
APPENDIX F

**Stormwater Management
Report**



FINAL

City of Hamilton

**Upper Wellington Street (Limeridge Road
East to Stone Church Road East)
Schedule C (Phases 1 – 4)
Municipal Class Environmental
Assessment**

Stormwater Management Report

2025-06-27



Upper Wellington Street
(Limeridge Road East to
Stone Church Road East)
Schedule C (Phases 1 - 4)
Municipal Class
Environmental Assessment

Stormwater Management
Report

City of Hamilton

FINAL

Project No.: CA-EI-IM20103037

Date: June 27, 2025

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- A Updated Existing Conditions Hydrology (2021)
- B Updated Existing Conditions Modelling Results
- C 2025 Modelling Updates (Existing Conditions)
- D Future Conditions Hydrology

1 INTRODUCTION

The City of Hamilton (City) has completed a Schedule 'C' Municipal Class Environmental Assessment (Class EA) to identify transportation and stormwater improvements to Upper Wellington Street from Limeridge Road East to Stone Church Road East (ref. **Figure 1-1**). Through this Class EA Study, the City has taken the opportunity to transform this segment of Upper Wellington Street into a "Complete Street", that is safe and comfortable for all road users: pedestrians, cyclists, transit users, and drivers, and people of all ages and abilities.

The existing roadway has an urban cross-section (curb and gutter) from Limeridge Road to Towercrest Drive / Sirente Drive and a rural cross-section (shallow, poorly defined ditches) from Towercrest Drive / Sirente Drive to Stone Church Road. The primary existing storm sewer system along Upper Wellington Street conveys runoff from 130 m (+/-) north of Desoto Drive southerly to Dragoon Drive, 160 m (+/-) south of Stone Church Road. A separate existing storm sewer system conveys runoff easterly on Sirente Drive, which crosses Upper Wellington Street.

This report has been prepared to assess the performance capacity of the existing major and minor drainage systems in order to document existing opportunities and constraints and better inform the subsequent assessment of stormwater management (SWM improvements and development of a preferred SWM strategy in further detail.



Figure 1-1: Study Area Location Plan

2 BACKGROUND INFORMATION

The following background information has been received from the City of Hamilton, obtained through the City of Hamilton's online records management platform Spatially Indexed Engineering Records 'SPIDER', or obtained from previous studies within the City of Hamilton completed by WSP (formerly Wood).

Summary Reports

- Stormwater Management Report, Jamesmount Gardens Development, Diconzo Construction Company Ltd. (Wood, May 2020)
- West Central Mountain Drainage Assessment – Supplemental Capacity Analysis and SWM Sizing, Mewburn and Sheldon Neighbourhoods (AMEC Environment & Infrastructure, October 2011)
- Stormwater Management Report for 1301 and 1315 Upper Wellington Street in the City of Hamilton (S. Llewellyn and Associates Limited, April 2008)

Engineering Drawings

- Vittoria Gardens Development drawings, Ralfana Home Corporation (S. Llewellyn & Associates Ltd., August 2008)
- Toscani Developments Ltd. (6 Sirente Drive), Site Servicing Plan (S. Llewellyn & Associates Ltd., December 2007)
- Stone Church Road East, Road & Sidewalk Reconstruction Upper James to Upper Wellington, City of Hamilton Public Works Department (AECOM, June 2007)
- Rose Gardens Phase 3, Storm Drainage Plan (Weslake Civil Engineers and Environmental Planners, January 2007)
- Desoto Gardens Development drawings, 456941 Ontario Ltd (S. Llewellyn & Associates Ltd., December 2004)
- Pineway Avenue Extension drawings, 456789 Ontario Ltd. (S. Llewellyn & Associates Ltd., September 2002)
- Crerar Park Phase #2, Drainage Area Plan (Urbex Engineering Limited, April 1, 2002)
- Dallia Estates Sewer Drainage Area (professional Engineers and Ontario Land Surveyors, January 31, 2000)
- Wellington Meadows Phase One drawings (Urbex Engineering Limited, August 25, 1997)
- Rose Gardens (Phase 2) Storm and Sanitary Drainage Plan (A.J. Clarke and Associates Ltd., April 1994)
- Oakwood Mews, Municipal Non-Profit (Hamilton) Housing Corporation (A.J. Clarke and Associates Ltd., November 1992)
- Da Vinci Place, Sons of Italy (Hamilton) Housing Corp (A.J. Clarke and Associates Ltd., September 1992)
- Primecan Estates General Plan of Services (A.J. Clarke and Associates Ltd., February 1992)
- Upper Wellington Street from Stone Church Road East to 380 m North, (The Regional Municipality of Hamilton-Wentworth Department of Engineering, August 17, 1990)
- Storm and Sanitary Area Plan Wentworth Place (Sidney W. Woods Inc., June 1987)

GIS and Mapping Data

- Topographic contour data
- Storm sewers, manholes, catch basins, and laterals
- Aerial imagery
- Roads
- Property parcels
- Building footprints

3 EXISTING CONDITIONS

3.1 Modelling Updates (2021)

3.1.1 Modelling Approach

Numerous potential modelling tools are available for the assessment of drainage and stormwater management. For the subject area, the potentially available models include the City of Hamilton's MIKE Urban model (a trunk sewer model developed as part of the City's Stormwater Master Plan, 2007), and a dual-drainage (overland flow and storm sewer links) PCSWMM model of the West Central Mountain area (as per "West Central Mountain Drainage Assessment – Supplemental Capacity Analysis and SWM Sizing, Mewburn and Sheldon Neighbourhoods" AMEC Environment & Infrastructure (now WSP), October 2011).

A meeting between WSP (formerly Wood) and City staff was held on December 1, 2020 to discuss the modelling options. Overall, WSP noted that the PCSWMM model was preferred, given that it included representation of both minor (storm sewer) and major (overland flow / roadway) hydraulic elements and inlet capacity restrictions (i.e. catchbasins), and that the modelling utilizes the EPA-SWMM computational engine, which can be opened and reviewed by the City using freely available EPA-SWMM software. Ultimately, City staff concurred with the recommendation; modelling has proceeded using PCSWMM accordingly.

The subject PCSWMM model was primarily focused on an assessment of the Mewburn-Sheldon Lands (largely between West 5th Street and Upper James Street, and Rymal Road and Stone Church Road) and an appropriate SWM strategy for these lands to mitigate impacts to the downstream trunk storm sewer system. The modelling extended trunk storm sewers and drainage areas beyond this area, to the Upper Wellington and Upper Wentworth Street areas, in order to adequately assess conditions.

This modelling was most recently updated to assess potential impacts from the Jamesmount Gardens Development for Diconzo Construction Company Ltd. (Wood, May 2020). In addition to the primary refinements in the vicinity of the subject development (Diconzo Drive between Regina Drive and Aquasanta Drive), the model also made further adjustments, most notably revising inlet functions (connection between overland road elements and storm sewers) to be orifices rather than inlet rating curves.

The results of this previous modelling were documented in the "West Central Mountain Drainage Assessment – Supplemental Capacity Analysis and SWM Sizing, Mewburn and Sheldon Neighbourhoods" (AMEC Environment & Infrastructure, October 2011) and the Jamesmount Gardens Development for Diconzo Construction Company Ltd. (Wood, May 2020). The modelling completed as part of those assessments has been used as the basis for the current assessment. Several additional updates and modifications have however been incorporated to reflect more current modelling approaches, as well as selected changes which have occurred in the drainage area to better reflect updated existing conditions. These updates are outlined further in subsequent sections.

3.1.2 Hydrology

The base PCSWMM modelling previously applied a Soil Conservation Service (SCS) Curve Number infiltration methodology, with a base Curve Number of 74 to represent open spaces in good condition for type "C" soils. The

total imperviousness was split into residential imperviousness and non-residential imperviousness (roadway, commercial, institutional) land uses. It was assumed that 60% of the residential land use area was directly connected to the subcatchment outlet, while all the non-residential land use areas were assumed to be directly connected to the subcatchment outlets. The subcatchment SCS Curve Numbers were increased accordingly using an area weighting approach to account for the difference between the total imperviousness and the directly connected imperviousness for residential areas.

Several developments have occurred in the West Central Mountain Drainage area since the completion of the 2011 assessment and several more have been proposed. For the purposes of this study, it has been anticipated that the proposed developments would apply SWM controls that would generally mimic pre-development flow conditions. As such, the drainage areas for those developments have been assumed to remain consistent with the pre-development land use scenario applied in the 2011 assessment. Further, assuming that the same assumptions for external lands are applied for all modelling scenarios, the overall findings relative to the study area should be reasonable.

The residential areas which directly contribute to the Upper Wellington Street study area or are directly downstream have been updated for the current study. These areas include Desoto Drive, Ralfana Crescent, Pineridge Drive, Chartwell Circle, and areas contributing to Sirente Drive between Upper Wellington Street and the Crerar Neighbourhood. Updated subcatchment boundaries and parameters have been developed accordingly. Hydraulic elements have also been updated, as described in **Section 3.1.3**.

The PCSWMM model subcatchment boundaries have been updated to reflect existing (2021) conditions based on available drawings (as available from the City's SPIDER database), the City's current 1 m contour data, and current aerial photography. Updated subcatchment boundaries and imperviousness information are presented in Drawings 1 to 3 (attached at the end of this report). The updates are summarized as follows:

- The extent of previous subcatchment C17 has been updated and split into catchments C17_1, C17_2, and C17_3 based on drainage areas shown on the Wellington Meadows Phase One drawings (Urbex Engineering Limited, August 25, 1997), drawing number 98-S-55_6, and available contour data.
- The extent of previous subcatchment C18 has been updated and split into subcatchments C18_1 to C18_13 and C18_23 to C18_26 based on drainage areas shown on the Vittoria Gardens Development drawings (S. Llewellyn & Associates Ltd., August 2008), drawing number 08-S-32_10, and available contour data and property parcel data has been used to identify the Rights-of-way (ROW) subcatchments.
- The extent of previous subcatchments C17, C18, and F02 have been updated and split at the Vittoria Gardens Development into subcatchments C18_14 to C18_22 based on drainage areas shown on the Vittoria Gardens Development drawings (S. Llewellyn & Associates Ltd, August 2008), drawing number 08-S-32_10, and available contour data.
- The west side of previous subcatchment F02 has been split through Jerome Neighbourhood Park to create subcatchments F02_4 and F02_9 based on the available contour data; Upper Wellington Street has been split to create subcatchments F02_1 to F02_3 based on available contour data and property parcel data has been used to identify the ROW subcatchments.
- The extent of the north side of previous subcatchment F02 has been updated and split into subcatchments F02_5 to F02_8 based on drawings for Oakwood Mews (A.J. Clarke and Associates Ltd., November 1992) and Da Vinci Place (A.J. Clarke and Associates Ltd., September 1992), drawing numbers LSP1711 and LSP1712 respectively, contour data, and property parcel data has been used to identify the ROW subcatchments.

- The eastern limit of previous subcatchment F02 has been split to create subcatchment F04_18 based on the Toscani Developments Ltd. Site Servicing Plan (S. Llewellyn & Associates Ltd., December 2007), drawing number LSP_2739.
- The downstream external subcatchments contributing to Sirente Drive, F04_01 to F04_17 have been added to the model based on the Crerar Park Phase #2 drawing (Urbex Engineering Limited, April 1, 2002), drawing number 02-S-21-04A, Rose Gardens Phase 3 (Weslake Civil Engineers and Environmental Planners, January 2007), drawing number 07-S-24_3R, Wentworth Place (Sidney W. Woods Inc., June 1987), drawing 88-S-35_4, Rose Gardens (Phase 2) (A.J. Clarke and Associates Ltd., April 1994), drawing 94-S-42_15.
- The Upper Wellington Street overpass of the Lincoln M. Alexander Parkway has been added to the PCSWMM model as subcatchments F02-5, contributing to Towercrest Drive / Sirente Drive, and H01, contributing to Limeridge Road East. Drawings 10-H-17_01 and 10-H-17_02, Upper Wellington Street Road Resurfacing and Sidewalk Construction (City of Hamilton Public Works Department, April 2010) indicate that a 300 mm storm sewer commences 85 m (+/-) south of Limeridge Road and discharges to the Limeridge Road storm sewer system. Drawing 10-H-25_8, Lincoln M. Alexander Parkway Road Resurfacing and Associated Works (City of Hamilton Public Works Department, March 2011) indicates that there are two catch basins 195 m (+/-) north of Towercrest Drive / Sirente Drive which discharge to the Lincoln M Alexander Parkway storm sewer system. The Upper Wellington Street drainage area conveyed to these two catch basins, approximately 680 m² (+/-), has been conveyed to the Limeridge Road storm sewer system, consistent with overland flow (Upper Wellington Street major system) during less frequent storm events. It is not anticipated that alterations to this area would significantly impact the Lincoln M Alexander storm sewer system due to the extensive upstream (external) contributing drainage area.

The imperviousness of the revised subcatchments has been updated to reflect existing (2021) conditions based on available aerial imagery, the City of Hamilton Road Parcel GIS layer, and the City of Hamilton Building (footprint) GIS layer. The updated existing conditions subcatchments have maintained the impervious methodology from the 2011 West Central Mountain (WCM) study and the 2020 Jamesmount Gardens SWM assessment, with 60% of the estimated imperviousness for residential land directly connected to the outlet (non-directly connected imperviousness is accounted for by increasing the pervious area Curve Number). The updates are summarized as follows (impervious calculations for the updated existing conditions are provided in **Appendix A**):

- The City's property parcel GIS layer has been applied to identify the ROW limits for Upper Wellington Street. Imperviousness for the ROW has been directly estimated for the extents of the ROW based on aerial imagery.
- A typical road ROW imperviousness for the external areas contributing to and downstream of Upper Wellington Street has been estimated based on ROW imperviousness values applied for the Jamesmount Gardens SWM assessment. Two typical 50 m (+/-) long sections of the road ROW have been used to estimate the ROW imperviousness of the roads within the updated subcatchments as 77.9% (+/-).
- The City of Hamilton Building GIS layer has been used to estimate the residential building footprint area within each of the updated subcatchments; the residential buildings within the identified subcatchments have been counted and the footprint areas have been summed and applied as the residential building impervious area.
- Typical residential property amenity areas (driveways, patios, sheds, pools, etc.) have been estimated based on measurements obtained for the Jamesmount Gardens SWM assessment. The imperviousness of amenity areas has been calculated as a ratio of the respective sample building footprint area and has been estimated to be 44.1% (+/-) amenity imperviousness for townhouses and 61.2% (+/-) amenity imperviousness for detached residential buildings.

- The aerial imagery has been compared to the City of Hamilton Building GIS layer and residential buildings on Chartwell Circle and Pinemeadow Drive (subcatchment F03), are shown on the aerial imagery, however these buildings are not shown on the Building GIS layer. The residential buildings have been included, and a unitary rate of the existing residential buildings footprint area has been used to estimate the impervious area of the unaccounted residential buildings. Similarly, the impervious amenity areas have been added to the unaccounted residential buildings.
- The imperviousness of subcatchments C17_3, C18_10, C18_26, F02_1, and F02_4 has been assumed to be 5% as these are primarily undeveloped grassed areas or parks.
- The imperviousness of subcatchments C18_4, F02_5, F04_18, and H01 have been manually measured based on aerial imagery as these subcatchments are primarily impervious areas.

3.1.3 Hydraulics

The existing conditions hydraulic elements and parameters have been updated based on the available drawings and the City's storm sewer and manhole GIS shapefiles, and are summarized as follows:

- The storm sewers and manholes that convey runoff from Pineridge Drive to Stone Church Road East have been added to the model based on the Wellington Meadows Phase One drawings (Urbex Engineering Limited, August 25, 1997), drawing number 98-S-55_6.
- The storm sewers and manholes on Upper Wellington Street, north of Stone Church Road have been added to the PCSWMM model as shown on the sanitary and storm sewer drawings for the Upper Wellington Street from Stone Church Road East to 380 m North, (The Regional Municipality of Hamilton-Wentworth Department of Engineering, August 17, 1990), drawing number 90-S-35.
- The storm sewers and manholes on Desoto Drive and Ralfana Crescent have been added to the PCSWMM model as shown on the Vittoria Gardens Development drawings (S. Llewellyn & Associates Ltd., August 2008), drawing number 08-S-32.
- Storm sewers discharging from the Oakwood Mews and Da Vinci developments have been added to the PCSWMM model based on the respective drawings, numbers LSP1711 and LSP1712.
- The number of catch basin and the types of catch basins which convey runoff to the Towercrest Drive storm sewers have been updated based on the City's catch basin GIS shapefile.
- The SWM facility and storm sewers discharging runoff from the Toscani Developments Ltd. site at 6 Sirente Drive have been added to the model based on the servicing drawing, number LSP_2739, and the rating curve for the SWM facility has been applied based on the values provided in the Stormwater Management Report for 1301 and 1315 Upper Wellington Street (S. Llewellyn and Associates Ltd., April 2008).
- The number of catch basin and the types of catch basins which convey runoff to the storm sewers on Sirente Drive and the downstream external subcatchments have been updated based on the City's catch basin GIS shapefile.
- Invert elevations for the major system conduits have been applied based on the rim elevations of the available manhole data and contour data where infrastructure data is not available.
- A surcharge depth has been applied to all PCSWMM model junctions to contain excess flow within the hydraulic system and prevent loss of flow from the model.

With respect to Upper Wellington Street specifically, major system conduits have been added between Stone Church Road East and Limeridge Road East. A standard 4 lane urban cross-section has been applied between Limeridge Road East and Towercrest Drive / Sirente Drive. A site-specific cross-section has been estimated for the rural section of Upper Wellington Street between Towercrest Drive / Sirente Drive and Stone Church Road East which has an approximate 0.5% cross slope in the ROW and a 1% overbank slope.

3.1.4 Precipitation Sensitivity Analysis

The PCSWMM model has been simulated using the City's 6-hour Chicago and 24-hour SCS 100-year design storm events (2004 IDF update), to determine which distribution generates the greatest peak flow rates within the study area. The storm events are consistent with the 2011 WCM study and the 2020 Jamesmount Gardens SWM assessment. The results for specific manhole locations are provided in **Table 3-1** and shown on the flow node location plan (ref. **Appendix B**) for both the major and minor drainage systems. A negative value indicates that the Chicago distribution generated a greater peak flow rate while a positive value indicates that the SCS distribution generated a greater peak flow rate.

As presented in **Table 3-1**, the SCS distribution generated peak flow rates are greater than the Chicago distribution peak flow rates at all the major and minor comparison locations, with the exception of the major system junction HI18B063 and minor system junctions HI18B026 and HI19B074. The SCS distribution has therefore been applied for the assessment of the Upper Wellington Street study area based on these results. Consistent with City of Hamilton standards, the 5-year design storm event has been used to assess the minor system (storm sewers) while the 100-year design storm event has been used to assess the major system (overland flow - roadways) in the subsequent analysis.

Table 3-1: Precipitation Sensitivity Analysis

LOCATION ID	JUNCTION NAME	MAJOR / MINOR SYSTEM	100 YEAR PEAK FLOW (m ³ /s)		DIFFERENCE	
			6 HR CHI	24 HR SCS	m ³ /s	%
1	HI18B063	Minor	4.51	4.55	+0.04	+0.7
		Major	5.16	4.92	-0.24	-4.7
3	HI18B026	Minor	6.59	6.57	-0.02	-0.3
		Major	3.83	3.92	+0.09	+2.5
4	HI19B074	Minor	0.82	0.82	0.00	0.0
		Major	0.74	0.75	+0.01	+1.3
5	HI19B068	Minor	2.50	2.58	+0.08	+3.4
		Major	1.29	1.36	+0.07	+5.0
6	HI20B119	Minor	26.96	27.17	+0.21	+0.8
		Major	3.07	3.23	+0.16	+5.1

3.1.5 Flow Comparison

The existing conditions Upper Wellington Street PCSWMM model has been simulated with the 5 and 100-year 24-hour SCS design storm events to validate the peak flow rates against the Jamesmount Gardens PCSWMM model. The 5-year storm event minor system results and the 100-year storm event major system are presented in **Table 3-2**; refer to Figure B1 (**Appendix B**) for location IDs.

Table 3-2: Comparison of Simulated Peak Flow Rates

LOCATION ID	JUNCTION NAME	5-YEAR STORM (24H SCS TYPE II) MINOR SYSTEM PEAK FLOW				100-YEAR STORM (24H SCS TYPE II) MAJOR SYSTEM PEAK FLOW			
		PEAK FLOW (m ³ /s)		DIFFERENCE		PEAK FLOW (m ³ /s)		DIFFERENCE	
		JG ¹	UW ²	M ³ /S	%	JG ¹	UW ²	M ³ /S	%
1	HI18B063	2.30	2.70	0.40	17.6	5.65	4.92	-0.73	-13.0
5	HI19B068	0.57	1.33	0.76	132.1	3.19	1.52	-1.66	-52.2
6	HI20B119	18.65	19.50	0.85	4.6	5.27	3.38	-1.89	-35.9
7	HI20T001	29.95	30.37	0.42	1.4	-	-	-	-

1. JG: Jamesmount Gardens (as per “Stormwater Management Report, Jamesmount Gardens Development”, Diconzo Construction Company Ltd. (Wood, May 2020)
2. UW: Upper Wellington Street (as per current report)

The results in **Table 3-2** indicate an increase in the peak flow rates within the storm sewer system during the 5-year design storm event. This can be attributed to the addition of the subcatchment refinements and the addition of the developed areas, resulting in an increase in the contributing drainage area imperviousness. The contributing drainage area to junction HI18B063 (Towercrest / Sirente) has decreased from 32.2 ha (+/-) to 28.5 ha (+/-), however the impervious area has increased from 5.6 ha (+/-) to 8.5 ha (+/-). The minor system results at junctions HI20B119 and HI20T001 demonstrate that the influence of the Upper Wellington Street refinements is less pronounced.

