CONSERVATION HALTON

HYDROLOGY REPORT

EAST BURLINGTON CREEKS FLOOD HAZARD MAPPING UPDATE

SEPTEMBER 18, 2023

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CONSERVATION HALTON

PROJECT NO.: WW21011057
DATE: SEPTEMBER 18, 2023

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TABLE OF CONTENTS

FIGURES

TABLES

DRAWINGS

APPENDICES

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 hydrologic modelling and associated documentation includes the following tasks:

— Modelling Approach Tasks (primarily submitted as Technical Memorandum #1)
  — Task 4.1: Discretize Subcatchments
  — Task 4.2: Establish Naming Conventions
  — Task 4.3: Establish Existing Condition Parameters
  — Task 4.4: Select Future Catchment Parameters
  — Task 4.5: Define Routing Elements
  — Task 4.6: Determine Event Rainfall
  — Task 4.7: Evaluate Potential Methods to Calibrate and/or Validate the Models
— Hydrologic Modelling Tasks (primarily submitted as Technical Memorandum #2)
  — Task 4.8: Build Hydrology Models & Sensitivity Analysis
  — Task 4.9: Model Calibration and Validation
  — Task 4.10: Existing Conditions Flows
  — Task 4.11: Update Existing & Define Future Conditions Flows
  — Task 4.12: Document Flows for Hydraulic Modelling
  — Task 4.13: Evaluation of Credited SWMFs
  — Task 4.14: Iterative Analysis for Inter-Basin Spills
  — Task 4.15: Endorse Modelling (Quality Assurance/Quality Control)

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 hydrology 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 hydraulic 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, the City of Burlington, Halton Region, and the Town of Oakville:

- **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)
  - Urban Burlington soil survey mapping (OMAFRA, 1971) – Based on Soils of Halton County (1971)
  - Urban Burlington detailed soil survey compilations data version 2 (CanSIS NSDB, 1990)
  - Watershed boundary shapefile (Conservation Halton, 2021)
  - Watercourses mapping (Conservation Halton, 2021)
  - Regulated wetlands mapping (Conservation Halton, 2020)
  - Rain gauge locations mapping (Conservation Halton, 2016)
  - Stormwater Management Pond Mapping (Conservation Halton, 2021)
  - ArcHydro catchments shapefile (Conservation Halton, 2021)
  - 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)
  - 2020 City of Burlington Official Plan (City of Burlington, 2020)
  - 2009 Town of Oakville Official Plan: Livable Oakville (Town of Oakville, 2009)

- **Monitoring Data**
  - Rainfall
    - Burlington Airport Rain Gauge 5-minute Precipitation Depth (2017-05-30 to 2021-08-25)
    - Burlington WPP Rain Gauge 5-minute Precipitation Depth (2012-05-16 to 2021-08-25)
    - Elizabeth Gardens PS Rain Gauge 5-minute Precipitation Depth (2008-12-18 to 2021-08-25)
    - Fourteen Mile Creek at Oakville Rain Gauge 5-minute Precipitation Depth (2004-02-11 to 2021-08-25)
    - Headon Reservoir Rain Gauge 5-minute Precipitation Depth (2019-10-09 to 2021-08-30)
    - Headon Reservoir Weighing Rain Gauge 15-minute Precipitation Depth (2011-02-30 to 2018-10-01)
    - Mainway Arena Rain Gauge 5-minute Precipitation Depth (1996-03-25 to 2021-08-27)
    - Mid-Halton WWTP Rain Gauge 5-minute Precipitation Depth (2004-02-11 to 2021-08-25)
    - Oakville SW WWTP Rain Gauge 5-minute Precipitation Depth (2004-12-01 to 2021-08-25)
    - Queen Elizabeth Community Centre Rain Gauge 5-minute Precipitation Depth (2011-06-21 to 2021-08-25)
    - Tyandaga Reservoir Rain Gauge 5-minute Precipitation Depth (2011-08-31 to 2021-08-25)
    - Tyandaga Reservoir Rain Gauge 15-minute Precipitation Depth (2011-08-31 to 2021-10-17)
    - Burlington Fire Station #1 15-minute Precipitation Depth (2017-11-24 to 2021-10-20)
    - Oakville Fire Station #4 15-minute Precipitation Depth (2008-11-24 to 2021-10-20)
    - Iroquois Ridge Community Center 15-minute Precipitation Depth (2008-11-20 to 2021-10-20)
    - McCraney Reservoir 15-minute Precipitation Depth (2009-11-09 to 2021-10-20)
    - Elizabeth Garden SPS 15-minute Precipitation Depth (2021-09-21 to 2021-10-17)
    - Headon Reservoir 15-minute Precipitation Depth (2021-08-01 to 2021-10-20)
    - Mainway Arena 15-minute Precipitation Depth (2021-08-01 to 2021-10-20)
  - Flow
    - Fourteen Mile Creek at Oakville (O2HB027) Daily Discharge (2002-05-01 to 2021-08-27)
    - Fourteen Mile Creek at Oakville (O2HB027) 15-minute Discharge (2002-01-01 to 2021-08-29)
    - Hager-Rambo Channel at QEW 5-minute Discharge/Water Level (2018-09-21 to 2021-10-20)
    - Morrison-Wedgewood Channel at Outlet 5-minute Discharge/Water Level (2019-04-30 to 2021-10-20)
  - Water Level
    - Sheldon Creek at Shell Park 5-minute Water Level (2021-10-07 to 2021-10-20)
- **Reports**
  - Area-Wide
    - DRAFT – Pond Crediting – East Burlington Creeks Flood Hazard Mapping Study (Conservation Halton, August 23, 2021)
    - 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)

- **Hydrologic Models**
  - Tuck Creek
    - 2- 100 year and Regional SWMHYMO hydrologic models (Aquafor Beech, 2012)
    - 2- 100 year and Regional OTTHYMO89 hydrologic models (Aquafor Beech, 1996)
  - Shoreacres Creek
    - GAWSER hydrologic model converted from OTTHYMO files (CH, 1993)
    - 2- 100 year, Regional, continuous GAWSER hydrologic models for existing conditions (CH, 1996)
    - 2- 100 year, Regional, continuous GAWSER hydrologic models for future conditions (CH, 1997)
  - Appleby Creek
    - GAWSER hydrologic model converted from OTTHYMO files (CH, 1993)

