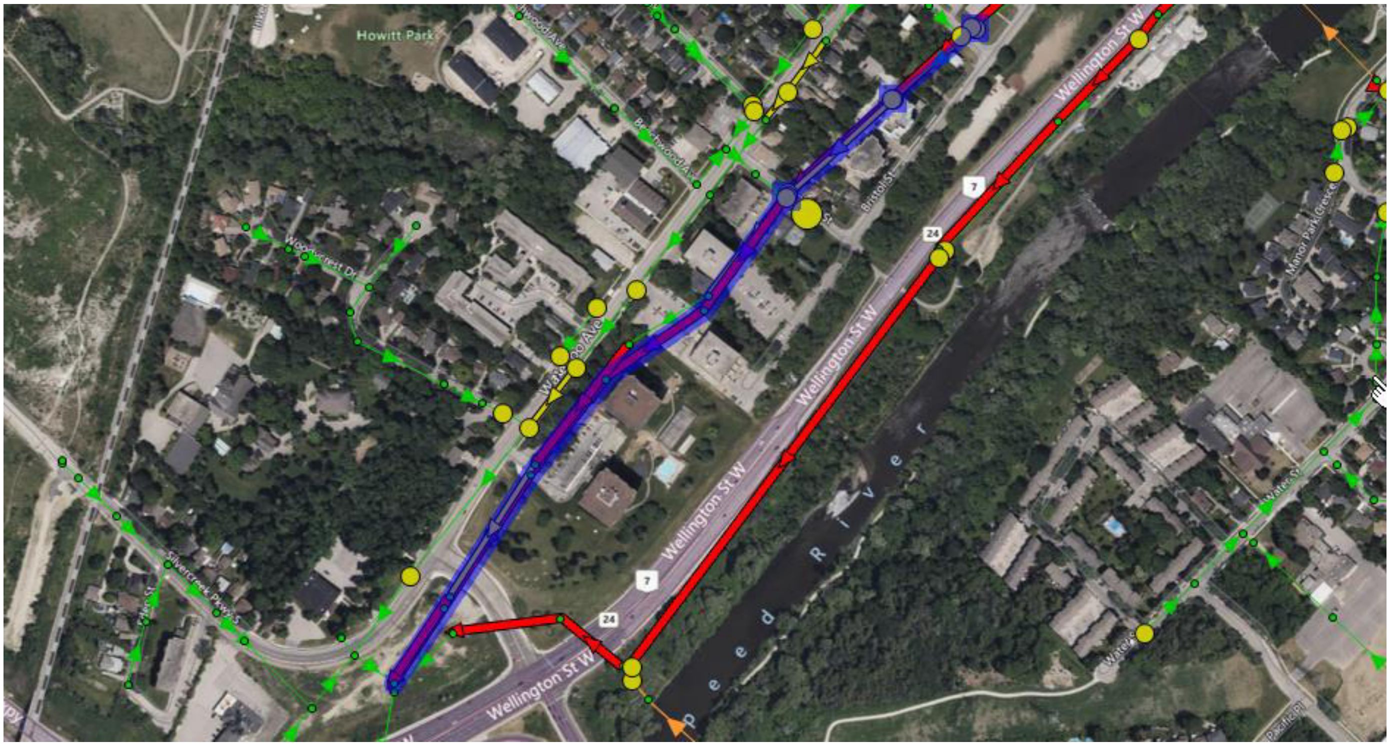


Figure 4-85 Future Conditions WWF Results - Area 5 Site Plan 2

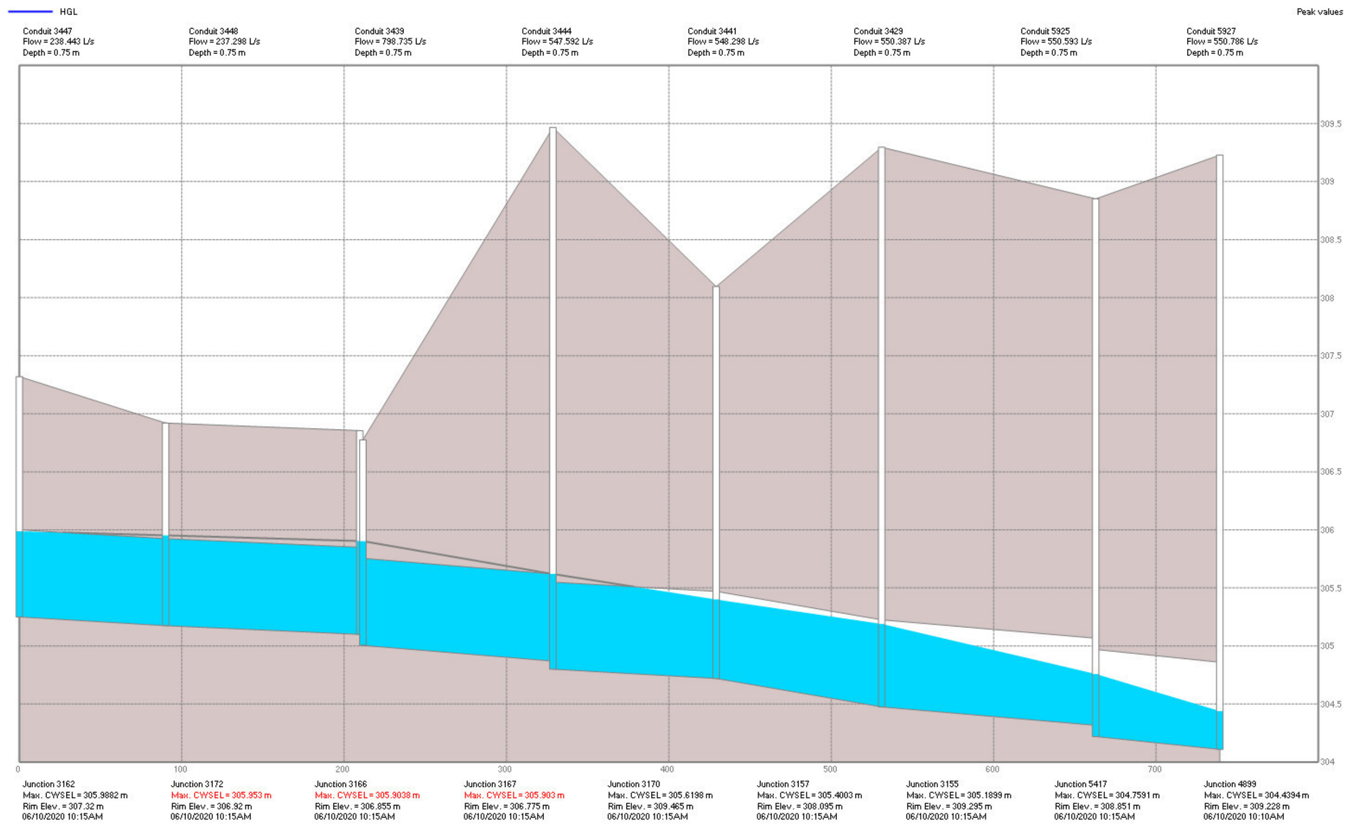


Figure 4-86 Future Conditions WWF Results - Area 5 HGL Profile 2

As shown above, there are 5 sewers flowing above FPC however surcharging is only observed in 2 MHs, with HGL reaching 0.08 m above the sewer obvert. There are 4 MH with less than 1.8 m of cover above the sewer obverts, and 3 MH with less than 1.0 m of cover above the sewer obverts.

The third reach in Area 5 is parallel to the second reach, with sewers ranging in diameter from 450 mm to 600 mm travelling from St Arnaud Street to Waterloo Avenue along Bristol Street, as shown in more detail below.