The peak flow rates at the Upper Wellington Street junctions, HI19B068 (Stone Church) and HI20B119 (Upper Wellington at Dragoon Drive), also indicate increases in simulated peak flow rates for the 5-year storm event, primarily because of the subcatchment refinements and increase in impervious area. The drainage area contributing to these junctions has increased from 21.2 ha (+/-) to 22.2 ha (+/-), with an increase in the impervious area from 4.7 ha (+/-) to 5.7 ha (+/-). The increases in peak flows for the 5-year event are logical given these changes, and the typical response to peakier type storm events. Conversely, these changes have resulted in a simulated decrease in the 100-year storm event. This may be attributable to the additional hydraulic elements in the updated model which may provide additional attenuation for more formative storm events. Hydrologic timing effects of different areas may also play a role.

Overall, the refined PCSWMM model is considered more discrete and representative of actual conditions along the Upper Wellington corridor than the previous coarser model version. Therefore, the results are reasonably consistent at the downstream junctions with the Jamesmount Gardens PCSWMM model and considered appropriate for this assessment.

3.1.6 Existing Conditions Performance Analysis

3.1.6.1 Minor System Performance Results

There are two primary storm sewer outlets for the study area. The Upper Wellington Street ROW from 460 m (+/-) north of Stone Church Road to Dragoon Drive is conveyed to the 2700 mm diameter storm sewer on Dragoon Drive. The Upper Wellington Street ROW from 185 m (+/-) south of Towercrest Drive / Sirente Drive to 165 m (+/-) north of Towercrest Drive / Sirente Drive is conveyed to the sag in the road at Towercrest Drive / Sirente Drive, and towards the 1350 mm diameter storm sewer on Sirente Drive. These two storm sewer systems are both conveyed easterly to a trunk sewer confluence on Upper Wentworth Street at Southpark Avenue. A third outlet has been identified at Limeridge Road for the minor portion of the Lincoln M Alexander Parkway overpass. Given the minor drainage area involved, hydraulic modelling has not been included in the PCSWMM model for this area, only hydrologic modelling has been included. Maintenance holes have been selected at key storm sewer locations on Upper Wellington Street and key Sirente Drive storm sewer confluences. These key locations are identified in **Table 3-3** (refer to Figure B1 in **Appendix B** for further details on node locations). Locations 1, 4, 5 and 8 are particularly critical as they are located within the study area limits.

Table 3-3: PCSWMM Model Junction Location Summary

LOCATION ID	MANHOLE ID	PIPE SIZE (mm)	DESCRIPTION
1	HI18B063	1350	Sirente Drive trunk storm sewer at Upper Wellington Street
2	HI18B073	1350	Sirente Drive trunk storm sewer at the confluence to Newport Crescent storm sewer
3	HI18B026	1500	Rockview Avenue trunk storm sewer at Merrilee Crescent confluence
4	HI19B074	675	Upper Wellington storm sewer at Desoto Drive confluence
5	HI19B068	975	Upper Wellington trunk storm sewer at Stone Church Road East confluence
6	HI20B119	2700	Upper Wellington trunk storm sewer at Dragoon Drive confluence
7	HI20T001	2750 x 3658	Delancey Boulevard trunk storm sewer downstream of the Brigade Drive confluence
8	OF_Limeridge	-	Major System outlet from Upper Wellington Street at Limeridge Road East

Of the eight locations identified in **Table 3-3**, one of these locations, OF_Limeridge, is not a modelled manhole in the PCSWMM model, rather a major system outlet from Upper Wellington Street to Limeridge Road East. The performance of this node has been assessed in **Section 3.1.6.2.6.2**.

The updated peak flow rates for the 5-year 24-hour SCS design storm event is presented in **Table 3-4**. A performance profile of the storm sewers is also provided in **Appendix B**; one profile is provided for the Towercrest Drive / Sirente Drive storm sewer, while a profile is provided for the Upper Wellington Street to Delancey Boulevard storm sewer. These peak flow rates and the performance will serve as a comparison to the results generated for the proposed conditions scenario.

Table 3-4: Existing Conditions Minor System 5-year SCS Storm Event Peak Flow Rate Summary

LOCATION ID	ROAD	MANHOLE ID	PIPE SIZE (mm)	DRAINAGE AREA (ha)	PEAK FLOW RATE (m ³ /s)
1	Upper Wellington Street	HI18B063	1350	28.9	2.70
2	Sirente Drive	HI18B073	1350	40.5	3.44
3	Rockview Avenue	HI18B026	1500	49.8	4.43
4	Upper Wellington Street	HI19B074	675	6.7	0.45
5	Upper Wellington Street	HI19B068	975	22.2	1.33
6	Upper Wellington Street	HI20B119	2700	302.9	19.50
7	Delancy Boulevard	HI20T001	2750 x 3658	412.8	30.37

As per **Table 3-4**, the simulated 5-year peak flow rates range from 2.70 m³/s at the intersection of Upper Wellington and Towercrest Drive / Sirente Drive to 4.43 m³/s at the Rockview Avenue confluence and 0.45 m³/s on Upper Wellington Street to 30.37 m³/s at Delancey Boulevard. The contributing drainage area to the storm sewer at Upper Wellington and Dragoon Drive is 302 ha (Location ID 6); the peak flow rate of 19.49 m³/s would be anticipated from such a large, developed area.

The Towercrest Drive / Sirente Drive storm sewer profile is presented in Figure B4 (ref. **Appendix B**). The simulated hydraulic gradeline (HGL) indicates that there is capacity in the upper end of the storm sewer system during the 5-year storm event, while the storm sewer is surcharged at manhole HI18B073, approximately 300 m east of the intersection of Upper Wellington and Towercrest Drive / Sirente Drive. The simulated surcharging corresponds to the confluence of the Newport Crescent drainage area contributing to the Sirente Drive storm sewer, resulting in a total drainage area of 41 ha (+/-) to the storm sewer at that location.

The Upper Wellington Street storm sewer profile is presented in Figure B5 (ref. **Appendix B**). The simulated HGL plot indicates that conveyance capacity is available during the 5-year storm event from the upstream end of the storm sewer system to manhole HI20B115, located 80 m (+/-) east of Upper Wellington Street on Dragoon Drive. The results indicate that the existing storm sewer sections within the study area have sufficient capacity to convey the 5-year storm event without surcharging, consistent with City of Hamilton standards.

3.1.6.2 Major System Performance Results

The updated peak flow rates for the 100-year 24-hour SCS design storm event is presented in **Table 3-5**. In addition to the results provided in the table, two figures of the roadway profile and 100-year major system HGL have been generated; refer to Figures B7 and B8 (ref. **Appendix B**).

Table 3-5: Existing Conditions Major System 100 Year SCS Storm Event Peak Flow Rate Summary

LOCATION ID	ROAD	JUNCTION ID	PEAK FLOW RATE (m ³ /s)
1	Upper Wellington Street	HI18B063	4.92
4	Upper Wellington Street	HI19B074	0.75
5	Upper Wellington Street	HI19B068	1.52
6	Upper Wellington Street	HI20B119	3.38
8	Upper Wellington Street	OF_Limeridge	0.15

The results in **Table 3-5** indicate the 100-year 24-hour SCS design storm peak flow rates range from 0.15 m³/s to 4.92 m³/s. In comparison to the results presented in Table 3-4, it is evident that the 100-year major system peak flow at ID 6 (Upper Wellington Street at Dragoon Drive) is notably less than the minor system 5-year flow. This reflects the trunk nature of the storm sewer and large capacity in this case, as well as potentially a lack of a continuous overland flow route from upstream areas.

The peak flow rate conveyed to the sag at the intersection of Upper Wellington Street and Towercrest Drive / Sirente Drive is the greatest of the reported peak flow rates at 4.92 m³/s. This simulated flow results in a ponding depth of 0.41 m (+/-) at the sag in the ROW as presented in Figure B7 (ref. **Appendix B**), which would exceed City of Hamilton standards (0 mm above the roadway crown for arterial roadways during the 100-year event). A major system outlet for the sag is not evident based on the available topographic information. Ponding up to 0.84 m is indicated until spill easterly would occur (from the high point along Sirente Drive, just east of Newport Crescent).

The contributing peak flow rate to Limeridge Road from Upper Wellington Street is 0.15 m³/s and as shown in Figure B7, results in a flow depth of 0.05 m (+/-) within the ROW.

The major system outlet for the southern portion of Upper Wellington Street (i.e. from the high point 280 m south of Towercrest / Sirente), is to the intersection of Upper Wellington Street and Bridage Drive, where the major system flow is conveyed easterly. The peak flow rate contributing to HI19B068 at the intersection of Stone Church Road East and Upper Wellington Street is 1.52 m³/s, resulting in a simulated 100-year flow depth within the ROW of 0.14 m relative to ditch bottom elevation (ref. Figure B8 in **Appendix B**). This would mean an approximate depth of 0.06 m relative to the road crown, given typical lane widths and crossfalls (i.e. 4 m lane at 2% crossfall = 0.08 m), which would slightly exceed City standards (i.e. 0 mm at the crown for the 100-year storm event for arterials). Further upstream (north) on Upper Wellington Street, these criteria would be achieved, however the higher simulated depth at the intersection of Upper Wellington and Stone Church Road reflect the impact of additional contributing flows from Stone Church Road to the west.

3.2 Modelling Updates (2025)

3.2.1 Additional Updates

Minor refinements were completed to the existing conditions PCSWMM model in 2025 to reflect improved data availability and to address model QA/QC observations. These updates included the following:

- Updated Subcatchment Imperviousness: The imperviousness of the road right-of-way (ROW) subcatchments was re-evaluated based on the most current aerial imagery available via Google™ and ESRI™ satellite views.

This refinement was applied to more accurately reflect existing surface conditions within the ROW, particularly along Upper Wellington Street and adjacent contributing roadways.

- Subcatchment Boundary Revisions: A small number of subcatchment boundaries representing road ROW areas were revised as part of routine QA/QC. These boundary adjustments led to minor updates to the associated subcatchment areas and calculated length and width parameters. These changes did not result in any material modifications to drainage direction or hydrologic routing.

All other model parameters from the 2021 existing conditions model have been retained, as the updates described above were limited in scope and did not necessitate recalibration or adjustment of additional model components. A summary of the updated subcatchment parameters is provided in **Appendix C** (2025 Modelling Updates) for reference.

3.2.2 Peak Flow Results

The refined 2025 PCSWMM model was simulated using the same SCS 24-hour design storm distribution applied in the 2021 assessment, including the 2-year, 5-year, 10-year, 25-year, 50-year, and 100-year storm events. Peak flow rates at key junctions were extracted and compared against those previously simulated in the 2021 model to assess the effects of the updated subcatchment parameters.

The results are summarized in **Table 3-6**, which presents minor system simulated peak flows for the key storm events at representative junctions within the study area.

Table 3-6: Existing Conditions Minor System 5 Year SCS Storm Event Peak Flow Rate comparison

LOCATION ID	ROAD	MANHOLE ID	5-YEAR PEAK FLOW RATE - 2021 (m ³ /s)	5-YEAR PEAK FLOW RATE - 2025 (m ³ /s)	% CHANGE
1	Upper Wellington Street	HI18B063	2.70	2.71	0.4%
2	Sirente Drive	HI18B073	3.44	3.45	0.3%
3	Rockview Avenue	HI18B026	4.43	4.44	0.2%
4	Upper Wellington Street	HI19B074	0.45	0.45	0.0%
5	Upper Wellington Street	HI19B068	1.33	1.32	-0.8%
6	Upper Wellington Street	HI20B119	19.50	19.50	0.0%
7	Delancy Boulevard	HI20T001	30.37	30.37	0.0%

The results summarized in **Table 3-7**, presents major system simulated peak flows for the key storm events at representative junctions within the study area.

Table 3-7: Existing Conditions Major System 100 Year SCS Storm Event Peak Flow Rate comparison

LOCATION ID	ROAD	JUNCTION ID	100-YEAR PEAK FLOW RATE - 2021 (m ³ /s)	100-YEAR PEAK FLOW RATE - 2025 (m ³ /s)	% CHANGE
1	Upper Wellington Street	HI18B063	4.92	4.89	- 0.61%
4	Upper Wellington Street	HI19B074	0.75	0.752	0.3%
5	Upper Wellington Street	HI19B068	1.52	1.52	0.0%
6	Upper Wellington Street	HI20B119	3.38	3.38	0.0%
8	Upper Wellington Street	OF_Limeridge	0.15	0.14	- 6.7%

As shown, peak flow values resulting from the 2025 model refinements are generally consistent with the 2021 results. Minor differences in the simulated peak flows can be attributed to changes in subcatchment imperviousness and geometry, but the overall system response remains aligned with prior assessments.

The updated (2025) existing conditions peak flows have been applied as the baseline values for the assessment of SWM quantity controls under proposed conditions, as described further in Section 4.

3.3 Stormwater Quality Control

Under existing conditions, there are no stormwater management facilities evident within the limits of the study area. Similarly, a review of the available drawings and GIS storm sewer manhole data has indicated that there are no oil-grit-separators shown within the study area.

The subject drainage area eventually outlets to Red Hill Creek and thus the existing Mountain Brow SWM facility (currently being reconstructed) that was intended to provide Level 2 (70% treatment of total suspended solids) for the contributing drainage area (S. Llewellyn and Associates Ltd., April 2008). Notwithstanding, as per comments from the Hamilton Conservation Authority (HCA; as discussed in subsequent sections) it is not considered appropriate to utilize the Mountain Brow SWM facility for the provision of quality control for the currently proposed road works.

4 FUTURE CONDITIONS

4.1 Overview of Proposed Changes

The preferred road design for the study area was identified through the Class EA process. The preferred design for Upper Wellington Street from Stone Church Road to Towercrest Drive / Sirente Drive proposes widening from two-lane cross-section to a three-lane cross-section, incorporating 3.3-meter vehicular through-lanes, a 3-meter center-turn lane. It also proposes 1.5-meter off-street bike lanes and 1.8-meter sidewalks on both sides, separated by a 0.6-meter buffer and physically separated from vehicular traffic lanes by 3-meter planting strips/boulevards.

The preferred design for Upper Wellington Street from Towercrest Drive / Sirente Drive to Limeridge Road proposes road diet, to reduce the existing four-lane cross-section to a two-lane cross-section with 3.3-meter vehicular through-lanes and dedicated turning lanes at intersections. The reclaimed space accommodates 1.8 metre bike lanes, 1.5 metre buffer for streetscaping elements, and maintaining existing 1.6-metre sidewalks with 0.6-metre buffer. Over the Lincoln M. Alexander Parkway overpass, the design transitions to leverage the existing 2.0-meter bridge sidewalks and repurpose the existing roadway platform for 1.8-meter bike lanes, physically separated from vehicular traffic by medians, reducing impacts to the existing bridge and structure.

The primary existing storm sewer system along Upper Wellington Street conveys runoff from 130 m (+/-) north of Desoto Drive southerly to Dragoon Drive, 160 m (+/-) south of Stone Church Road. A separate existing storm sewer system conveys runoff easterly on Sirente Drive, which crosses Upper Wellington Street.

4.2 Modelling Updates

4.2.1 Hydrology

To identify future conditions for roadway right-of-way (ROW) limits, a GIS layer was created based on proposed ROW drawings for changes in land cover, specifically impervious surfaces. The PCSWMM model subcatchments for the ROW have been updated as per the GIS layer to reflect future conditions and changes in land cover (drainage boundaries remain consistent with those from existing conditions).

The imperviousness of the revised subcatchments has been updated to reflect future conditions based on available aerial imagery without any changes in the subcatchment boundaries and drainage areas. The updated imperviousness values for future conditions are provided in **Appendix D**.

The subcatchment SCS Curve Numbers for the future conditions ROW subcatchments were increased accordingly using an area weighting approach. With a base Curve Number of 74 to represent open spaces in good condition for type “C” soils and Curve Number 79 that represents fair conditions for the grass cover. As a result, curve numbers used for the future conditions model have been maintained close to the revised existing conditions values since there is little difference between the pervious landcover and soil type under both conditions.

Existing and future conditions ROW imperviousness details contributing to drainage outlets are presented in **Table 4-1**.

Table 4-1: Existing and Future Conditions ROW Impervious Area

DRAINAGE OUTLET NODE ID	DRAINAGE LOCATION	EXISTING ROW IMPERVIOUS AREA (ha)	PROPOSED ROW IMPERVIOUS AREA (ha)	ADDITIONAL ROW IMPERVIOUS AREA (ha)
HI19B070 & HI19B070-S	Intersection of Upper Wellington Street & Stone Church Road East	0.70	0.93	+0.23
HI18B063 HI18B063-S	Intersection of Upper Wellington Street & Sirente Drive	0.53	0.66	+0.13
OF_Limeridge	Intersection of Upper Wellington Street & Limeridge Road East	0.36	0.36	0

4.2.2 Hydraulics

For future conditions, the existing conditions hydraulic elements and parameters were used to assess the model. No information was available at this point with respect to proposed roadway profile, therefore no proposed storm sewer networks were added to the future conditions model. The vertical profile was not developed as part of the Class EA Study due to the lack of detailed topographic survey. Numerous driveway and road connections create constraints that limit the opportunity to revise the profile without major development works. The vertical road profile for the Study Corridor will be developed during detailed design phase and it is assumed it will generally follow the existing ground profile to minimize grading and property impacts, and to match into the existing road network. Storms sewers have therefore not been assessed or designed as part of the current study but should be completed as part of the future detailed design phase.

For the SWM assessment (quantity and quality control), which is the focus of the current summary, flows were compared with existing conditions at locations where flows from project site contribute to existing storm water network. Comparison between existing and future conditions peak flows are presented in **Table 4-2**. Percentage difference between existing and future conditions peak flows are presented in **Table 4-3**.

Table 4-2: Future Conditions Uncontrolled Peak Flows Key Locations for SCS 24-hour Rainfall Distribution (m³/s)

DRAINAGE NODE ID	DRAINAGE LOCATION	TOTAL DRAINAGE AREA (ha)	ROW DRAINAGE AREA (ha)	EXTERNAL DRAINAGE AREA (ha)	DESIGN STORM EVENT PEAK FLOWS (m³/s)					
					2-YEAR	5-YEAR	10-YEAR	25-YEAR	50-YEAR	100-YEAR
HI19B070 & HI19B070-S	Intersection of Upper Wellington Street & Stone Church Road East	9.10	1.33	7.77	0.68	1.08	1.33	1.79	2.04	2.24
HI18B063 & HI18B063-S	Intersection of Upper Wellington Street & Sirente Drive	27.98	0.90	27.08	2.23	4.01	5.32	7.28	8.48	9.44
OF_Limeridge	Intersection of Upper Wellington Street & Lime Ridge Road East	0.41	0.41	0.00	0.06	0.08	0.10	0.12	0.13	0.14

Table 4-3: Future Conditions Uncontrolled vs Existing Conditions Peak Flows Comparison Percent Difference under SCS 24-hour Rainfall Distribution (%)

DRAINAGE NODE ID	DRAINAGE LOCATION	TOTAL DRAINAGE AREA (ha)	ROW DRAINAGE AREA (ha)	EXTERNAL DRAINAGE AREA (ha)	DESIGN STORM EVENT PEAK FLOWS (%)					
					2-YEAR	5-YEAR	10-YEAR	25-YEAR	50-YEAR	100-YEAR
HI19B070 & HI19B070-S	Intersection of Upper Wellington & Stone Church Road East	9.10	1.328	7.77	4.9%	3.5%	3.8%	3.1%	2.5%	2.4%
HI18B063 & HI18B063-S	Intersection of Upper Wellington & Sirente Drive	27.98	0.899	27.08	1.8%	1.4%	1.1%	1.4%	0.4%	0.3%
OF_Limeridge	Intersection of Upper Wellington & Lime Ridge Road East	0.41	0.405	0.00	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%

5 STORMWATER MANAGEMENT OPPORTUNITIES

5.1 General Stormwater Management Opportunities

Stormwater Management practices (SWMPs) for the management of roadway runoff generally fall into two categories: those that address stormwater quantity (including erosion) and those that manage the stormwater quality of surface runoff. In addition, Low Impact Development (LID) best management practices (BMPs) are designed to provide water quality treatment and quantity control, including erosion control, for smaller, more frequent storm events (i.e., typically up to and including the 25 mm storm event).

Stormwater quantity management issues relate to the proper sizing of minor (sewer) and major (overland flow) conveyance systems for roadway runoff. In addition, stormwater quantity management strategies can include the need for facilities to address downstream flow constraints and erosion potential from alterations of the roadway right-of-way.

Based on the proposed changes to the Upper Wellington Street right-of-way (ROW) contributing to the City's storm sewer network and overland flows, and the results presented in the previous section, quantity controls are required to reduce or maintain peak flows under proposed conditions to existing conditions values.

In terms of stormwater quality, the SWMPs relate to the treatment of new pavement. Typically, the treatment level is related to the standards defined in a watershed or subwatershed planning study, which are dependent on the quality and sensitivity of the receiving stream system. Water quality standards are defined in the Ministry of the Environment's 2003 SWM Planning and Design Manual and refer to the average annual degree of Total Suspended Solids (TSS) removal. The subject site ultimately drains to Red Hill Creek, which in turn drains into Hamilton Harbour. As per Table 6.3 in the City's "Comprehensive Development Guidelines and Financial Policies Manual" (2019), this watershed requires "Enhanced" treatment, or 80% TSS removal. This is also consistent with comments received from the Hamilton Conservation Authority (HCA) on February 19, 2021. HCA also noted that site specific quality controls be provided, given that the downstream Mountain Brow SWM facility is not expected to have sufficient capacity to accommodate the proposed development. Thus, enhanced (80%) treatment has been applied as the water quality target for the site area.

In terms of erosion/volume control, a review will be undertaken to confirm the feasibility of retaining the 25 mm storm event from the additional impervious area. This will however depend on soil characteristics, groundwater and bedrock elevation as well as other factors.

5.2 Alternative Stormwater Management Practices

5.2.1 Quantity Management

Quantity control impacts (in this case increased runoff from additional proposed hard (impervious) surfaces including roadway, sidewalks, and intersection improvements) can typically be mitigated by on-site storage and infiltration techniques.

