CONSERVATION HALTON

HYDRAULICS REPORT

EAST BURLINGTON CREEKS FLOOD HAZARD MAPPING UPDATE

SEPTEMBER 18, 2023

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EAST BURLINGTON CREEKS FLOOD HAZARD MAPPING UPDATE

CONSERVATION HALTON

PROJECT NO.: WW21011057

DATE: SEPTEMBER 18, 2023

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1 INTRODUCTION AND SCOPE

WSP E&I Canada Limited (WSP; formerly Wood Environment & Infrastructure Solutions,Canada Limited) has been retained by Conservation Halton (CH) to undertake an update to the flood hazard mapping for the “East Burlington Creeks” area, namely Tuck, Shoreacres, Appleby and Sheldon Creeks. The scope of work includes the development of new hydrologic and hydraulic models (both 1-dimensional (1D) and 2-dimensional (2D) for each of the watersheds, ultimately leading to the preparation of floodline delineation and flood hazard mapping preparation. The study also includes public consultation and engagement and documentation.

As per the approved scope of work for this project, the hydraulic modelling and associated documentation includes the following tasks:

- Task 5.1: Identify Cross-Section Naming Convention
- Task 5.2: Identify Cross-Section Alignment, Centreline, and Overbanks
- Task 5.3: Identify Key Hydraulic Modelling Parameters
- Task 5.4: 1D Steady State Hydraulic Model Development
- Task 5.5: 1D/2D or 2D Models
- Task 5.6: Iterative Analysis for Inter-basin spills
- Task 5.7: Endorse Modelling (Quality Assurance/Quality Control) and Model Validation

Following completion of the preceding (hydraulic modelling), flood hazard mapping is to be prepared for both 1D and 2D generated floodlines (Task 6).

In addition to review by CH, a Technical Advisory Committee (TAC) consisting of representatives from the City of Burlington, Town of Oakville, and Halton Region has also reviewed key deliverables and provided comments and input.

The current hydraulics report builds upon previously submitted Technical Memoranda and reflects CH and TAC input on the previous draft submittals. The report should be read in conjunction with the companion report on hydrologic modelling, specifically with respect to the estimation of inter-watershed spill flows.

This project received support through the National Disaster Mitigation Program, however the views expressed in this material do not necessarily reflect the views of the Province of Ontario or the Government of Canada.

2 BACKGROUND REVIEW

2.1 INFORMATION RECEIVED

The following currently available information which is relevant to the current reporting has been provided by Conservation Halton, City of Burlington, Halton Region, and Town of Oakville.

It should be noted that in addition to the information listed below, an extensive suite of record drawings have been provided for hydraulic structures from the member municipalities as well as applicable agencies (407ETR, Metrolinx, CN Rail).

- **Mapping Data**
  - 1 X 1 m full feature LiDAR data (Conservation Halton, 2018)
  - 1 X 1 m bare earth LiDAR data (Conservation Halton, 2018)
  - 0.5m topographic contour mapping (Conservation Halton, 2018)
  - 2019 Orthophotos (Conservation Halton, 2019)
  - Urban Burlington land cover mapping (Conservation Halton, 2021)
  - Urban Burlington building footprint mapping (Conservation Halton, 2021)
  - Watercourses mapping (Conservation Halton, 2021)
  - Regulated wetlands mapping (Conservation Halton, 2020)
  - ArcHydro drainage nodes shapefile (Conservation Halton, 2021)
  - ArcHydro drainage lines shapefile (Conservation Halton, 2021)
  - HECRAS cross section locations mapping (Conservation Halton, 2021)
  - Current spill directions mapping (Conservation Halton, 2021)
  - Current floodplain mapping (Conservation Halton, 2021)
  - Observed floodlines mapping (Conservation Halton, 2014)
  - Roads shapefile (Conservation Halton, 2020)
  - Railway shapefile (Conservation Halton, 2012)
  - MNR parcels mapping (Conservation Halton, 2020)
- **Reports**
  - Area-Wide
    - Urban-Area Flood Vulnerability, Prioritization and Mitigation Study (Amec Foster Wheeler, July 2017)
    - August 4th, 2014 Storm Event, Burlington, (Conservation Halton, 2015)
  - Tuck Creek
    - Project Updates: Tuck Creek Flood Mitigation Hydro Right of Way to Spruce Avenue (City of Burlington, 2020)

    - Tuck Creek Flood Assessment and Crossing Upgrades at Rockwood Drive and Rexway Drive Municipal Class Environmental Assessment, Schedule 'B' (IBI Group, 2020)
    - Tuck Creek Flood Assessment and Crossing Upgrades between New Street and Spruce Avenue Class Environmental Assessment Final Report (Aquafor Beech, 2016)
    - Tuck Creek Erosion Control Municipal Class Environmental Assessment Final Report (Aquafor Beech, June 2012)
    - Tuck Creek Erosion Control Study Hydrology and Hydraulics (Aquafor Beech, June 1996)
  - Shoreacres Creek
    - Shoreacres Creek Floodline Mapping Update Final Report (Environmental Water Resources Group Ltd., July 1997)
  - Appleby Creek
    - Appleby Creek Erosion Control Environmental Assessment Project File Report – Final (Aquafor Beech, August 2020)
    - Schedule B Class EA: Municipal Class Environmental Assessment for Appleby Creek Flood Mitigation between Fairview Street and New Street (Aquafor Beech, 2019)
    - Appleby Creek Floodline Mapping Update Final Report (Environmental Water Resources Group Ltd., July 1997)
  - Sheldon Creek
    - Sheldon Creek Flood Mitigation Opportunities Study Final Report (Wood, October 13, 2020)
    - Sheldon Creek Watershed – Hydrologic and Hydraulic Study – Final Hydrology Report (Amec Foster Wheeler, Revised October 2019)
    - Sheldon Creek Watershed – Hydrologic and Hydraulic Study – Final Hydraulics Report (Amec Foster Wheeler, Revised October 2019)
    - Sheldon Creek Watershed Master Plan (Philips Planning and Engineering Limited, October 1993)
- **Hydraulic Models**
  - Tuck Creek
    - TuckCreek_Ph1-HECRAS(2019_01_08).prj, HEC-RAS Version: 5.07 (Aquafor Beech, Approved 2021)
    - TuckCreek.prj, HEC-RAS Version: 5.0.3 (R.J. Burnside and Associates Limited, Approved 2018)
    - TuckCreek.prj, HEC-RAS Version: 4.10 (R.J. Burnside and Associates Limited, Approved 2018)
    - TuckCreek_Harvester-Billings(2017-03-21).prj, HEC-RAS Version: 4.10 (Aquafor Beech, Approved 2017)
    - TuckEastReach1.prj, HEC-RAS Version: 4.10 (Draft)
    - TuckEastReach2.prj, HEC-RAS Version: 4.10 (Draft)
    - Landmark.prj, HEC-RAS Version: 4.00 (Amec Foster Wheeler, Approved 2011)
    - tuck.prj, HEC-RAS Version: 4.00 (Cole Engineering, Approved 2010)
    - SUB.DAT, HEC-2 Version: 4.6.2 (Phillips Engineering, Approved 2009)
    - TUCK-P3.DAT, HEC-2 Version: 4.6.2 (S. Llewellyn and Associates Limited, Approved 2004)

- — TUCK-A4.DAT, HEC-2 Version: 4.6.2 (Aquafor Beech, Approved 1996)

— Shoreacres Creek

- — Existing_Future.prj, HEC-RAS Version: 4.10 (Counterpoint Engineering Inc., Approved 2017)
- — 100106AB.prj, HEC-RAS Version: 4.10 (Aquafor Beech, Approved 2016)
- — Prop.prj, HEC-RAS Version: 4.10 (MTE Consultants Inc, Approved 2016)
- — ShoreacresEastReach1.prj, HEC-RAS Version: 4.10 (Draft)
- — ShoreacresWestReach1.prj, HEC-RAS Version: 4.10 (Draft)
- — ShoreacresWestReach2.prj, HEC-RAS Version: 4.10 (Draft)
- — shoreacre.prj, HEC-RAS Version: 4.00 (Cole Engineering, Approved 2010)
- — shoreacre.prj, HEC-RAS Version: 4.00 (Cole Engineering, Approved 2010)
- — Shoreacres.prj, HEC-RAS Version: 3.13 (AMEC Foster Wheeler, Approved 2010)
- — SHR731.DAT, HEC-2 Version: 4.6.2 (J. McHenry, Approved 2004)
- — SHR731_PH2.DAT, HEC-2 Version: 4.6.2 (Metropolitan Consulting Inc., Approved 2003)
- — Shore.dat, HEC-2 Version: 4.6.2 (Philips Engineering Limited, Approved 2003)
- — SHORE.97H, HEC-2 Version: 4.6.2 (Environmental Water Resources Group, Approved 1997)

— Appleby Creek

- — Appleby_Lower.prj, HEC-RAS Version: 3.13 (Aquafor Beech, Approved 2015)
- — ApplebyMainBranch.prj, HEC-RAS Version: 4.10 (Valdor Engineering Inc, Approved 2014)
- — 10465_ApplebyGo.prj, HEC-RAS Version: 4.00 (MTE Consultants Inc, Approved 2010)
- — 20060724_3110_407_proposed_final.prj, HEC-RAS Version: 3.12 (Counterpoint Engineering Inc., Approved 2006)
- — 20060724_3110_proposed_final.prj, HEC-RAS Version: 3.12 (Counterpoint Engineering Inc., Approved 2006)
- — ApplebyEastReach1.prj, HEC-RAS Version: 4.10 (Brian Evans, Draft 0)
- — Apple97.prj, HEC-RAS Version: 4.00 (Imported Plan)
- — MC_G.prj, HEC-RAS Version: 4.10 (Philips Engineering Limited, Approved 2005)
- — Applebe-Post-Development-2.prj, HEC-RAS Version: 3.12 (J and B Engineering, Approved 2004)
- — REV-FUT2.DAT, HEC-2 Version: 4.6.2 (Philips Engineering Limited, Approved 2004)
- — 03150-R2.DAT, HEC-2 Version: 4.6.2 (Philips Engineering Limited, Approved 2004)
- — APPLEB97.DAT, HEC-2 Version: 4.6.2 (Environmental Water Resources Group, Approved 1997)
- — EASTULT2.DAT, HEC-2 Version: 4.6.2 (Philips Engineering Limited, Approved 1997)

— Sheldon Creek

- — SheldonCrek_Apr2020.prj, HEC-RAS Version: 5.01 (Wood, 2020)
- — MSheldon.prj, HEC-RAS Version: 4.10 (GHD, Approved 2014)
- — W1W2.prj, HEC-RAS Version: 4.10 (Urbantech Consulting, Approved 2010)

- MainSheldon.prj, HEC-RAS Version: 4.00 (Marshall Macklin Monaghan, Approved 2010)
- sheldon.prj, HEC-RAS Version: 4.00 (Cole Engineering, Approved 2010)
- WestSheldon.prj, HEC-RAS Version: 4.00 (MMM Group Limited, Approved 2010)
- shelpr2.dat, HEC-2 Version: 4.6.2 (Totten Sims Hubicki Associates Limited, Approved 2008)
- A_07_B_77Proposed.prj, HEC-RAS Version: 4.00 (MMM Group Limited, Approved 2008)
- 3rd-Pro1106.txt, HEC-2 Version: 4.6.2 (Trafalgar Engineering Limited, Approved 2007)
- W1W2Scenario2R.prj, HEC-RAS Version: 3.12 (Stantec, Approved 2006)
- ShelPROP.dat, HEC-2 Version: 4.6.2 (Marshall Macklin Monaghan, Approved 2005)
- CNR-ULT2.dat, HEC-2 Version: 4.6.2 (Philips Engineering Limited, Approved 2005)
- PR13-2C.DAT, HEC-2 Version: 4.6.2 (Philips Engineering Limited, Approved 2004)
- AMFinal.prj, HEC-RAS Version: 2.20 (Marshal Macklin Monaghan, Approved 2002)
- Final_A_01_B_1.prj, HEC-RAS Version: 3.0.1 (Marshall Macklin Monaghan, Approved 2001)
- ESHFU-1.HEC, HEC-2 Version: 4.6.2 (Philips Engineering Limited, Approved 1993)

2.2 SUMMARY OF PREVIOUS STUDIES

2.2.1 TUCK CREEK WATERSHED

Tuck Creek Erosion Control Study Hydrology and Hydraulics (1996)

In 1996, a hydrologic and hydraulic analysis was undertaken for Tuck Creek to update and expand the existing hydrologic model for the study area, to estimate flow conditions associated with existing and ultimate land use scenarios, and to update floodplain mapping of the watercourse (ref. Tuck Creek Erosion Control Study Hydrology and Hydraulics. Aquafor Beech, June 1996). The hydraulic model developed for this study consisted of a HEC-2 model, which was updated from the previous hydraulic model developed in 1985 as part of the Tuck, Shoreacres, Appleby & Sheldon Creeks Watershed Study (ref. Proctor and Redfern Group, M.M. Dillon Ltd., and MacLaren Plansearch Inc., 1985). Updates to the model included refinements based upon field investigation for bridges/culverts, revised topographic mapping and local survey, and merging with other sub-models developed as part of site specific projects such as bridge/culvert upgrades or erosion control works.

The revised HEC-2 model extended from Lake Ontario to the Headon Forest community (north of Headon Road), and along the west tributary up to the Highway 403 (407) corridor and the east tributary up to Highway 5 (ref. Aquafor Beech, 1996). The boundary condition at the outlet to Lake Ontario was set to a water surface elevation of 74.7 m (topographic mapping from 1989, vertical datum unknown), which was the mean annual water level of Lake Ontario at the time of study (ref. Aquafor Beech, 1996).

The revised HEC-2 model was simulated for the 2-100 year design storm events, as well as the Regional Storm; flows were sourced from the hydrologic modelling developed as part of the same study (refer to companion Hydrology Report for further details). The hydraulic results were then used to establish floodplain limits, and identify flood susceptible buildings and hydraulic structures (bridges/culverts), as well as the potential for spills from the watercourse during the 100-year and Regional storm events. The results of this study included the following outcomes (ref. Aquafor Beech, 1996):

- Out of the eighteen (18) hydraulic structures modelled, fifteen (15) overtopped during the Regional Storm and eleven (11) overtopped during the 100-year storm.

- Spills from the watercourse were closely related to the overtopping structures (backwater).

- Approximately 223 buildings were identified to be susceptible to flooding during the Regional Storm, and approximately 78 buildings were susceptible during the 100-year storm. The majority of these flood susceptible areas were identified between Lakeshore Road and Fairview Street.

- Six (6) hydraulic structures were recommended for upgrades as part of the 1985 study, including structures at Lakeshore Road, Regal Road, New Street, Fairview Street, CNR (Oakville) and Upper Middle Road. As part of the 1996 study, the structure upgrades carried forward from the 1985 study recommendations were assessed to quantify the improvement for flow conveyance and flood levels.

Tuck Creek Erosion Control Municipal Class Environmental Assessment (2012)

In 2012, the City of Burlington retained Aquafor Beech to complete an erosion control Municipal Class Environmental Assessment (EA) for Tuck Creek (ref. Tuck Creek Erosion Control Municipal Class Environmental Assessment, Aquafor Beech, June 2012). This EA was initiated in response to the City-wide Creeks Inventory and Erosion Assessment completed in December 2008 which identified two reaches of Tuck Creek as high priority areas. The local study area for the EA included Tuck Creek from Spruce Avenue on the downstream end, up to the Hydro Corridor upstream of New Street; the purpose of this study was to develop alternatives to address the erosion concerns at both the identified problem sites.

The hydraulic modelling portion of the 2012 EA study was based upon the HEC-2 model developed as part of the previous erosion control study (ref. Aquafor Beech, 1996). The HEC-2 model was imported into a newer version of HEC-RAS and updates were made to the road decks, manning's n value in culverts and to the bridge modelling approach to minimize the difference between the HEC-2 and HEC-RAS results, due to model conversion (ref. Aquafor Beech, 2012). The input flows for the hydraulic model were then updated based upon updates to the hydrology model (refer to companion Hydrology Report). Updates to the cross-sections were then made by creating additional cross-sections at key-points along the channel as well as the inclusion of low-flow channels for the cross-sections within the primary study area; all edits to cross-sections were based upon survey data collected in 2010 (ref. Aquafor Beech, 2012). This revised HEC-RAS model was then used to evaluate a variety of alternatives to address the erosion concerns within the study area.

Tuck Creek Local Flood Assessments

Subsequent to the erosion control EA conducted in 2012, the City of Burlington initiated two (2) Flood Assessment and Crossing Upgrade Municipal Class Environmental Assessments for two (2) different reaches within Tuck Creek, including between New Street and Spruce Avenue (ref. Aquafor Beech, 2016) and between Rockwood Drive and Rexway Drive (ref. IBI Group, 2020). The purpose of these studies was to evaluate a variety of alternatives to mitigate flood risk within the subject reaches; the alternatives evaluated included bridge/culvert upgrades and in-channel works such as channel deepening, channel widening and combination approaches. The hydraulic modelling efforts as well as the recommendations from both studies are summarized in the following sections.

"Tuck Creek Flood Assessment and Crossing Upgrades between New Street and Spruce Avenue Class Environmental Assessment Final Report" (Aquafor Beech, 2016)

The hydraulic modelling for the Flood Assessment and Crossing Upgrade between New Street and Spruce Avenue was based upon the HEC-RAS model updated for the subject reach as part of the previous Erosion Control EA completed in 2012 (Aquafor Beech, 2012). Updates to the model included geo-referencing the model, which included Tuck Creek from the Hydro Corridor to the outlet at Lake Ontario. The boundary condition was updated using Lake Ontario monthly mean water level records (from 1918 to 2013) obtained from the Canadian Hydrographic Service; the average maximum water level of 75.158 m (vertical datum of IGLD 1985) over the past 20-year period was used as the model boundary condition.

The cross-section geometry in the 2012 model was refined using survey data collected after the August 4th, 2014, storm event; this included topographic survey during the fall of 2014 for the Hydro Corridor to Regal Road, and in the fall of 2015 for Regal Road to Lake Ontario. The inclusion of the survey data captured the channel formation changes as a result of the August 4th, 2014, storm event, as well as the replacement of a recreational trail bridge located at the Hydro Corridor (ref. Aquafor Beech, 2012).

The resulting preferred alternative for this segment of Tuck Creek included three (3) crossing upgrades (New Street, Regal Road and Spruce Avenue) and channel widening; this alternative was found to provide a 95% reduction in the number of buildings flooded for the 100-year storm event, and a 38% reduction for the Regional Storm. Implementation for the preferred alternative was recommended to be completed in three (3) phases based upon the availability of City funding (ref. Aquafor Beech, 2012).

"Tuck Creek Flood Assessment and Crossing Upgrades at Rockwood Drive and Rexway Drive Municipal Class Environmental Assessment, Schedule 'B'" (IBI Group, 2020)

The hydraulic modelling for the Flood Assessment and Crossing Upgrade between Rockwood Drive and Rexway Drive was built upon the HEC-RAS modelling developed for the EA completed in 2016 for Tuck Creek from the Hydro Corridor to Lake Ontario (ref. Aquafor Beech, 2016). The model developed in 2016 was extended upstream to represent the subject reach, and cross-sections for the channel were established based upon the DEM provided by Conservation Halton and were supplemented by topographic survey of the creek corridor completed by IBI Group in June 2017. The boundary condition at the outlet to Lake Ontario was set to 75.0 m (vertical datum unknown), based upon known water surface elevations of the lake. Two (2) hydraulic structures were added to the model based upon field inventory and topographic survey (ref. IBI Group, 2020).

The resulting preferred alternative for this segment of Tuck Creek including upgrading the crossings at Rockwood Drive and Rexway Drive, once the existing structures reach their end-of-life cycles. Recommendations for structure upgrades included constructing wider crossing openings, and minor channel grading, which were noted to provide a reduction in the 100-year floodplain for the areas directly upstream of both structures. Other short-term flood protection measures were also discussed, including backflow preventers, roof leader disconnection, lot grading improvements, flood walls, temporary barriers, floodproofing (wet/dry) and structure relocation (ref. IBI Group, 2020).

2.2.2 SHOREACRES CREEK WATERSHED

The Shoreacres Creek Floodline Mapping Update (Environmental Water Resources Group Ltd., July 1997) developed an updated HEC-2 hydraulic model based upon the previous hydraulic model developed in 1985 as part of the Tuck, Shoreacres, Appleby & Sheldon Creeks Watershed Study (ref. Proctor and Redfern Group, M.M. Dillon Ltd., and MacLaren Plansearch Inc., 1985). The model refinements were based upon field investigation of the channel (geometry, floodplain, vegetation, hydraulic structures) as well as topographic survey at select locations, including vertical control survey completed at seven (7) structures. Starting water surface elevations for Lake Ontario at the model outlet were based upon the greater elevation of either the HEC-2 elevation at the model limits or the starting water surface elevation flood elevations, based upon available data for Lake Ontario water levels.

The updated HEC-2 model was executed for the 2- through 100-year storm events, as well as the Regional Storm; the computed water surface elevation results was then used to establish floodplain limits and characterize the floodplain characteristics. The results and outcomes of the hydraulic modelling established the following:

- Seven (7) overland flow and spill zones were located along Shoreacres Creek, including the areas of Longmoor Drive, Harvester Road, QEW, Heritage Road, Walkers Line (West Branch), Headon Road (West Branch) and Dundas Street (West Branch & East Tributary).

- Approximately twelve (12) buildings are identified as flood susceptible during the Regional Storm event, and one (1) building is flood susceptible during the 100-year storm event. Recommended mitigation included building floodproofing.

2.2.3 APPLEBY CREEK WATERSHED

Appleby Creek Floodline Mapping Update (1997)

The Appleby Creek Floodline Mapping Update (Environmental Water Resources Group Ltd., July 1997) developed an updated HEC-2 hydraulic model based upon the previous hydraulic model developed in 1985 as part of the Tuck, Shoreacres, Appleby & Sheldon Creeks Watershed Study (ref. Proctor and Redfern Group, M.M. Dillon Ltd., and MacLaren Plansearch Inc., 1985). The model refinements were based upon field investigation of the channel (geometry, floodplain, vegetation, hydraulic structures) as well as topographic survey at select locations, including vertical control survey completed at five (5) structures. Starting water surface elevations for Lake Ontario at the model outlet were based upon the higher elevation between the HEC-2 elevation at the model limits and the starting water surface elevation flood elevations, based upon available data for Lake Ontario water levels.

The updated HEC-2 model was executed for the 2- through 100-year storm events, as well as the Regional Storm; the computed water surface elevation results was then used to establish floodplain limits and characterize the floodplain characteristics. The results and outcomes of the hydraulic modelling established the following:

- Ten (10) overland flow and spill zones were identified along Appleby Creek, including the areas of New Street, Pinedale Avenue, Fairview Street, Harvester Road, QEW (West and East), Appleby Line (West and East) and CN Halton (West and East).

- Approximately ten (10) buildings were identified as flood susceptible during the Regional Storm event, and no buildings were identified to be within the 100-year floodplain. Recommended mitigation included building floodproofing and channel refinements.

Local Flooding & Erosion Studies

In recent years, the City of Burlington has initiated two (2) studies specific to Appleby Creek for evaluation of flood mitigation and erosion control along certain priority reaches of Appleby Creek. These two (2) studies were completed through the Municipal Class Environmental Assessment process and incorporated the evaluation of a variety of alternatives to mitigate issues related to flooding and erosion within the subject reaches of Appleby Creek. The hydraulic modelling efforts as well as the recommendations from both studies are summarized in the following sections.

"Schedule B Class EA: Municipal Class Environmental Assessment for Appleby Creek Flood Mitigation between Fairview Street and New Street" (Aquafor Beech, 2019)

The hydraulic modelling applied in the Flood Mitigation EA between Fairview Street and New Street was based upon the 2006 HEC-RAS model (called "generic regulation model"), provided to Aquafor Beech by CH for use in the flood risk analysis; this base model was assumed to be based upon the 1997 floodplain study. Updates to the base model were made in conjunction with CH, as CH provided a number of recommended updates prior to the use of the model for flood risk analysis / floodplain mapping. The updates were focused within the primary study area limits and incorporated topographic survey, low flow channels, additional sections and updated hydraulic modelling parameters.

The updated HEC-RAS model was used to simulate the 100-year and Regional Storm for floodline generation within the study area, which was split into three (3) different reaches, located from New Street to the Railway. Based upon the existing conditions floodplain limits, thirty-seven (37) buildings were identified within the Regional floodplain, and two (2) buildings were identified within the 100-year floodplain. The alternative assessment included a review of a variety of opportunities, including upgrades to roadway crossings, channel widening and floodplain enhancements/connectivity. The preferred alternative for all three (3) reaches included the widening of existing hydraulic structures, including the bridges at New Street, Pinedale Avenue and Fairview Road, in order to improve flow conveyance and mitigate flood risk to the surrounding area.

"Appleby Creek Erosion Control Environmental Assessment Project File Report – Final" (Aquafor Beech, August 2020).

In 2020, the City of Burlington retained Aquafor Beech to complete an erosion control Municipal Class Environmental Assessment (EA) for Appleby Creek (ref. Appleby Creek Erosion Control Class Environmental Assessment, Aquafor Beech, August 2020). This EA was initiated in response to the updated City-wide Creeks Inventory and Erosion Assessment completed in 2016, which identified one (1) reach as high priority and four (4) moderate priority sites along Appleby Creek, between Lake Ontario and the South Service Road. The local study area for the EA included Appleby Creek, extending from Lake Ontario to South Service Road; the purpose of this study was to develop alternatives to address the erosion hazard concerns at the identified problem sites.

The hydraulic modelling portion of the 2020 EA study was based upon the HEC-RAS model developed as part of the previous erosion control EA study (ref. Aquafor Beech, 2019). The HEC-RAS model was updated to include the extended study area and included additional cross-sections in order to capture the identified erosion site locations identified through the study. The model geometry was refined using topography survey data to accurately represent the existing erosional concerns along the subject reach of Appleby Creek. The revised HEC-RAS model was then used to evaluate a variety of alternatives, including replacement of bank protection, channel widening, natural channel design, enhancement of aquatic habitat and improvements of riparian cover. The preferred alternatives for the twelve (12) identified erosion sites, consisted of local restoration works and comprehensive reach-based works.

2.2.4 SHELDON CREEK WATERSHED

The City of Burlington retained Amec Foster Wheeler (now WSP E&I Canada Limited) to undertake a Hydrologic and Hydraulic Study of the Sheldon Creek Watershed (October 2019), building off the work completed as part of the previous Sheldon Creek Watershed Master Plan (1993). The Master Plan involved the development of a model using the HEC-2 modelling platform, which has been supplemented and updated numerous times since its creation in 1993, including georeferencing and geometry updates completed by Conservation Halton based upon the 2002 DEM. The updated base model was then further refined as part of the hydrologic and hydraulic modelling update in 2019, to include hydraulic structure survey from 2016, additional cross-sections, updates to modelling parameters (updating to current standard practice), among other model updates to generate a refined hydraulic model to be used for floodline mapping, spill zones and overtopping structures.

The previously noted modelling was further refined by Wood (now WSP E&I Canada Limited) within the Town of Oakville as part of the "Sheldon Creek Flood Mitigation Opportunities Study" (October 2020). Updates to the hydraulic modelling included incorporating additional topographic survey, incorporation of in-channel works, hydraulic structure edits, and other minor modelling adjustments. The hydraulic modelling underwent additional validation to document the changes in results from the 2019 HEC-RAS model versus the updated 2020 HEC-RAS model.

The updated 2020 HEC-RAS model was then used to establish the mapping for the 100-year frequency flow condition, the Regional Storm event as well as the August 4th, 2014 storm event (based upon hydrologic simulation). The resulting floodplain mapping was then used to identify existing spill locations and flood vulnerable properties during each of the mapped events; the results of this analysis identified five (5) properties with major flood risk during 100-year event, seventeen (17) properties during the August 4th event, and thirty-one (31) properties during the Regional event. These results and the updated models were then used to evaluate a variety of flood mitigation opportunities, for which the preferred alternatives included a range of non-structural emergency preparedness recommendations as well as floodplain and/or channel improvements.

2.2.5 AREA-WIDE

The study for August 4th 2014 Storm Event was conducted by Conservation Halton in 2015 (ref. August 4th, 2014 Storm Event, Burlington. Conservation Halton, 2015). The area of the storm was approximately 200 km^2, centred over the middle and upper portions of Roseland Creek and Tuck Creek just east of Highway 407. The watercourses most impacted were Tuck Creek, Shoreacres Creek, and Appleby Creek. Homes were flooded by runoff that entered the buildings through the sanitary/storm sewer system, and from surface flooding from overflowing watercourse banks. Several watercourse crossings were also identified to have overtopped, generating large backwater impacts and spills from the watercourses.

Detailed field reconnaissance and cataloguing public input was completed by CH and partners to characterize the flood damages caused by the storm. The study indicates that there were no stream flow gauges in place within the affected area during the August 4th, 2014 event. Conservation Halton staff delineated high water marks and debris lines during post-storm reconnaissance. This information was translated onto contour mapping and used to estimate observed maximum water surface elevations within each of the impacted watershed systems.

In 2017, Amec Foster Wheeler (WSP) was retained by the City of Burlington to undertake the City-Wide Flood Vulnerability, Prioritization and Mitigation Study, in response to the August 4th, 2014 storm event (ref. City-Wide Flood Vulnerability, Prioritization and Mitigation Study. Amec Foster Wheeler, July 2017). As part of this study, hydraulic analyses were completed to identify flood vulnerable areas associated with both riverine and urban flooding conditions across all ten (10) watershed systems within the City.

The riverine hydraulic analysis was completed using the most current models available at the time of study; relevant to the current study (2021), those models included:

- Tuck Creek Watershed: June 2012 HEC-RAS Model (ref. Tuck Creek Erosion Control Municipal Class Environmental Assessment Final Report, Aquafor Beech, June 2012)

- Shoreacres Creek Watershed: 1997 HEC-2 Model (ref. Shoreacres Creek Floodplain Mapping Update, EWRG, 1997) and 2008 HEC-RAS Model (New to Lakeshore) (ref. Shoreacres Creek Erosion Control and Stream Restoration New Street to Lake Ontario Class Environmental Assessment Final Report, TSH, Jan 2009)

- Appleby Creek Watershed: 1996 HEC-RAS Model (ref. Appleby Creek Floodplain Mapping Update, EWRG, 1997)

- Sheldon Creek Watershed: 2017 HEC-RAS Model (ref. Sheldon Creek Watershed Hydrologic and Hydraulic Study, June 2017)

The findings of this study identified a variety of flood vulnerable roadways and buildings based upon land use category and the regulatory floodplain limits (riverine flooding). This analysis resulted in the identification of 195 flood vulnerable buildings in Tuck Creek, 86 in Appleby Creek, 28 in Shoreacres Creek and 18 in Sheldon Creek, with the vast majority of the identified structures designated as low density residential. The riverine flood vulnerability of each watershed system was analyzed to generate a priority list of the top twenty (20) flood vulnerable areas to be considered for mitigation. Of those twenty (20) areas, eight (8) were located in the Tuck Creek watershed, with one (1) in both Appleby Creek and Sheldon Creek.