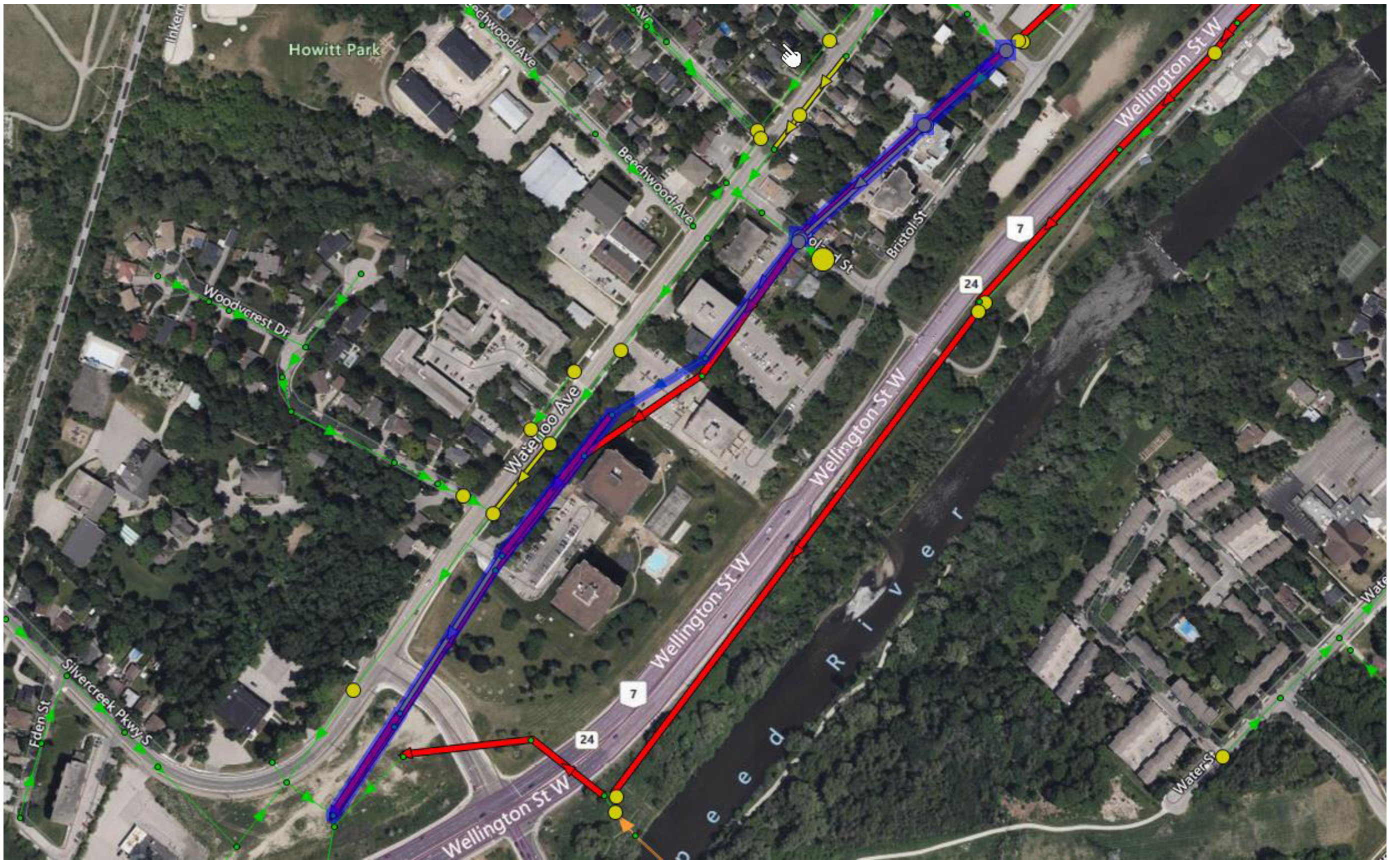


Figure 4-87 Future Conditions WWF Results - Area 5 Site Plan 3

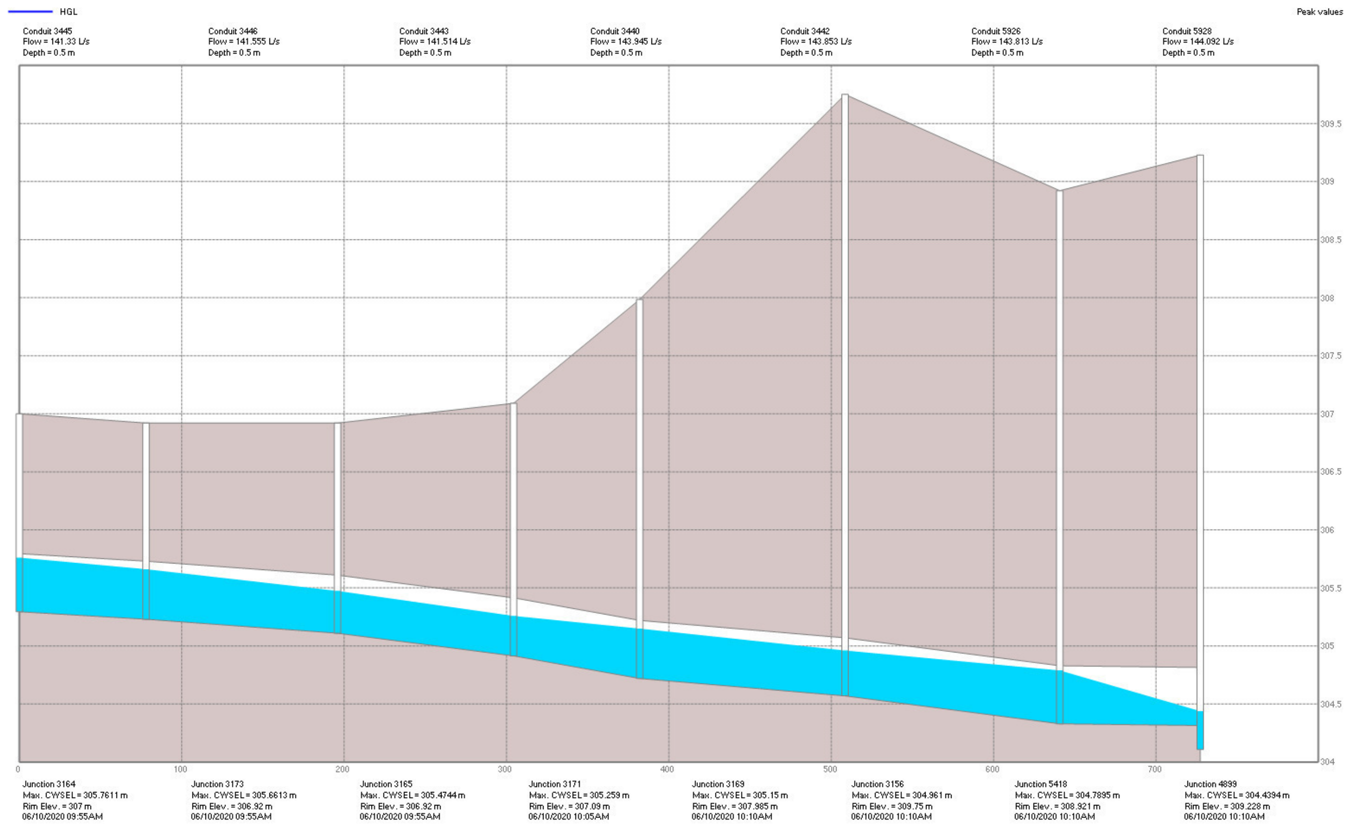


Figure 4-88 Future Conditions WWF Results - Area 5 HGL Profile 3

As shown above, there are 4 sewers flowing above FPC however no surcharging is observed. This is because these sewers are generally flowing only slightly above capacity and the constraint is not enough to cause surcharge before the capacity improves downstream. There are 4 MH with less than 1.8 m of cover above the sewer obverts.

The fourth reach in Area 5 includes 300 mm and 375 mm sewers along Quebec Street and Wyndham Street South, as shown in more detail below.

Figure 4-89 Future Conditions WWF Results - Area 5 Site Plan 4

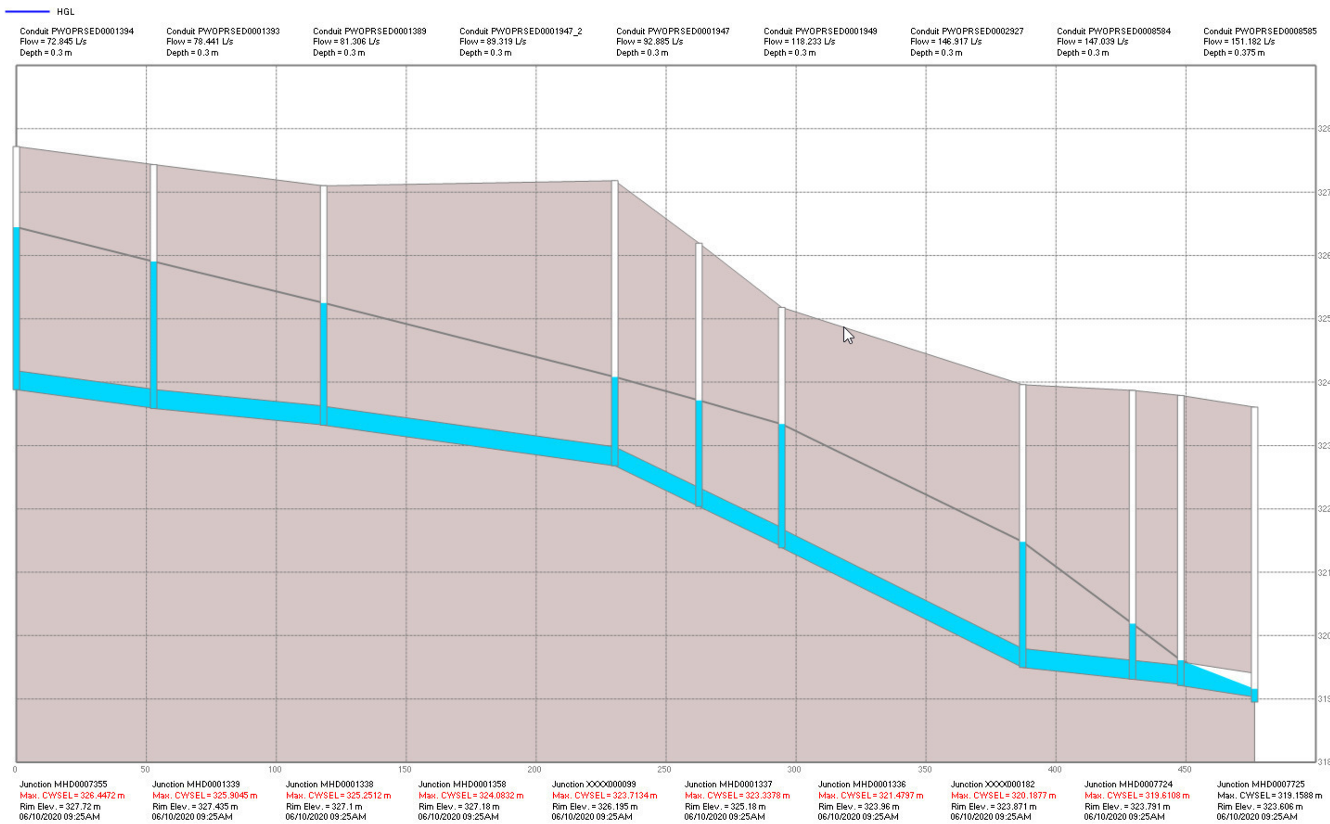


Figure 4-90 Future Conditions WWF Results - Area 5 HGL Profile 4

As shown above, there are 7 sewers flowing above FPC, and surcharging observed in 9 MHs with HGLs reaching 2.27 m above the sewer obverts. There are 2 MHs with less than 1.8 m freeboard.