    - 2- 100 year, Regional, continuous GAWSER hydrologic models for existing conditions (CH, 1996)
    - 2- 100 year, Regional, continuous GAWSER hydrologic models for future conditions (CH, 1997)
- Sheldon Creek
    - HSP-F hydrologic model (Continuous Simulation and Regional Storm) (Wood, October 2019)
    - PCSWMM hydrologic/hydraulic model; Town of Oakville Stormwater Master Plan (Wood, September 2019)

2.2 SUMMARY OF PREVIOUS STUDIES

Tuck Creek Watershed

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 hydrologic model for the study was the HYMO based INTERHYMO model which is an updated version of OTTHYMO. The CN* approach was used to determine direct runoff from pervious areas. Nash unit hydrographs (NASHYD) were applied to simulate runoff responses from rural areas, with the time-to-peak determined using basin lag time. Standard unit hydrographs (STANDHYD) were applied to simulate runoff responses from urban areas. The hydrologic model was calibrated using "observed" peak flows at Spruce Avenue for an event on October 5, 1995. In the study, a critical storm duration and distribution analysis compared Atmospheric Environment Service (AES) 1-hour distribution, AES 12-hour distribution, and Chicago 3-hour distributions (design storms). Results indicated that the 3-hour Chicago distribution produced the largest peak flow rates throughout the watershed for all return period events. Therefore, the 3-hour Chicago distribution were used to determine the 2 through 100-year return period flows. The 12-hour version of the Regional Storm (Hurricane Hazel) of 212 mm was used to determine the Regional Storm flow.

The INTERHYMO model developed in 1996 for the Tuck Creek Erosion Control Study was updated in 2012 for the Tuck Creek Erosion Control Municipal Class Environmental Assessment (ref. Tuck Creek Erosion Control Municipal Class Environmental Assessment Final Report. Aquafor Beech, June 2012). The hydrologic analysis in the 2012 study was undertaken using the SWMHYMO hydrologic model, which was an updated version of the INTERHYMO model. Changes were made to catchments A1-N and A2-N to reflect development that had occurred since 1996. The remaining catchments were unchanged based on the review of the parameters. The IDF curve applied was the most current version (dated from June 1998) as per the City of Burlington's webpage at that time.

There was no updated hydrologic modelling generated for later studies such as the "Tuck Creek Flood Assessment and Crossing Upgrades between New Street and Spruce Avenue Class Environmental Assessment Final Report" (Aqufor Beech, 2016) and the "Tuck Creek Flood Assessment and Crossing Upgrades at Rockwood Drive and Rexway Drive Municipal Class Environmental Assessment, Schedule 'B'" (IBI Group, 2020)

Shoreacres Creek Watershed

The Shoreacres Creek Floodline Mapping Updates (Environmental Water Resources Group Ltd., July 1997) used GAWSER 6.5 to determine peak flow rates and streamflow hydrographs. The 3-hour duration 2 to 100-year return period design storms were developed using the Keifer & Chu method (i.e. Chicago Design Storm). A 48-hour Regional Storm (Hurricane Hazel) was applied to determine the Regional Storm peak flows.

Appleby Creek Watershed

The Appleby Creek Floodline Mapping Updates (Environmental Water Resources Group Ltd., July 1997) used GAWSER 6.5 to determine peak flow rates and streamflow hydrographs. The 3-hour duration 2 to 100-year return period design storms were developed using the Keifer & Chu method (i.e. Chicago Design Storm). A 48-hour Regional (Hurricane Hazel) storm was applied to determine the Regional Storm peak flows.

There was no updated hydrologic modelling generated for the later studies such as "Schedule B Class EA: Municipal Class Environmental Assessment for Appleby Creek Flood Mitigation between Fairview Street and New Street" (Aquafor Beech, 2019) and "Appleby Creek Erosion Control Environmental Assessment Project File Report – Final" (Aquafor Beech, August 2020).

Sheldon Creek Watershed

The City of Burlington retained Amec Foster Wheeler to undertake a hydrologic and hydraulic modelling update of the Sheldon Creek watershed (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 United States Environmental Protection Agency (US-EPA) Hydrologic Simulation Program – Fortran (HSP-F) program. A continuous simulation modelling approach was applied, using 29 years of meteorological data. Frequency analysis was used to determine the 2 through 100-year return period flows. The Regional Storm (Hurricane Hazel) was simulated as a discrete event.

This base model was further refined as part of the 2019 study, including refining subcatchment boundaries to reflect more recent development. Channel routing elements were also updated. A model calibration was undertaken based on the data generated from a flow monitoring program. The continuous simulation dataset was extended through additional data from 1991 to 2003 (42 years), with results validated against typical flows from other studies. A future land use scenario was simulated. Assessments of stormwater management systems, and potential impacts of climate change, were also undertaken.

The previously noted modelling was further refined within the Town of Oakville as part of the "Sheldon Creek Flood Mitigation Opportunities Study" (October 2020), with a further model validation effort completed. The modelling was also used to simulate local historic extreme storm events, including the August 4th, 2014 Burlington Storm.

Area-Wide – August 4th, 2014 Storm Event

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 runoff that overflowed watercourse banks. The August 4th, 2014 storm event was characterized using data from approximately 34 rainfall gauges and two radar stations. There were only three gauges close to the storm centre. Due to the narrow width of the storm cell, the storm missed most of the rainfall gauges within the Burlington area. Radar data was used to refine the rainfall distribution across Burlington. NexRAD radar data was obtained for the Buffalo station through the National Oceanic and Atmospheric Administration (NOAA), and radar data was obtained from the King City (Environment Canada) station. Total rainfall amounts, from the 34 rainfall gauges, were compared to rainfall amounts from both the NexRAD radar, and the King City radar. The NexRAD total rainfall values were closer to the observed rain gauge totals than the King City radar and thus were used in the radar calibration exercise.

