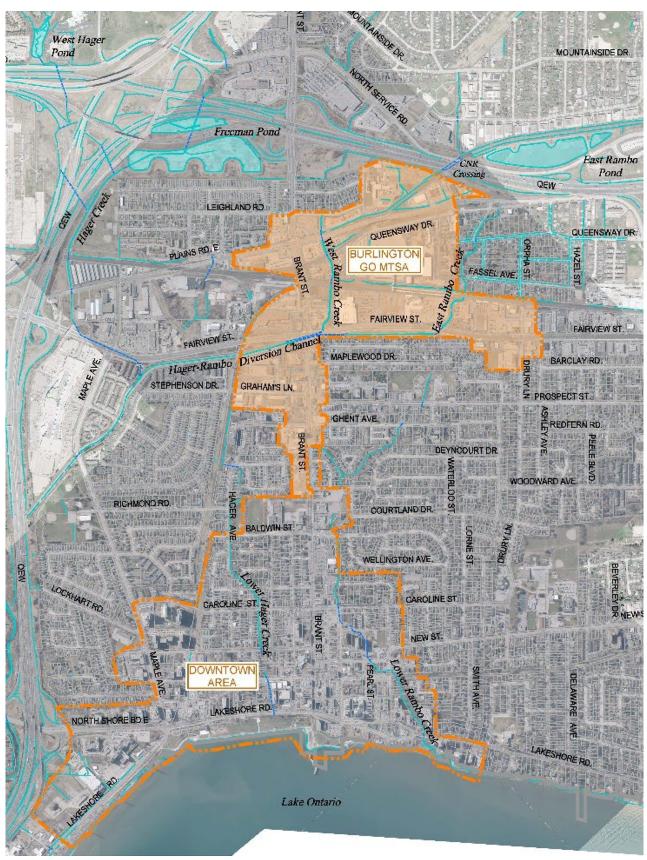
CITY OF BURLINGTON

MAJOR TRANSIT STATION AREA (MTSA) PHASE 2 FLOOD HAZARD ASSESSMENT

BURLINGTON GO AND DOWNTOWN

MARCH 06, 2023



WSP

MAJOR TRANSIT STATION AREA (MTSA) PHASE 2 FLOOD HAZARD ASSESSMENT

BURLINGTON GO AND DOWNTOWN

CITY OF BURLINGTON

PROJECT NO.: WW21011078
DATE: MARCH 06, 2023

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SIGNATURE AND DISCLAIMER

APPROVED[1] BY *(must be reviewed for technical accuracy prior to approval)*

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1 INTRODUCTION

The City of Burlington (City) has undertaken a land use planning study for three (3) Major Transit Station areas (MTSAs), previously referred to as Mobility Hubs. These are areas located around the City's GO stations including Appleby GO, Burlington GO, Aldershot GO, and also includes the Downtown area where re-development and intensification are expected.

A planning study was undertaken commencing in 2017 (led by Brook McIlroy Inc). This work included the preparation of a series of Scoped Environmental Impact Studies (EIS) for each of the four (4) areas cited above. The purpose of the Scoped EISs was to document existing environmental conditions and assess potential environmental impacts and mitigation strategies related to the expected development and re-development in these areas. SGL Planning subsequently undertook a scoped review of the Downtown area ("Taking a Closer Look at the Downtown: Themes, Principles and Land Use Concepts", October 2019).

In support of this effort, WSP E&I Canada Limited (WSP; formerly Wood Environment & Infrastructure Solutions Canada Limited) prepared a series of flood hazard and scoped stormwater management assessments for each of the three (3) MTSAs (Appleby, Burlington and Aldershot GO) and the Downtown area:

— "Flood Hazard and Scoped Stormwater Management Assessment – Burlington GO Mobility Hub and Downtown" (Wood, September 22, 2020)
— "Flood Hazard and Scoped Stormwater Management Assessment – Aldershot GO Mobility Hub" (Wood, August 9, 2021)
— "Flood Hazard and Scoped Stormwater Management Assessment – Appleby GO Mobility Hub" (Wood, August 9, 2021)

These documents summarized existing flood hazards for areas of anticipated development, and also developed preliminary stormwater management strategies, including consideration for drainage infrastructure service capacity and associated improvements, where feasible and required.

As noted above, a single report was prepared by WSP for two (2) of the four (4) areas, specifically the Downtown area and Burlington GO MTSA. These two (2) areas are located directly adjacent to each other, and although these areas are separated by the Hager-Rambo Diversion Channel floodwater under extreme conditions can potentially spill from this feature (within the Burlington GO MTSA) and thereby have the potential to impact the Downtown area; as such these areas were assessed jointly. Drawing 1 presents the boundaries of these two (2) planning areas along with the area watercourses and the existing three (3) flood control facilities (West Hager, Freeman, and East Rambo Ponds).

The analyses documented within the resulting report ("Flood Hazard and Scoped Stormwater Management Assessment – Burlington GO Mobility Hub and Downtown" Wood, September 2020) provided details and context with respect to the overall flood risk within the Burlington GO MTSA and Downtown area, and the potential implications to the proposed intensification development in these areas. This report is now generally referred to as the "Phase 1" Flood Hazard Assessment, given additional follow-on analyses requested by Conservation Halton (CH) which were deferred to a later date (i.e. referred to as Phase 2).

In conjunction with the preceding, WSP prepared the "Hager Rambo Flood Control Facilities Assessment" (September 2020) for the City of Burlington. That report is considered a companion document to this process to support the potential for crediting the attenuative effects of the flood control facilities in the area's flood management.

Although the Phase 1 Flood Hazard Assessment reporting and its recommendations to support the Official Plan Amendment (OPA) was ultimately accepted, CH noted the need for a follow-up (Phase 2) study to further assess flood hazards and stormwater management (SWM) requirements for these areas based on enhanced local detail including topographic mapping. A Terms of Reference (TOR) for the Phase 2 study was developed jointly by the City, CH and WSP (then Wood), and ultimately finalized August 6, 2021 (refer to Appendix B). The scope of work includes both hydrologic and hydraulic modelling updates, including:

- Hydrology
  - Use of the City's current (2020) rainfall intensity-duration-frequency (IDF) data
  - Confirmation of impact of spills from Roseland Creek at the QEW to the Hager-Rambo System;
  - Assessment of three (3) scenarios for the three (3) flood control facilities – fully credited, 50% low flow blockage, and fully removed.
  - Confirmation of future land use conditions and potential impacts to the Burlington GO MTSA area only (Downtown Area not considered)
- Hydraulics
  - Updated through the use of higher resolution topographic data (2018 LiDAR DTM Halton (Package B)) for both 1-dimensional and 2-dimensional hydraulic modelling
  - Updated assessment of spill flows and floodplain mapping
  - Assess impacts of filling in spill areas to off-site flood potential through three (3) different intensification/re-development scenarios

The outcomes of the current Phase 2 assessment are intended to further support an updated identification of flood hazards within the study areas and, where feasible, identify recommendations and/or requirements for flood mitigation and floodproofing to support the future land use(s) and zoning being contemplated for the MTSAs. This document is intended to provide additional input and support to the local land use planning and development effort for these areas.

The current report documents the tasks completed as part of the approved work plan and associated findings and mapping.

2 BURLINGTON GO MTSA

2.1 HYDROLOGY

2.1.1 MODELLING UPDATES

2.1.1.1 OVERVIEW

Prior to 2017 (commencement of the previous Phase 1 Study), the approved hydrologic modelling for the Hager-Rambo system was the 1997 OTTHYMO model (ref. "Technical Summary, Updated Hydrology, Hager Rambo System, Roseland Creek" Philips Planning and Engineering, 1997). As part of the July 31, 2017 Urban Area Flood Vulnerability, Prioritization and Mitigation Study for the City of Burlington (in response to the August 4, 2014 storm event), WSP updated the 1997 OTTHYMO model to a SWMHYMO format. Furthermore, as part of the previous Phase 1 Study (September 2020), WSP also undertook a more detailed review and refinement of the SWMHYMO version of the model, including making revisions and updates as required, to represent current land use and infrastructure conditions more accurately. Reference is made to that report for further details.

The SWMHYMO model was initially updated as part of the current Phase 2 study to remove the channel routing element (NHYD 583, previously Node H; refer to Drawing 4) which represents 600 m of the West Hager Creek watercourse between the outlet of the Freeman Pond and the confluence with the Hager-Rambo diversion channel. Node H was then represented by the ADDHYD 582, with no attenuation/routing between the Freeman Pond outlet and the confluence with the diversion channel. The reason for this update was that this routing element was determined to generate an instability in the outputs by reducing the routed flow by approximately 15% and causing a 1-hour shift in the peak flow during the 100-year storm event, which was not considered reasonable nor representative of field conditions. This was however further addressed as part of subsequent modelling updates.

The base hydrologic modelling generally reflects existing land use conditions and is consistent with the base parameterization noted in the Phase 1 study report (refer to that report for further details). A further discussion of future land use conditions is provided in subsequent sections of this report.

In addition to the preceding, an assessment of the impacts of spill flows from the adjacent Roseland Creek watershed has been considered as part of the current (Phase 2) study. A spill at the QEW from Roseland Creek into the Hager-Rambo system was previously identified in the approved modelling for Roseland Creek. A new 2-dimensional (2D) hydraulic model (described in subsequent sections) has been developed to assess this spill flow. The total Roseland Creek hydrograph at the QEW (Node R11.2; refer to Drawings 3 and 4) has been extracted from the hydrologic modelling and included in the 2D hydraulic modelling to determine the impact of the spill on the Upper Hager\Rambo system, including potential spill along and across the QEW.

The approved Roseland Creek OTTHYMO model was originally prepared in 1997 and updated to SWMHYMO in 2009 as a part of the Roseland Creek Flood Control Class Environmental Assessment prepared for the City of Burlington by Philips Engineering Ltd. (February 2009). The Roseland Creek 2009 Class EA considered a potential future stormwater management facility north of the QEW referred to as the 'Leon's Pond'. This recommendation has not yet been implemented and as such, the existing conditions (without Leon's Pond) modelling for the 2009 Class EA has been used as the baseline condition for flow generation.

Drawing 3 presents the drainage area boundaries for the Upper Hager, Upper Rambo, and Roseland Creek systems, and also depicts key hydrologic nodes (locations) of interest based on the flows generated from the updated hydrologic modelling. Drawing 4 presents the updated hydrologic modelling schematic, based on the previously completed studies.

Partway through the Phase 2 Study, Conservation Halton (CH) staff recommended that WSP consider converting the SWMHYMO based model to Visual OTTHYMO (VO). This recommendation was considered reasonable and advantageous as the VO platform is generally accepted to be a more modern modelling platform and results are generally consistent as each software applies the HYMO-based engine (i.e., same unit hydrographs for rural and urban land uses). Several of the key advantages of using VO versus SWMHYMO are summarized below:

- VO has a more functional graphical user interface than SWMHYMO's text-based interface,
- VO does not have the same limitation concerning the number of hydrographs which can be preserved at any one time. SWMHYMO is limited to preserving only (10) hydrographs which renders model setup more complex for larger watersheds and makes modifications more challenging,
- VO allows results to be extracted and compared within the software, SWMHYMO requires modification of the model to extract results and comparison is typically done separate from the model.
- VO is more commonly applied within the industry in Ontario and the software is being actively maintained and updated by the software developer,

As part of the collaborative approach to this study as discussed on April 20, 2022, and May 5, 2022, CH staff prepared conceptual models in VO (Version 6.2) for the Study Team's consideration. Note that the previously noted modelling instability at a route channel element (NHYD 583) was resolved and thus maintained in the VO model.

The converted VO model by CH has been reviewed by WSP for consistency with the parent model to verify the conversion and model updates. A comparison of the base VO model to the base SWMHYMO model is discussed further in Section 2.1.1.2.

2.1.1.2 *SCOPED VALIDATION*

Although SWMHYMO and VO generally incorporate similar unit hydrograph routines and other calculation methodologies, a scoped model validation has been considered warranted to confirm that any resulting modelling differences between the two (2) models are minor and that the results are generally consistent.

Comparison nodes have focused on areas of interest outside of the Burlington GO MTSA, given that the current VO model does not include routing of flows through this area. Subcatchment flows are generated for this area separately, and then input into the HEC-RAS 2-dimensional (2D) model for routing given the complex spill mechanics in this area (this is discussed further in subsequent sections). Results for selected nodes are presented in Tables 2.1.1 (Peak Flows) and 2.1.2 (Runoff Volume).

Note the following VO results do not include additional updates related to the area Flood Control Facilities (consideration of debris blockage at West Hager Facility) and hydraulic flow routing (2D modelling) as discussed in subsequent sections. Results are presented for the purposes of the comparison to SWMHYMO only.

Table 2.1.1. Comparison of Peak Flow Results between SWMHYMO and VO (m^3/s)

LOCATION	SWMHYMO		VO		DIFFERENCE	
	100Y	REGIONAL 12HR	100Y	REGIONAL 12HR	100Y	REGIONAL 12HR
East Rambo Pond Inlet (Node Q)	77.2	64.0	67.0	54.7	-10.2 (-13%)	-9.3 (-5%)
West Rambo Creek at QEW (Node P)	18.2	11.9	17.0	11.8	-1.2 (-7%)	-0.1 (0%)
Freeman Pond Outlet (Node G1)	18.0	38.1	18.2	43.2	+0.2 (+1%)	+5.1 (+3%)
West Hager at CNR (Node H3)	33.5	57.1	34.9	63.7	+1.4 (+4%)	+6.6 (+3%)
Freeman / West Hager Conf. (Node H)	36.3	58.6	37.7	65.6	+1.4 (+4%)	+7.0 (+4%)

Table 2.1.2. Comparison of Simulated Runoff Volume Results between SWMHYMO and VO (mm)

LOCATION	SWHMYMO		VO		DIFFERENCE	
	100Y	REGIONAL 12HR	100Y	REGIONAL 12HR	100Y	REGIONAL 12HR
East Rambo Pond Inlet (Node Q)	84.5	194.5	86.4	195.0	+1.9 (+2%)	+0.5 (+0.3%)
West Rambo Creek at QEW (Node P)	94.7	200.6	94.7	200.0	0 (0%)	-0.6 (-0.3%)
Freeman Pond Outlet (Node G1)	85.3	197.2	85.5	196.1	+0.2 (+0.2%)	-1.1 (-0.6%)
West Hager at CNR (Node H3)	84.8	196.9	84.7	195.4	-0.1 (-0.1%)	-1.5 (-0.8%)
Freeman / West Hager Conf. (Node H)	84.8	196.8	84.7	195.4	-0.1 (-0.1%)	-1.4 (-0.7%)

The results for runoff volume (Table 2.1.2) indicate nominal differences, typically less than 1% with one (1) exception. For peak flow (Table 2.1.2), the results indicate a greater difference, but typically 5% or less, with two (2) exceptions. In all cases, the differences are considered nominal and acceptable.

Overall, the VO model conversion was deemed reasonable, and the updated VO modelling has been used for the generation of estimated peak flows for the Hager-Rambo and Roseland Creek systems within the Burlington GO MTSA

2.1.2 RAINFALL AND INTENSITY-DURATION-FREQUENCY (IDF) DATA

The City of Burlington's previously approved rainfall Intensity-Duration-Frequency (IDF) parameters were sourced from the City's 1994 Storm Drainage Design Manual (as prepared by Philips Planning and Engineering Limited, now WSP). These IDF values were based on the Hamilton RBG rain gauge for a period of record from 1964 to 1990 (26 years). These values were approved for use by the City in 1999.

In 2004, WSP (then Philips Engineering) also completed an IDF review for the City of Burlington to update the available dataset from 1962 to 1996 (35 years). This updated dataset was however never formally approved for use by the City.

The Roseland Creek Flood Control Class EA completed in 2009 (as prepared by Philips Planning and Engineering Limited, now WSP) applied the 2004 updated IDF curve for generating the 24-hour SCS Type II rainfall distribution used in the assessment documented in the report. The proposed flood control facility (Leon's Pond) was sized using the 1982 scaled storm (100-year) and the SCS Type II distribution as noted in the report to be provided for information purposes only.

The City engaged WSP (then Amec Foster Wheeler) in 2014 to complete the Urban-Area Flood Vulnerability, Prioritization and Mitigation Study (Amec Foster Wheeler, July 2017). The report documents the City-wide hydrologic and hydraulic assessment using existing modelling tools to assess flood vulnerable areas and develop mitigation strategies. That assessment used (then) current IDF data from the Hamilton RBG station from 1964 to 2007 for the assessment of frequency storms.

As noted earlier, WSP was subsequently retained by the City (through Brook McIlroy) in 2017 to complete a series of Flood Hazard and Scoped Stormwater Management Assessment reports for the three (3) Mobility Hub areas (Aldershot, Burlington and Appleby GO stations) and the Downtown area. The 24-hour SCS Type II design storm model files used in these studies ("Phase 1") were generated using the previously noted 2004 IDF curve data (data were included in the appendices of the Mobility Hub reports).

The City of Burlington recently (2020) updated its rainfall Intensity-Duration-Frequency (IDF) curves (ref. Stormwater Management Design Guidelines, City of Burlington, May 2020). City staff reviewed climate change adjusted rainfall patterns using the publicly available IDFCC tool, and determined that based on these forecasted values, an average increase of 15% to existing IDF values (i.e., those from 1999) was considered appropriate to reflect the potential impacts of climate change. The depth of rainfall associated with the updated 2020 IDF curves for the 100-year storm event are summarized in Table 2.1.3, along with a comparison to the previously generated (but not formally adopted) 2004 IDF update, which was applied as part of the Phase 1 Study (2020).

Table 2.1.3. Comparison of City of Burlington 100-Year Event Rainfall Depths (mm)

DURATION (hours)	2004 UPDATE (mm)	2020 UPDATE (mm)	DIFFERENCE (mm)	DIFFERENCE (%)
1	42.5 (1)	53.5	+11.0	+25.9
2	60.8 (1)	65.0	+4.2	+6.9
6	92.4 (2)	86.3	-6.1	-6.6
12	103.6 (2)	102.4	-1.2	-1.2
24	122.9 (2)	121.1	-1.8	-1.5

1. Values from City of Burlington IDF Relationships and Design Storms Memorandum by Philips Engineering, December 10, 2004
2. Values from SCS design storm modelling files accompanying the December 10, 2004 Memorandum prepared by Philips Engineering.

The results presented in Table 2.1.3 indicate that the updated 2020 IDF generated rainfall depths for the 100-year storm event are generally consistent with the 2004 IDF for the 12-hour and 24-hour durations, however the 2020 update resulted in a greater increase in rainfall depth for the 1 and 2-hour durations.

The 24-hour duration design storm was determined to produce the greatest peak flows in the Hager-Rambo SWMHYMO model based on the sensitivity analysis completed as part of the Mobility Hubs Phase 1 Study and as such has been advanced in this Phase 2 study (since it is understood that the VO modelling now employed should be generally consistent). Based on the preceding results, it should be noted that the 2020 Update IDF generated 24 -hour rainfall depth is marginally reduced by 1.5 mm (1.2% of the total depth) as compared to the 2004 IDF employed in the Phase 1 study. This is generally considered nominal, however.

A scoped sensitivity check has been completed by WSP using the current 2020 IDF. This effort confirmed that the 24-hour SCS Type II distribution remains the governing design storm distribution (i.e., generates the highest peak flows). This is consistent with the Phase 1 study findings, and also with the results for the Roseland Creek modelling files (as per the 2009 study). As such, this design storm distribution has been maintained for the estimation of frequency flows for this Phase 2 study.

2.1.3 FUTURE CONDITIONS MTSA LAND USE ASSESSMENT

2.1.3.1 IMPERVIOUSNESS CALCULATIONS

The Study TOR (ref. Item 1d - as included in Appendix B) includes the requirement for a hydrologic assessment of the future land use condition contemplated for the Burlington GO MTSA area only. The assessment is to consider the highest imperviousness which can reasonably be expected to occur under current zoning and/or Official Plan specifications. As noted in the Phase 1 Study (September 2020), the Burlington GO MTSA area is already almost completely built out (refer to Drawing 1 for aerial photography).

The estimated existing land use initially applied for hydrologic modelling parameterization of imperviousness has also assumed a consistently high imperviousness, as per Drawing 2. This land use is also considered reasonably consistent with any expected changes under future conditions, as per the developed precinct plan (refer to Appendix A). The draft precinct plan (as provided by the City to WSP) also includes several new potential park sites (i.e., future greenspace/pervious areas), although the exact numbers and sizes are not specified. These potential park sites have conservatively been omitted from the calculations of imperviousness given uncertainty regarding their potential sizes and locations.

Further, any re-development within the study area would need to comply with the City's Stormwater Management Design Guidelines (2020), which require post to pre-peak flow on-site control or greater for all storm events, which would further be expected to mitigate the runoff impacts associated with any minor localized changes in imperviousness.

Notwithstanding, as discussed with CH and City staff (ref. meeting of November 19, 2021), CH has requested a comparison of the calculated imperviousness for the various land use scenarios. This requirement was reiterated in CH's comments of May 19, 2022 and considered in the "Proposed Approach to Finalize Reports" document of June 16, 2022 (refer to Appendix B). The three (3) scenarios to be assessed are as follows:

a) "Modelled Existing" (Existing Conditions – Imperviousness based on Estimated Land Use)

"Modelled Existing" conditions imperviousness is based on the assumed land use presented in Drawing 2, and using the assumed impervious coverages presented in the Phase 1 (September 2020) study. These values are reproduced in Table 2.1.4.

b) **"Actual Existing" (Existing Conditions – Measured Imperviousness)**

"Actual Existing" imperviousness coverage has been calculated based on a review of current (at the time of this report) aerial photography. For the purposes of the assessment, railway corridors have been considered to be pervious (due to the typical granular rail bedding material); this is consistent with the land use assumptions presented in Drawing 2. Pervious and impervious areas have been manually digitized in GIS accordingly from the aerial photography. A graphical presentation of the estimated actual impervious/pervious coverage is presented in Figure 2.1.1 (pervious areas in purple, rail corridors in green, all other areas deemed to be impervious).

c) "Future Intensification" (Assumed Full Build Out of Burlington GO MTSA)

Contemplated future intensification of the Burlington GO MTSA has also been assessed, with imperviousness estimated based on coverage ratios for differing land uses in the precinct plans supplied by the City (refer to Appendix A). A graphical presentation of the area of interest is presented in Figure 2.1.2. Impervious coverages have not been assigned to the designated land uses as part of the planning effort. Given this uncertainty, and in order to consider a potential full build out assessment of the area, potential development areas have been assumed to be 90% impervious uniformly in areas of contemplated future development; consistent with the approach presented in the "Proposed Approach to Finalize Reports" (June 16, 2022). Natural open space and rail corridors have been assumed a nominal 10% imperviousness. Assumptions are summarized in Table 2.1.5; note that total and directly connected imperviousness have been assumed to be identical given the high assumed values. Resulting imperviousness for the three (3) land use scenarios are presented in Table 2.1.6.