The fifth reach in Area 5 includes 225 mm diameter sewers along Woolwich Street from Wyndham Street West to Thorp Street, as shown in more detail below.

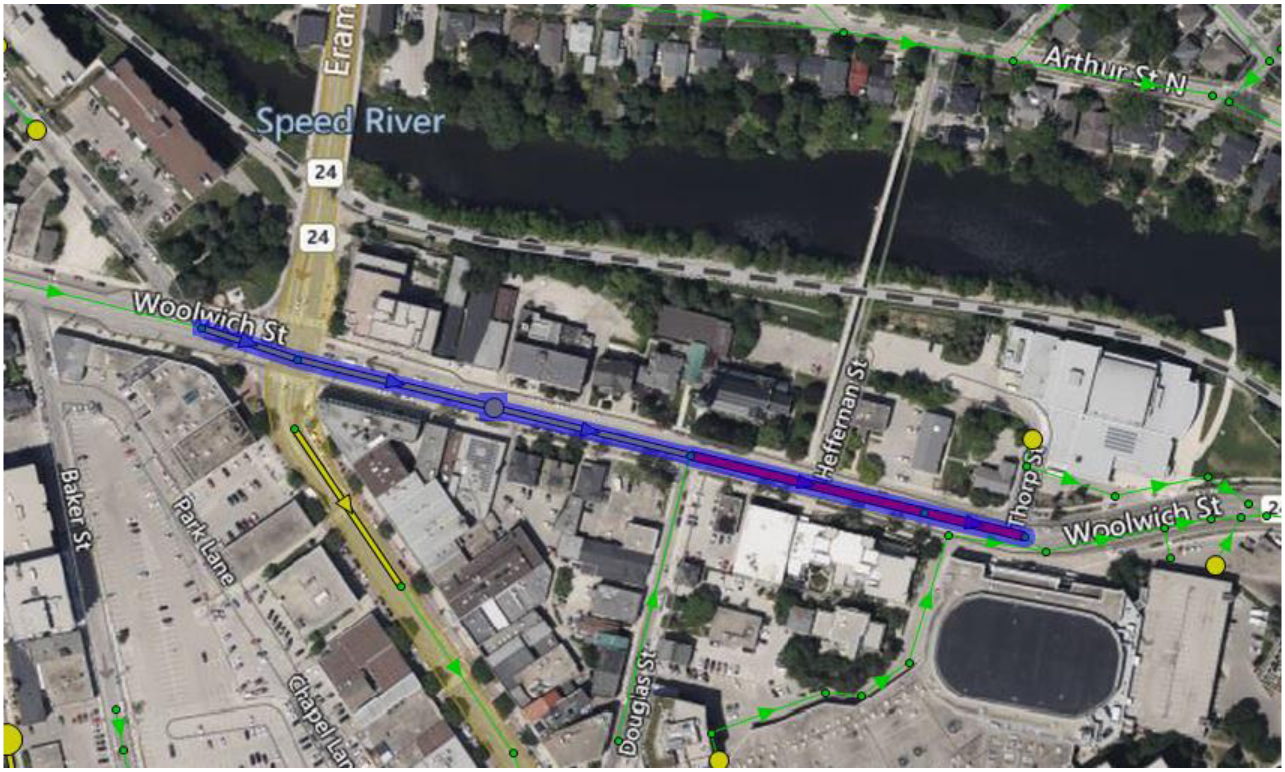


Figure 4-91 Future Conditions WWF Results - Area 5 Site Plan 5

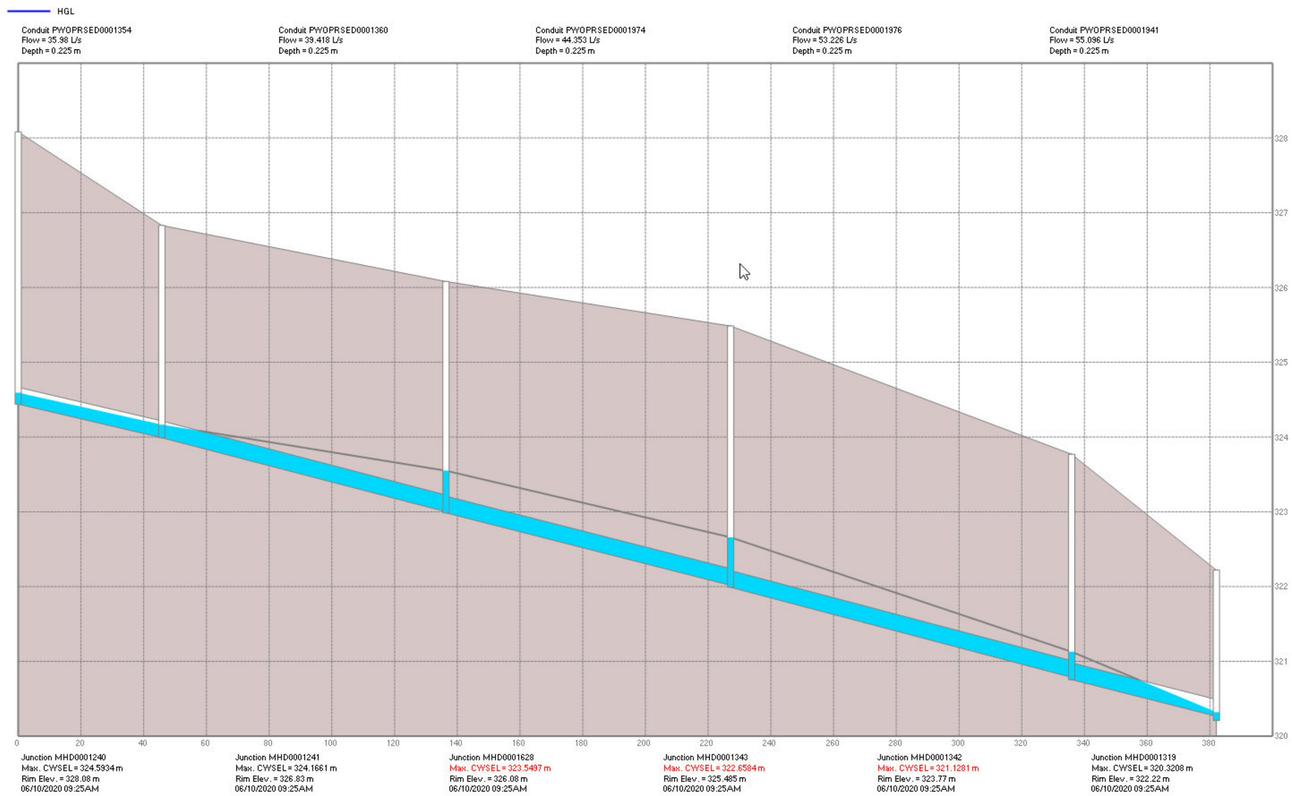


Figure 4-92 Future Conditions WWF Results - Area 5 HGL Profile 5

As shown above, there are 2 sewer flowing above FPC and surcharging observed in 3 MHs, with HGLs reaching 0.45 m above the sewer obverts.

The sixth reach in Area 5 includes 375 mm and 450 mm diameter sewers along Wyndham Street and Wellington Street East, as shown in more detail below.