On August 4th, 2014 rainfall started at approximately 2:00 pm and ended by 9:00 pm. The NexRAD radar (maximum cell) rainfall total was approximately 196 mm while the maximum recorded rainfall gauge total was approximately 192 mm. The NexRAD cell rainfall totals were used to determine the following watershed average rainfall totals:

Watershed	**Average Rainfall (mm)**
Tuck Creek	150
Shoreacres Creek	140
Appleby Creek	130
Sheldon Creek	100

Area-Wide – City of Burlington Flood Vulnerability Study

In 2017, Amec Foster Wheeler 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). The August 4th, 2014 event produced average subwatershed rainfall depths up to 150 mm over 6 -7 hours and a maximum localized rainfall depth of nearly 200 mm. The hyetograph of the August 4th, 2014 storm event represents a storm duration of 6.5 hours and 196 mm of total rainfall. The hydrologic analysis was based on the most currently available models at that time including the following:

- Tuck Creek Watershed: June 2012 SWMHYMO Model (ref. Tuck Creek Erosion Control Municipal Class Environmental Assessment Final Report. Aquafor Beech, June 2012),
- Shoreacres Creek Watershed: July 1997 GAWSER model (ref. Appleby Creek Floodline Mapping Update, Environmental Water Resources Group Ltd., July 1997),
- Appleby Creek Watershed: August 1997 GAWSER model (ref. Appleby Creek Floodline Mapping Update, Environmental Water Resources Group Ltd., July 1997), and J
- Sheldon Creek Watershed: June 2017 HSP-F model (Sheldon Creek Watershed Hydrologic and Hydraulic Study, June 2017).

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. Water surface elevations were then converted to estimated flow rates using stage-flow rating curves from available hydraulic modelling used in previous floodplain mapping studies. The estimated flow rates were compared to 3-hour Chicago Storm peak flow rates and 6-hour SCS Type II peak flow rates. Due to the limited accuracy of the observed data, a direct conclusion could not be drawn. The 2017 study recommended that more data collection and refined modelling (with calibration) be completed in the future.

Area-Wide – Appleby GO Mobility Hub (Major Transit Station Area)

Beginning in 2017, the City of Burlington undertook a land use planning study for four (4) Mobility Hub areas (now referred to as Major Transit Station Areas or MTSAs). These areas are located around major transit hubs within the City (Appleby GO, Burlington GO, Aldershot GO and the Downtown area) where re-development and intensification was expected. In support of the planning effort (lead by Brook McIlroy Inc), and the scoped Environmental Impact Studies (EIS – lead by Dillon Consulting Limited), a series of flood hazard and scoped stormwater management assessments were prepared by Wood. Of relevance to the current study, the report for the Appleby GO area considered the area watercourses, namely Shoreacres Creek, Appleby Creek, and Sheldon Creek.

The study utilized existing approved hydrologic and hydraulic models for the study area. These models were updated and refined as required to better define the expected flood hazards for the study area. Potential spill areas were also identified through the hydraulic modelling effort. Updated floodplain mapping was prepared to define flood hazard limits. An overall flood management strategy for the area was proposed based on these results. Potential hydraulic structure improvements were noted. An overall stormwater management strategy for the area was also proposed to mitigate potential impacts; the strategy was largely consistent with the directives provided in the City's updated Stormwater Management Design Guidelines (as finalized May 2020).

Reporting ("Flood Hazard and Scoped Stormwater Management Assessment – Appleby GO Mobility Hub", Wood) was largely finalized in April 2019, however a finalized/updated version of the report was issued August 2021.

3 HYDROLOGIC 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 are also not incorporated as part of the elevation features. 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. The horizontal and vertical datums to be used for hydrologic and hydraulic models (and associated mapping and reporting) are indicated as the following:

— Horizontal datum: North American Datum (NAD) 1983 coordinate system expressed 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.

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. The finer resolution 0.5 x 0.5 m LIO Halton DTM would be more appropriate to use for hydraulic analyses, as discussed separately.