Table 2.1.4. Phase 1 Estimated "Modelled Existing" Imperviousness (Burlington GO MTSA)

LAND USE CLASSIFICATION	TOTAL IMPERVIOUSNESS (%)	DIRECTLY CONNECTED IMPERVIOUSNESS (%)
Apartment Buildings	60%	60%
High Density Detached	60%	30%
Low Density Detached	40%	20%
Downtown High Density	60%	60%
Downtown Low Density Residential	35%	15%
High Impervious	90%	90%
Institutional	60%	60%
Park/Corridor	10%	10%
Semi Detached and Town Homes	60%	60%
Roadways	90%	90%

Table 2.1.5. Estimated "Future Intensification" Imperviousness (Burlington GO MTSA)

PRECINCT PLAN LAND USE	EQUIVALENT LAND USE	TOTAL AND DIRECTLY CONNECTED IMPERVIOUSNESS (%)
Burlington GO Central	High Impervious	90%
Corridor	Roadways	90%
Drury Node	High Impervious	90%
Fairview Frequent Transit Corridor	High Impervious	90%
Leighland Node	High Impervious	90%
Mid-Rise Residential	Apartment Buildings	90%
Natural Open Space	Park/Corridor	10%
Queensway Main Street	High Density Detached	90%
Rail Corridor	Park/Corridor	10%
Residential High Density	High Density Detached	90%
Residential Medium Density	Semi Detached and Town Homes	90%
Upper Brant	High Impervious	90%
Urban Employment	High Impervious	90%

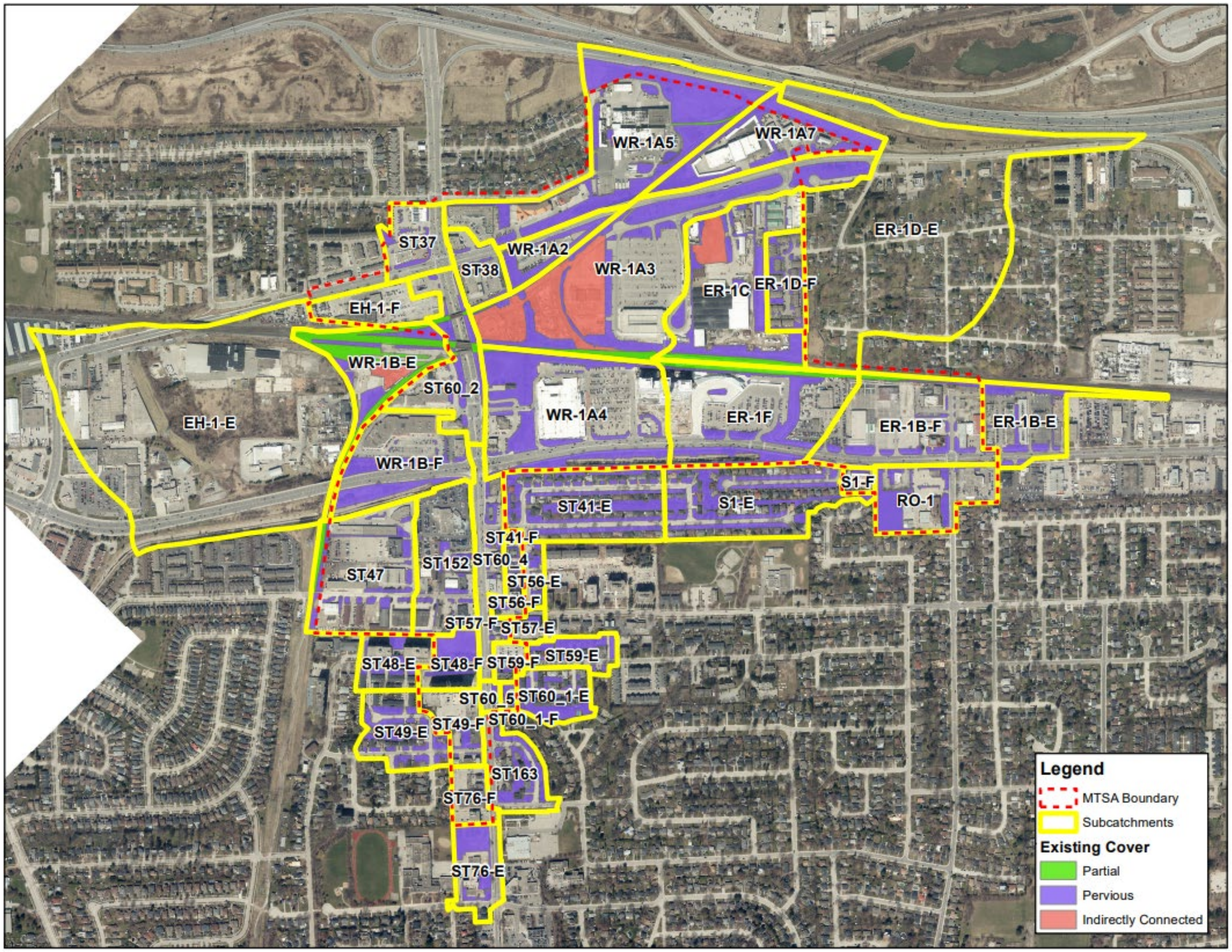

Figure 2.1.1. Existing Pervious Areas within Burlington GO MTSA (Hager-Rambo Watershed)

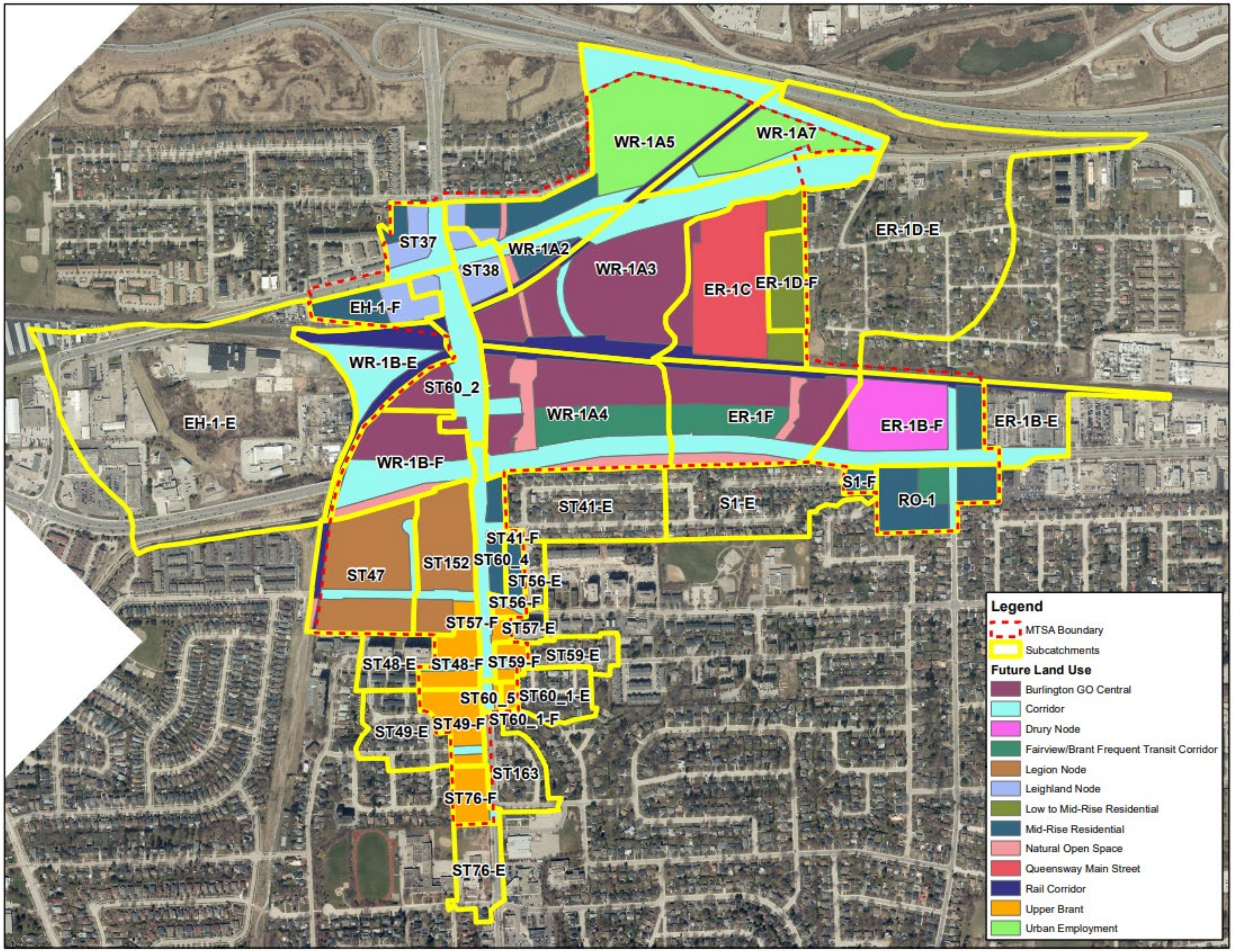

Figure 2.1.2. Future Precinct Areas within Burlington GO MTSA (Hager-Rambo Watershed)

Table 2.1.6. Comparison of Estimated Imperviousness for Burlington GO MTSA

WATERSHED	SUBCATCHMENT	AREA (ha)	IMPERVIOUSNESS (%)		
			MODELLED EXISTING	ACTUAL EXISTING[1]	FUTURE INTENSIFICATION[2]
West Rambo Creek	WR-1A7	4.66	81.4%	49.1%	82.1%
	WR-1A5	11.56	82.4%	56.2%	85.6%
	WR-1A2	1.79	71.1%	53.8%	72.1%
	WR-1A3	13.80	77.5%	67.8%	79.4%
	WR-1A4	10.34	70.1%	65.8%	74.3%
	WR-1B-F	4.99	69.3%	69.3%	79.1%
	TOTAL	**47.13**	**76.4%**	**62.3%**	**79.8%**
East Rambo Creek	ER-1C	8.08	83.8%	66.1%	83.8%
	ER-1D-F	1.85	61.5%	63.3%	90.0%
	ER-1B-E	3.34	78.1%	72.5%	78.1%[3]
	ER-1B-F	6.59	85.3%	80.9%	85.3%
	ER-1F	8.21	75.5%	70.6%	72.9%
	TOTAL	**28.06**	**79.6%**	**71.4%**	**80.7%**
Lower Rambo Creek	ST37	2.16	89.7%	82.4%	90.0%
	ST38	1.59	84.0%	84.1%	84.0%
	ST60_2	4.04	85.7%	72.3%	87.0%
	ST60_4	2.85	89.9%	80.4%	90.0%
	ST56-F	0.69	63.8%	64.9%	90.0%
	ST57-F	0.29	90.0%	80.3%	90.0%
	ST152	4.22	90.0%	87.0%	90.0%
	ST47	6.12	83.5%	81.6%	83.5%
	ST48-F	1.48	90.0%	47.6%[4]	90.0%
	ST49-F	1.71	90.0%	80.1%	90.0%
	ST76-F	1.11	89.7%	94.8%	90.0%
	ST59-F	0.55	90.0%	81.9%	90.0%
	ST60_1-F	0.23	89.9%	55.2%[5]	90.0%
	ST60_5	0.31	90.0%	78.3%	90.0%
	TOTAL	**27.35**	**86.9%**	**79.1%**	**87.8%**
East Hager Creek	EH-1-F	2.01	88.1%	96.0%	96.0%[3]
ALL	**TOTAL**	**104.55**	**80.2%**	**69.8%**	**82.4%**

1. **Rail corridors assumed to be fully pervious.**
2. **Impervious coverage based on Table 2.1.5.**
3. **Impervious for future condition set to existing condition impervious to avoid reducing the impervious coverage in the future condition.**
4. **Re-development site near Ghent Avenue; indicated as pervious cover under existing conditions.**
5. **Subcatchment includes a large amount of pervious land use.**

The results in Table 2.1.6 indicate that the modelled existing impervious coverage is clearly higher as compared to the actual existing estimated value (i.e. as estimated\measured from aerial photography). This may suggest that the peak flows generated by the baseline model (existing conditions) may be higher than actual existing conditions, given the difference in impervious coverage. The modelled imperviousness is an average of 80%, whereas the actual (measured) existing impervious coverage is approximately 70%. This likely reflects the use of simplified assumptions on land use coverage in Drawing 2 (i.e., some larger pervious areas are included as part of the overall "High Impervious" land classification, such as the parcel immediately west of the CNR line, south of the QEW).

As would be expected, the future intensification scenario yields the highest overall imperviousness but is only slightly greater than the impervious coverage generated by the "modelled existing" scenario (between 1 and 3% difference overall).

It should be noted that the Future Intensification Scenario does not include the additional greenspace proposed through the various potential park nodes, which would tend to reduce imperviousness further. In addition, the preceding does not consider the potential benefits of on-site stormwater management (SWM) controls applied to re-developing areas; this is assessed further in Section 2.1.4.2.

2.1.3.2 *SWM SIZING AND FLOW ASSESSMENT*

The original study TOR (refer to Appendix B) noted the need to complete a hydrologic assessment to verify that there will be no increased peak flows to the Hager-Rambo system due to potential future intensification, by assessing the expected on-site stormwater management (SWM) quantity controls (2 through 100-year storms), as per City of Burlington requirements (i.e. as per the City's current Stormwater Management Design Guidelines of May 2020). WSP has previously noted that the study area is already largely developed, and that the modelled existing land use assumes higher impervious coverage as compared to the actual existing land coverage. WSP further notes that given the magnitude of the upstream drainage areas of the Hager-Rambo watershed, relative to the study area and associated hydrograph timing effects, that there may not be a particular hydrologic sensitivity to the proposed land use changes in this area (hydrograph peaks are unlikely to be synchronous). However, this has required further assessment to validate and confirm this consideration.

Notwithstanding, as per the "Proposed Approach to Finalize Reports" document (June 16, 2022; refer to Appendix B) an assessment of the SWM strategy's effectiveness has been undertaken.

The "actual existing" land use modelling described in the preceding section has been used as the basis for the simulation of subcatchment based peak flow targets. Based on the City of Burlington's current (May 2020) Stormwater Design Guidelines, on-site quantity controls for subcatchments within the Burlington GO MTSA would be developed using two (2) different approaches:

- For subcatchments which may potentially outlet directly to a <u>watercourse receiver</u>, post to pre peak flow control would be required (i.e., using the "actual existing" scenario) for the 2 through 100-year storm events
    - Applies to Subcatchments ER-1C, ER-1D-F, WR-1A2, WR-1A3, WR-1A5, WR-1A7, and WR-1B-F
- For subcatchments potentially outletting to a <u>storm sewer system receiver</u>, 100-year to 5-year overcontrol is required for peak flows, where the 5-year target should be based on a 36% imperviousness (assumed greenfield condition as per City's Stormwater Management Design Guidelines), unless the "actual existing" imperviousness is lower
    - Applies to Subcatchments ER-1B-F, ER-1F, WR-1A4

Detailed summary results on a subcatchment basis have been included in Appendix D. For those subcatchments subject to the over-control criteria, a reduction in peak flow is clearly evident (60-70% reduction in simulated peak flows) relative to actual existing conditions. For other locations, resultant subcatchment peak flows are generally within 5% of the target values.

The resultant flow scenarios have been combined with the overall watershed flows to assess the effectiveness of the on-site SWM controls and which scenario yields the most conservative results for the receiving Hager-Rambo system. For the purposes of an initial high-level review, the assessment has been completed by using manual addition of the Burlington GO MTSA area hydrographs with the hydrographs from contributing upstream areas (i.e., at key watercourse nodes), namely:

— Node P (West Rambo Creek at QEW)

— Node Q_{out} (Total Discharge from the East Rambo Pond in VO Modelling)

Spill flow from Roseland Creek has been omitted for the purposes of the assessment for consistency between scenarios and to ensure the focus is on the Hager-Rambo system directly, given the high-level nature of the initial assessment. The preceding, in addition to the local subcatchment flows for the area between the QEW and Fairview Street (i.e., the Burlington GO MTSA) would effectively represent the combined flow to the Hager-Rambo Diversion Channel at approximately Node K (refer to Drawing 3), notwithstanding omission of flow routing effects and spills (including Roseland Creek). Results are presented in Table 2.1.7.

Table 2.1.7. Total Peak Flows (m^3/s) for Hager-Rambo System (Node K) for Different Land Use Scenarios using Manual Hydrograph Addition (Roseland Creek Excluded)

SCENARIO	2YR	5YR	10YR	25YR	50YR	100YR	REGIONAL 12H
Actual Existing	21.0	30.7	38.4	48.2	55.3	62.4	77.3
Modelled Existing	23.0	33.2	41.2	51.2	58.5	66.1	77.0
Future Intensification (Uncontrolled)	23.3	33.6	41.6	51.8	59.2	66.7	77.0
Future Intensification (With SWM)	18.8	27.7	34.9	43.8	50.3	57.1	77.6

1. Note: Results presented are for screening purposes of land use scenarios and do not reflect final peak flows applied for floodplain mapping purposes.

The results indicate the "future land use with SWM" scenario (as per City's Stormwater Design Guidelines) is expected to control peak flows below actual existing values for the 2 through 100-year storm events. For the Regional Storm Event, the results indicate that the Future with SWM scenario actually generates a slightly higher peak flow, however the differences are considered by WSP to be nominal. This may be attributable to hydrograph timing (greater synchronization of peak flows) or the nature of the assumptions regarding the overflow ordinates for the hypothetical SWM facilities.

For the purposes of subsequent riverine peak flow estimation and development of conservative floodlines, the preceding results suggest that the future uncontrolled scenario should be applied for the 2–100-year storm events, and that the actual existing land use scenario should be applied for the Regional Storm Event.

Following the completion and submission of the preceding initial analyses by WSP, staff from CH undertook a supplementary analysis using the submitted hydrologic and hydraulic modelling files. While the preceding analysis by WSP employed a simplified, manual hydrograph addition process, CH applied the various time-varying hydrographs for the different scenarios and then applied them to the 2-dimensional HEC-RAS modelling to better assess the hydrograph timing implications of flow routing through the East and West Rambo Creek areas. CH's additional analyses also considered potential changes in simulated peak flows at multiple nodes and locations. The results of this additional analysis were initially discussed with WSP staff December 22, 2022 (along with copies of updated hydrologic and hydraulic modelling files); and again, along with City staff January 10, 2023. A summary of results has been included with CH's formal comment letter of January 23, 2023 (included in Appendix B of the current report; refer to Table 1).

The results provided by CH indicate that for the 100-year storm event, the application of the preliminary proposed SWM quantity controls (as per City of Burlington 2020 Guidelines) may increase peak flows along the Hager-Rambo System as compared to existing conditions. Simulated peak flow increases range from 0.6% to 9.0%. The largest simulated increases are indicated along West Rambo Creek (Nodes P1 to P3 as per Drawing 3; increases of between 6.7 and 9.0%). Downstream along the Hager-Rambo Diversion Channel increases of approximately 4% are indicated. By comparison, for the future intensification scenario without SWM quantity controls in place, lesser peak flow increases are generally indicated (between 0.6% and 3.2% with the exception of one (1) location).

The simulated results provided by CH indicate that flow routing through the Burlington GO MTSA (as represented by inclusion of flow routing through the HEC-RAS 2D hydraulic modelling) does have an influence on hydrograph timing and ultimately yields different conclusions with respect to the preliminary effectiveness of the direct application of the City's Stormwater Management Guidelines for on-site SWM quantity controls.

Although not directly included in the results generated by CH, CH has also expressed concern that future intensification may also result in increases in Regional (Regulatory) Storm peak flows to watercourse systems. Although not assessed as part of CH's additional review, the potential need for Regulatory Event quantity controls to mitigate such simulated increases has been noted. As per Provincial Guidelines (i.e. MNR, 2002), the Province (MNRF) recommends that quantity controls typically not be included or credited for the Regulatory Event. Further, given the lack of public land in this area, it is considered likely that if required (and approved and credited) Regulatory Event controls would need to be implemented on private property. This would require further review to ensure that both the City and CH are satisfied that these controls are properly operated and maintained in perpetuity. As per CH's comments (ref. Appendix B) only ponds and underground tanks would be considered for crediting. Given the expected form of future intensification, it is considered likely that only underground tanks would be proposed (again, if required and approved and credited). Further consideration of Regulatory Event Controls, and the policy requirements to enable such controls on private property is beyond the scope of the current study but would need to be determined as part of future study.

In general, the City has expressed a preference for evaluating the requirements for future developments within the Burlington GO MTSA on a case-by-case basis, as has historically been the case. It is recommended that the modelling tools developed as part of this study be applied for future assessments accordingly. Such assessments could either be completed by representatives of the private developer, or by a representative of the City. In either case, modelling updates should be completed and consider cumulative impacts; a "current' set of modelling files should be maintained to ensure that new developments consider the assessments and results of previously approved developments.

Based on the preceding results, it is conceivable that some sites may not require SWM quantity controls for some or all of the events assessed (i.e. uncontrolled discharge may be more appropriate to mitigate peak flows to downstream receivers). Future assessments will need to determine the most appropriate approach for the full suite of the 2 through 100-year storm events, as well as the Regional Storm Event. In addition, where sites discharge to intermediate conveyance systems (i.e. local storm sewers), the potential negative impacts to these systems from uncontrolled discharges may need to be balanced against the simulated impacts to the downstream receivers\watercourses.

2.1.4 FLOOD CONTROL FACILITIES

2.1.4.1 BACKGROUND

The previously completed hydrologic modelling updates to the Hager-Rambo model (as part of the Phase 1 study) also included refinements to the rating curves representing the three (3) Flood Control Facilities (FCFs) within the watershed, namely the East Rambo Pond, the Freeman Pond, and the West Hager Pond. Those rating curves were updated based on field measurements of the associated outlet structures and topographic mapping to establish a refined stage-storage-discharge relationship.

Subsequent to the Mobility Hubs Phase 1 Report, the Hager-Rambo Flood Control Facilities Study Report (Wood, September 2020) documented field investigations and assessments for hydraulic, geotechnical and structural considerations in the three (3) FCFs to further support crediting of the FCFs in the hydrologic models used for Regulatory flow determination. The FCF rating curves were adjusted slightly as part of this effort, primarily due to the revised stage-storage information associated with more current LiDAR based topography data (as per the now publicly available Halton 2018 Package B DTM, part of the GTA 2014-2018 project by the Province of Ontario). The updated storage-discharge rating curves presented in the September 2020 FCF Study Report remain the most current and have therefore been applied in the Visual OTTHYMO modelling for this Phase 2 study.

As part of the Phase 1 study, two (2) scenarios were considered for the East Rambo Pond. Scenario 1 assumed that the subject FCF would function as per existing conditions, with the majority of the spill being directed to the West Rambo Creek. Scenario 2 assumed a potential upgrade to the culvert capacity beneath the QEW such that all flows are directed to East Rambo Creek. WSP (then Wood) subsequently prepared a retrofit feasibility assessment (included as Appendix E to the Phase 1 report) which provided WSP's professional opinion that such a retrofit was considered neither desirable nor practically feasible. CH subsequently reviewed and approved this conclusion as part of the overall Phase 1 reporting. As such, all assessments are based on the Scenario 1 approach (i.e., East Rambo FCF functions as per existing outlet structures and overflows).

The East Rambo FCF was represented in the hydrologic modelling using the rating curve and flow split as per Scenario 1 (existing conditions) from the 2020 Flood Control Facility Study Report. Notwithstanding the preceding, given the complexities of the hydraulics of this area due to spill flows from Roseland Creek, a locally specific 2D hydraulic modelling assessment of this area has been completed separately and is described in Section 2.1.5.

As a component of the update to the Visual OTTHYMO (VO) modelling platform, in order to prevent confusion for future users, the conceptual model provided by CH and reviewed by the Study Team, was modified to remove the East Rambo FCF and routing through the MTSA area in favour of an integrated VO and HEC-RAS 2D modelling approach. The catchments within the MTSA area and node flows entering the 2D mesh described in Section 2.1.5 have been disconnected from the downstream routing in VO to allow HEC-RAS 2D to perform the hydraulic routing for the Study Area. Subsequent to running the HEC-RAS 2D hydraulic model for routing, the output hydrograph from HEC-RAS was re-imported into VO at the diversion channel and Brant Street (ref. Node K on Drawing 3), or else the external contributing flows from VO manually added to the routed flows from the HEC-RAS 2D modelling.

The base rating curve for the Freeman Pond has been represented using the current rating curve developed in the 2020 Flood Control Facility Study Report.

The base rating curve for the West Hager Pond has been represented using the fully restored (i.e., removal of sediment at the outlet) rating curve developed in the 2020 Flood Control Facility Study Report.

2.1.4.2 *DEBRIS BLOCKAGE CONSIDERATIONS*

As required by CH and included in the study TOR (refer to Appendix B), consideration has been given to consider the potential for debris blockage to affect facility performance. The TOR indicate that three (3) scenarios were to be considered: FCFs at full capacity, FCFs with partial blockage, and all FCFs removed.

As per CH's comments of May 19, 2022 (refer to Appendix B), CH indicated that it was expected that each FCF would be reviewed individually, and then a recommendation developed as to the likelihood of debris blockage and develop modified rating curves accordingly. CH then noted that it was "envisioned that these recommendations will be used to inform the flood hazard mapping should FCFs ultimately be credited". As such, it is understood that any recommended debris blockage rating curves should become the basis for estimated Regulatory flood flows and associated Regulatory flood hazard limits in the current study.

CH provided its initial opinion as part of the May 19, 2022 comment letter. Considerations were also discussed further with CH staff at the team meeting of May 26, 2022. Based on the preceding and WSP's further review, the following has been determined. This approach is also consistent with that outlined in the "Proposed Approach to Finalize Reports" document (June 16, 2022; refer to Appendix B).