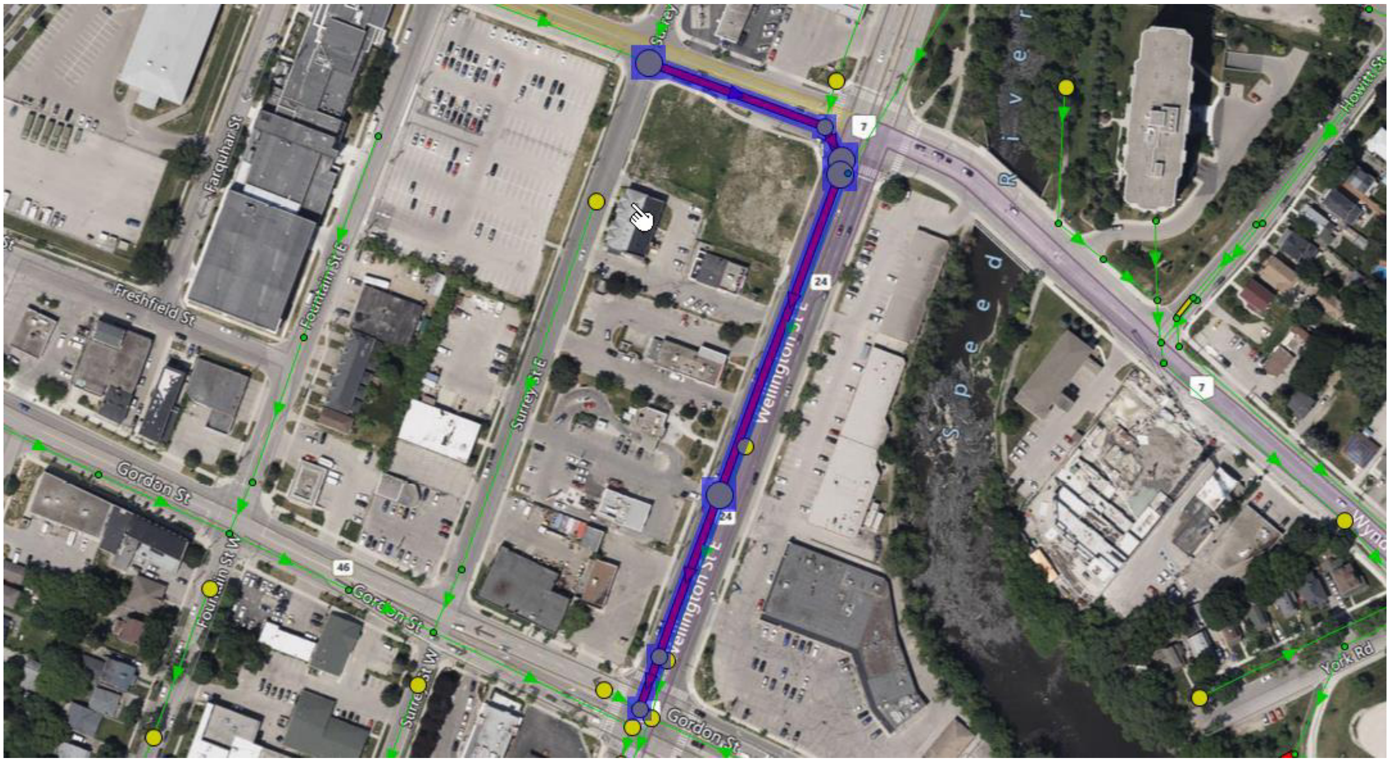


Figure 4-93 Future Conditions WWF Results - Area 5 Site Plan 6

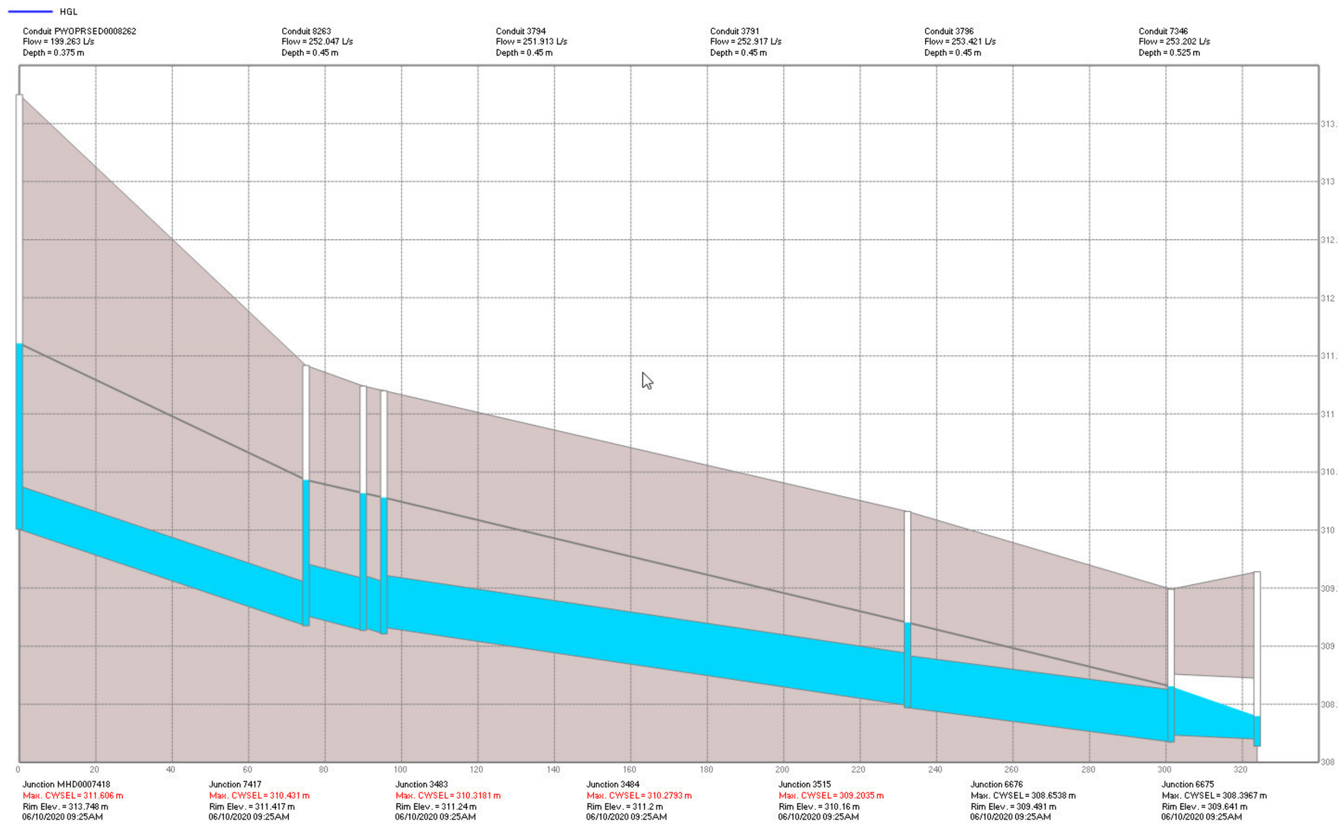


Figure 4-94 Future Conditions WWF Results - Area 5 HGL Profile 6

As shown above, there are 5 sewers flowing above FPC and surcharging observed in 5 MHs, with HGLs reaching 1.23 m above the sewer obverts. There are 4 MHs with less than 1.8 m freeboard in this location.

The seventh reach in Area 5 includes 1050 mm diameter sewers that travel adjacent to the Eramosa River, from Waterworks Place to the confluence with the Speed River, upstream of reach 1 as shown in more detail below.