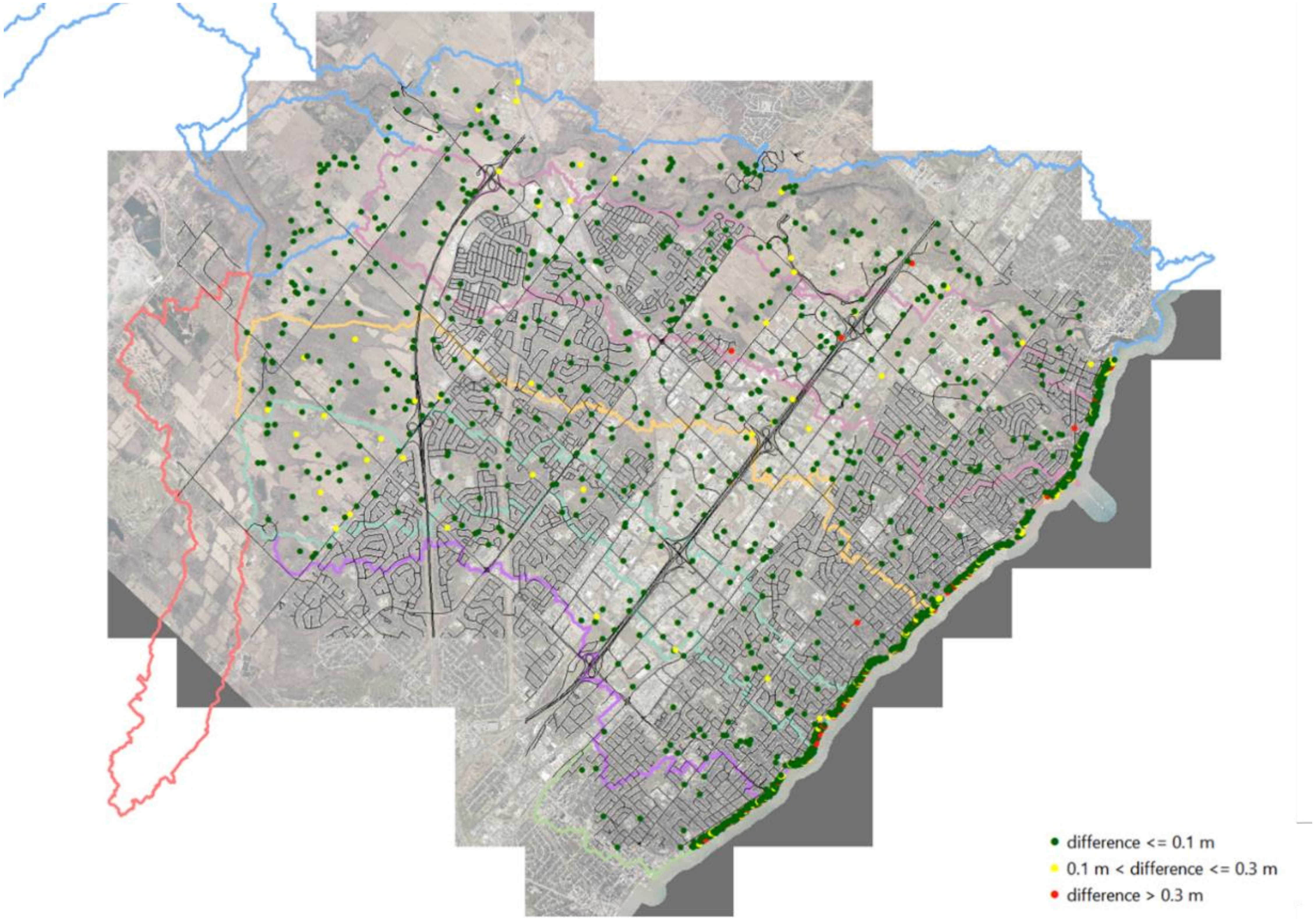


Figure 3.1. Point Elevation between Conservation Halton LiDAR and LIO Halton DTM

3.1.2 SUBCATCHMENT BOUNDARIES

The ArcHydro subcatchment boundaries provided by Conservation Halton for reference in this study have been reviewed and refined within the extent of the study area by WSP, based on the Conservation Halton bare earth LiDAR data and contour mapping, and adjustments necessary to define flows to key nodes and locations of interest.

Through discussions with CH, it has been determined that a threshold of approximately 50 ha per subcatchment has been applied in other watershed scale studies. This suggests the possibility for additional refinement as to the number and sizes of the subcatchments in each watershed. Initial subcatchments have been reviewed and refined based upon the aerial imagery, DEM/contours, storm sewer infrastructure, stormwater management facility locations (for possible hydrologic model inclusion) and watercourse mapping to determine an appropriate level of discretization.

Focus for refinement has been placed on subcatchments of less than 20 ha, to determine if these areas can be reasonably combined with neighboring subcatchments, as well as detailed refinement for subcatchments draining directly to or containing reaches which are to be mapped as part of the subsequent hydraulic modelling effort (i.e., downstream of headwaters). Refinements for these areas have included splitting subcatchments at internal drainage boundaries such as roadways and crossings, which may present the need for additional flow nodes as part of future hydraulic modelling initiatives.

Table 3.1 provides a summary of the refined subcatchments generated by WSP, which has resulted in subcatchments at an average size of 22 to 29 ha, with the standard deviation about 16 to 24 ha.

Table 3.1. Subcatchment Boundary Characteristics

WATERSHED	TOTAL NUMBER OF SUBCATCHMENTS	MINIMUM SUBCATCHMENT SIZE (ha)	MAXIMUM SUBCATCHMENT SIZE (ha)	AVERAGE SUBCATCHMENT SIZE (ha)	STANDARD DEVIATION (ha)
Tuck Creek	49	1.26	70.33	22.53	+/-15.99
Shoreacres Creek	58	4.5	85.6	23.8	+/-17.10
Appleby Creek	46	1.12	71.04	26.02	+/-18.33
Sheldon Creek	64	0.48	90.19	27.63	+/-22.79

The proposed subcatchments are notably at a finer resolution compared to the previous studies. Previous studies generally had less than 40 subcatchments per watershed. Specifically, the 1997 study for Appleby Creek had 31 subcatchments, the 1997 study for Shoreacres Creek had 40 subcatchments, the 1996 study for Tuck Creek had 16 subcatchments.

It should be noted that through review of the subcatchments, a number of small catchment areas along the Lake Ontario shoreline (i.e., south of Lakeshore Road) are recommended for exclusion from the hydrologic modelling as these areas are considered to drain directly towards the lake and would not contribute to the respective creek systems. There are also some areas at the downstream extents of the watersheds that indicate storm sewer servicing which outlets directly to Lake Ontario, and not to the respective creek systems. These areas have been considered as part of the previous summary.

A further detailed summary of subcatchment boundaries is provided with respect to each of the watersheds in subsequent sections, including any areas which may include minor/major splits.

3.2 SUBCATCHMENT NAMING CONVENTION

The hydrologic modelling platform, Visual OTTHYMO Version 6 (VO6 – as discussed further in Section 3.3.1) allows input for both hydrographic number (NHYD) and a description. A 5-digit number has been applied for NHYD, including a suitable prefix to indicate watershed name plus the four digits as the descriptive name. The first digit in NHYD identifies the watershed which the subcatchment lies within. The last digit indicates the sub-area of a subcatchment, which for the current study will remain as a trailing zero (0), providing flexibility for future model refinements after the current study completion. The NHYD and descriptive name for subcatchments is described as follows:

1	0 0 1	1
Watershed	Subcatchment ID	future sub-area

TU	0 0 1	1
Watershed	Subcatchment ID	future sub-area

Table 3.2 illustrates the proposed subcatchment naming convention applied for this study.

Similarly, a FIVE-digit number was applied for flow nodes and route channel elements. The first digit identifies the type of the elements (i.e. 5xxxx represents flow nodes, 6xxxx represents route channel, 7xxxx represents route reservoir, 8xxxx represents route pipe, 9xxxx represents dual hydrograph). The last digit indicates whether there is a sub-section of the element, which for the current study will remain a trailing zero (0). The descriptive name replaces the first digit in the NHYD with an indicative letter and adds the watershed abbreviation in front. The HHYD and descriptive name for the VO elements other than subcatchments is described as follows:

5	0 0 1	1
type of element	element ID	future sub-section

TU	J	0 0 1	1
watershed	type of element	element ID	future sub-section

Table 3.3 illustrates the naming convention applied for VO elements other than subcatchments.