3 HYDRAULIC MODELLING APPROACH

3.1 TOPOGRAPHIC DATA

3.1.1 LIDAR DATASET

Bare earth LiDAR and full feature LiDAR were provided by CH for use in this study. The bare earth LiDAR shows bare ground with buildings and vegetation removed, and in some locations, road decks have also been removed. The full feature LiDAR includes elevations from vegetation, buildings, structures, roads and other features on the landscape. Both LiDAR datasets are at a horizontal resolution of 1 x 1 m and apply the CGVD2013 geodetic datum.

In discussion with Conservation Halton, Conservation Halton supports applying a correction factor of -0.40 m to convert from CGVD1928:78 (the vertical datum applied by the City of Burlington and Town of Oakville) to CGVD2013 (the vertical datum used for the LiDAR dataset) where required. This is reasonably consistent with the datum adjustment of 0.423 m calculated by Wood (now WSP E&I Canada Limited) using Provincial Survey Benchmarks from the COSINE network (Hager-Rambo Flood Control Facilities Study Report, September 2020).

The horizontal and vertical datums to be used for the current hydraulic modelling study are as follows:

- Horizontal datum: North American Datum (NAD) 1983 coordinate system in UTM Zone 17N projection (ESPG Coordinate Number 26917)
- Vertical datum: Canadian Geodetic Vertical Datum of 2013 (CGVD2013).

The provided bare earth LiDAR from Conservation Halton has been compared with the Land Information Ontario (LIO) Halton Digital Terrain Model (DTM) by point elevation check. The LIO Halton DTM is a freely available dataset (available via the Ontario GeoHub) prepared for the Ontario Ministry of Natural Resources and Forestry (MNRF) from data collected using LiDAR in the spring of 2018, by Airborne Imaging. It is understood that this data was collected as part of the same flight as the Conservation Halton dataset, but was processed separately by others. The LIO Halton DTM is at a horizontal resolution of 0.5 x 0.5 m resolution and also applies the Canadian Geodetic Vertical Datum 2013 (CGVD2013).

The comparison between the datasets indicates that the differences are found to be generally less than 0.1 m, which would be expected given the common original data source (flight). Larger differences are noted more often near hydraulic crossings and outfalls along the Lake. These larger differences can be attributable to different resolutions, post-processing methodologies and extracting point elevation near cell faces with abrupt elevation changes. The differences of the point elevation check are presented in Figure 3.1.

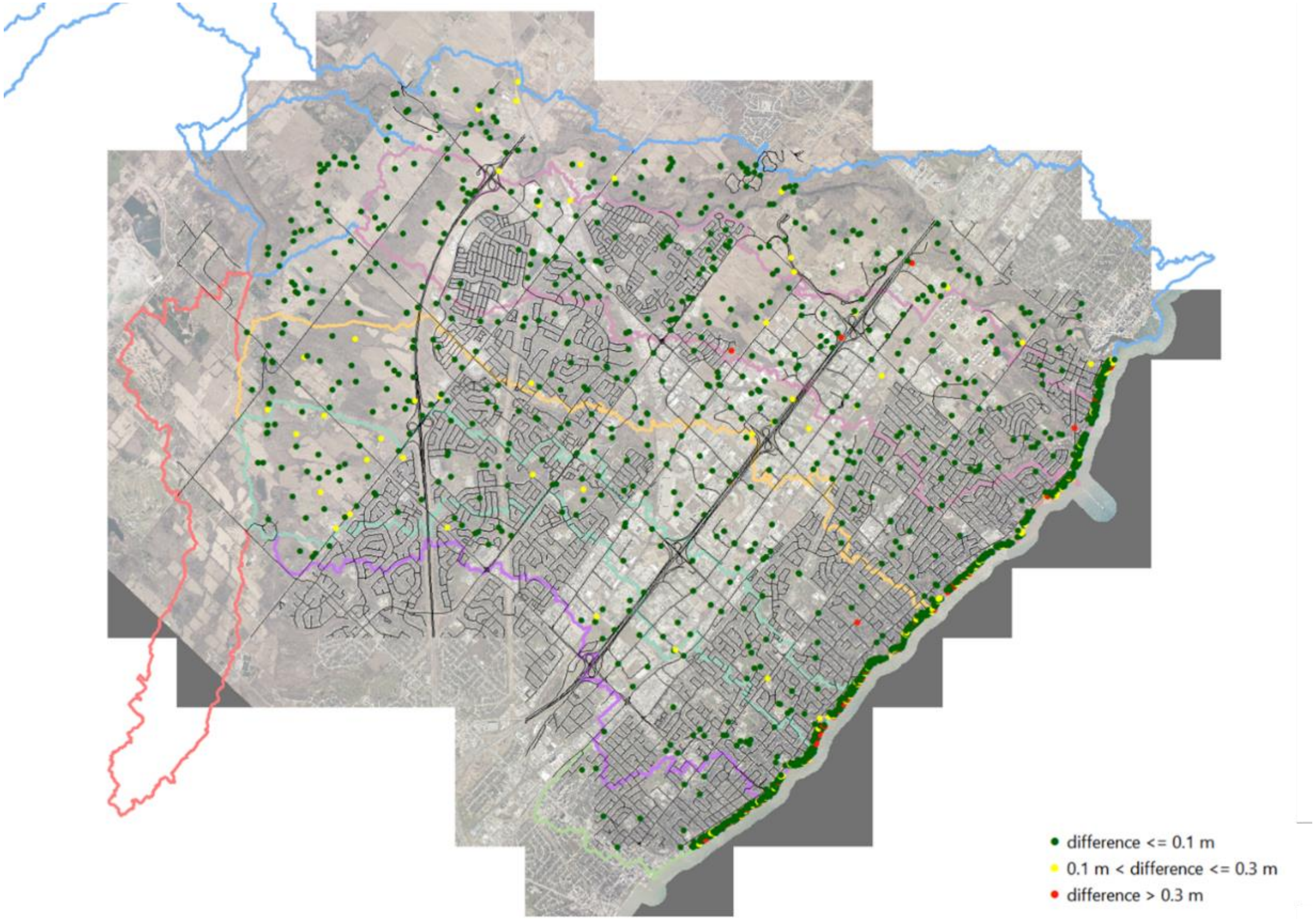


Figure 3.1. Point Elevation Check between Conservation Halton LiDAR and LIO Halton DTM

Through review of the datasets, it has been determined that the 1 x 1 m bare earth LiDAR data (Conservation Halton) is appropriate to use for subcatchment delineation (hydrologic modelling). The finer resolution 0.5 x 0.5 m LIO Halton DTM would be more appropriate to use for hydraulic analyses. This includes both the 1-dimensional and 2-dimensional modelling requirements.

3.1.2 TOPOGRAPHIC SURVEY VALIDATION

In support of the model development and data validation effort, WSP completed a topographic survey in the CGVD2013 vertical datum, to remain consistent with the LiDAR data provided for use in the current study. Topgraphic survey has been completed for key elevations of hydraulic structures as well as selected open channel sections; discussion regarding channel cross-sections can be found in a subsequent section.

The topographic survey points focused upon hydraulic structures have been categorized into three (3) categories or zones for data type – namely the following:

- Roadway Points (i.e., Centreline, Edge of Pavement, Orignal Ground)
- Upstream Structure (i.e. Inverts, Obverts, Headwalls, Wingwalls, etc.)
- Downstream Structure (i.e. Inverts, Obverts, Headwalls, Wingwalls, etc.)

Survey points related to the upstream and downstream zones of hydraulic structures are discussed in a subsequent section (ref. Section 3.2.4). The focus for validating the LiDAR data provided for use in this study has been to compare the survey points along the roadway, as these are fixed features which are typically clear of vegetation / blockages from the flight, and provide a consistent comparison point to verify the elevation results.

The hydraulic structures for which survey was completed have been reviewed to identify which locations have the road deck included within the LiDAR, and which have had the road deck removed (i.e., burned out). The structures which have the road deck maintained with the LiDAR have been used as the focus of this comparison and LiDAR verification.

For each of these locations, the LiDAR elevation has been extracted at all available roadway points to compare to the surveyed elevation; this comparison has been completed with both the LIO LiDAR and CH's Bare Earth. A summary of this comparison is shown in Table 3.1 below.

Based upon the results in Table 3.1, the surveyed elevations range from approximately +0.18 m to -0.08 m difference from the LiDAR elevations, and are on average approximately +0.04 m higher than the LiDAR (both LIO and CH sources). These differences are considered to be relatively minor, and can likely be attributed to the accuracy of the survey equipment or differences in the exact point of measurement; as such, this demonstrates a reasonable match to the LiDAR and indicates that there is no need for any adjustment to the based LiDAR topographic data for use in the current study.

Table 3.1. Survey and LiDAR Verification Summary

WATERSHED & SURVEY POINT	DIFFERENCE BTW SURVEY & LIO LIDAR			DIFFERENCE BTW SURVEY & CH BARE EARTH			# OF SURVEY POINTS
	AVG	MAX	MIN	AVG	MAX	MIN	
Appleby Average	**0.05**	**0.13**	**-0.08**	**0.05**	**0.13**	**-0.07**	**71**
CL	*0.05*	*0.13*	*-0.04*	*0.05*	*0.13*	*-0.05*	*69*
EP	*-0.02*	*0.04*	*-0.08*	*-0.01*	*0.04*	*-0.07*	*2*
Sheldon Average	**0.04**	**0.11**	**-0.03**	**0.04**	**0.11**	**-0.03**	**89**
CL	*0.04*	*0.11*	*-0.03*	*0.04*	*0.11*	*-0.03*	*89*
Shoreacres Average	**0.05**	**0.15**	**0.00**	**0.05**	**0.15**	**0.00**	**63**
CL	*0.06*	*0.15*	*0.00*	*0.06*	*0.15*	*0.01*	*60*
EP	*0.04*	*0.11*	*0.00*	*0.03*	*0.09*	*0.00*	*3*
Tuck Average	**0.05**	**0.18**	**-0.04**	**0.05**	**0.17**	**-0.03**	**54**
CL	*0.05*	*0.18*	*-0.02*	*0.05*	*0.17*	*-0.03*	*46*
EP	*0.01*	*0.03*	*-0.03*	*0.01*	*0.05*	*-0.03*	*3*
GO	*0.01*	*0.10*	*-0.04*	*0.03*	*0.10*	*-0.02*	*5*
Study Area Total	**0.04**	**0.18**	**-0.08**	**0.04**	**0.17**	**-0.07**	**277**

3.2 1-DIMENSIONAL (1D) HYDRAULIC MODELLING

1-Dimensional (1D) hydraulic modelling is proposed to be completed in the most recent non-beta version of HEC-RAS, which at the time of the completion of this study, was version 6.3.1.

3.2.1 HYDRAULIC MODEL NAMING CONVENTION

The hydraulic modelling platform, HEC-RAS developed by the US Army Corps of Engineers, allows for an input for both a "river" and a "reach" naming convention. Reaches can be a subset of segments along the primary river being modelled. The naming convention is intended to be generally consistent with the approach applied in the hydrologic model development (refer to the companion Hydrology Report), and to provide flexibility for future model refinements subsequent to the current study completion. The river and reach naming for each of the four (4) watersheds is outlined in Table 3.2, and presented visually on Drawing 1 (attached).

Table 3.2. River and Reach Naming in HEC-RAS

CREEK	SEGMENT (#)	TRIBUTARY	NAME FROM CH GIS LAYER	WATERSHED	BRANCH	REACH ID	SUB-REACH ID	HECRAS REACH NAME
Tuck	1	East Branch	Tuck East Branch	TU	E	10	1	TU_E_101
	2	East Branch	Tuck East Branch	TU	E	10	2	TU_E_102
	3	East Branch	Tuck East Branch	TU	E	10	3	TU_E_103
	4	West Branch	Tuck West Branch	TU	W	10	0	TU_W_100

CREEK	SEGMENT (#)	TRIBUTARY	NAME FROM CH GIS LAYER	WATERSHED	BRANCH	REACH ID	SUB-REACH ID	HECRAS REACH NAME
	5	Main Branch	Tuck Main Branch	TU	M	10	0	TU_M_100
Shoreacres	1	West Branch	Shoreacres West Branch	SA	W	10	1	SA_W_101
	2	West Branch	Shoreacres West Branch	SA	W	10	2	SA_W_102
	3	West Branch	Shoreacres West Branch	SA	W	10	3	SA_W_103
	4	East Branch	Shoreacres East Branch	SA	E	10	0	SA_E_100
	5	Main Branch	Shoreacres Main Branch	SA	M	10	0	SA_M_100
Appleby	1	East Branch	Appleby East Branch	AP	E	10	1	AP_E_101
	2	East Branch	Appleby East Branch	AP	E	10	2	AP_E_102
	3	East Branch	Appleby East Branch	AP	E	10	3	AP_E_103
	4	West Branch	Appleby West Branch	AP	W	10	0	AP_W_100
	5	Main Branch	Appleby Main Branch	AP	M	10	0	AP_M_100
Sheldon	1	West Branch _R2	Sheldon West Branch Reach 2	SD	W	20	1	SD_W_201
	2	West Branch _R1	Sheldon West Branch Reach 1	SD	W	10	1	SD_W_101
	3	West Branch _R1	Sheldon West Branch Reach 1	SD	W	10	2	SD_W_102
	4	West Branch _R1	Sheldon West Branch Reach 1	SD	W	10	3	SD_W_103
	5	West Branch	Sheldon West Branch	SD	W	20	0	SD_W_200
	6	East Branch	Sheldon East Branch	SD	E	10	0	SD_E_100

CREEK	SEGMENT (#)	TRIBUTARY	NAME FROM CH GIS LAYER	WATERSHED	BRANCH	REACH ID	SUB-REACH ID	HECRAS REACH NAME
	7	Main Branch	Sheldon Main Branch	SD	M	10	0	SD_M_100

The cross-section naming has been based upon the river and reach naming outlined previously, as well as river stationing which is based upon the cross-section's location along the modelled reach (distance based); these river stations have been established to two (2) decimal places, to provide sufficient detail for uniqueness while limiting the length of the IDs. In order to ensure there are no duplicate cross-section IDs across the four (4) models to limit any post-processing errors, a leading placeholder representing the watershed has been added to ensure naming is unique across all four (4) models. Additional review of the cross-section river stationing within each individual model has been completed to ensure there are no duplicates within the subject models. An example of this cross-section naming approach is provided in Table 3.3.

Table 3.3. Cross-Section Naming Approach

WATERSHED	LEADING PLACEHOLDER	HECGEORAS GENERATED XS ID (EXAMPLE)	HECRAS – XS ID (EXAMPLE)
Tuck	1 x	5.95 - 7340.69	10005.95 - 17340.69
Shoreacres	2 x	5.95 - 7699.06	20005.95 - 27699.06
Appleby	3 x	5.95 - 6573.03	30005.95 - 36573.03
Sheldon	4 x	5.95 - 7935.14	40005.95 - 47935.14

3.2.2 CROSS-SECTION ALIGNMENT, CENTRELINE AND OVERBANKS

Four (4) separate hydraulic models have been developed for each watershed (Tuck, Shoreacres, Appleby and Sheldon Creeks). The base models have been developed in HEC-RAS version 6.3.1, which includes built-in GIS tools which were previously only available in the separate HEC-GeoRAS model platform. The following subsections further describe the base model development approach.

3.2.2.1 *WATERCOURSE CENTRELINE*

A base watercourse centreline for the "watercourses to be mapped" was provided by CH at the start of this study, which was based upon the ArcHydro GIS analysis of subcatchments and drainage direction within each of the watersheds. Because the "watercourses to be mapped" layer was generated in GIS based upon the DEM, the line feature was jagged in areas due to the processing against the DEM tiles. Therefore, the "watercourses to be mapped" layer has been reviewed against the DEM and the aerial imagery to simplify the shape and confirm the accurate centreline location.

Through review and refinement of the centreline, several locations have been found where the "watercourses to be mapped" line, differed from the aerial imagery and/or the DEM; these locations have been presented to CH to receive their input and preferred approach for modelling these areas.

A primary identified location is along Shoreacres Creek, downstream of Millcroft Park Drive (ref. Figure 3.2). It was confirmed through discussions with CH (ref. e-mail Jin-Senior, August 20, 2021) that a watercourse re-alignment had been completed subsequent to the LiDAR flight conducted in 2018; therefore, CH provided the design drawings (City of Burlington Project WR13-0898), survey and 0.25 m contour information representing the watercourse re-alignment which could be integrated into the DEM for cross-sections within this area. The elvations have been converted from their original CGVD1928:78 vertical datum to CGVD2013 (for the current study DEM) by subtracting 0.40 m as specified by CH. It should be noted that additional topographic survey has also been collected this area of

Shoreacres Creek by WSP, in order to confirm the elevations and channel formation to apply in the hydraulic modelling.

Figure 3. 2. Watercourse Centrelines on LiDAR and Aerial Photo (Shoreacres Creek Downstream of Millcroft Park Drive)

Two (2) other locations have been identified where there appeared to be secondary channels or low points based upon the DEM which did not align with the “watercourses to be mapped” line. The first location is along Shoreacreas Creek north of Highway 407 (Ref. Figure 3.4). The second location is along Sheldon Creek south of Upper Middle Road (Ref. Figure 3.5). These areas have been presented to CH and it was determined that the DEM and aerial imagery would be the preferred method for assigning the centreline (ref. e-mail Jin-Zhang, August 26, 2021).

Figure 3.3. Watercourse Centrelines on LiDAR and Aerial Photo (Shoreacres Creek West Branch Upstream of Highway 407)

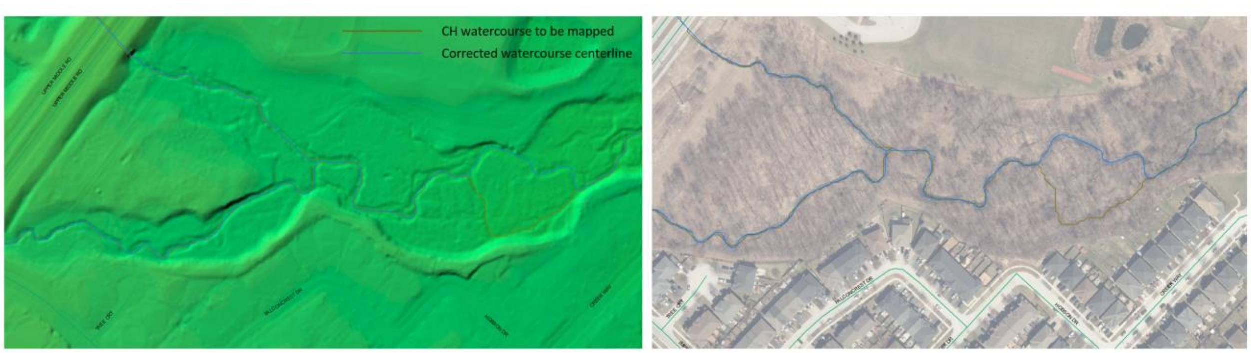


Figure 3. 4. Watercourse Centrelines on LiDAR and Aerial Photo (Sheldon Creek South of Upper Middle Road)

3.2.2.2 *LOW FLOW CHANNEL ADJUSTMENTS*

The hydraulic cross-sections have been initially cut based upon the DEM and the previously developed watercourse centreline. The completed field inventory has included a low flow channel cross-section (not surveyed) at each structure (upstream and downstream), as well as an observed water level at the time of inventory, where accessible; refer to Appendix A for further details.

It has been assumed that the threshold for low flow channels can be based upon depths observed in the field, with a focus on areas where channel depths of > 0.30 m +\- are noted, to incorporate more formative low flow channels. To assist with this review and scoping the requirement for low flow channel adjustment, a review of the water level measurement identified in the field inventory has been completed to characterize the observed water depth within each watershed system, to provide an indication of which areas might be impacted in the LiDAR data.

Based upon the current findings, the water level recordings have been grouped into three (3) categories of low flow channel potential, including the following:

- Negligible = Water Level < 0.15 m
- Minimal = Water Level (0.15 – 0.30 m)
- Warranted = Water Level > 0.30 m

A summary of the water level depth recordings and categorization is provided in Table 3.4, and a visual representation of the findings is provided in Figure 3.6.

Table 3.4. Water Level Summary Comparison

WATERSHED	WATER LEVEL MEASUREMENT (M)			LOW FLOW CATEGORY (#)		
	MAXIMUM	MINIMUM	AVG	NEGLIGIBLE	MINIMAL	WARRANTED
Appleby	0.30	0.00	0.13	20	22	1
Sheldon	0.25	0.00	0.11	40	27	0
Shoreacres	0.40	0.05	0.20	4	26	4
Tuck	0.50	0.05	0.23	3	16	10

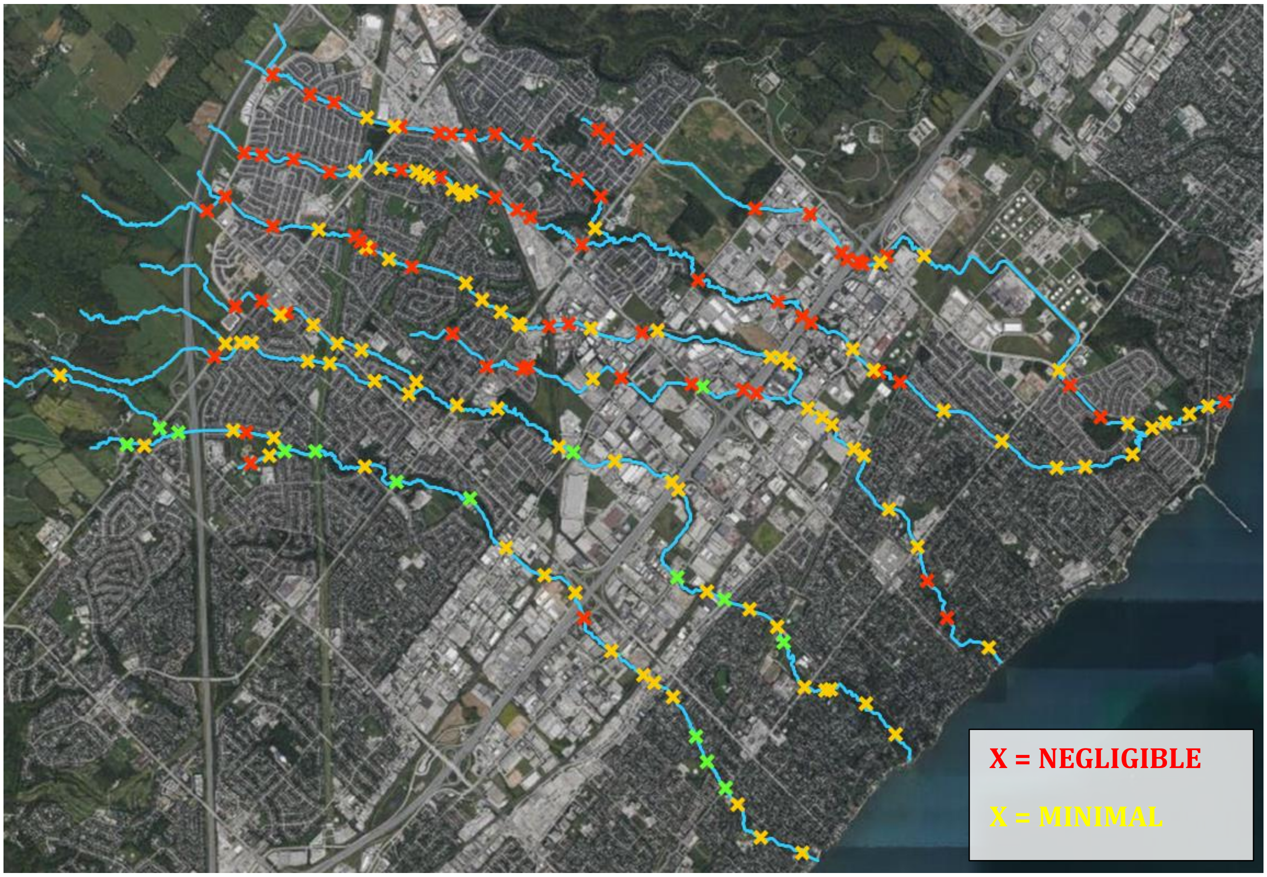


Figure 3.5. Low Flow Channel Categorization amount all Four (4) Watersheds

Based upon the results of this analysis, the key reaches of priority for low flow channel adjustment would include the lower reaches of Tuck Creek and Shoreacres Creek, which both demonstrated more consistent flow depths greater than 0.15 m at the time of assessment.

This summary and recommendation for areas of focus was presented to CH for discussion (ref. Haug-Kindellan-Jin, September 21st, 2021). CH then indicated that the area of concern is largely surrounding the LiDAR in the downstream reaches as a result of high lake levels during the time of the LiDAR capture (2018). As such, supplemental survey was undertaken in December 2021 to attempt to capture surveyed cross-sections downstream of Lakeshore Road within the watersheds, as well as complete surveyed cross-sections in the identified reaches for all four (4) creeks, through public access points such as Sweetgrass Park, Tuck Park and Nelson Park, and at Lakeshore Road. Collected cross sections included:

- Tuck Creek
  - One (1) cross-section in Sweetgrass Park
  - Two (2) cross-sections in Tuck Park
  - No cross-sections were collected downstream of Lakeshore Road as the location was inaccessible due to private property constraints
- Shoreacres Creek

  - Two (2) cross-sections upstream of the East\West confluence (south of Millcroft Park Drive)
  - Two (2) cross-sections in Nelson Park
  - One (1) cross-section downstream of Lakeshore Road
- Appleby Creek
  - One (1) cross-section downstream of Lakeshore Road
- Sheldon Creek
  - One (1) cross-section downstream of Lakeshore Road

WSP compared the survey data to the base DEM and noted only minimal differences (primarily less than 0.30 m +\-). The sections near Lakeshore Road also indicated consistent channel form to the DEM, suggesting that high Lake Ontario water levels in the spring of 2018 had minimal impact on the accuracy of the LiDAR data. Based upon the preceding, WSP concluded (ref. e-mail Haug-Kindellan-Jin, January 18, 2022) that the DEM provides an accurate representation of channel conditions, and no low flow channel corrections were warranted. CH subsequently confirmed agreement with this approach (meeting of January 19, 2022).

3.2.2.3 *OVERBANK LINES*

The overbank lines have been delineated for each watershed system through review of the DEM and aerial imagery to establish bank lines along both the left and right banks of the system; this has been established based upon the bank-full width. The overbank lines have been used as part of the subsequent model building stages to assign bank stations within each of the cross-sections. Further review has also been undertaken following the development of the base HEC-RAS model in order to ensure accuracy with respect to the bank-full stations and the cross-section elevations.

3.2.2.4 *CROSS-SECTIONS*

Cross-section locations have been established for each of the four (4) watershed systems. Preliminary cross-sections developed as part of previous stages have been further refined based on the comments provided by CH.

The cross-section locations and extents have been established based upon a variety of information, including the watercourse centreline, topographic information (contours), aerial imagery, building footprints, the existing floodplain and previously identified spill locations. The cross-section cutting approach has been applied looking downstream, from left to right, stopping at the high point on either end of the cross-section. The cross-section lengths have been established based upon the topographic information and the existing floodplain limits, which can provide an indication of the flood limits expected within each section of the model; these cross-section extents have been subsequently refined as needed through the model development.

The cross-sections have been cut to ensure that there are 4 bounding cross-sections for each hydraulically significant structure to be included in the modelling (2 upstream and 2 downstream), representing the contraction and expansion zones approaching each hydraulic structure. It should be noted that for hydraulic structures with wingwalls and/or other structural elements associated with the structure, cross-sections have been cut outside of these structures to ensure the true channel form approaching the structure is captured. Best efforts have also been made to ensure that cross-sections bounding the structures do not cross the road deck or embankment.

As part of previous hydraulic studies (ref. Section 2.2), several potential spill locations have been identified across the watershed systems; these locations and spill directions were provided by CH as part of the background review for the current study. The mapping provided by CH has been used to interpret the cross-section extents within the spill zones, and to ensure sufficient cross-section density at the spill locations, for subsequent 2D hydraulic modelling (ref. Section 3.3). These cross-sections (as well as other potential spills) have been reviewed and refined as part of subsequent model building stages, to ensure the cross-sections end at the spill point, and can be accurately used to establish spill areas to support subsequent 2D models.

For adjacent watercourses where floodlines may overlap (i.e., confluences), the proposed approach has been to keep the reaches modelled separately, by cutting cross-sections independently, ensuring no overlap, based upon the topography and floodplain associated with the subject reach. This approach helps to keep the hydraulic computations for each reach independent of one another and allows for conservative floodline delineation along both watercourse reaches.

Based upon the approaches outlined above, cross-section locations for each of the four (4) watershed systems have been established. An overview of the cross section locations is presented on Drawing 2 (attached), and a summary of the cross-section density for each model presented in Table 3.5.

As evident form Table 3.5, average cross section spacing ranges from 27.3 to 57.4 m (+/-), which is considered reasonable and appropriate. The cross-section spacing has been determined based upon standard conventions, whereby channel length is determined based upon the river centre line, and overbank distances have been determined along the path of the centre of mass for overbank flow. These are automatically calculated as part of the base hydraulic model development, and is consistent with standard hydraulic modelling practices. Additional cross-sections have been added throughout the modelling iterations as part of QA/QC processes where warranted.

Table 3.5. Cross-Section Density per Watershed Model

CREEK	SEGMENT (#)	TRIBUTARY	NAME FROM CH GIS LAYER	HEC-RAS REACH NAME	CENTRELINE LENGTH (M)	NO. OF XS (#)	AVERAGE XS SPACING (M)
Tuck	1	East Branch	Tuck East Branch	TU_E_101	2165.8	68	31.8
	2	East Branch	Tuck East Branch	TU_E_102	866.3	25	34.7
	3	East Branch	Tuck East Branch	TU_E_103	1239.0	21	59.0
	4	West Branch	Tuck West Branch	TU_W_100	480.2	17	28.2
	5	Main Branch	Tuck Main Branch	TU_M_100	7092.4	196	36.2
Shoreacres	1	West Branch	Shoreacres West Branch	SA_W_101	2152.1	59	36.5
	2	West Branch	Shoreacres West Branch	SA_W_102	2084.8	56	37.2
	3	West Branch	Shoreacres West Branch	SA_W_103	2049.8	69	29.7
	4	East Branch	Shoreacres East Branch	SA_E_100	3480.9	106	32.8
	5	Main Branch	Shoreacres Main Branch	SA_M_100	7552.3	234	32.3
Appleby	1	East Branch	Appleby East Branch	AP_E_101	1779.6	31	57.4
	2	East Branch	Appleby East Branch	AP_E_102	396.6	8	49.6
	3	East Branch	Appleby East Branch	AP_E_103	6608.1	188	35.1
	4	West Branch	Appleby West Branch	AP_W_100	4379.9	110	39.8
	5	Main Branch	Appleby Main Branch	AP_M_100	3466.2	127	27.3
Sheldon	1	West Branch _R2	Sheldon West Branch Reach 2	SD_W_201	4657.3	145	32.1
	2	West Branch _R1	Sheldon West Branch Reach 1	SD_W_101	248.1	5	49.6

CREEK	SEGMENT (#)	TRIBUTARY	NAME FROM CH GIS LAYER	HEC-RAS REACH NAME	CENTRELINE LENGTH (M)	NO. OF XS (#)	AVERAGE XS SPACING (M)
	3	West Branch _R1	Sheldon West Branch Reach 1	SD_W_102	523.9	12	43.7
	4	West Branch _R1	Sheldon West Branch Reach 1	SD_W_103	4488.4	134	33.5
	5	West Branch	Sheldon West Branch	SD_W_200	6643.6	195	34.1
	6	East Branch	Sheldon East Branch	SD_E_100	7693.5	252	30.5
	7	Main Branch	Sheldon Main Branch	SD_M_100	1003.5	35	28.7

3.2.2.5 *OTHER GEOMETRY FACTORS*

As part of subsequent model development stages, both ineffective flow areas and blocked obstructions have been included as part of the model geometry. Blocked obstructions were included in the cross-section geometry based upon the building footprint layer provided by CH for use in the current study. These have been assigned a standard height (i.e., 5 m) in the cross-section geometry to represent the building structure obstructions in the floodplain, which would inherently result in the reduction of available flow area. The base cross-sections have already been established with consideration for the adjacent buildings, to ensure those that may be within the floodplain are included in the modelling. This has been reviewed at subsequent modelling stages to ensure that any changes in floodplain limits / buildings within the floodplain are included in the model geometry.