For the current project, quantity controls are required for the proposed ROW sections to address predicted peak flows increases compared to existing conditions, therefore temporary storage and infiltration-based stormwater management measures as per the following have been considered.

5.2.2 Quality Management

There are numerous stormwater management practices which can be used to treat contaminated stormwater runoff from roadway surfaces. These include the following:

- Wet ponds / wetlands / hybrids (generally linear facilities)
- Enhanced grass swales
- Filter strips
- Oil and grit separators
- Off-site stormwater management facilities (existing, retrofitted and/or proposed)
- Catch basin shields
- LID BMPs – (Bioretention systems, permeable pavement and other infiltration systems)

The respective characteristics, advantages and disadvantages of the foregoing have been well documented in existing Municipal and Provincial literature and hence this information has not been repeated within this document. Some brief advantages and disadvantages, though, are discussed in the following section.

5.3 General Assessment

5.3.1 Quantity Control

The following SWM quantity controls have been assessed for feasibility and appropriateness for the current project.

1 Wet Ponds / Wetlands / Hybrids

Constructing a new wet pond, wetland or hybrid pond is not feasible for this study area based on space constraints are costly to implement and maintain. As such this alternative has not been considered further.

2 Super Pipe Storage

Super pipe storage would require either upgrading existing storm sewers to a larger storm sewer or sizing a new sewer capable of storing additional runoff to meet erosion control targets. Super pipe storage is one of the costliest methods of providing underground storage. Notwithstanding, it can be simpler for municipalities to maintain and implement given concerns with sub-surface storage systems which have larger footprints and tend to be shallower. It is understood that the City generally prefers super pipes to sub-surface storage systems. As such, this alternative has been advanced for further consideration.

3 Conventional Underground Storage (Concrete Tanks)

Conventional underground storage for the project site would require multiple storage tanks (concrete). The storage tanks would be connected to the downstream end of the proposed storm sewers to maximize the contributing drainage area to the storage elements. Underground concrete tanks are considered costly to implement and may also be challenging to implement in a roadway ROW given utilities and space constraints. In addition, conventional underground tanks do not filter, or infiltrate captured runoff. As such conventional underground storage (concrete) tanks have been screened for further consideration.

4 Conventional Underground Storage (Cellular Systems)

Notwithstanding the preceding, more cost-effective underground storage systems could be considered to achieve erosion control requirements. This includes cellular type tank systems such as StormconTM, BrentwoodTM, CultecTM, or TritonTM systems. These systems warrant further consideration.

5 Low Impact Development Best Management Practices (LID BMPs)

LID BMPs can address erosion control requirements by retaining, infiltrating and filtering stormwater runoff for more frequent storm events, but may not provide sufficient control for larger more formative events. These options have been discussed further in the subsequent section with respect to quality control, however, may also be a feasible alternative for water budget\balance and erosion control of smaller storm events.

5.3.2 Quality Control

1 Wet Ponds / Wetlands / Hybrids

Further to the assessment in the previous section with respect to quantity control, wet ponds and related features are not considered feasible for the current study area. An off-site SWM facility does existing (Mountain Brow SWMF)

however as per previous discussion and comments from the HCA, it is not considered appropriate to provide quality control for the subject works.

2 Oil and Grit Separators (OGS)

These end-of-pipe systems tend to service smaller drainage areas (2 ha +/-) and provide varying levels of stormwater quality treatment depending on the model selected. OGS units are typically encouraged as part of a “treatment train” approach; many municipalities and regulators will not credit the full TSS removal function of OGS units accordingly (i.e., typical maximum credit of 50% to 70% TSS removal). Disadvantages include the need for frequent maintenance, as well as relatively high capital costs and the ability to service smaller drainage areas. As a pre-treatment approach for other stormwater quality measures, or for providing water quality treatment for pavement areas greater than the proposed additional paved areas, OGS have been carried forward for further consideration.

3 CB Shields™

CB Shields™ are a proprietary product that involves a catch basin insert to shield accumulated sediment in the catch basin sump from resuspension and washout. The CB shields can increase TSS capture by up to 50% as shown in Environmental Technology Verification (ETV) testing. The application of CB shields is not to be applied as a stand-alone treatment approach, however, can be combined with other treatment technologies to mitigate water quality. Implementation costs would be comparatively low to other forms of water quality treatment and frequent maintenance would be required to remove accumulated sediment from the catch basin sump to ensure acceptable long-term performance. These measures have been carried forward for further consideration.

4 Low Impact Development Best Management Practices (LID BMPs)

LID represents the application of a suite of BMPs normally related to source and conveyance stormwater management controls to promote infiltration and pollutant removal on a local site-by-site basis. These measures rely on eliminating the direct connection between impervious surfaces such as roads and the storm drainage system, as well as the promotion of infiltration of road drainage. General design guidelines and considerations for source and conveyance controls have been advanced since the early 1990’s as part of the MMAH “Making Choices” and in 1994 as part of the Ministry of the Environment’s original Best Management Practices Guidelines.

Subsequent to the 1994 MOE Guidelines, technologies and standards have been developed further for the application of source and conveyance controls. These have evolved into a class of Best Management Practices (BMPs) referred to as Low Impact Development (LID) practices, which have advanced as an integrated form of site planning and storm servicing to maintain water balance and providing stormwater quality control for urban developments. Initial results from studies in other settings have demonstrated that LID practices provide benefits by way of reducing the erosion potential within receiving watercourses and thereby reducing the total volume of end-of-pipe stormwater erosion control requirements. In addition, due to volumetric controls afforded by LID BMP’s, water quality is also improved through a reduction in mass loading. The benefits from LID stormwater management practices are generally focused on the more frequent storm events (e.g., 2-year storm) of lower volumes as opposed to the less frequent storm events (e.g., 100-year storm) with higher volumes. It is also recognized that the forms of LID practices which promote infiltration or filtration through a granular medium provide thermal mitigation for storm runoff.

Guidelines regarding the application of LID practices and techniques have been developed within various jurisdictions in the United States and Canada. The Toronto and Region Conservation Authority (TRCA) and Credit Valley Conservation (CVC) have produced the 2010 Low Impact Development Stormwater Management Manual,

for the design and application of LID measures, which is used by southern Ontario conservation authorities). Various LID techniques, as well as their function that are applicable to road projects, are summarized in **Table 5-1**.

Table 5-1: LID Source and Conveyance Controls

TECHNIQUE	FUNCTION
Enhanced Grassed Swales	Enhanced Grassed Swales are vegetated open channel designed to convey, treat and attenuate runoff. They may include check dams or other measures to slow flow and promote infiltration.
Filter Strips	Typically applied as part of a treatment train, filter strips are designed to filter sheet flow prior to discharge to a surface drainage receiver.
Bio-retention Cell	Vegetated technique for filtration of storm runoff. Stormwater quality control is provided through filtration of runoff through soil medium and vegetation. Infiltration / evapotranspiration / water balance maintenance and additional erosion control may be achieved if no subdrain provided.
Infiltration Trenches	Infiltration technique to provide stormwater quality control and maintain water balance. Erosion controls may be achieved depending upon soil conditions.
Tree Planter\Soil Retention Cells	Cellular soil storage systems are typically modular cells that allow for less compact soil and are used to capture runoff from paved areas or roadways and then either infiltrate or use the runoff for irrigation of trees planted within the cell.
Permeable Pavers / Pavement	Infiltration technique to reduce surface runoff volume. Benefits to stormwater quality and erosion control can be determined based on the runoff volume captured and infiltrated through an infiltration media beneath the permeable pavers/ pavement. Permeable pavement can be in the form of porous concrete or asphalt and permeable granular with a binding agent.
Exfiltration Pipes	Technique to reduce storm runoff through the implementation of perforated pipes as part of the storm sewer system (typically a separate lower perforated pipe, with the conventional storm sewer as the “overflow”). Commonly referred to as Etobicoke Exfiltration system. Promotion of infiltration maintains water balance and provides stormwater quality and erosion control benefits.

– Enhanced Grassed Swales

Grassed swales designed with a trapezoidal geometry and flat longitudinal profiles with largely un-maintained turf can provide excellent filtration and treatment for storm runoff from roadways. It is generally conceded that treatment levels are at a minimum, Normal (formerly Level 2) 70% TSS removal water quality treatment and combined with other practices can provide Enhanced (Level 1) 80% TSS removal stormwater quality treatment. Their application in linear corridors is also particularly appropriate and can be further enhanced through the introduction of check dams to provide additional on-line storage. Their application in urbanized roadway cross-sections (i.e., curb and gutter) often requires alternative grading and roadway configurations which can compromise the function of the roadway itself and are therefore typically not preferred in those cases. Notwithstanding, gutter outlets along outside lanes have been demonstrated to function effectively where the right-of-way can accommodate the design. Based on the proposed ultimate urbanized road ROW and spatial constraints, enhanced grassed swales are not considered feasible and have been screened.

– Filter Strips

Filter strips are typically designed for small drainage areas (less than 2 ha +/-) and are applied as part of a treatment train. Filter strips require flat areas with slopes ranging from 1 to 5% and are usually in the range of 10 to 20 m in

length in the direction of flow. Flow leaving filter strips should be a maximum of 0.10 m depth, based on a 10 mm storm event. Based on the limited space within the Upper Wellington Street ROW, filter strips are not considered a practical stormwater quality solution and have been screened for further consideration.

– **Bioretention Systems**

Bioretention systems provide effective removal of pollutants by sedimentation, filtering, soil absorption, microbial processes and plant uptake. Bioretention systems should be approximately 10 to 20% in size of the contributing drainage area, with typical drainage areas of 0.50 ha and a maximum drainage area of 0.8 ha. Slopes within bioretention systems are typically 1% to 5%. Bioretention systems are preferred in areas that have reasonable infiltration properties (15 mm/ hr, 1×10^{-6} cm/s), but can be implemented in all soil types as long as the water quality event can be temporarily stored (typical depths 0.15 m to 0.25 m) before infiltrating and an underdrain is provided.

Bioretention systems are not considered feasible in this case given the form of the proposed roadway section. Bioretention systems have been screened from further consideration.

– **Infiltrative Trenches**

Infiltrative Trenches could be implemented as they are similar to bioretention systems but would be located sub-surface, and likely within boulevard areas (landscaping, bike lanes, sidewalks, etcetera) notwithstanding any potential utility conflicts. Any connected catchbasins would need to be fitted with goss traps to filter floatable debris. The infiltration trench could be designed to capture the 25 mm storm event with no discharge by setting the overflow to the storm sewer system above the 25 mm storm event capture storage depth.

The infiltrative cell is sized based on the target for infiltration, with consideration to bedrock and groundwater elevations and the amount of runoff capture provided by the catch basin. The cell can be a stone system, with a 40% void ratio or it can be an open cellular system that provided close to 100% volume for retention. Pre-treatment of runoff can be provided by CB Shields or an equivalent measure and goss traps. Infiltrative Trenches have been carried forward for further consideration.

– **Tree Planter\Soil Retention Cells**

Tree Planter\Soil Retention Cells (such as Silva CellsTM) are typically modular suspended paved systems with a cellular soil storage system providing structural support and allows for overland road and pavement drainage to be captured and infiltrated within the cellular soil storage system. Trees are planted within the cellular soil storage system which also use the collected drainage and provide evapotranspiration. These cells can be used in confined spaces within urban environments and provide additional stormwater quality benefits. Typically, these measures would not be considered to be a standalone water quality measure, and similar to infiltrative trenches require some form of pre-treatment such as catchbasin inserts. Tree Planter\Soil Retention Cells have been carried forward for further consideration.

– **Permeable Pavers / Pavement**

Permeable pavement could be considered for portions of the sidewalk and separated bike lanes (permeable pavement is typically not supported for active roadways). As a standalone LID BMP, a permeable paved bike lane\sidewalk would not meet either stormwater quality and/or erosion control targets as it would treat a limited area and would not treat the roadway itself (which would be expected to generate the highest contaminant loadings). However, permeable surfaces would reduce the runoff volume from paved surfaces within the urban road ROW. This LID BMP would have to be selected by the City to complement other SWM measures during the detailed design stage

for road sections that would incur increased roadway pavement area in addition to the potential application to the bike lanes/sidewalks. Based on the preceding, this alternative has been screened from further consideration.

– **Exfiltration Pipes**

Exfiltration pipes could be used in combination with other LID measures; pre-treatment (including catchbasin measures) is recommended to limit issues of debris and clogging. As a standalone SWM measure, exfiltration pipes can be a cost-effective and relatively simple method to accomplish erosion control and infiltration requirements, while eliminating the need for surface space within the right-of-way. Exfiltration pipes have been carried forward for further consideration.

Based on the foregoing, the following LID BMPs have been short listed for further consideration:

- Infiltration Trenches
- Tree Planter/Soil Retention Cells
- Exfiltration Pipes (used with infiltration trenches)

The following figures illustrate typical examples of the recommended LID BMP source controls:



Figure 5-1: Infiltration Trench Construction (Low Impact Development Stormwater Management Planning and Design Guide, Version 1.0, CVC and TRCA, 2010)



Figure 5-2: Tree Planter/Soil Retention Cell Construction (www.smartcitiesdive.com, 2020)



Figure 5-3: Soil Retention/Tree Planter Cell Cross Section (info.cambrianrisevt.com, 2020)

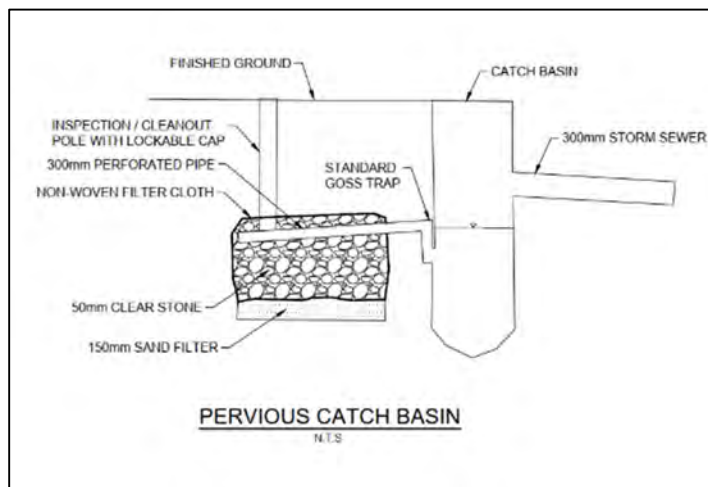


Figure 5-4: Pervious Catch Basin Cross Section

5.3.3 *Summary of Short-Listed Measures*

Based on the preceding evaluation, the following measures have been short-listed:

- Super pipes
- Conventional Underground Storage (Cellular Systems)
- Oil/Grit Separators
- CB Shields™
- Low Impact Development Best Management Practices (LID BMPs)
 - Infiltration Trenches
 - Tree Planter\Soil Retention Cells
 - Exfiltration Pipes

6 SHORT-LISTED DRAINAGE SYSTEM AND STORMWATER MANAGEMENT ALTERNATIVES ASSESSMENT

6.1 Design and Assessment Approach

The vertical profile for the roadway was not developed as part of the Class EA Study due to the lack of detailed topographic survey. The vertical road profile for the Study Corridor will be developed during the subsequent detailed design phase and it is expected it will generally follow the existing ground profile to minimize grading and property impacts, and to match into the existing road network. Storm sewers and collection/conveyance system details will need to be designed as part of this effort and have not been assessed as part of the current study, which has focused on overall stormwater management (SWM) measures only.

Accordingly, the proposed road drainage requirements will need to be assessed during the detailed design phase once road profile and grades are finalized. The drainage system assessment shall include the following:

- New storm sewers shall be designed and implemented to provide conveyance of minor system flows as per the City of Hamilton design criteria.
 - Since the majority of the existing conditions road right-of-way has a rural section (ditching) and hence no existing storm sewer network for the most part, the proposed storm sewer drainage system would supersede these features.
 - The major (overland flow) system should be re-assessed as well to ensure City of Hamilton design criteria are met including conveyance depths, particularly at roadway sag points.
-

6.2 Quantity Controls and Flow Conveyance

6.2.1 Modelling Assessment

The proposed stormwater management requirements have been assessed using the analytical and modelling tools described in the previous sections.

Quantity control storage is required to mitigate potential downstream impacts due to potential increases in the peak flows (due to increased roadway imperviousness) under various storm events. As per the previous screening and review, it is expected that storage would need to be provided within underground storage tanks, which are designed to store and discharge in such manner that the post-development peak flows are controlled to pre-development values for all return periods (2 through 100-year storms).

To determine the required quantity controls, the SCS 24-hour design storm distributions have been applied, as it is typically expected that longer duration storms will yield the greatest required storage volumes.

Model simulation results are presented in the subsequent summary tables. Table 6-1 and Table 6-2 indicate the simulated peak flows for existing conditions and future conditions for the SCS 24-hour rainfall distribution since

that is the governing distribution for quantity controls. Table 6-3 indicates the percentage difference between uncontrolled future conditions compared to the existing conditions simulated peak flows. As evident from the results, without any quantity controls in place, peak flows would be expected to increase, which is not supportable.

Quantity controls are therefore required. For the purposes of the current study, preliminary storage systems have been proposed at:

- Upper Wellington at south side of Sirente Road
- Upper Wellington north of Desoto Drive

In the case of Upper Wellington at Desoto Drive, it was generally considered preferable to locate the storage at the upstream end of the system, to provide overall quantity control for the section of roadway draining to Stone Church Drive. Implementing the storage in this location avoids the requirement to replace or upgrade existing larger diameter storm sewers at the downstream limits, as discussed further in subsequent sections. The comparison of target flows have been compared at the downstream limit of the section however, to ensure that the controls function as intended.

Hydrologic modelling results with theoretical storage systems in place are presented in Table 6-4; a comparison to existing conditions results is presented in Table 6-5. As evident, the proposed storage systems would control peak flows under proposed conditions to existing conditions values for all storm events.

The details of the proposed storage systems are presented in Table 6-6 to 6-8 respectively, including contributing drainage area, storage volumes, and storage-discharge characteristics.

Table 6-1: Existing Conditions Peak Flows at Key Locations for SCS 24-hour Rainfall Distribution (m³/s)

DRAINAGE OUTLET	DRAINAGE LOCATION	TOTAL DRAINAGE AREA (ha)	ROW DRAINAGE AREA (ha)	EXTERNAL DRAINAGE AREA (ha)	EXISTING CONDITIONS DESIGN STORM EVENT PEAK FLOWS (m³/s)					
					2-YEAR	5-YEAR	10-YEAR	25-YEAR	50-YEAR	100-YEAR
HI19B070 & HI19B070-S	Intersection of Upper Wellington Street & Stone Church Road East	9.10	1.33	7.77	0.71	1.12	1.41	1.80	1.90	2.07
HI18B063 & HI18B063-S	Intersection of Upper Wellington Street & Sirente Drive	27.98	0.90	27.08	2.19	3.95	5.26	7.18	8.45	9.41
OF_Limeridge	Intersection of Upper Wellington & Lime Ridge Road East	0.41	0.41	0.00	0.06	0.08	0.10	0.12	0.13	0.14

Table 6-2: Future Conditions Uncontrolled Peak Flows at Key Locations for SCS 24-hour Rainfall Distribution (m³/s)

DRAINAGE OUTLET	DRAINAGE LOCATION	TOTAL DRAINAGE AREA (ha)	ROW DRAINAGE AREA (ha)	EXTERNAL DRAINAGE AREA (ha)	FUTURE CONDITIONS UNCONTROLLED DESIGN STORM EVENT PEAK FLOWS (m³/s)					
					2-YEAR	5-YEAR	10-YEAR	25-YEAR	50-YEAR	100-YEAR
HI19B070 & HI19B070-S	Intersection of Upper Wellington Street & Stone Church Road East	9.10	1.33	7.77	0.75	1.17	1.50	1.84	1.94	2.11
HI18B063 & HI18B063-S	Intersection of Upper Wellington Street & Sirente Drive	27.98	0.90	27.08	2.23	4.01	5.32	7.28	8.48	9.44
OF_Limeridge	Intersection of Upper Wellington Street & Limeridge Road East	0.41	0.41	0.00	0.06	0.08	0.10	0.12	0.13	0.14

Table 6-3: Future Conditions Uncontrolled vs Existing Conditions Peak Flows Comparison Percent Difference under SCS 24-hour Rainfall Distribution

DRAINAGE OUTLET	DRAINAGE LOCATION	TOTAL DRAINAGE AREA (ha)	ROW DRAINAGE AREA (ha)	EXTERNAL DRAINAGE AREA (ha)	DESIGN STORM EVENT PEAK FLOW (m ³ /s) PERCENT DIFFERENCE (%)					
					2-YEAR	5-YEAR	10-YEAR	25-YEAR	50-YEAR	100-YEAR
HI19B070 & HI19B070-S	Intersection of Upper Wellington Street & Stone Church Road East	9.10	1.33	7.77	4.7%	3.8%	3.0%	2.0%	2.1%	2.0%
HI18B063 & HI18B063-S	Intersection of Upper Wellington Street & Sirente Drive	27.98	0.90	27.08	1.8%	1.4%	1.1%	1.4%	0.4%	0.3%
OF_Limeridge	Intersection of Upper Wellington Street & Limeridge Road East	0.41	0.41	0.00	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%

Table 6-4: Future Conditions Peak Flows with Stormwater Management under SCS 24-hour Rainfall Distribution (m³/s)

DRAINAGE OUTLET	DRAINAGE LOCATION	TOTAL DRAINAGE AREA (ha)	ROW DRAINAGE AREA (ha)	EXTERNAL DRAINAGE AREA (ha)	DESIGN STORM EVENT PEAK FLOW (m ³ /s)					
					2-YEAR	5-YEAR	10-YEAR	25-YEAR	50-YEAR	100-YEAR
HI19B070 & HI19B070-S	Intersection of Upper Wellington Street & Stone Church Road East	9.10	1.33	7.77	0.65	1.03	1.38	1.82	1.91	2.07
HI18B063 & HI18B063-S	Intersection of Upper Wellington Street & Sirente Drive	27.98	0.90	27.08	2.17	3.91	5.24	7.14	8.44	9.37

Table 6-5: Future Conditions with Stormwater Management vs Existing Conditions Peak Flows Comparison Percent Difference for SCS 24-hour Rainfall Distribution

DRAINAGE OUTLET	DRAINAGE OUTLET LOCATION	TOTAL DRAINAGE AREA (ha)	ROW DRAINAGE AREA (ha)	EXTERNAL DRAINAGE AREA (ha)	DESIGN STORM EVENT PEAK FLOW (m ³ /s) PERCENT DIFFERENCE (%)					
					2-YEAR	5-YEAR	10-YEAR	25-YEAR	50-YEAR	100-YEAR
HI19B070 & HI19B070-S	Intersection of Upper Wellington Street & Stone Church Road East	9.10	1.33	7.77	-5.8%	-9.1%	-2.7%	1.2%	1.5%	0%
HI18B063 & HI18B063-S	Intersection of Upper Wellington Street & Sirente Drive	27.98	0.90	27.08	-0.9%	-1.3%	-0.4%	-0.6%	-0.1%	-0.3%

Table 6-7: Summary of Recommended Stormwater Quantity Management Sizing for Future conditions – Sirente Drive

TANK 1		
Drainage Outlet	HI18B063 & HI18B063-S	
Drainage Outlet Location	Upstream of Intersection of Upper Wellington Street & Sirente Drive	
Total Drainage Area (ha)	27.98	
ROW Drainage Area (ha)	0.90	
External Drainage Area (ha)	27.08	
Stage	Storage (m³)	Discharge (m³/s)
2-Year	57	0.059
5-Year	59	0.112
10-Year	64	0.149
25-Year	67	0.202
50-Year	90	0.242
100-Year	95	0.283

Table 6-8: Summary of Recommended Stormwater Quantity Management Sizing for Future conditions – Desoto Drive

TANK 2		
Drainage Outlet	HI19B070 & HI19B070-S	
Drainage Outlet Location	Upstream of Intersection of Upper Wellington Street & Desoto Drive	
Total Drainage Area (ha)	9.10	
ROW Drainage Area (ha)	1.33	
External Drainage Area (ha)	7.77	
Stage	Storage (m³)	Discharge (m³/s)
2-Year	126	0.09
5-Year	197	0.14
10-Year	205	0.32
25-Year	216	0.44
50-Year	218	0.51
100-Year	220	0.58

It should be noted that the preliminary storage system at Upper Wellington at Desoto Drive would only capture the drainage from upstream subcatchments (i.e., C18_12, C18_13, C18_23, C18_24, C18_25, C18_26 and C18_27 ; refer to attached drawings). Consequently, the remaining subcatchments draining to outlet “HI19B070 & HI19B070-S” (i.e. Stone Church Road) from the road Right-of-Way would be expected to drain uncontrolled (i.e. without quantity control storage).