— **East Rambo Pond:** The grate covering the East Rambo Pond FCF outlet pipe serves to protect the low flow outlet culvert behind from becoming blocked (consistent with the comments provided by CH in its May 19, 2022 letter). Based on WSP's field reconnaissance, the grate surface area is also more than twice as large as the culvert behind it. It is therefore expected that the grate would protect the low flow outlet from experiencing a blockage during the Regulatory Event. This is suggested considering the size and efficacy of the grate at preventing blockage and City staff's commitments to monitor and remove potential debris blockages at the grate. Further, the grate would need to experience more than a 50% blockage for it to begin limiting conveyance capacity (capacity of the low flow outlet controls). As such, blockages are not expected to negatively affect the performance of the East Rambo FCF. Given that the developed rating curve is based on the culvert dimensions behind the grate, the full opening width is to be used for flow conveyance. Note that the routing through the East Rambo Pond has been completed using the HEC-RAS 2D modelling for this area, due to the complex spill flow hydraulics including interaction with spills from Roseland Creek. This is described further in conjunction with the hydraulic modelling results.

— **Freeman Pond:** As noted in CH's comments of May 19, 2022, given "the large conveyance capacity of the low flow outlet", the urban land cover present upstream, the minimal large woody debris present in the Freeman Pond, and the upstream barriers to large debris, debris blockage is considered to be less likely for the Freeman Pond. As such, the full rating curve for this FCF is proposed to be applied to hydrologic model simulations.

— **West Hager Pond:** As noted by CH in its comments of May 19, 2022, at this FCF "the potential for blockage may be higher" given the land cover present upstream (naturalized\treed valley) and larger vegetation has more of a potential to accumulate at the culvert\FCF control structure. As such, a 50% blockage of the culvert width has been assumed. This has been applied on the basis of the "restored outlet" rating curve (accumulated sediment depth removed; refer to Hager-Rambo Control Facilities Study Report, Wood, September 22, 2020). A modified rating curve has been developed accordingly. Details are included in Appendix D.

The preceding approach has been employed for all subsequent simulations of the "FCFs credited" scenario. In addition, the "no FCF" scenario has also been considered, as discussed further in Section 2.1.6. No third scenario of FCFs is considered required based on the preceding.

2.1.5 MODEL RESULTS WITH FCFS CREDITED

2.1.5.1 EAST RAMBO POND AREA

The updated combined Visual OTTHYMO (VO) model for Roseland Creek (to the QEW) and East Rambo Creek has been simulated for the 2 to 100-year 24-hour SCS Type II distribution design storms (2020 IDF), as well as the Regional Storm Event. The Regional Storm Event has been simulated for the governing 12-hour distribution using AMC III (saturated) infiltration parameters.

Based on the analysis noted in Section 2.1.3.2, the frequency (2-100-year) storm events have been simulated for the Future Intensification Land Use (90% imperviousness; without SWM), while the Regional Storm Event has been simulated for the Actual Existing Land Use scenario, as these generate the greatest peak flows respectively based on WSP's analysis. As discussed in Section 2.1.3.2, additional analyses were subsequently completed by CH (refer to that section and CH comments of January 23, 2023 in Appendix B) that generated slightly different conclusions. These results are included with CH's comment letter included in Appendix A but have not been analyzed or assessed further by WSP.

The peak flows at the key locations from both models are provided in Table 2.1.8. Refer to Drawings 3 and 4 for further details on specific locations. Note that the presented drainage areas are total areas; VO may calculate varying areas depending on flow splits and the percentage of drainage diverted.

It should be noted that flows are generated at Node R8.1 in the combined Roseland Creek and Hager-Rambo Creek VO model. The flows from Node R8.1 are included in the flow to Node Q, while the local subcatchment ER-1E flow contributes to the model downstream of the East Rambo FCF (i.e., not included in Node Q). The flows from Node R11.2 have not been added to additional flows in the Hager-Rambo VO model (rather this spill flow is determined by the HEC-RAS 2D model separately) and hence the peak flows presented in Table 2.1.8 do not include the influence of the potential spill along the QEW (hydrologic modelling results only). The spill flow from the Roseland Creek Node R11.2 is understood to contribute to the spill flows from the East Rambo Creek over the QEW and impact the hydraulics at the outlet of the East Rambo FCF, which is explored in detail in subsequent sections on hydraulic modelling.

The inflows to the East Rambo FCF have been disaggregated from Node Q into component flows based on the inlet locations to the facility. The VO model schematic was reviewed to determine appropriate hydrographs to represent the inlets to the East Rambo FCF. The East Rambo FCF has three designed inlets (not including any potential spill inflows from Roseland Creek via the QEW):

— The East Rambo Main Channel - inlets at the northwest of the FCF

— The CNR culverts - distributed along the north boundary of the FCF

— The Roseland Creek diversion - pipe inlets at the northeast of the FCF

Further details and associated peak flows are noted in Table 2.1.8.

The East Rambo Main Channel is best represented by hydrograph routing element 5102 in the VO model which contains the runoff contributing to the East Rambo channel prior to the inclusion of the CNR culvert major/minor split. The CNR culvert flows are split into major and minor in hydrograph 800. The CNR culvert flows are split into major/minor based on a total culvert capacity of 10.2 m^3/s (as per the currently approved model) to represent the four (4) contributing pipes. Flows greater than 10.2 m^3/s would be expected to spill and have been assumed to be routed westerly along the CNR tracks to the East Rambo Main Channel and contribute to the northeast inlet of the East Rambo FCF. The local subcatchment (ER-1G) has been added to the distributed culvert inflows for simplicity, as the local runoff would be best represented by a distributed inflow similar to the four culverts.

Table 2.1.8. Updated Peak Flows at Nodes of Interest for Hager-Rambo and Roseland Creek Systems

LOCATION (Node)	DRAINAGE AREA[1] (ha)	SIMULATED PEAK FLOW (m^3/s) FOR RETURN PERIOD (YEARS) OR STORM						
		2	5	10	25	50	100	REGIONAL STORM (12 HR AMC III)
Roseland Creek at QEW (R11.2)	447.90	11.70	18.58	23.34	27.73	30.50	34.53	42.39
Local NSR Drainage (ER-1E)	12.00	0.45	0.69	0.87	1.14	1.32	1.50	1.47
Total Inflow to East Rambo FCF[2] (Q)	641.78[2]	24.85	36.82	42.25	53.05	60.06	67.00	54.69
Node Q Disaggregated Flows for 2D Hydraulic Model by Inflow Location								
East Rambo Main Channel to East Rambo FCF (Hyd. 510/9006)	377.54	11.66	15.49	17.01	19.12	20.76	22.35	21.00
CNR Culverts Major Spill to East Rambo Main Channel (Hyd. 800 Maj.)	198.02	0	5.09	8.58	14.08	17.36	21.62	26.83
CNR Culverts Minor to East Rambo FCF (Hyd. 800 Min.)	198.02	9.85	10.20	10.20	10.20	10.20	10.20	10.20
Direct East Rambo FCF Drainage (ER-1G)	13.90	1.26	1.74	2.18	2.62	2.95	3.27	1.93
Roseland Area 8 Diversion to East Rambo FCF (R8.1)	52.32	3.34	4.98	5.99	7.57	9.47	10.31	7.98

1. Drainage areas are totals based on subcatchment boundary plan (refer to Drawing 3), do not reflect variations due to spills, flow splits, or other factors.
2. Does not include drainage area from Roseland Creek spill.

2.1.5.2 *EAST AND WEST RAMBO CREEKS*

In addition to the primary inflows to the East Rambo FCF area, additional inflows are required for the areas downstream of the East Rambo FCF to represent both tailwater conditions and also to support the 2D hydraulic modelling assessment of this area, as described in subsequent sections. Given the nature of the hydraulic modelling, combined inflows from the hydrologic modelling cannot be used; rather the discrete inputs from individual subcatchments are required.

The peak flows for the catchments through the Burlington GO MTSA (i.e., East and West Rambo Creeks) are presented in Table 2.1.9; refer to Drawing 3 for locations. The Roseland Creek spill and other combined upstream inflows were presented previously in Table 2.1.8. Subcatchment flows are based on the land use assumptions noted previously, namely the application of the future intensification (without SWM) scenario flows for the 2–100-year storm events, and the application of the actual existing scenario flows for the Regional Storm Event.

It should be noted that under typical conditions, drainage from the Brant Street underpass is serviced by a gravity storm sewer system which drains southerly down Brant Street and to the Lower Rambo Creek. As such, this area is included in the Downtown hydrologic modelling (as per Section 3) rather than the Hager-Rambo Diversion VO modelling. These flows have been extracted from the Downtown PCSWMM modelling accordingly.

Table 2.1.9. Updated Peak Flows at Nodes of Interest for East and West Rambo Creek Systems

LOCATION (NODE)	DRAINAGE AREA (ha)	SIMULATED PEAK FLOW (m^3/s) FOR RETURN PERIOD (YEARS) OR STORM						
		2	5	10	25	50	100	REGIONAL STORM (12 HR AMC III)
ER-1A[1]	28.40	2.13	3.01	3.78	4.64	5.26	5.87	3.82
ER-1B-E[1]	3.34	0.44	0.59	0.69	0.83	0.92	1.02	0.48
ER-1B-F[1]	6.59	0.90	1.23	1.43	1.70	1.97	2.17	0.95
ER-1D-E	21.79	2.08	3.00	3.57	4.47	5.07	5.64	3.10
ER-1D-F	1.85	0.29	0.39	0.45	0.53	0.59	0.65	0.26
ER-1C	8.08	1.12	1.57	1.83	2.17	2.43	2.67	1.17
ER-1F	8.21	0.96	1.32	1.55	1.87	2.10	2.32	1.17
WR-1A7	4.66	0.63	0.85	0.99	1.18	1.32	1.51	0.67
WR-1A5	11.56	1.54	2.10	2.45	2.98	3.33	3.71	1.66
West Rambo Creek at QEW (P)	84.10	5.39	7.96	10.38	12.86	14.91	16.99	11.83
WR-1A6	12.47	1.12	1.58	1.94	2.36	2.68	3.00	1.75
WR-1A2	1.79	0.22	0.29	0.34	0.41	0.46	0.51	0.25
WR-1A3	13.80	1.72	2.35	2.75	3.30	3.69	4.07	1.96
WR-1A4	10.34	1.29	1.76	2.06	2.58	2.89	3.20	1.48
Brant Street Underpass[2]	7.68	1.21	1.62	1.88	2.21	2.46	2.70	1.12
WR-1B-E	4.17	0.50	0.69	0.81	0.98	1.10	1.21	0.60
WR-1B-F	4.99	0.67	0.91	1.06	1.32	1.47	1.62	0.72

1. Subject to a major/minor flow splits as described further in this section.
2. From PCSWMM modelling for Downtown area (subcatchments only), refer to Section 3 for further details.

At the subcatchment level, peak runoff response for the frequency design storms is high in comparison to the Regional Storm's response. In this regard, subcatchments' response to the Regional Storm is approximately equivalent to, or less than, the response predicted from the 10-year frequency design storm.

This is attributed to the high impervious coverage simulated through this area which tends to produce higher peak flows under high rainfall intensity events compared to high rainfall volume events.

In the 2D model, the runoff from ER-1A is applied upstream of the CN tracks, with the 1050 mm diameter storm sewer explicitly included to determine the actual minor/major flow split. Drainage on the south side of the CNR tracks (ER-1B) would also undergo a minor/major split, however based upon discussion with CH (February 2, 2022), it was agreed that the full flow from ER-1B would be applied directly to the Hager-Rambo Diversion Channel to better represent riverine flood conditions, rather than more localized urban flooding.

The runoff hydrographs for local catchments between the QEW and the Hager-Rambo diversion channel (as per Table 2.1.9) have been extracted from the VO model and simulated as inflows to the HEC-RAS 2D mesh (in addition to the primary inflows at the East Rambo FCF described in Table 2.1.8). The application locations of the inflow hydrographs are discussed in subsequent sections.

2.1.5.3 *DOWNSTREAM HAGER-RAMBO*

The HEC-RAS 2D model (as described further in subsequent sections) has been applied (using the inflows described in the previous section) to determine the resulting simulated outflow hydrographs at the Hager-Rambo diversion channel to account for the complex hydraulics through the Burlington GO MTSA. In order to eliminate the loss of flow associated with hydraulic structure attenuation in an unsteady state simulation (and remain consistent with Provincial Policy), a "hydroburned" version of the model has been employed which removes the hydraulic structures. This approach is described further in subsequent sections.

The output hydrographs from the HEC-RAS 2D modelling have been combined with the discrete flows from the VO modelling for downstream areas (i.e. subcatchments as well as flow from Hager Creek) to generate the resulting peak flows, which have been used within the 1D HEC-RAS analysis for the Hager-Rambo Diversion channel (west of Brant Street) as described further in Section 2.3.

As no spill flows have been confirmed to meet CH criteria for crediting (i.e. it is assumed these spills could potentially be eliminated in the future), where necessary, these spill flows have also conservatively been re-inserted into the modelling.

As noted, the flows from West and East Hager Creeks have been simulated directly in the VO model (these areas do not require consideration of complex spill flow routing as is the case for the East and West Rambo Creeks area). The resulting peak flows for the nodes of interest in the West Hager and East Hager Creeks (i.e., from VO – prior to any combination with upstream flows from the Rambo Creek system) are provided in Table 2.1.9. Refer to Drawing 3 (attached) for node locations.

The results in Table 2.1.10 indicate that for the flows from both the Freeman and West Hager Ponds, the Regional Storm Event will continue to govern over the 100-year storm event (i.e., is the Regulatory Event). The 100-year inflow to the Freeman Pond is in fact higher than the Regional Storm Event, however the volume of the Regional Storm Event is much greater, and as such the attenuation provided by the Freeman Pond for that event is notably less, resulting in a greater overall discharge.

The Hager Creek flows downstream of the Freeman and West Hager FCFs are similarly governed by the Regional Storm Event which is consistent with the results for the upstream contributing areas to both FCFs. The East Hager Creek flows which contribute to the diversion channel (Node 535) are however governed by the 100-year storm event, consistent with the other smaller, urbanized (high imperviousness) drainage areas within the Burlington GO MTSA.

Table 2.1.10. Updated Peak Flows at Nodes of Interest for Hager-Rambo Creek Watershed with Flood Control Facilities Credited

LOCATION	DRAINAGE AREA[1] (HA)	SIMULATED PEAK FLOW (m^3/s)	
		100Y	REGIONAL STORM (12 HR AMC III)
Hager-Rambo Diversion at CNR (Node K)[2]	887.60	52.81	75.97
East Hager Creek (535)	62.17	13.58	8.17
Hager-Rambo Diversion U/S of Hager Creek (Node L)	949.77	66.39	84.14
Freeman Pond Inflow (Node G)	367.91	80.55	71.02
Freeman Pond Outflow (Node G1)	367.91	18.21	43.24
West Hager Pond Inflow (Node H1)	155.00	16.84	18.55
West Hager Pond Outflow (Node H2)	155.00	6.42	18.51
Hager Creek at CNR (Node H3)	599.71	30.92	66.29
Hager Creek at H-R (Node H)	622.29	34.04	67.73
Hager-Rambo Diversion D/S of Hager Creek (Node M)	1.572.06	100.43	151.87
WH-1B	28.67	4.33	3.78
Hager-Rambo Diversion at QEW (Node N)	1,600.73	104.76	155.65

1. Drainage areas are totals based on subcatchment boundary plan (refer to Drawing 3), do not reflect variations due to spills, flow splits, or other factors. Roseland Creek spill area (node R11.2) excluded.
2. Based on output from HEC-RAS 2D modelling; refer to subsequent sections.

2.1.6 MODEL RESULTS WITH FCFS NOT CREDITED

As requested by CH and per the Study TOR (refer to Appendix B), a "no Flood Control Facility" (no FCF) scenario has been required, in order to assess impacts should FCFs not be credited. It should however be clearly understood that the City of Burlington has previously noted that it does not support the application of a "no FCF" scenario, however CH has indicated the requirement to present these results for comparison purposes. CH has received comprehensive documentation from the City (February 9, 2023) which details the ownership and maintenance responsibilities (between the City, CH and MTO) of the Hager-Rambo Flood Control System facilities which include the East Rambo Pond, Freeman Pond and West Hager Pond.; this remains under review by CH.

A No FCF scenario for the East Rambo FCF has been assessed in the HEC-RAS 2D model due to the complexities of the spill mechanics in this case, which cannot be reasonably modelled in a hydrologic model. Based on discussions with CH, and as indicated in CH's review comments of January 27, 2022, CH's preferred approach involves placing a "storage area" element within the 2D mesh and assigning a rating curve characteristic of a typical channel section through the length of the facility to effectively exclude a large portion of the facility's flood storage volume. The "storage element" maintains connection with the primary low flow outlet (3.0 m x 1.5 m box culvert) as well as high flow/spill pathways (i.e. CNR underpass). This is described further in subsequent sections. For the Freeman and West Hager FCFs, the "No FCF" scenario involves the removal of these features from the hydrologic (VO) modelling.

The resulting flows from the HEC-RAS 2D modelling are described in subsequent sections; the flows from the VO modelling, with specific reference to the West Hager and Freeman Flood Control Facilities, are described herein. Resulting peak flows are presented in Table 2.1.11. Results for inflows to the FCFs (Freeman and West Hager FCFs), East Hager Creek (535) and Subcatchment WH-1B would be the same in all cases and have therefore not been reproduced from the results previously presented in Table 2.1.10. The resulting differences, as compared to the "with FCF" (Flood Control Facilities Credited) results (as per Table 2.1.11), are also presented in Table 2.1.11.

Table 2.1.11. Peak Flows at Nodes of Interest for Hager-Rambo Creek Watershed without Flood Control Facilities Credited and Resulting Per cent Differences

LOCATION	DRAINAGE AREA[1] (HA)	SIMULATED PEAK FLOW (m^3/s)					
		WITHOUT FCFS		ABSOLUTE DIFFERENCE TO RESULTS WITH FCFS		PERCENT DIFFERENCE TO RESULTS WITH FCFS	
		100Y	REGIONAL STORM (12 HR AMC III)	100Y	REGIONAL STORM (12 HR AMC III)	100Y	REGIONAL STORM (12 HR AMC III)
Hager-Rambo Diversion at CNR (Node K)[2]	887.60	76.85	80.30	+24.04	+4.33	+46%	+6%
Hager-Rambo Diversion U/S of Hager Creek (Node L)	949.77	90.43	88.47	+24.04	+4.33	+36%	+5%
Freeman Pond Outflow (Node G1)	367.91	80.55	71.02	+62.34	+27.78	+342%	+64%
West Hager Pond Outflow (Node H2)	155.00	16.84	18.55	+10.42	+0.04	+162%	+0.2%
Hager Creek at CNR (Node H3)	599.71	106.80	98.02	+75.88	+31.73	+245%	+48%
Hager Creek at H-R (Node H)	622.29	108.77	100.74	+74.73	+33.01	+220%	+49%
Hager-Rambo Diversion D/S of Hager Creek (Node M)	1.572.06	198.20	189.21	+97.77	+37.34	+97%	+25%
Hager-Rambo Diversion at QEW (Node N)	1,600.73	202.53	192.99	+97.77	+37.34	+93%	+24%

1. Drainage areas are totals based on subcatchment boundary plan (refer to Drawing 3), do not reflect variations due to spills, flow splits, or other factors. Roseland Creek spill area (node R11.2) excluded.
2. Based on output from HEC-RAS 2D modelling ("hydroburned" model which eliminates structure attenuation); refer to subsequent sections.

For the “no FCF” scenario, the results consistently indicate higher peak flows for all scenarios as compared to the “with FCF” scenario, as would be expected. The results also indicate that under the “no FCF” scenario, the 100-year storm event would govern for the Freeman Pond, due to the lack of peak flow attenuation and sharply peaked nature of the simulated hydrograph for that event. For the West Hager Pond, the peak flows for the 100-year and Regional Storm are very similar, however the 12-hour Regional Storm continues to govern. Further downstream, the 100-year storm event again governs, given the preceding trends for the Freeman Pond, and the much larger contributing drainage area for this facility, as compared to the West Hager Pond. Peak flows are however similar for both events.

2.2 2-DIMENSIONAL HYDRAULICS AND SPILLS

2.2.1 BASE 2D MODEL DEVELOPMENT

2.2.1.1 HYDRAULICS

On August 30, 2021, Conservation Halton (CH) provided a draft 2-dimensional (2D) hydraulic model in HEC-RAS Version 5.0.7 that was developed by CH to assess the potential overland flow spill occurring from Roseland Creek at the QEW, due to the capacity restrictions of the existing QEW enclosure, which conveys flows to the downstream section of open channel, beyond Harvester Road (approximately 450 m). This model contained two (2) separate 2D areas (shown as areas 1 and 2 in Figure 2.2.1) in two (2) different plans and geometries within HEC-RAS. An internal boundary condition line was used to apply the Regional Storm inflow hydrograph just downstream of the CNR and upstream of North Service Road. A boundary condition line was used to convey the spills from area 1 into area 2. WSP has used this model as the base for the current analyses. HEC-RAS Version 6.3.1 has been used for all 1D and 2D hydraulic modelling, as it is the most current and most stable version of the program.

Figure 2.2.1. Base 2D Model Study Areas

The topography from the base 2D model indicates that while most of the spill from Roseland Creek would drain south towards Queensway Drive at the QEW-Guelph Line off-ramp, a spill is also identified towards, and into, the East Rambo Pond. Spill inflows to this system were not considered as part of the previously completed hydrologic modelling (then SWMHYMO) of the Hager-Rambo System (Mobility Hubs Phase 1 Study); the updated rating curve for the East Rambo Pond indicated the potential for a third outlet via the low point on the North Service Road (Node Q3). The updated results from the Flood Control Facilities report however indicated that this spill would not be expected under the Regional Storm Event. The preceding however did not include spill flows from Roseland Creek via the QEW, as is being considered in the current assessment.

The results of the current assessment have indicated that the two (2) watersheds have the potential to interact which further complicates the assessment. Therefore, in a meeting with the City and CH (October 19, 2021), it was agreed to further update and expand the base 2D model to include the East Rambo Pond, including the low flow outlet to East Rambo Creek and the CNR crossing spill to West Rambo Creek, in order to better assess interactions between the various systems. The 2D area has been extended to Industrial Street in the northwest, near the East Rambo Pond, Brant Street in the west. To the south, the model has been extended generally to Fairview Street from its intersection with Brant Street. A further model extension to the south was determined to be necessary to properly map spill flows from both West and East Rambo Creeks to the receiver (Lower Rambo Creek). As such, the model has been further extended to include the residential areas south of Argon Court near Prospect Street, as well as the area along Brant Street. The model terminates along Lower Rambo Creek downstream of Blairholm Avenue and the associated enclosure underneath the St. John Catholic Elementary School site ref. Figure 2.2.2).

The extents of the model have been developed, given the expectation that the modelling will be used not only to assess the spills from Roseland Creek and the East Rambo FCF, but also to determine flooding extents around both East and West Rambo Creeks and the Hager-Rambo Diversion channel, as well as the spills southerly to Lower Rambo Creek, to the limits noted (i.e. Blairholm Avenue). The impacts of the spills to Lower Rambo Creek are however considered further, separately as part of the Downtown area assessment (as per Section 3).

The accuracy and detail of the terrain model is critical in creating an accurate and detailed 2D model. High resolution (0.5 m) processed LiDAR surface mapping with buildings has been provided by CH to use as terrain in the HEC-RAS model, which is understood to be sourced from the Provincial Lidar DTM Halton 2018 Package B. The LiDAR data use a vertical datum of CGVD:2013, which differs from the typical City standard datum of CGVD28:78. This was discussed as part of the 2020 FCF report; elevations in the CGVD:2013 datum were noted to be approximately 0.426 m lower than those in the CGVD28:78 datum based on a survey of local Provincial Benchmarks. This is generally consistent with the recommended conversion of 0.40 m suggested by CH as part of other studies.

Within the LiDAR data, hydraulic structures that allow flow through an embankment are typically not represented. Because of this, incorporating the raw LiDAR data into the 2D terrain model can overestimate storage behind these embankments and redirect flow erroneously. Therefore, structures have been appropriately coded within the 2D HEC-RAS model based on the dimensions and elevations included in the previously approved 1D HEC-RAS modelling (both for the Hager-Rambo and Roseland Creek systems) or information from record drawings supplied by the City (relevant drawings are included in Appendix C). Elevations have been converted to the CGVD:2013 datum accordingly. A vertical datum conversion factor of 0.43 m has been used for the culvert inverts if the drawings/survey were in the 1978 datum as opposed to the current study (2013 datum). In some cases, the elevation from the LiDAR dataset has been used to determine the expected culvert invert, given the high resolution of the data. Figures 2.2.3 and 2.2.4 indicate the location of structures that have been coded within HEC-RAS, in addition to the structures that were already present in the base model.