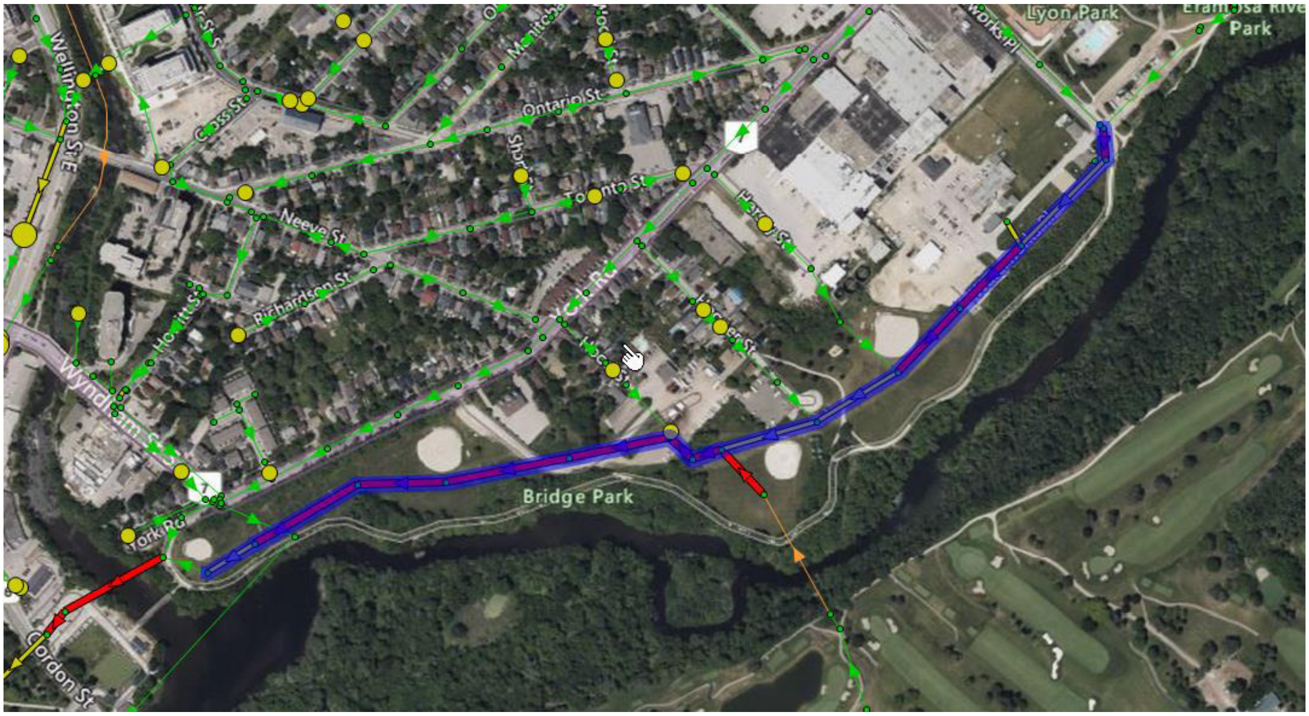


Figure 4-95 Future Conditions WWF Results - Area 5 Site Plan 7

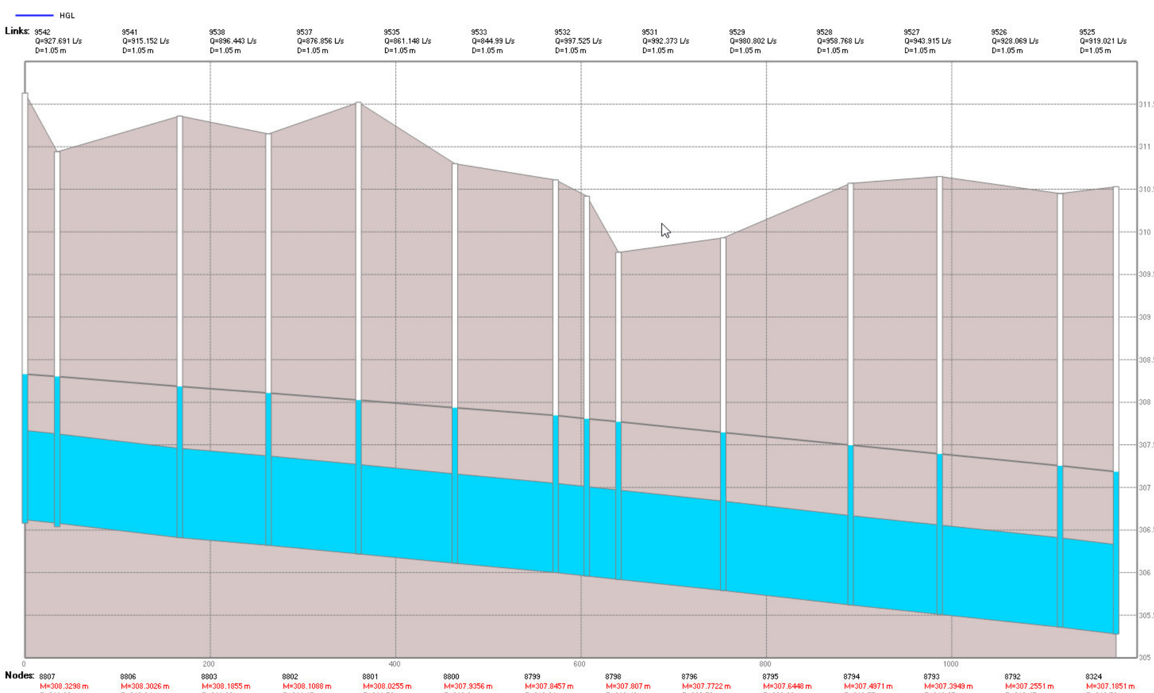


Figure 4-96 Future Conditions WWF Results - Area 5 HGL Profile 7

As shown above, 9 of the sewers in the reach are flowing above FPC, and all MHs are surcharged. This surcharge condition is a continuation of the surcharge observed in reach 1. All MHs in this reach have greater than 1.8 m of freeboard.

The eighth and final reach in Area 5 includes only 3 sewers with 1200 mm diameter near the WRRC, as shown in more detail below.

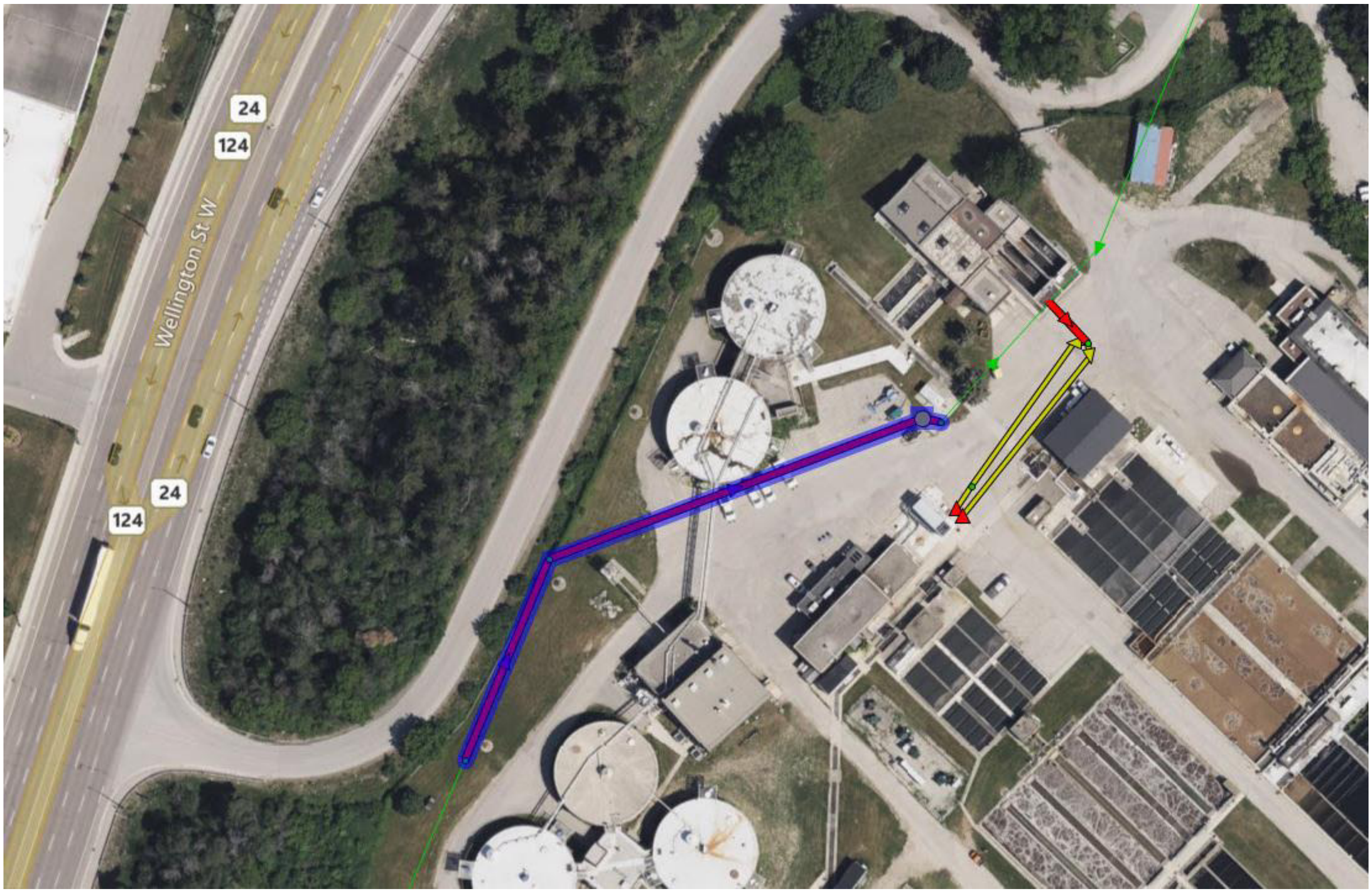


Figure 4-97 Future Conditions WWF Results - Area 5 Site Plan 8

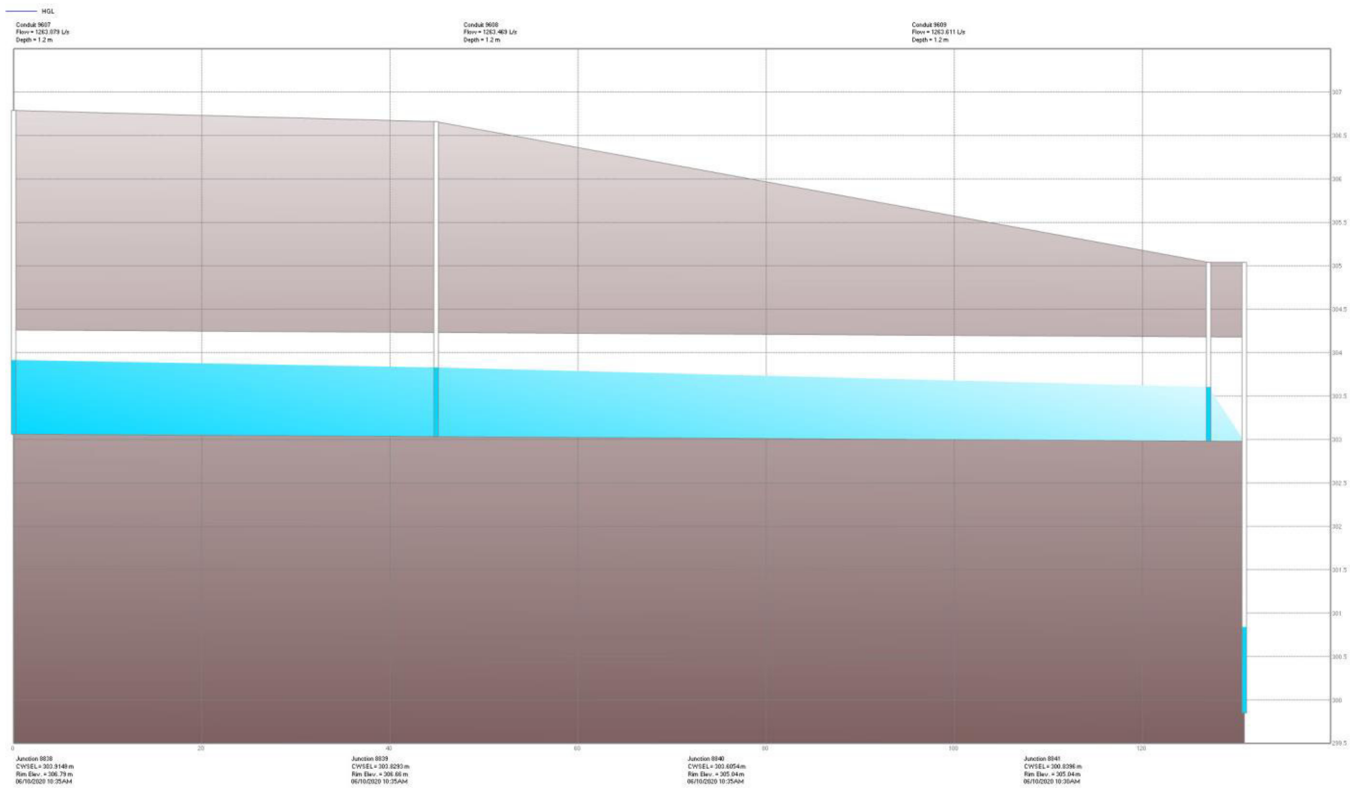


Figure 4-98 Future Conditions WWF Results - Area 5 HGL Profile 4

Similar to the existing conditions assessment, all three sewers are flowing above FPC however the HGL remains within the sewer obverts. This is because they are very flat sewers with slopes of approximately 0.05% - 0.06%. Two of the MHs have less than 1.8 m of cover above the sewer obverts.

The sixth location, noted as Area 6 on Figure 4-65 is a reach including sewers ranging in diameter from 250 mm to 675 mm, travelling along College Avenue and Scottsdale Drive, and across We Hamilton Park as shown in more detail below.