Similarly, the storage system at Upper Wellington Street and South of Sirente Road would only capture the drainage from Subcatchment F02_2 and F02_3 (refer to attached drawings).

6.2.2 Conceptual Design

6.2.2.1 Overview and Approach

At the request of the City of Hamilton, the preliminary storage volumes assessed previously have been used to develop a conceptual design of the required storm sewers and storage volumes. Based on feedback from the City, the preliminary design has focused on the implementation of superpipes for storage.

However, WSP would recommend that sub-surface storage chamber options be considered further as part of the subsequent preliminary and detailed design phase. Chambers do not need to be located in one location given spatial requirements and potential utility conflicts. Rather, these features could be connected to catchbasins and located along the periphery of the road right-of-way to reduce spatial requirements. Pre-treatment through CB Shield™ inserts or similar measures would likely be required to limit sedimentation.

As noted previously, there is no topographic survey, proposed road profile, utility information, or geotechnical/hydrogeological information to support design work. As such, the current approach should be understood to be conceptual only. The design has assumed that the existing profile would generally be maintained under the proposed re-design. This, and the other preceding considerations would all require further review and assessment as part of the subsequent preliminary/detailed design phase.

6.2.2.2 Sirente Drive

As noted previously, a storage volume of approximately 95 m³ would be required to achieve post to pre peak flow control for this location, based on the proposed roadway expansion.

Based on the preceding, approximately 55 m +/- of 1500 mm diameter superpipe would be required to provide the estimated storage volume. Given the steeper grade of the hill south of the intersection, a shorter, larger pipe section is preferred to avoid deeper excavation at the end of the superpipe.

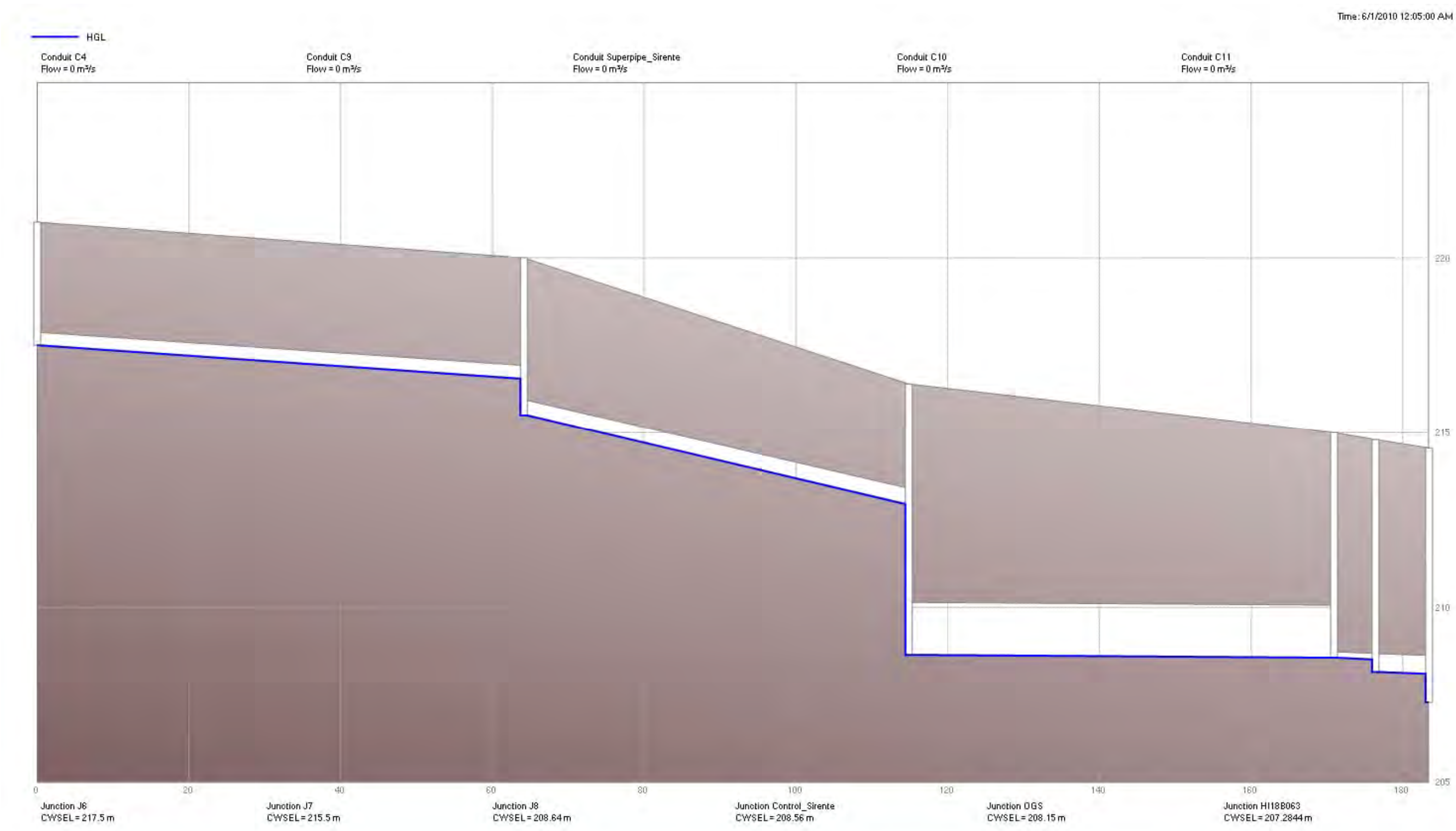
A 170 mm diameter +/- low flow orifice control would be required or equivalent, with a high flow overflow to be incorporated into the design. It is assumed the system would ultimately discharge into a new 525 mm storm sewer connection to the trunk storm sewer on Sirente Drive; potentially to the existing MH at this location to avoid an additional MH replacement or modification to the existing trunk sewer.

The conceptual layout is provided in the attached Drawings as well as in Figure 6.1 below. The preliminary profile is also shown in Figure 6.2. Superpipes typically require very shallow grades (0.1 to 0.2%) to ensure the most efficient use of the linear storage.

Figure 6-1: Conceptual Layout of Proposed Storm Sewers and Superpipe System – Sirente Drive



Figure 6-2: Conceptual Profile of Proposed Storm Sewers and Superpipe System – Sirente Drive



As per Figures 6-1 and 6-2, it is assumed that conventional storm sewers will also need to be installed upstream of the super pipe system to provide road drainage and to direct flows to the super pipe. Given the minor contributing drainage area and generally steep slopes, 375 mm and 450 mm diameter storm sewers have been estimated as shown. The second section (450 mm diameter) has been adjusted to keep pipe slopes below 5%, given the steep road grade in this area. A small drop feature may therefore be required. Sewer depths in this area can be much shallower than the super pipe, and more consistent with typical City standards (minimum cover of 2.75 m).

As noted, the proposed road profile for Upper Wellington Street has not yet been developed. However, as per Figure 6-2, it may be necessary to create a localized minor sag point to ensure full capture of overland flow and direction to the superpipe system. This would need to be reviewed as part of the subsequent preliminary and detailed design phases.

6.2.2.3 Desoto Drive

As noted previously, a storage volume of approximately 210 m³ would be required to achieve post to pre peak flow control for this location, based on the proposed roadway expansion.

As noted, the preferred approach in this case would be to locate this storage at the most upstream limits of the section of roadway draining towards Stone Church Road, to avoid the need to upgrade or replace the larger diameter storm sewers further downstream. Based on the modelling results, the existing storm sewers downstream of Desoto Drive have sufficient capacity to meet City standards (5-year storm event conveyance without surcharging) and thus subject to confirmation of the structural integrity/condition of the sewers, and under the assumption that the proposed roadway profile would generally be similar to the existing profile. This approach would generally require over controlling the most upstream flows, such that overall post to pre peak flow control is achieved at the downstream limits (i.e. at Stone Church Road East). This would mean road segments downstream of Desoto Drive would not require any quantity controls (discharge uncontrolled).

Based on the preceding, approximately 150 m +/- of 1350 mm diameter superpipe would be required to provide the estimated storage volume. A 150 mm diameter +/- low flow orifice control would be required or equivalent, with a high flow overflow to be incorporated into the design. It is assumed the system would ultimately discharge into the existing 525 mm storm sewer near Desoto Drive; likely the last segment of piping connecting to the existing MH (which drains the 600 mm diameter storm sewer from Desoto Drive) could be preserved to avoid an additional MH replacement in this area.

The conceptual layout is provided in the attached Drawings as well as in Figure 6.3 below. The preliminary profile is also shown in Figure 6.4. Superpipes typically require very shallow grades (0.1 to 0.2%) to ensure the most efficient use of the linear storage.

As noted, the proposed road profile for Upper Wellington Street has not yet been developed. However, as per Figure 6-4, it may be necessary to create a localized minor sag point to ensure full capture of overland flow and direction to the superpipe system. This would need to be reviewed as part of the subsequent preliminary and detailed design phases.

Figure 6-3: Conceptual Layout of Proposed Storm Sewers and Superpipe System – Desoto Drive

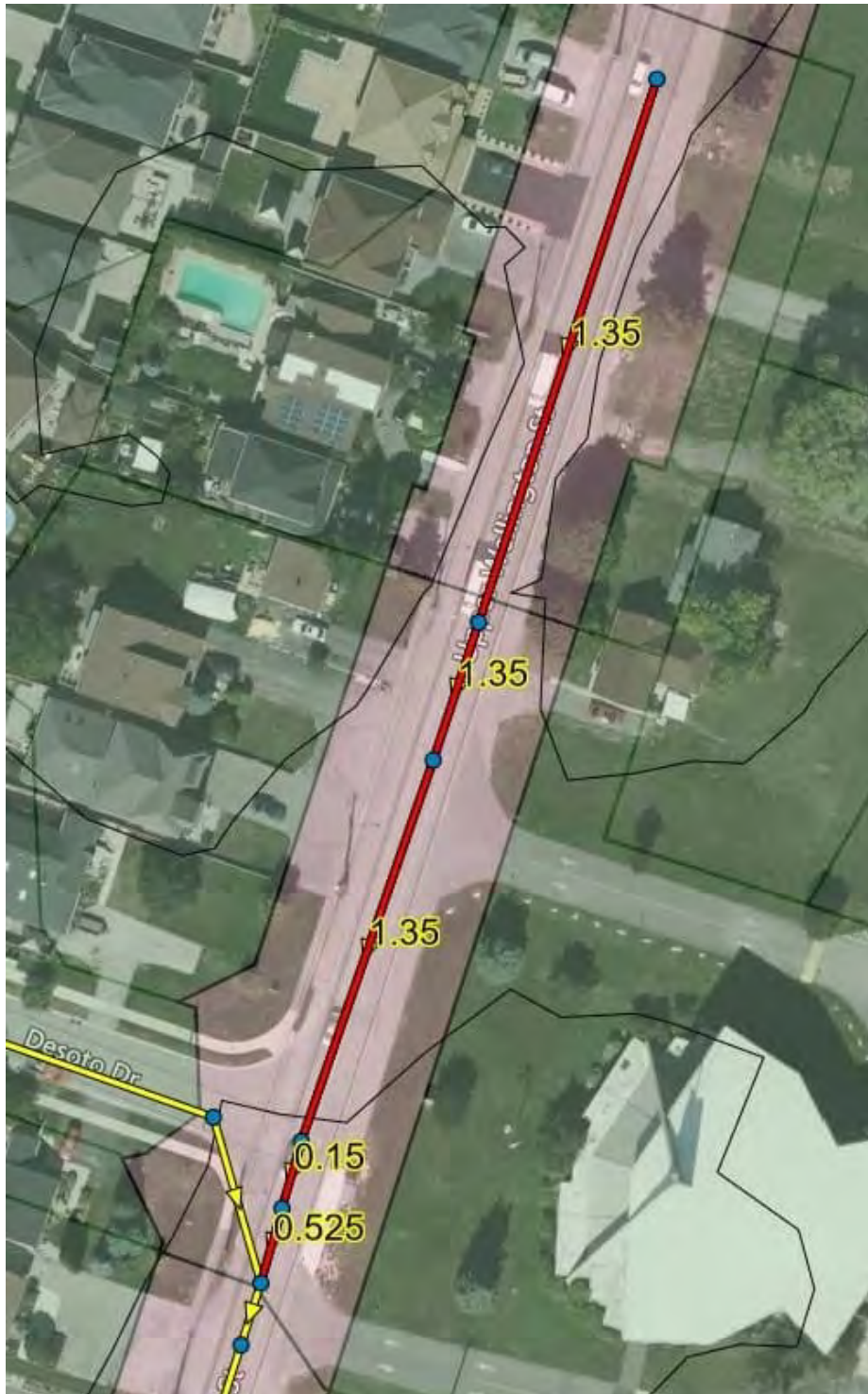
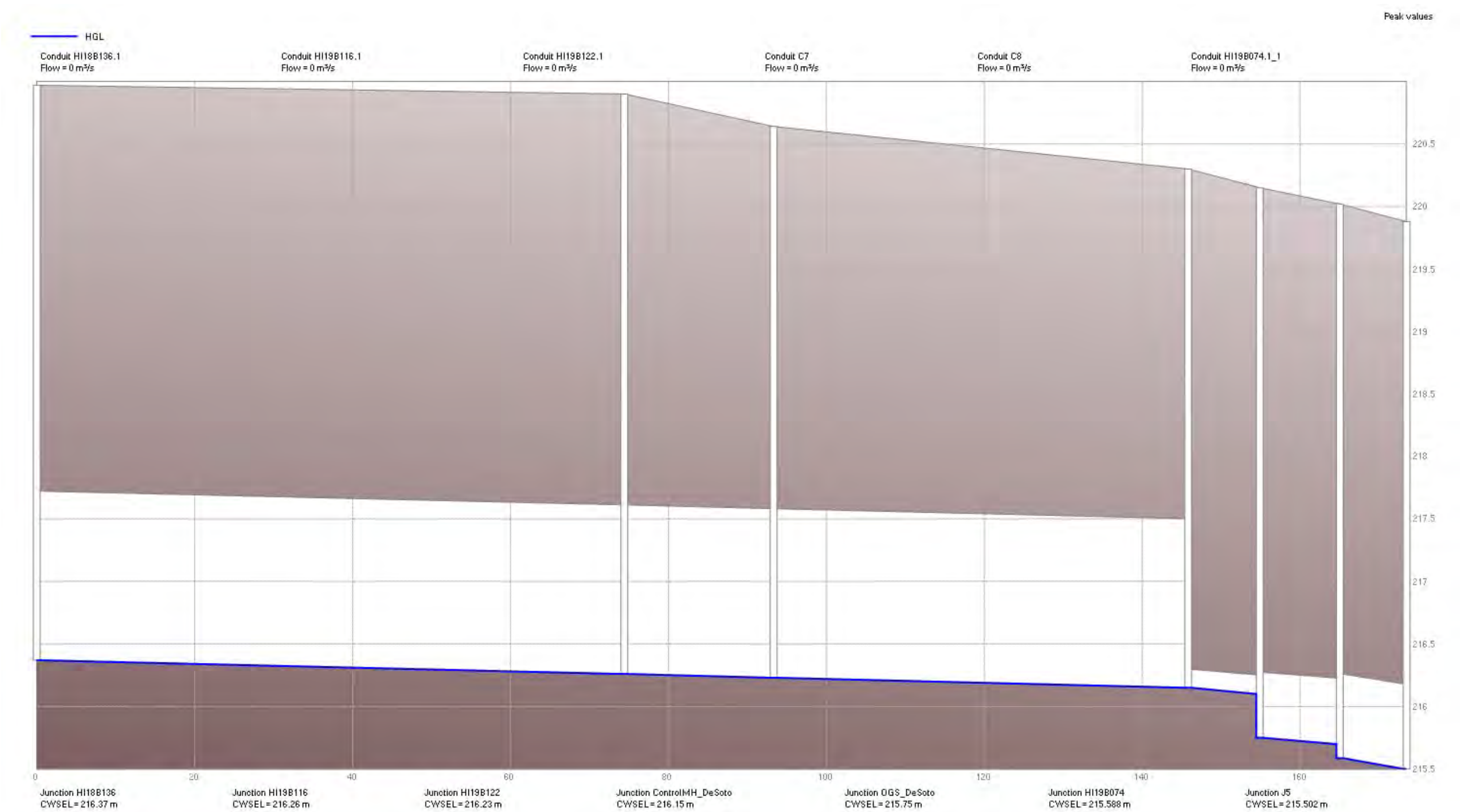


Figure 6-4: Conceptual Profile of Proposed Storm Sewers and Superpipe System – Desoto Drive



6.3 Erosion and Volume Control

Erosion control may be supportable through infiltrative BMPs, where groundwater and bedrock elevations permit. A detailed Geotechnical Investigation report will be required to verify the groundwater depths along the road right-of-way at the detailed design stage; this information was not available for the current Class EA.

In locations where infiltrative BMPs are not possible due to high groundwater and bedrock elevations, or where infiltration capacity does not permit, the quantity control storage tanks should be used to retain and release 25 mm storm event over a 24-hour period.

It is proposed to locate infiltrative BMPs (infiltration trenches and tree planter cells) under the pedestrian bike lanes and sidewalks to the extent possible to minimize the impact of BMP maintenance and operations on traffic flow. Alternatively, exfiltration pipes could be considered and located within the primary road right-of-way.

Infiltrative features should receive drainage from a pre-treatment unit, such as an oil/grit separator (OGS) CB Shield™ or if not feasible, catchbasin goss traps. Once the cells reach full capacity, they will overflow into the storm sewer drainage network. The proposed infiltration cells should have a bottom depth of at least 1 meter above the groundwater level or bedrock elevation throughout the project. The cells would typically be filled with 50 mm clear stone and have a void ratio of 0.4. Any portion of the 25 mm event which cannot be managed through infiltration should be retained in storage tanks and released over a 24-hour period through an orifice control structure.

Table 6-9 indicates the required erosion control volume as well as the necessary infiltration and retention volumes for each drainage outlet.

Table 6-9: Erosion Control Volume for Additional Impervious Area

DRAINAGE NODE	DRAINAGE OUTLET LOCATION	ROW DRAINAGE AREA			ADDITIONAL IMPERVIOUS AREA (ha)	EROSION CONTROL VOLUME REQUIRED (m³)
		TOTAL (ha)	IMP (%)	IMP (ha)		
HI19B070 & HI19B070-S	Intersection of Upper Wellington Street & Stone Church Road East	1.33	70	0.93	0.23	57
HI18B063 & HI18B063-S	Intersection of Upper Wellington Street & Sirente Drive	0.90	74	0.66	0.13	34

The volumes presented in Table 6-9 are preliminary and it is expected that further refinement will be required as part of the detailed design stage, including considerations of site constraints and grading and layout feasibility. It should be noted that the erosion control volumes provided above would be in addition to the quantity control volumes. Therefore, the total volume required for each outlet would be the sum of erosion and quantity control volumes provided. The potential balance of infiltrative and storage volumes will also require further consideration, once more site-specific data is available during the detailed design stage.