Ineffective flow areas have been assigned at each hydraulic structure crossing, applied to both the upstream and downstream bounding cross-sections. The approach is consistent with the HEC-RAS methodology, where a 1:1 contraction rate has been applied for placing the ineffective flow areas on both sides of the structure face. On the upstream side, the ineffective flow area elevation has been assigned based upon the low point (spill point) in the roadway deck, whereas on the downstream side the elevation has been assigned based upon the midpoint between the bridge/culvert obvert and the deck low point, as WSP has applied in other floodplain mapping modelling.

The ineffective flow areas within the bounding cross-sections at structures have been assigned as “non-permanent”, based upon guidance and collaborative discussion with CH as to the modelling methodologies for these elements (ref. Appendix D). Certain structures have had the associated ineffective flow areas set to “permanent”, if the modelling results demonstrate more stable results under a “permanent” condition, these scenarios have been determined in consultation with CH.

Permanent ineffective flow areas have also been included in the overbank zones throughout the model where the cross-section includes off-line ponds and/or non-conveying flow areas (i.e., flat areas/wetlands where water will pond and have minimal velocity close to zero). This will ensure that any off-line storage is not included as part of the hydraulic analysis and resulting floodplain mapping limits.

3.2.3 HYDRAULIC PARAMETERS

3.2.3.1 ROUGHNESS COEFFICIENTS

Initial estimation of Manning's roughness coefficients has been based upon urban land use mapping, field observation and review of aerial imagery. CH has provided an extensive urban land use layer (ref. Technical Memorandum #1), which has been used as the base for developing a roughness map of the subject watersheds, in conjunction with CH's standard parameters (ref. Table 8: Manning's n Values for Channelized Flow), outlined in Table 3.6 below.

Table 3. 6. CH's Standard Parameters (Manning's n Values for Channelized Flow)

CHANNEL COMPONENT	EXISTING CONDITION	n
Channel	Concrete	0.015
	Armourstone or gabions	0.025
	Vegetated or Natural Rock	0.035
Floodplain	Asphalt/Concrete	0.02
	Manicured Grass/Lawns – rural, within 30 m of watercourse*	0.08
	Manicured Grass/Lawns – urban, within 30 m of watercourse and in public ownership or large estate lots*	0.08
	Manicured Grass/Lawns, Other	0.045
	Pasture – within 30 m of watercourse*	0.08
	Pasture – other	0.045
	Crop – within 30 m of watercourse*	0.08
	Crop – other	0.045
	Field/Meadow – within 30 m of watercourse*	0.08
	Field/Meadow – other	0.055
	Brush and Wooded	0.08

The urban land use layer provided for the study area has a total of 28 different land use categories, these are proposed to be combined into a higher category associated with the floodplain manning's n values outlined in CH's standard parameter table. The urban land cover layer has been used as provided to produce the roughness map; manual review and adjustment of the horizontally varying manning's n has been completed to ensure cross-sections do not exceed the maximum number of manning's n coefficients (max of 20). The initially proposed re-categorization of this information is presented in Table 3.7.

The Manning's roughness map has been developed based upon the categories outlined in Table 3.7 and has been subsequently reviewed for refinements within the 30 m buffer from the watercourse, as outlined in the footnotes of CH's standard parameters.

Table 3.7. Land Cover and Assumed Roughness Category

LAND USE IN GIS LAYER	LAND USE IN REFERENCE TABLE	MANNING'S n VALUE
Channel		
-	Concrete	0.015
-	Armourstone or gabions	0.025
-	Vegetated or Natural Rock	0.035
Floodplain		
Agricultural	Crop – within 30 m of watercourse Crop -other	0.08 0.045
Bare Soil	Asphalt/Concrete	0.02
Barn	Asphalt/Concrete	0.02
Cemetery	Field/Meadow – within 30 m of watercourse Field/Meadow – other	0.08 0.055
Commercial	Asphalt/Concrete	0.02
Confinement Yard	Asphalt/Concrete	0.02
Extraction	Asphalt/Concrete	0.02
Field	Field/Meadow – within 30 m of watercourse Field/Meadow – other	0.08 0.055
Forest	Brush and Wooded	0.08
Golf Course	Manicured Grass/Lawns, within 30 m of watercourse Manicured Grass/Lawns - other	0.08 0.045
Grass	Manicured Grass/Lawns, within 30 m of watercourse Manicured Grass/Lawns - other	0.08 0.045
High Density Residential	Asphalt/Concrete	0.02
Impervious	Asphalt/Concrete	0.02
Industrial	Asphalt/Concrete	0.02
Institutional	Asphalt/Concrete	0.02
Marsh	Brush and Wooded	0.08
Nursery	Pasture – within 30 m of watercourse Pasture - other	0.08 0.045
Parking Lot	Asphalt/Concrete	0.02
Pasture	Pasture – within 30 m of watercourse Pasture - other	0.08 0.045
Plantation	Brush and Wooded	0.08
Private Road	Asphalt/Concrete	0.02
Railway	Pasture – other	0.045
Recreational	Manicured Grass/Lawns, Other	0.045
Rural Residential	Manicured Grass/Lawns – within 30 m of watercourse Manicured Grass/Lawns	0.08 0.045
Transportation	Asphalt/Concrete	0.02
Urban Residential	Asphalt/Concrete & Manicured Grass/Lawns	0.035
Water	Asphalt/Concrete	0.02
Wetland	Brush and Wooded	0.08
Building	-	10

3.2.3.2 *EXPANSION AND CONTRACTION COEFFICIENTS*

Expansion and contraction coefficients for normal channel cross-sections has been set to 0.1 and 0.3, respectively. For cross-sections bounding hydraulic structures and for locations where there is a rapid change in cross-section/valley geometry, expansion and contraction coefficients has been set to 0.3 and 0.5, respectively. These ratios are used by HEC-RAS in the computation of energy losses due to flow contraction and expansion between adjacent cross-sections. The noted values are consistent with those recommended in the HEC-RAS Technical Reference Manual.

It should also be noted, with regard to structure coding, that coefficients of 0.3 and 0.5 (expansion and contraction respectively) have been applied to the two (2) cross-sections upstream of a structure, and one (1) cross-section immediately downstream of a structure. This application of expansion and contraction coefficients reflects the anticipated rapid changes occurring at these cross-sections. This approach is consistent with other floodplain mapping work WSP has completed in southern Ontario.

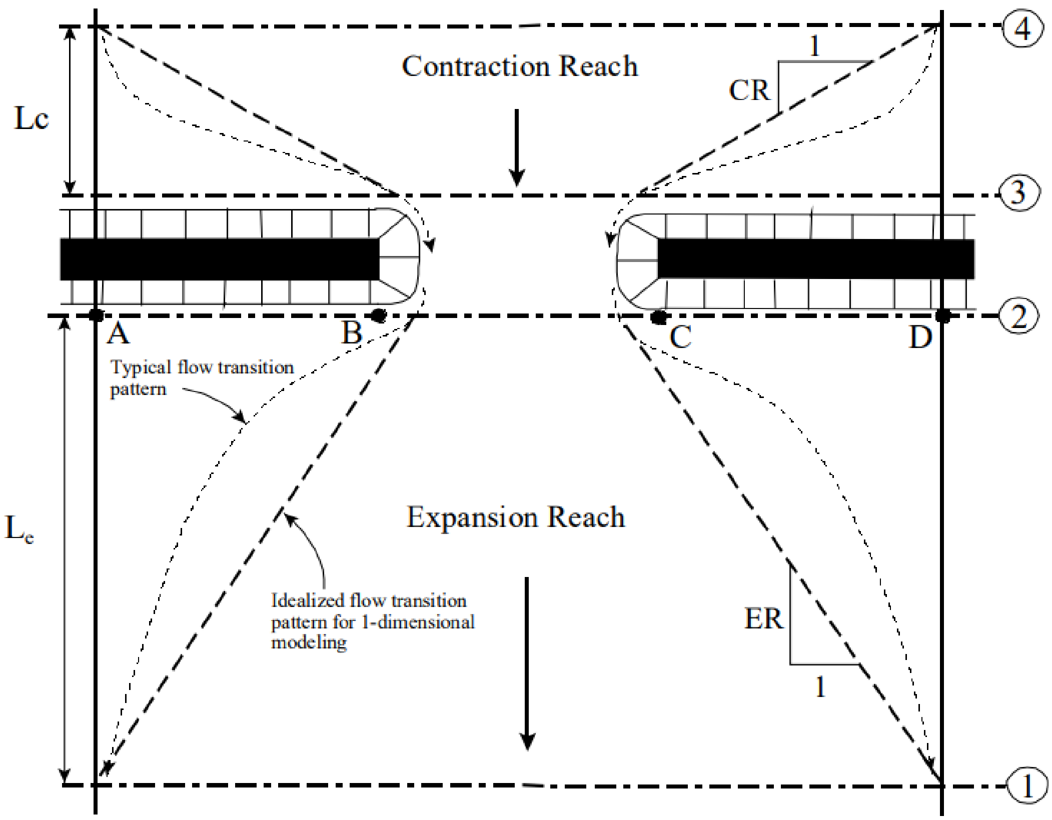


Figure 3. 6. Cross-Section Locations at a Bridge (ref. Figure 5-1, HEC-RAS Hydraulics Reference Manual)

3.2.4 HYDRAULIC STRUCTURES

3.2.4.1 DATA COLLECTION

An inventory of hydraulic structures within each of the four (4) watershed systems has been completed. As part of previous reporting it was identified that there were 206 structures within the study area based upon the itemized list provided in the RFP, however as part of the field inventory it was identified that two (2) of the structures were redundant / repeat points duplicating other structures (i.e., ID#27 and ID#108) and another structure (ID#197) was visited and was found to be removed from the channel.

As such, in total there were 203 structures identified within the study area for the inventory. Of those, there were:

- Thirty-two (32) identified in Tuck Creek,
- Thirty-nine (39) identified in Shoreacres Creek,
- Fifty-one (51) identified in Appleby Creek, and
- Eighty-one (81) identified in Sheldon Creek.

These consist of a variety of crossing types, including highway, local roads, inline weirs, railways and pedestrian crossings. The locations for all hydraulic structures currently identified within the study area is presented on Drawing 3 (attached).

The intent of the field inventory is to confirm the structure geometry (i.e., type, end treatments, opening width, span, distance from obvert to top of road, etc.) as well as identify any other pertinent observations such as low flow channel geometry, vegetation and formation of overbank zones, categorizing the road deck, among others. This information has been used as the primary source for hydraulic structure coding into the HEC-RAS models, which can be supplemented by topographic survey, as-built drawings, previous modelling and aerial imagery, where available. It should be noted that in some circumstances, field measurements may have been approximated given safety / access issues at structures (i.e., along Highway 407 and/or the Railway Crossings); in these situations, the field investigation has been used as a verification of secondary information (i.e., as-built drawings / previous modelling). The inventory sheets for those structures for which the inventory has been completed has been attached to this report (ref. Appendix A).

Based on the results of the field inventory, 193 structures were able to be accessed and included as part of the initial hydraulic field inventory. The remaining ten (10) structures, which were inaccessible, these were largely located along the Highway 407, and on private property. Given further review of drawings / data provided for the 407 ETR, additional field investigation was completed on December 16, 2021 to confirm the structure size and shape for accurate hydraulic modelling of the eight (8) culvert crossings along the 407 ETR. The remaining two (2) structures that were inaccessible were limited to structures on private property. The inaccessible structures include TU_205 (Private Driveway Structure in the headwaters of Tuck Creek west of Guelph Line, opposite Burlington Memorial Gardens) and SD_186 (Private Driveway structure for Sahi Express Trucking, located on Sheldon Creek immediately north of Rebecca Street).

Concurrent to the field inventory, topographic survey work was finalized for a short-list of structures (i.e., public roads), where safe access could be established and vegetation coverage would not pose any issues for equipment. The focus for topographic survey has been to capture the structure invert and obvert (both upstream and downstream) and establish the road deck centrelines to be included as part of the structure coding. The majority of pedestrian bridges were excluded from survey as the focus of the survey effort was placed on larger structures with more prominent road decks.

Based on the preceding, a total of 100 structures were safely accessed for survey of at least one of the three (3) survey zones identified in Section 3.1.2 (roadway, upstream and downstream). However, not all crossings were able to have fulsome survey completed due to limitations in safe access points, steep embankments, busy streets or vegetation coverage potentially impacting the survey results. A summary of the number of structures for which survey was completed and the key survey points captured to date is provided in Table 3.8.

Table 3.8. Topographic Survey Data Summary

WATERSHED	TOTAL # OF STRUCTURES	TOTAL # SURVEYED	TOTAL # WITH INVERTS OR OBVERTS	TOTAL # WITH ROADWAY CL
Tuck Creek	32	16	16	12
Shoreacres Creek	39	22	22	18
Appleby Creek	51	24	23	18
Sheldon Creek	81	37	37	27
Total	**203**	**99**	**98**	**75**

Structure coding in the HEC-RAS model has been completed for hydraulic structures denoted as hydraulically significant. The hydraulic significance of structures has been determined based upon the opening type, the structure deck and the expected impact to flow conveyance and floodplain limits. For example, pedestrian bridges, including both pre-constructed open types as well as informal crossings (i.e., some identified in the Millcroft Golf Course), have been generally proposed to be excluded from the modelling; these have been evaluated on a case by case basis and have been documented as part of the structure coding process.

Based on the results of the field inventory, a suite of proposed hydraulic structures for inclusion in the modelling has been developed. The primary focus has been placed upon bridges, culverts and inline structures which have been deemed to be hydraulically significant. A summary of the structure type identified in the field/mapping/background data, and the number of structures included in the hydraulic models (hydraulically significant) is presented in Tables 3.9 and 3.10.

Table 3.9. Summary of Structure Type Identified in the Field per Watershed

WATERSHED	TOTAL # OF STRUCTURES	TOTAL # OPEN BRIDGES	TOTAL # OF PEDESTRIAN BRIDGES	TOTAL # CULVERTS	TOTAL # INLINE STRUCTURES
Tuck	31[2]	6	7	18	0
Shoreacres	39	4	4	29	2[1]
Appleby	51	3	14	33	1
Sheldon	81	15	22	40	4
Total	**203**	**28**	**47**	**120**	**7**

Note: [1] Through field inventory, an inline weir structure was identified beneath a pedestrian bridge (SA_053), which was not identified as part of the preliminary structure list.

[2] Note, private driveway structure (TU_205) located upstream in Tuck was not visited during the field investigation and no other information was available; excluded from current summary due to unknown structure type.

Table 3.10. Summary of Structure Type included in Modelling per Watershed

WATERSHED	TOTAL # OF STRUCTURES	TOTAL # OPEN BRIDGES	TOTAL # OF PEDESTRIAN BRIDGES	TOTAL # CULVERTS	TOTAL # INLINE STRUCTURES
Tuck	24	6	0	18	0
Shoreacres	35	4	0	29	2
Appleby	41	3	4	33	1
Sheldon	61	15	2	40	4
Total	**161**	**28**	**6**	**120**	**7**

Of the structures identified in the field, the majority of the structures have been included in the hydraulic modelling, which has over 160 structures coded in the 1D models. There are a large number of pedestrian crossings identified throughout the study area; these are generally located within parks, public lands and the Millcroft Golf Course. The majority of these pedestrian bridges have been excluded from the modelling, following review of deck thickness and width over the channel to determine potential hydraulic significance.

The hydraulic structures have been given ID numbers based upon the initial list outlined in the TOR. These ID numbers have been maintained given the extent of their current use; however a leading identifier has been added related to the watershed system, much like those applied in the hydrologic modelling and the river/reach naming. Examples of the structure naming conventions are presented in the Table 3.11.

Table 3.11. HEC-RAS Structure Naming Convention

WATERSHED	IDENTIFIER	FIELD INVENTORY ID (3 DIGIT EXAMPLE)	GENERAL STRUCTURE NAME	INLINE STRUCTURE NAME
Tuck	TU_	001	TU_ST001	TU_INL001
Shoreacres	SA_	032	SA_ST032	SA_INL032
Appleby	AP_	071	AP_ST071	AP_INL071
Sheldon	SD_	123	SD_ST123	SD_INL123

The preceding naming conventions has been applied to the supporting GIS layers, field inventory sheets and future mapping/summary to remain consistent throughout the study.

3.2.4.2 *MODELLING APPROACH (STANDARD STRUCTURES)*

Culvert vs Bridge Methodology

HEC-RAS provides two (2) methods for modelling hydraulic structures, namely culvert method or bridge method. Based upon review of the currently completed field inventory, the majority of the structures within the study area consist of culverts, rather than large open footing bridges. Structures have been reviewed on a case by case basis, in order to determine whether the culvert or bridge method is considered more appropriate. In general, bridge method has been applied for open footing structures with a width of 6 m or greater, whereas the culvert method has been applied for structures matching one of the nine (9) different culvert geometries offered by the HEC-RAS modelling platform. However, adjustments may have been made based upon model stability and results. If structures appear to be culverts but have been noted as open bottom as part of the field inventory, a natural channel manning's n value (i.e., 0.035) has been applied to the bottom 0.1 m depth of the culvert.

Bridge Modelling Approach

The modelling approach for hydraulic structures for low flow methods includes the Energy (Standard Step) approach, as well as the Momentum and/or Yarnell approach for structures with bridge piers. For high flow methods, the Energy Only (Standard Step) was applied, followed by a review of any overtopping structures; structures overtopping during the Regulatory Storm event have been assigned a Pressure and/or Weir approach, to more accurately represent the flow overtop of the roadway during high flow conditions.

Weir Coefficients

The HEC-RAS Hydraulic Reference Manual recommends a weir flow coefficient of 2.6 (1.44 metric) representing weir flow over a typical bridge deck. Whereas for flow over elevated roadway approach embankments, a weir flow coefficient of 3.0 (1.66 metric) is recommended. Other studies which WSP has completed have applied 1.44 and 1.7 as the typical values for weir flow coefficients. For the current modelling effort, weir coefficients of 1.44 have been applied to all structures.

Bridge Skew

A bridge on a skew refers to a condition when a bridge opening is not perpendicular to the direction of flow or, similarly, when a pier is not aligned with the flow. The HEC-RAS User's Manual indicates that skew angle (θ) is defined as the angle between the flow path as water goes through the bridge opening and the line perpendicular to the cross sections bounding the bridge (ref. Figure 3.8).

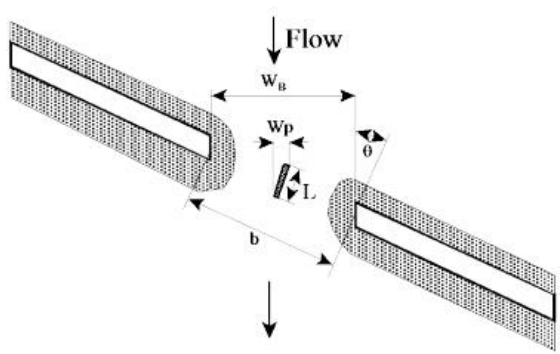


Figure 3. 7. Example of a Bridge on a Skew (ref. HEC-RAS) Manual)

A skew angle assessment was completed using Google Earth Pro™ for all crossings included in the new HEC-RAS models. Where crossings were deemed to have a skew angle greater than 30°, a skew angle was defined in the HEC-RAS model.

From review of the mapping and field inventory as part of base structure coding, no structures with skews greater than 30° have been identified. There are structures where the road deck may be skewed, but the culvert is aligned with the direction of flow. Therefore, no structures have been assigned a skew in the current hydraulic modelling.

3.2.4.3 *MODELLING APPROACH (COMPLEX AND INLINE STRUCTURES)*

Long Structures

In some locations, long enclosures are present. Such structures are typically not well represented by HEC-RAS in 1-D steady-state simulations, owing to the complex hydraulics of the enclosure (higher potential for pressure and reverse flow) as compared to the simplified hydraulic approach in HEC-RAS (i.e. energy equation).

Based upon the field inventory, a total of seven (7) long structures have been identified in the study area; it should be noted that structure IDs have been given to the upstream and downstream end separately, as outlined in the RFP. These long structures are summarized in Table 3.12.

Table 3.12. Summary of Long Structures

WATERSHED	D/S ID #	U/S ID #	LOCATION	GEOMETRY
Tuck Creek	TU_ST011	TU_ST012	QEW Enclosure	Approx 245.5 m long Concrete Box Culvert (U/S 4.1 m span x 1.8 m rise) (D/S 4.3 m span x 1.6 m rise)
	TU_ST021	TU_ST022	407 Enclosure	Approx 505 m long Concrete Pipe / Box Culvert (U/S 2.2 m Diameter Pipe) (D/S 3 m span x 1.8 m rise)
Shoreacres Creek	SA_ST043	SA_ST044	QEW Enclosure	Approx 93 m long Concrete Box Culvert (U/S 4.8 m span x 2.4 m rise) (D/S 4.1 m span x 2.3 m rise)
	SA_ST055	SA_ST056	Headen Forest Drive	Approx 104.5 m long Double Concrete Box Culvert (2.5 m span x 1.8 m rise & 1.5 m span x 1.8 m rise)
Appleby Creek	AP_ST109	AP_ST110	Appleby Line / North Service Road	Approx 112.5 m long Concrete Box Culvert (2 m span x 2 m rise)
Sheldon Creek	SD_ST191	SD_ST192	Burloak Drive / QEW	Approx 81 m long Concrete Box Culvert (U/S 3 m span x 2 m rise) (D/S 3 m span x 3 m rise)
	SD_ST194	SD_ST195	QEW Enclosure	Approx 103 m long Concrete Box Culvert (3.3 m span x 1.8 m rise)

Each of these enclosures / long structures have been coded into the 1D HEC-RAS model by applying the most conservative opening size (if upstream and downstream measurements differ).

Based on comments received from CH (December 21, 2021), the primary long enclosures of concern are:

- Tuck Creek at 407ETR (TU_ST021 to TU_ST022)
- Shoreacres Creek at Headon Forest Drive (SA_ST055 to SA_ST056)
- Appleby Creek at Appleby Line (AP_ST109 to AP_ST110)

The Tuck Creek and Shoreacres Creek long enclosures (including the QEW enclosures described in Table 3.12) have been modeled as part of the separate 2D HEC-RAS model for the 407ETR area (Area 1; refer to Section 5.0 and Drawing 6). The long enclosure at Appleby Line has been included in a separate proposed 2D HEC-RAS model (Area

5 as per Drawing 6). Long enclosures have been added as 2D connections within the 2D mesh, utilizing a normal 2D equation domain as the overflow computation method and culvert barrels were traced based on GIS data or available drawings. The embankment data was added as per survey collected by WSP and the LiDAR added as terrain within the 2D model.

Inline Structures

Inline structures associated with on-line ponds are generally incorporated into the modelling based upon the field inventory and subsequent topographic survey (where available). These structures are coded based upon geometry only and do not apply internal rating curves / boundary conditions for water levels within the model.

Based upon the field inventory completed throughout the study area, there are a total of seven (7) inline structures which have been coded into the model. A summary of the locations and type of these structures is presented in Table 3.13.

Table 3.13. Summary of Inline Structures in Study Area

WATERSHED	STRUCTURE ID	TYPE / NOTES
Shoreacres	SA_INL053	Inline Channel Weir found beneath Pedstrian Bridge – Opening Approx. 4.8 m top span x 1.4 m rise
	SA_INL066	Inline Channel Weir – Trapezoidal Opening Approx. 0.4 m bottom span x 0.3 m rise
Appleby	AP_INL118	Millcroft Golf Course Online Pond Outlet – Approx. 1.2 m rise Weir along edge of Pond
Sheldon	SD_INL146	Online Pond (ID#805 – Not Included in Hydrology) Berm Outlet and rectangular opening – Approx. 0.3 m dia
	SD_INL158	Online Pond (ID#804 – Included in Hydrology) Berm Outlet – Approx. 0.6 m dia
	SD_INL175	Online Pond (ID#818 – Not Included in Hydrology) Berm Outlet – Approx. 0.7 m dia
	SD_INL201	Online Pond (ID#808 – Not Included in Hydrology) Berm Outlet – Approx. 0.5 m span X 3.2 m rise

Of the inline structures identified within the study area, two (2) are associated with channel control structures, whereas the other five (5) are associated with online ponds (either formal or informal SWM control). One (1) of the structures located within the Sheldon Creek watershed is associated with a SWM facility (Pond 804) which has been included in the hydrologic modelling based on the screening completed by CH. These structures have been coded based upon the geometry observed / measured in the field as part of the structure inventory.

3.2.5 STEADY FLOW TABLE

The steady flow table has been developed based upon the peak flows generated as part of the hydrologic modelling for each subject watershed which has been completed in parallel to the hydraulic modelling. The hydraulic modelling has been simulated for the 2, 5, 10, 25, 50, 100-year and Regional Storm events, and the August 4th, 2014, storm event. As per the study TOR, the preceding has been based on the future land use scenario from the hydrologic modelling. Return period flows for the future land use scenario will apply the City of Burlington's currently approved rainfall IDF, which includes an additional 15% above current values in order to account for the impacts of climate change.

It should be noted that the steady flow input for both the Regional Storm and the 100-year event has been developed based upon results from up to three (3) different hydrologic scenarios and the iterative analysis with 2D modelling results, which include:

- **Base (No Spills) Scenario** – no external flows from spill (local watershed flows only).

- **Inter-Basin Spill Scenario** – includes inter-basin spills are occuring from one watershed to another (i.e., Appleby to Sheldon), when the spill flow meets to applicable spill criteria (greater than 5 m^3/s and/or represents greater than 10% of the receiving system flow).

- **Blended / Intra-Basin Spill Scenaro** – in addition to any inter-basin spills, this scenario includes any intra-basin spills which occur within the same watershed (i.e., flows spill from one branch to another in the same watershed, e.g.,Shoreacres East to Shoreacres West) which meet the same threshold criteria outlined above.

The steady flow table for both the Regional Storm and the 100-year events have been generated with the consideration of these spill scenarios to ensure that the input to the 1D hydraulic modelling is consistent with that of the results of the 2D modelling, and is aligned with CH's Spill Modelling Approach (ref. Appendix D). This ensures a conservative modelling scenario to represent the flood hazard limits for these riverine systems.

The current flow change locations have been established based upon a review of all available flow nodes from the hydrologic models, noting key locations throughout the watershed (i.e. upstream of confluences, at roadways, downstream of ponds included in hydrologic modelling, etc.) and including additional flow change locations in between key points when the flow change moving downstream exceeds the identified 10% threshold for the regulatory event. The flow changes have been applied at the upstream extent of the reach / subcatchment, which allows for the most conservative modelling approach for the subject reach. Best efforts have been made to locate flow change locations outside of the four (4) cross-sections bounding a hydraulic structure, to ensure that a consistent flow rate is applied throughout the structure.

A detailed review has previously been conducted in order to identify any locations where the change in peak flow between flow nodes moving downstream exceeds the 10% threshold based upon the Regional Storm event. Based upon the steady flow table, there are several flow change locations which exceed the 10% threshold moving downstream; however, the majority of the larger percent exceedances are relatively minor when the absolute change in peak flow is considered (less than 5 m^3/s). Through discussions with CH, flow change locations where the change in flow moving downstream is both greater than 10%, as well as greater than 5 m^3/s have been reviewed. For the most part, these increases are as a result of urban drainage confluence points, downstream of SWM facilities, or other conditions where the timing influences of contributing runoff result in larger differences. These areas have previously been reviewed with CH and it has been determined that there are minimal opportunities to mitigate the increases at this scale of study.

A visual representation of the selected flow nodes and the flow change locations (cross-sections) has been presented on Drawing 4 (attached). The governing/regulatory storm for all reaches within the four (4) watersheds is the Regional Storm under the Base Model conditions except for:

- Several nodes located on an Appleby reach (AP_W_100),

- Three (3) Sheldon Creek reaches (SD_W_201, SD_W_103, SD_E_100)

- One node located on a Tuck Creek reach (TU_W_100)

The full steady flow tables along with regulatory event information and any exceedances for each watershed hydraulic model can be found in Appendix B.

3.2.6 BOUNDARY CONDITIONS

As documented in the Great Lakes Technical Guide Part 3 Flooding Hazard (Ontario, 2001):

> *"Determining the relevant flooding hazard limit at the junction of a lake and river or stream is based on an evaluation of which flooding hazard limit governs the site, namely the flooding hazard limit for large inland lakes or the flooding hazard limit for river and stream systems. In other words, the decision on which limit applies is based on which factors most influence the level of the flood risk or hazard at a given location.*
>
> *Determining which flooding hazard limit applies is based on the same principles outlined in the Technical Guide for River and Stream Systems (MNR 1996) and are as follows:*
>
> *Rivers flowing into large inland lakes require an analysis of the respective river and lake flood levels. Where the high water conditions at the junction are generated by two independent flood events, the flooding hazard limit should be based on the higher of:*
>
> *i. mean annual lake level and the river and stream systems flooding hazard limit as shown in Figure 4.12, Section A-A';*
>
> *or*
>
> *ii. large inland lakes flooding hazard limit as shown in Figure 4.12, Section B-B'.*

Figure 4.12 from the Great Lakes Technical Guide is replicated as Figure 3.9 in this report.

The aforementioned guidance is consistent with guidance provided in the *Technical Guidelines for Flood Hazard Mapping* (EWRG, 2017).

As noted previously, the downstream limit of the new hydraulic models is Lake Ontario. The surface area of Lake Ontario is about 19,000 square kilometres[1]. Given the size of Lake Ontario, it can be reasonably concluded that a flood event impacting Lake Ontario and one impacting the Tuck, Shoreacres, Appleby and Sheldon Creek Watersheds would be independent events.