Table 3.2. Example of Subcatchment Naming Convention

SUBCATCHMENT NO.	VO NHYD	VO DESCRIPTIVE NAME
Tuck Creek Watershed		
1	10010	TU0010
1-5 (future sub-area)	10015 (future sub-area)	TU0015 (future sub-area)
2	10020	TU0020
2-5 (future sub-area)	10025	TU0025
...	...	...
10	10100	TU0100
10-5 (future sub-area)	10105 (future sub-area)	TU0105 (future sub-area)
...	...	...
999	19990	TU9990
999-5 (future sub-area)	19995 (future sub-area)	TU9995 (future sub-area)
Shoreares Creek Watershed		
1	20010	SA0010
1-5 (future sub-area)	20015 (future sub-area)	SA0015 (future sub-area)
2	20020	SA0020
2-5 (future sub-area)	20025 (future sub-area)	SA0025 (future sub-area)
...	...	...
10	20100	SA0100
10-5 (future sub-area)	20105 (future sub-area)	SA0105 (future sub-area)
...	...	...
999	29990	SA9990
999-5 (future sub-area)	29995 (future sub-area)	SA9995 (future sub-area)
Appleby Watershed		
1	30010	AP0010
1-5 (future sub-area)	30015 (future sub-area)	AP0015 (future sub-area)
2	30020	AP0020
2-5 (future sub-area)	30025 (future sub-area)	AP0025 (future sub-area)
...	...	...
10	30100	AP0100
10-5 (future sub-area)	30105 (future sub-area)	AP0105 (future sub-area)
...	...	...
999	39990	AP9990
999-5 (future sub-area)	39995 (future sub-area)	AP9995 (future sub-area)
Sheldon Watershed		
1	40010	SD0010
1-5 (future sub-area)	40015 (future sub-area)	SD0015 (future sub-area)
2	40020	SD0020
2-5 (future sub-area)	40025 (future sub-area)	SD0025 (future sub-area)
...	...	...
10	40100	SD0100
10-5 (future sub-area)	40105 (future sub-area)	SD0105 (future sub-area)
...	...	...
999	49990	SD9990
999-5 (future sub-area)	49995 (future sub-area)	SD9995 (future sub-area)

Table 3.3. Example of Naming Convention for VO Routing Elements

VO ROUTING ELEMENT	VO NHYD	VO DESCRIPTIVE NAME
Tuck Creek Watershed		
ADD HYD	50010 - 59990	TUJ0010 – TUJ9990
ROUTE CHANNEL	60010 - 69990	TUC0010-TUC9990
ROUTE RESERVOIR	70010 - 79990	TUSU & CH SWM ID
ROUTE PIPE	80010 - 89990	TUP0010 – TUP9990
DUHYD	90010 - 99990	TUD0010 – TUD9990
Shoreacres Creek Watershed		
ADD HYD	50010 - 59990	SAJ0010 – SAJ9990
ROUTE CHANNEL	60010 - 69990	SAC0010-SAC9990
ROUTE RESERVOIR	70010 - 79990	SASU & CH SWM ID
ROUTE PIPE	80010 - 89990	SAP0010 – SAP9990
DUHYD	90010 - 99990	SAD0010 – SAD9990
Appleby Creek Watershed		
ADD HYD	50010 - 59990	APJ0010 – APJ9990
ROUTE CHANNEL	60010 - 69990	APC0010-TUC9990
ROUTE RESERVOIR	70010 - 79990	APSU & CH SWM ID
ROUTE PIPE	80010 - 89990	APP0010 – APP9990
DUHYD	90010 - 99990	APD0010 – APD9990
Sheldon Creek Watershed		
ADD HYD	50010 - 59990	SDJ0010 – SDJ9990
ROUTE CHANNEL	60010 - 69990	SDC0010-SDC9990
ROUTE RESERVOIR	70010 - 79990	SDSU & CH SWM ID
ROUTE PIPE	80010 - 89990	SDP0010 – SDP9990
DUHYD	90010 - 99990	SDD0010 – SDD9990

3.3 GENERAL MODELLING APPROACH

Four (4) separate hydrologic models have been developed for each watershed (Tuck, Shoreacres, Appleby and Sheldon Creeks). The models have been developed using the latest version of Visual OTTHYMO (VO) (i.e., VO 6.2) which has improved stability and GIS capabilities. Compared with GAWSER or SWMHYMO which was adopted in previous studies, VO has many advantages including a graphic user interface (GUI), the capability of both watershed scale assessment and local scale assessment, and the capability of continuous simulation as well as synthetic design storms.

The following summarizes the hydrologic modelling methodology for this study:

- Urban runoff responses use STANDHYD command (imperviousness > 20%)
- Rural runoff responses use NASHYD command (imperviousness < 20%)
- ROUTE CHANNEL/MUSKINGUM CUNGE command added where hydrograph is routed through a typical channel cross section (to be generalized based on topographic data)
- ROUTE RESERVOIR command added where hydrograph is routed through a stormwater pond

- ROUTE PIPE command will be added where hydrograph is routed through longer enclosures (where applicable)
- DIVERT HYD will be added where a spilt flow exists (i.e. due to spill conditions; this may require further future iteration based on results of the hydraulic modelling)

There are three (3) options in VO for the calculation of soil infiltration loss: Horton Equation, Modified Curve Number (CN) Method, and Proportional Loss Coefficient Method. The Modified CN method is proposed for use in this study as it is a simplified approach that is well understood by practitioners and typically applied for hydrologic modelling studies. Many practitioners have advocated for the application of the CN* approach, which accounts for the fact that the CN approach is premised on an Initial Abstraction of 0.2S (S being a storage parameter), which in many cases is unrealistically high. The CN* methodology is not proposed for the current study, rather the CN values and Initial Abstraction/Depression Storage values should be viewed as calibration/validation parameters. This is discussed further in subsequent sections.

The hydrologic modelling has been simulated firstly for calibration and validation purposes. As outlined in Section 2.1, there is currently limited available calibration/validation data. Potential calibration events include the August 4^{th} 2014 storm event (however there were no flow gauges available for this storm event), flow data from adjacent systems, and flow data collected from new gauges (2 gauges installed by Conservation Halton in 2021 to support the current study, however as the rating curves were still being developed during the calibration phase of this study, flow information was not available for the current study). Potential validation datasets do however exist for adjacent watersheds. Model calibration and validation is discussed further in subsequent sections.

The calibrated and validated model has been executed for the 2 to 100-year return periods using an agreed upon design storm distribution (discussed further in subsequent sections) as well as the Regional Storm event (Hurricane Hazel).

3.4 SUBCATCHMENT PARAMETERIZATION

3.4.1 EXISTING SUBCATCHMENT PARAMETERIZATION

Hydrologic model parameterization has been undertaken in consideration of the documentation of Conservation Halton Standard Parameters provided for use in this study, along with relevant standards and guidelines. Refer to Appendix A for a complete suite of applied parameter tables specific for this study.

Subcatchment drainage area, slope, and flow length have been determined based on the topographic mapping, using GIS tools. Consideration for major/minor splits has also been included and is discussed further with respect to specific watersheds in subsequent sections.

The subcatchment total impervious coverage and directly connected impervious coverage has been determined using the existing land use mapping as well as CH's Table of standard values (CH Table 7) corresponding to each land use type. The land cover types in the Urban Burlington Land Cover layer have been categorized into the groups outlined in standard parameter CH Table 7. The existing land use conditions are presented on Drawing 1 (attached). Proposed total impervious coverage values and directly connected impervious coverage values corresponding to each land use type are included in Appendix A (refer to Tables A1a and A1b).