6.4 Quality Control

6.4.1 Base Quality Control Assessment

Water quality measures to provide an Enhanced Level or 80% TSS removal of water quality protection for the proposed increase in the pavement to each drainage outlet have been selected with consideration to the contributing drainage area, magnitude in the increase in paved area, ROW spatial constraints, and effectiveness of water quality measures.

To provide stormwater quality control, proposed systems must provide a minimum of 80% TSS removal for new pavement/impervious surfaces.

Pre-treatment for any proposed infiltration systems (LID BMPs, if proposed) would need to be provided (i.e. creditable measures such as oil/grit separators (OGS units) and/or CB Shields™ or other non-credited measures such as Littatrap or goss traps) depending upon location.

In the case of OGS units, it is understood that the City typically only accepts 50% TSS removal efficiency for OGS units based on the New Jersey Department of Environmental Protection (NJDEP), that said oil grit separators (e.g., Stormceptor™ EF Units) tested under the Environmental Technology Verification (ETV) do provide greater than 50% TSS removal. It is generally accepted based on MECP guidance and treatment standards that OGS units will appropriately treat up to 2 ha (+/-); as the drainage area to an OGS unit increase, the peak flows will also increase and could exceed the flow capacity of the OGS unit, with the exceeded flows bypassing the treatment function of the OGS.

Equivalent TSS removal rates required for the proposed ROW subcatchments are indicated in **Table 6-10**.

Table 6-10: Equivalent TSS Removal Required for the Proposed ROW Subcatchments

DRAINAGE NODE	DRAINAGE OUTLET LOCATION	ROW DRAINAGE AREA			CHANGE IN PAVED AREA (ha)	TSS REMOVAL REQUIRED (%)	EQUIVALENT TSS REMOVAL REQUIRED (%)
		TOTAL (ha)	IMP (%)	IMP (ha)			
HI19B070 & HI19B070-S	Intersection of Upper Wellington Street & Stone Church Road East	1.33	70	0.93	0.23	80	20
HI18B063 & HI18B063-S	Intersection of Upper Wellington Street & Sirente Drive	0.90	74	0.66	0.13	80	16

Per the water quality control criteria, 80% of TSS removal must be provided for the additional paved surfaces (i.e., impervious area). As indicated in **Table 6-10**, to determine the required infiltration to achieve the 80% TSS removal target, an equivalent TSS removal rate is calculated by weighting the additional impervious area against the total impervious area under future conditions. For example, consider drainage outlet –Node HI19B070 & HI19B070-S. for that stretch of the road, providing 80% TSS removal on the additional 0.23 ha impervious area is equivalent to providing 20% TSS removal on the total impervious area of 0.93 ha under the future conditions. Hence, if water

quality measures can provide at least 20% of TSS removal for the total impervious area of 0.93 ha under future conditions, that infiltration volume would be deemed sufficient to provide the required water quality control.

As per the attached drawings, under proposed conditions it has been assumed that new oil/grit separators would be located downstream of the superpipe storage systems. Further review with City staff would be required as part of the subsequent preliminary\detailed design phase to determine whether a stand-alone OGS unit could be accepted for quality control measures or secondary treatment measures would be required. This would be more challenging if a superpipe system is proposed for quantity control however (as opposed to sub-surface storage chambers).

In general, given the size of the roadway contributing, it is expected that secondary quality control measures will be required downstream of Desoto Drive (i.e. to Stone Church Road East) regardless, as the area of roadway to be treated by the OGS unit upstream of Desoto Drive would generally be insufficient to provide the overall level of treatment required given the expected new roadway\pavement area. As such, secondary measures such as CB Shield™ inserts may be the most cost-effective solution in this area. However, as noted this would require further review through subsequent preliminary and detailed design phases.

6.4.2 Water Quality Control through Infiltrative Measures

As such, it is likely that only one form of quality control would be required (i.e. a treatment train is not necessary). From a quality control perspective, it is expected that an oil/grit separator (OGS) unit or other engineered methods could likely achieve the quality control targets presented in **Table 6-10**. However, these measures would not provide any infiltrative or erosion control benefit, as would be provided by LID BMPs.

The TSS removal efficiency of infiltrative BMPs has been estimated using Table 3.2 of the MOE (now MECP) 2003 Stormwater Management Planning and Design Manual. The estimated infiltration volume required to achieve 80% TSS removal (without any formalized\credited pre-treatment) for the additional impervious areas are summarized in **Table 6-11**.

To achieve this, an equivalent TSS removal rate is calculated by weighting the additional impervious area against the total impervious area under the future conditions. For example, providing 80% TSS removal on the additional 0.22 ha impervious area is equivalent to providing 20% TSS removal on the total impervious area of 0.93 ha under future conditions.

Table 6-11: Infiltration Feature Sizing Requirement for Water Quality Only

DRAINAGE OUTLET	DRAINAGE OUTLET LOCATION	FUTURE ROW DRAINAGE AREA			EQUIVALENT TSS REMOVAL REQUIRED (%)	CHANGE IN PAVED AREA (ha)	INFILTRATION VOLUME ADDITIONAL IMP AREA 80% TSS REMOVAL (m³)
		TOTAL (ha)	IMP (%)	IMP (ha)			
HI19B070 & HI19B070-S	Intersection of Upper Wellington Street & Stone Church Road East	1.33	70	0.93	20	0.22	5

DRAINAGE OUTLET	DRAINAGE OUTLET LOCATION	FUTURE ROW DRAINAGE AREA			EQUIVALENT TSS REMOVAL REQUIRED (%)	CHANGE IN PAVED AREA (ha)	INFILTRATION VOLUME ADDITIONAL IMP AREA 80% TSS REMOVAL (m ³)
		TOTAL (ha)	IMP (%)	IMP (ha)			
HI18B063 & HI18B063-S	Intersection of Upper Wellington Street & Sirente Drive	0.90	74	0.66	16	0.13	3

As discussed in Section 6.3, infiltrative features (where possible) are proposed to be located under the sidewalks and bike lanes (infiltration cells and tree planter cells) or within the roadway ROW if exfiltration pipes are determined to be feasible.

Table 6-12 presents the required volumes for the infiltration cells to satisfy both the erosion control volumes as indicated in **Table 6-9** and the quality controls according to **Table 6-11**. Overall, it is evident that erosion control sizing requirements would govern over water quality sizing requirements. However, if an infiltrative LID BMP is provided for erosion\volume control purposes, it could also easily provide the required quality control function as well, potentially avoiding the need for more costly engineered water quality measures (i.e. such as an OGS unit). Notwithstanding, pre-treatment of some type would still be required.

Table 6-12: Minimum Infiltration Cell Sizing Required

DRAINAGE OUTLET	DRAINAGE OUTLET LOCATION	REQUIRED INFILTRATION CELL VOL. FOR 80% TSS REMOVAL QUALITY CONTROL (m ³)	REQUIRED INFILTRATION CELL VOL. FOR 25MM EROSION CONTROL (m ³)
HI19B070 & HI19B070-S	Intersection of Upper Wellington Street & Stone Church Road East	5	57
HI18B063 & HI18B063-S	Intersection of Upper Wellington Street & Sirente Drive	3	34

7 SUMMARY AND RECOMMENDATIONS

7.1 Existing Conditions

An updated hydrologic / hydraulic model was developed in PCSWMM for the Upper Wellington Street Class EA study area between Limeridge Road East to the north and 50 m south of Stone Church Road East. Both hydrologic and hydraulic modelling elements have been refined to assess drainage capacity within the study area limits.

Overall, the updated modelling generated minor system peak flow rates that indicated differences from previous versions of the model. These differences are attributed to the refinements to the subcatchment boundaries, addition of recent development areas, and additions / revisions to hydraulic modelling elements (storm sewers and overland flow routes / roadways). Overall, the current updated model is considered appropriate to use for this assessment.

The performance results for the major and minor drainage systems have been reviewed for the study area. The performance of the existing Towercrest Drive / Sirente Drive storm sewer system, simulated for the 5-year design storm event (consistent with City of Hamilton standards), indicates available capacity in the vicinity of the study area, however, the storm sewer is surcharged approximately 300 m (+/-) downstream of the study area. Similarly, there is available capacity in the Upper Wellington Street storm sewer system south of the study area however, the storm sewer is surcharged on Dragoon Drive, 80 m (+/-) east of Upper Wellington Street. Within the limits of the study area however, no storm sewer surcharging is indicated for the City's design standard event.

The major system's performance is generally acceptable for the study area, except for the sag at the intersection with Towercrest Drive / Sirente Drive. During the 100-year design storm event, the simulate major system flow depth reaches approximately 0.41 m (+/-), which significantly exceeds the City's standard of 0 mm at the crown for arterial roadways. A minor exceedance of this criterion is indicated at the southern limits of the study area (at Stone Church Road), however, the majority of the southern portion of the study area does not indicate a significant depth accumulation and remains below the City's standards when considering typical roadway cross-falls.

As such, any subsequent design work should consider the following:

- A notable major system depth accumulation has been indicated at the sag point at Upper Wellington Street and Towercrest/Sirente
- Grading improvements or increased inlet capacity (i.e. catchbasins and ditch inlets) should be considered.
- The potential to incorporate storage into the depressed areas in the south-west corner of the intersection may also warrant further assessment, however this could potentially result in adverse impacts to trees and vegetation.

Stormwater quality control treatment within the study area is limited, based on a review of available drawings and GIS storm sewer data. The existing Mountain Brow SWM facility, currently being reconstructed, was intended to provide Level 2 (70% treatment of total suspended solids) for the contributing drainage area, including the study area (S. Llewellyn and Associates Ltd., April 2008).

7.2 Future Conditions

For future conditions, the proposed improvements would include the following:

- Upper Wellington Street from Stone Church Road to Towercrest Drive / Sirente Drive: Widening from two-lane cross-section to a three-lane cross-section, incorporating 3.3-meter vehicular through-lanes, a 3-meter center-turn lane, 1.5-meter off-street bike lanes and 1.8-meter sidewalks on both sides, separated by a 0.6-meter buffer and physically separated from vehicular traffic lanes by 3-meter planting strips/boulevards.
- Upper Wellington Street from Towercrest Drive / Sirente Drive to Limeridge Road: Road diet to reduce the existing four-lane cross-section to a two-lane cross-section with 3.3-meter vehicular through-lanes and dedicated turning lanes at intersections. The reclaimed space would accommodate 1.8 metre bike lanes, 1.5 metre buffer for streetscaping elements, and maintaining existing 1.6-metre sidewalks with 0.6-metre buffer. Over the Lincoln M. Alexander Parkway overpass, the design would transition to leverage the existing 2.0-meter bridge sidewalks and repurpose the existing roadway platform for 1.8-meter bike lanes, physically separated from vehicular traffic by medians, reducing impacts to the existing bridge and structure.

Future improvements will increase the ROW impervious area by approximately 0.46 hectares (this may be refined as part of subsequent detailed design). A future conditions updated hydrologic / hydraulic model has been developed in PCSWMM for the study area between Limeridge Road East to the north and 50 m south of Stone Church Road East to assess the impacts of these changes. Based on the results of this assessment, quantity control storage is required. In addition, water quality controls are required. It is also recommended that erosion/volume controls be provided; it is recommended that the 25mm rainfall be captured through a combination of infiltration and active storage volume, with a preference for infiltrative measures if feasible.

Potential SWM measures were assessed and a short-list of measures prepared for further consideration.

Based on the preceding, the following has been recommended for further refinement at the detailed design stage with respect to SWM measures:

- Two (2) separate superpipe systems are expected to be required to control post-construction peak flows to pre-development values; preliminary requirements are as follows:
 - *Sirente Drive: 95 m³ +/- (south of the Intersection of Upper Wellington Street & Sirente Drive) – 55 m of 1500 mm pipe +/-*
 - *Desoto Drive: 220 m³ +/- (north of Intersection of Upper Wellington Street & Desoto Drive) – 150 m of 1350 mm pipe +/-*

As noted previously, these layouts and sizings are considered conceptual only given the limited information available at this stage of the assessment. We would recommend this be re-assessed as part of the subsequent preliminary/detailed design phase, including consideration for using sub-surface storage chamber systems (rather than superpipes) along the periphery of the roadway right-of-way to limit the construction cost and footprint (as opposed to one large central chamber system that may conflict with utilities).

Given the proposed road urbanization (curb and gutter) additional storm sewers and associated conveyance appurtenances (i.e. catchbasins and maintenance holes) may be required. We would note that in addition to the proposed superpipe systems previously described):

- The storm sewers south (downstream) of Desoto Drive would generally be proposed to be preserved based on an overview of capacity, subject to further review of the proposed roadway profile and sewer condition as part of subsequent preliminary/detailed design.
- Additional storm sewers would be proposed upstream of Sirente Drive (from the roadway high point to the south), in the area that does not currently have storm sewer servicing.

- No new storm sewers are proposed to the north of Sirente Drive given that no overall change in imperviousness is proposed in this area, and the storm sewers in this area are therefore generally considered adequate.
- It is expected that numerous additional catchbasins will be required as part of the subsequent detailed road design to ensure collection of stormwater from the new urbanized road section.

For quality control, two (2) oil/grit separator units have been proposed to provide treatment in conjunction with the locations of the proposed superpipe systems. It is expected that at a minimum, additional secondary water quality controls will be required south of Desoto Drive (to Stone Church Road East) to accommodate the additional paved\impervious area in this location. CB Shield™ inserts may be the most direct solution in this case, however secondary LID BMPs could also be considered (such as tree planter\soil retention cells, exfiltration pipes etcetera). Further discussion with City staff would also be required to determine the degree to which OGS units can be accepted\credited for overall TSS removal.

If sub-surface storage chambers are ultimately determined to be feasible for quantity control, as noted it is considered the most efficient to combine these features with an infiltration function to provide water quality and erosion\volume control requirements. As such, while alternative LID BMP measures were short-listed (i.e. tree planter\soil retention cells, exfiltration pipes) it is expected that these would be less preferred if sub-surface storage chambers can be accommodated. Similarly, while other quality control measures were short-listed (i.e. OGS units, CB Shields™) these would not be required if quality control requirements can be met by the infiltrative function of the chambers.

As noted, the preceding is subject to the collection of additional site-specific data to assess feasibility as well as further analysis and design, to be completed as part of the subsequent detailed design stage. It is expected that additional considerations would include (but are not limited to):

- Geotechnical investigation (types of surficial soils and infiltration potential, depth to bedrock, etcetera)
- Hydrogeological investigation (depth to groundwater and seasonal variation, infiltration potential of surficial soils)
- Topographic survey (to inform the subsequent road profile and grading)
- Subsurface Utility Engineering (SUE) investigation (to confirm locations of utilities and potential conflicts with proposed works)

The base PCSWMM models developed as part of this study should ideally be utilized and updated as part of the subsequent detailed design effort, including not only re-confirming SWM sizing and specifics, but also determining storm sewer sizing requirements and re-assessing major system overland flow capacity, which was not feasible as part of the current phase of study.

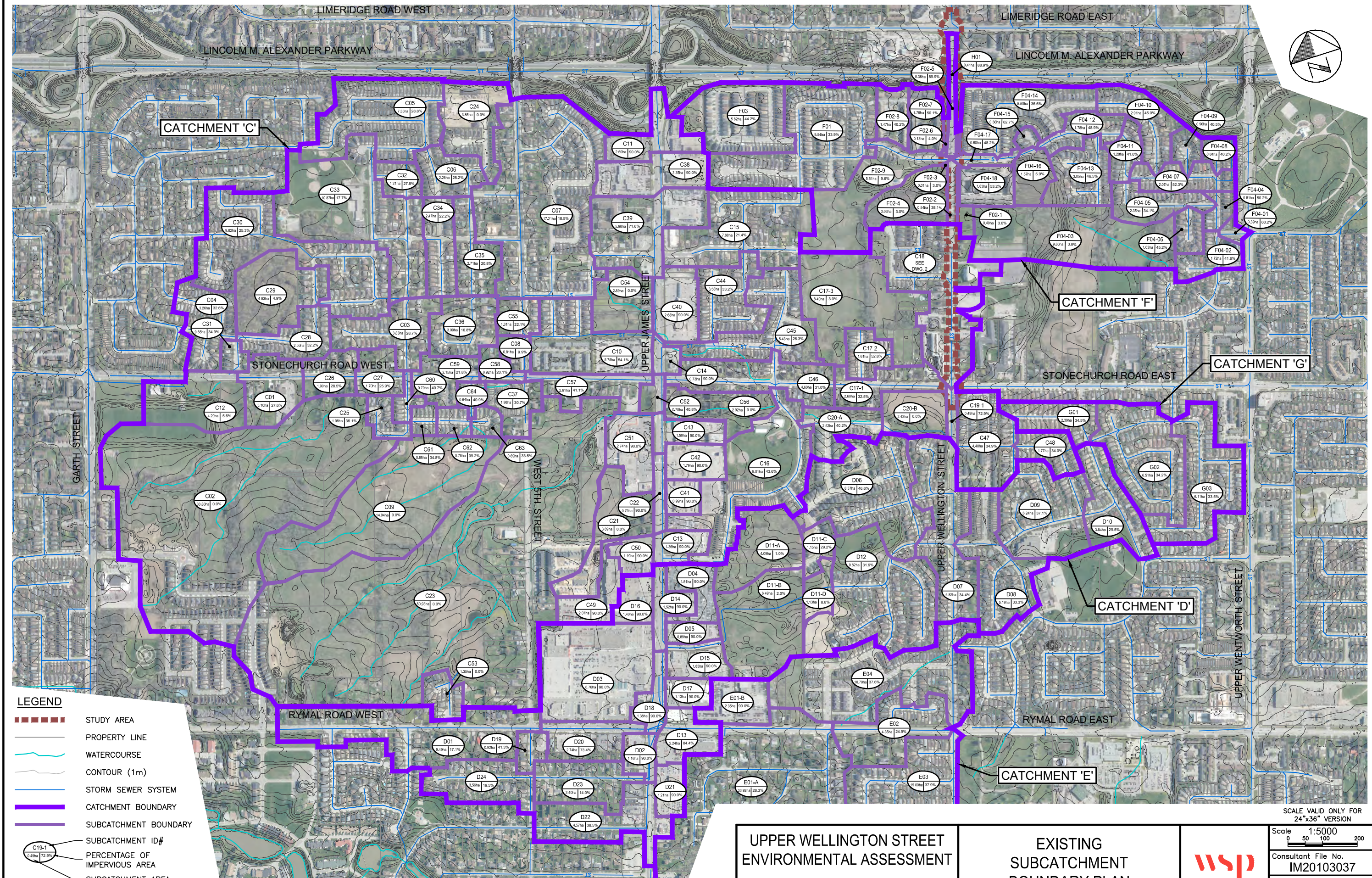
It should also be noted that the preceding analyses did not consider the potential impacts of climate change and more intense rainfall. As part of the subsequent detailed design stage, the City may wish to incorporate this into the design to provide a factor of safety and additional resiliency against the potential impacts of climate change.