The Lake Ontario monthly mean water level based on recorded water surface elevations over the period 1918 to 2019, referred to International Great Lakes Datum 1985 (IGLD 1985), is 74.77 m.[2]

[1] Source: https://www.thecanadianencyclopedia.ca/en/article/lake-ontario

[2] Source (accessed October 2021) : Fisheries and Oceans Canada via URL http://www.tides.gc.ca/C&A/network_means-eng.html

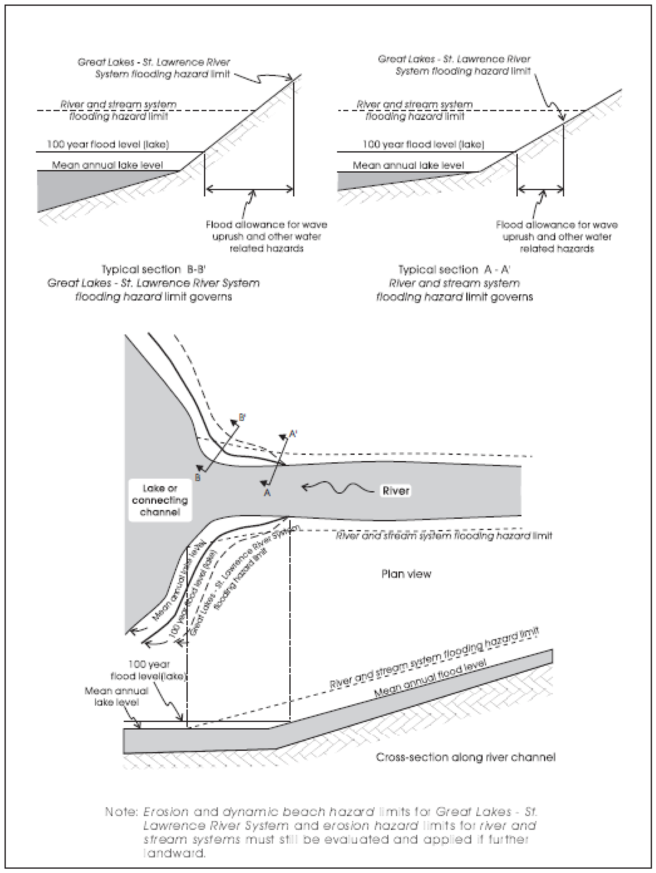


Figure 3. 8. Flooding Hazard Limits at Junction of River and Lake [source Figure 4.12, Great Leaks Technical Guide Part 3 Flooding Hazard (Ontario, 2001)]

In June of 2019, the Lake Ontario water surface elevation was recorded as 75.91 m (IGLD85), which is the highest water level for Lake Ontario on record since reliable records began in 1918. Locally, this exceeded the documented Ministry of Natural Resources and Forestry's 1 in 100-year instantaneous lake level.

Based upon this and the direction from CH, the Lake Ontario boundary condition for each of the new hydraulic models is proposed to be set to an elevation of 74.275 m (CGVD2013) for the current project, which is representative of the mean average water level (1918 to 2019) for Lake Ontario (approximately 74.77 m in IGLD85 based on a conversion of -0.495 m, based upon NRC Station 59U9526). This boundary condition at the model outlets remains consistent for all simulated events.

3.3 2-DIMENSIONAL (2D) HYDRAULIC MODELLING

Consistent with the approach to 1-Dimensional (1D) hydraulic modelling, 2-dimensional (2D) hydraulic modelling was completed in the most recent non-beta version of HEC-RAS, which at the time of the completion of this study, was version 6.3.1.

3.3.1 APPROACH TO DEFINING SPILL AREAS

The MNRF's Technical Guide River & Stream Systems: Flooding Hazard Limit (2002) (ref. Section 4.13 of the guidelines) defines a spill as occurring when flood levels overtop the banks of a watercourse and spill overland away from the watercourse channel. Frequently, this spill will move into another watershed or join the originating watercourse at a distance downstream. Further, the guidelines describe that:

"The effect of spills moving into another watershed should be assessed to determine the potential flood risks. Alternative measures should be investigated to prevent the spill moving into the adjacent watershed. If the amount of spill is relatively small, less than 10% of the peak flow, the floodplain mapping for the watercourse should be based on the original flow, without any deduction for the spill. For larger spills, allowance for the reduced flow should only be made where the review of alternatives proves that the spill cannot be prevented, either because there are no feasible alternatives or the costs, when compared to the potential benefits, are too high. Where the spill re-joins the watercourse further downstream, the route of the spill should be examined to determine the potential harmful effects of overland flow. No reduction should be made for the spill in the downstream floodplain computations."

1D hydraulic models incorporating all hydraulic structures and topographic survey have been reviewed to identify cross-sections which are uncontained during the simulated events. These areas have subsequently been reviewed with CH and the TAC to identify areas with significance for large inter-basin spills and/or areas having a high concentration of adjacent spills, which were prioritized for modelling using 2D techniques, as described in the sections that follow.

3.3.2 MODELLING APPROACH

For the identified spills to be modelled a pure 2D modelling approach is considered to be preferred as opposed to a combined 1D-2D approach. It should be noted that there are limitations in HEC-RAS when applying an integrated 1D/2D model, whereby spill occurring at a hydraulic crossing within the 1D portion of the model will be transferred downstream (within the 1D model) and not to the 2D model flow areas. Further, given the complexities of linking the two systems through lateral structures, the benefits of modelling the primary channel in 2D, as well as the more robust hydraulic calculations for structures, such as long enclosures, 2D models as opposed to 1D-2D models were developed for spill areas. A total of seven (7) 2D models have been developed for the identified potential spill locations in HEC-RAS 6.3.1.

Spills have been assessed using the actual (unsteady state/time varying) hydrographs. This is consistent with initial comments from CH (ref. September 15, 2021, Comment 10, which notes consistency with the approach taken for the assessment of other studies such as the Morrison-Wedgewood Diversion Channel) and the CH-WSP team meeting of October 25, 2021.

Notwithstanding the preceding, it is acknowledged that in some cases undersized hydraulic structures may limit flows to downstream receivers using the unsteady state approach (i.e. with the hydrograph time series). Modelling results have been reviewed to determine the impacts of hydraulic structures, and whether the degree of attenuation is considered significant or not.

Several potential approaches were reviewed and considered, including theoretical hydraulic structure upgrades, or the implementation of quasi-steady state hydrographs to consume available storage volume. Both methods were considered to be fairly onerous and cumbersome however; theoretical hydraulic structure upgrades also provide an additional degree of uncertainty.

Based on subsequent discussions with CH and the approach employed on other studies (such as the Major Transit Station Area analysis for the Burlington GO and Downtown areas within the Hager-Rambo system) the preferred approach has been determined to generate an alternative version of each 2D modelling area that modifies the terrain within RAS Mapper to manually remove all of the structures ("hydroburning") and constriction caused by structure embankments to allow free flow of water similar to an open channel. This allows for a calculation of the resulting unattenuated flows. These have been compared against the base 2D modelling flows, i.e., 2D model with structures to determine the magnitude of the flow attenuation. Where a notable attenuation has been indicated, the difference in the hydrographs has been added back into the base 2D modelling.

3.3.3 2D MODEL DEVELOPMENT

3.3.3.1 DEM AND TERRAIN REFINEMENT

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 with buildings has been provided by CH to use as terrain in the HEC-RAS model. The LiDAR data uses a vertical datum of CGVD:2013, which differs from the typical City standard datum of CGVD28:78.

The LiDAR was reviewed after it was imported in RAS Mapper and modifications were made near structures to appropriately represent culvert inverts and generate clean bridge upstream and downstream bounding cross-sections as well as internal bridge cross-sections.

Further, to model the 407ETR and QEW median barriers, the LiDAR data within RAS Mapper has been modified. The solid sections of the barrier have been considered to be a closed "tall-wall concrete barrier" (confirmed from Google Street View), and has therefore been assumed to have a height of 1.05m (taken from Ontario Provincial Standard Drawing 911.132, ref. Figure 3.10). The 407-ETR concrete median barrier was confirmed to run across the 2D model extents, in contrast to the QEW barrier where there are sections of open guard rail. The sections of open guard rail, having an opening height of 0.465 m, were assumed to allow flow through them (as per MTO Detail 925.100, ref. Figure 3.11). The width of the concrete barriers have been assumed to be 1 m with a side slope of 0.1 to appropriately represent the geometry within HEC-RAS. The 2D cells were enforced with the help of breakline placement on the top of the barrier.

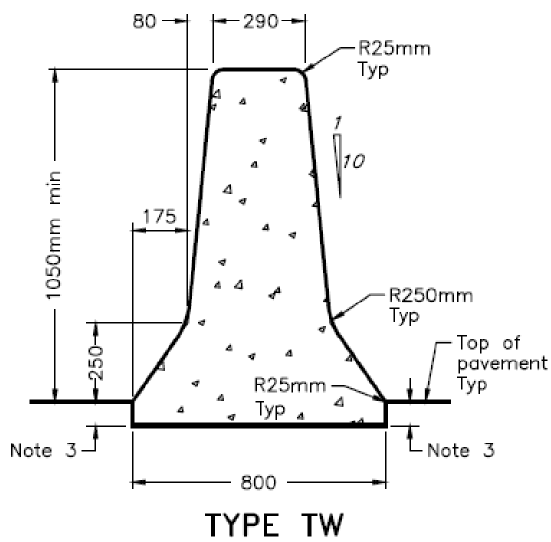


Figure 3.9. Tall-Wall Concrete Barrier Cross-Section (ref. Appendix C).

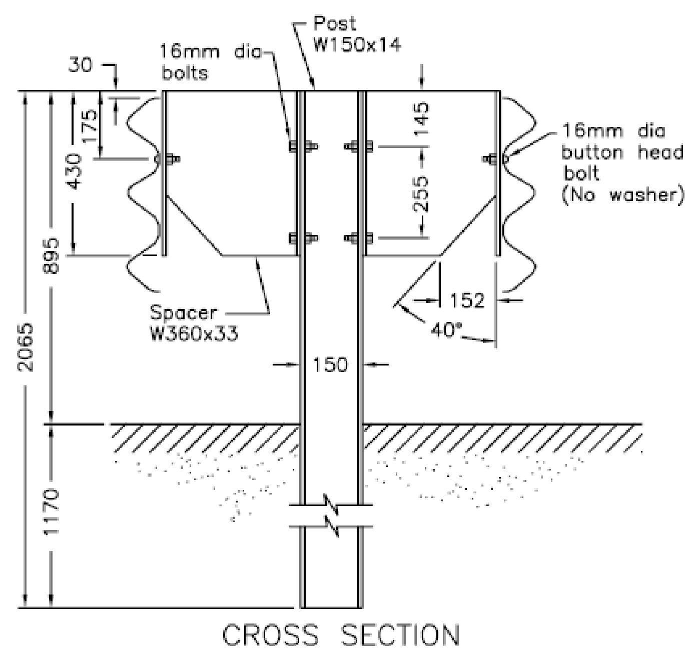


Figure 3.10. Thrie Beam Guide Rail Median Barrier Cross-Section (ref. Appendix C)

The LiDAR surface provided by CH already included buildings except for a few recent developments that were not included. These were added later within RAS Mapper by modifying the terrain. If a building seemed significant to obstruct the flow of water, additional breaklines were added to enforce cells in the 2D mesh for better flowpaths around the building.

Further, the terrain at underpasses within the 2D mesh has been adjusted if needed using terrain modification tools in RAS Mapper to avoid unrealistic accumulation of water. The LiDAR at the bridge faces has also been adjusted within RAS Mapper if needed to generate clean upstream and downstream bounding cross-sections as well as internal bridge cross-sections.

3.3.3.2 *MESH DEVELOPMENT*

The basin boundaries for 2D models have been generated using ArcPro GIS software to setup the 2D mesh for the potential spill areas. Model boundaries have been created so that they cover greater extents without accumulation of excess water and end on high ground. A cell resolution of 5 m has been chosen as default to generate the computational 2D mesh. Breaklines and refinement areas were applied to adjust the cell faces and provide a detailed mesh representative of varying topographic features. The purpose of applying breaklines and refinement areas at featured locations such as road centerlines, top and bottom of curbs, barriers, and other stypes of slopes is to be representiatvie of the actual flow paths. Examples would be to capture significant barriers to flows (e.g. New Jersey Barriers) and flow potential within narrow concentrated flow corridors. As deemed necessary, transportation breaklines have been enforced using a 3 m near spacing and 5 m far spacing with single or multiple repeat cells. Refinements to the mesh have been applied carefully, since a finer resolution could easily increase the number of cells within a model potentially leading to longer run times. Stream breaklines have been enforced with a finer resolution of 3 m far and near spacing with single repeat cells to cover the entire width of the channel.

3.3.3.3 *STRUCTURES IN 2D*

Structures in the 2D models have been added by utilizing SA-2D Connections and have been enforced in the mesh with a finer resolution or a default resolution of 5 m as deemed necessary. A normal 2D equation domain was chosen as the overflow computation method. Culvert barrels were traced based on GIS data, areal imagery or available drawings. Embankment data has been added as per survey collected by WSP and the LiDAR added as terrain within the 2D model.

Bridges inside of 2D flow areas can handle full range of flow regimes. Bridge data has been entered inside of 2D areas similarly to the modeling of bridges in a 1D model. However, for 2D modeling, the bridge's curves are used to obtain a water surface difference through the bridge for each set of cells being used to model the bridge. This water surface difference is then equated to a force. That force is distributed and inserted into a special version of the momentum equation for each set of cells spanning the bridge centerline. Instead of calculating friction forces, pressure forces, and spatial acceleration forces, these forces are obtained from the bridge curves. Then the 2D equations are solved as they are normally solved at any cell/face in the model. This approach used for 2D modeling allows for equivalent forces to be computed for low flow, pressure flow, and combined pressure flow/weir flow, or even low flow/weir flow. The amount of force given to each cell is based on the percentage of the total flow passing through that particular set of cells. This 2D modeling approach allows for varying flow, water surface, and velocity at each of the cells around the centerline of the bridge opening. Therefore, the flow is still computed as two-dimensional flow through and over top of the bridge. Flow can pass at any angle through the bridge opening based on the hydraulics of the flow and the number of cells being used to represent the bridge opening. This 2D modeling approach allows for modeling highly skewed bridges without requiring a special option for users to enter bridge skew (which is necessary for 1D modeling). Hydraulic property table (HTab) parameters for bridges have been chosen so as to result in a most stable run and hydrographs at the structure. The number of points on a free flow curve, number of submerged curves and number of points on each submerged curve were set on a trial-and-error basis to generate smooth free flow/tail water curves. The head water elevations have been set slightly higher than the deck elevations to prevent any curves from crossing each other, and similarly the maximum flow that could pass through the bridge was slightly higher. The geometry pre-processor has been re-executed each time the HTab parameters were changed and the bridge family or rating curves were reviewed for hydraulic accuracy. The LiDAR at the bridge faces has also been adjusted within RAS Mapper if needed to generate consistent and clear upstream and downstream bounding cross-sections as well as internal bridge cross-sections.

3.3.3.4 *SURFACE ROUGHNESS*

As outlined previously, a shapefile for the land use has been provided by CH for use in the current study. A roughness layer has been prepared, as summarized previously in Section 3.2.3.1 (Tables 3.6 and 3.7). This roughness data layer has been similarly applied for 2D modelling by importing into RAS Mapper Channel roughness has been applied by using a buffer of streamlines and then imported as “calibration/override regions” in RAS Mapper. The sections of the channel roughness layer have been clipped so that they do not overwrite the transportation Manning’s or culvert Manning’s n values.

3.3.3.5 *BOUNDARY CONDITIONS*

In general, it is expected that 2D models will extend sufficiently far to convey flows back to the channels, such that at the point of overlap, the 1D model would govern. Therefore, it is not expected that the boundary conditions for the riverine system at the downstream limits will be overly sensitive; a normal depth boundary condition is therefore likely reasonable in most cases. Additionally, normal depth boundary condition lines have been applied at each section along the 2D area boundary where water has been expected to leave the system to avoid artificial rise of water surface elevations. Channel slope in the LiDAR elevation data, as deemed appropriate, has been applied as normal depth slope for each boundary condition.

3.3.3.6 *INFLOW HYDROGRAPHS*

Inflow hydrographs extracted from VO models developed during the hydrology development part of the study have been applied at appropriate locations (similar to 1D models) utilizing internal boundary condition lines with an energy slope of 0.01 to distribute flows. Internal boundary condition lines were placed in the center of channel longitudinally covering 4 to 5 cells to distribute flows.

To account for appropriate volume of water in reaches, full hydrographs have been applied at upstream extent of the 2D models whereas for subsequent downstream locations, the upstream hydrographs have been subtracted from downstream hydrographs to only account for additional volume of water.

3.3.3.7 *SPILL MODELING AND FLOW BALANCING WITHIN 2D MODELS*

As noted in Section 3.3.2, an additional adjustment is required to address the loss of flow and volume associated with the attenuation behind hydraulic structures. In order to be consistent with Provincial Policy (i.e. MNRF, 2002) and the approach for steady-state 1D hydraulic modelling, this loss cannot be included given the potential for future hydraulic structure upgrades or structure wash-out during a major flooding event.

Based on subsequent discussions with CH and the approach employed on other studies (such as the Major Transit Station Area analysis for the Burlington GO and Downtown areas within the Hager-Rambo system) the preferred approach has been determined to generate an alternative version of each 2D modelling area that modifies the terrain within RAS Mapper to manually remove all of the structures (“hydroburning”) and constriction caused by structure embankments to allow free flow of water similar to an open channel. This allows for a calculation of the resulting unattenuated flows. These have been compared against the base 2D modelling flows, i.e., 2D model with structures to determine the magnitude of the flow attenuation. Where a notable attenuation has been indicated, the difference in the hydrographs has been added back into the base 2D modelling.

In order to bring the 2D modeling into greater conformance with the Provincial guidelines, modeling has been completed based on the “Spill Modelling Approach” guidelines memorandum provided by CH dated May 19, 2022 (ref. Appendix D).

Per CH’s guidelines, three separate model scenarios have been developed for each 2D area:

- **Base Model** – This model scenario represents the extent of flooding likely to occur based on the current system conditions, (i.e., existing crossings have full flow capacity, any attenuation associated with the crossing structures reduces downstream flows, and when spills leave the channel, the flow is not maintained downstream).

- **Hydro-burn Model** – This model scenario represents an idealized situation where every riverine crossing has been 'replaced' with a natural valley cross section. This model represents the maximum potential conveyance capacity of the valley system.

- **Balanced Flow Model** – This model scenario represents a melding of information from the base and hydro-burn models. In this scenario, existing infrastructure (crossings) are modelled as in the base model, however the system flows are adjusted to better align with Provincial direction.

The base model flows are adjusted as follows:

- inter-basin spills are added into the receiving river system (as per screening criteria noted in Appendix D),

- flows downstream of structures are increased based on the flow predicted within the hydro-burn model to eliminate any routing associated with storage behind the structure, and

- flows downstream of non-credited spills, that leave the system, are increased to add spill flows back into the riverine model.

3.3.3.8 *SIMULATION SETTINGS*

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 the computation is inherently more stable. However, there are applications where the 2D SWE could be used for greater accuracy.

The Full Momentum Equation should produce more accurate results if there are highly dynamic flood waves, abrupt contractions and expansions or flat sloping river systems. However, this may take more computational power and potentially lead to longer run times. The selection of Diffusion Wave Equation or Full Momentum Equation depends upon the comparison of initial simulation results. WSP has conducted tests on using the two different 2D equation solver set that have been discussed in subsequent sections and used the 2D Diffusion Wave Equation for all the 2D models. Further, all 2D models were executed using a 1 second base time step for a 24-hour duration of model simulation time to allow enough time for the flood wave to reach peak before the downstream end of the modeled area.

Over the course of this Study, Conservation Halton has advanced spills policy updates. Work on the spills policies is ongoing; however, it is anticipated that an enhanced understanding of depth and velocities within the spill zone will be beneficial to support a risk-based policy approach. It is therefore recommended that future studies consider the use of the more complex Full Momentum Equations. Use of this study's 2D models (which are based on the Diffusion Wave Equation) remain appropriate for regulatory modeling and mapping purposes until such a time that updated modeling becomes available.

4 1-DIMENSIONAL (1D) MODELLING RESULTS

4.1 SENSITIVITY AND UNCERTAINTY ANALYSIS

With respect to the 1D hydraulic modelling, WSP has completed a scoped sensitivity analysis to assess model sensitivity and confirm validity of results. Similar to the work completed for other Conservation Authorities, the sensitivity analysis focused upon key parameters, namely peak flow, Manning's Roughness Coefficients, and boundary conditions. The analysis has considered the resulting impact to water surface elevations. The analysis has also considered how the number of critical depth occurrences is impacted. Further details are provided in Appendix E.

4.2 EXISTING CONDITIONS 1D MODEL VALIDATION

4.2.1 MODELLING APPROACH

The initial draft 1D hydraulic model had been simulated at earlier stages of the project using the existing conditions hydrologic flows for the 100-year and Regional (12-hour and 48-hour) storm events, as well as the August 2014 storm event in order to support overall hydraulic model validation (separate from the hydrologic model validation noted in the separate Hydrology Report).

The 12-hour Regional Storm event generally generated the highest peak flows for all four (4) watersheds and has therefore been used as the primary source for delineating initial inundation limits for validation purposes. The RAS Mapper function in HEC-RAS has been used to plot the simulated inundation boundary for the 12-hour Regional Storm event, based upon the model terrain and the computed water surface elevations at each cross-section. This has been completed to enable an initial comparison between the initial (existing condition) draft models and existing floodlines (i.e., CH's currently approved floodplain limits). In addition, a comparison has been made to the field observations identified as part of the August 2014 event, demonstrated by the observed estimated floodline layer (as per Conservation Halton, 2015), as well as photos / videos submitted as part of the recent Public Information Centre (PIC) held on October 14, 2021.

Details regarding the initial 1D hydraulic modelling results in terms of comparison to existing floodlines, field observations and preliminary identification of spill zones is provided in the subsequent sections.

It should be noted that the results presented in the following sections are based upon an initial version of the modelling developed in earlier stages of the project, and does not reflect the results from the final approved modelling. Further details regarding the results for the final flood hazard delineation is provided in Section 4.3.

4.2.2 COMPARISON TO EXISTING FLOODLINES

The initial modelling generated floodlines for the 12-hour Regional event for each of the four (4) 1D hydraulic models which have been compared against the existing floodplain limits provided by CH. As would be expected with a modelling update, there are differences observed in the initial floodlines when comparing the existing limits. In general, the differences appear relatively minor (ref. Figure 4.1 as an example), which can likely be attributed to improvements in the DEM used for both modelling and plotting of the floodlines as well as differences in the applied flow rates.

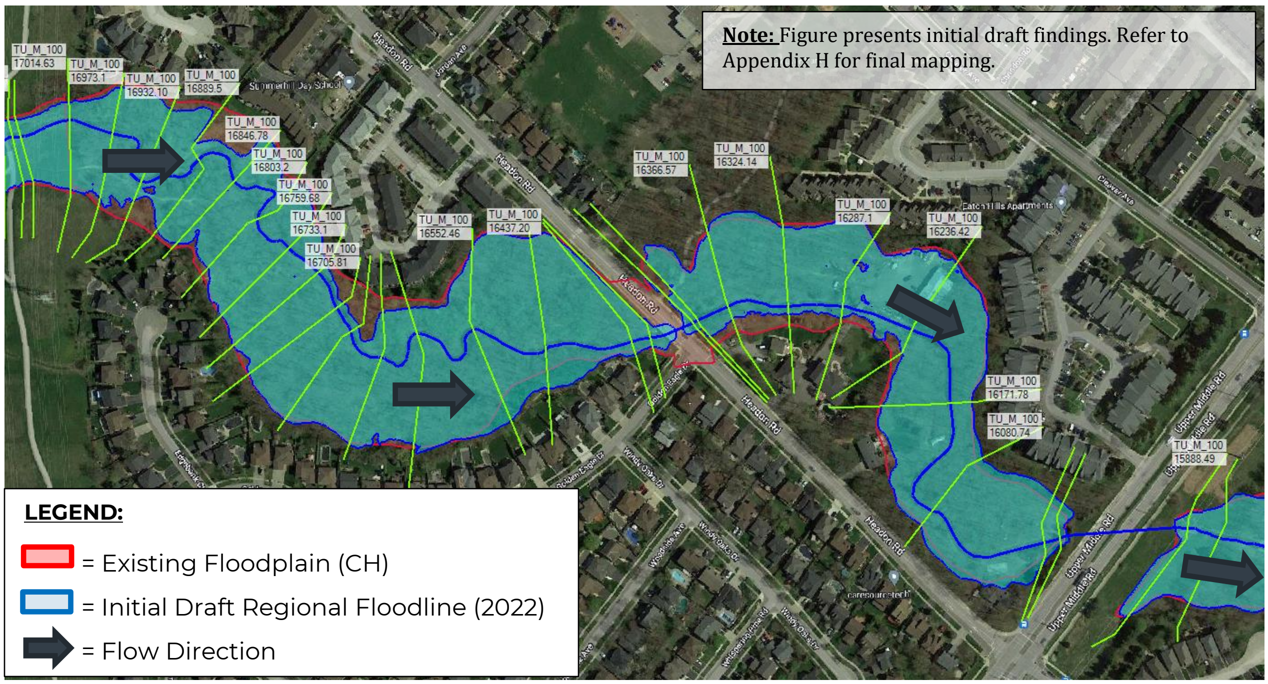


Figure 4.1. Example of Minor Differences in Floodlines on Tuck Creek (TU_M_100)

However, in certain areas there were more notable differences in floodlines, which may include roadway structures either newly overtopping or alternatively no longer overtopping, expansion / contraction of floodlines, and/or the identification of new spill areas. Refinement in this initial draft modelling and floodline validation notwithstanding, some example areas of change based upon the previous initial draft modeling included:

- Shoreacres Creek – Newport Park – Expansion of the floodplain.
- Shoreacres Creek – Nelson Park – Expansions of the floodplain.
- Shoreacres Creek – Upstream of Shoreacres Road – Expansions of the floodplain.
- Appleby Creek – Millcroft Golf Course (Holes 10 & 11) – Expansions of the floodplain.
- Appleby Creek – Upper Middle Road (AP_090) – Roadway overtopped.
- Appleby Creek – Frontenac Park – Expansion of the floodplain.
- Sheldon Creek – Millcroft Golf Course (Holes 13 & 14) – Expansions of the floodplain.

- Sheldon Creek – Appleby Line (SD_144 & SD_173) – Roadway overtopped.

- Sheldon Creek – Shell Park – Expansions of the floodplain.

These differences are likely attributable to the the refined DEM applied in the current study, potential hydraulic structure updgrades / roadworks since the completion of the previous study, and the changes in peak flows from the updated hydrologic modelling.

As summarized in the Hydrology Report, the existing conditions peak flows from the new hydrologic models compared to the previous flows are as follows:

— Tuck Creek

  — 100-Year Storm Peak Flow Differences from the 2012 Study (SWMHYMO) ranged from -41% to +25% throughout the watershed, with -14% at the Lake Ontario outlet.

  — Regional Storm Peak Flow Differences from the 2012 Study (SWMHYMO) ranged from -19% to +4% throughout the watershed, with -6% at the Lake Ontario outlet.

— Shoreacres Creek

  — 100-Year Storm Peak Flow Differences from the 1997 Study (GAWSER) ranged from -58% to +42% throughout the watershed, with +16% at the Lake Ontario outlet.

  — Regional Storm Peak Flow Differences from the 1997 Study (GAWSER) ranged from +10% to +37% throughout the watershed, with +11% at the Lake Ontario outlet.

— Appleby Creek

  — 100-Year Storm Peak Flow Differences from the 1997 Study (GAWSER) ranged from -39% to +17% throughout the watershed, with +16% at the Lake Ontario outlet.

  — Regional Storm Peak Flow Differences from the 1997 Study (GAWSER) ranged from +2% to +33% throughout the watershed, with +17% at the Lake Ontario outlet.

— Sheldon Creek

  — Regional Storm Peak Flow Differences from the 2019 Study (HSP-F) ranged from -11% to +37% throughout the watershed, with +27% at the Lake Ontario outlet.

The rationale for differences in peak flows is discussed further in the Hydrology Report, and also in Section 6.3 of the current Hydraulics Report.

As part of the subsequent hydraulic model updates, future condition flows have been applied in the hydraulic modelling, which represents a conservative flow scenario incorporating both future land use conditions and climate change influenced rainfall. These changes in peak flow data will inherently have an effect on the hydraulic performance of the riverine systems, but in general, the initial simulated flood limits described in this section generally follow the existing floodplain limits currently mapped for the study area. Further discussion regarding observed differences and some example mapping is provided in the subsequent sections.

4.2.3 COMPARISON TO FIELD OBSERVATIONS

In addition to the 12-hour Regional Storm event, the initial draft 1D models had been simulated and RAS Mapper had been used to plot inundation limits for the August 4th 2014 storm event, based upon the flows produced as part of the hydrologic modelling. The computed inundation limits have been compared to the “Observed Flood Lines” from the August 4th 2014 event (as per Conservation Halton, 2015), which were provided by CH for use and reference in this study. The observed flood lines consist of areas within the Tuck Creek watershed (between the highway 407 and the CNR, and further downstream near New Street) and the Shoreacres Creek watershed, (upstream of the CNR to the QEW). Examples of the comparison for simulated versus observed flood limits for the August 2014 event are provided in Figures 4.2 to 4.8.