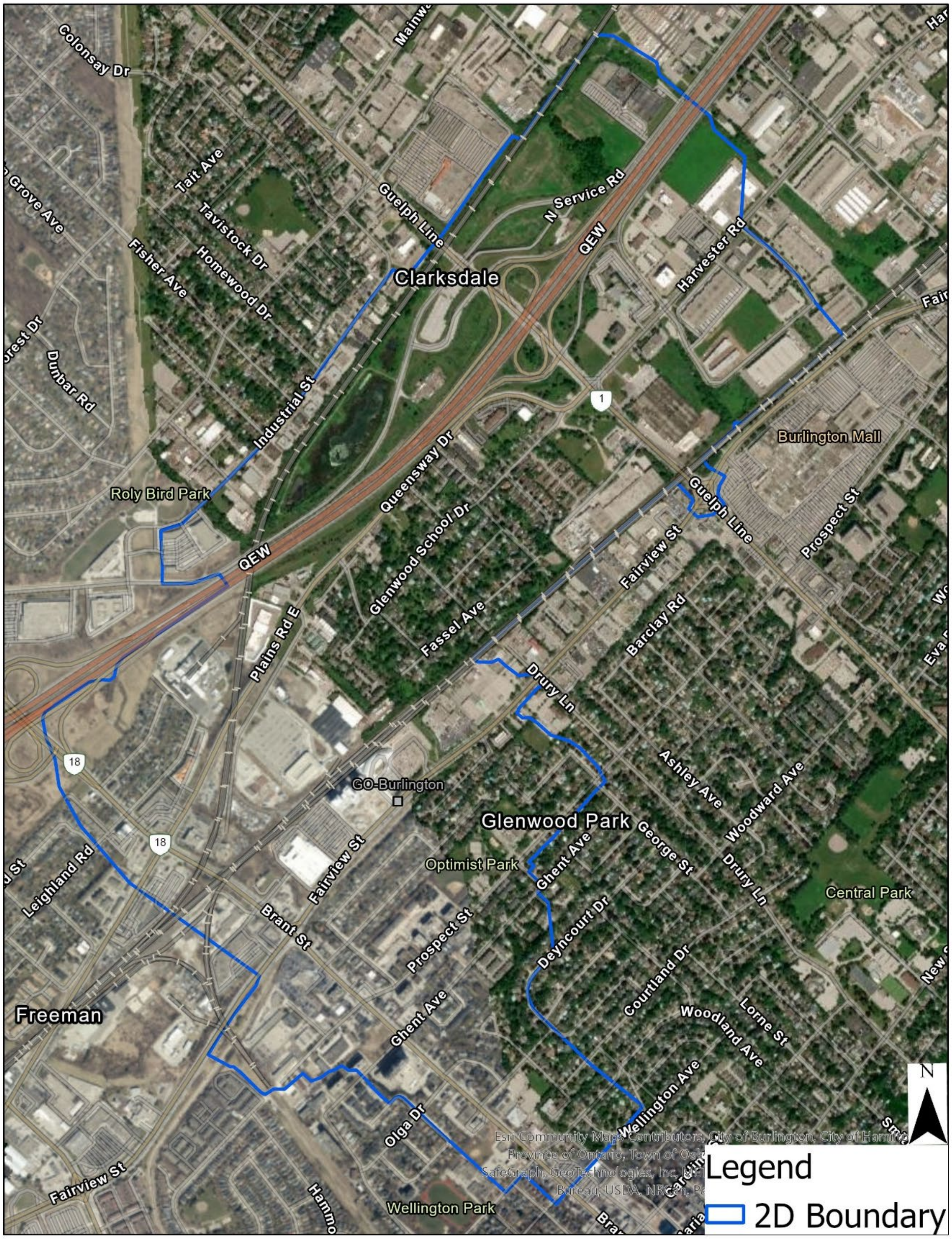

Figure 2.2.2. Updated 2D Model Area

Figure 2.2.3. Location of Modelled Structures in HEC-RAS (East Rambo Creek and Roseland Creek)

Figure 2.2.4. Location of Modelled Structures in HEC-RAS (West Rambo Creek Area)

Appendix D (Table D1) presents the details of the hydraulic structures that have been coded within the HEC-RAS model. Culvert dimensions and inverts have been extracted from multiple sources (drawings and other data provided in appendices) as noted previously.

A local storm sewer (ID #11) that drains south across the QEW (near Reimer Common) and then westerly along Queensway Drive (as an 825 mm to 1200 mm storm sewer) to East Rambo Creek at the upstream end into the box culvert close to the outlet to the open channel, has also been added in the HEC-RAS model based on drawings provided by the City (refer to Appendix C). WSP previously omitted this culvert, under the assumption that it would likely be full for major storm events. Based on discussions with City and CH however (November 19, 2021), it was agreed that the culvert should be included as a 1200 mm diameter circular conduit. A smaller crossing to the east (outletting to the same storm sewer on Queensway Drive, near the Best Western Hotel) has been neglected, as it is considered that the western crossing is the primary crossing, and both would ultimately be constrained by the capacity of storm sewer along Queensway Drive. The record drawings for these crossings have been included in Appendix C for reference purposes, along with the results of the field investigation completed by WSP.

The storm sewer (ID #17) that drains subcatchment ER-1A (refer to Drawing 3) south across the CNR and westerly along Fairview Street (connecting to the Hager-Rambo diversion channel west of Argon Court) has also been added to the model based on drawings shared by the City. This conduit has been used to determine the major/minor flow split from the flow from subcatchment ER-1A, which is added to the mesh upstream of this point. A local opening in the 2D mesh has been generated to match the estimated invert of the CNR crossing at this point by modifying the terrain in the ditch just upstream. Since the limiting section of the storm sewer was found to be 1050 mm diameter, this geometry was used in HEC-RAS to represent the entire length of the culvert. The storm sewer (ID #24) that drains subcatchment WR-1A3 towards West Rambo Creek has been included in the 2D model. Structure dimensions for the CNR culvert along West Rambo Creek, east of Brant Street (ID #25) were taken from a previous structural engineering study ("Burlington Railway Crossings – Inspection Summary Report" Wood, March 2020, and Wood Internal Memorandum (Penney-Galloway) of February 12, 2020) considering culvert rehabilitation completed by WSP for the City of Burlington. WSP was separately retained by the City of Burlington to undertake the inspection and design of four (4) stormwater railway crossings. One (1) of the four (4) crossings is the primary CNR tracks crossing of West Rambo Creek (referred to as WR6 in the current study). Based on the field work completed as part of this study, it was confirmed that the culvert is actually comprised of three (3) distinct sections, rather than one (1) section which was assumed in previous versions of the current study, based on the limited field reconnaissance of the downstream face of the crossing (upstream face was not accessible). The most upstream section is a 2850 mm diameter circular CSP pipe 13.6 m in length, the middle section is a 3100 mm span by 2850 mm rise masonry arch 14.6 m in length with exposed concrete, and the most downstream section is a 3100 mm span by 2850 mm rise concrete arch 4 m in length. Based on the preceding, the upstream section (2850 mm diameter circular CSP pipe) would be the critical conveyance section based on opening area and Manning's Roughness Coefficient (corrugated steel pipe as compared to concrete/masonry).

The West Rambo Creek crossing of Fairview Street (ID #27) has been modelled separately from the Hager-Rambo crossing (HR Diversion channel) as shown in Figure 2.2.5 from the HEC-RAS model. The separation of these structures has been confirmed based on drawings supplied by the City of Burlington, as well as aerial photography which clearly shows the West Rambo crossing outletting further downstream from the primary Hager-Rambo Diversion culvert crossing (ID #15).

The storm sewer (ID #36) that drains the CNR underpass on Brant Street has also been added using dimensions and elevations from the PCSWWM model developed for the Phase 1 Study (which in turn was based on record drawings supplied by the City) and has been set to outlet at the upstream end of Lower Rambo Creek downstream of Blairholm Avenue. This is a gravity storm sewer which drains across the Hager-Rambo Diversion channel (not connected) and outlets into the Downtown area (Lower Rambo Creek) near Blairholm Avenue. To model this storm sewer, the terrain was modified to represent the inlet/outlet areas.

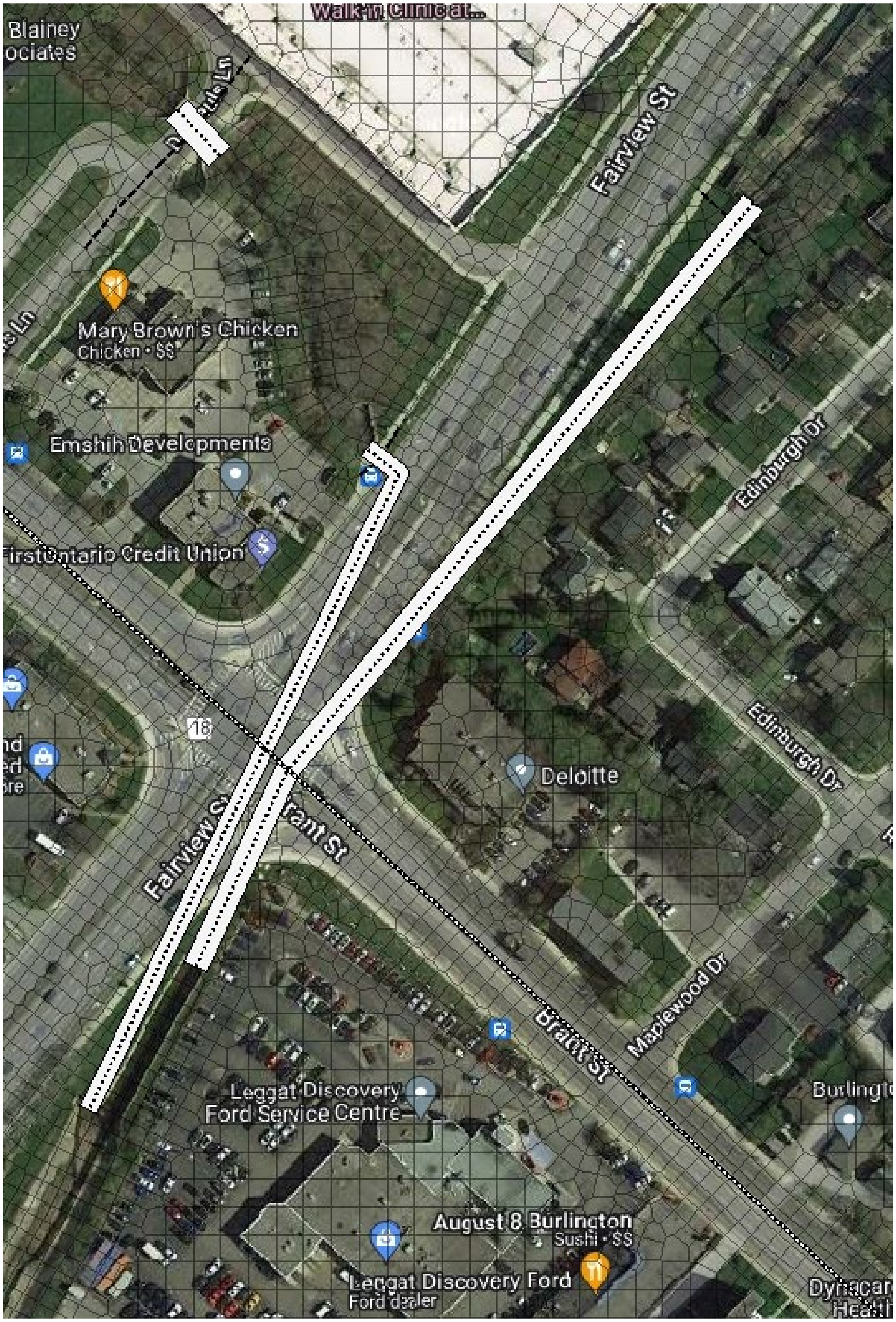

Figure 2.2.5. West Rambo Creek and Hager-Rambo (HR Diversion Channel)

Figure 2.2.6. Highway Barriers

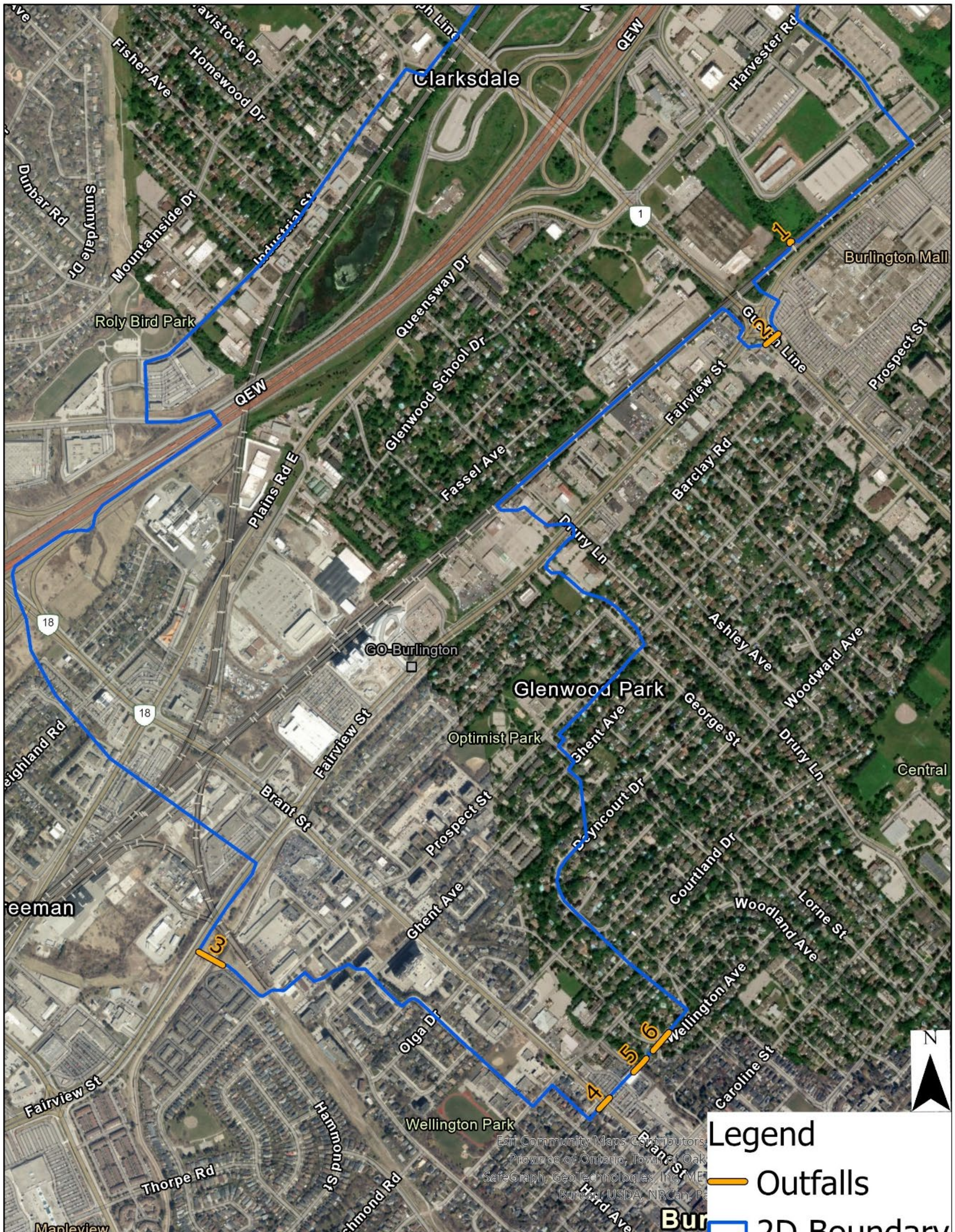

Figure 2.2.7. Location of Normal Depth Boundary Condition Lines (Outfalls)

In addition to the primary hydraulic structures, concrete barriers along the QEW have been added to the model within the terrain using terrain modification tools in RAS Mapper to appropriately represent obstructions caused in the flow of water (ref. Figure 2.2.6). The 2D connection height for solid highway barriers have been set at 1.05 m higher than the terrain (as per OPSD 911.132 for "tall wall" systems). Open metal guard-rail barriers, having an opening height of about 43.5 cm (as per MTOD 925.100 for three beam guide rail), have been assumed to allow conveyance of flow relatively freely given expected flow depths and therefore have not been represented in the terrain. No correction has been made to account for the width of the wooden support posts in the current modelling, as their impact is generally considered to be minor. Additionally, a sensitivity analysis to determine the impact of raising the Manning's n-value along the open guard rail section to 0.05 and 0.08 from 0.02 has been conducted and no significant impact on water depths or velocities has been observed. The output of the sensitivity analysis showing water depth and velocity has been included in Appendix D.

The LiDAR data at various overpasses were also corrected using terrain modification tools in RAS Mapper to avoid artificial ponding of water, creating a generally smooth surface, along with adjusted bridge abutments to better match vertical walls.

Manning's roughness coefficients have been defined across the study area for use in the calculations in the base 2D model provided by CH. The Manning's layer has been updated where the mesh was extended to ensure the correct application of roughness within HEC-RAS. In particular, the rail line (granular bedding) area originally had a roughness of 0.08 in the modelling received from CH. Based on WSP's review, and CH's standard table of roughness parameters, a value of 0.035 has been considered more appropriate; the modelling has been updated accordingly.

The 2D computational mesh (5 m X 5 m) for the extended area has been generated within HEC-RAS using the LiDAR elevation data. All hydraulically significant embankments, such as roads, have been enforced in the mesh using breaklines to ensure that the crests of these embankments are represented in the cell faces. This approach allows for a more detailed model than a standard square mesh can produce.

A normal depth boundary condition has been applied in the HEC-RAS model at each section along the 2D area boundary where water can leave the system (Figure 2.2.7). The channel slope in the LiDAR elevation data, as deemed appropriate, has been applied as the normal depth slope for each boundary condition.

HEC-RAS 2D models solve either the Saint Venant equations (Full Momentum) or the Diffusion Wave (simplification) equations. The HEC-RAS 2D computation module has the option of running the following equation sets: 2D Diffusion Wave equations; Shallow Water Equations (SWE-ELM) with a Eulerian-Lagrangian approach to solving for advection; or a new Shallow Water Equation solver (SWE-EM) that uses an Eulerian approach for advection (ref. HEC-RAS User's Manual Version 6.0). The default is the 2D Diffusion Wave equation set. In general, many flood applications will function adequately with the 2D Diffusion Wave equations and the Diffusion Wave equation set will run faster and is inherently more stable. However, there are applications where the 2D SWE could be used for greater accuracy. The two different set of equations have been tested for the base model and it has been determined that the 2D Diffusion Wave equation ran more stably and did not generate significant water surface cells errors in contrast to the original SWE solver. Therefore, all plans have been set to use the default set of equations using a fine time step of 1 to 3 seconds to generate the most stable run while balancing computation times.

Notwithstanding, any 2D models that are developed in the future for smaller development sites (2D model size that is anticipated to be smaller than what was developed in this study) should use the 2D Shallow Water equation (SWE-Eulerian-Lagrangian Method) using a fine time step of 1 second or smaller while able to justify computation times and power used, to generate the most stable and accurate simulation.

Additionally, a restart file has been used for all model plans and the simulation time has been set to allow sufficient time (24 hours for each plan run) for the flood wave to reach the peak and drain out of the 2D system completely.

2.2.1.2 *FLOWS*

Inflow hydrographs from the hydrologic model (VO) as described previously, have been used to represent appropriate inflows into the HEC-RAS model (locations shown in Figures 2.2.8 and 2.2.9). Subcatchment inflow hydrographs per recommendations from CH have been applied within the 2D mesh at outlet locations where flows have concentrated and can reasonably be assumed to form part of a regulated watercourse's floodplain or spill. Inflows include:

- Flow hydrographs from Roseland Creek at Node R11.2 have been applied as internal boundary conditions in the 2D mesh along Roseland Creek just downstream of the CNR and upstream of the North Service Road. No further downstream flows from Roseland Creek have been included.
- Flow hydrographs to represent inflow into the East Rambo Pond at node Q (ref. Figure 2.2.8), split into multiple hydrographs from:
  - The East Rambo Creek main channel, applied upstream of the CN tracks.
  - Major flows from the culvert inlets to the East Rambo Pond, applied within CN track north ditch.
  - Inflow to the East Rambo Pond (minor flows from the CNR culverts plus the flow from subcatchment ER-1G), distributed within the Pond.
  - Inflow from the Roseland Creek Area 8 Diversion, applied at eastern forebay.
- Runoff from the local subcatchment (ER-1E) has also been applied in the 2D mesh in the north QEW ditch and south of intersection of North Service Road and Guelph Line, as per Figure 2.9a.
- Runoff hydrographs for downstream areas have been applied at suitable locations within the respective subcatchments.
- Full subcatchment ER-1B flows have been applied just downstream of Fairview Street culvert in the H-R Diversion Channel.
- Node P flow (i.e., West Rambo Creek just downstream of the QEW) combined with WR-1A6 has been applied at the upstream extent of the West Rambo Creek.
- Flow hydrographs for the Regional Storm for the Brant Street underpass have been applied from the PCSWMM model developed (ref. Figure 2.2.10) for the Downtown area as part of the Phase 1 Study (further details are included in Section 3) and applied within the 2D mesh along Brant Street south of Leighland Road. Flows for ST_37, ST_38 and ST_60.2 have been applied within the 2D mesh for the 12-hour AMC III Regional Storm (ref. Figure 2.2.9). This approach was confirmed based on discussion with CH staff (ref. meeting of October 13, 2022).

It should be noted that while the limits of the 2D model extend down to Wellington Avenue, the model does not include any additional flow contributions to the mesh from this area (i.e., from the Downtown area PCSWMM modelling). As such the presented inundation extents from the hydraulic modelling should be understood to be riverine spill inundation limits only. The resulting spill flows have also been extracted for application to the Lower Rambo hydrology model (i.e., PCSWMM) at the points of spill, as discussed further in subsequent sections.