The soils within the study area consist of predominantly clay loam and loam, which are classified as SCS Type 'C' and Type "D" soils, exhibiting relatively low rates of infiltration and comparatively high rates of runoff. The soil mapping is presented on Drawing 2 (attached).

The Curve Number (CN) values have been determined from the land use and soils based on CH Table 3 from Conservation Halton Standard Parameters, and Chart 1.09 of the MTO Drainage Management Manual (Ministry of Transportation, 1997), based on AMC-II (normal) conditions as a starting value. In general, given that imperviousness is accounted for directly for urban areas, the CN value represents only the pervious area (i.e. typically lawns or grassed areas, or forested areas in some cases). As such, a relatively consistent CN value has been applied in these cases. For rural headwater areas, a greater range of values would occur, based on the land use in question (farmland, forest, open fields/meadows, etcetera). CN values for urban built-up areas have been estimated based on soil groups from adjacent lands. Surficial geology mapping and soil maps have been further reviewed for areas largely missing the soil group information.

CN values corresponding to each land use type are included in Appendix A (refer to Tables A2a, A2b, and A2c). Using the method for assigning CN values described in the VO6 User's Manual, the actual applied CN values herein are area weighted values which account for both pervious and impervious lands. The associated range of CN values (assuming CN includes consideration of impervious coverage/land use) is presented on Drawing 3 (attached). Notwithstanding the preceding, when impervious area is explicitly accounted for separately, the CN value represents the runoff potential from the pervious land segment only. This is discussed further in subsequent sections.

Initial abstraction (IA) values have been reviewed based on CH Table 2 from Conservation Halton Standard Parameters. Proposed IA values corresponding to each land use type are included in Appendix A (refer to Tables A3a and A3b).

Manning's Roughness Coefficient ("n") for Sheet flow have been reviewed based on CH Table 6 from Conservation Halton Standard Parameters, and the Visual OTTHYMO User's Manual Version 6.0 (CIVICA, 2019). Proposed Roughness Coefficient values corresponding to each land use type are included in Appendix A (refer to Tables A4a and A4b).

The drainage system within the existing developed areas is characterized with storm sewer drainage systems and overland flow drainage systems. For urban subcatchments where STANDHYD applies, the overland flow length (LGP) has been determined as the average length over which flows from pervious areas would travel before being intercepted by channels, sewers, or roads (ref. Visual Otthymo Reference Guide Version 6.0, Civica Infrastructure, January 2019). Overland flow length for each subcatchment has been measured based on the contour mapping and has been limited to the range between 40 and 150 m based on standard industry practice.

For rural subcatchments where NASHYD applies, the time-to-peak for the subcatchments have been calculated using the Bransby Williams Formula or Airport Equation (Equations 8.15 and 8.16 from the MTO Drainage Management Manual, 1997), based on the runoff coefficient associated with the drainage area (i.e. $C > 0.4$ or $C < 0.4$ respectively). Specifics for each subcatchment are provided on an overall watershed basis as per subsequent sections.

VO 6.2 allows importing GIS layers into the program. As such, the parameters can be edited and updated using a GIS tool outside of the VO platform, for ease of usability and tracking. GIS layers (shapefiles) have been created to represent each of the subcatchment elements (STANDHYD and NASHYD). The shapefiles that contain attributes corresponding to the parameters are stored as GIS features in VO 6.2, for record keeping and review by CH. The auto-connections among the elements through importing have been reviewed and revised in VO 6.2. WSP has verified that the attribute table data has been correctly transferred between programs.

3.4.2 FUTURE CONDITIONS SUBCATCHMENT PARAMETERIZATION

3.4.2.1 *OVERVIEW*

An initial approach to future land use conditions was discussed with the TAC at a meeting on September 8, 2021. It was generally agreed that Official Plan (OP) data will be the primary source of data, with adjustments for known infill areas (specifically the Appleby GO Major Transit Station Area). Official plan data has been provided by both the City of Burlington and Town of Oakville for this purpose. In addition, data related to the Appleby GO Major Transit Station Area has been provided by the City.

WSP has reviewed the existing land use conditions (as well as existing aerial photography) against the proposed ultimate land use condition, to identify areas where development, and an associated increase in imperviousness would be expected. Areas of expected infill and intensification have been identified in Drawing 4a (attached). Drawing 4b presents a composite overlay of existing and future development areas, including existing pervious area.

A further discussion of changes in impervious coverage and runoff potential is provided in the subsequent sections specific to each of the four (4) watersheds. The subcatchment total and directly connected impervious coverage have been updated by adding these additional areas into the base land use mapping layer; values have been developed consistent with the approach for existing land use as per CH's Table of standard values (Table 7) corresponding to each land use type. The land cover types in the Urban Burlington Land Cover layer have been categorized into the groups outlined in CH's standard parameter Table 7. Proposed total impervious coverage values and directly connected impervious coverage values corresponding to each land use type are included in Appendix A.

It is understood that the majority of the future development areas will support commercial or industrial uses. Although on-site SWM quantity controls would generally be required as part of Municipal guidelines, these controls would be private. As such, it is uncertain whether future SWM control measures could be included in the future land use simulation, as per the approach described in Section 3.5.2 and elsewhere. While there are a few minor residential infills, these areas are generally small and it is considered likely that SWM controls would similarly be private, and therefore have not been supported for hydrologic model inclusion in this study.

As evident from Drawing 4a, there are no future land use changes identified throughout the entire Tuck Watershed. The expected land use changes in other watersheds are noted in subsequent sub-sections.

3.4.2.2 *SHOREACRES CREEK*

Within the Shoreacres Creek Watershed, the open land immediately south of Highway 407 is identified as general employment (Business Corridor) in the future land use plan. As per the direction of CH (ref. e-mail Jin-Senior, January 28, 2022) the natural hazard area around the two (2) branches of Shoreacres Creek were widened from the current flood hazard limit to a more realistic 75 m corridor, roughly centred on the existing watercourses, to represent potential future changes and required setbacks. These areas have been denoted as "Forest" land cover under future land use condition.

The area adjacent to Upper Middle Road east of Walkers Line is classified as high density residential in the future land use plan but is currently undeveloped. Land use data from the City's Official Plan mapping has been employed.