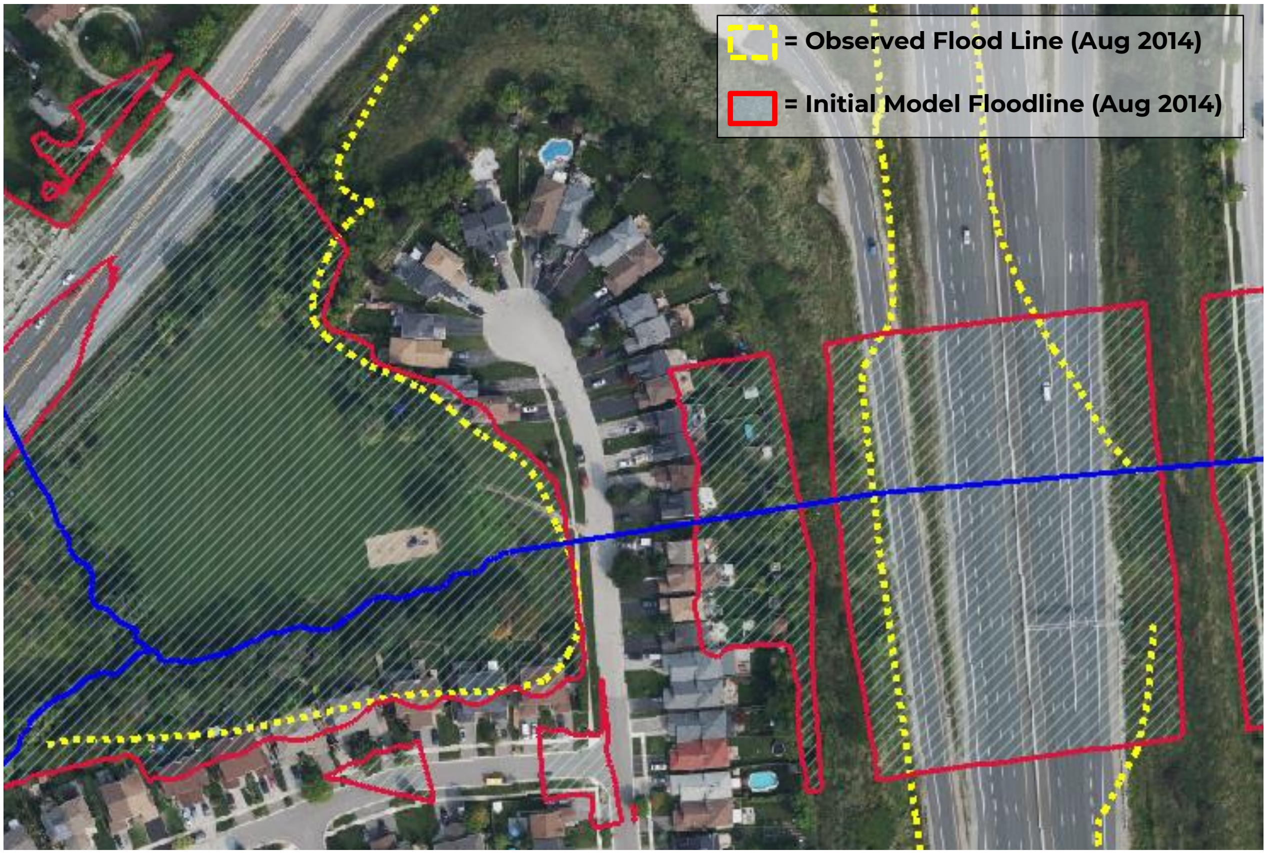


Figure 4.2. Comparison of Initial Existing Floodlines (1D) to Observed Flood Lines for the Aug 2014 Event – Tuck Creek Upstream of the 407 Enclosure (TU_ST022)

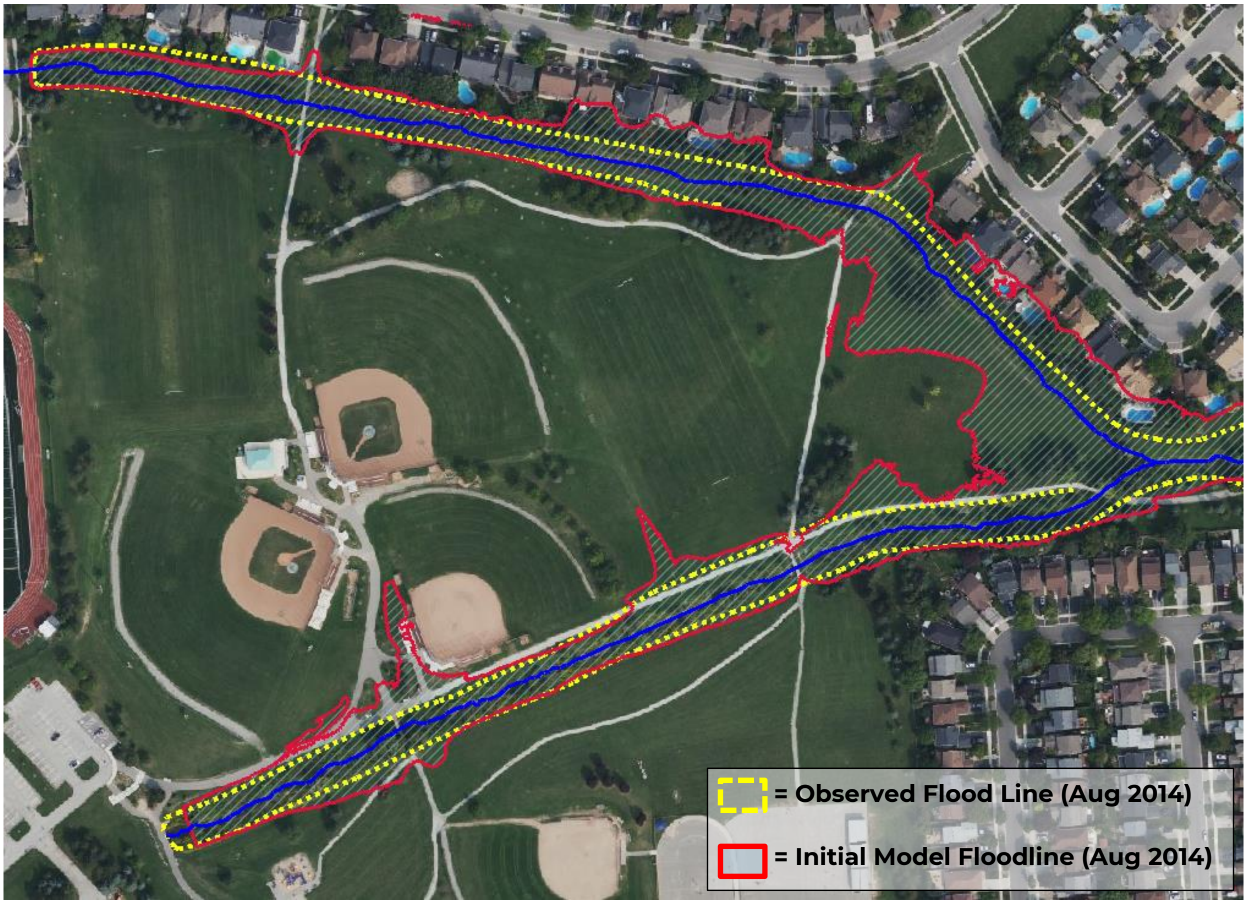


Figure 4.3. Comparison of Initial Existing Floodlines (1D) to Observed Flood Lines for the Aug 2014 Event – Tuck Creek Downstream of the 407 Enclosure (TU_ST021) to Confluence Point (Ireland Park)

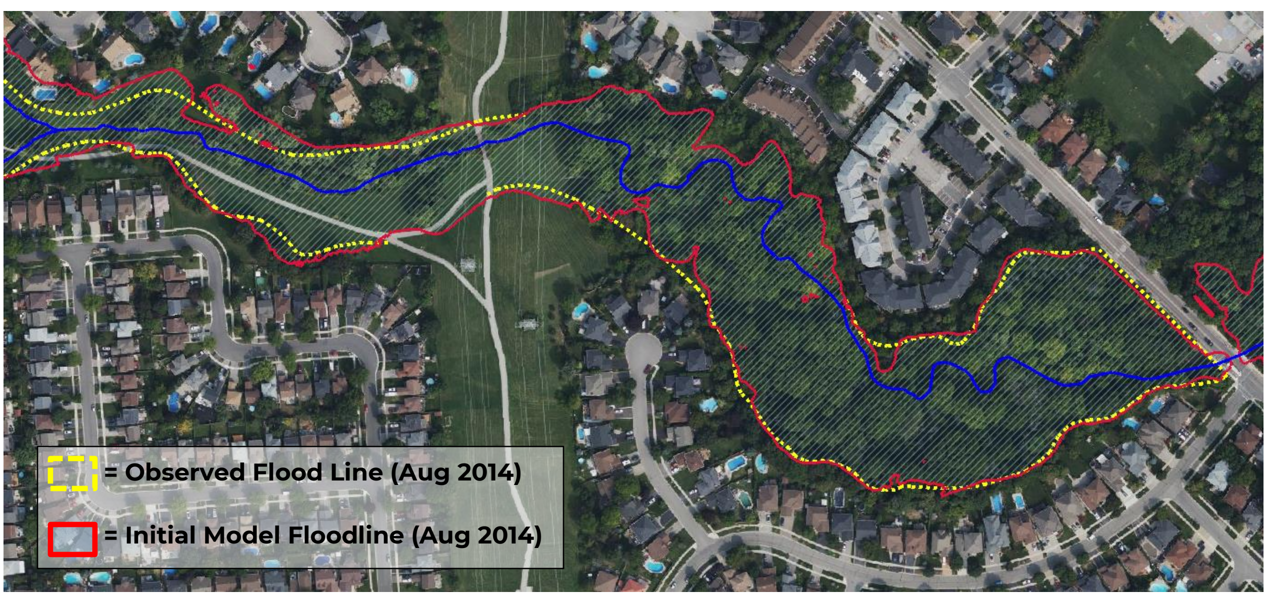


Figure 4.4. Comparison of Initial Existing Floodlines (1D) to Observed Flood Lines for the Aug 2014 Event – Tuck Creek Downstream of Confluence Point (Ireland Park) to Head Road (TU_ST017)

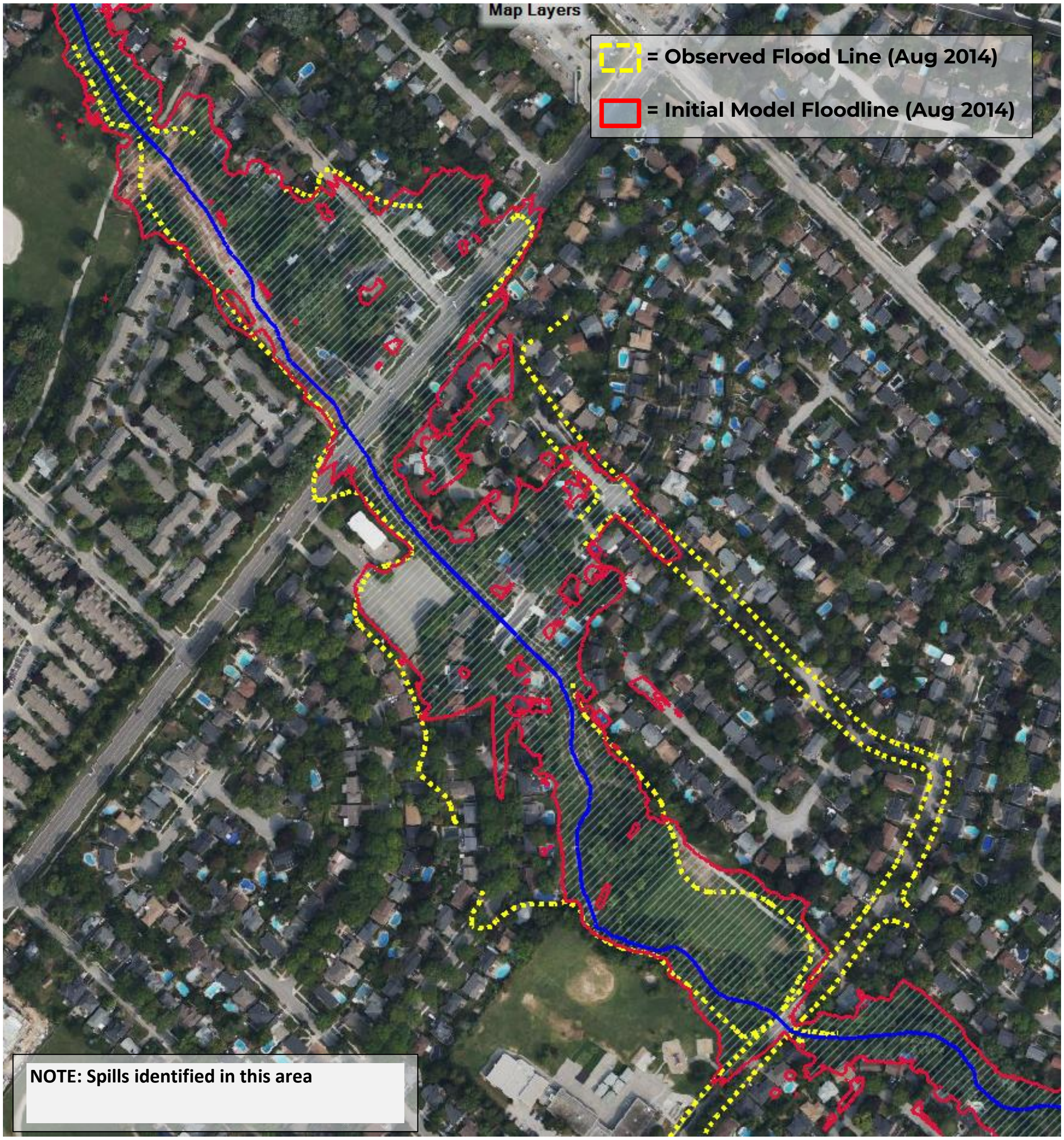


Figure 4.5. Comparison of Initial Existing Floodlines (1D) to Observed Flood Lines for the Aug 2014 Event – Tuck Creek from Sweetgrass Park (TU_ST005) to Spruce Avenue (TU_005)

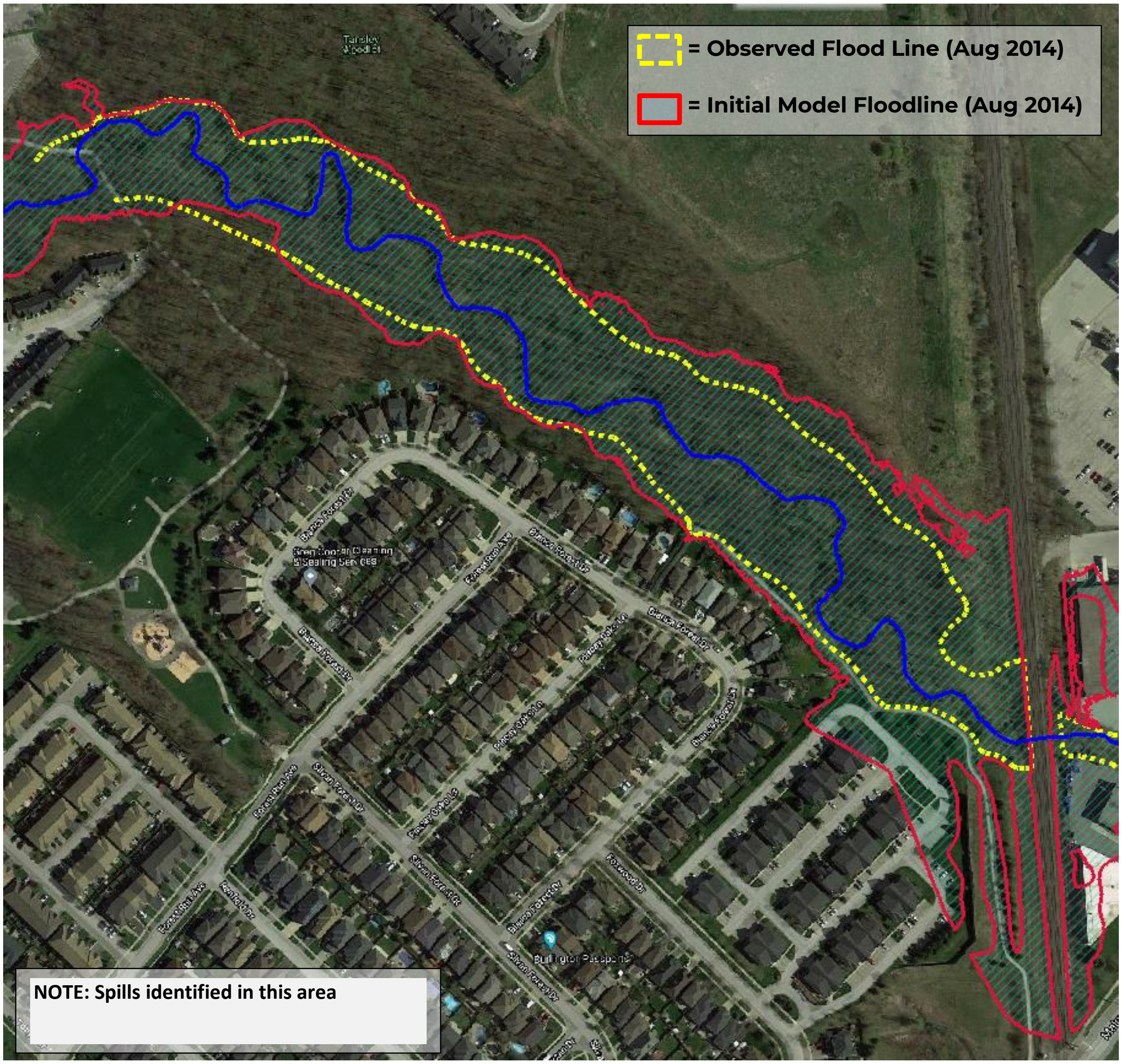


Figure 4.6. Comparison of Initial Existing Floodlines (1D) to Observed Flood Lines for the Aug 2014 Event – Shoreacres Creek from Itabashi Way to CNR (SA_ST048)

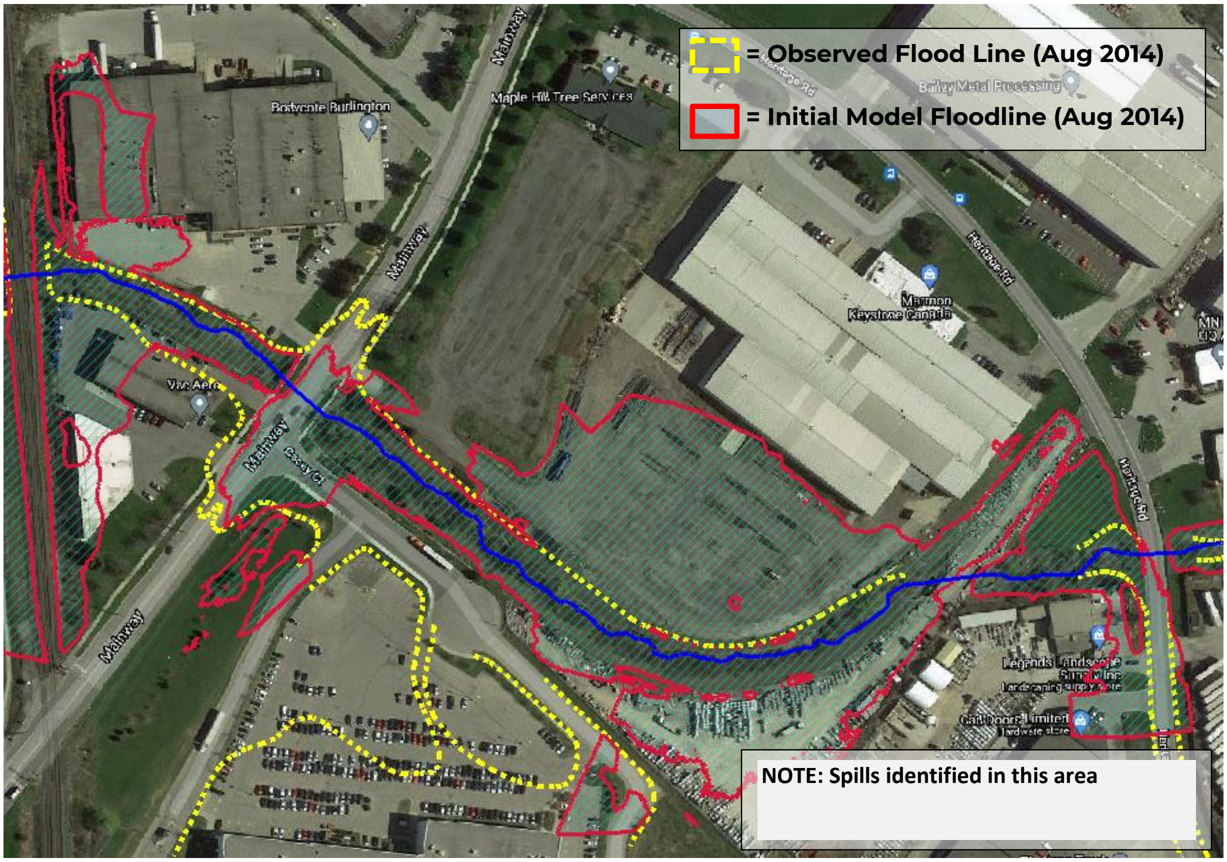


Figure 4.7. Comparison of Initial Existing Floodlines (1D) to Observed Flood Lines for the Aug 2014 Event – Shoreacres Creek from CNR (SA_ST048) to Heritage Road

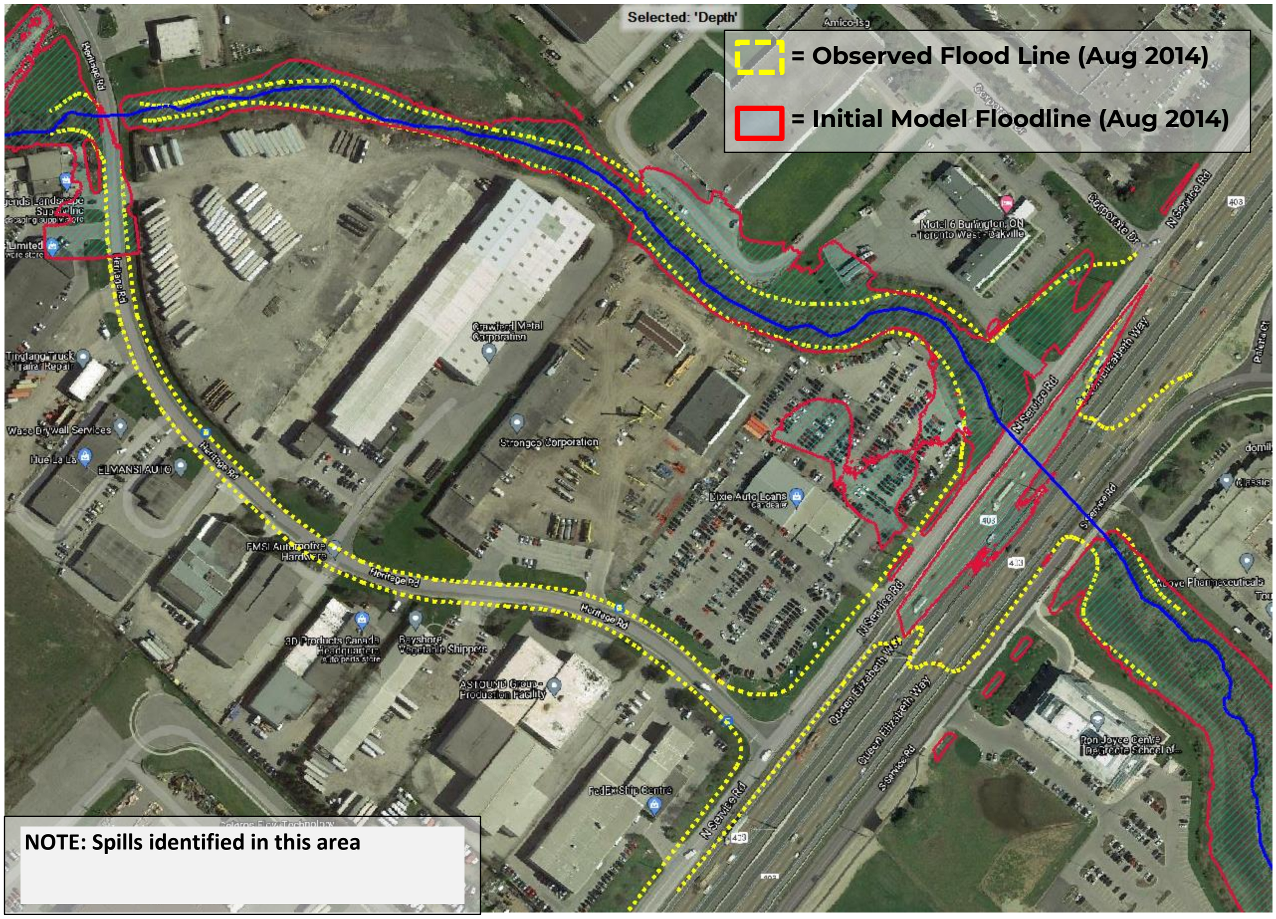


Figure 4.8. Comparison of initial Existing Floodlines (1D) to Observed Flood Lines for the Aug 2014 Event – Shoreacres Creek from Heritage Road (SA_ST045) to QEW (SA_ST044)

As evident from Figures 4.2 to 4.5, the Tuck Creek hydraulic modelling results demonstrate a good match to the observed flood lines from the August 2014 event. In the upstream section, overtopping of Highway 407 is not indicated in the 1D results; however observed inundation in this location may have been the result of pluvial (rather than fluvial) flooding or spills from other areas along the Highway 407. In the downstream section of the watershed surrounding New Street, there are several spill areas that have been previously identified by CH; these have been generally confirmed through the observed flood limits demonstrating structures overtopped and spill flows within residential streets and properties.

With respect to Shoreacres Creek, Figures 4.6 to 4.8 demonstrate some differences as compared to what was observed during the August 2014 event, including spills upstream of the CNR and within the adjacent industrial properties. The spill identified at Heritage Road (ref. Figure 4.8) is confirmed in the modelling. This area has been investigated as part of subsequent 2D modelling exercises to represent the spills along these roadways, rejoining the reach further downstream (ref. 2D Area 3).

Numerous photographs and video footage of historic flooding events within the study area was submitted to CH by members of the public as part of PIC #1 (October 14, 2021) for use and reference in this study. CH has reviewed and sorted the received information to provide the best use for the data to confirm results of this study. The photos/videos received are largely located within the vicinity of the Millcroft Golf Course, as several residential neighborhoods back onto the golf course lands. This area is largely within the Appleby Creek watershed, however the easternmost portion of this area lies within the Sheldon Creek watershed.

The photographs and videos provided by the public have been reviewed in order to compare against the draft floodlines and assist in validating the current model simulation results. The focus has been placed on photographs demonstrating more formative flood events, to allow for a reasonable comparison to the results of the Appleby Creek Regional event initial simulation. Examples of this comparison are presented in Figures 4.9 to Figure 4.11.

The comparison between the observed flood limits to the current simulation results demonstrate a reasonably accurate representation of the systems flood response throughout the eastern and western branches of Appleby Creek, flowing through the Millcroft Golf Course and lands downstream (notwithstanding the uncertainty with respect to the magnitude of the storm event and timing of the photographs).

Figure 4.9. Millcroft Golf Course (Hole 2) Demonstrating Flood Pathways & Inundation Limits – Confirmed with Draft Regional Floodplain Mapping (Appleby Creek West – AP_W_100)

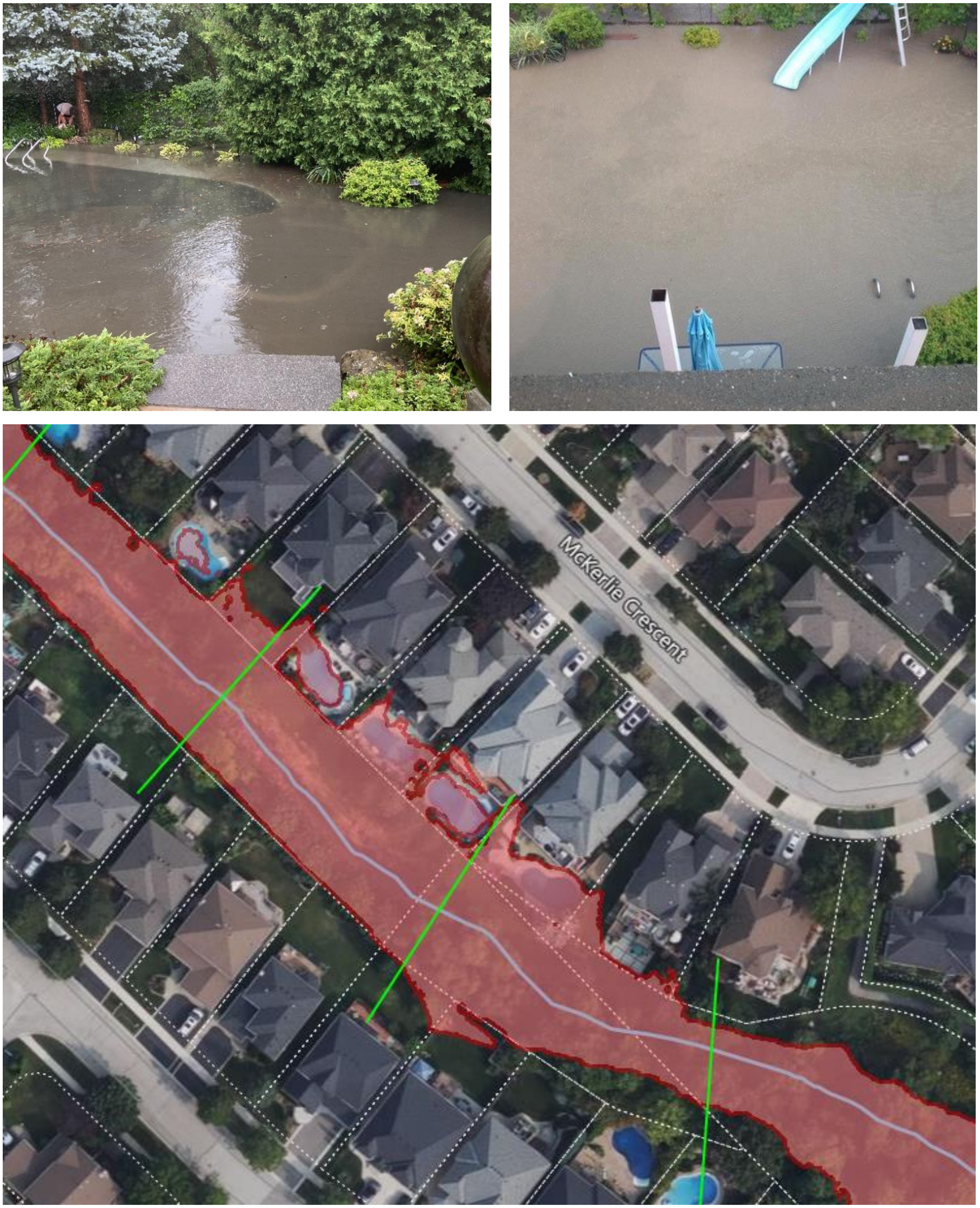


Figure 4.10. McKerlie Crescent Resident Photos Demonstrating Flooding into Backyards/Pools – Confirmed with Draft Regional Floodplain mapping (Appleby Creek West – AP_W_100)

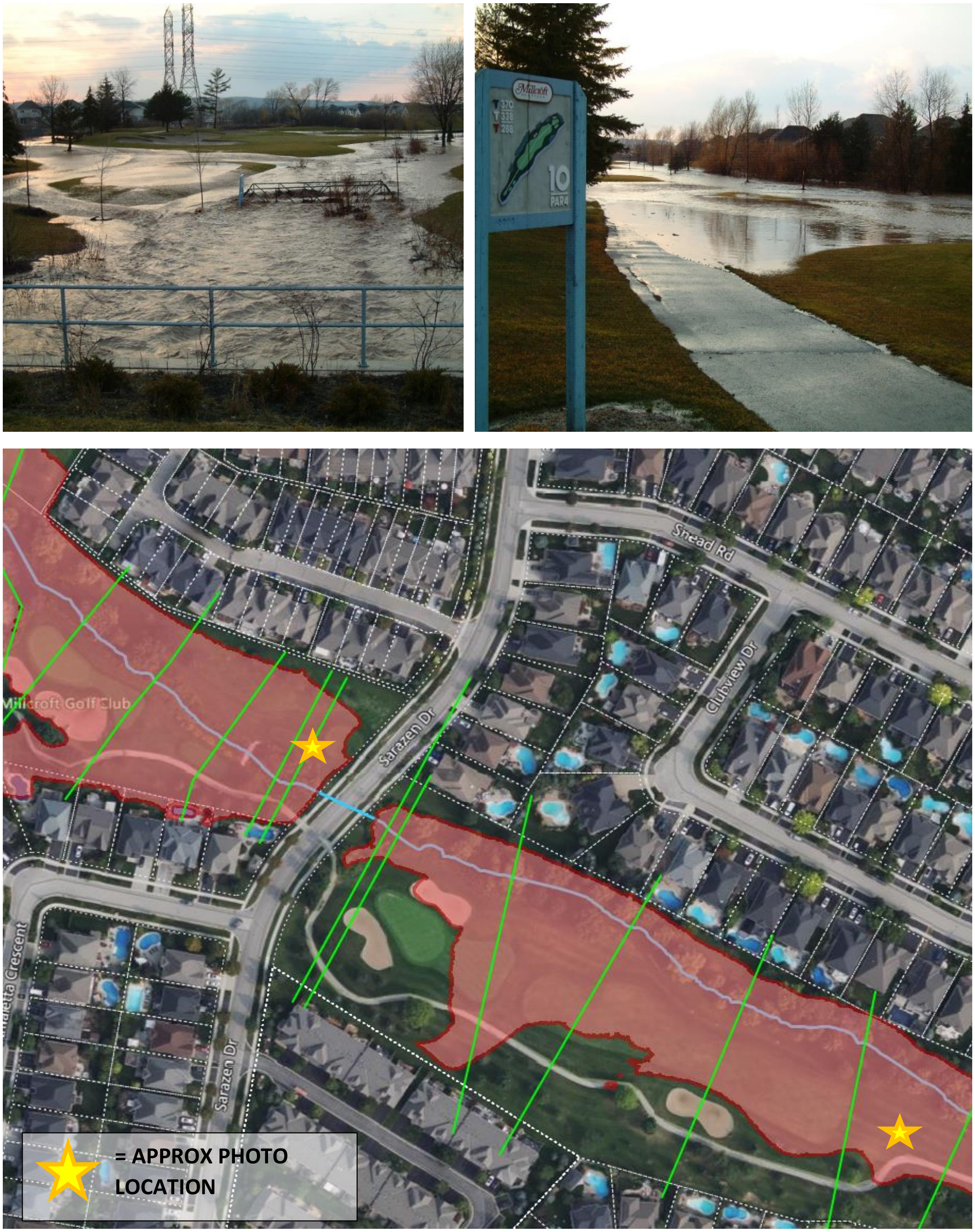


Figure 4.11. Millcroft Golf Course (Hole 10 & 11) Demonstrating Flood Pathways & Inundation Limits – Confirmed with Draft Regional Floodplain Mapping (Appleby Creek East – AP_E_103)

4.3 FUTURE CONDITIONS 1D MODEL

4.3.1 QUALITY ASSURANCE AND QUALITY CONTROL

Following the preceding model validation for existing conditions, the 1D hydraulic model has been simulated with the future conditions flows, as documented in the Hydrology Report. Where the Regional Storm is the Regulatory Event (the majority of the watercourses), the Future Condition Flows do not result in a notable difference (generally less than 1%) as the model is not particularly sensitive to the changes in land use. Where the 100-year is the Regulatory Event, more notable changes have been indicated due to the combination of increased rainfall intensity and impervious coverage (which is more sensitive, due to the rainfall intensities associated with the 100-year storm event).