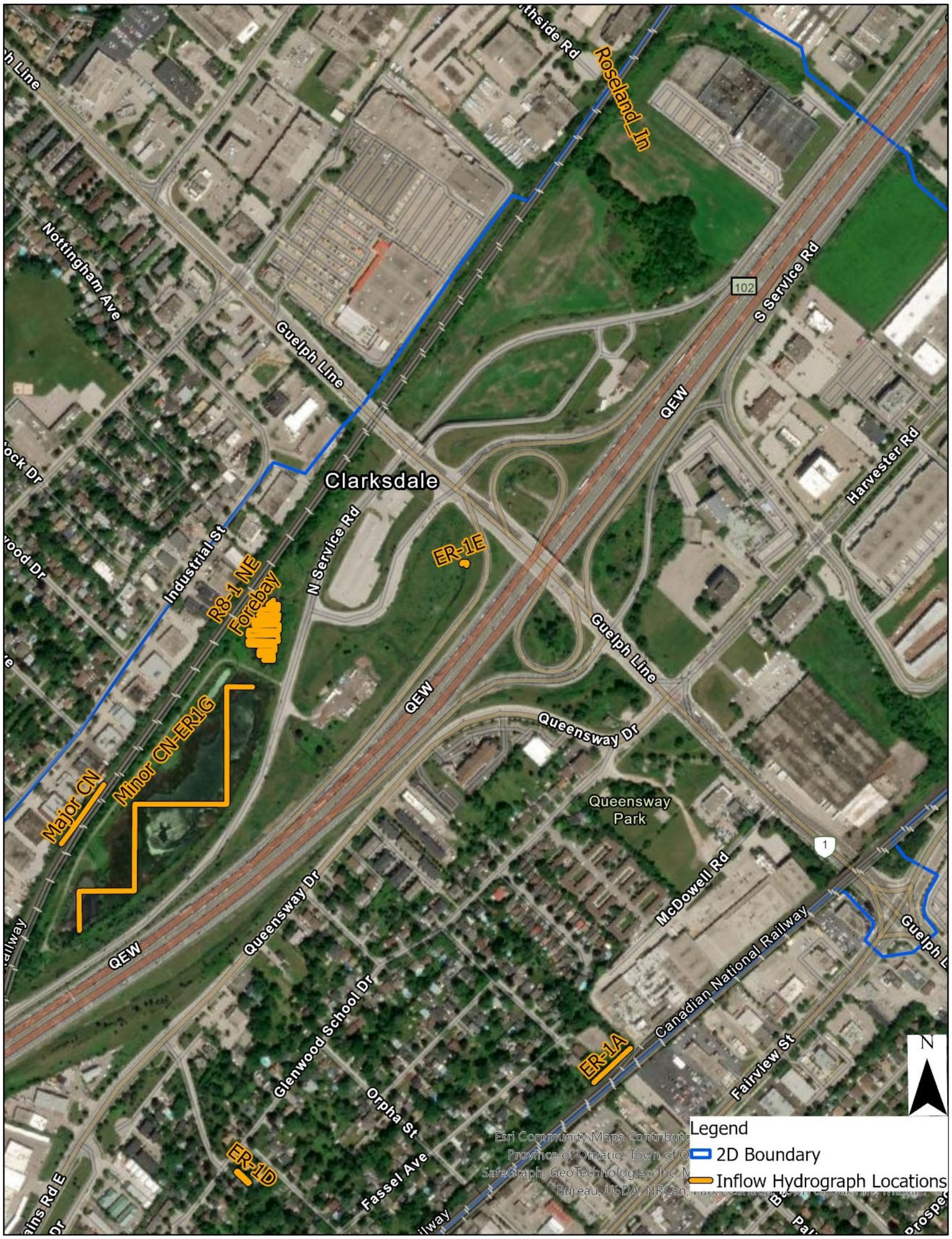

Figure 2.2.8. Inflow Locations in the HEC-RAS Model

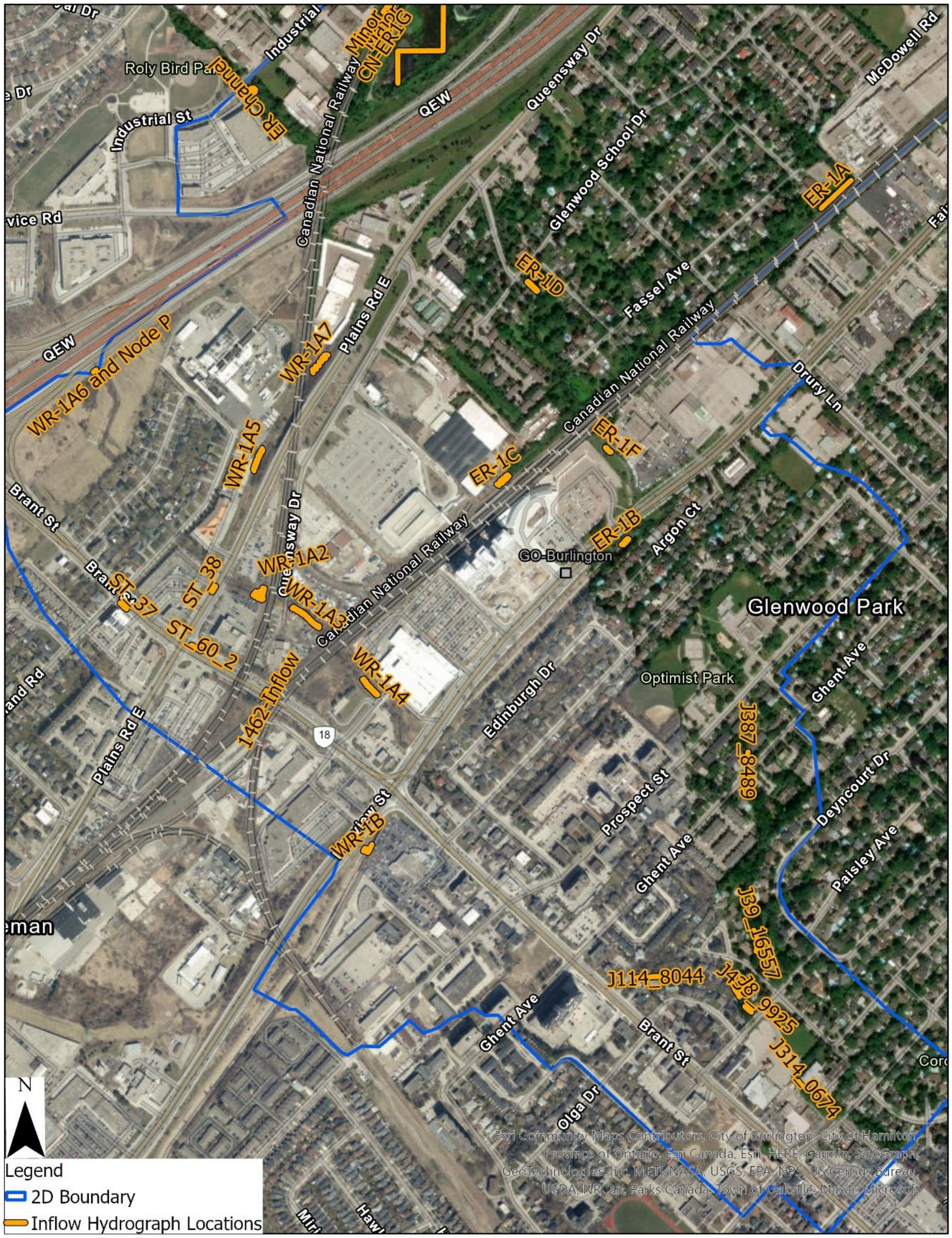

Figure 2.2.9. Inflow Locations in the HEC-RAS Model

Figure 2.2.10. Inflow into the Brant Street Underpass Subcatchment (from PCSWMM)

The Energy slope used with Manning's equation to distribute flow within the 2D mesh has been derived from the terrain data. This slope was then entered in the unsteady flow data editor at the corresponding flow location.

Based on previous discussions with CH staff, and as noted in CH's comments of January 27, 2022 (ref. Appendix B), a key concern relates to ensuring flows are consistent with Provincial Guidance (i.e., "Technical Guide: River & Stream Systems: Flooding Hazard Limit", Ministry of Natural Resources, 2002). Fundamental to that document and the conventionally applied hydraulic modelling approach (i.e., 1D steady state) is the conveyance of full flows to downstream areas, with no crediting of flow attenuation due to man-made hydraulic structures. In addition, only credited spills (i.e., that exceed 10% of the total flow and cannot be mitigated based on sufficient technical assessment) should be credited. The 2D unsteady state modelling approach inherently accounts for volume and storage losses due to both structure attenuation and spills, which then results in reduced peak flows and volumes downstream, contrary to the previously noted provincial guidance.

Notwithstanding, it has also been recognized that the majority of the flows being assessed through the subject area are spill flows from external areas, rather than direct spills from the area watercourses. CH provided further comments and recommendations with respect to this matter in its letter of July 7, 2022 (refer to Appendix B). As noted in that summary, "*strict application of current provincial guidance surrounding spill elimination poses a significant challenge for the MTSAs due to the prevalence of spills as well as their potential interactions with adjoining watersheds*". Ultimately, the agreed upon approach differs from that being applied by CH in other areas (such as the East Burlington Creeks Flood Hazard Mapping) in that it has not been recommended to re-add spill flows to downstream areas (i.e., those "losing" the spill flow). However, it was agreed that further review was warranted to confirm whether hydraulic structure attenuation was considered to be "significant" at existing crossings, and to address these losses accordingly, as necessary. As per the "Proposed Approach to Finalize Reports" (Wood, June 16, 2022), a threshold of 5% was recommended by WSP to consider where attenuation is considered "significant".

To support this effort, an alternate version (plan) of the HEC-RAS 2D model has been developed which has removed all hydraulic structures along the watercourses by manually "hydroburning" the terrain to remove them and thereby ensure a free flow pathway, consistent with the approximate dimensions of the watercourse. This approach eliminates any attenuation associated with the hydraulic structures and provides unimpeded flows, consistent with the outputs of a typical hydrologic model. This approach also allows for a comparison of peak flows from the "base" modelling (i.e., with hydraulic structures included) and the associated degree of flow attenuation at each location. For each structure, profile lines have been used to extract the total flow "in" to the structure and then the total flow "out" of the structure, considering spill flows as required. This difference has been calculated both for the base model and the hydroburned model; the resulting difference in flows has therefore considered to be attributable only to the structure, and therefore the estimated degree of hydraulic structure flow attenuation. A summary of the simulated degree of hydraulic structure flow attenuation for the 100-year and Regional Storm Events is presented in Table 2.2.1.

Table 2.2.1. Estimated Hydraulic Structure Attenuation from HEC-RAS 2D Modelling Peak Flows (m^3/s)

HYDRAULIC STRUCTURE	SCENARIO	WITH FCFS		WITHOUT FCFS	
		100-YEAR	REGIONAL	100-YEAR	REGIONAL
East Rambo – Glenwood Dr	Base Model	0	0	0	0
	Hydoburned Model	0	0	0	0
	Difference	0 (0%)	0 (0%)	0 (0%)	0 (0%)
East Rambo – CNR	Base Model	4.1	4.8	7.5	7.54
	Hydoburned Model	3.9	2.4	1.9	5.34
	Difference	+0.2 (+1%)	+2.4 (+7.9%)	+5.6 (+20.4%)	+2.2 (+7.2%)
West Rambo – Churchill Ave and Leighland Rd	Base Model	3.54	2.1	4.9	0.5
	Hydoburned Model	3.05	2.1	3.9	0.7
	Difference	+0.49 (+3.1%)	0 (0%)	+1.0 (+4.4%)	-0.2 (-1.0%)
West Rambo – Plains Road	Base Model	2.7	0.1	0.2	-0.3
	Hydoburned Model	3.3	0.2	-0.3	0.05
	Difference	-0.6 (-2.1%)	-0.1 (-0.2%)	0.5 (+1.0%)	-0.35 (-0.8%)
West Rambo – CNR Spur Line	Base Model	0	0.54	0.2	-0.4
	Hydoburned Model	-0.2	0.24	-0.3	-0.5
	Difference	+0.2 (+1.2%)	+0.30 (+1.9%)	+0.5 (+2.9%)	+0.1 (+0.6%)
West Rambo – CNR Main Line	Base Model	-0.2	-1.0	-2.2	-1.2
	Hydoburned Model	1	-0.9	-0.1	-1.1
	Difference	-1.2 (-7.1%)	-0.1 (-0.5%)	-2.3 (-11.3%)	-0.1 (-0.5%)
West Rambo – DePauls Lane	Base Model	-0.1	-0.3	0.1	0
	Hydoburned Model	-0.2	-0.4	-0.2	-0.5
	Difference	+0.1 (+0.6%)	+0.1 (+0.4%)	+0.3 (+1.5%)	+0.5 (+2.1%)
West Rambo and Hager-Rambo – Fairview\Brant	Base Model	0	-0.5	0.4	-0.7
	Hydoburned Model	0.9	-1.0	1	-0.9
	Difference	-0.9 (-2.3%)	+0.5 (+0.7%)	-0.6 (+0.9%)	+0.2 (+0.3%)

As evident from Table 2.2.1, only one (1) hydraulic structure is considered to cause "significant" attenuation (i.e., > 5% peak flow reduction); this is the East Rambo Creek crossing of the CNR. Based on the preceding results, hydrographs for the calculated attenuation flows have been extracted and then re-inserted into the HEC-RAS modelling immediately downstream of the subject structure. Subsequent hydraulic modelling results reflect this additional flow correction. No other flow corrections for spills or hydraulic structure attenuation have been included in the modelling.

2.2.2 2D MODEL RESULTS WITH FCFS CREDITED

2.2.2.1 EAST RAMBO POND AND ROSELAND CREEK SPILL ASSESSMENT

The preceding HEC-RAS modelling has been simulated using the flows presented in Section 2.1 and 2.2.1.2 with the Roseland Creek spill flow included. For clarity, the model results for the upper portion of the subject area (i.e., East Rambo Pond and the QEW; outside of the Burlington GO MTSA boundary) have been discussed separately from the lower portion of the subject area (i.e., downstream of the QEW and within the Burlington GO MTSA boundary; refer to Section 2.2.2.2 for further discussion).

It should be noted that the additional local flows have not been included for Roseland Creek downstream of the QEW, given the study focus upon the Burlington GO MTSA. As such any simulated peak flows or overland flow conveyance estimates for Roseland Creek should be noted as such and are presented for information purposes only.

Results are presented in Table 2.2.2. Key locations of interest and an inundation boundary map for the Regional Storm Event (12H AMC III) including local flows downstream of the QEW, are presented in Figure 2.2.11.

The base HEC-RAS model indicates that an overland spill does not occur from Roseland Creek along the QEW and towards the East Rambo Pond for the Regional Storm event. Spill flows rather pond within the QEW right-of-way and ultimately spill southerly.

The currently approved hydrologic modelling for the Roseland Creek watershed (2009 Flood Control Class EA; ref. Table 5.1) estimated an enclosure capacity of 18.40 m^3/s, based on the critical 2.46 m x 1.85 m box culvert link between the South Service Road and the outlet on the south side of Harvester Road), which was noted to have a 2-year storm event capacity (the primary 3.73 m x 1.85 m QEW crossing was estimated to have a 5-year capacity of 19.86 m^3/s). The results presented in Table 2.2.2 indicate a more variable capacity based on storm event, with spill flow indicated for the 100-year event with a fairly consistent simulated culvert flow of 19.8 m^3/s. An estimated flow of 20.8 m^3/s is achieved for the Regional Storm Event, likely due to the higher head associated with this storm. The modelling results indicate that no spill occurs into the East Rambo Pond from the Roseland Creek spill (ref. ID 5) since the flows have the ability to be stored in the ditches along the QEW due to the additional culverts (IDs 4, 5 and 6; ref. Figure 2.2.3 and also due to the relief provided by the southerly spill towards Queensway Drive).

The Roseland Creek spill on to the QEW Is indicated as 17.2 m^3/s for the Regional Storm Event. This spill is generally split between the storm sewer flow across QEW near Reimer Common (location 3b – 3.0 m^3/s, which includes the contributions from subcatchment ER-1E) and spill overland south from the QEW (location 3a – 9.8 m^3/s). Furthermore, some of the flow is conveyed south of the QEW towards Harvester Road underneath the guard rail barriers. It should also be noted that flow is also stored within the ditches along the QEW and within the QEW roadway due to the size of the area and confinement due to the existing concrete barriers. Approximately 2.1 m^3/s of flow for the Regional Storm would be conveyed south through the guard rail barrier opening only after the flows reach the storage capacity on the west-bound lane of QEW at location #2b shown in Figure 2.2.11. The spill from the QEW, west of the Guelph Line Ramp, along with local flow from subcatchment (ER-1A) results in surface ponding south of Glenwood School Drive.

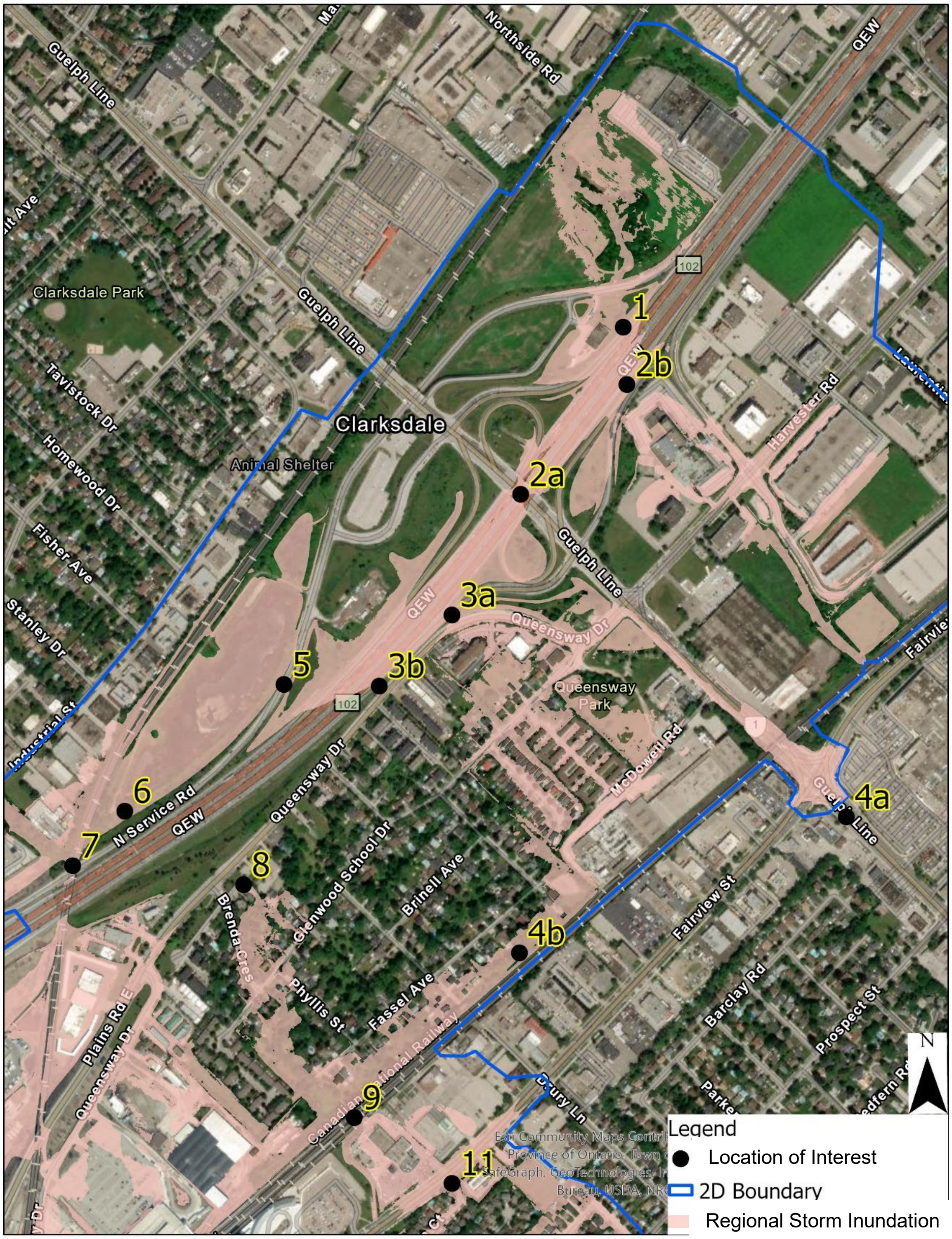

Figure 2.2.11. Locations of Interest for East Rambo Pond Area

Table 2.2.2. Simulated Peak Flow Results (m^3/s) for East Rambo Pond Area from HEC-RAS 2D Base Model (with Roseland Creek Spill Included)

ID	LOCATION (NODE)	SIMULATED PEAK FLOW (m^3/s) FOR SPECIFIED RETURN PERIOD (YEARS) OR STORM	
		100-YEAR	REGIONAL STORM 12H AMC-III
1	Roseland Creek Enclosure	19.8	20.8
2a	Roseland Creek QEW Spill at Guelph Line Underpass	11.7	17.2
2b	QEW Spill through Guard Rail Opening	0.4	2.1
3a	Overland Spill from QEW to Queensway Drive	3.5	9.8
3b	Storm Sewer flow across QEW (near Reimer Common)	2.5	3.0
4a	Guelph Line Spill at Fairview Street/CNR Underpass[1]	0	2.4
4b	Spill back to East Rambo Creek	2.3	5.8
5	Overland Spill into East Rambo Pond	0	0
6	Culvert Discharge from East Rambo Pond to East Rambo Creek	17.6	19.0
7	Spill from East Rambo Pond via CNR crossing	18.4	38.0

1. Does not include additional local flow inputs to Roseland Creek

With respect to the spill from the south side of the QEW, the results indicate that some of the flow would be directed back to East Rambo Creek along the CNR embankment, south of Fassel Avenue. Of the 9.8 m^3/s of spill under the Regional Storm Event, 5.8 m^3/s is indicated as being directed to this location (location 4b) and only 2.4 m^3/s being directed back towards Roseland Creek via the Guelph Line underpass (location 4a; does not include local flow contributions from Roseland Creek), with the balance reflecting the impacts of conveyance and storage.

Low flow culvert discharges from the East Rambo Pond to East Rambo Creek indicate a peak flow of 17.6 m^3/s for the 100-year storm event, and 19.0 m^3/s for the Regional Storm. These are generally consistent with the capacity limits developed from the previous hydrologic modelling (i.e., 17.1 and 18.5 m^3/s respectively).

As noted in Table 2.2.2, the simulated Regional Storm spill flow via CNR crossing using the HEC-RAS 2D modelling with the inclusion of inflow from Roseland Creek is 38.0 m^3/s.

2.2.2.2 *EAST AND WEST RAMBO CREEK AREAS*

Simulated peak flow results for East and West Rambo Creeks, including local flow contributions are presented in Table 2.2.3. Relevant locations are presented in Figure 2.2.12.

Simulated inundation limits and flood depths (a re-classified depth grid from RAS Mapper to commonly used depth thresholds) for the governing Regional Storm Event are presented in Figures 2.2.13 and 2.2.14. Refer to Drawings 5B and 5D for the Regional Storm Inundation limits and flood depths as well.

100-year storm event results are provided in Appendix D (Figures D1 to D2) and in Drawings 5B and 5C respectively.

It should be noted that the extents of flooding indicated for Roseland Creek downstream of the QEW do not include additional local flows and are therefore presented for information purposes only; actual flood extents would be expected to be greater due to the additional local flows.

Table 2.2.3. Simulated Peak Flow Results (m^3/s) from HEC-RAS 2D Base Model for East and West Rambo Creek

ID	LOCATION (NODE)	SIMULATED PEAK FLOW (m^3/s) FOR SPECIFIED RETURN PERIOD (YEARS) OR STORM	
		100-YEAR	REGIONAL STORM 12H - AMC-III
8	East Rambo Creek at Plains Road	20.1	21.8
9	East Rambo Creek at CNR (Node J1)	21.0	30.3
10	Hager-Rambo Diversion at Fairview (Node J)	22.6	27.5
11	Fairview Street Spill towards Argon Court and Joyce Street	0.5	6.0
12	Spill to Lower Rambo Creek Just South of Maplewood Drive	0.4	2.6
13	WRC at Plains Road[1] (Node P3)	16.4	16.4
14	Spill into the Brant Street Underpass	13.3	24.9
15	West Rambo Creek at CNR (Node P2)	16.9	20.5
16	West Rambo Creek at Fairview (Node P1)	17.2	22.7
17	Total Hager-Rambo Diversion West of Brant (Node K)	39.8	51.1
18	Spill Flow from CNR Underpass South on Brant Street	1.6	20.6
19	Spill flow onto Brant Street at Fairview Street	0	19.1

1. Riverine flow on the downstream side of Plains Road, Regional Storm may include overflow west of the WRC channel.
2. Does not include structure attenuation flows.