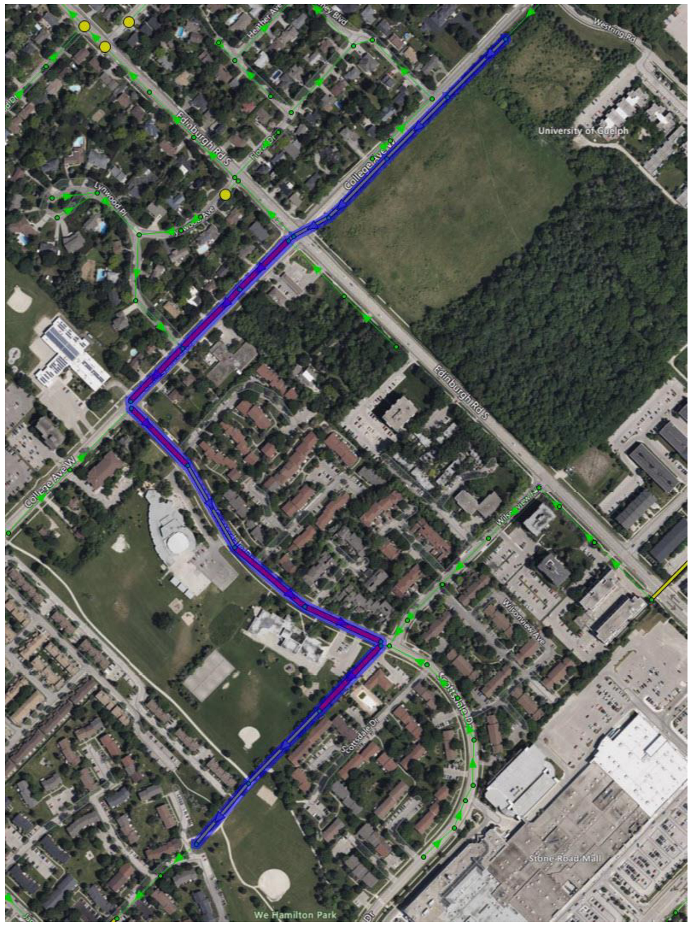


Figure 4-99 Future Conditions WWF Results - Area 6 Site Plan

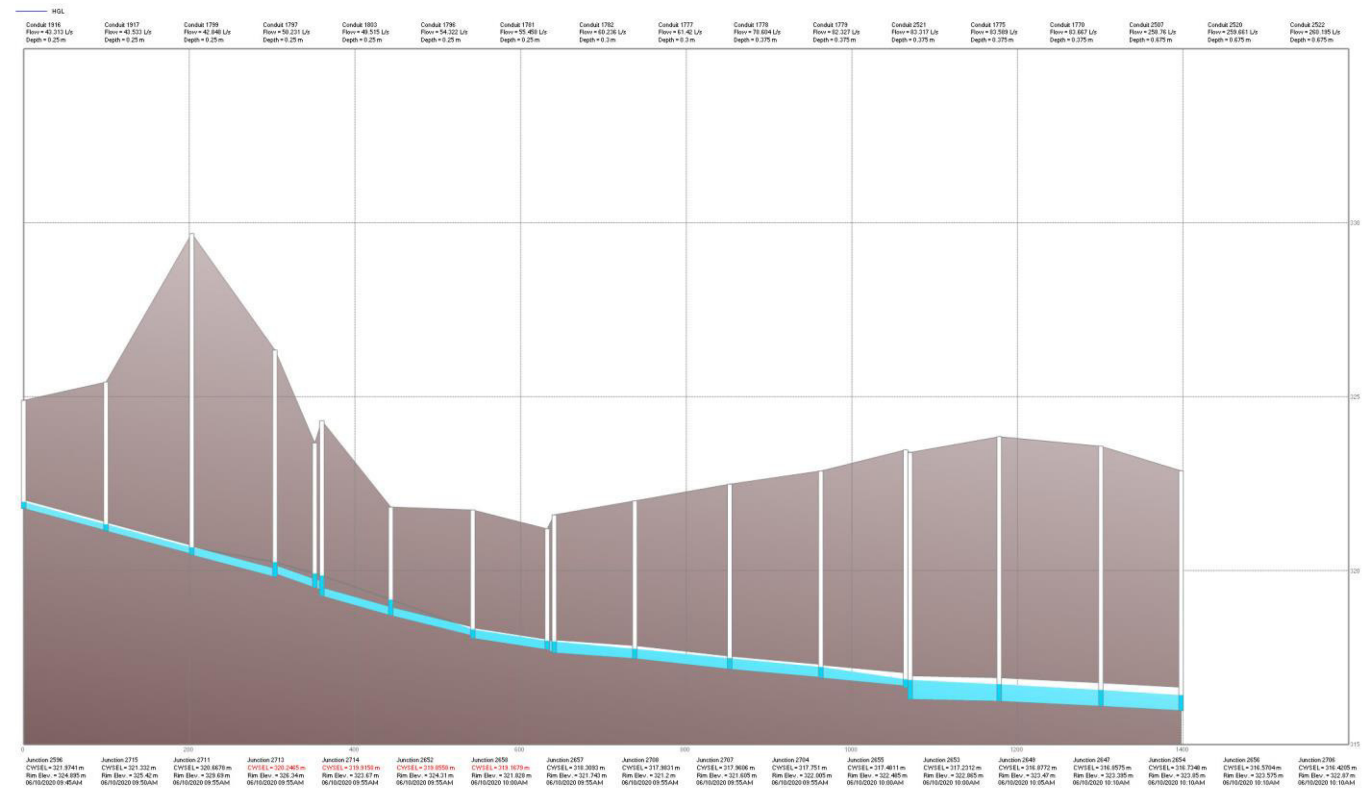


Figure 4-100 Future Conditions WWF Results - Area 6 HGL Profile

As shown above, there are 7 sewers flowing above FPC, but only minor surcharging is observed in 4 MHs with HGLs reaching 0.45 m above the sewer obverts. There are other individual sewers flowing above FPC in Area 6 however there are no surcharging issues related to them identified.

The seventh and final location, noted as Area 7 on Figure 4-65 is a reach of 200 mm and 450 mm sewers travelling along Farley Drive and Clairfields Drive West, as shown in more detail below. This is the location of connection point for the Future Clair Maltby development area.