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Plotted By: wds_richard.bartolo
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LEGEND

- STUDY AREA
- PROPERTY LINE
- WATERCOURSE
- CONTOUR (1m)
- STORM SEWER SYSTEM
- CATCHMENT BOUNDARY
- SUBCATCHMENT BOUNDARY
- SUBCATCHMENT ID#
- PERCENTAGE OF IMPERVIOUS AREA
- SUBCATCHMENT AREA



UPPER WELLINGTON STREET
ENVIRONMENTAL ASSESSMENT
CITY OF HAMILTON

EXISTING
SUBCATCHMENT
BOUNDARY PLAN



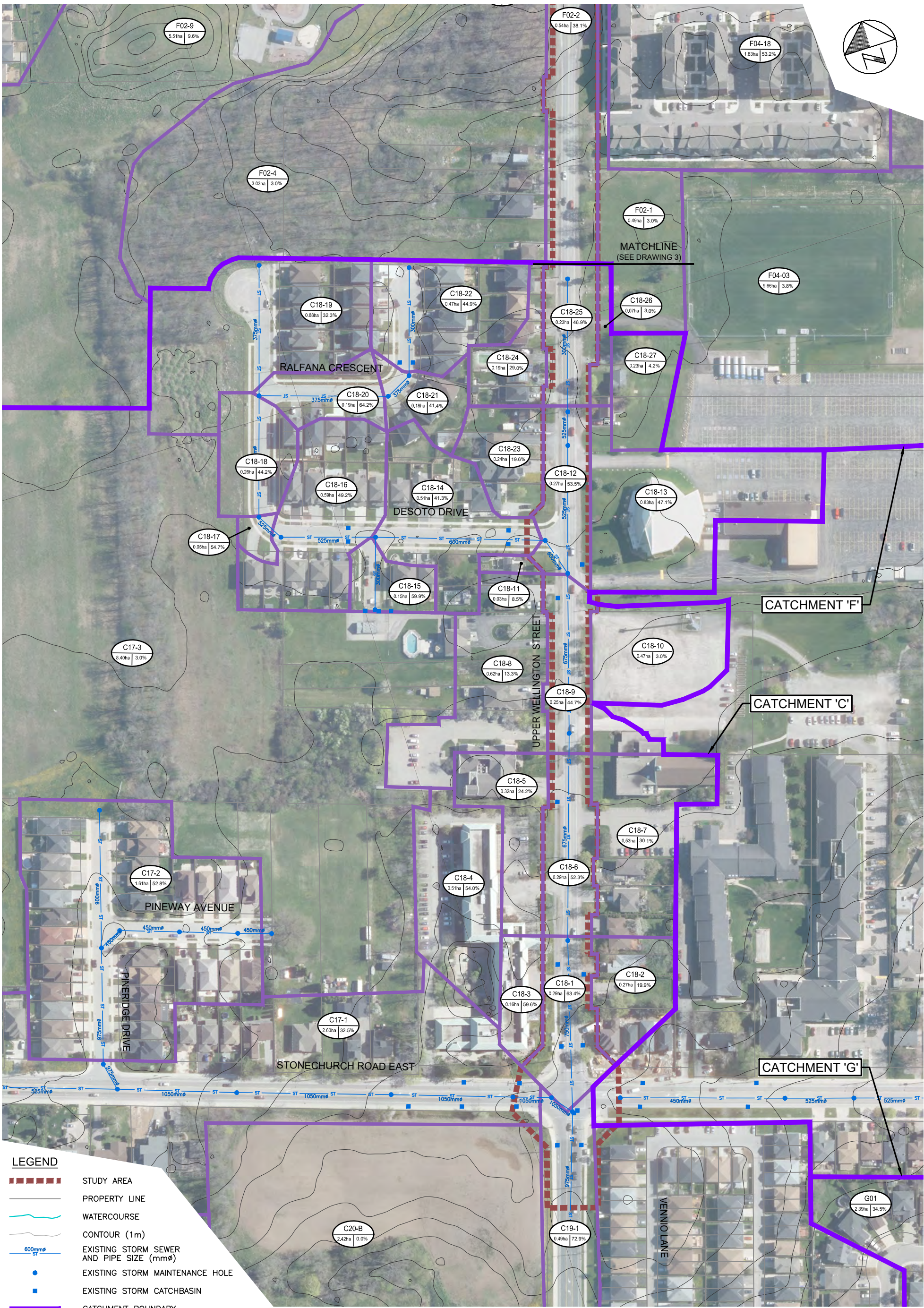
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Drawing No.
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wds_richard.bortolo
Plotted By: wds_richard.bortolo
2025-06-27
Last Saved: 2025-06-27
Plotted: 2025-06-27



LEGEND

- STUDY AREA
- PROPERTY LINE
- WATERCOURSE
- CONTOUR (1m)
- EXISTING STORM SEWER AND PIPE SIZE (mmØ)
- EXISTING STORM MAINTENANCE HOLE
- EXISTING STORM CATCHBASIN
- CATCHMENT BOUNDARY
- SUBCATCHMENT BOUNDARY
- SUBCATCHMENT ID#
- PERCENTAGE OF IMPERVIOUS AREA
- SUBCATCHMENT AREA

UPPER WELLINGTON STREET ENVIRONMENTAL ASSESSMENT
CITY OF HAMILTON

EXISTING SUBCATCHMENT BOUNDARY PLAN
(NORTH OF STONECHURCH ROAD E)



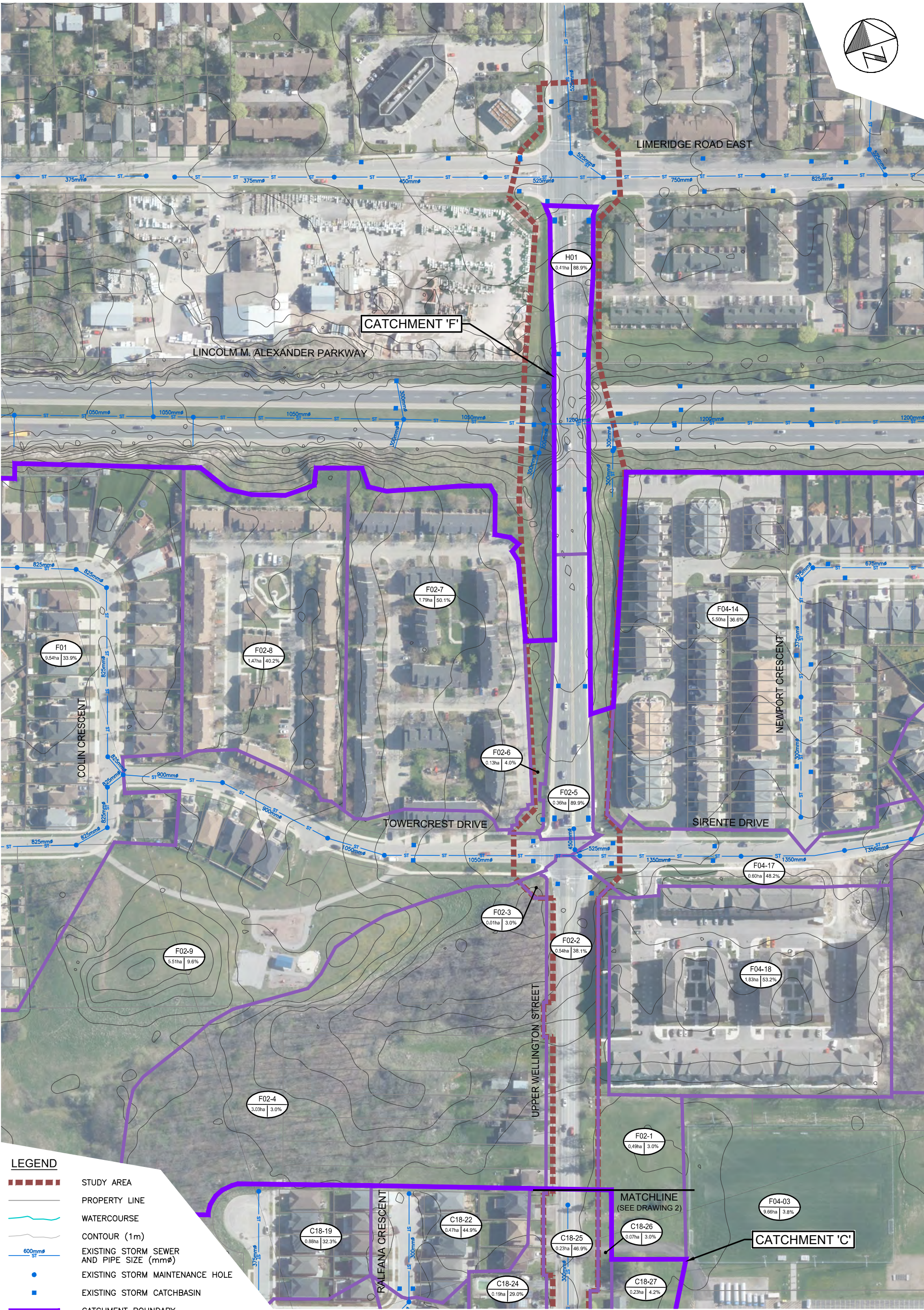
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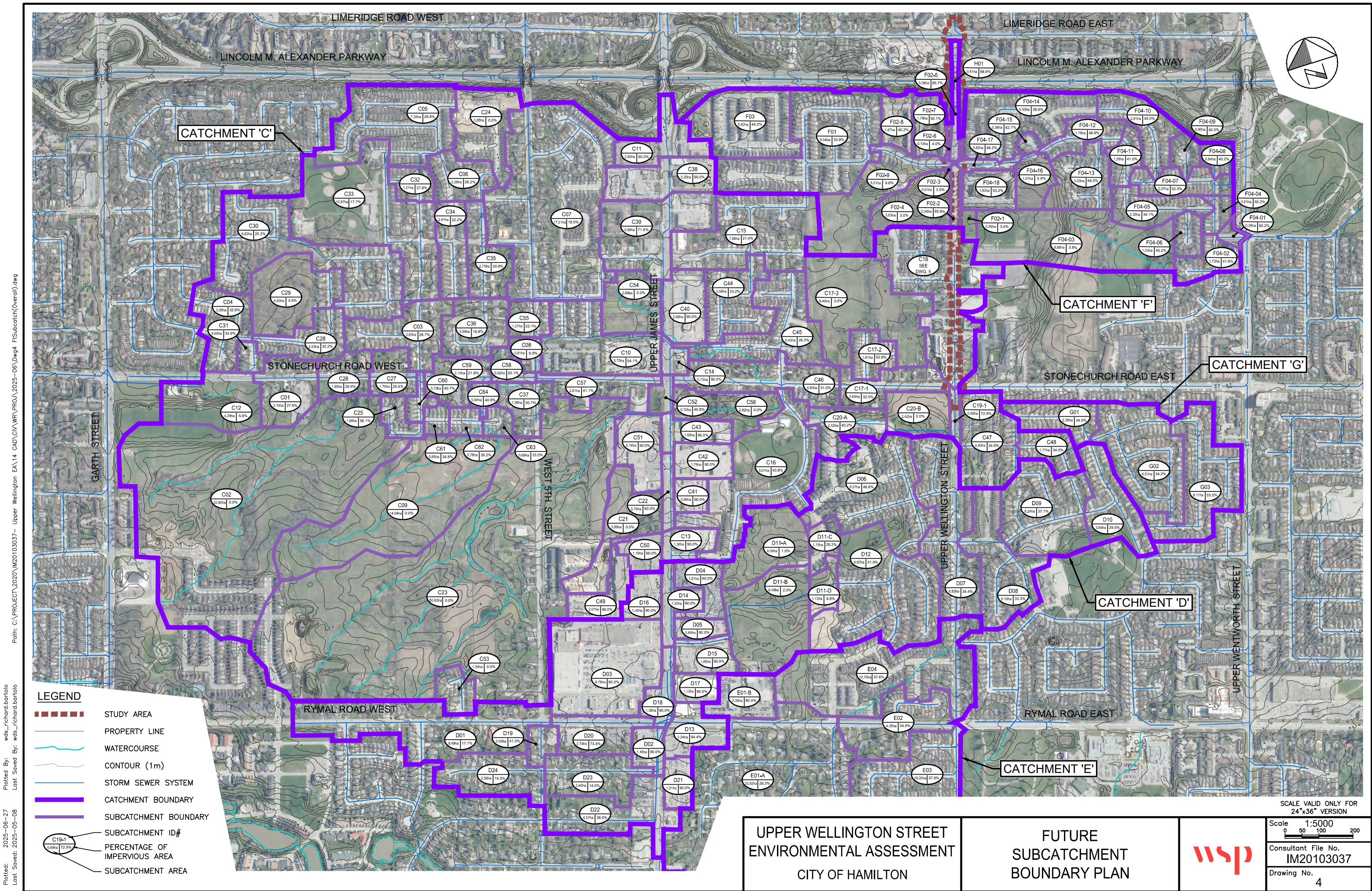
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2025-06-27
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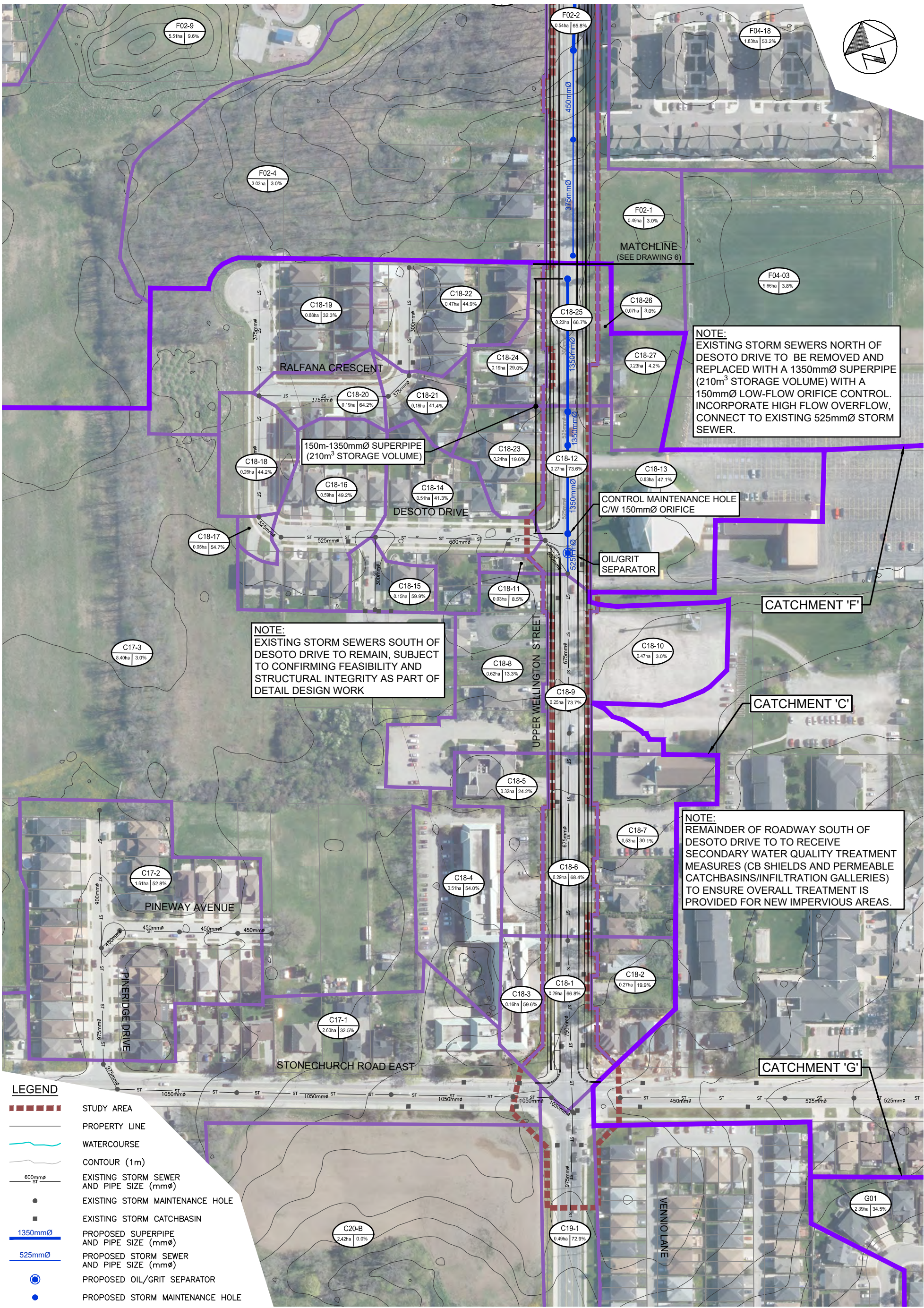
UPPER WELLINGTON STREET
ENVIRONMENTAL ASSESSMENT
CITY OF HAMILTON

EXISTING
SUBCATCHMENT
BOUNDARY PLAN
(SOUTH OF LIMERIDGE ROAD E)

wsp



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2025-06-27
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LEGEND

- STUDY AREA
- PROPERTY LINE
- WATERCOURSE
- CONTOUR (1m)
- EXISTING STORM SEWER AND PIPE SIZE (mmØ)
- EXISTING STORM MAINTENANCE HOLE
- EXISTING STORM CATCHBASIN
- PROPOSED SUPERPIPE AND PIPE SIZE (mmØ)
- PROPOSED STORM SEWER AND PIPE SIZE (mmØ)
- PROPOSED OIL/GRIT SEPARATOR
- PROPOSED STORM MAINTENANCE HOLE
- CATCHMENT BOUNDARY
- SUBCATCHMENT BOUNDARY
- SUBCATCHMENT ID#
- PERCENTAGE OF IMPERVIOUS AREA
- SUBCATCHMENT AREA

UPPER WELLINGTON STREET
ENVIRONMENTAL ASSESSMENT
CITY OF HAMILTON

FUTURE
SUBCATCHMENT
BOUNDARY PLAN
(NORTH OF STONECHURCH ROAD E)



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5



Appendix A

Updated Existing Conditions Hydrology (2021)



Table A1 January 2021 Updated Subcatchment Imperviousness Calculations															
Subcatchment Name	Area (ha)	Number of Townhouses	Number of Detached Houses	Houses Not Identified in Building Layer	Detached House Area (ha)	Houses Not identified in Building Layer (ha)	Detached House Amenity Area (ha)	Townhouse Impervious Area (ha)	Townhouse Amenity Area (ha)	Total Residential Impervious Area (ha)	Total ROW Area (ha)	Impervious Road Area Manually Measured (ha)	Impervious Road Area (ha)	Total Impervious Area (ha)	Total Imperviousness (%)
C17_1	2.60	0	15	0	0.25	0.00	0.15	0.00	0.00	0.41	0.77	0.00	0.60	1.00	38.7
C17_2	1.61	0	32	0	0.53	0.00	0.33	0.00	0.00	0.86	0.43	0.00	0.34	1.20	74.1
C17_3	8.40	0	0	0	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.00	0.01	0.01	5.0
C18_1	0.29	0	0	0	0.00	0.00	0.00	0.00	0.00	0.00	0.29	0.18	0.18	0.18	64.2
C18_10	0.49	0	0	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	5.0
C18_11	0.03	0	1	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	14.1
C18_12	0.26	0	0	0	0.00	0.00	0.00	0.00	0.00	0.00	0.26	0.14	0.14	0.14	53.8
C18_13	0.84	0	2	0	0.21	0.00	0.13	0.00	0.00	0.34	0.00	0.00	0.00	0.66	78.4
C18_14	0.51	0	4	0	0.07	0.00	0.04	0.00	0.00	0.12	0.18	0.00	0.14	0.26	50.4
C18_15	0.15	0	2	0	0.03	0.00	0.02	0.00	0.00	0.05	0.07	0.00	0.06	0.11	74.4
C18_16	0.59	0	13	0	0.21	0.00	0.13	0.00	0.00	0.34	0.11	0.00	0.09	0.43	72.4
C18_17	0.05	0	0	0	0.00	0.00	0.00	0.00	0.00	0.00	0.03	0.00	0.03	0.03	54.6
C18_18	0.26	0	1	0	0.02	0.00	0.01	0.00	0.00	0.02	0.13	0.00	0.10	0.13	48.0
C18_19	0.88	0	9	0	0.15	0.00	0.09	0.00	0.00	0.23	0.18	0.00	0.14	0.38	42.9
C18_2	0.27	0	4	0	0.05	0.00	0.03	0.00	0.00	0.09	0.00	0.00	0.00	0.09	32.7
C18_20	0.19	0	2	0	0.03	0.00	0.02	0.00	0.00	0.05	0.12	0.00	0.09	0.14	74.5
C18_21	0.18	0	4	0	0.05	0.00	0.03	0.00	0.00	0.08	0.03	0.00	0.03	0.11	59.6
C18_22	0.47	0	7	0	0.13	0.00	0.08	0.00	0.00	0.21	0.11	0.00	0.09	0.30	62.6
C18_23	0.24	0	3	0	0.05	0.00	0.03	0.00	0.00	0.08	0.00	0.00	0.00	0.08	32.6
C18_24	0.20	0	4	0	0.06	0.00	0.04	0.00	0.00	0.10	0.00	0.00	0.00	0.10	48.3
C18_26	0.07	0	0	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	5.0
C18_27	0.23	0	1	0	0.01	0.00	0.01	0.00	0.00	0.02	0.00	0.00	0.00	0.02	7.0
C18_29	0.22	0	0	0	0.00	0.00	0.00	0.00	0.00	0.00	0.22	0.11	0.11	0.11	48.0
C18_3	0.16	0	1	0	0.10	0.00	0.06	0.00	0.00	0.16	0.00	0.00	0.00	0.16	99.3
C18_4	0.51	0	1	0	0.15	0.00	0.09	0.00	0.00	0.24	0.00	0.00	0.00	0.24	90.0
C18_5	0.33	0	1	0	0.08	0.00	0.05	0.00	0.00	0.13	0.00	0.00	0.00	0.13	40.3
C18_6	0.27	0	0	0	0.00	0.00	0.00	0.00	0.00	0.00	0.27	0.15	0.15	0.15	53.7
C18_7	0.54	0	3	0	0.17	0.00	0.10	0.00	0.00	0.27	0.00	0.00	0.00	0.27	50.1
C18_8	0.64	0	5	0	0.09	0.00	0.05	0.00	0.00	0.14	0.00	0.00	0.00	0.14	22.1
C18_9	0.22	0	0	0	0.00	0.00	0.00	0.00	0.00	0.00	0.22	0.11	0.11	0.11	49.6
C19_1	0.49	0	0	0	0.00	0.00	0.00	0.00	0.00	0.00	0.46	0.00	0.36	0.36	72.9
F02_1	0.49	0	0	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	5.0
F02_2	0.38	0	0	0	0.00	0.00	0.00	0.00	0.00	0.00	0.38	0.16	0.16	0.16	42.2
F02_3	0.12	0	0	0	0.00	0.00	0.00	0.00	0.00	0.00	0.12	0.05	0.05	0.05	40.8
F02_4	3.08	0	5	0	0.06	0.00	0.03	0.00	0.00	0.09	0.00	0.00	0.00	0.09	5.0
F02_5	0.42	0	0	0	0.00	0.00	0.00	0.00	0.00	0.00	0.42	0.00	0.33	0.33	100.0

[illegible]

Table A2 January 2021 Updated Existing Conditions Subcatchment SCS CN Calculations

Subcatchment Name	Area (ha)	XIMP	Base CN	Adjusted CN	Total Imp (%)	Total Imp (ha)	Road or Commercial (ha)	Residential (ha)
C17_1	2.60	32.5	74	76.2	38.71	1.00	0.60	0.41
C17_2	1.61	52.8	74	84.8	74.15	1.20	0.34	0.86
C17_3	8.40	3.0	74	74.5	5.00	0.42	0.01	0.41
C18_1	0.29	64.2	74	74.0	64.20	0.18	0.18	0.00
C18_10	0.49	3.0	74	74.5	5.00	0.02	0.00	0.02
C18_11	0.03	8.5	74	75.5	14.11	0.00	0.00	0.00
C18_12	0.26	53.8	74	74.0	53.85	0.14	0.14	0.00
C18_13	0.84	47.1	74	88.2	78.42	0.66	0.00	0.66
C18_14	0.51	41.2	74	77.7	50.37	0.26	0.14	0.12
C18_15	0.15	59.8	74	82.7	74.37	0.11	0.06	0.05
C18_16	0.59	49.2	74	84.9	72.36	0.43	0.09	0.34
C18_17	0.05	54.6	74	74.0	54.65	0.03	0.03	0.00
C18_18	0.26	44.2	74	75.6	47.95	0.13	0.10	0.02
C18_19	0.88	32.2	74	77.8	42.92	0.38	0.14	0.23
C18_2	0.27	19.9	74	77.8	32.74	0.09	0.00	0.09
C18_20	0.19	64.2	74	80.9	74.46	0.14	0.09	0.05
C18_21	0.18	41.4	74	81.4	59.58	0.11	0.03	0.08
C18_22	0.47	44.9	74	81.7	62.57	0.30	0.09	0.21
C18_23	0.24	19.6	74	77.9	32.63	0.08	0.00	0.08
C18_24	0.20	29.0	74	80.5	48.27	0.10	0.00	0.10
C18_26	0.07	3.0	74	74.5	5.00	0.00	0.00	0.00
C18_27	0.23	4.2	74	74.7	6.97	0.02	0.00	0.02
C18_29	0.22	48.0	74	74.0	47.97	0.11	0.11	0.00
C18_3	0.16	59.6	74	97.6	99.34	0.16	0.00	0.16
C18_4	0.51	54.0	74	92.8	90.00	0.46	0.00	0.46
C18_5	0.33	24.2	74	79.1	40.31	0.13	0.00	0.13