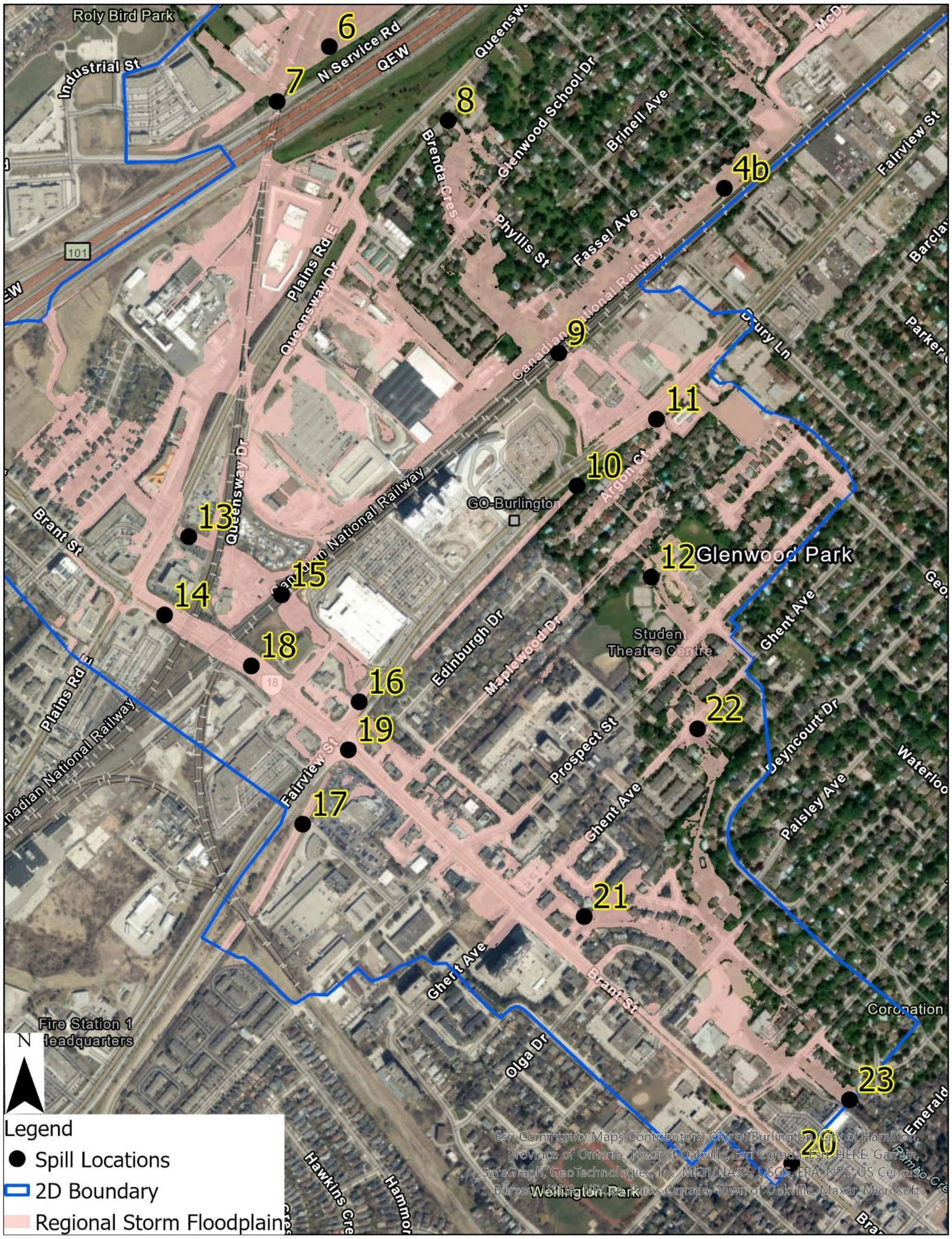

Figure 2.2.12. Locations of Interest for East and West Rambo Creeks

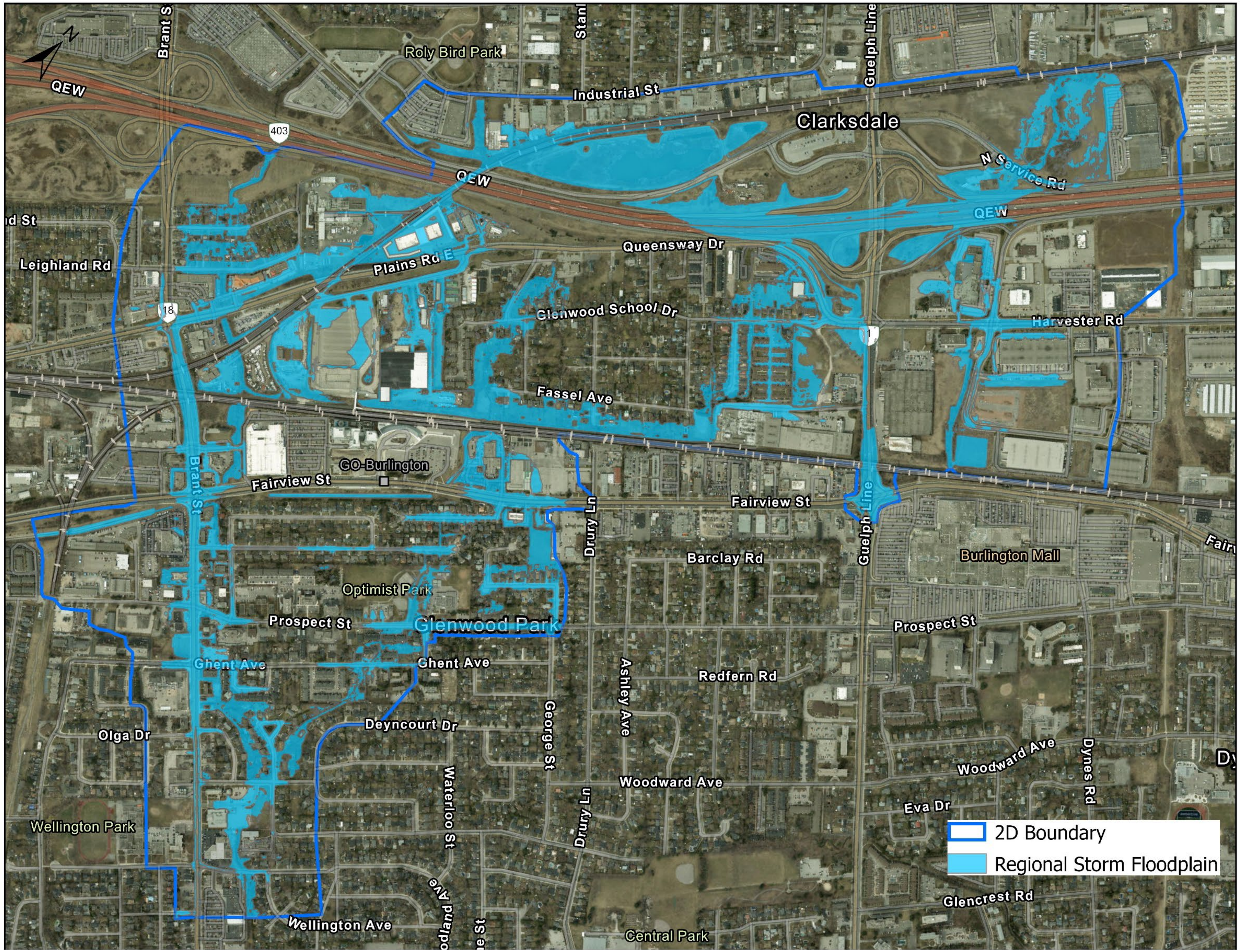

Figure 2.2.13. Simulated Inundation Limits for the Regional Storm Event (Shown to Limits of the Model)

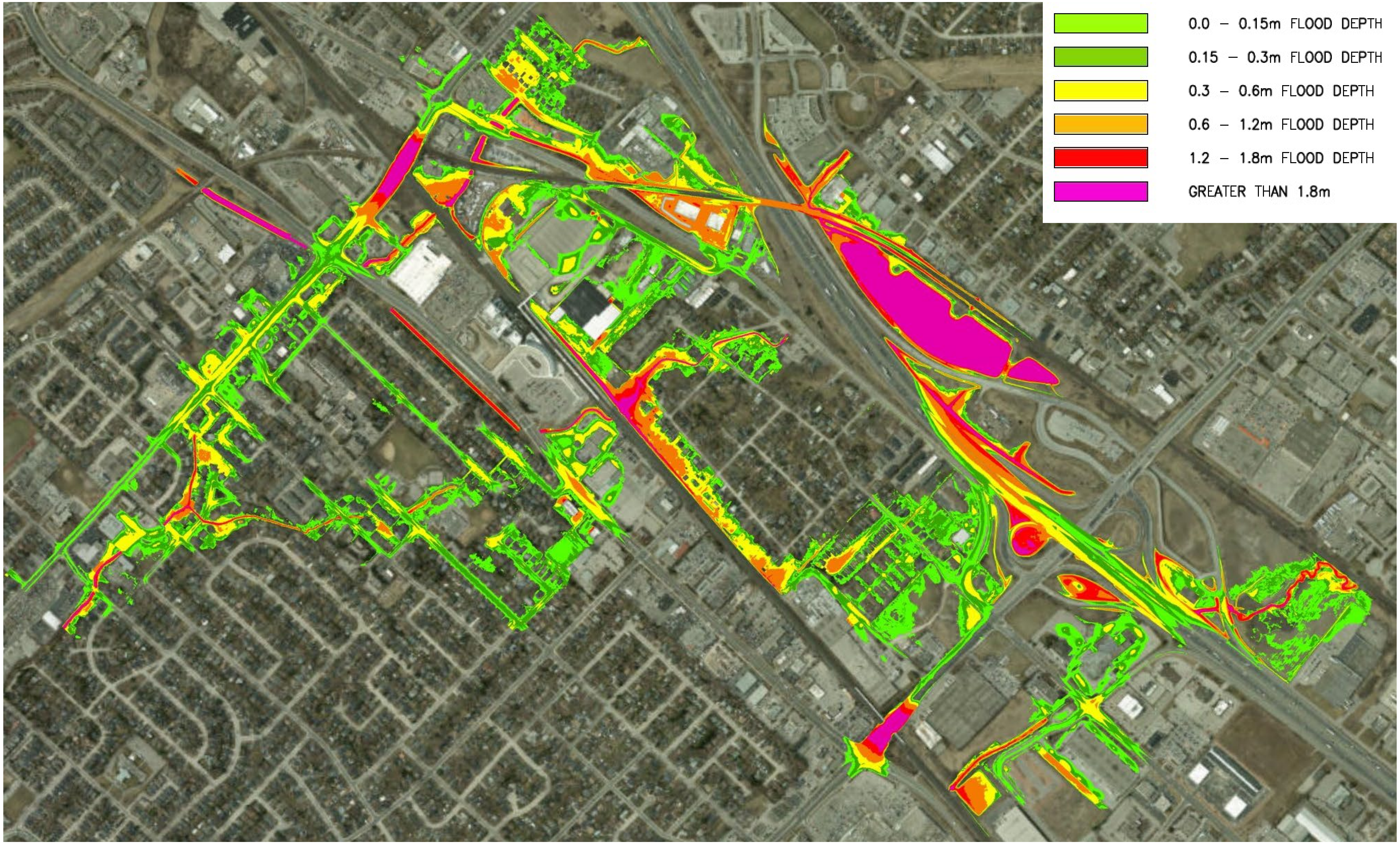

Figure 2.2.14. Simulated Flood Depths for the Regional Storm Event (Shown to Limits of the Model)

The flow just downstream of Plains Road includes flow from the 1200 mm diameter storm sewer across the QEW and along Queensway Drive (3.0 m^3/s for the Regional Storm) and the culvert discharge from East Rambo Pond (19.0 m^3/s for the Regional Storm as per Table 2.2.3); the resulting combined peak flow at location 8 is 21.8 m^3/s as per Table 2.13. The East Rambo Creek does not have sufficient channel capacity to convey the 100-year storm, nor the Regional Storm, and along with the sub-catchment ER-1D flows, spills are indicated in the residential neighborhood just north of Glenwood School Drive as flow is conveyed southerly down to the CNR ditch, where greater depth accumulations are noted. It should also be noted that the CNR is not overtopped at any location.

The CNR spill at the QEW from the East Rambo Pond splits east and west, resulting in about 8.9 m^3/s for the Regional Storm spilling onto Plains Road East and south along Queensway Drive and then finally entering the ditch along the CNR and flowing into East Rambo Creek. Further, the spill on the west side of CNR results in flooding along Leighland Road. Similar to the East Rambo Creek, the West Rambo Creek does not have sufficient channel capacity to convey flows from subcatchment WR-1A6 (local flows in the vicinity of Leighland Road) and Node P (i.e., flow immediately downstream of the QEW) for the Regional Storm and the 100-year event. This results in flooding along Churchill Avenue, Leighland Road and nearby areas further leading to a spill onto Plains Road East and Brant Street. The limiting capacity of the Plains Road enclosure is also a contributing factor. Spill flow onto Plains Road spills west and then south to the CNR underpass on Brant Street. The storm sewer in this area cannot reasonably drain all of the inflow to this location, which ponds until it reaches the spill elevation along Brant Street south of the CNR. Therefore, as per Table 2.2.3, the 2 100-year storm event (and storm events less than the 100-year event) are stored under the CNR underpass, and the spills are significantly lower for these storms when comparing inflow (Node 14) to outflow (Nodes 18 and 19).

The Brant Street spill at the south side of the 2D mesh near the Hager-Rambo Diversion Channel (Node 19) is approximately 19.1 m^3/s for the Regional Storm. It should also be noted that some of the Brant Street spill splits and further spills east and west of Brant Street, as depicted on Figure 2.2.13. Additional 2D modelling extents have been included in this area to better assess the route of this spill flow between this point and the ultimate receiver within the Downtown area (i.e., Lower Rambo Creek between Ghent Avenue and Rambo Crescent).

Peak flows are likely affected by available storage within the Burlington GO MTSA area. As evident from Figure 2.2.12 ponding areas include the area immediately south of the CNR crossing, the area around the Burlington GO north parking lot, the CNR railway embankment, and the Brant Street underpass, as well as several other locations.

2.2.2.3 *FILL ANALYSIS*

As part of the agreed Study Terms of Reference (ref. Appendix B), an impact assessment of the potential impacts of filling (due to future re-development) is required. The intent of this assessment is to determine what the impacts of filling would be, and also what degree of filling may be permissible (as compared to what should be avoided and/or minimized).

A filling analysis, conservatively assuming filling of all lands scheduled for intensification, was previously completed as part of the March 2022 version of the MTSA Flood Hazard Assessment Reporting. A summary of the methodology and results has been included in Appendix D. It has not been considered warranted to update the analysis for the current version of the hydrologic\hydraulic modelling, as the findings are generally expected to be consistent.

Overall, the results included in Appendix D suggest that West Rambo Creek would experience the greatest increases in peak flows and flood depths due to the theoretical infilling scenario. Given the uncertainty as to the timing and form of future intensification and development within the Burlington GO MTSA, it is suggested that the 2D modelling tools developed as part of the current study be applied to further assess the impacts of any proposed infilling and associated cut\fill and mitigation measures to demonstrate that proposed developments are adequately floodproofed and also do not result in any negative flooding impacts to off-site properties and areas. It is expected this will occur as part of site-specific development applications as they are proposed.

2.2.3 2D MODEL RESULTS WITHOUT FCFS CREDITED

In September 2020, WSP (then Wood) completed the "Hager-Rambo Flood Control Facilities Study Report" for the City of Burlington. The intent of that report was to document a series of supporting analyses for the three (3) major flood control facilities (West Hager, Freeman, and East Rambo Ponds), including structural, geotechnical, and hydrotechnical, to confirm that the ponds are technically sound and can reasonably be supported for crediting of their flood control function for the 2-100 year and Regional Storm Events.

CH has received comprehensive documentation from the City (February 9, 2023) which details the ownership and maintenance responsibilities (between the City, CH and MTO) of the Hager-Rambo Flood Control System facilities which include the East Rambo Pond, Freeman Pond and West Hager Pond. CH is currently reviewing the documentation to confirm that maintenance responsibilities are clearly identified and that the quantity control function of the three (3) flood control facilities will be accepted and included in the hydrologic modelling, and the associated peak flows used in hydraulic modelling and flood hazard mapping.

As such, as part of the current study and agreed Study Terms of Reference (ref. Appendix B), alternative FCF scenarios have been assessed as a form of sensitivity analysis and to assist CH in understanding the impacts of partial crediting and complete removal of these facilities. As per the TOR, this study has focused primarily on the resulting difference in flows only; updated floodplain extents for these scenarios have not been considered. CH is of the opinion that all flood hazard mapping should be based on the exclusion of FCFs in the estimation of peak flows, pending final arrangements for operation and maintenance of FCFs.

Notwithstanding, flow routing through the East and West Rambo Creek areas requires application of the previously described HEC-RAS 2D modelling, however the "hydroburned" version of the modelling has been applied for flow routing purposes to ensure no loss of flows, consistent with the approach applied for the modelling results with FCFs credited.

The results from this modified scenario for the East Rambo FCF in HEC-RAS have been combined with those for the external areas (West Hager and Freeman Ponds) modelled in VO to assess the overall resulting differences in flows. Overall results are included in the hydrologic modelling discussion (refer to Section 2.1.6)

The removal of East Rambo FCF within the 2D HEC-RAS model has been achieved by using a storage area within the 2D mesh to effectively "block out" the potentially available storage. A conceptual 10 m wide channel has been incorporated to ensure flows can be conveyed to the 3.0 m x 1.5 m box culvert (low flow outlet). The overflow via the CNR crossing of the QEW remains unchanged. The HEC-RAS 2D geometry that was developed by CH (ref. email Irwin-Senior, January 27, 2022) for this scenario was used for this model run. The extents of the storage area are presented in Figure 2.2.15.

As noted previously, the combined results of the routed peak flows for this scenario are presented along with the external flows from the hydrologic (VO) modelling in Section 2.1.6; reference is made to that section for further details and findings.

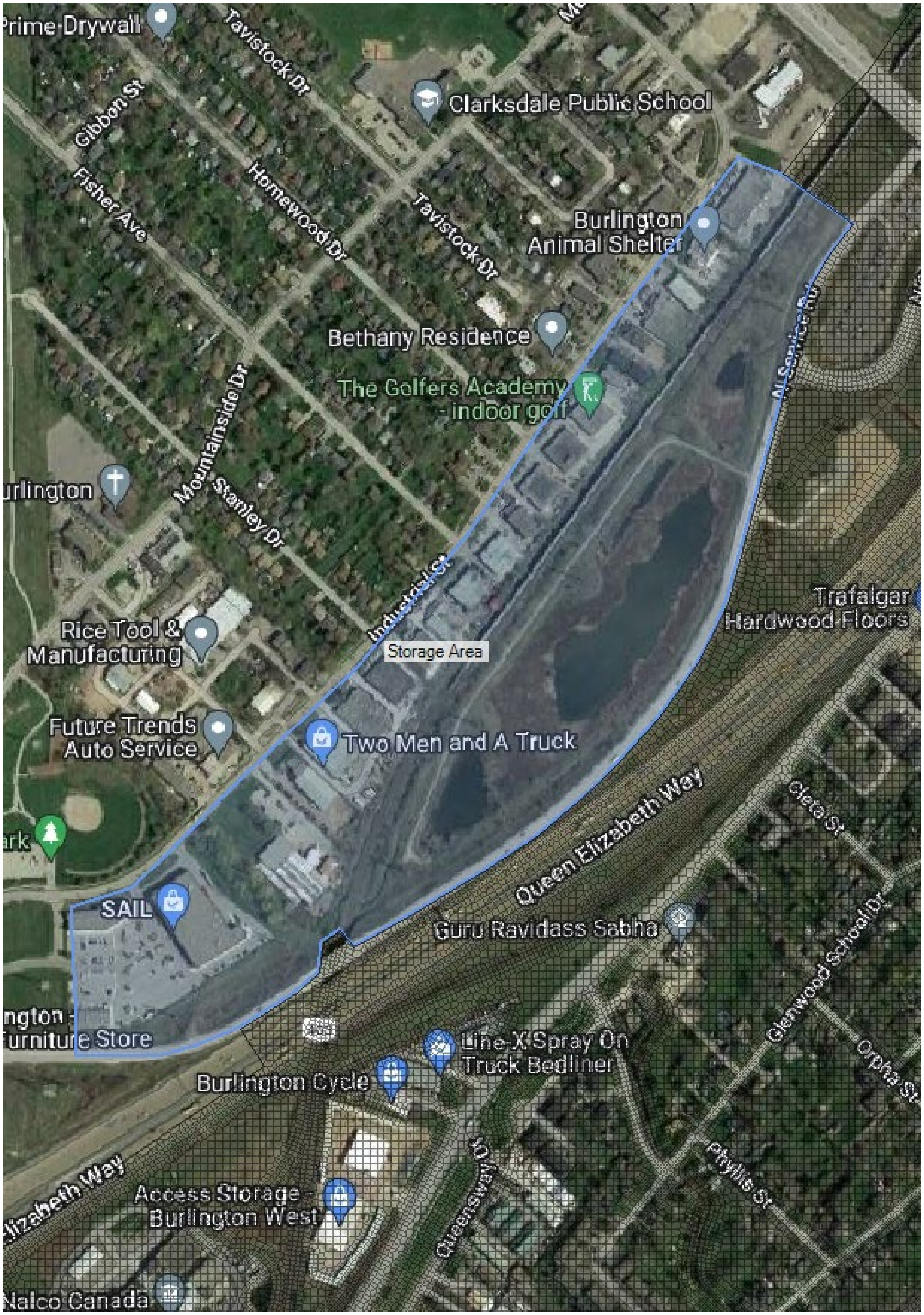

Figure 2.2.15. Modelled Storage Area in HEC-RAS for FCF Removal Scenario

2.3 1-DIMENSIONAL HYDRAULICS

2.3.1 MODEL DEVELOPMENT

A 1-dimensional hydraulic model (HEC-RAS) of the study area, including East and West Rambo Creeks, the Hager-Rambo Diversion Channel, and Hager Creek was previously prepared using HEC-2 (Philips Planning and Engineering, 1984). More recently, Conservation Halton (CH) prepared a hydraulic model of the channel using HEC-GeoRAS, based on a 2002 digital elevation model (DEM). CH further updated the model, including incorporating hydraulic structures based on data available from the 1984 HEC-2 model. A memorandum summarizing the model updates was provided to WSP by CH as part of the Urban Area Flood Study (March 18th, 2014 (2015)), including a number of disclaimers related to its use (provided in "as-is" condition). The memorandum is included as part of Appendix B of the Phase 1 study report.

The HEC-GeoRAS model provided by CH was updated as part of the Phase 1 study, as noted in that reporting. The updates included use of the Region of Halton's 2015 DEM, field inspection and verification of hydraulic structures, refinement of Manning's Roughness Coefficients, and other modelling adjustments based on subsequent review comments from CH.

Since the current HEC-RAS 2D mesh extends west to the intersection of Fairview Street and Brant Street, WSP previously recommended that the 1D model be truncated to include only Hager Creek and the Hager-Rambo Diversion Channel where it ties in with the 2D model. WSP also recommended that the current 2D model be used for floodplain mapping purposes within the upstream area (East and West Rambo Creeks) considering the complex nature of spills and the fact that the 2D model results would be more accurate than 1D steady state results.

Notwithstanding, based on subsequent discussions with CH (January 26, 2022, meeting), CH noted that in order to ensure compliance with Provincial Guidelines (MNR, 2002) some form of 1D model was still required for East and West Rambo Creeks in order to support the delineation of floodplains and identify potential spills. Beyond the top of bank\spill point, spill arrows are to be indicated, with the resulting flood limits defined by the 2D modelling described previously.

The 1D model developed during Phase 1 of the study has been updated using the same 2018 LiDAR data used for the 2D modelling. Similar to 2D modeling, HEC-RAS Version 6.3.1 has been used for all 1D modeling updates (most current and stable version of the program). Approximately 33 cross sections were adjusted to better represent the geometry and structures within that model.

As part of the collaborative approach between WSP and CH on this project, CH staff undertook an update to the 1D hydraulic modelling for the subject area (based on WSP's March 2022 submission), as provided to WSP May 19, 2022. CH undertook a complete revision to the modelling, including all new cross-sections cropped to the riverine valley "top of bank". CH also included the application of "lids" for some portions of enclosures for Hager Creek and the Hager-Rambo Diversion Channel which were considered to better model in this manner than using the typical hydraulic structure routines (i.e. bridges\culverts) in HEC-RAS. All lateral structures (included in the original 2015 modelling) have been removed from the model. WSP has also undertaken an update of Manning's Roughness coefficients to address CH comments.

Structure dimensions have been verified from record drawings supplied by the City (relevant drawings are included in Appendix C). Elevations have been converted to the CGVD:2013 datum accordingly. A vertical datum conversion factor of 0.43 m has been used for the culvert inverts if the drawings/survey were in the previous

CGVD28:78 datum as opposed to the current study (CGVD:2013 datum). In some cases, the elevation from the LiDAR dataset has been used to determine the expected culvert invert, given the high resolution of the data.

A combination of the previously described "hydroburned" model flows (East and West Rambo Creek area) and VO flows (East and West Hager Creeks) have been applied in the 1D model, as described in Section 2.1. Note that all modelling results include crediting of the FCFs. Flow profiles for the "no FCF" scenario have been included in the 1D HEC-RAS modelling but have not been discussed or presented in the current reporting.

2.3.2 MODEL RESULTS WITH FCFS CREDITED

2.3.2.1 EAST AND WEST RAMBO CREEKS

As noted previously, due to the extensive simulated spills and complex hydraulics, the results of the 1D hydraulic modelling are not considered suitable to reasonably delineate the floodplain extents for East and West Rambo Creeks. Notwithstanding, CH has requested that the 1D model include these areas to ensure compliance with Provincial guidance. The 1D modelling results are therefore considered to represent the primary 1D channel flood limits, which generally reflects the limits of the channel to the high points along each overbank, which are limited by levees within the modelling. The delineated 1D flood limit\top of bank as presented on Drawings 5A-5D should be interpreted; accordingly, flood hazard limits beyond the primary channel are better defined by the 2D modelling discussed in Section 2.2.

Notwithstanding the preceding, the simulated 1D hydraulic modelling results have been used to estimate the conveyance capacity of hydraulic structures within the primary Burlington GO MTSA. The ratio of the simulated peak headwater (HW) to structure diameter or height (D) has been extracted along with the estimated freeboard between the peak headwater and the spill elevation (typically top of road or railway at the crossing). The results are presented in Table 2.2.4. A HW/D ratio greater than 1 indicates that the culvert would be submerged during the storm event; a greater ratio indicates a larger degree of submergence.