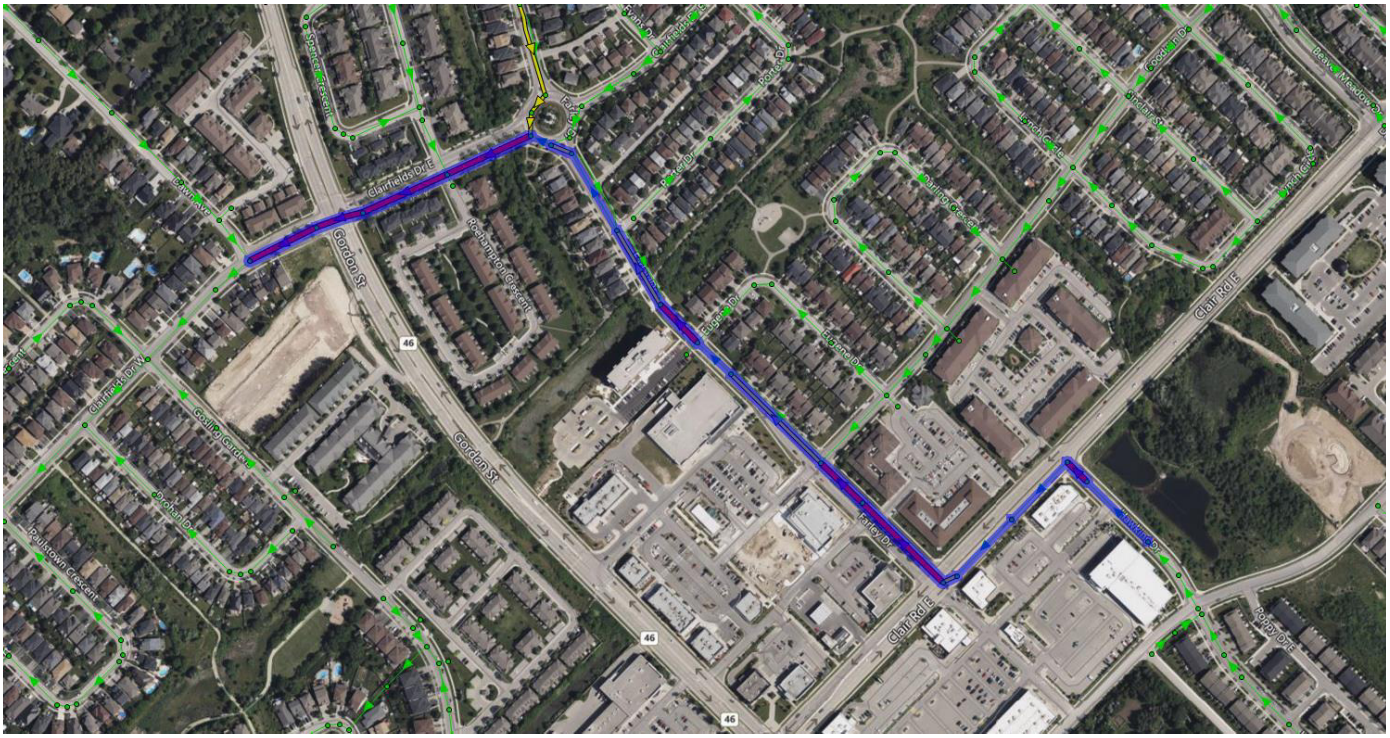


Figure 4-101 Future Conditions WWF Results - Area 7 Site Plan

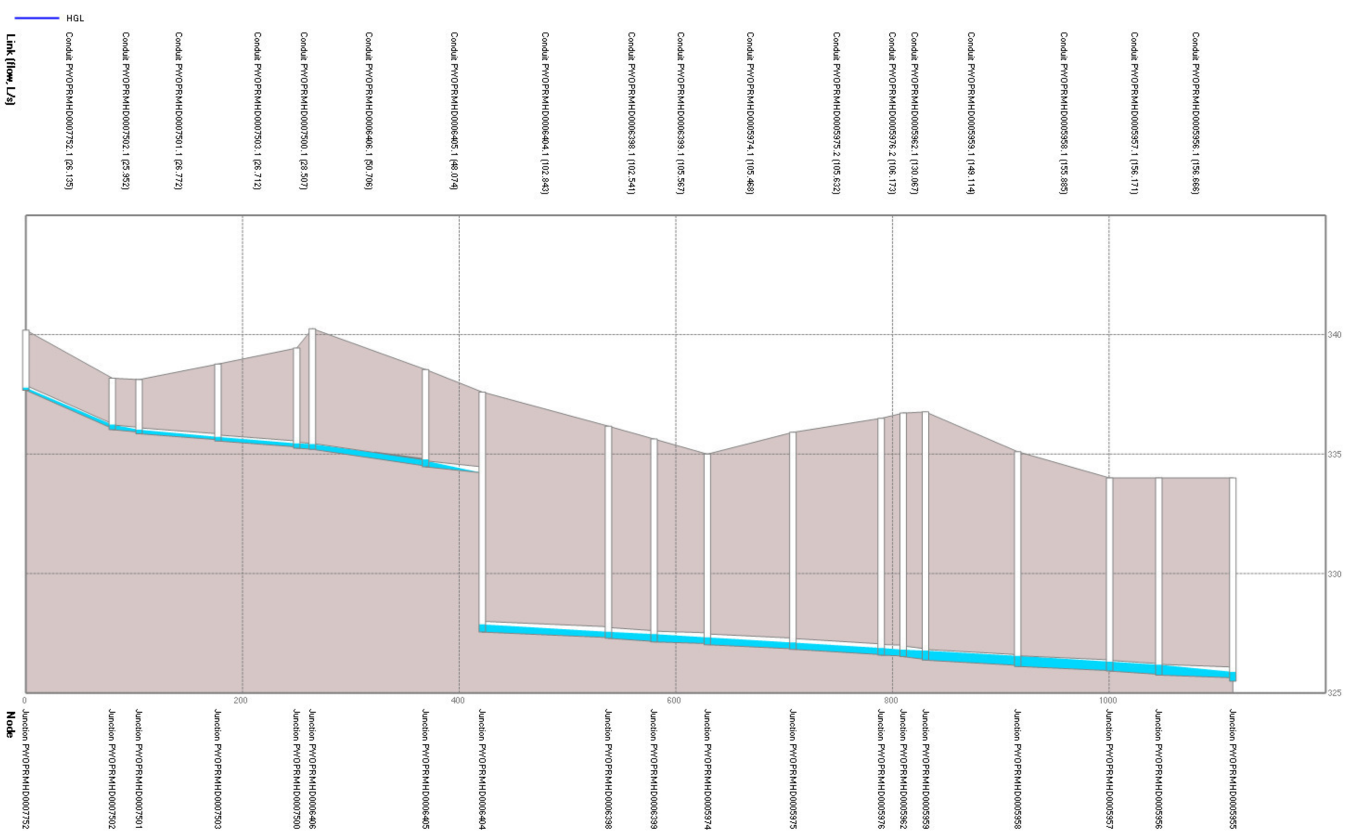


Figure 4-102 Future Conditions WWF Results - Area 7 HGL Profile

As shown above, there are 8 sewers flowing above FPC, but only minor surcharging is observed in 3 MHs with HGLs reaching 0.06 m above obverts. This condition is due to the projected Claire Maltby flows exceeding the capacity of the existing receiving sewers. It should be noted that there are other individual sewers flowing above FPC scattered around the City however a single sewer flowing above capacity is rarely a cause of surcharge and these have all been reviewed and it was determined that they did not pose a risk of surcharge or flooding.

4.2.5 Pump Stations

4.2.5.1 Pump Stations Existing Conditions Analysis

The assessment of the pump stations for the existing conditions assessment was carried out by comparing the peak DWF and WWF flows to the firm capacities of the pump stations. See Table 4-5 below.

Table 4-5 Sanitary Pump Station Existing Results

Sanitary Pump Station	Address	Capacity (L/s)	Peak DWF Inflow (L/s)	Peak WWF Inflow (L/s)
Barton Estates	49 Robin Road	8.9 L/s	0.5 L/s	1.0 L/s
Gazer Mooney	672 Speedvale Avenue East	14.9 L/s	0.6 L/s	11.5 L/s
Kortright Heights	1005 Victoria Road South	130.6 L/s	9.1 L/s	77.5 L/s
Landfill Site on Eastview	186 Eastview Road	19.6 L/s from annual pump data	19.6 L/s	19.6 L/s
NiMa Trails	Shakespeare Drive	Existing: Temporary SPS of unknown capacity Future: 26 L/s	0.7 L/s	11.8 L/s
Northern Heights	68 Ingram Drive	33.0 L/s	4.8 L/s	25.2 L/s
Terraview	51 Terraview Crescent	13.0 L/s	1.2 L/s	1.7 L/s

As shown in Table 4-5, all sanitary pump stations have sufficient capacity to manage the peak incoming flows in both the existing DWF and WWF scenarios. It should be noted that the landfill site includes three pump stations: Main, West, and South as discussed in TM2b. Each station contains two pumps. Weekly pumped volume data (provided by the City) was used to determine the average weekly flow between 2017 and 2019. It is assumed that the pumps work in an alternating sequence. The weekly average flow was calculated by dividing the recorded volumes with the pump's total runtime. The results show that the highest recorded weekly flow was 19.6 L/s. This was conservatively loaded to the model as a constant flow.

4.2.5.2 Pump Stations Future Conditions Analysis

The assessment of the pump stations for the future conditions assessment was carried out by comparing the peak DWF and WWF flows to the firm capacities of the pump stations. See Table 4-6 below.

Table 4-6 Sanitary Pump Station Future Results

Sanitary Pump Station	Address	Capacity (L/s)	Peak DWF Inflow (L/s)	Peak WWF Inflow (L/s)
Barton Estates	49 Robin Road	8.9 L/s	0.5 L/s	1.0 L/s
Gazer Mooney	672 Speedvale Avenue East	14.9 L/s	0.7 L/s	11.5 L/s
Kortright Heights	1005 Victoria Road South	130.6 L/s	9.2 L/s	77.5 L/s
Landfill Site on Eastview	186 Eastview Road	19.6 L/s from annual pump data	19.6 L/s	19.6 L/s
NiMa Trails	Shakespeare Drive	Existing: Temporary SPS of unknown capacity Future: 26 L/s	0.7 L/s	11.8 L/s
Northern Heights	68 Ingram Drive	33.0 L/s	4.8 L/s	25.2 L/s
Terraview	51 Terraview Crescent	13.0 L/s	2.1 L/s	2.5 L/s

As shown in Table 4-6, all sanitary pump stations have sufficient capacity to manage the peak incoming flows in both the existing DWF and WWF scenarios. It should be noted that the Landfill Site pump station was setup based on analysis of the Landfill pump data provided by the City. This is discussed in detail in TM2b.

4.2.6 WRRC

4.2.6.1 WRRC Existing Conditions Analysis

The assessment of the WRRC for the existing conditions assessment was carried out by computing the average dry weather flow (ADWF), peak dry weather flow (PDWF) and peak wet weather flow (PWWF) to use a comparison with the current WRRC capacity. The volumes presented represent the total volume of wastewater flow entering the WRRC for a 48-hour period. The results of the analysis are presented below.

Table 4-7: Existing Conditions WRRC Model Results.

Characteristic	Average DWF (L/s)	Peak DWF (L/s)	Peak WWF (L/s)	DWF Volume (48hrs) (m^3)	WWF Volume (48hrs) (m^3)
Model Result	816.9	1,008	2,812	141,700	171,600

4.2.6.2 WRRC Future Conditions Analysis

The assessment of the WRRC for the future conditions assessment was carried out by computing the ADWF, PDWF and PWWF to use a comparison with the current WRRC capacity. The volumes presented represent the total volume of wastewater flow entering the WRRC for a 48-hour period. The results of the analysis are present below.