Updates and refinements have been made to the model geometry throughout the course of the study based upon review and comment by CH, supplemental field investigation for structures, as well as cross-section extensions to contain the floodplain. In addition to base model updates, the initial results have been reviewed as part of QA/QC processes to identify areas where consecutive cross-sections have defaulted to critical depth, as well as cross-sections where the computed water surface elevations change by greater than +/- 0.50 m between cross-sections.

These areas have been reviewed within the modelling and several additional cross-sections have been added, which can improve model stability as part of subcritical flow analyses. It should also be noted that modelling results can be quite sensitive to the bridge versus culvert editor modelling methodology at structures; therefore, in certain areas where oscillations in results were observed near structures, the coding methodology has been adjusted to achieve greater stability.

Several of the oscillations have been resolved through these edits, however certain reaches continue to default to critical depth; a summary of the occurrences of critical depth within each of the four (4) models under the Regional event is presented in Table 4.1.

As noted in Table 4.1, there are several reaches still defaulting to critical depth, however all reaches have a longitudinal slope greater than 0.5%, and majority of the reaches are greater than 1%. Slopes within this range are unlikely to achieve a valid subcritical result, and therefore indicates the potential for supercritical flow conditions.

It should be noted that for the purposes of flood hazard mapping, cross-sections which default to critical depth have been reviewed further to identify if the downstream cross-section has a valid subcritical answer which results in a higher water surface elevtion; if this is the case, the critical depth cross-section has been reviewed for manual adjustment in the mapping process. Further description for additional manual adjustments to the floodplain mapping is summarized in Section 6.

Table 4.1. Summary of Critical Depth Occurrences (Regional)

MODEL SUMMARY	RIVER	TOTAL # XS	TOTAL # CRITICAL DEPTH	% DEFAULTING TO CD	AVG SLOPE (%)
Tuck	TU_E_11	68	45	66%	2.5%
	TU_E_12	25	11	44%	2.3%
	TU_E_13	21	7	33%	1.1%
	TU_W_10	17	7	41%	1.0%
	TU_M_10	196	92	47%	1.0%
Shoreacres	SA_W_11	59	34	58%	2.1%
	SA_W_12	56	32	57%	1.8%
	SA_W_13	69	17	25%	1.1%
	SA_E_10	106	32	30%	1.2%
	SA_M_10	234	81	35%	1.0%
Appleby	AP_E_11	31	16	52%	1.3%
	AP_E_12	8	4	50%	1.2%
	AP_E_13	188	43	23%	0.9%
	AP_W_10	110	27	25%	1.1%
	AP_M_10	127	47	37%	1.0%
Sheldon	SD_W_21	145	14	10%	0.7%
	SD_W_11	5	1	20%	2.5%
	SD_W_12	12	2	17%	1.3%
	SD_W_13	134	26	19%	0.7%
	SD_W_20	195	52	27%	0.8%
	SD_E_10	252	62	25%	0.8%
	SD_M_10	35	8	23%	0.7%

4.3.2 PRELIMINARY SPILL AREA IDENTIFICATION

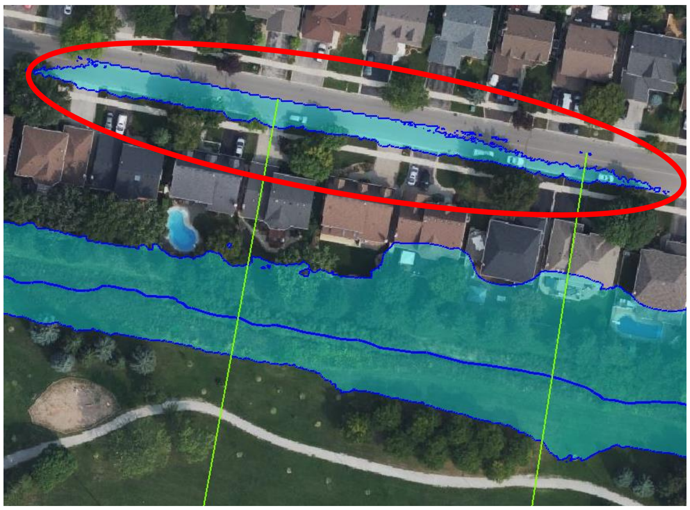
As part of the initial hydraulic model development which was completed using existing conditions flows, a long list of potential spill areas was established to be evaluated as part of subsequent model review and refinement. As part of the future conditions 1D model development, these areas have been reviewed further to identify which represent "true spills" and may warrant additional 2D modelling to confirm spill pathways and flows contributing to neighboring watersheds systems.

The MNRF's Technical Guide River & Stream Systems: Flooding Hazard Limit (2002) (ref. Section 4.13 of the guidelines) defines a spill as occurring when flood levels overtop the banks of a watercourse and spill overland away from the watercourse channel. Frequently, this spill will move into another watershed or join the originating watercourse at a distance downstream. Further, the guidelines describe that:

> "The effect of spills moving into another watershed should be assessed to determine the potential flood risks. Alternative measures should be investigated to prevent the spill moving into the adjacent watershed. If the amount of spill is relatively small, less than 10% of the peak flow, the floodplain mapping for the watercourse should be based on the original flow, without any deduction for the spill. For larger spills, allowance for the reduced flow should only be made where the review of alternatives proves that the spill cannot be prevented, either because there are no feasible alternatives or the costs, when compared to the potential benefits, are too high. Where the spill re-joins the watercourse further downstream, the route of the spill should be examined to determine the potential harmful effects of overland flow. No reduction should be made for the spill in the downstream floodplain computations."

For the current study, an iterative assessment of potential spill areas has been undertaken by WSP in conjunction with CH.

As part of the future conditions 1D hydraulic model development and review of draft floodlines, another primary task has been to identify potential spill areas and propose a priority list of spills for consideration to be carried forward as part of subsequent 2D modelling efforts. The review of spill zones has been completed by generating a HEC-RAS geometry file with levees applied at select cross-sections to represent only the active flow area with a continuous floodline (i.e. not from backwater or depressions in the DEM). An example of these scenarios is provided in Figure 4.12 below.



Note: Figure presents initial draft findings. Refer to Appendix H for final mapping.

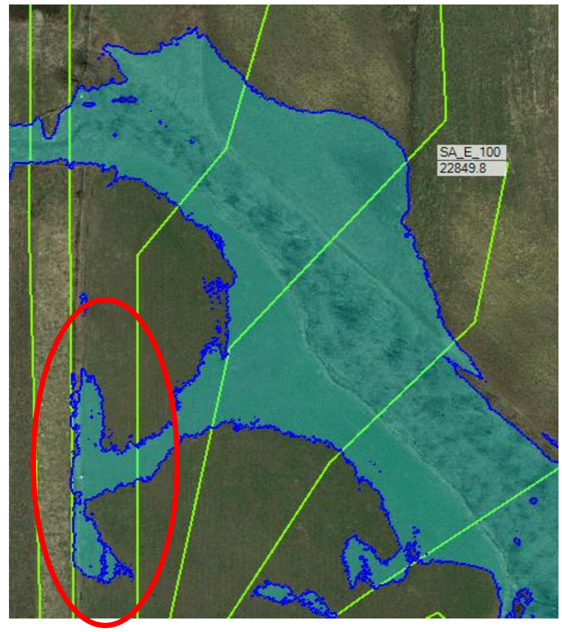


Figure 4.12: (a) Example of Dead-Storage based upon DEM – Not connected to the floodplain. (b) Example of Backwater – Not a part of active flow area for select cross-sections

This approach has allowed for a review of continuous flow paths, and assists with the identification of verified levees/constrictions of flow, as well as potential backwater and spill pathways to be considered as part of subsequent modelling. It should be noted that the floodlines generated in RAS mapper are limited to the extent of the modelling, as such cross-section extensions may have been required in order to close certain floodlines in support of final mapping. This has been completed as part of final QA/QC with the final approved flows, as well as the floodplain mapping process.

At the onset of the study, CH provided a shapefile demonstrating currently identified spill zones within each of the four (4) subject watersheds. These areas have been reviewed against the current draft floodlines to identify whether the spill is still occurring in the draft models (i.e. confirmed) or whether the previously identified spill is no longer occurring, as well as identify any new / additional spills not identified as part of previous modelling/mapping. It should be noted that as part of subsequent model review and refinement with future flows, there are a number of areas in the model which have been resolved through the extension of cross-sections, or through the addition of cross-sections to mitigate oscillations in the modelling.

The initial spill areas that have been identified throughout the four (4) watershed models which are unable to be contained (i.e., true spills) as part of the initial draft modelling (does not represent the final modelling / results), have been summarized in Table 4.2. For simplicity as part of initial reporting, initial draft spill areas have been grouped based upon common location (i.e., multiple spills identified surrounding a single structure / crossing have been grouped together).

Table 4.2. Summary of Spill / Uncontained Areas within Study Area (Initial 1D Results)

WATERSHED	IDENTIFIED IN PREVIOUS CH MAPPING	INITIAL DRAFT MODELS / MAPPING [1]			
		PREVIOUS SPILLS CONFIRMED	DIFFERENCE IN SPILLS	NEW SPILLS IDENTIFIED	SPILLS NOT SHOWN IN MODELLING
Tuck	13	9	1	1	3
Shoreacres	8	4	2	3	2
Appleby	9	1	1	6	7
Sheldon	6	5	0	19	1
Total	**36**	**19**	**4**	**29**	**13**

Note: [1] The results presented within this Table are based upon preliminary draft 1D models, and therefore do not represent the formal / final modelling which has been refined through subsequent study tasks. These results and those shown on Drawing 5 demonstrate the initial results which helped to support the subsequent 2D modelling effort required to determine true spill areas.

Overall, a total of 52 initial spills or uncontained areas within the initial draft 1D modelling had been identified across the four (4) watersheds, which include 19 previously identified spills confirmed, 4 differences in spills, and 29 new spills identified. A further 13 spills (previously identified) were found to be no longer occurring in the initial draft modelling.

Tuck Creek continues to have the greatest number of previously identified spills confirmed through the draft modelling; WSP also identified an additional potential spill area within the Tuck Creek watershed based upon the preliminary 1D modelling results. A total of 3 potential new spills have also been identified for the Shoreacres Creek watershed, and 6 potential new spills within the Appleby Creek watershed, as well as a much larger total of 19 new potential spills have been identified within the Sheldon Creek watershed, based upon the initial draft modelling.

Details regarding the initial locations, type of spill, and initial spill priority (i.e., “Low, Medium or High” based upon a review of depth of spill and potential impacted lands) were previously identified at earlier stages of the project, to help support the screening, identification prioritization of potential areas of concern to be considered for subsequent detailed 2D modelling (ref. Section 5.0). A visual representation of the previously identified spill locations across the study area is presented on the attached Drawing 5. It should be noted that the spill areas identified on Drawing 5 represent the initial draft 1D hydraulic modelling results established at interim stages of the project, and were subject to change and further refinement as part of subsequent modelling, and particularly the iterative analysis with 2D modelling and hydrologic analysis.

Given the number of initial spills previously identified, it has been considered warranted to simulate multiple spill zones as part of one comprehensive 2D model, focusing on areas with several spills in close proximity. Such an approach is currently being proposed for ongoing work in ongoing watersheds within CH’s jurisdiction (i.e. the Roseland and Hager-Rambo Creek systems as part of the Burlington GO MTSA and Downtown area study). This approach is more proactive in identifying the spill flows contributing to downstream and/or adjacent reaches, as these areas are interconnected systems.

Based on a preliminary review of the initial potential spill areas (ref. Table 4.2 & Drawing 5) and through consultation with CH, 2D modelling areas have been developed and are presented in Table 4.3. Visual representations of the proposed modelling extents are presented in Drawing 6 (ref. attached).

Table 4.3. Summary of 2D Modelling Areas

WATERSHED	AREA ID#	DESCRIPTION	NUMBER OF STRUCTURES	AREA (ha)
Tuck, Shoreacres, Appleby and Sheldon	1	407ETR Sheldon to Tuck Inter Basin Model	29	550
Tuck	2	Mainway to Spruce Avenue	12	543
Shoreacres	3	CNR/Mainway to Centennial Bikeway	15	464
Appleby and Sheldon	4	Dundas Street to Mainway	21	284
Appleby	5	QEW to Pineland Park near Spruce Avenue	9	331
Sheldon	6/7	Sheldon Park/Riverside Drive and Rebecca Street to Lakeshore Road West	12	217
	8	QEW to Michigan Drive	10	268

Further discussion regarding the 2D model development, preliminary results and integration with the subsequent iterative analysis with both 1D hydraulics and hydrology is provided in Section 5.0.

5 2-DIMENSIONAL (2D) MODELLING RESULTS

5.1 SENSITIVITY ANALYSIS

With respect to choosing the 2D modeling set of equations and other parameters within HEC-RAS 6.3.1, a sensitivity analysis has been conducted to inform the most stable run and produce accurate results. Different mesh sizes and the two (2) different sets of equations have been tested using the Area 1 2D model.

The HEC-RAS 2D modeling capability uses a Finite-Volume solution scheme. This algorithm was developed to allow for the use of a structured or unstructured computational mesh. This means that the computational mesh can be a mixture of 3-sided, 4-sided, 5-sided, etc., computational cells (HEC-RAS has a maximum of 8 sides in a computational cell). However, users typically select a nominal grid resolution to use (e.g., 10 m x 10 m cells), and the automated tools within HEC-RAS will build the computational mesh. After the initial mesh is built, the user can refine the grid with break lines, refinement regions, and the mesh editing tools.

The Area 1 test model (version 1) was developed using a 10 m x 10 m cell resolution which contained 74,417 cells and another test model for Area 1 (version 2) was developed using a 5 m x 5 m cell resolution which contained 231,698 cells. The resulting grid outputs (depth grids) for the version 1 model did not indicate a smooth connection between cells when compared to the version 2 model. Although the version 1 model ran approximately 40% faster than the version 2 model, there were several cell errors noted within the output computation log file. All hydraulically significant embankments, such as roads and channel centerlines (streamlines), when enforced in the mesh using breaklines in both the versions of the Area 1 model, however, the version 2 model allowed for a more detailed model than the version 1 model due to the finer cell resolution. The Manning's n values were correctly assigned to the 5 m cell faces in contrast to the 10 m cell faces. This is because, when the cell faces are processed, the Manning's n value selected will be based on finding the cell face center, then corresponding Manning's n value from the land cover layer, therefore, a finer resolution mesh will capture detailed Manning's n. Thus, recognizing the benefits of a finer mesh resolution, all 2D models were therefore set up using a finer mesh resolution 5 m x 5 m cell mesh resolution).

Further, two test versions of the Area 1 2D model that had same geometry (cell resolution, landuse, topography, breaklines, number of structures etc.), simulation run time settings (1 second) and simulation time window for a true comparison were compared and, the run times, outputs, cell errors and model stability of the two sets of 2D modeling equations were evaluated. The version 1 model was executed using the shallow water equation "2D Unsteady SWE-ELM Equation Set (faster)" equation and version 2 model was executed using the "2D Unsteady Diffusion Wave Equation Set (fastest)" equation. It was determined that the version 1 model reported a significant number of cell errors ranging from 0.001 m to 0.7 m in contrast to the version 2 model which reported only six cell errors equal to 0.003 m. Additionally, the version 1 model took 11 hours and 51 minutes to run when compared to the version 2 model which took approximately 8 hours to run. Furthermore, when some of the culvert flow hydrographs selected at random were compared for the two versions of the test models, both versions generated similar results. The 2D diffusion wave equation set was therefore used for all the 2D models due to faster computation times and stable runs to produce results.

5.2 2D MODELLING RESULTS

5.2.1 AREA 1 – 407ETR SHELDON TO TUCK INTER BASIN MODEL

The Area 1- 407ETR model (550 ha in size) has been developed as a multiple watershed system model that includes all four (4) watersheds in the study. The model extents of this 2D HEC-RAS model covers all reaches upstream of the 407ETR.

The Tuck Creek Watershed reaches just downstream of Berkshire Lane and Headon Forest Drive have been included in the 2D model. Shoreacres reaches just downstream of Headon Forest Drive and Constable Henshaw Boulevard have been included. Appleyby reaches and Sheldon reaches just downstream of Palladium Way have been included in the 2D model.

A total of 29 structures (including long culverts across the 407ETR and Headon Forest Drive) have been included. General model development and modeling methods have been already described in the previous sections. This model has been executed for the Regional Storm (12H AMC III Future Conditions flows) and 100-year storm (12H AMC II SCS distribution, future conditions flow).

Model adjustments have been implemented to reflect inter-watershed spill flows, or to correct for the impacts of flow and volume attenuation behind several hydraulic structures within this model. Some of the significant inter-basin spills include the Appleby to Sheldon watershed spill and one spill from the east to the west branch within the Sheldon Creek Watershed near the 407ETR (SW_W_201 and AP_E_102). It should be noted that the eastern branches within the Sheldon Creek Watershed do not spill into eachother (SD_W_102 - SD_W_101 and SD_W_101 – SD_W_201). The ponding on the 407ETR lanes have also been accounted for during balancing of flows for the Tuck Creek Watershed reaches, namely, TU_E_13 and TU_W_10 just downstream of Berkshire Lane and Headon Forest Drive respectively.

Based on the model results, (ref. Figure 5.1), water is stored within the 407ETR east and west bound lanes and near Tuck Creek just downstream of Driftwood Drive. The median barrier is indicated as overtopped in this area, however water does not over top the high ground on the south side of the highway. Also, flows get stored within the ditches along the 407ETR.

The floodplain is mostly connected in all reaches along the north side ditches of the highway except at the two reaches of Sheldon Creek just west of Appleby Line. It should also be noted that water is stored at a sag point on the 407ETR near the Guelph Line underpass and does not spill towards adjacent watersheds (i.e. Roseland Creek). Further, at the crossing of Guelph Line and Dundas Street on Tuck Creek (reach TU_E_12), the structure at Guelph Line is overtopped and water travels on Guelph Line and then eastward on Dundas Street before finally entering the reach just downstream of Dundas Street.

Figure 5.1. Area 1 2D Floodplain Inundation Limits (Regional Storm – Future Conditions)

5.2.2 AREA 2 – TUCK CREEK (MAINWAY TO SPRUCE AVENUE)

The Area 2- Tuck Creek model (543 ha) has been developed as a single watershed system model. The model extents of this 2D HEC-RAS model include a single reach just downstream of Palmer Drive to Spruce Avenue.

A total of 12 hydraulic structures, including a long culvert across the QEW have been modeled. General model development and modeling methods have been already described in the previous sections. Currently, this model has been executed for the Regional Storm (12H AMC III Future Conditions flows) and 100-year storm (12H AMC II SCS distribution, future conditions flow)

Based on the model results, (ref. Figure 5.2), flows from Area 3 spills into Area 2 at the QEW and the North Service Road at the intersection with Walkers Line.

Balancing of flows was required at two locations, the long culvert on QEW (ST011-012) and the triple culverts at CNR (ST009). Flows at ST011-012 were balanced with the hydroburn version as per methodology suggested by CH and supported by WSP and the TAC to better align with the MNRF guidance. However, the hydroburn version at ST009 estimated flows lower than the hydrologic model and therefore had to be adjusted. Adjustment of the hydroburn hydrograph is carried out through the equation Qhb * (QmaxVO/QmaxHB). The difference between the adjusted hydroburn hydrogaph and the base model was then added to balance the flows downstream of ST009.

The Tuck Creek culvert at Mainway appears to be a critical structure which is overtopped, and flows spill over Mainway inundating the industrial area near north CNR and then travel west along the north CNR ditch. The open channel through this section does not appear to have sufficient capacity to convey the Regional Storm flows. Some of this flow appears to over top the North CNR east of Pioneer Street and then combines with the flows that travel west along the QEW ditch and finally westwards.

It should be noted that the City of Burlington is responsible for two (2) storm sewer\culvert crossings of the CNR in this area, a 1350 mm diameter crossing at Pioneer Road and a 1050 mm diameter crossing at Blair Road. WSP (then Wood) previously undertook a structural rehabilitation review of these crossings for the city (ref. “Burlington Railway Crossings – Inspection Summary Report” Wood, March 2020). Both of these structures have been included in the 2D model.

Water is also indicated as being stored on the QEW and travelling south through the openings in the median open guard rail system. Moving downstream along the reach, near Harvester Road a similar scenario is observed where water spills over Harvester Road, splits between the open channel and travels south on Walkers Line and Harrington Crescent. The south CNR line doesn’t appear to be overtopped. Further, Fairview Street, Rockwood Drive, New Street and Spruce Avenue, are all overtopped and flows spill onto the neighboring streets and move southward parallelly.

Flows were observed exiting the system flowing westerly along the north side of the CNR, QEW, and south of the CNR adjacent to Fairvew Street. These spill flows would therefore be expected to continue to spill westerly into the adjacent watershed system (Roseland Creek). Based on hydraulic modelling of this area recently completed (“Major Transit Station Area Phase 2 Flood Hazard Assessment – Burlington GO and Downtown”, WSP, March 2023), it is expected that spill along the QEW may continue westerly towards the East Rambo Pond and the Hager-Rambo Creek watershed. However, further hydraulic modelling would be required to confirm this, which is beyond the scope of the current study.

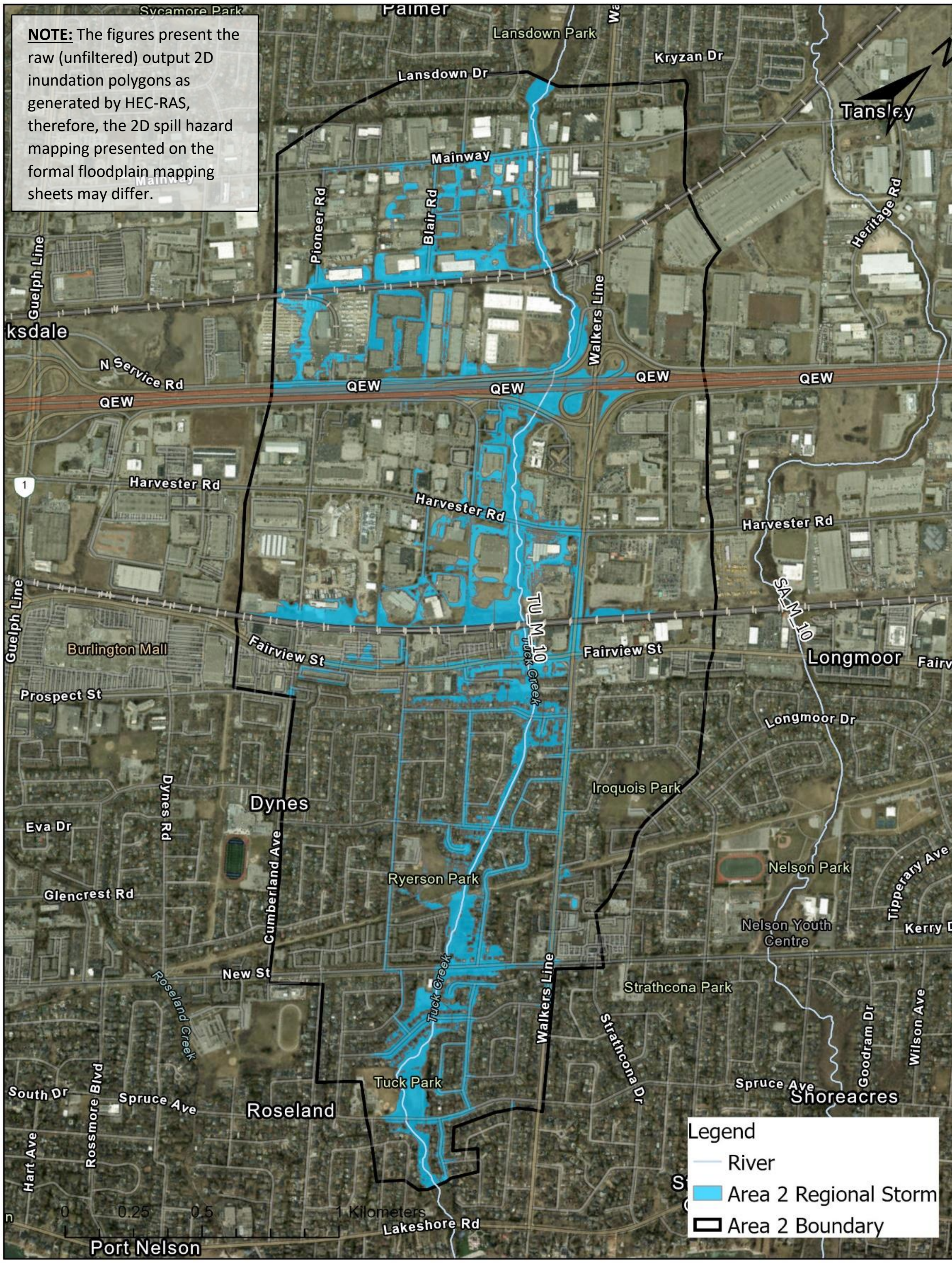


Figure 5.2. Area 2 2D Floodplain Inundation Limits (Regional Storm – Future Conditions – Including Inflows from Area 3)

5.2.3 AREA 3 – SHOREACRES CREEK (CNR|MAINWAY TO CENTENNIAL BIKEWAY)

The Area 3- Shoreacres Creek model (464 ha) has been developed as a single watershed system model. The model extents of this 2D HEC-RAS model cover a single reach just upstream of the CNR to New Street.

A total of 15 hydraulic structures have been modeled. This includes one bridge and nine inline culverts (i.e., associated with a modelled watercourse), the remaining six being floodplain culverts (i.e., drainage conveyance culverts, not associated with a modelled watercourse), modelled for hydraulic connectivity. General model development and modeling methods have been described in the previous sections. This model has been executed for the Regional Storm (12H AMC III Future Conditions flows) and 100-year storm (12H AMC II SCS distribution, future conditions flow).

Based on the model results (ref. Figure 5.3), flows from Area 5 (refer to Section 5.2.5) spill into Area 3 at the QEW.

Balancing of flows was required at three locations, the CNR (Halton Subdivision – to the north) culvert (ST048), the long culvert at the QEW (ST043-44) and the triple culverts at CNR (Oakville Subdivision to the south) (ST041). Flows at ST048 and ST043-44 were balanced with the hydroburn version as per methodology suggested by CH and supported by WSP and the TAC to better align with the MNRF guidance. At ST041, however, the hydroburn version estimated flows lower than the hydrologic model and therefore had to be adjusted. Adjustment of the hydroburn hydrograph is carried out through the equation as described in Section 5.2.2. The difference between the adjusted hydroburn hydrogaph and the base model was then added to balance the flows downstream of ST041.

Spills were observed at Mainway, QEW, Harvester Road and Fairview Street. At Mainway, the flows upstream of the structure travels along the CNR Halton Subdivision tracks to Walkers Line where it finally exists westward into Area 2.

At Heritage Road, a split flow scenario occurs and flow spills south onto Heritage Road and is stored on QEW, where flows travel south through the openings in the median open guard rail system into the industrial area along the South Service Road. The spill on South Service Road then connects with spills along Harvester Road. These spills travel further south, to the south CNR. However, these spills exit the system at the western and eastern watershed boundaries.

The structure at Fairview Street appears to be constrictive, forcing water to overtop and then spill onto Ingram Common. Spill flow splits to travel south along the residential neighbourhood streets such as Longmoor Drive, Greg Drive, and Belvenia Road before finally exiting the system at the southern limits of the 2D mesh.