Table A2 January 2021 Updated Existing Conditions Subcatchment SCS CN Calculations

Subcatchment Name	Area (ha)	XIMP	Base CN	Adjusted CN	Total Imp (%)	Total Imp (ha)	Road or Commercial (ha)	Residential (ha)
C18_6	0.27	53.7	74	74.0	53.67	0.15	0.15	0.00
C18_7	0.54	30.1	74	80.9	50.13	0.27	0.00	0.27
C18_8	0.64	13.3	74	76.4	22.09	0.14	0.00	0.14
C18_9	0.22	49.6	74	74.0	49.58	0.11	0.11	0.00
C19_1	0.49	72.9	74	74.0	72.92	0.36	0.36	0.00
F02_1	0.49	3.0	74	74.5	5.00	0.02	0.00	0.02
F02_2	0.38	42.2	74	74.0	42.24	0.16	0.16	0.00
F02_3	0.12	40.8	74	74.0	40.83	0.05	0.05	0.00
F02_4	3.08	3.0	74	74.5	5.00	0.15	0.00	0.15
F02_5	0.42	91.2	74	98.0	100.00	0.42	0.33	0.09
F02_6	0.15	69.1	74	74.0	69.06	0.10	0.10	0.00
F02_7	1.79	40.3	74	81.1	58.02	1.04	0.25	0.79
F02_8	1.47	47.6	74	81.9	64.86	0.95	0.32	0.63
F02_9	5.51	9.6	74	74.3	10.72	0.59	0.44	0.15
F03	5.62	44.2	74	80.1	58.26	3.27	1.29	1.98
F04_01	0.39	60.2	74	80.6	71.11	0.28	0.17	0.11
F04_02	1.72	41.6	74	80.6	57.71	0.99	0.30	0.69
F04_03	9.66	3.7	74	74.6	6.21	0.60	0.01	0.59
F04_04	0.81	50.2	74	83.4	69.66	0.57	0.17	0.40
F04_05	2.35	34.1	74	78.4	46.07	1.08	0.38	0.71
F04_06	1.03	45.2	74	82.8	65.36	0.67	0.15	0.52
F04_07	2.07	52.3	74	84.3	72.76	1.50	0.45	1.06
F04_08	0.84	40.2	74	81.1	57.98	0.48	0.11	0.37
F04_09	0.90	40.5	74	80.5	56.51	0.51	0.15	0.36
F04_10	2.91	45.0	74	82.0	63.23	1.84	0.51	1.33
F04_11	1.28	41.0	74	80.3	56.53	0.72	0.23	0.50

Table A2 January 2021 Updated Existing Conditions Subcatchment SCS CN Calculations

Subcatchment Name	Area (ha)	XIMP	Base CN	Adjusted CN	Total Imp (%)	Total Imp (ha)	Road or Commercial (ha)	Residential (ha)
F04_12	1.78	48.9	74	82.6	67.24	1.20	0.38	0.81
F04_13	3.05	46.5	74	82.2	64.78	1.98	0.59	1.39
F04_14	5.50	36.5	74	79.5	51.21	2.81	0.80	2.01
F04_15	0.36	62.7	74	88.6	85.43	0.31	0.10	0.21
F04_16	1.57	5.8	74	74.0	5.85	0.09	0.09	0.00
F04_17	0.60	48.2	74	74.7	49.75	0.30	0.28	0.02
F04_18	1.83	53.2	74	92.2	88.70	1.62	0.00	1.62
H01	0.33	60.0	74	98.0	100.00	0.33	0.00	0.33



Appendix B

Updated Existing Conditions Modelling Results



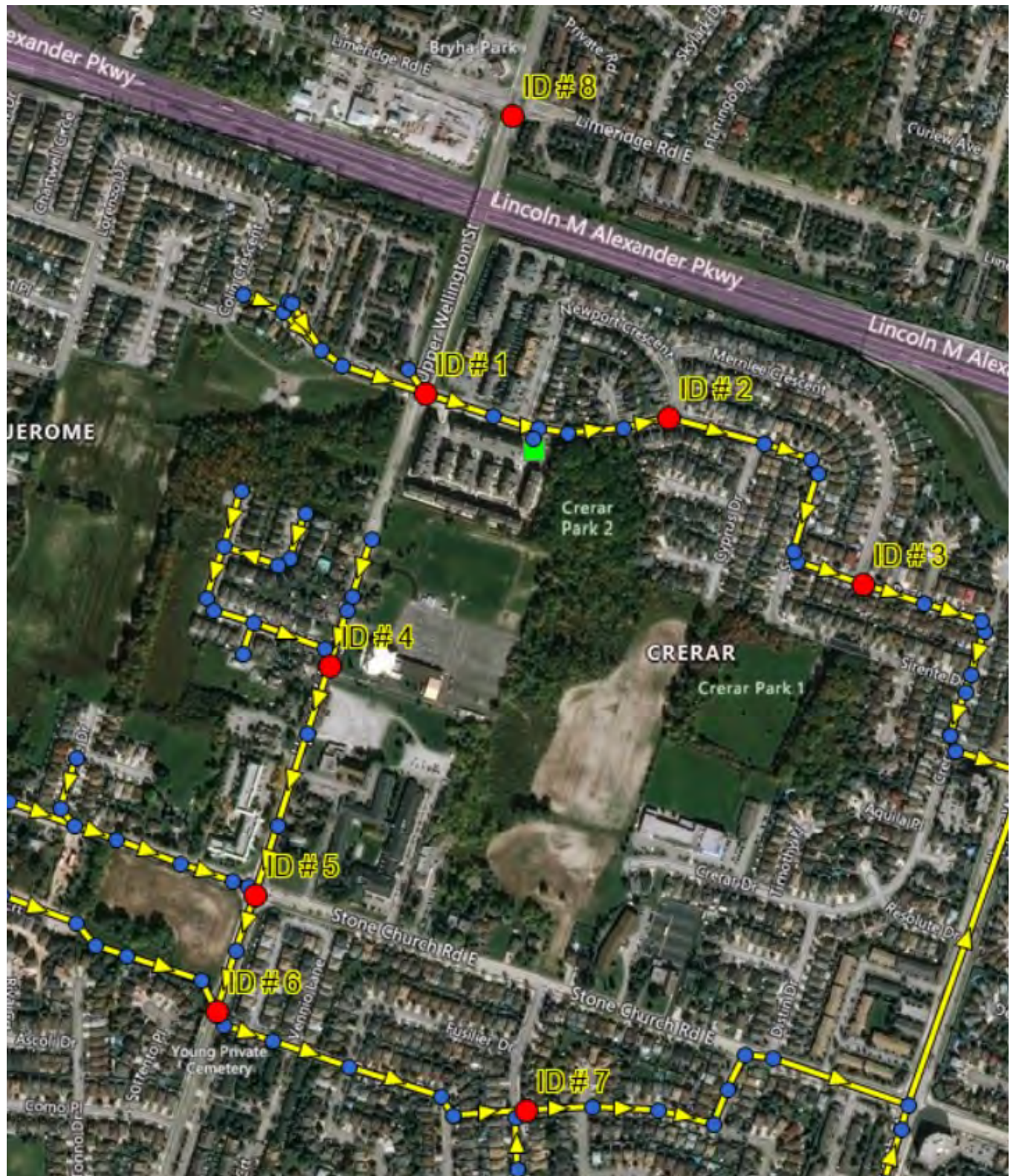


Figure B1 – Study area flow node location plan

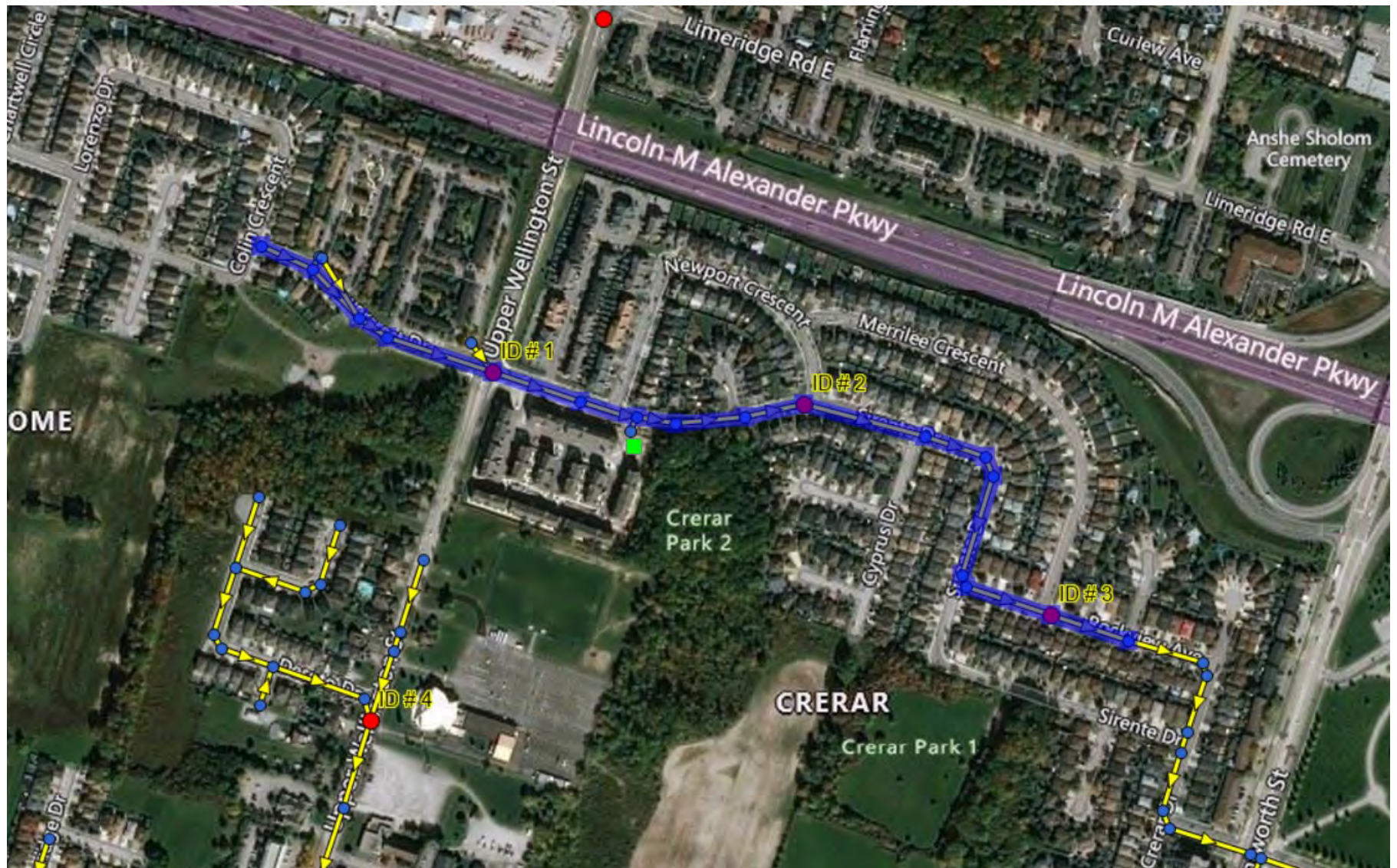


Figure B2 – Towercrest Drive/Sirente Drive storm sewer system plan of the profile path

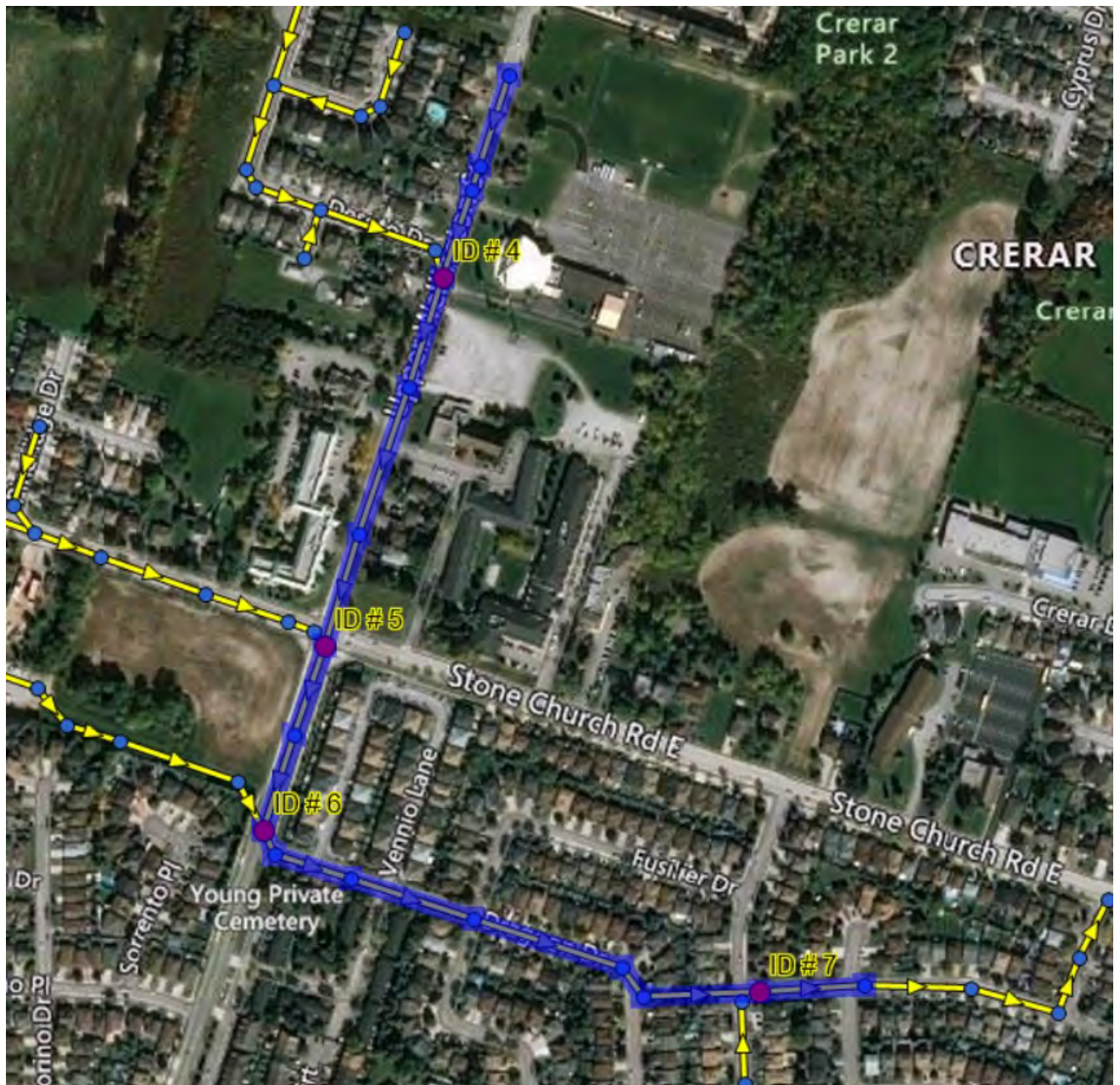


Figure B3 – Upper Wellington Street to Delancey Drive storm sewer system plan of the profile path

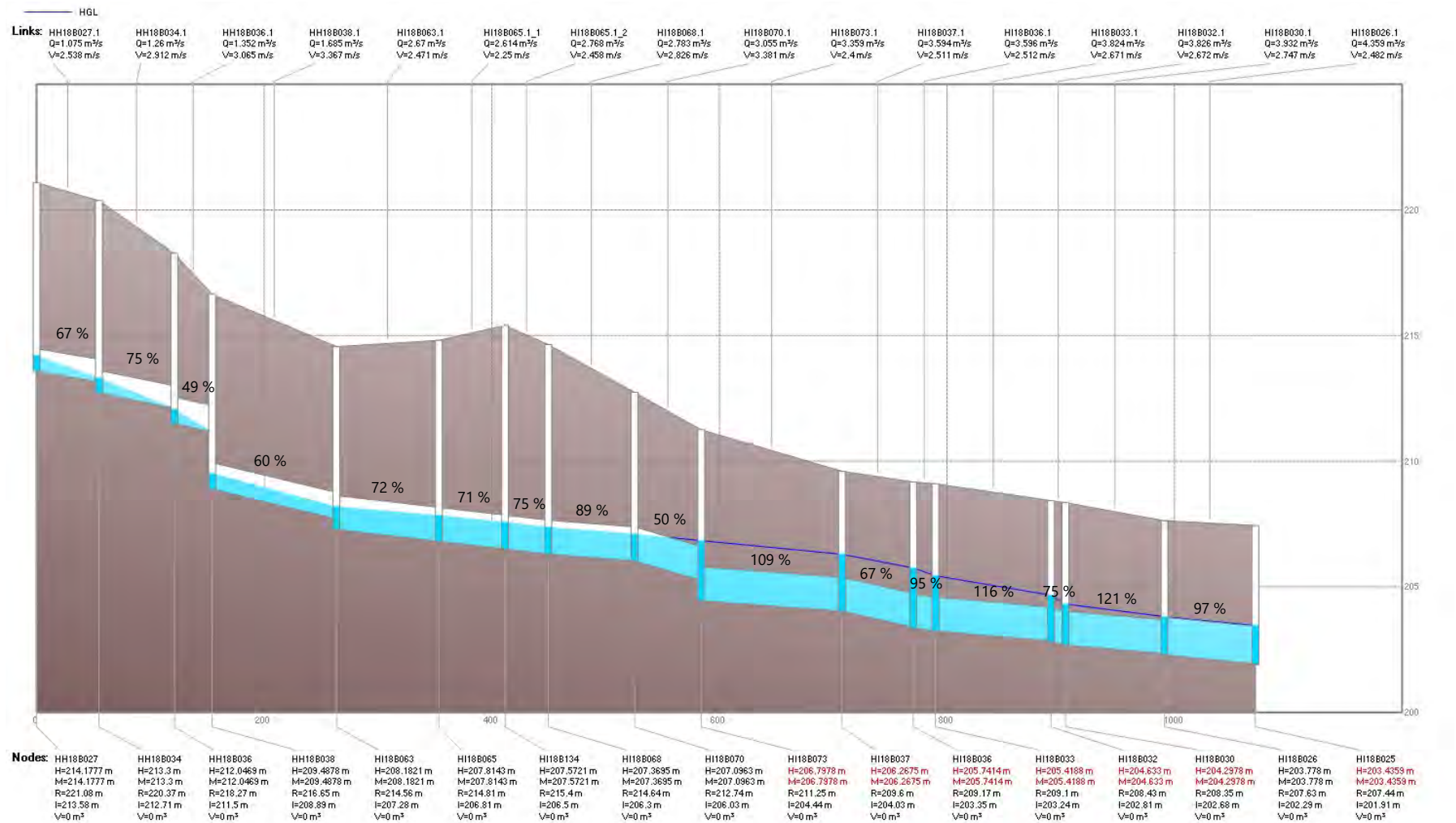


Figure B4 – 5 year 24 hr SCS storm sewer system performance profile, Towercrest Drive/Sirente Drive; maximum flow rate shown as a percentage of the full flow rate

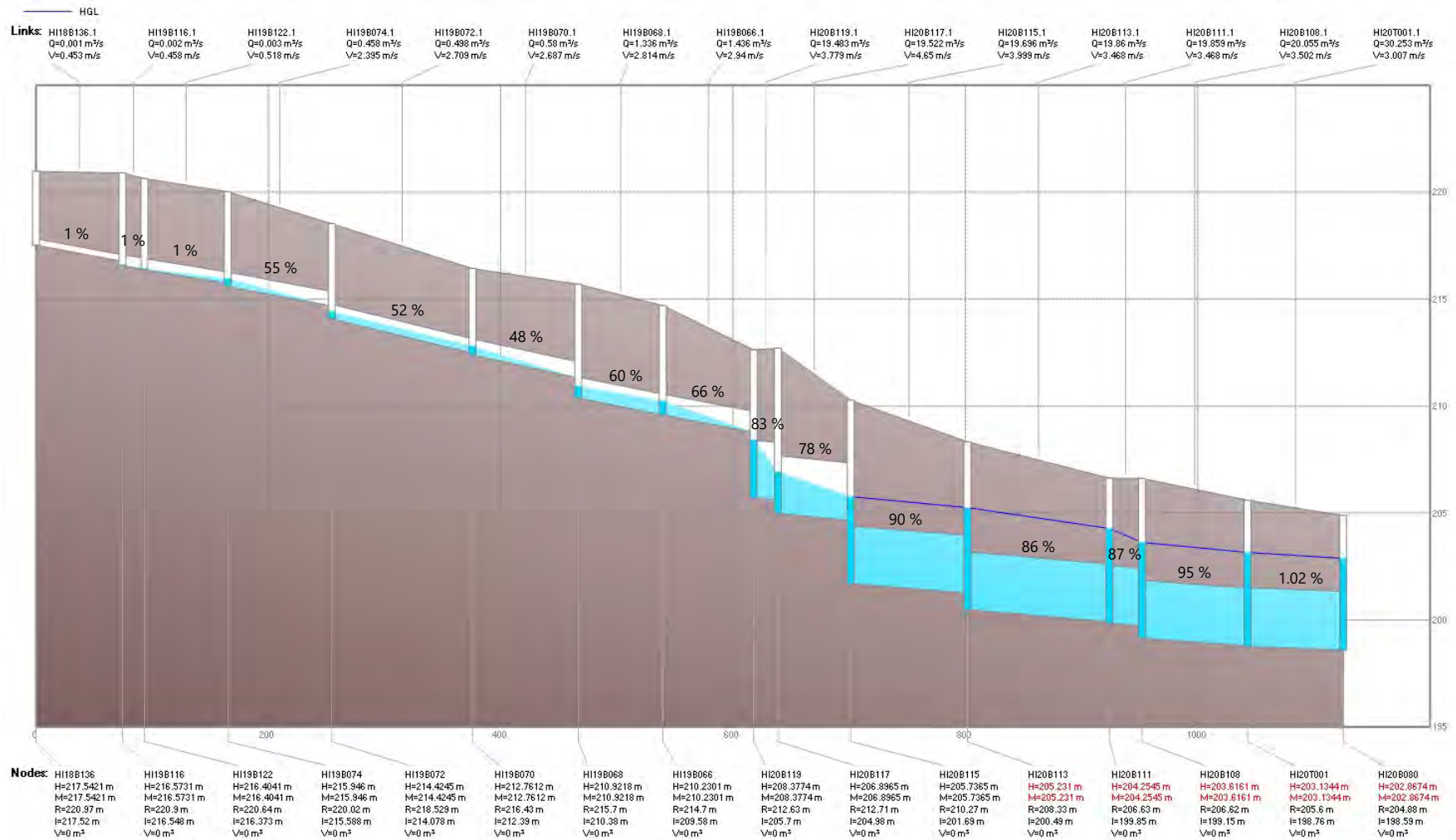


Figure B5 – 5 year 24 hr SCS storm sewer system performance profile, Upper Wellington Street to Delancey Drive; maximum flow rate shown as a percentage of the full flow rate