As per the direction of CH (ref. e-mail to the TAC, December 21, 2021), existing land use conditions were assumed for park and school lands which were nonetheless indicated as future development lands in the City's Official Plan Mapping. This includes school property (both Sacred Heart of Jesus Catholic Elementary and Florence Meares Public School) and Tansley Woods Park. It has been assumed that these areas would remain in their current state given public ownership.

Additional currently undeveloped parcels of land have been identified through a mapping review (approximately between Mainway and Fairview Street). In addition, undeveloped parcels of land were previously identified through the Appleby GO MTSA (as per the Phase 1 Flood Hazard and Scoped Stormwater Management Assessment, Wood, August 2021). These areas were considered variously as General Employment, Business Corridor, and Urban Corridor based on OP mapping information. Minor adjustments to the property along Morris Drive were made to consider the existing creek block and floodplain extents.

3.4.2.3 *APPLEBY CREEK*

Within the Appleby Creek watershed, a minor area south of Highway 407 along Palladium Way is slated to become Business Corridor in the future land use plan.

The land between Sarazen Drive and Clubview Drive (immediately north of Taywood Park) has been noted as a potential future High Density Residential infill and has been assessed as such in this study (refer to Drawing 4a). Notwithstanding it is noted that in the City of Burlington Official Plan this area is designated as Residential – Low Density. The exact form of the future development would require further review with the City of Burlington to confirm; the assumed land use for this area within the current study is for watershed-scale hydrologic impact assessment purposes only.

The land at the corner of Appleby Line and Mainway (1309 Appleby Line) is currently undeveloped but has been suggested to be Mixed Use Commercial under future land use conditions (as per CH and City of Burlington direction); refer to Drawing 4a. It is noted however that this area is identified as “Uptown Business Corridor Employment” and “Uptown Residential Medium Density” in the City of Burlington Official Plan. As noted, the assumed land use for this area within the current study is for watershed-scale hydrologic impact assessment purposes only. As the east branch of Appleby Creek traverses this property, the current floodplain limits (prior to the current study) were used to delineate the approximate creek block; this area was designated as a Forest Land use under future conditions.

A number of additional currently undeveloped properties were noted within this area, generally between Mainway and Fairview similar to the Shoreacres Creek watershed. Properties south of the QEW are generally located within the Appleby GO MTSA noted previously. These areas have been assumed to be either General Employment or Business Corridor land uses based on OP mapping.

It has been assumed that Sherwood Forest Park will remain undeveloped under future conditions, given that the property is owned by the City of Burlington.

3.4.2.4 *SHELDON CREEK*

Within the Sheldon Creek Watershed, Business Corridor land use is again planned for the areas south of Highway 407. A few minor residential infills have been noted based on existing land use in the vicinity of Dundas Street. The property at 4853 Thomas Alton Boulevard has been assumed to be High Density Residential as per the City’s OP. The other infills in this area have been designated as “Urban Residential” based on assumed consistency with the adjacent land use.

A large portion of undeveloped land between Upper Middle Road and Mainway (north-south) and West Sheldon Creek and Burloak Drive (west-east) is currently indicated as General Employment in the City’s Official Plan mapping. This area is referred to as the Bronte Creek Meadows lands. As per the direction of CH (ref. e-mail Mayes-TAC, December 21, 2021) it is understood that an Area Specific Plan or other supporting studies would need to be completed for this area prior to any development occurring, which would include comprehensive review of functional servicing, natural heritage systems, and hazardous lands. As such, it was considered reasonable to maintain this area under existing land use conditions in the future land use scenario, given the assumption that future on-site SWM measures would be implemented to maintain this property to existing condition flows.

A number of currently vacant/undeveloped parcels have been identified in the industrial area between Mainway and the QEW. These parcels have been assumed to become General Employment lands based on the OP mapping and current adjacent land uses.

Some additional minor parcels of undeveloped land have also been identified withing the Appleby GO MTSA, immediately west of Sheldon Creek upstream of the CNR tracks (as shown in Drawing 4a).

Future development areas are also indicated in the area between Wyecroft Road and Rebecca Street within the Town of Oakville. These areas are primarily designated as Business Employment. It should be noted that many of these areas would be serviced by Town-owned SWM facilities (as per Drawing 5) which have been considered appropriate for hydrologic model inclusion based on further review (reference Appendix B). As such, the impact of development from these areas under future conditions is expected to be reduced as compared to other areas.

The area contributing to Pond 823 has been revised under future conditions. Under this scenario, subcatchment SD0800 has been split into two (2) subcatchments to reflect the area contributing to the pond and the area that continues to drain to the confluence point downstream of Pond 823.

3.5 ROUTING ELEMENTS

3.5.1 CHANNEL ROUTING ELEMENTS

A typical cross-section of ROUTE CHANNEL has been determined based on the LiDAR data. Typical cross sections have been manually inputted in the format of paired distance-elevation dataset. The paired datasets have been determined from LiDAR and managed using a spreadsheet approach. The spreadsheet records the paired dataset and corresponding HYHD/descriptive name of the ROUTE CHANNEL commands. Cross-sections have been simplified to ensure that there are less than 20 points, consistent with VO requirements.

The roughness coefficient (Manning's n) of 0.035 have been applied for the main channel and 0.08 for the overbanks of the hydrologic routing elements. Specific lengths and slopes for each particular section have also been determined from the available LiDAR data, within the channel location.

3.5.2 RESERVOIR ROUTING ELEMENTS

Based on the initial data provided by Conservation Halton, thirty-one (31) stormwater management facilities (SWMFs) are located within the study area. Based on the long-list of 31 SWMFs currently noted:

- None are located within the Tuck Creek watershed;
- Seven (7) are within Shoreacres Creek watershed, and three (3) off-line quantity control facilities have been considered for inclusion in the hydrologic modelling for 2 to 100-year storm events;
- Five (5) are within Appleby Creek watershed, and one (1) off-line quantity control facility has been considered for inclusion in the hydrologic modelling for 2 to 100-year storm events;
- Nineteen (19) are within Sheldon Creek watershed, and six (6) quantity control facilities (including four (4) off-line facilities and two (2) on-line facilities) have been considered for inclusion in the hydrologic modelling for the 2 to 100-year storm events. One (1) of these ponds is also proposed for inclusion in the hydrologic modelling for the Regional Storm.

The locations of the above-noted quantity control facilities are presented in Drawing 5.