The results presented in Table 2.2.4 indicate that with the exception of the Hager-Rambo Diversion crossing at Fairview, all of the crossings would have HW/D ratios greater than 1.0, indicating surcharge. Crossings on West Rambo Creek generally indicate the highest HW/D ratios, and also frequently negative freeboard values. This likely reflects the high inflows associated with the East Rambo Creek spill via the CNR, which would not have been considered as part of their original design.

Table 2.2.4. Hydraulic Structure Capacity Evaluation Summary (1D Modelling)

ID	TYPE	WATER COURSE	LOCATION	100-YEAR			REGIONAL STORM		
				WSE (m)	HW/D	FB (m)	WSE (m)	HW/D	FB (m)
12	Culvert (1RCB)	ER Creek	Glenwood School Drive	100.72	1.10	+0.13	100.61	1.04	+0.24
13	Culvert (1 RCB)	ER Creek	CNR, just u/s of Fairview Street	99.06	1.12	+1.01	99.37	1.25	+0.70
14	Culvert (1 RCB)	HR Diversion	Fairview Street	95.04	0.54	+1.96	95.65	0.77	+1.35
19	Culvert (1 RCB)	WR Creek	Parking Lot, just u/s of Plains Road E	100.72	1.91	-0.47	100.74	1.92	-0.49
21	Culvert (1 RCB)	WR Creek	Plains Road E	100.45	2.18	-0.52	100.52	2.22	-0.59
22	Culvert (1 RCB)	WR Creek	CNR, just d/s of Plains Road E	100.21	2.28	+0.38	100.47	2.44	+0.12
25	Culvert (1 CMP)	WR Creek	CNR, just u/s of De Paul's Ln	99.78	1.76	-0.20	99.94	1.70	-0.04
26	Culvert (1 RCB)	WR Creek	De Paul's Ln behind Walmart	96.26	1.66	-0.20	96.63	1.93	-0.57

1. Negative freeboard indicates overtopping, positive freeboard indicates depth below overtopping depth.
2. Some crossings may indicate HW/D > 1 (submerged/surcharging) but have a positive freeboard depending on the estimated embankment height to the spill point.

The designed flow capacity is typically set based on design standards, which in turn depend on the ownership of the road (City, Region, Province, or Railway) as well as the classification of the road (Local, Collector, Arterial, Highway), and also typically the span of the structure. The Ministry of Transportation's 2008 Highway Drainage Design Standards are a common resource for roadway conveyance criteria; Section WC-1 provides typical standards which may range from a 10-year storm event for a local road with a span less than 6 m, to a 100-year storm event for a freeway or urban arterial with a span greater than 6 m.

There are no commonly accepted design standards for railway crossings in Ontario. Based on WSP's experience, the American Railway Engineering and Maintenance-of-way Association (AREMA) Manual for Railway Engineering (2019) has been used for the hydraulic design of rail crossing on other projects in Ontario.

For Regulated (by Conservation Authority) watercourses, consideration of the Regional Storm is however still required as indicated by Conservation Halton.

The majority of the hydraulic structures noted previously have spans less than 6 m and would thus have design standards ranging between a 10 and a 50-year storm events based on the MTO criteria. Freeboard for culverts (as per MTO Section WC-7) is typically 0.3 m or greater for local roads and 1.0 m for freeways, arterials, and collectors, measured from the edge of travelled right-of-way. Clearance varies depending on the type of roadway and vulnerability, but varies between 0 and 1.0 m. Depending on the degree of ground cover and type of road, clearance may be more restrictive than freeboard.

Overall, the preceding results confirm that there are several deficient hydraulic structures that may result in worsened spill conditions or elevated flood elevations and associated greater floodplain extents. This should be considered as part of future drainage improvements in the area by associated infrastructure managers, including not only the City, but also Halton Region, MTO, and CNR among others.

2.3.2.2 *HAGER-RAMBO DIVERSION CHANNEL*

The simulated floodplain extents for the Hager-Rambo Diversion Channel are presented in Drawing 5E. Unlike the results for East and West Rambo Creeks (which are typically indicative of top of bank), the 1D extents for the Hager-Rambo Diversion channel reflect full flows and are considered representative of the flood hazard along this section of channel. As evident from Drawing 5E, the full flows are generally conveyed by the diversion channel, with the floodplain extents generally limited to the channel block. A spill is indicated for the Regional Storm Event between Brant Street and the rail crossing due to the inclusion of full flows (i.e., application of the "hydroburned" modelling and addition of spill flows as required). Under existing conditions, the spill would occur upstream of this point in reality, as presented as part of the 2D modelling results. As such, this section of channel has sufficient capacity for flows which remain within the system, meaning the spill would only occur if all upstream spills were eliminated without improving downstream capacity (which would generally not be recommended).

In the current 1D hydraulic modeling, the structure at Maple Avenue has been modelled with a "lid", and exhibits overtopping for the Regional Storm Event. The structure at Thorpe Road shows a minor overtopping at the internal cross-sections of HEC-RAS. A spill is indicated at Maple Avenue; no spill is indicated at Thorpe Road. The upstream hydraulic structure at the railway corridor is able to convey the flows without overtopping.

2.3.2.3 *HAGER CREEK*

The simulated floodplain extents for Hager Creek (Freeman Pond to the Diversion Channel) are presented in Drawing 5F. Consistent with the results presented in the Phase 1 study, upstream of the CNR an extensive floodplain is indicated along the eastern overbank, which would be expected to result in spill flow towards the CNR underpass at Plains Road. Spill extents have not been mapped as part of the current study (as per the TOR included in Appendix B), as the extents are not expected to impact the primary Burlington GO MTSA study area.

The City of Burlington has reportedly recently completed an analysis of the flood conditions affecting Leighland Park as part of the proposed park improvement project. Findings from that analysis (2D) may more accurately reflect the inundation/flooding anticipated in that area. WSP has not received or reviewed the study in question; the City of Burlington should be contacted for any further details related to this work.

Downstream of the CNR, the 100-year floodplain extents are contained to the channel area. The Regional Storm floodplain extents are however indicated as extending beyond the primary channel and in the floodplain. Spills are indicated in Drawing 5F for the Regional Storm upstream of Fairview Street. The simulated results indicate that the structures along Hager Creek have the capacity to convey the 100-year design storm event however all structures would be overtopped for the Regional Storm.

3 DOWNTOWN AREA

3.1 HYDROLOGY

3.1.1 MODEL DEVELOPMENT

3.1.1.1 HYDROLOGIC ELEMENTS

Prior to the completion of the Phase 1 Study, there were no hydrologic or hydraulic models available for the Downtown area (i.e., Lower Hager or Lower Rambo Creeks). As part of the Phase 1 study, new hydrologic and hydraulic modelling was developed, using an integrated approach in PCSWMM.

Subcatchment boundaries were developed on the basis of the trunk storm sewer network (generally pipes 600 mm in diameter and greater), and topographic data supplied by the City of Burlington at that time (2015 elevation data from the Region of Halton). Given the resolution of the hydraulic system, subcatchment discretization is also relatively resolute. A total of 182 subcatchments (440.8 ha) were discretized for the PCSWMM model, with an average drainage area of 2.4 ha +\-. Details on hydrologic model development and parameterization, undertaken as part of the Phase 1 Study, are presented in the Phase 1 Study Report (September 2020).

Consistent with the approach applied for the Burlington GO MTSA, updated 24-hour SCS Type II design storms, using the City of Burlington's current (2020) rainfall intensity-duration-frequency (IDF) curves have been employed for the estimation of the 100-year return period peak flow.

As part of the current (Phase 2) study, slight modifications to subcatchments were completed to reflect the updated Burlington GO MTSA boundary, which in turn altered the Downtown area study boundary. Updated subcatchment boundaries are presented in Drawing 6 (attached). The current modelling includes a total of 191 subcatchments (average area of 2.3 ha +\-).

In addition, based on the findings of the future land use assessment completed for the Burlington GO MTSA (refer to Section 2.1.3), it was determined that the future intensification land use scenario (i.e. assumption of 90% imperviousness for all development areas) resulted in the highest peak flow rates for the majority of the system for the estimation of frequency flows (2-100 year storm events), while the actual existing land use scenario was resulted in the highest peak flows rates for the estimation of the Regional Storm Event. To ensure consistency, the same assumptions have been applied for subcatchments within Lower Rambo Creek which are part of the Burlington GO MTSA (i.e., along the Brant Street corridor). As such, the imperviousness for these subcatchments have been developed using the preceding approach, and the associated values presented previously in Table 2.1.5.

Notwithstanding the preceding, CH completed a separate analysis of the supplied hydrologic modelling files for the Downtown area and included these findings in its comments of January 23, 2023 (refer to Appendix B; and Tables 2 to 4 specifically). CH's results indicate that the Future Intensification scenario does in fact generate the largest peak flows for the Regional Storm Event, albeit the differences are considered by WSP to be nominal. Differences in Table 3 of CH's summary are generally between 0.01 and 0.02 m^3/s, or less than 0.1%. Differences in Table 4 of CH's summary are generally between 0.02 and 0.05 m^3/s, or 0.1 to 0.2%. Given the minimal difference, WSP's previously proposed flows and approach have been maintained.

In its comments of January 23, 2023, CH also noted that the current study has not assessed or verified a Stormwater Management Strategy for Downtown Burlington. As per the approved Terms of Reference for this study (refer to Appendix B) such an assessment was not part of the approved scope. As documented in the Phase 1 Study (September 2020), re-development in the area was "not expected to result in any observable change in impervious coverage, given the existing urbanized/developed nature of the downtown study area". As such, the City of Burlington's requirements for quantity controls (as per the City's approved Stormwater Management Guidelines) were considered sufficient, particularly given its requirements for over-control for areas discharging directly to the storm sewer system (as would be expected to be the case for the majority of the sites in the downtown). The effectiveness of such controls has however not been assessed as part of the current study, for the reasons previously noted. Given the concerns identified by CH with respect to the Burlington GO MTSA and potential hydrograph timing effects, this may require future consideration to ensure that any on site quantity controls do not in fact result in unintended increases in peak flows to Lower Rambo Creek. Notwithstanding, any consideration of uncontrolled discharge to area storm drainage systems (i.e. the storm sewer system) would need to consider local capacity constraints and potential impacts from such an approach and balance these against simulated impacts to watercourse receivers.

It is recommended that the impacts of on-site quantity controls be assessed on a case-by-case basis, consistent with the approach currently applied by the City of Burlington, and the approach recommended for the Burlington GO MTSA (as per Section 2.1.3.2). It is recommended that the modelling tools developed as part of this study be applied for future assessments accordingly. Such assessments could either be completed by representatives of the private developer, or by a representative of the City. In either case, modelling updates should be completed cumulatively; a "current' set of modelling files should be maintained to ensure that new developments consider the assessments and results of previous developments.

Based on the preceding results, it is conceivable that some sites may not require SWM quantity controls for some or all of the events assessed (i.e. uncontrolled discharge is more appropriate to mitigate peak flows to downstream receivers). Future assessments will need to determine the most appropriate approach for the full suite of the 2 through 100-year storm events, as well as the Regional Storm Event. In addition, where sites discharge to intermediate conveyance systems (i.e. local storm sewers) the potential negative impacts to these systems from uncontrolled discharges may need to be balanced against the simulated impacts to the downstream receivers\watercourses.

If determined to be required, considerations around crediting of Regional Storm Controls, and particularly on private property would require further consideration, as noted in Section 2.1.3.2.

3.1.1.2 *HYDRAULIC ELEMENTS*

As an integrated hydrologic/hydraulic model, PCSWMM also requires that routing and conveyance elements be included explicitly. Given the urbanized nature of the Downtown area, this generally includes urban drainage components (i.e., storm sewers and roadways), as well as some riverine components (open channels/creeks). All hydraulic elements are modelled as 1-dimensional conduits in the modelling. The minor (storm sewer) and major (roadway) systems are linked through bottom draw orifices at junction nodes, sized to represent the number of inlets. Further details are provided within the Phase 1 Study Report (September 2020).

With respect to riverine (open channel/creek) sections within the Downtown Hub, a hydraulic model was previously developed as part of the Phase 1 study using HEC-GeoRAS. This modelling was completed based on elevation data supplied by the City of Burlington (2015 elevation data from the Region of Halton in geodatabase format: more recent than CH source). Ultimately, the modelling developed in HEC-GeoRAS was imported into PCSWMM to represent the hydrologic routing of open channel areas and connected to the hydraulic modelling for the urban areas (i.e., storm sewer outfalls, major overland flow route spills to watercourses).

Although the updated surface hydraulics completed as part of the current (Phase 2) study will use the more current 2018 Halton LiDAR DTM, the PCSWMM modelling has not been updated for the current study, given that it is being used for hydrologic modelling only. It is noted that the PCSWMM model remains in the CGVD28:78 vertical datum, whereas the updated DTM data uses the CGVD:2013 datum. As such, boundary conditions, where necessary, have required updating accordingly to ensure consistency.

Two different versions of the system hydraulics were originally modelled in PCSWMM. One version excluded all hydraulic structures (i.e., roadway culverts and bridges), while the other included these features. For the current study purpose, the first version has been employed for all modelling given that it is being applied to develop estimates of riverine peak flows (i.e., input to the HEC-RAS modelling), in order to not include the storage available behind structures, and thus attenuate flows. This approach is consistent with current Provincial guidance and the comments received from Conservation Halton (ref. September 12, 2017, letter).

As noted in the Phase 1 report, the “no structures” modelling also includes the removal of the Blairholm Avenue enclosure, as requested by CH. The longer enclosures further downstream along Lower Rambo Creek (Caroline to James Street) and Lower Hager Creek (Elgin Street to Lake Ontario) have been left in the model given that they are less likely to be replaced or updated in the future.

Further to the collaborative nature of this assessment between WSP and CH, the current iteration of the PCSWMM modelling also includes additional hydraulics updates completed by CH (as per the modelling files received May 19, 2022, and associated comment letter; refer to Appendix B for further details). CH’s edits primarily related to additional modifications necessary to ensure no inadvertent loss of flow from the modelling, and refinements to overland spill pathways. CH also noted a preference that spill flows be added into the PCSWMM modelling where they occur, such that PCSWMM routes and conveys these spill flows to their ultimate riverine receivers. The resulting updated flows reflect this approach.

As part of the recent modelling updates, WSP has revised all weir flow coefficients to a standard metric value of 1.75. In addition, any spill hydrograph inputs from the Hager-Rambo Diversion Channel area (as described in Section 2) have been revised accordingly. Where necessary, additional surcharge depth has been added to nodes to ensure no loss of flow under the revised modelling.

3.1.2 MODELLING RESULTS

Given the resolution and scope of the developed hydrologic/hydraulic modelling, a detailed summary of hydrologic modelling parameters has been presented in Appendix F. A summary of simulated flows at key riverine nodes (Lower Hager and Lower Rambo Creeks) modelling is presented in Table 3.1.1. The flows presented within Table 3.1.1 also do not include any spill flows from the Hager-Rambo Diversion Channel; these results are presented in Table 3.1.2.

In general, simulated 100-year and Regional Storm Event flows are relatively consistent in magnitude with the exception of smaller upstream areas, and towards the downstream limits (i.e. at Lakeshore Road) where the difference becomes greater. The governing (i.e., Regulatory) flow depends on the location selected, however the 100-year storm event tends to be the Regulatory event in this area for the “without spills” scenario.

In addition to Table 3.1.1, consideration also needs to be given to the additional spill flows from the Hager-Rambo watershed via Brant Street, and the more minor spill via Argon Court, as identified by the HEC-RAS 2D hydraulic modelling completed for this area (as per Section 2.3).

Table 3.1.1. Simulated Peak Flows (m^3/s) at Riverine Nodes of Interest (Downtown Area) without Spill Flows from Hager-Rambo Diversion Channel

WATERCOURSE	LOCATION AND MODEL ID	DRAINAGE AREA (ha)[1]	100-YEAR	REGIONAL
Lower Hager Creek	Baldwin Street (J752.2523)	36.5	6.71	4.59
	Birch Avenue (J426.0621)	44.5	7.29	5.63
	Caroline Street (J306.062)	51.8	7.36	6.08
	Ontario Street (J97.03847)	76.4	8.93	7.42
Lower Rambo Creek	Brant Street Underpass	7.8	2.74	1.14
	West Branch Upstream of Blairholm Ave (J114.8044)	22.7	2.91	1.35
	East Branch at Ghent Avenue (J387.8489)	25.5	7.05	3.67
	East Branch at Courtland Place (J39.16557)	30.0	7.04	4.30
	Confluence of East and West Branches (J54.32442)	80.7	9.87	5.63
	Blairholm Ave - Upstream (J8)	90.8	17.44	12.77
	Blairholm Ave - Downstream (J433.9925)	100.2	19.37	14.12
	Victoria Avenue (J314.0674)	109.7	20.36	14.78
	Caroline Street (J2.929314))	141.5	20.73	16.77
	James Street (J552.0832)	151.5	26.21	20.96
	Martha Street (J498.3497)	219.3	32.44	26.38
	South of Waterfront Trail (J419.5095)	224.0	32.88	27.05
	Lakeshore Road – Upstream (J149.5095)	227.2	37.94	28.40
	Lakeshore Rd Downstream / Lake Ontario (J59.22536)	259.1	42.63	32.26

Note: 1. As estimated from PCSWMM. Note that due to interconnectivity of minor and major systems, actual representative drainage area based on flow splits may differ somewhat.

In order to accurately assess the flood hazard limits within the HEC-RAS 2D modelling, the local inflows for the Brant Street underpass from the PCSWMM modelling were added into the HEC-RAS 2D modelling, for the Regional Storm Event only, given that the local flows would be expected to contribute to the overall spill flows down Brant Street. As such, for the simulation of storm events which result in spills along Brant Street, the underpass area must be removed in the PCSWMM modelling to avoid double-counting flows. In addition, the HEC-RAS 2D model includes an approximation of drainage via the sag point catchbasins and associated gravity storm sewer; this flow must also be considered in the PCSWMM modelling for the simulation of the "with spills" scenario. Where spills via Brant Street are not expected the PCSWMM modelling remains consistent with that used under this base conditions.

As per the direction of CH (ref. May 19, 2022, comments), spill flows should be added into the PCSWMM modelling at the location at which they occur. As such, spill flows have been added at up to three (3) different locations depending on the storm event in question:

- Spill flow at Brant Street has been added on the major system high point on Brant Street (Node 3684-S)
- Drainage from the underpass sag point gravity storm sewer has been added to the corresponding point in the minor system (Node 3684)
- Spill flow from Argon Court to Lower Rambo Creek has been added to the first open channel node on the east branch of Lower Rambo Creek, downstream of Maplewood Drive (Node J106.8931)

These additional flows have been added to the PCSWMM modelling as external time-varying hydrographs. Peak flows are presented in Table 3.1.2 for both the with and without FCF scenarios. All spill flows are from the "base" hydraulic modelling described in Section 2.3.

Table 3.1.2. Simulated Spill Flow (m^3/s) from Hager-Rambo Diversion Channel (HEC-RAS 2D) to Lower Rambo Creek (PCSWMM)

LOCATION	SCENARIO	100-YEAR	REGIONAL
Overland Flow Spill at Brant Street	With FCF	0	19.1
	Without FCF	20.7	23.4
Storm Sewer Flow at Brant Street Sag	With FCF	N/A[1]	5.2
	Without FCF	5.2	5.3
Spill at Fairview\Argon Court	With FCF	0.4	6.4
	Without FCF	5.8	6.4

Note: Storm sewer flow for this scenario assessed via PCSWMM modelling rather than representation in HEC-RAS modelling due to lower estimated flows.

As evident, only a minor spill flow is indicated at Argon Court for the 100-year storm event with FCFs credited, and no spill is indicated at Brant Street for this event. For the Regional Storm Event with FCFs credited, much larger spills are indicated both at Argon Court and at Brant Street.

As would be expected, if FCFs are not credited, spill flows are uniformly increased, with the exception of the Regional Storm spill flow at Argon Court.

The resultant combined peak flows incorporating spills are presented in Table 3.1.3. Note that results have been included for Lower Rambo Creek only, since all spill flows would be directed to this watercourse (none to Lower Hager Creek).

For the "with FCFs" scenario, the results for the 100-year storm event with spill are identical to those without spills. This is attributable to hydrograph timing. Due to the much larger contributing drainage area, the peak flow within East Rambo Creek peaks much later than that in Lower Rambo Creek, and as such the spill flow is not coincident with the peak from the local drainage systems. The minor spill magnitude also does not result in the governing peak flow for the watercourse, which is due to local inflows.

For the "with FCFs" scenario, the results for the Regional Storm Event, the spill flows are notable and result in increased flows within Lower Rambo Creek. For the east branch of Lower Rambo Creek, hydrograph timing is again a factor, as the spill flow occurs much later than the local peak flow. However, in this case due to its magnitude, the spill flow becomes the dominant flow in the watercourse (larger than local flows) for a large portion of the upstream area (approximately to James Street). Further downstream, the spill flows continue to dominate, with notable increases to the previous Regional Storm flows (without spills) and resulting in flows larger than the simulated 100-year storm event in all locations other than the most downstream node at Lake Ontario. In some cases, due to routing effects, slight reductions in peak flows are indicated. However, for consistency full flows would be applied for the subsequent hydraulic modelling in Sections 3.2 and 3.3.

The results for the no FCF scenario are presented for information purposes only; these flows have not been used for the subsequent hydraulic modelling. The simulated peak flows with FCFs have been applied for the estimation of the flood hazard limits through hydraulic modelling, as described further in Sections 3.2 and 3.3.

Table 3.1.3. Simulated Peak Flows (m^3/s) at Riverine Nodes of Interest (Lower Rambo Creek) including Spill Flows from Hager-Rambo Diversion Channel

LOCATION AND MODEL ID	WITH FCFS CREDITED		WITHOUT FCF CREDITING	
	100-YEAR	REGIONAL	100-YEAR	REGIONAL
West Branch Upstream of Blairholm Ave (J114.8044)	2.91	12.25	7.33	13.56
East Branch at Ghent Avenue (J387.8489)	7.05	8.24	7.05	8.53
East Branch at Courtland Place (J39.16557)	7.04	8.65	7.03	8.93
Confluence of East and West Branches (J54.32442)	9.87	20.79	13.61	22.31
Blairholm Ave – Upstream (J8)	17.44	30.59	22.40	33.72
Blairholm Ave – Downstream (J433.9925)	19.37	31.59	22.38	35.11
Victoria Avenue (J314.0674)	20.36	34.74	24.53	38.51
Caroline Street (J2.929314))	20.73	34.55	23.94	38.36
James Street (J552.0832)	26.21	36.39	26.27	40.76
Martha Street (J498.3497)	32.44	40.75	32.53	46.17
South of Waterfront Trai (J419.5095)	32.88	40.99	32.96	46.65
Lakeshore Road – Upstream (J149.5095)	37.94	41.19	38.05	47.14
Lakeshore Rd Downstream / Lake Ontario (J59.22536)	42.63	43.83	42.75	50.29

3.2 1-DIMENSIONAL HYDRAULICS

3.2.1 MODEL DEVELOPMENT

A 1-dimensional hydraulic model was developed in HEC-GeoRAS) as part of the Phase 1 study in order to develop estimated floodplain mapping, using the elevation data supplied at that time by the City of Burlington (2015 Region of Halton DEM).

It should be noted that the integrated hydrologic/hydraulic modelling (PCSWMM, as per Section 3.1) is also capable of generating simulated floodplain elevations. However, as per the comments provided by Conservation Halton as part of the Phase 1 study (ref. September 12, 2017 letter), it is understood that a preference is for the floodplain analysis to be completed in a steady-state HEC-RAS model, given concerns with respect to flow attenuation and storage, due to the more complex hydrodynamic modelling routines used in PCSWMM (full dynamic wave routing) as opposed to 1D steady-state HEC-RAS modelling (energy equation approach). This approach has been confirmed as per the Phase 2 TOR (refer to Appendix B). Note that the PCSWMM modelling is generally consistent with the Phase 1 Study modelling (notwithstanding the edits noted in Section 3.1.1) and therefore does not reflect the updated topographic data used for hydraulic modelling (i.e. HEC-RAS) for the Phase 2 study. Given that the PCSWMM model is being applied for hydrology only (and not for any type of floodplain modelling for the reasons noted previously) this is considered reasonable.

A key component of the Phase 2 scope of work is to update the preceding modelling with the more current and resolute 2018 Halton LiDAR DEM. This dataset employs the CGVD:2013 vertical datum, unlike the previous modelling which applied the CGVD28:78 datum (City of Burlington standard).

The 1D model developed during Phase 1 of the study was initially updated using the 2018 LiDAR data with buildings provided by CH. Approximately 39 cross sections were adjusted to better represent the geometry and structures within this model. Since building obstructions are already present in the LiDAR mapping and represented in the cross-section geometry, additional obstructions have not been used.