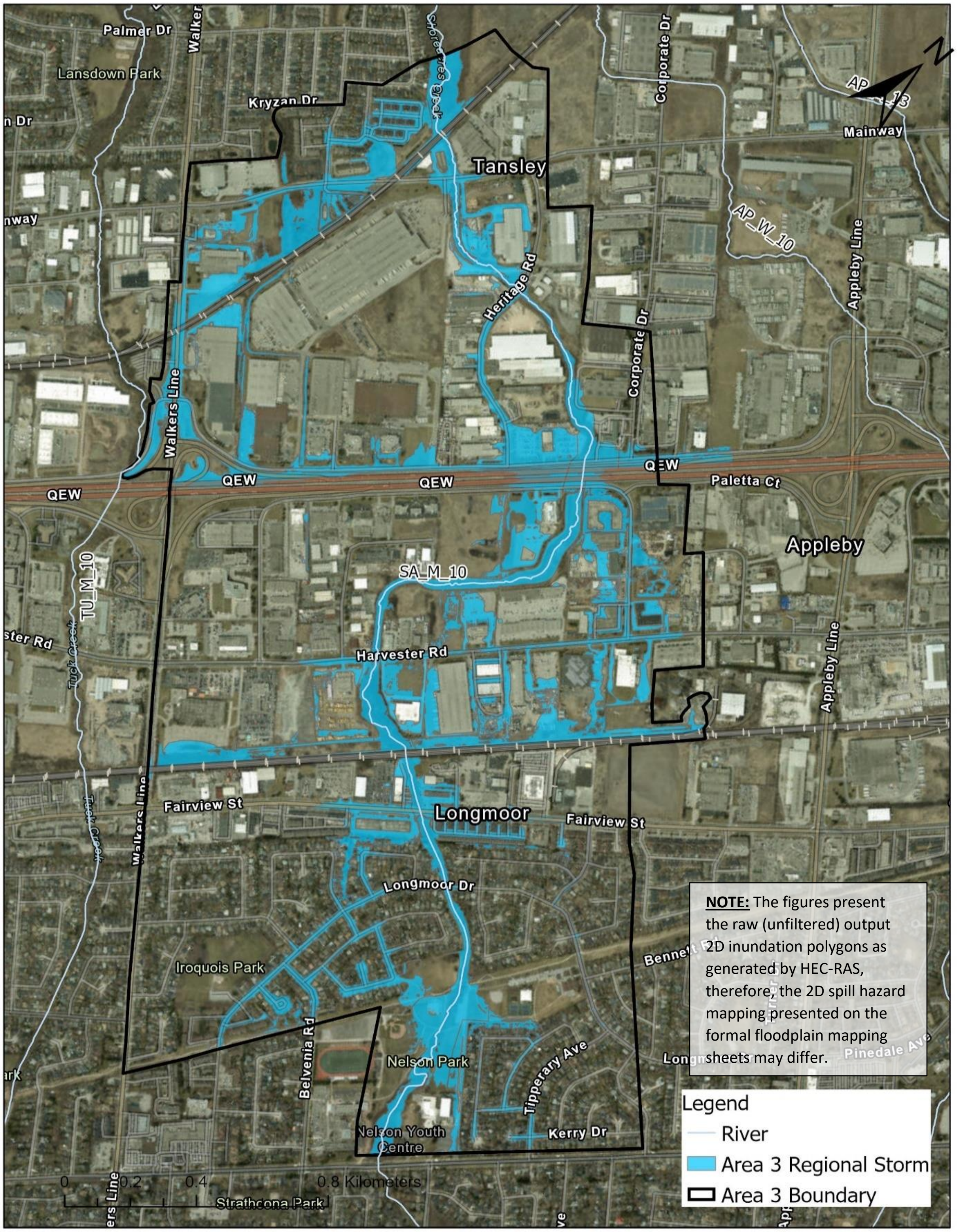


Figure 5.3. Area 3 2D Floodplain Inundation Limits (Regional Storm – Future Conditions)

5.2.4 AREA 4 – SHELDON & APPLEBY CREEKS (DUNDAS STREET TO MAINWAY)

The Area 4 Sheldon and Appleby Creek model (284 ha) has been developed as a dual watershed system model with the Sheldon branch having two sub-watersheds that confluence. The Sheldon Creek branches commence just upstream of Dundas Street and continue southeast along Appleby Line where they confluence at Upper Middle Road and flow downstream to Mainway. The Appleby Creek branch starts at Upper Middle Road and flows southeast to just beyond Mainway.

A total of 21 hydraulic structures have been modeled. This includes one bridge and nineteen inline culverts (i.e., associated with a modelled watercourse), the remaining one being a floodplain culvert (i.e., drainage conveyance culvert, not associated with a modelled watercourse), modelled for hydraulic connectivity. General model development and modeling methods have been described in the previous sections. This model has been executed for the Regional Storm (12H AMC III Future Conditions flows) and 100-year storm (12H AMC II SCS distribution, future conditions flow).

Based on the model results (ref. Figure 5.4), no lateral spill flows into the system were observed.

Balancing of flows was required at six locations with this model. Flow balancing was carried out on the Sheldon Creek branch SD_W_102 at the culvert on the CNR (ST171), the Sheldon Creek branch SD_W_201, at the culvert on the Appleby Line (ST144) and the remaining four balancing of flows was carried out on the Appleby Creek branch AP_E_103, at Upper Middle Road (ST89-90), CNR Rail (ST088), Ironside Drive (ST087) and Appleby Line (ST085). Flows were adjustedas per methodology suggested by CH and supported by WSP and the TAC to better align with the MNRF guidance.

The area includes a number of Stormwater Management (SWM) ponds; refer to Table 3.13 for further details. Two (2) of these ponds in particular are online within the limits of the mesh; Pond 804 upstream of Dundas Street, and the Pond within the Millcroft Golf Course immediately upstream of Upper Middle Road. These ponds were filled in, using the Terrain Modification tool within RAS Mapper, to avoid double-counting of their storage capacities within the hydrologic model (VO Model; refer to Hydrology Report) and within the terrain in the HEC-RAS 2D model. Pond 804 to an elevation of 152.8 m.a.s.l and the pond on Appleby Creek at Upper Middle Road to and elevation of 136.5 m.a.s.l to align with the emergency spillway weir elevation.

Note that the 100-year quantity control function of Pond 804 was included in the hydrologic modelling, and assessed using the developed 2D modelling (without filling). This is detailed as part of the "SWM Pond Review Report" (WSP, March 2023) which forms Appendix B of the overall Hydrology Report.

For simplicity, only the 2D modelling results pertaining to the Regional Storm have been discussed here. The spills at Dundas Street overtop the road and inundate the intersection of Cornerstone Drive and Appleby Line. The spill on Appleby Line is fed by both the northern and southern branches of Sheldon Creek until Upper Middle Road, after which it inundates the neighbouring residential areas and finally spills into Appleby Creek.

The Sheldon Creek branch SD_W_201 has a large floodplain that extends westwards and inundates the plaza at southwest of the intersection of Dundas St. and Appleby Line, crossing the CNR line and inundating Millcroft Park on the other side. Lastly, spills were observed exiting the system at the southwestern boundary of the study area, towards the parallel branch of Appleby Creek AP_E_103.

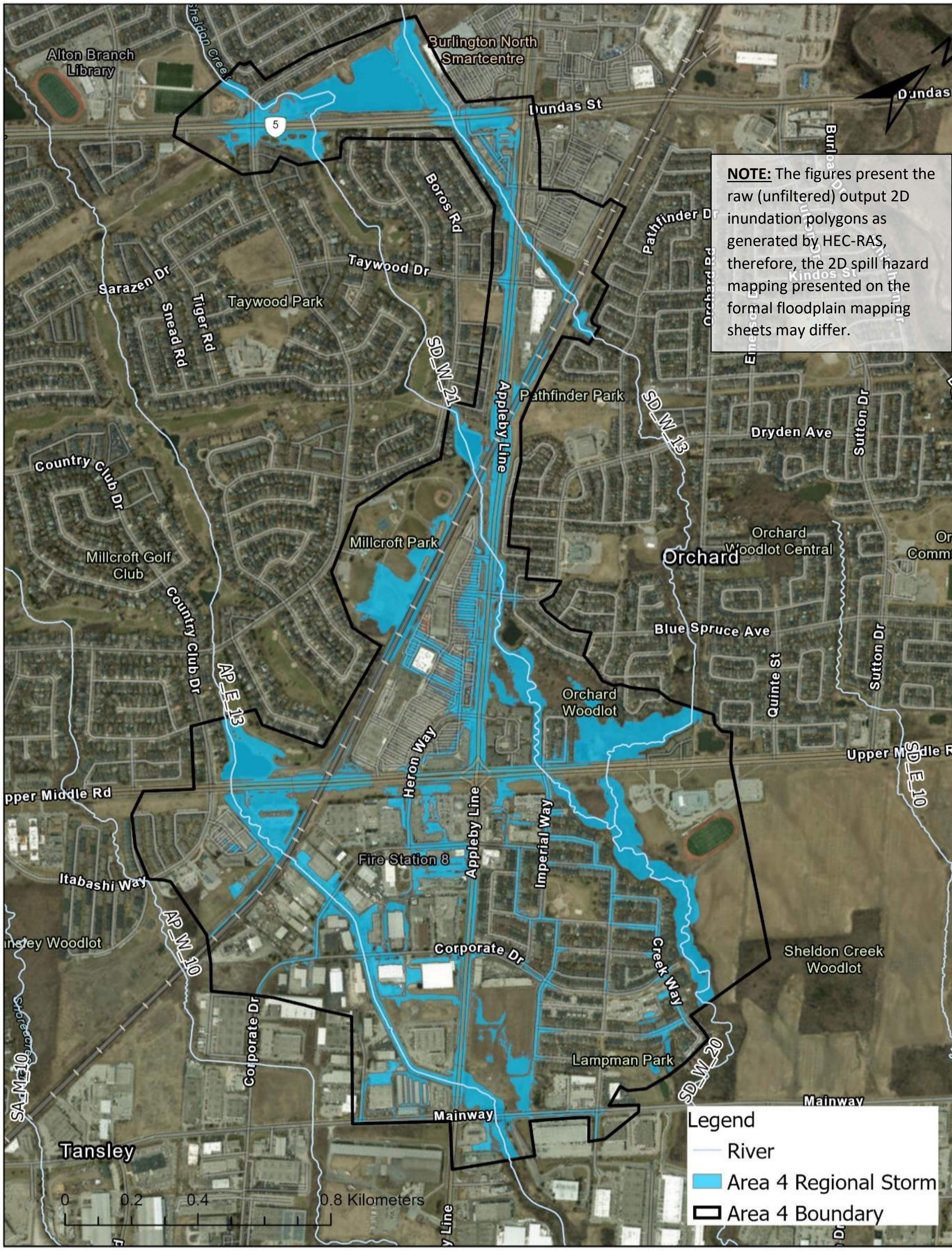


Figure 5.4: Area 4 2D Floodplain Inundation Limits (Regional Storm – Future Conditions)

5.2.5 AREA 5 – APPLEBY CREEK (QEW TO PINELAND PARK NEAR SPRUCE AVENUE)

The Area 5- Appleby Creek model (331 ha) has been developed as a single watershed system model. The model extents of this 2D HEC-RAS model covers the area just north of the QEW to just south to Pineland Park near Meadowhill Road and Spruce Avenue.

A total of nine (9) hydraulic structures have been modeled which include two bridges and seven culverts. General model development and modeling methods have been described in the previous sections. This model has been executed for the Regional Storm (12H AMC III Future Conditions flows) and 100-year Storm (12H AMC II SCS distribution, future conditions flow).

Model adjustments have been implemented to account for spill flows or to correct for the impacts of flow and volume attenuation behind hydraulic structures within this model. Flow balancing was carried out as per methodology suggested by CH and supported by WSP and the TAC to better align with the MNRF guidance. One such notable spill was added back to the model at the culvert ST109-110 at Appleby Line and N Service Road. Flows for structure attenuation was corrected for the QEW culvert and the CNR culvert near the Appleby GO station on the AP_E_13 reach.

The Regional Storm flows inundate the west and east bound lanes of the QEW east of Appleby Line. Flows also inundate the industrial area just south of Harvester Road, however, it should be noted that the CNR crossing is not overtopped. Further, the channel does not have the capacity to contain the Regional Storm flows just south of the CNR and flows spill west and east onto Fairview Street. Spill follows multiple flow paths south along several streets (ref. Figure 5.5 below).

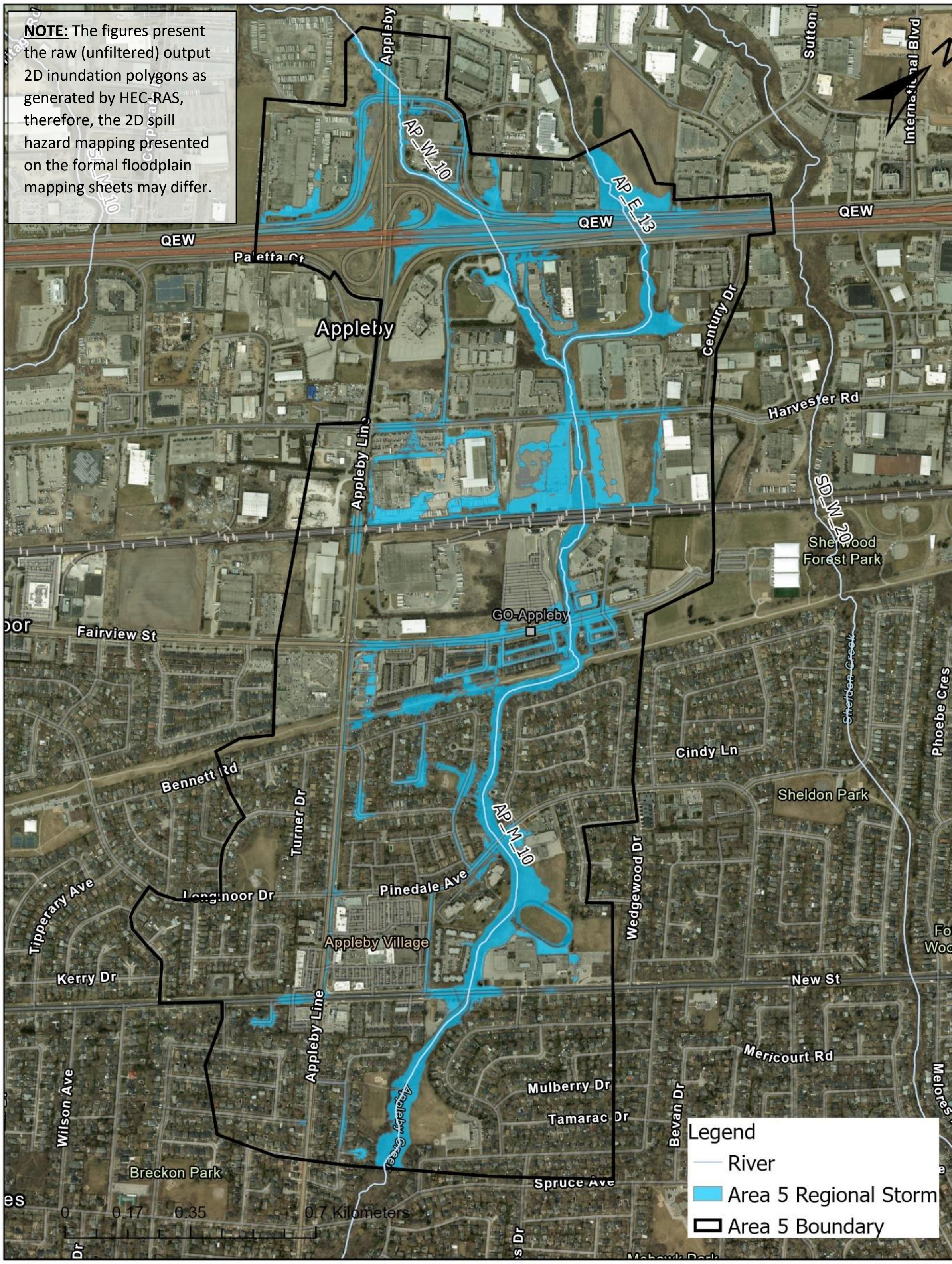


Figure 5.5: Area 5 2D Floodplain Inundation Limits (Regional Storm – Future Conditions)

5.2.6 AREA 6/7 – SHELDON CREEK (SHELDON PARK/RIVERSIDE DRIVE AND REBECCA STREET TO LAKESHORE ROAD WEST)

The Area 6/7- Sheldon Creek model (217 ha) has been developed as a single watershed system model. The model extents of this 2D HEC-RAS model cover two reaches from upstream of New Street to Lake Ontario.

A total of 12 hydraulic structures have been modeled. This includes six (6) bridges and six (6) inline culverts. General model development and modeling methods have been described in the previous sections. This model has been executed for the Regional Storm (12H AMC III Future Conditions flows) and 100-year Storm (12H AMC II SCS distribution, future conditions flow).

Based on the model results (ref. Figure 5.6), no lateral spill flows into the system were observed.

Balancing of flows was required at two locations. These flow balancings involved accounting for spills exiting the system towards the west at Linbrook Road and towards the south at Calvert Place, Randolph Crescent, Foxbar Road, Euston Road and Burloak Drive. Flow balancing was carried out as per methodology suggested by CH and supported by WSP and the TAC to better align with the MNRF guidance.

For simplicity, only the results pertaining to the Regional Storm have been discussed here. Between Sherwood Park and Burloak Drive, the western reach does not appear to have sufficient capacity to convey the Regional Storm. The inundation overtops the creek banks and inundates the residential neighbourhoods on both sides; this was also observed in the hydroburn model.

The eastern reach overtops Rebecca Street and inundates the residential neighbourhood further south, however, it is largely relegated to the cul-de-sacs and as it overtopped Rebecca Street a split flow, of sorts, occurred, where the flow now travels south on Great Lakes Boulevard, and finally drains back into the east reach at Creek Path Avenue.

The east and west branches confluence east of Wilmot Park, here Sheldon Creek is fairly contained until downstream of Lakeshore Road, where it spills onto Shelburn Place.

It should be noted that the inundation extents presented in Figure 5.6 assume application of full flows at the upstream limit of the mesh, downstream of 3361 Rebecca Street. A separate version of the 2D modelling has been completed which considers application of full flows upstream of this location. Under this scenario, an extensive spill is indicated across Rebecca Street and towards Nautical and Village Wood Parks, continuing easterly towards Bronte Creek; refer to Table 5.1 for further details. Both of these inundation results are also presented on the generated floodplain mapping sheets for this project.

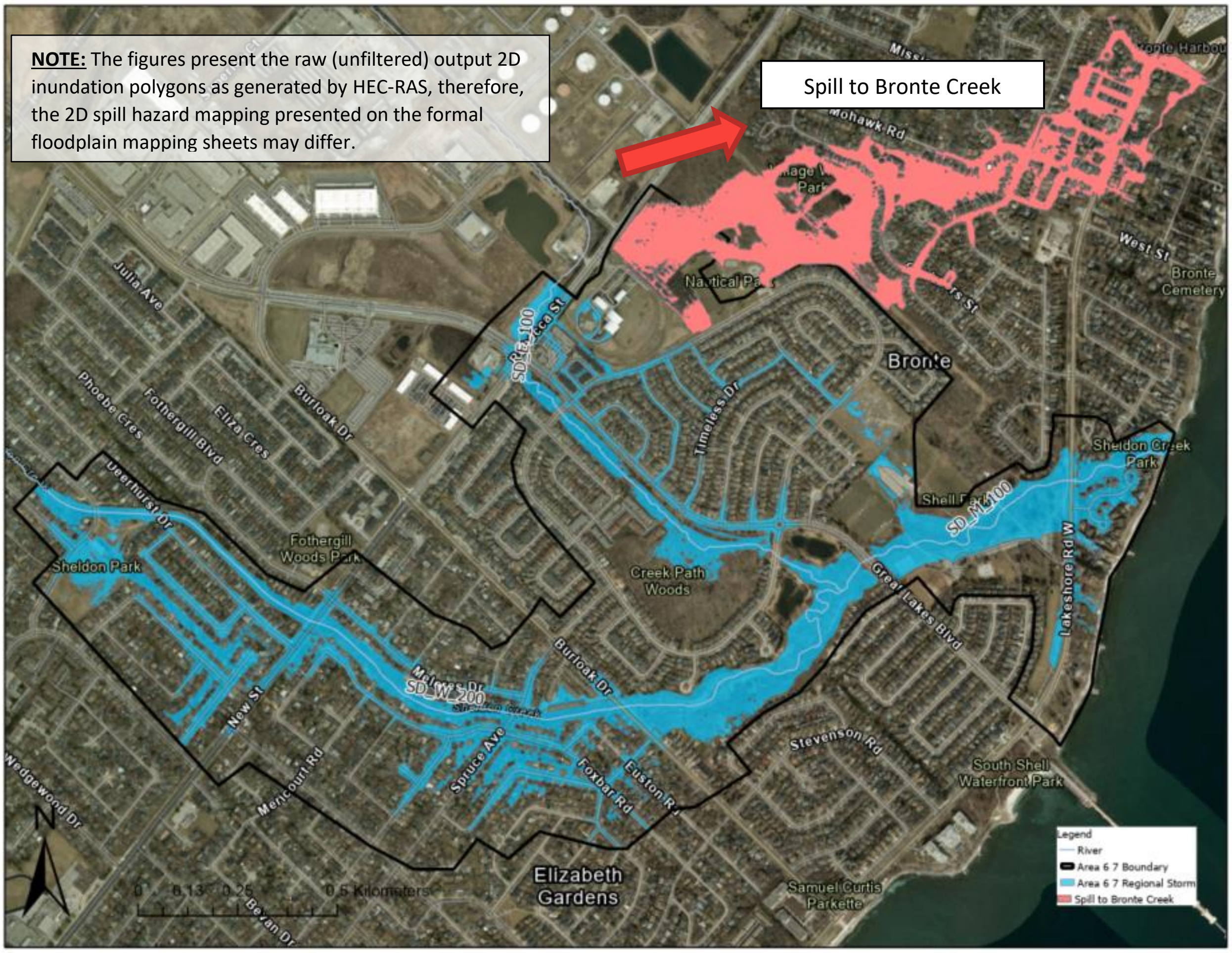


Figure 5.6: Area 6/7 2D Floodplain Inundation Limits (Regional Storm – Future Conditions)

5.2.7 AREA 8 – SHELDON CREEK (QEW TO MICHIGAN DRIVE)

The Area 8- Sheldon Creek model (268 ha) has been developed as a single watershed system model. The model extents of this 2D HEC-RAS model cover two (2) reaches from upstream of the QEW. The model extents terminate at Sherwood Forest Park for one of the reaches and downstream of Michigan Drive near Rebecca Street for the other.

A total of ten (10) hydraulic structures have been modeled which include two(2) bridges and eight (8) culverts. General model development and modeling methods have been described in the previous sections. This model has been executed for the Regional Storm (12H AMC III Future Conditions flows) and 100-year Storm (12H AMC II SCS distribution, future conditions flow).

Model adjustments have been implemented to account for spill flows or to correct for the impacts of flow and volume attenuation behind hydraulic structures as per methodology suggested by CH and supported by WSP and the TAC to better align with the MNRF guidance. Flows have been adjusted for three (3) structures, namely, the QEW, Burloak Drive and CNR on the SD_E_10 reach. The Regional Storm Flows interact between Area 5 and this model on the west bound lanes of the QEW, however, flows do not appear to overtop the median barrier and stay within the west bound lanes and the north ditch of the QEW. Some spills are noted onto Harvestor Road and the CNR south of Wyecroft Road (ref. Figure 5.7 below).

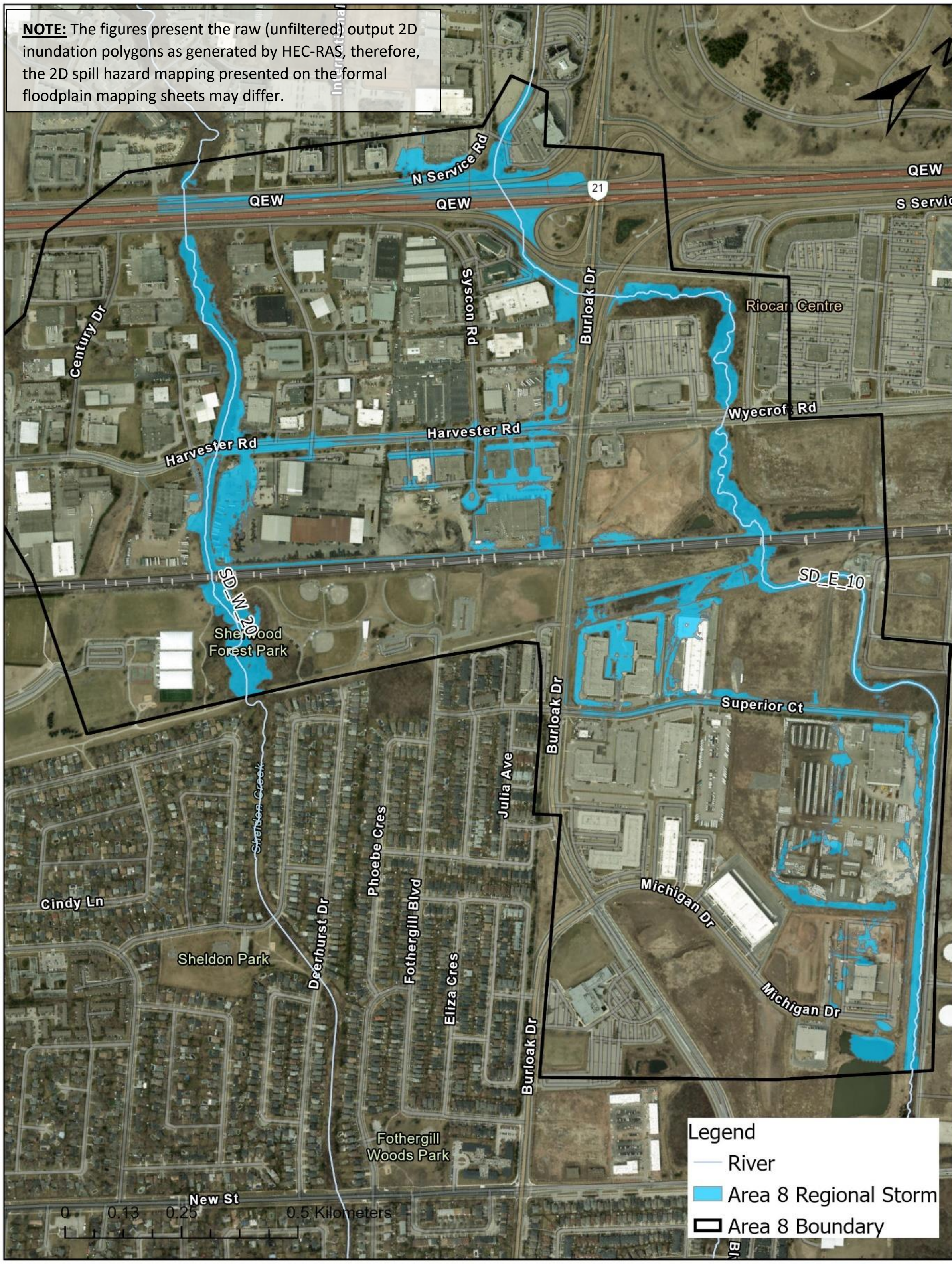


Figure 5.7: Area 8 2D Floodplain Inundation Limits (Regional Storm – Future Conditions)

5.2.8 SUMMARY OF SPILL AREAS

Model adjustments have been implemented to account for spill flows or to correct for the impacts of flow and volume attenuation behind hydraulic structures within the 2D models as described in the preceding sections. Based upon the results of the 2D modelling, spill areas have been identified and evaluated to determine if they meet the criteria for inclusion in the hydrologic modelling and subsequent 1D hydraulic modelling, in accordance with modelling methodology suggested by CH and supported by WSP and the TAC to better align with the MNRF guidance. (refer to Appendix D).

A summary of the resulting spills (inter-basin and intra-basin) occurring during the Regional Storm which have met the criteria for inclusion as part of the iterative analysis has been provided in Table 5.1. Further discussion and details related to the spill results and impacts to overall hydrology, as well as any spills during the 100-year storm can be found within the Hydrology Report.

Table 5.1: Summary of Final Spill Flows (Regional)

2D AREA MODEL	INTER-BASIN SPILLS – REGIONAL STORM				
	SPILL TYPE	SPILL FROM	SPILL TO	PEAK FLOW	INCLUDED AS INFLOW IN VO?
Area 1	Inter-Basin	Sheldon Creek	Bronte Creek	4.2 m^3/s at Highway 407ETR	N/A
	Inter-Basin	Appleby Creek	Sheldon Creek	4.3 m^3/s at Highway 407ETR	Yes
	Intra-Basin	Shoreacres Creek West – East Trib	Shoreacres Creek West – West Trib	3.7 m^3/s at Highway 407ETR	Yes
	Intra-Basin	Shoreacres Creek West Branch	Shoreacres Creek East Branch	5.6 m^3/s upstream of Dundas Street	Yes
Area 2	Inter-Basin	Tuck Creek	Roseland Creek	6.9 m^3/s at CNR-Pioneer Road 24.9 m^3/s at QEW	N/A
Area 3	Inter-Basin	Shoreacres Creek	Tuck Creek	18.7 m^3/s at QEW	Yes
Area 4	Intra-Basin	Sheldon Creek	Sheldon Creek	9.0 m^3/s at Upper Middle Road 9.1 m^3/s at Pond 804	Yes
Area 5	Inter-Basin	Appleby Creek	Shoreacres Creek	5.3 m^3/s at QEW	Yes
Area 6/7	Inter-Basin	Sheldon Creek	Bronte Creek	30.0 m^3/s at Rebecca Street	N/A
Area 8	-	-	-	-	-

6 FLOOD HAZARD MAPPING PROCESS

6.1 1D FLOODPLAIN MAPPING

Following the completion of hydraulic modelling (as described in previous sections), floodplain mapping extents have been generated by WSP. Initial (draft) floodlines have been generated using RASMapper within HEC-RAS for the 1D component of the modelling efforts. Following comments from CH and the TAC, the modelling and mapping have been further refined, and a more rigorous process for 1D floodline delineation has been applied.

Manual quality checks have been undertaken with special attention paid to areas where infilling was required to remedy topographic highs that artificially cut off sections of floodlines that would otherwise be connected or vice-versa where floodlines had to be clipped to account for true topographic highs. Other typical areas of screening include:

- Floodline exceeds cross-section extents
- Gaps in the floodline
- Irregularly shaped floodline extents
- Correct rendering of floodline at hydraulic structures (i.e. whether or not deck is overtopped)
- Floodplain “islands” (high points within the flood inundated area)
- Floodplain extent excessively close to watercourse centreline
- Connectivity between adjacent floodplain areas
- Potential or confirmed spill areas
- Water Surface Elevation (WSE) labels ensuring no decreasing WSE results

This “clean-up” has been applied to both the 100-year and the Regional storm event flood hazard limits and not the flood inundation limits for the other design storms (2-, through to the 50-year) which have been generated for use in flood risk screening and characterization only.

Formal floodplain mapping sheets have been developed to present the resulting floodplain limits A total of 50 - 24”x36” (Arch D) mapping sheets have been prepared by WSP (1:2,000 scale) to cover the study area limits.