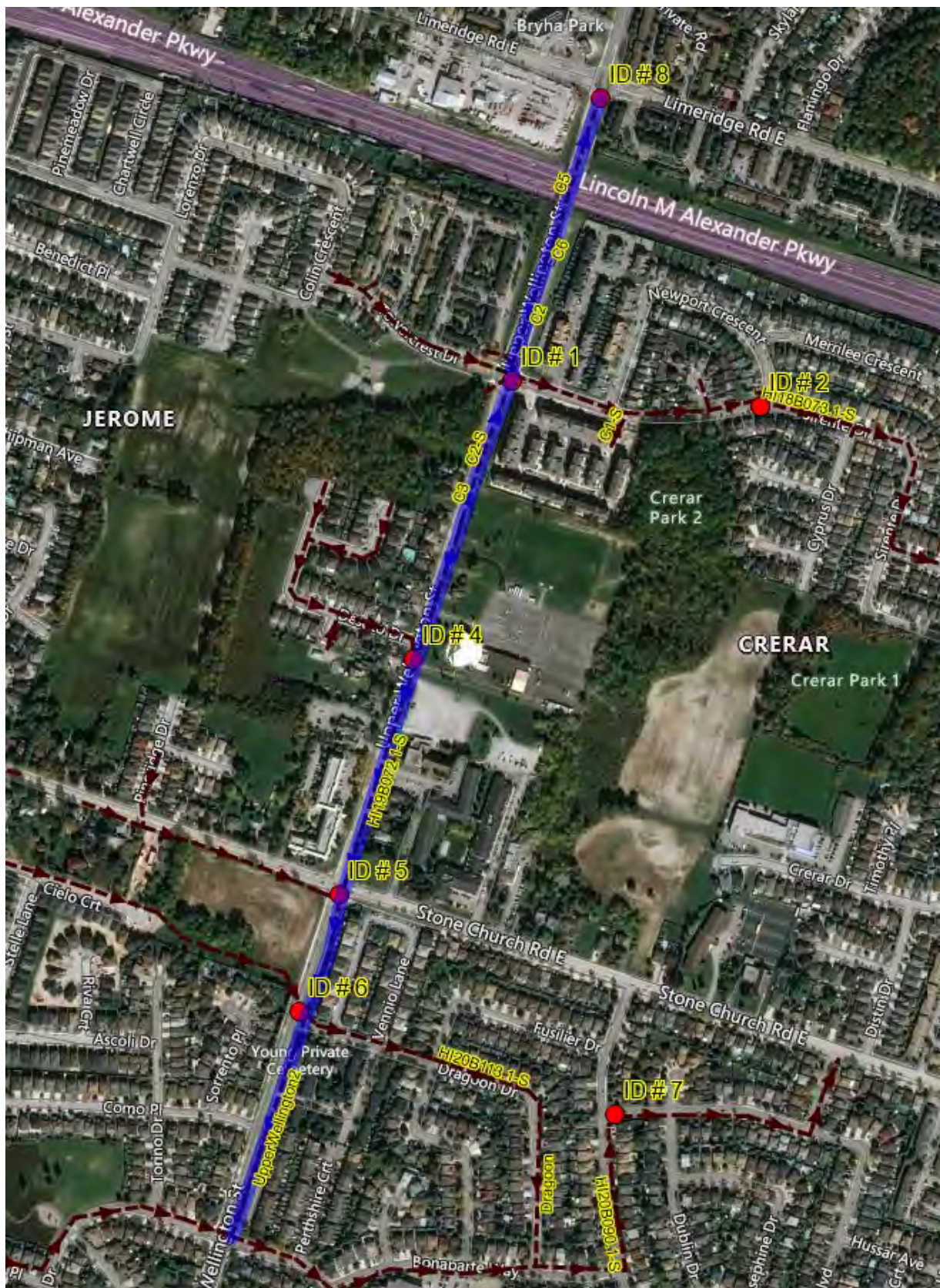


Figure B6 – Upper Wellington Street major system plan of the profile path

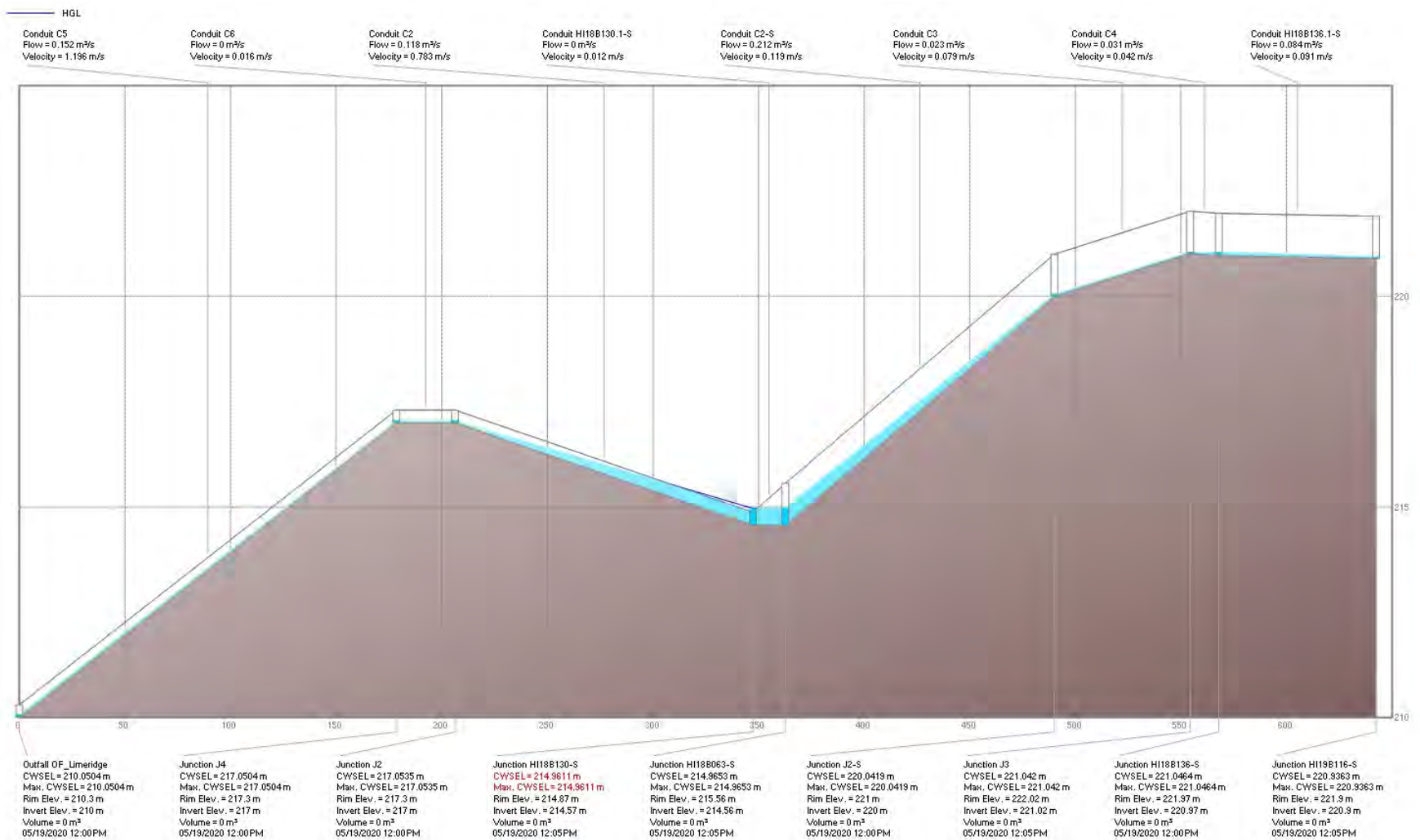


Figure B7 – 100 year 24 hr SCS major system performance profile, Upper Wellington Street from Limeridge Road East to 280 m south of Towercrest Drive/Sirente Drive

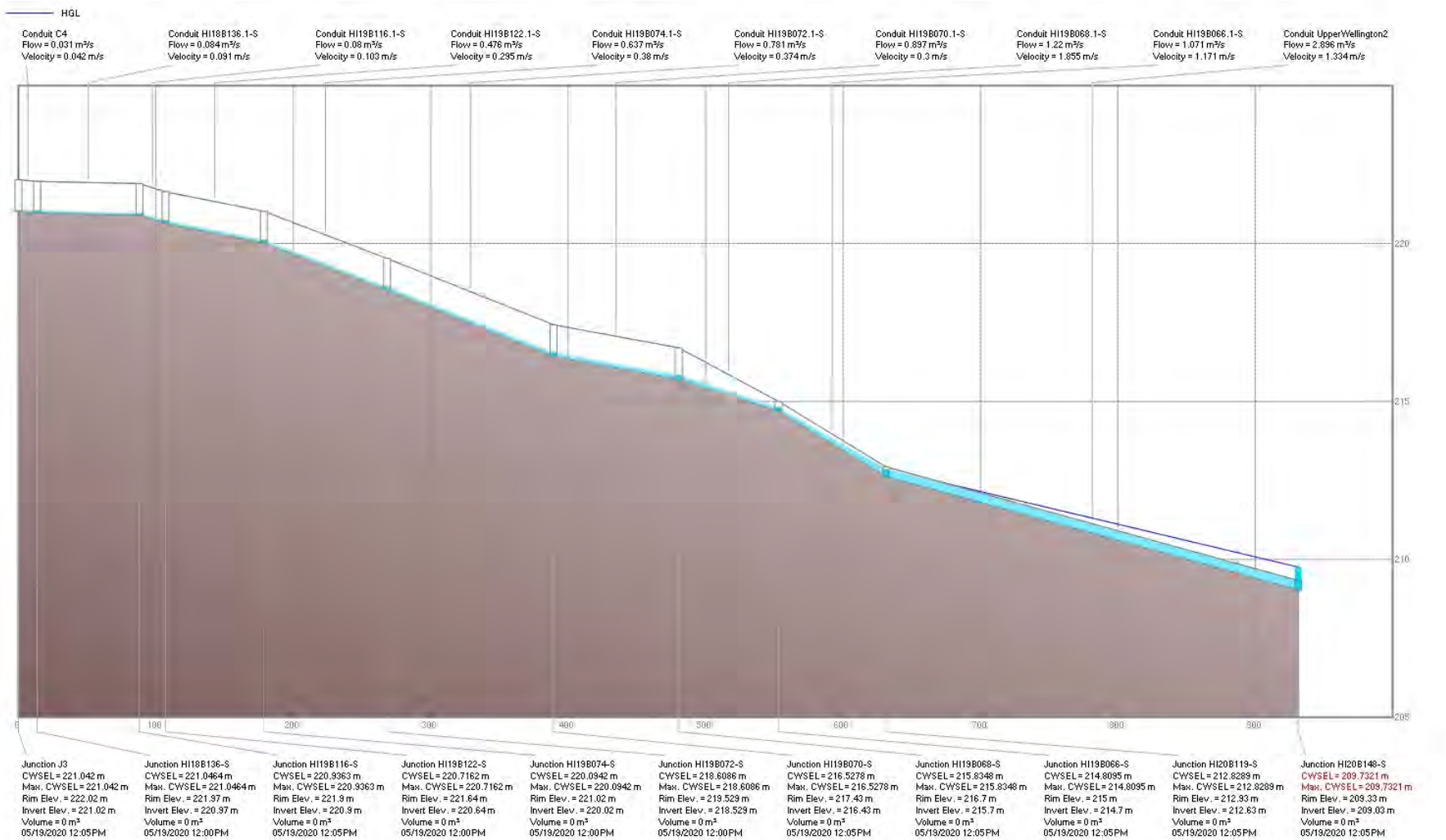


Figure B7 – 100 year 24 hr SCS major system performance profile, Upper Wellington Street from 280 m south of Towercrest Drive/Sirente Drive to Brigade Drive

Appendix C

2025 Modelling Updates (Existing Conditions)

Updated Existing Conditions PCSWMM Subcatchment Parameters (2025)																						
Name	Area (ha)	Width (m)	Flow Length (m)	Slope (%)	Imperv. (%)	N Imperv	N Perv	Dstore Imperv (mm)	Dstore Perv (mm)	Zero Imperv (%)	Subarea Routing	Percent Routed (%)	Infiltration Method	Max. Infil. Rate (mm/hr)	Min. Infil. Rate (mm/hr)	Decay Constant (1/hr)	Drying Time (days)	Max. Volume (mm)	Suction Head (mm)	Conductivity (mm/hr)	Initial Deficit (frac.)	Curve Number
F02_5	0.36	246.7	14.7	2	89.887	0.013	0.2	1	5	25	OUTLET	100	CURVE_NUMBER	3	0.5	4	7	0	3.5	0.5	0.25	96.6
H01	0.41	48.6	83.4	2	88.949	0.013	0.2	1	5	25	OUTLET	100	CURVE_NUMBER	3	0.5	4	7	0	3.5	0.5	0.25	93.3
F02_2	0.54	127.9	41.9	2	38.098	0.013	0.2	1	5	25	OUTLET	100	CURVE_NUMBER	3	0.5	4	7	0	3.5	0.5	0.25	74.0
C18_12	0.27	20.8	130.0	2	53.543	0.013	0.2	1	5	25	OUTLET	100	CURVE_NUMBER	3	0.5	4	7	0	3.5	0.5	0.25	74.6
C18_9	0.25	31.1	80.0	2	44.692	0.013	0.2	1	5	25	OUTLET	100	CURVE_NUMBER	3	0.5	4	7	0	3.5	0.5	0.25	74.2
S18_25	0.23	51.0	45.0	2	46.896	0.013	0.2	1	5	25	OUTLET	100	CURVE_NUMBER	3	0.5	4	7	0	3.5	0.5	0.25	74.2
C18_6	0.29	38.2	75.0	2	52.321	0.013	0.2	1	5	25	OUTLET	100	CURVE_NUMBER	3	0.5	4	7	0	3.5	0.5	0.25	74.3
C18_1	0.29	58.5	50.0	2	63.373	0.013	0.2	1	5	25	OUTLET	100	CURVE_NUMBER	3	0.5	4	7	0	3.5	0.5	0.25	74.1



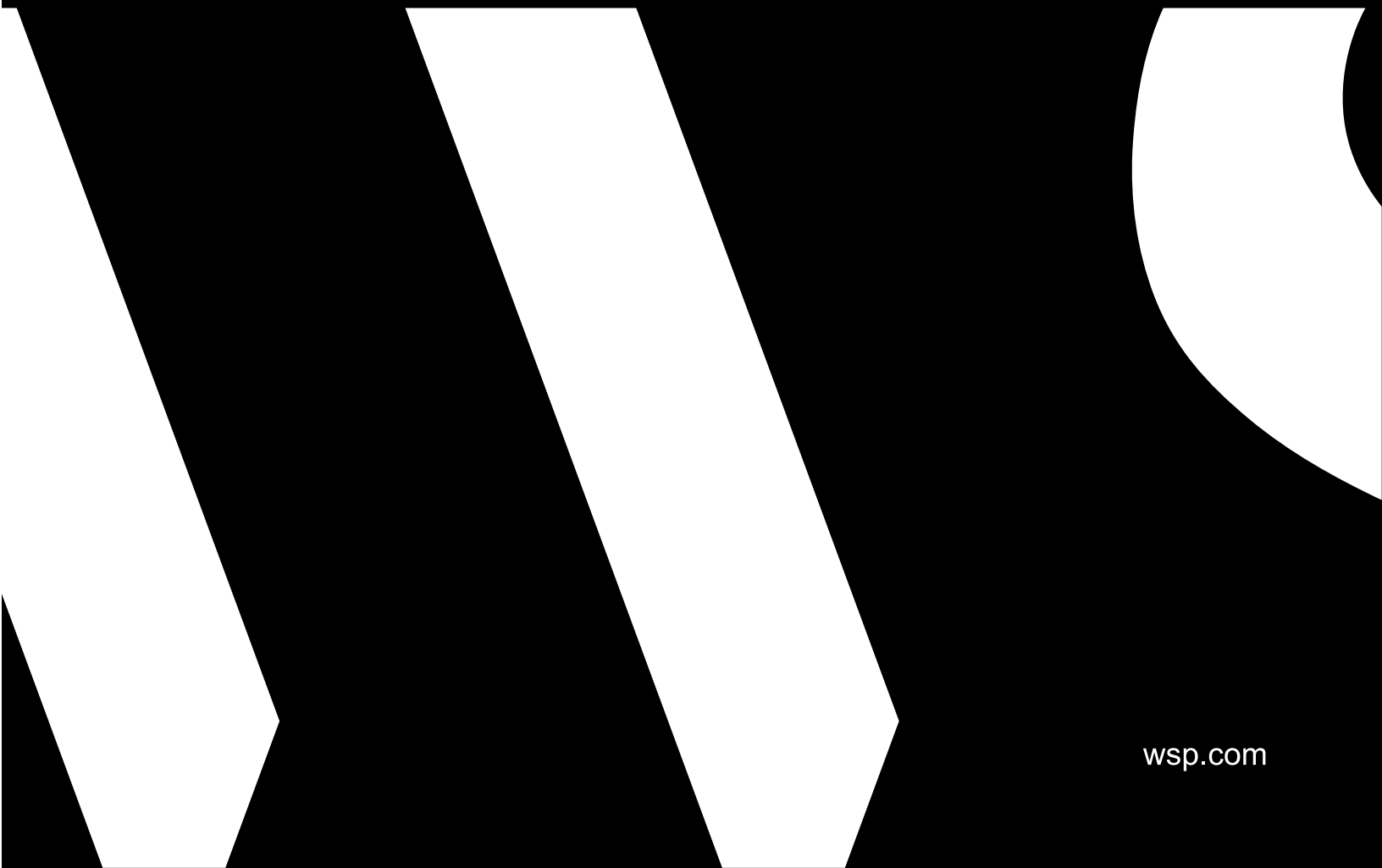
Appendix D

Future Conditions Hydrology



Future Conditions PCSWMM Subcatchment Parameters (2025)																						
Name	Area (ha)	Width (m)	Flow Length (m)	Slope (%)	Imperv. (%)	N Imperv	N Perv	Dstore Imper (mm)	Dstore Perv (mm)	Zero Imperv (%)	Subarea Routing	Percent Routed (%)	Infiltration Method	Max. Infil. Rate (mm/hr)	Min. Infil. Rate (mm/hr)	Decay Constant (1/hr)	Drying Time (days)	Max. Volume (mm)	Suction Head (mm)	Conductivity (mm/hr)	Initial Deficit (frac.)	Curve Number
F02_5	0.363	246.7	14.7	2	85.7	0.013	0.2	1	5	25	OUTLET	100	CURVE_NUMBER	3	0.5	4	7	0	3.5	0.5	0.25	94.2
H01	0.405	48.6	83.4	2	88.9	0.013	0.2	1	5	25	OUTLET	100	CURVE_NUMBER	3	0.5	4	7	0	3.5	0.5	0.25	95.2
F02_2	0.537	127.9	41.9	2	65.8	0.013	0.2	1	5	25	OUTLET	100	CURVE_NUMBER	3	0.5	4	7	0	3.5	0.5	0.25	74.0
C18-12	0.271	20.8	130.0	2	73.6	0.013	0.2	1	5	25	OUTLET	100	CURVE_NUMBER	3	0.5	4	7	0	3.5	0.5	0.25	74.6
C18_9	0.249	31.1	80.0	2	73.7	0.013	0.2	1	5	25	OUTLET	100	CURVE_NUMBER	3	0.5	4	7	0	3.5	0.5	0.25	74.2
C18_25	0.230	51.0	45.0	2	66.7	0.013	0.2	1	5	25	OUTLET	100	CURVE_NUMBER	3	0.5	4	7	0	3.5	0.5	0.25	74.2
C18_6	0.287	38.2	75.0	2	68.4	0.013	0.2	1	5	25	OUTLET	100	CURVE_NUMBER	3	0.5	4	7	0	3.5	0.5	0.25	74.3
C18_1	0.292	58.5	50.0	2	66.8	0.013	0.2	1	5	25	OUTLET	100	CURVE_NUMBER	3	0.5	4	7	0	3.5	0.5	0.25	74.1

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