As part of the collaboration between CH and WSP with respect to this study, CH generated an updated 1D HEC-RAS model specifically for Lower Rambo Creek, which was supplied in conjunction with its comments of May 19, 2022. New cross-sections were developed for all of the watercourses, in addition the “lid” option was used to model both the Blairholm and Caroline enclosures rather than using the more typical hydraulic structure routines in HEC-RAS. It should be noted that previously WSP had excluded the section between Caroline Street and James Street from the 1D hydraulic model due to the fact that this section is dominated by the sub-surface enclosure, with the exception of the short section of open channel behind 497 Elizabeth Street). WSP previously applied a fixed boundary condition at the upstream limits of the enclosure, based on the results of the dual drainage PCSWMM modelling. As part of its updates, CH has included open channel sections throughout this area, and has used the “lid” option to represent the enclosure. WSP has generally deferred to the updated modelling approach proposed by CH; this has been used for the current iteration of the assessment and reporting. Further discussion is provided in Section 3.2.2.

Hydraulic structure dimensions have been verified from survey completed by WSP as part of the Phase 1 study, and data supplied by CH as completed by R. Avis Surveying Inc. in 2016 (relevant drawings are included in Appendix E). Elevations have been converted to the CGVD:2013 datum accordingly. A vertical datum conversion factor of 0.43 m has been used for the culvert inverts if the drawings/survey were in the original CGVD28:78 vertical datum as opposed to the current study (CGVD:2013 datum). In some cases, the elevation from the LiDAR dataset has been used to determine the expected culvert invert, given the high resolution of the data.

Similar to 2D modeling, HEC-RAS Version 6.3.1 has been used for all 1D modeling updates (most current and stable version of the program). The updated 1D HEC-RAS modelling has also utilized the updated peak flows presented in Section 3.1.2. Hydraulic modelling and simulated floodplain extent results are discussed further in Section 3.2.2.

3.2.2 FLOODPLAIN EXTENTS

Simulated floodplain extents from the 1D HEC-RAS modelling (are presented in Drawing 7 (Lower Hager Creek), 8A (Lower Rambo Creek – without spills) and 8B (Lower Rambo Creek – with spills, with FCFs credited) respectively.

The results for Lower Hager Creek are generally consistent with those generated as part of the Phase 1 study (2020). The most notable result is a large spill indicated between Birch and Caroline Streets. The spill is indicated for both the 100-year and Regional Storm Events. As per the Study Terms of Reference (reference Appendix B), the spill has not been mapped further, however.

The results for Lower Rambo Creek (without spills) are also generally consistent with those presented in the Phase 1 study. A spill across Blairholm Avenue is indicated for both the 100-year and Regional Storm Events, due to the capacity constraints associated with the enclosure in this area. Downstream of this location the results are again similar and indicate that both the 100-year and Regional Storm floodplain extents are generally confined to the channel block. Overtopping is however indicated for the Regional Storm and the 100-year storm event at all roadway crossings up to the Caroline Street enclosure, which does not have sufficient capacity to convey flows without overtopping, nor does the Maria Street enclosure. 100-year road overtopping is indicated again at Martha Street. Some roadway spill on to Lakeshore Road is indicated

The current 1D modelling results do indicate a more extensive floodplain immediately upstream of the Caroline Street enclosure as compared to previous model version which employed a fixed water surface elevation boundary condition at the upstream limit of the enclosure (and removed this section from the modelling). The updated approach (HEC-RAS) appears to predict a greater backwater impact from Caroline Street than the previous approach (PCSWMM) and greater upstream flood extents accordingly. The 1D results appear to be slightly conservative (more extensive) as compared to the 2D extents indicated on Drawings 8A and 8B; this is discussed further in the subsequent section.

It is noted that the estimated 1D floodplain limits along the enclosure (Caroline Street to James Street) should be considered approximate given that the 1D cross-sections cannot fully contain the flow, and the complex spill pathways in this area. It is considered that the 2D generated inundation limits are a better indicator of flooding limits in this specific area. This is discussed further in the subsequent section.

3.3 2-DIMENSIONAL HYDRAULICS

3.3.1 MODELING OVERVIEW AND RESULTS

3.3.1.1 LOWER RAMBO CREEK (TO BLAIRHOLM AVENUE)

The 2D model extents for the East and West Rambo creek areas (as described in Section 2.2) have been extended suitably south to reasonably map the spills from the Hager-Rambo Diversion channel and the expected flood inundation limits, as the flows drain towards Lower Rambo Creek.

The HEC-RAS 2D model is based on riverine spill flows only; local flows from the PCSWMM modelling for the Lower Rambo Creek watershed have not been included. As such, the simulated extents (as presented both on Drawings 5B to 5D, and 8B) should be understood as riverine spill inundation limits only.

As discussed in the preceding section, the long enclosure at Blairholm Avenue does not have sufficient capacity to convey the Regional Storm event, resulting in overtopping and overland spill. The associated inundation limits in this area generated by the 2D modelling are considered more appropriate than those from the 1D modelling, given the better representation of surface grades and pathways.

The 2D inundation limits have been added to Drawing 8B for comparison to the 1D generated limits. As evident, the 2D model indicates more spill flows being conveyed along roadways than is captured by the 1D flood inundation limits. However, primary floodplain extents are generally quite consistent between the two (2) modelling sources.

3.3.1.2 *LOWER RAMBO CREEK (AT CAROLINE STREET)*

As noted in the 1D modelling summary (Section 3.2.2), the long enclosures at Caroline Street and Maria Street/James Street do not have sufficient capacity to convey the Regional Storm flows with the inclusion of spills from the upstream Hager-Rambo Diversion Channel area.

A new HEC-RAS 2D model (ref. Figure 3.2.1 for model extents) has been developed for this area to identify spills. This 2D model has also been developed using the 2018 LiDAR provided by CH and contains 2D modelling components such as a detailed 5 X 5 m 2D mesh, use of breaklines, hydraulic structures (including the Caroline Street and Maria Street enclosures), a Manning's n layer, and normal depth boundary condition lines to allow water to exit.

The Regional Storm (with FCFs credited) inflow hydrographs for the 2D area have been extracted from the dual-drainage PCSWMM modelling described in Section 3.1 (for both the with and without spill flow scenarios). The resulting hydrographs have been applied within the 2D mesh at appropriate locations. The combined flow from the PCSWMM modelling at the upstream limits of the model (Victoria Ave) has been used as a starting point, with incremental hydrographs (based on the difference between the total simulated creek flow and the upstream flow) added at key riverine nodes further downstream.

The Regional Storm flood extents generated by the 2D modelling are presented in Drawings 8A (without upstream spill flows) 8B (with upstream spill flows) and in Figure 3.2.2.

A spill is indicated at Caroline Street under both the with and without spill flow scenarios. For the "without upstream spill" scenario (Drawing 8A), the spill is relatively limited and confined to the area around Caroline Street and Elizabeth Street.

For the "with upstream spill flows" scenario (Drawing 8B) the Regional Storm spill flow is much more extensive. The spill occurring at Caroline Street flows towards Lakeshore Road, traveling via Elizabeth Street, John Street and Locust Street. Spills onto James Street, Martha Street and Lakeshore Road have also been indicated by the modelling. The flood depth summary is presented in Figure 3.2.2 and Drawing 8C.

The simulated 2D inundation limits upstream of Caroline Street are somewhat less than the 1D limits and may be a more accurate representation of the flood limits in this area given the complexities of the hydraulics in this location and given that the 1D modelling does not allow flows to leave the system (spill) whereas the 2D modelling does. Similarly, the flood inundation limits generated by the 2D modelling are considered a more accurate representation of the flood hazard limit along the Caroline Street enclosure (i.e. Caroline Street to James Street) given the complexities of spill flows and the limitation of the 1D modelling in this area.

It should be noted that the simulated flood inundation limits have been truncated at Lower Hager Creek since the interaction of the spill flow with the Lower Hager Creek floodplain has not been assessed.

3.3.2 FILL ANALYSIS

A scoped fill analysis was completed for the East Rambo, West Rambo, Hager-Rambo Diversion Channel and Upper Rambo Creek area as part of the Burlington GO MTSA as discussed briefly in Section 2.2.2.3, and presented further in Appendix D.

An assessment of the potential impact of filling in the Lower Rambo Creek area south of Victoria Avenue (i.e., the Downtown area) due to the expected spill at Caroline Street (Regional Storm Event with spills scenario) has not been assessed as part of the current summary. This may be considered further as part of future study or through site-specific assessments as required. As noted in Section 2.2.2.3, given the uncertainty as to the timing and form of future intensification and development within the Downtown Area, it is suggested that the 2D modelling tools developed as part of the current study be applied to further assess the impacts of any proposed infilling and associated cut\fill and mitigation measures to demonstrate that proposed developments are adequately floodproofed and also do not result in any negative flooding impacts to off-site properties and areas. It is expected this will occur as part of site-specific development applications as they are proposed.

Figure 3.2.1. Boundary of Burlington Downtown HEC-RAS 2D Model

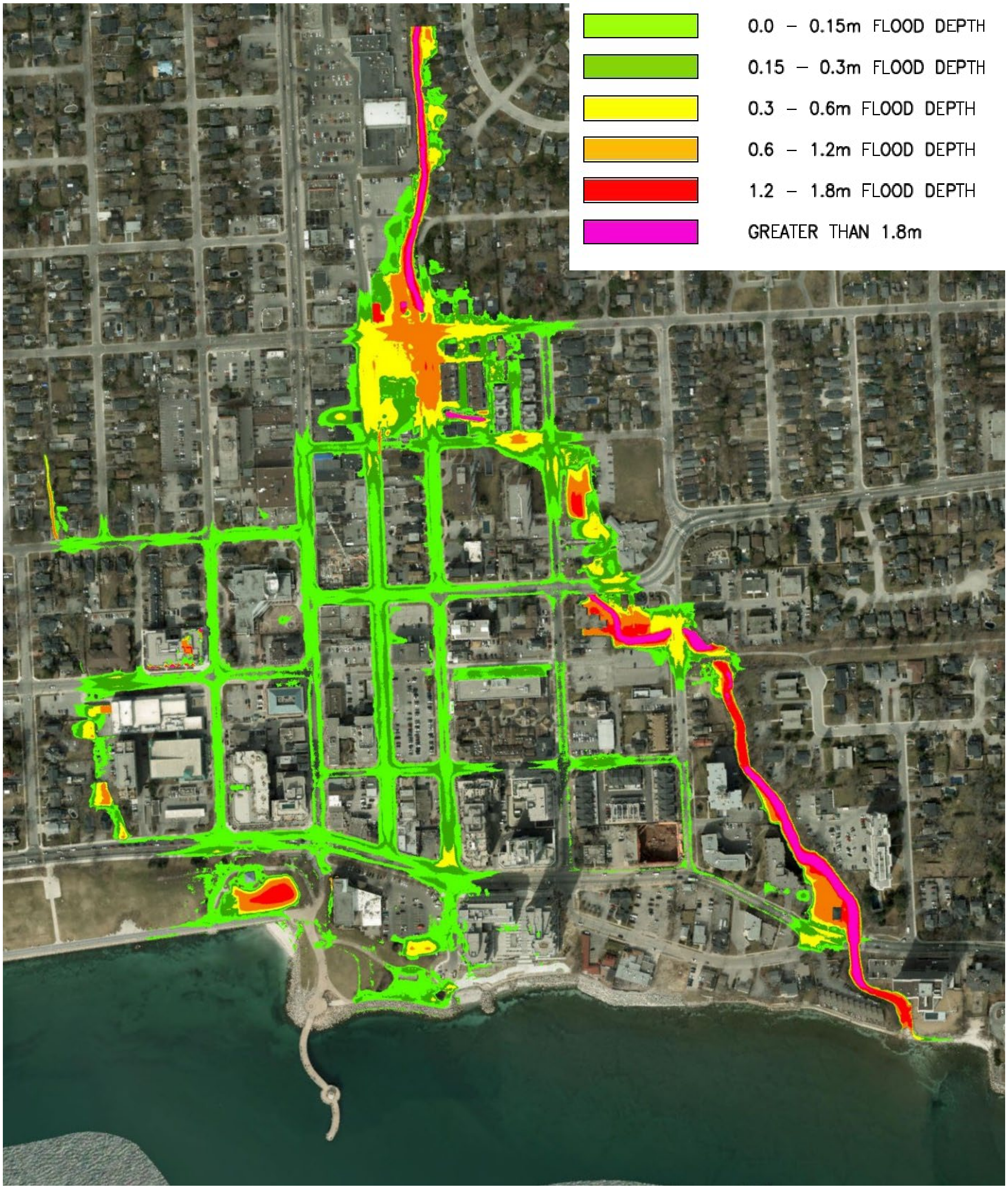

Figure 3.2.2. Regional Storm (with Spills and FCFs Credited) Inundation Limits and Flood Depths

4 SUMMARY

4.1 ANALYSES AND MODELLING

The preceding analyses have permitted a current understanding of the expected hydrologic impacts of intensification, as well as the expected flood hazard limits within both the Burlington GO MTSA and Downtown areas. Ultimately the updated modelling and flood inundation mapping developed through this study should become the basis for the identification and assessment of flood risk within the study areas and should supersede hazard mapping developed through the previous Phase 1 study.

The modelling tools should be kept up to date to reflect ongoing changes as they occur. This includes application of the modelling tools on a "case by case" basis to assess the SWM quantity control strategies for development sites for the 2-100 year storm events (and potentially the Regional Storm Event), to ensure that there are no adverse impacts to downstream watercourses. This may include allowing sites to discharge uncontrolled for certain storm events, however the requirements for downstream watercourse receivers may need to be balanced against the potential impacts to intermediary conveyance features (i.e. storm sewers and overland flow routes).

The City, as proponents of the current study, may be most suited to maintain the most current versions of the modelling tools. Further review with Conservation Halton may be required to confirm ownership and associated responsibilities. The findings from this technical study should also inform other City, Region, Conservation Authority, and Province-led initiatives.

A preliminary infilling assessment has been completed as part of the current study. The intention of the filling assessment was to ensure that any constraints to future development could be reasonably addressed as part of the study. However, during the completion of the assessment, it was determined that there was too great an uncertainty with respect to the form of future development (i.e., size and extents of buildings, site grading, etcetera) as well as the sequencing, to be able to reasonably identify expected impacts and mitigation measures. It is expected that any proposed development that may be located within an area expected to be impacted by spills should be assessed on a case-by-case basis using the tools developed through the current Phase 2 study. This assessment should consider both on-site and off-site impacts. Further confirmation with CH would be required to confirm what can be reasonably deemed and accepted as a nominal impact (as absolute zero impact may not be feasible in all cases); it is understood that this is still subject to confirmation from CH. CH's pending formal spill policy may provide further direction in this regard. Until CH's spills policy is finalized, CH is operating under an interim spills policy which is generally on a case-by-case basis.

Overall, it is expected that the modelling tools developed as part of this study should be leveraged by those development proponents in identified flood hazard areas to support site-level assessments as development applications are proposed and submitted. This would include verification of on-site SWM quantity controls (as noted previously), as well as incremental verification of proposed development and grading to ensure that proposed development is adequately floodproofed to the requirements of the City and CH, and that safe ingress and egress can be achieved, as per the requirements of CH (and the guidelines of the 2002 Flood Hazard Guidelines by MNR). As noted previously, these assessments should also confirm that proposed filling does not negatively impact flow conveyance and the flood risk adjacent and downstream properties, subject to consideration of what can be reasonably deemed and accepted as a nominal impact (if an absolute zero impact can be demonstrated to be infeasible). As noted previously, the City may be most suited to maintain the most current versions of the modelling tools as updates are applied to reflect ongoing developments.

4.2 POLICY

As noted previously, it is expected that the findings of this technical study should inform other City, Region, Conservation Authority, and Province-led initiatives, including policies.

The results of the hydrologic analysis of future intensification within the Burlington GO MTSA (including supplementary analyses completed by CH as documented in their January 23, 2023 comments included in Appendix B) have indicated that there is the potential for adverse impacts to downstream receivers from potential future intensification. The supplementary analyses completed by CH (refer to Appendix B) further indicate that on-site controls may further increase off-site peak flows. As noted previously, the City continues to advocate for a “case by case” approach whereby development proponents use the modelling tools developed through this study to assess potential off-site impacts and confirm if the requirements of the City’s Stormwater Management Guidelines are adequate. If deemed necessary, alternative SWM strategies may include uncontrolled discharge for some events, however simulated impacts to downstream receivers would need to be balanced against impacts to intermediary conveyance infrastructure (i.e. storm sewers and overland flow routes).

A similar assessment has not been completed for the Downtown area given the agreed-upon Terms of Reference for this study, generally premised on the highly developed nature of the downtown area. The supplementary hydrologic results provided by CH (refer to Appendix B) indicate only nominal increases in peak flows to receivers as noted in Section 3.1.1.1. Notwithstanding, no assessment of the impacts of on-site SWM quantity controls (as per City Stormwater Management Guidelines) has been completed. Consistent with the approach for the Burlington GO MTSA, a “case by case” approach is recommended, whereby development proponents utilize the analytical tools developed through this study.

The necessity for on-site Regional Storm quantity controls has not yet been definitively confirmed. As noted, this will require a further “case by case” assessment as development applications are submitted. If Regional Storm controls are determined to be warranted, a further policy review will be required to confirm the necessary requirements to allow CH to support formal crediting, including consideration of ownership and operation and maintenance considerations.

The identified flood hazard limits have been developed on the basis of the “with flood control facilities” (i.e., all three (3) flood control facilities (FCFs) in place – West Hager Pond, Freeman Pond and East Rambo Pond). A review of the potential for debris blockage at each of the three (3) FCFs has been undertaken as part of this study. It has been determined that based on the nature of the West Hager Pond, a 50% debris blockage factor is appropriate, and has been applied in the assessment of the expected performance of that facility. The other two (2) FCFs are considered to either be of low risk (Freeman Pond) or have other redundancies to address the concern (East Rambo Pond and the debris grate in front of the outlet culvert). All of the “with FCF” scenarios reflect the preceding assumptions, and the operational rating curves confirmed through previous studies.

The City has clearly indicated as part of the Phase 1 study and subsequent work, which given the legacy and original intent of these FCFs, all mapping should include crediting. The “Hager-Rambo Flood Control Facilities Study Report” (Wood, September 2020) was prepared to further support crediting. That report was signed by all professional engineers which completed the report, including geotechnical, structural, and water resources engineering professionals. Notwithstanding, as per comments from CH (ref. e-mail Smith-Malik, February 23, 2023), the report has been requested to be re-issued including Professional Engineering Stamping from the leads for the various disciplines (all other elements of the 2020 report remain unchanged). This remains in process at the time of the finalization of this report. Nonetheless, WSP confirms that the findings and recommendations generated by the 2020 study remain valid, including that:

- The proposed stage-storage-discharge relationships proposed in that report remain valid based on the best available information (notwithstanding, as per the current Phase 2 Study Report, the proposed West Hager FCF rating curve should apply a 50% debris blockage factor).

- The FCFs pose a limited risk of failure under a Regional Storm Event.

- Inclusion of the three (3) online FCFs within the hydrologic modelling to attenuate peak flows to be applied in downstream hydraulic modelling and associated flood hazard mapping is considered reasonable and appropriate.

CH has received comprehensive documentation from the City (February 9, 2023) which details the ownership and maintenance responsibilities (between the City, CH and MTO) of the Hager-Rambo Flood Control System facilities which include the East Rambo Pond, Freeman Pond and West Hager Pond. CH is currently reviewing the documentation to confirm that maintenance responsibilities are clearly identified and that the quantity control function of the three (3) flood control facilities will be accepted and included in the hydrologic modelling, and the associated peak flows used in hydraulic modelling and flood hazard mapping.

As such, as part of the current study and agreed Study Terms of Reference (ref. Appendix B), alternative FCF scenarios have been assessed as a form of sensitivity analysis and to assist CH in understanding the impacts of partial crediting and complete removal of these facilities. As per the TOR, this study has focused primarily on the resulting difference in flows only; updated floodplain extents for these scenarios have not been considered.

Ideally, confirmation of agreement on the "with FCFs" crediting of all parties should be obtained in conjunction with the finalization of this report and study; however, it is understood that discussions remain ongoing, including CH review of the comprehensive documentation provided by the City.

As noted previously it is understood that CH is in the process of developing a formal spills policy. Previously, CH did not formally map or apply its regulation to most spills, as it was not feasible with the resources available at the time. As indicated in CH's overall Policies and Guidelines for the Administration of Ontario Regulation 162/06, CH now indicates that any proposed development in spill areas may require CH's permission. Currently, CH is operating under an interim spills policy. It is expected that the forthcoming spills policies will further elaborate on requirements.

4.3 HYDRAULIC STRUCTURES

As demonstrated in previous sections, several hydraulic structures have been identified which do not have sufficient capacity to convey design flood flows. In many locations, this results in elevated flood levels and extents, which may potentially affect adjacent lands. Further, undersized hydraulic structures may also affect spills, which can generate additional flood hazards for roadways and other properties located outside of the typical riverine flood hazard limits.

Based on the modelling results, the hydraulic structure conveying West Rambo Creek across Plains Road is of particular concern, given the magnitude of spill flows indicated from the East Rambo Pond via the CNR crossing of the QEW, which results in additional flows to this location which were likely not considered or known in the original design. Any flows not captured by the crossing, spill overland westerly down Plains Road and to the Brant Street underpass, and ultimately southerly down Brant Street to Lower Rambo Creek and the Downtown area. Increasing the conveyance capacity of this structure or implementing a secondary diversion structure to direct flows to West Rambo Creek would reduce spill flows accordingly. Additional hydraulic structure upgrades further downstream on West Rambo Creek (i.e., the CNR crossings and De Paul's Lane) may be required as well. Notwithstanding, it is understood that any such hydraulic structure upgrades will need to consider capital budgeting requirements by the City and the local site constraints related to property ownership and existing utilities, among other considerations.

CH has previously noted an interest in further assessing a hydraulic structure upgrade scenario, however it is considered that this will be deferred to future study.

4.4 FUTURE STUDIES

As noted previously, it is expected that the modelling tools developed by this study will be applied on a "case by case" basis by future proponents to both confirm requirements for on-site SWM quantity controls (to mitigate impacts to downstream receivers) and to ensure that cut\fill balancing and floodproofing for safe ingress\egress are adequately addressed.

It is understood that Conservation Halton is planning an overall update of the hydrologic and hydraulic modelling of the Hager-Rambo and Roseland Creek systems in the near future, however a definitive timeline has not been established. It is not expected that any future update work will generate notably different results from the current study (unless notable changes in hydrology and peak flows occur), however this potential will need to be considered once any updated (and approved) modelling becomes available.

It is noted that the current study has included a conversion of the previous SWMHYMO hydrologic modelling to a more current Visual OTTHYMO platform. Notwithstanding, the current effort has focused on the primary study area only; CH may have an interest in revisiting the entirety of the watershed hydrologic modelling including upstream areas.

The hydraulic modelling (HEC-RAS; both 1D and 2D) reflects the most current topographic data available and uses the most current hydraulic modelling tools, and therefore it is expected that any future work would leverage these to the extent possible. This includes both any watershed-wide update (as initiated by Conservation Halton), as well as localized site development application verifications (as initiated by developers and their representatives).

As discussed previously, future study (potentially led by the City or CH) may consider the potential benefits of hydraulic structure upgrades, particularly along West Rambo Creek and at Plains Road.

In addition, the modelling (in particular the 2D modelling) should be applied as part of future site development applications to validate any future infill development in spill areas and confirm that the proposed development is adequately floodproofed and also does not result in any negative impacts to off-site areas (subject to confirmation of an acceptable nominal impact as noted previously). Any updates based on CH's forthcoming spills policy may also be relevant.

As noted in the Phase 1 study, the City and CH may also wish to consider undertaking further field monitoring and data collection efforts to support hydrologic model calibration to ensure that the simulated flows are consistent with the actual watershed response. This would include drainage systems within the Burlington GO MTSA and Downtown area, as well as potentially areas further upstream. Based on WSP's previous experience, CH typically only supports model calibration if a sufficiently large storm event is recorded (i.e., 2-year or greater) to ensure consistency with the expected response for formative flood events used for regulation. This typically requires a sufficiently long period of monitoring to ensure that storms of such a magnitude are captured. Commencing such a program as early as possible, and well in advance of any planned hydrologic modelling update work, is therefore recommended. These data could support future watershed modelling update studies planned by CH (i.e., Roseland and Hager-Rambo Creek systems) and the associated model calibration\validation process.

Respectfully submitted,

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