6.2 2D SPILL HAZARD MAPPING

Following the completion of 2D hydraulic modelling (as described in previous sections), 2D spills mapping extents have been generated by WSP for the Regional Storm Event. These have been generated using RASMapper within HEC-RAS. The spill mapping extents were defined based on guidelines from CH (Approach to Mapping Spills

Including Criteria for Defining the Limit of the Flood Hazard Associated with Spills, CH Memorandum dated May 24, 2022), which were supported by WSP and the TAC. The CH guidelines and process followed to generate 2D spills are presented in **Appendix D.**

The 2D models have been used to define the spill flow pathways associated with the extent of riverine flooding but does not reflect other mechanisms of flooding. Based on CH's guidance, the limits of mapped spills have been terminated where erosion and public safety are not of concern, even when considering potential for cumulative impacts.

Typically, spills have been mapped where the combined spill flows leaving the system are greater than 5 m^3/s, and terminated where all points downstream along the spill pathway meet the following criteria outlined in Table 6.1.

Table 6.1: Spill Mapping Criteriaa (ref. CH Memorandum)

CRITERIA	PUBLIC ROW	PUBLIC LANDS	PRIVATE PROPERTY
Depth	<0.3 m (exception for ditches)	<0.05 m	<0.05 m
Velocity	<1.7 m/s on paved surfaces <0.9 m/s on vegetated surfaces	<0.9 m/s	<0.9 m/s
Depth Velocity Product	<0.37 m^2/s	<0.37 m^2/s	<0.37 m^2/s

It should be noted that in Section 5, the figures present the raw (unfiltered) output 2D inundation polygons as generated by HEC-RAS, therefore, the 2D spill hazard mapping presented on the formal floodplain mapping sheets may differ.

6.3 SUMMARY OF MAPPING CHANGES

This work provides a more detailed understanding of the flood hazard associated with Tuck, Shoreacres, Appleby and Sheldon Creeks. The modelling has been developed from detailed topographic data collected using LiDAR technology and applies modern software that allows for a more sophisticated analysis of the riverine floodplain and spills that comprise the flood hazard. In some areas the known extent of the hazard has decreased, while in other areas it has increased. There are many factors that have influenced these changes, including:

- New affordable tools and technology have enabled mapping of significant spills, which were previously only represented by an arrow or opening in the floodplain limit at the point of spill. The spill mapping extents have been generated by following a robust case by case analysis and utilizing current 2-Dimensional modeling.

- LiDAR data have provided a better understanding of topography, allowing the flood hazard limit to be drawn with greater precision.

- The HEC RAS 6.3.1 model platform improves upon previous model versions, incorporating numerous refinements and new analytical tools that support both model development and quality assurance reviews.

- New hydrologic models were developed to define the flows associated with the regulatory flood. This is discussed in greater detail in the Hydrology Report, however, differences in flows are generally attributed to:

  - application of different rainfall distributions and areal reduction factors (including representation of a climate adjusted future condition per the City of Burlington's "Storwmater Management Design Guidelines" for the 2-100 year storm event flows)

  - use of a different modelling platform (i.e., VO 6.2 vs. SWMHYMO, GAWSER, or HSP-F)

  - differences in parameterization and methodology, including application of an assumed future land use condition

  - differences in contributing drainage areas due to refined topographic information, including split flow drainage areas, and

  - differences in modelling approaches applied for SWM infrastructure (e.g., some past studies recognized private lot level controls including parking lot and rooftop storage and private ponds). In this study, stormwater management controls were considered in detail based on current guidelines and practices, and nine (9) municipally owned ponds have been recognized in the 1:100 year analysis, while one (1) municipally owned pond, which was designed to provide quantity controls under the Hurricane Hazel storm, has been accounted for in the Regional storm scenario.

6.4 EVALUATION OF STRUCTURES (BUILDINGS) AT RISK

Based upon the results of the updated flood hazard modelling and mapping, an evaluation of structures (buildings) at risk has been completed for the 1D modelling results, representing the 100-year and Regional flood hazard limits, and the 2D modelling results which represents the Spill Inundation Limits. This has been completed through GIS analysis to determine the number of buildings which are located within the flood hazard limits (i.e., touching at any point), based upon the building footprint mapping provided by CH for use in the current study. It should be noted that this GIS analysis does not account for the depth of the floodplain surrounding the buildings, and is rather a locational analysis based upon which watershed the building is located in, and the flood hazard limits determined as part of this study.

A summary of the buildings at risk per watershed is provided in Table 6.2.

Table 6.2: Summary of Estimated Buildings at Risk per Watershed

WATERSHED[1]	ESTIMATED NUMBER OF BUILDINGS WITHIN THE FLOOD HAZARD		
	100-YEAR 1D FLOOD HAZARD	REGIONAL 1D FLOODPLAIN	SPILL INUNDATION LIMITS
Tuck Creek	109	190	181
Shoreacres Creek	15	55	174
Appleby Creek	27	43	131
Sheldon Creek	73	181	267
Total	**224**	**469**	**753**

Note:
1. Buildings which are located within two (2) watersheds (i.e., along the watershed boundary) have been accounted for in both the subject watershed results.
2. Does not include properties affected by spill from Sheldon Creek to Bronte Creek.
3. Numbers are estimated given issues with selecting properties in close proximity to boundary limits.

The results of this analysis demonstrate that there are approximately a total of 224 (+/-) buildings located within the 100-year flood hazard limit, a total of 469 (+/-) buildings located within the Regional floodplain and a total of 753 (+/-) buildings located within the spill inundation limits across the four (4) primary watersheds. As noted however, there may be buildings that are located along watershed boundaries that may have been counted twice.

Tuck Creek was found to have the most buildings at risk under both the Regional and the 100-year storms, followed by Sheldon Creek, whereas Shoreacres Creek and Appleby Creek had similar levels of buildings at risk under the Regional and 100-year storms. Under spill conditions, Sheldon Creek had the highest number of buildings located within the spill inundation limits, followed by Shoreacres Creek, Tuck Creek and Appleby Creek, respectively.

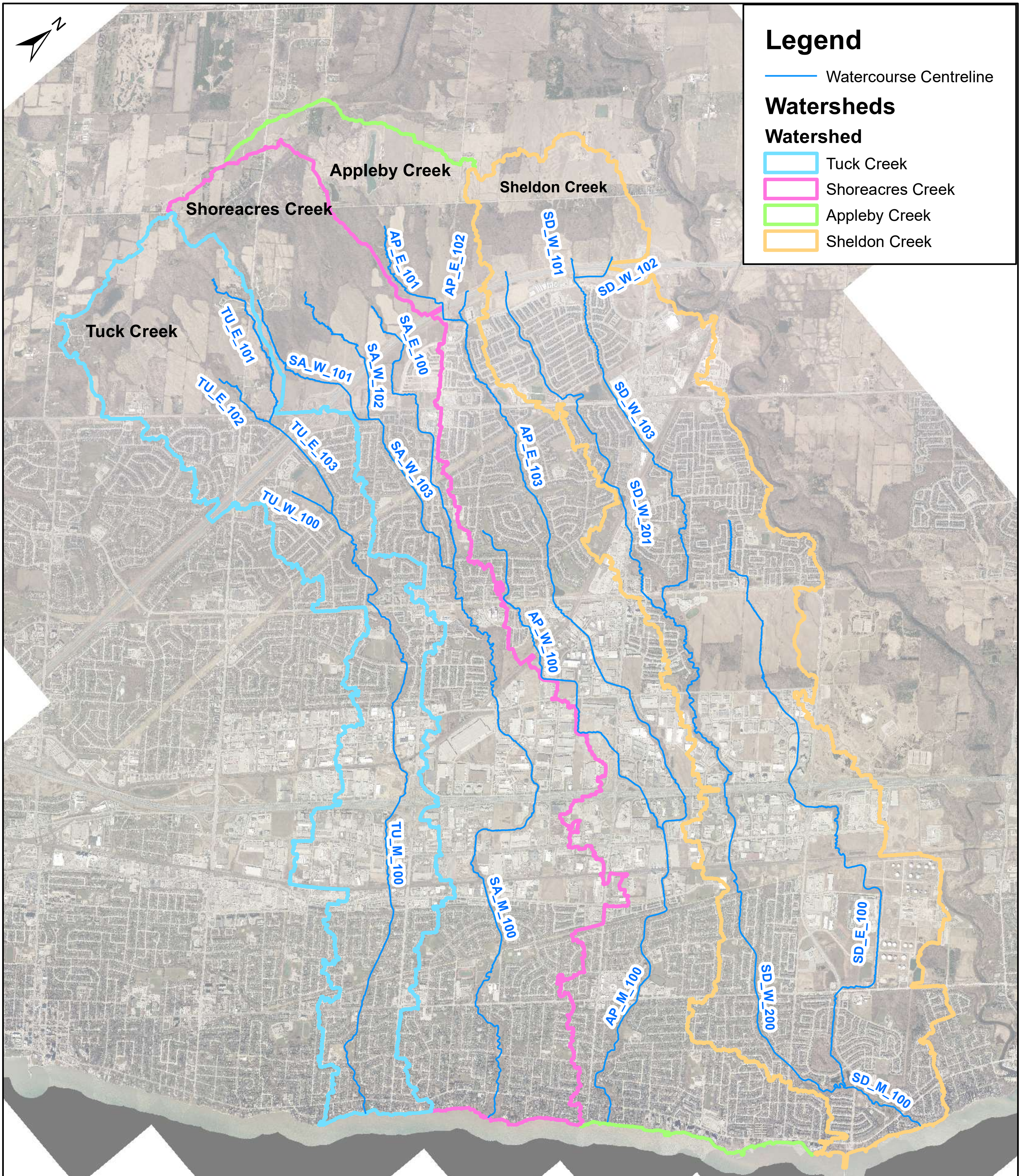
In addition to the buildings identitied in Table 6.2, there are an additional approximately 176 (+/-) buildings located within the spill inundation limits in external watershed systems (i.e., 161 +\- to Bronte Creek and 15 +\- to Roseland Creek). These buildings at risk are all demonstrated on the flood hazard mapping sheets attached to this report (ref. Appendix H).

7 SUMMARY AND CONCLUSIONS

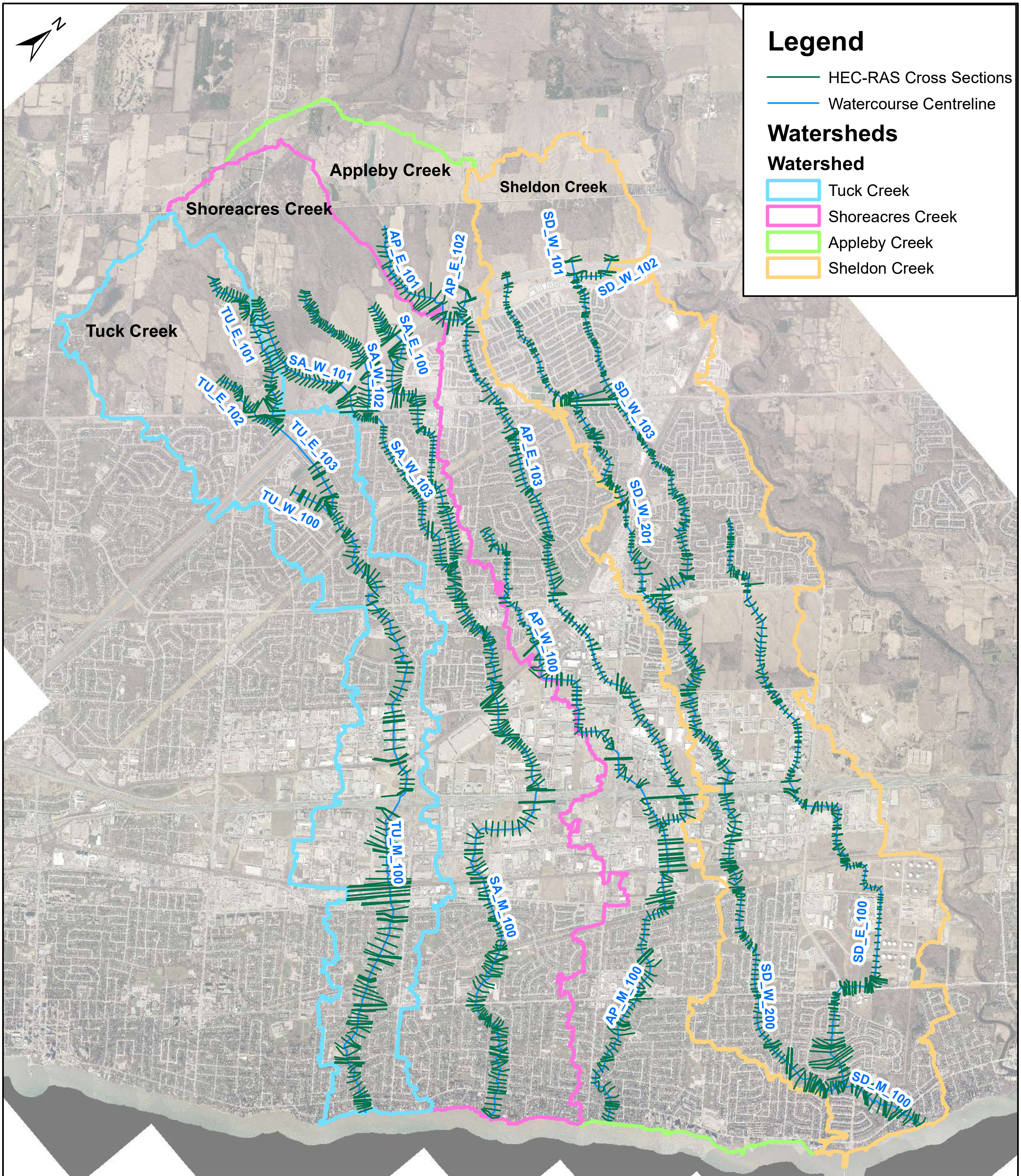
New hydraulic models have been developed in HEC-RAS for the study area, using current best practices and standards, including the specifications of Conservation Halton and is in keeping with the Technical Guide for River & Stream Systems: Flooding Hazard Limit (2002). The hydraulic modelling includes 1-dimensional (1D) models for the primary watercourses of interest within each of the four (4) subject watersheds (Tuck, Shoreacres, Appleby and Sheldon Creeks). In addition, due to the number of identified spills (both inter-watershed and intra-watershed), a total of eight (8) different 2D modelling areas have been modelled to better assess spill pathways and expected inundation areas. The 2D modelling has been developed iteratively with the hydrologic modelling using the balanced approach proposed by CH and supported by WSP and TAC for this study to ensure consistency.

Future studies should consider the use of the more complex Full Momentum Equations to provide an enhanced understanding of depth and velocities within the spill zone.

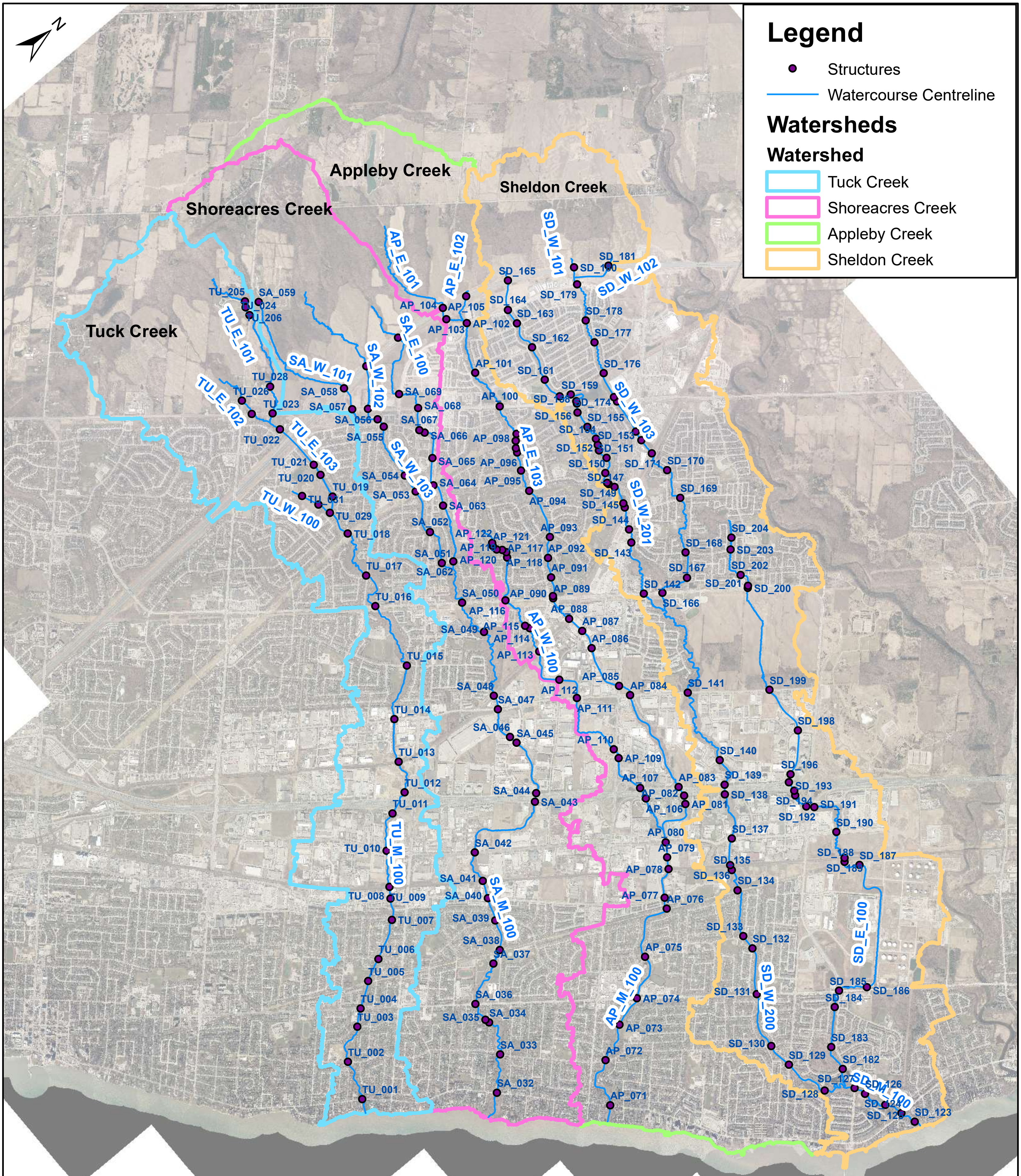
Notwithstanding the recommendation that the Full Momentum Equations be considered for use in future study, the presented hydraulic modelling results are considered valid and appropriate and have been applied in conjunction with the estimated flows for the subject areas (refer to the companion Hydrology Report) to develop flood hazard mapping, as attached separately in Appendix H. The modelling and mapping produced as part of this study are considered appropriate for use in the administration of Ontario Regulation 162/06 and land use descision making.



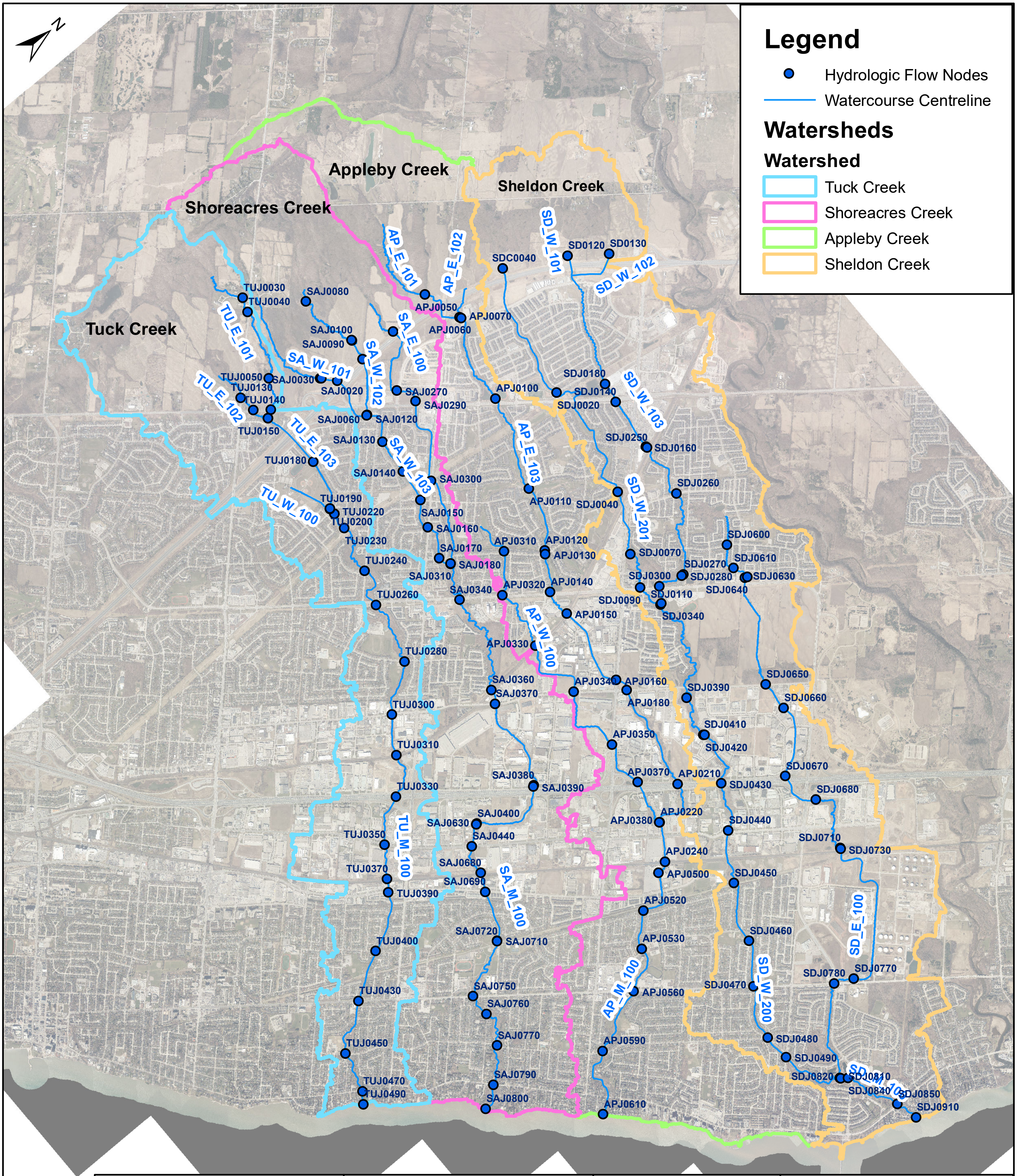
East Burlington Creeks Flood Hazard Mapping Update Conservation Halton	HEC-RAS Watercourse Centrelines	wsp	Scale 1:50,000 Project No. WW21011057 Drawing No. 1



East Burlington Creeks Flood Hazard Mapping Update **Conservation Halton**	**HEC-RAS Cross Section Location Plan**	wsp	Scale **1:50,000** Project No. **WW21011057** Drawing No. **2**



East Burlington Creeks Flood Hazard Mapping Update Conservation Halton	Hydraulic Structure Location Plan	wsp	Scale 1:50,000 Project No. WW21011057 Drawing No. 3

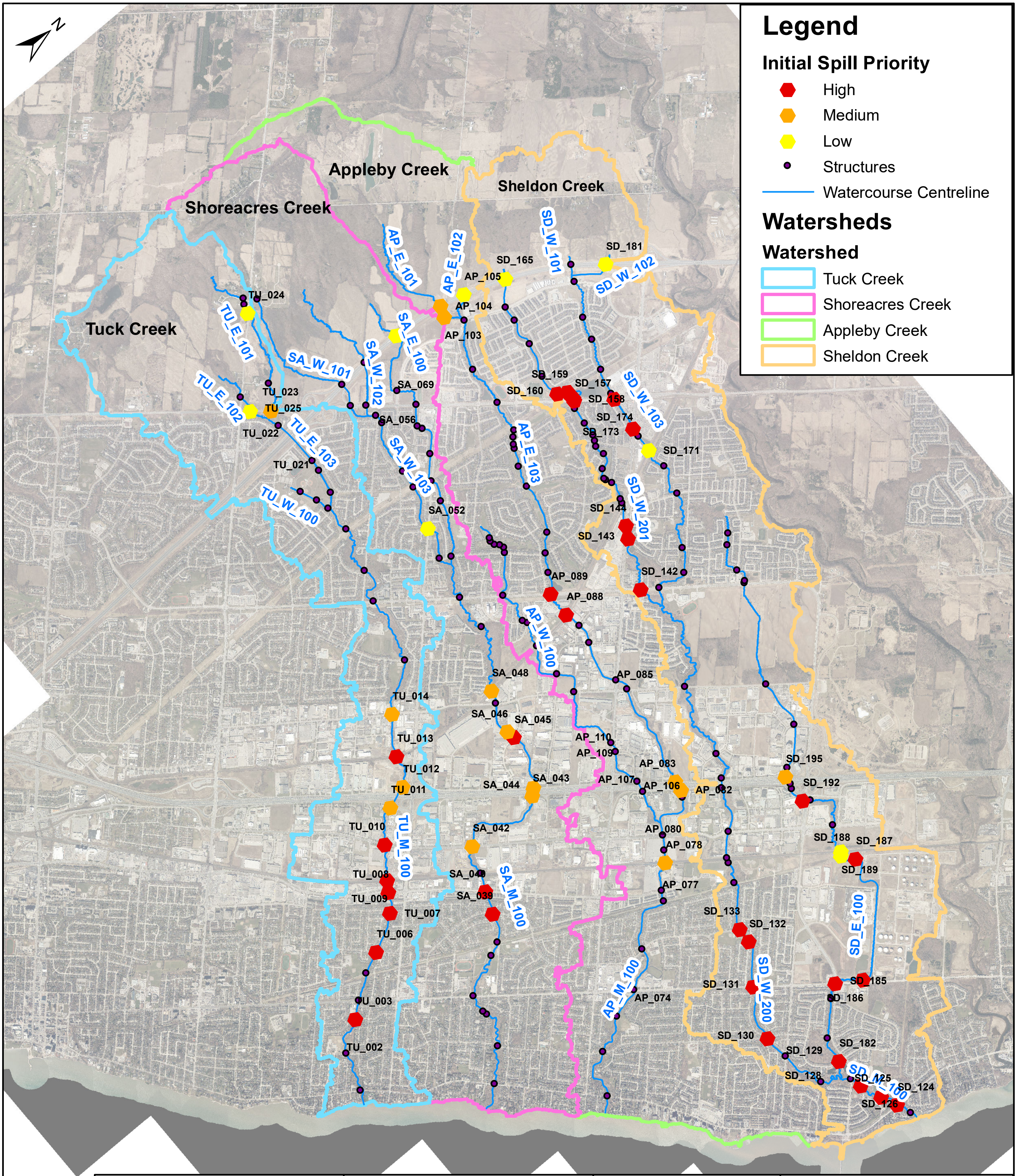


East Burlington Creeks Flood Hazard Mapping Update

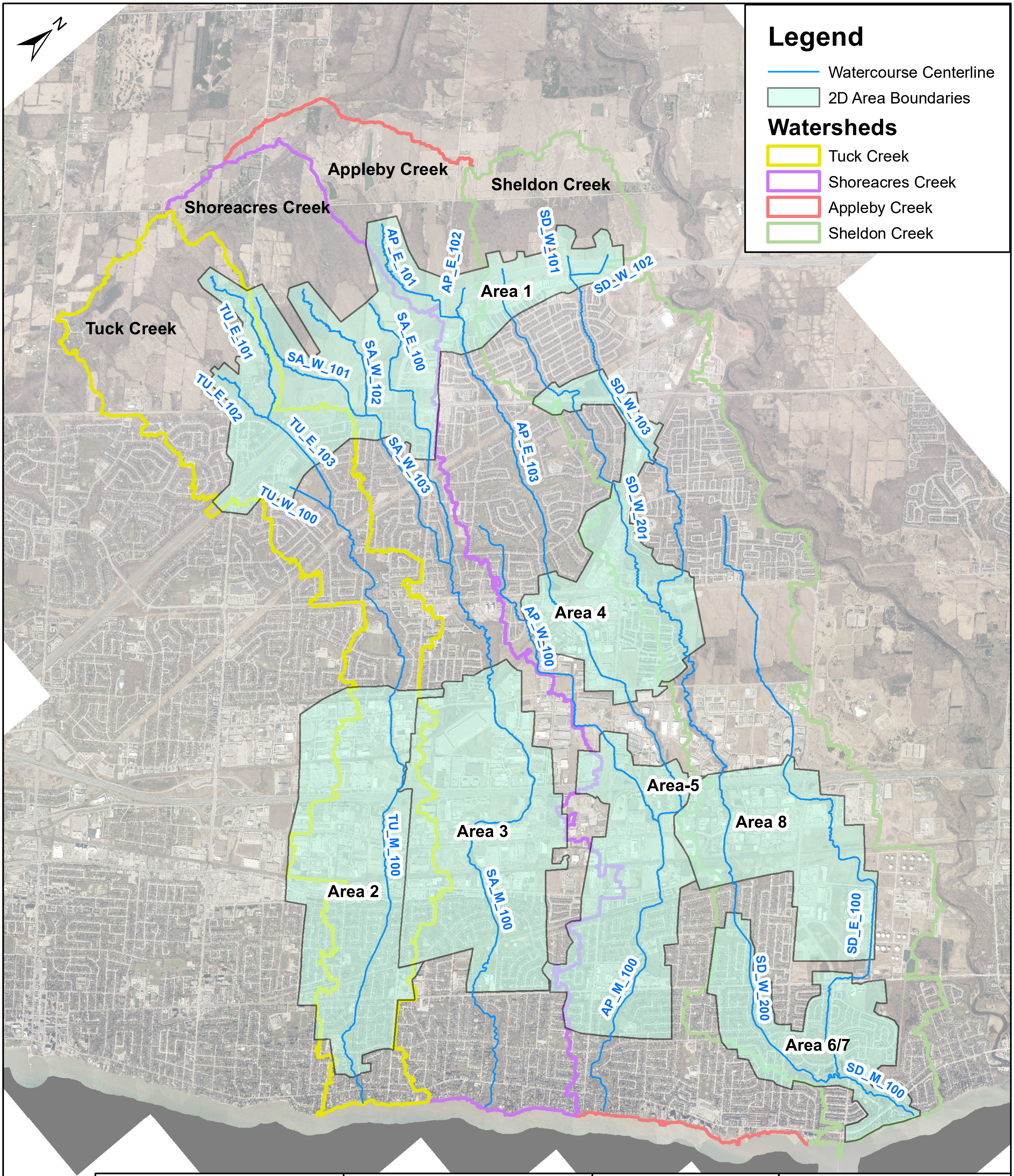
Conservation Halton

Steady Flow Node Location Plan

Scale	1:50,000
Project No.	WW21011057
Drawing No.	4



East Burlington Creeks Flood Hazard Mapping Update Conservation Halton	Initial Spill Area Location Plan	wsp	Scale 1:50,000 Project No. WW21011057 Drawing No. 5



East Burlington Creeks Flood Hazard Mapping Update Conservation Halton	HEC-RAS 2D Area Model Boundaries	wsp	Scale 1:50,000 Project No. WW21011057 Figure No. 6