Table 4-8 Future Conditions WRRC Model Results

Characteristic	Average DWF (L/s)	Peak DWF (L/s)	Peak WWF (L/s)	DWF Volume (48hrs) (m^3)	WWF Volume (48hrs) (m^3)
Model Result	1,418	1,682	3,543	245,900	291,800

5.0 CONCLUSIONS AND NEXT STEPS

5.1 Water System

Based on the analysis of the water system under existing and 2051+ demand conditions, the following conclusions were made:

1. Storage:
    a. Based on the desktop analysis, there is sufficient storage under existing and 2051+ demand conditions.
    b. Based on the hydraulic analysis, limitations were seen in the model in regard to balancing the Zone 1 storage between the Verney and Clair ETs. The Verney ET was found to overflow while the Clair ET level dropped to 45% full under 2051+ MDD conditions. This is due to the hydraulic connectivity of the system as the Verney ET is located closer and is better connected to the Woods PS than the Clair ET. The Woods Reservoir also drained throughout the day as the Woods PS struggled to maintain the Clair ET level.
    c. The hydraulic analysis showed that in Zone 2, the Paisley Reservoir and Verney ET levels were maintained under existing and 2051+ conditions. The Clythe Reservoir was found to drop below 50% full during peak hour under existing conditions, but this was mitigated under 2051+ conditions with the expanded Clythe Reservoir, inflow from the Clythe Well and the additional supply sources on the east side of Zone 2.
2. Supply:
    a. The desktop analysis showed that the existing and planned future supply sources, as per the 2021 WSMP, are sufficient to meet the projected 2051+ demands.
    b. The hydraulic analysis showed that the sources which supply the Woods Reservoir via the Arkell Aqueduct (Arkell Wells, Glen Collector, Lower Collector and Carter Wells) were not sufficient to maintain the Woods Reservoir level under 2051+ MDD conditions. This was due to the Verney ET overflowing, causing water loss in the system.
3. Pump Capacity:
    a. The desktop analysis showed sufficient pump capacity under existing and 2051+ conditions.
    b. The hydraulic analysis showed that under 2051+ MDD conditions, the pump capacity in the south end of Zone 1 was not sufficient to maintain the Clair ET level, while the north end of Zone 1 was being oversupplied, causing the Verney ET to overflow.
    c. The hydraulic analysis showed sufficient Zone 2 pump capacity under 2051+ MDD conditions, with Paisley, Verney and Clythe PSs running well below their planned firm capacities.
    d. The Clair PS was found to have sufficient capacity to supply Zone 3 2051+ MDD.
4. Fire Flow
    a. Localized fire flow concerns were seen in the model under both existing and 2051+ conditions in areas with small cast iron watermains.
5. Watermain Capacity
    a. Limited watermain capacity and increased demands under 2051+ conditions prevented the Woods PS from being able to supply the south end of Zone 1 to sufficiently maintain the Clair ET level.
    b. The headloss was found to exceed 2 m/km in a number of watermains throughout Zone 2 under 2051+ MDD conditions.
    c. Velocity was not found to exceed 3 m/s under existing or 2051+ conditions.
6. Pressure
    a. Under existing MDD conditions, pressure below 40 psi was seen in the model in pockets of Zone 1 with ground elevations above 344m and one high elevation area on the east side of Zone 2 with ground elevations above 357m.

   b. Under existing MDD conditions, pressure above 100 psi was seen in the model in areas of Zone 1 along the Speed River with elevations below 310m.
   c. Existing low pressure concerns in the south end of Zone 1 were found to worsen under 2051+ conditions when the Clair ET level dropped during peak hour
   d. Existing high pressure concerns along the Speed River in Zone 1 were found to worsen under 2051+ conditions when the Woods PS ran at a higher flow to meet demands and fill the Clair ET.
7. The water system failure analysis showed the following infrastructure to be critical to the system's performance and will be taken into consideration when developing proposed future projects:
   a. High Criticality:
      i. Woods PS
      ii. Arkell Aqueduct
   b. Medium Criticality:
      i. University Watermain River Crossing
      ii. Paisley PS and Reservoir
      iii. Clythe PS
      iv. Clair BPS (2051+ conditions only)

5.2 Wastewater System

The model developed and calibrated as part of the ongoing Master Plan was used as a basis for an assessment of the City's collection system. This was completed by revising the model to include recently completed capital works, as well as the addition of growth-related flows. A failure condition analysis was also completed. A summary of the overall findings includes:

1. There were minor capacity issues (q / Q > 100 %) under the DWF condition for the existing of future scenarios, with minor surcharging observed in one location.
2. Multiple locations are identified as being under capacity under the WWF condition for both the existing and future scenarios.
3. The City's pump stations appear to have adequate capacity for the DWF and WWF conditions under both the existing and future scenarios.
4. The failure analysis provides the extent of surcharge and the spill point for the 20 locations of interest provided by the City.

Of note, it is suggested however that the maintenance hole (MH) top of grade (TOGs) (i.e., surface elevations) be validated in several locations. Approximately 15% of the MH TOGS are shown to be within 1.8m (~6ft) of surface based on the GIS database provided. This is important to verify as our assessment considers the proximity of the hydraulic grade line (HGL) to the surface and how this may reflect increases to basement or surface flooding.

5.3 Next Steps

The findings presented provide the baseline conditions to allow water and wastewater system upgrades to be developed. The next steps for the Master Plan include:

1. Presentation of the Existing and Future Scenario assessment results to the City.
2. Development of preliminary servicing strategies to accommodate growth
3. Presentation of these to the City and selection of the preferred servicing strategy.

6.0 REFERENCES

AECOM, 2021
Final Draft Water Supply Master Plan Update. December 2021. Report Prepared for: City of Guelph

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C3 Water Inc. and Stantec, 2020
Design Criteria, LOS and Sensitivity Analysis: City of Guelph Water and Wastewater Servicing Master Plan. December 2020. Prepared for the City of Guelph

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Drinking Water Works Permit. Guelph Drinking Water System. June 2020.

APPENDIX A –

Water System Failure Analysis Results

Pressure Zone	Failure Scenario	Infrastructure Shutdown	Facility Type/Size	Rationale	Demand Scenario	Duration of Shutdown	Impact on System Pressure	Impact on Fire Flow	Impact on Velocity	Impact on ET Level	Impact on Hospital	Impact on St Josephs Retirement Home	Impact on Southgate Data Centre	Total Score
							% of Guelph that falls < 40 psi	% Decrease			Y/N	Y/N	Y/N	
Zone 1	1	FM Woods PS	Pumping Station/Supply	Largest source in the City	ADD	24-hours	< 81%	No FF in 80% of City	0%	Y	Y	Y	Y	High
	2	Arkell Aqueduct Shutdown	Pumping Station/Supply	Largest supply in the City, Woods Reservoir is available	ADD	1-week	< 81%	No FF in 80% of City	0%	Y	Y	Y	Y	High
	3	FM Woods Reservoir (2 cells offline)	Pumping Station/Supply	Largest supply in the City, less stoarge available	ADD	24-hours	0.0%	0.0%	0%	N	N	N	N	Low
	4	Park St Wells	Pumping Station/Supply	Second largest supply in the City and located near downtown	MDD	24-hours	0.0%	1.9%	0%	N	N	N	N	Low
	5	Stevenson Feedermain	600mm	Connection to the Verney ET. Emma Street.	MDD	24-hours	0.3%	0.1%	0%	N	N	N	N	Low
	6	Metcalfe Watermain (Railway Crossing)	300mm	Extended shutdown duration due to railway. Near downtown.	MDD	72-hours	0.0%	0.1%	0%	N	N	N	N	Low
	7	Downtown River Crossing (Wyndham)	350mm	Extended shutdown duration due to river. Key downtown connection.	MDD	72-hours	0.0%	0.0%	0%	N	N	N	N	Low
	8	University Watermain (River Crossing)	400mm	Extended shutdown duration due to river. Key Woods connection.	MDD	72-hours	1.2%	4.7%	0%	N	N	N	N	Medium
	9	Victoria Rd Watermain	400mm	Connection to Clair ET	MDD	24-hours	0.1%	1.7%	0%	N	N	N	N	Low
	10	Stone Rd Watermain	250mm	Key east-west Zone 1 connection. Near Stone Road Mall.	MDD	24-hours	0.0%	0.4%	0%	N	N	N	N	Low
	11	Wellington Watermain (Crossing at Hanlon)	500mm	Paisley Reservoir fill.	MDD	24-hours	0.1%	1.5%	0%	N	N	N	N	Low
	12	Hanlon Watermain (South of Downey)	400mm	Connection to Clair ET.	MDD	24-hours	0.7%	1.7%	0%	N	N	N	N	Low

Pressure Zone	Failure Scenario	Infrastructure Shutdown	Facility Type/Size	Rationale	Demand Scenario	Duration of Shutdown	Impact on System Pressure	Impact on Fire Flow	Impact on Velocity	Impact on ET Level	Impact on Hospital	Impact on St Josephs Retirement Home	Impact on Southgate Data Centre	Total Score
							% of Guelph that falls < 40 psi	% Decrease			Y/N	Y/N	Y/N	
	13	York Feedermain	600mm	Clythe Reservoir fill.	MDD	24-hours	0.0%	0.0%	0%	N	N	N	N	Low
Zone 2	14	Paisley PS and Reservoir	Pumping Station/Supply	Zone 2 Supply and Storage. Largest Zone 2 Storage.	ADD	24-hours	0.3%	1.8%	0%	Y	N	N	N	Medium
	15	Clythe PS	Pumping Station/Supply	Zone 2 Supply and Storage. Main supply on east side of Zone 2.	ADD	24-hours	7.2%	7.3%	0%	N	N	N	N	Medium
	16	Speedvale ET	Storage	Zone 2 floating storage.	ADD	24-hours	0.0%	2.3%	0%	N	N	N	N	Low
	17	Paisley Feedermain	500mm	Key connection between Paisley PS and Speedvale ET.	MDD	24-hours	0.0%	0.0%	0%	N	N	N	N	Low
	18	Speedvale Feedermain	400mm	Key connection between Robertson PS and Speedvale ET.	MDD	24-hours	0.0%	0.6%	0%	N	N	N	N	Low
	19	Woodlawn Watermain	300mm	Key east-west Zone 2 connection.	MDD	24-hours	0.0%	2.9%	0%	N	N	N	N	Low
	20	Watson Pkwy Watermain	400mm	Key Clythe connection.	MDD	24-hours	0.0%	0.0%	0%	N	N	N	N	Low
Zone 3	21	Clair Rd BPS	Pumping Station/Supply	Only Zone 3 supply.	ADD	24-hours	0.4%	0.6%	0%	N	N	N	N	Low
	21	Clair Rd BPS	Pumping Station/Supply	Only Zone 3 supply.	2051+ ADD	24-hours	2.5%	3.8%	0%	Y	N	N	N	Medium
	22	Poppy Rd Watermain	400mm	Key Clair PS connection.	MDD	24-hours	0.4%	0.4%	0%	N	N	N	N	Low