Notwithstanding, not all SWMFs are designed to provide quantity control. SWMFs were screened by Conservation Halton in consultation with the study Technical Advisory Committee (TAC) to identify potential low risk facilities designed to provide quantity control for the 1:100-year storm or Regional Storm, and to confirm the storage-discharge curves associated with each SWMF's final design. Further screening incorporated municipal screening of ownership and maintenance/inspection records, resulting in a short list of 10 SWMFs.

This short-list was further reviewed through a desktop level engineering assessment by WSP (refer to Appendix B for a full copy of that report). The assessment involved a more rigorous analysis to confirm screened facilities would have a limited risk of failure under a regulatory event (to the limitations of the scope of the assessment), confirm refined SWMF rating curves as required based on updated information, and confirm whether or not the short-listed SWMF is appropriate for inclusion in the hydrologic modelling. Subsequently, municipal partners were requested to formalize their support for the inclusion of the subject SWMFs and subject rating curves, as documented through signed letters included in the Appendix of that report, which confirm that the SWMFs are being inspected and maintained in keeping with industry best practices.

As documented further in the SWM Pond Evaluation Report (refer to Appendix B), of the short list of ten (10) quantity control facilities, the majority (nine (9)) have been included in the hydrologic modelling. The exception is Pond 808, located within the Sheldon Creek watershed, which was indicated as spilling for the 100-year event, with spills not expected to return to the same downstream receiver (spill to be conveyed westerly along Upper Middle Road). Pond 808 may however be considered for inclusion for lesser storm events (2 through 50-year storm events). However, for the purposes of the current study, Pond 808 has been excluded for the simulation of all events, including the 2–50-year storm events. The potential to include Pond 808 for these and other storm events should be considered as part of future study.

It is noted that one (1) SWM facility (Pond 823) within the Sheldon Creek watershed has been included in the hydrologic modelling of the Regional Storm Event given the originally approved design intent (refer to Appendix B, and also as per December 1, 2021 TAC meeting). All other stormwater management facilities (i.e. all those other than Pond 823) have been removed from the hydrologic model for the simulation of the Regional Storm.

3.6 POTENTIAL METHODS TO CALIBRATE AND VALIDATE THE MODEL

3.6.1 DATA AVAILABILITY

The hydrologic modelling has been simulated firstly for calibration and validation purposes based on current conditions. There is currently limited available calibration/validation data. While two (2) new permanent flow gauges were installed in Shoreacres Creek and Sheldon Creek over the course of this study, the available data is still limited to water level information as stage-discharge rating curves are being developed and refined. Calibration to the August 4th 2014 storm event would be ideal given it represents an observed high flow event, however there were no active flow gauges available during this storm event.

In the 1996 Tuck Creek Erosion Control Study, the hydrologic model was calibrated to an event on 5 October 1995 at Spruce Avenue, using surveyed high-water marks and stream cross sections immediately upstream and downstream of Spruce Avenue. The "observed" peak flow which could result in the differences in upstream and downstream water levels was determined using the HEC-2 hydraulic model for Tuck Creek. A similar approach was applied for the August 4th 2014 storm event in absence of available observed flow data.

During the calibration stage of this study, limited data was available from the Sheldon Creek system. The new Sheldon Creek gauge was installed in the fall of 2021 and has water level data only, as a rating curve has not yet been well established. However, available water level data was applied to support a general understanding of watershed response timing, however given the limited data, calibration on this basis was considered inappropriate. In addition, the magnitude of observed storm events and flows is considered insufficient for application to the current study, given the focus on major flood events.

Recognizing the data limitations, Conservation Halton suggested that model validation be completed at the global scale by using flow records from nearby watersheds, including a long-standing WSC gauge at Fourteen Mile Creek and a CH gauge installed since September 2018, near the downstream limit of the Hager Rambo Diversion Channel. CH gauge data is also available from the Morrison-Wedgewood diversion channel within the Town of Oakville.

In addition to the preceding, WSP has a database of simulated 100-year and Regional Storm flow rates from other studies across Southern Ontario, which have been normalized by area. This includes data for the Morrison/Wedgwood and Grindstone watersheds provided by CH for use in this study. The results are presented in Table 3.4; plots of the normalized flows are presented in Figures 3.2 and 3.3.

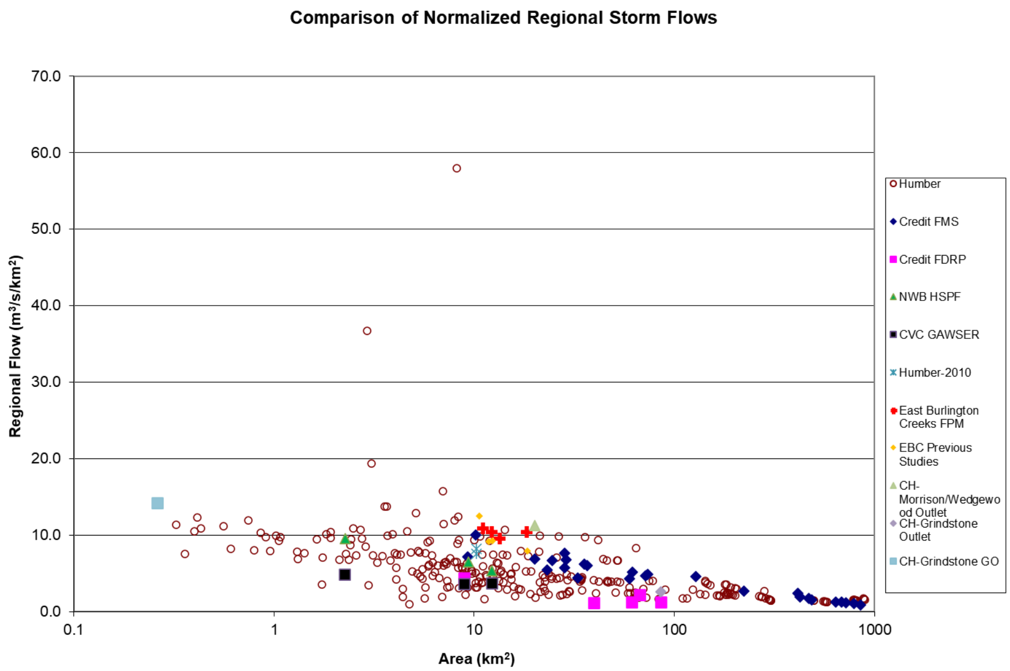


Figure 3.2. Comparison of Normalized Regional Storm Unitary Peak Flows

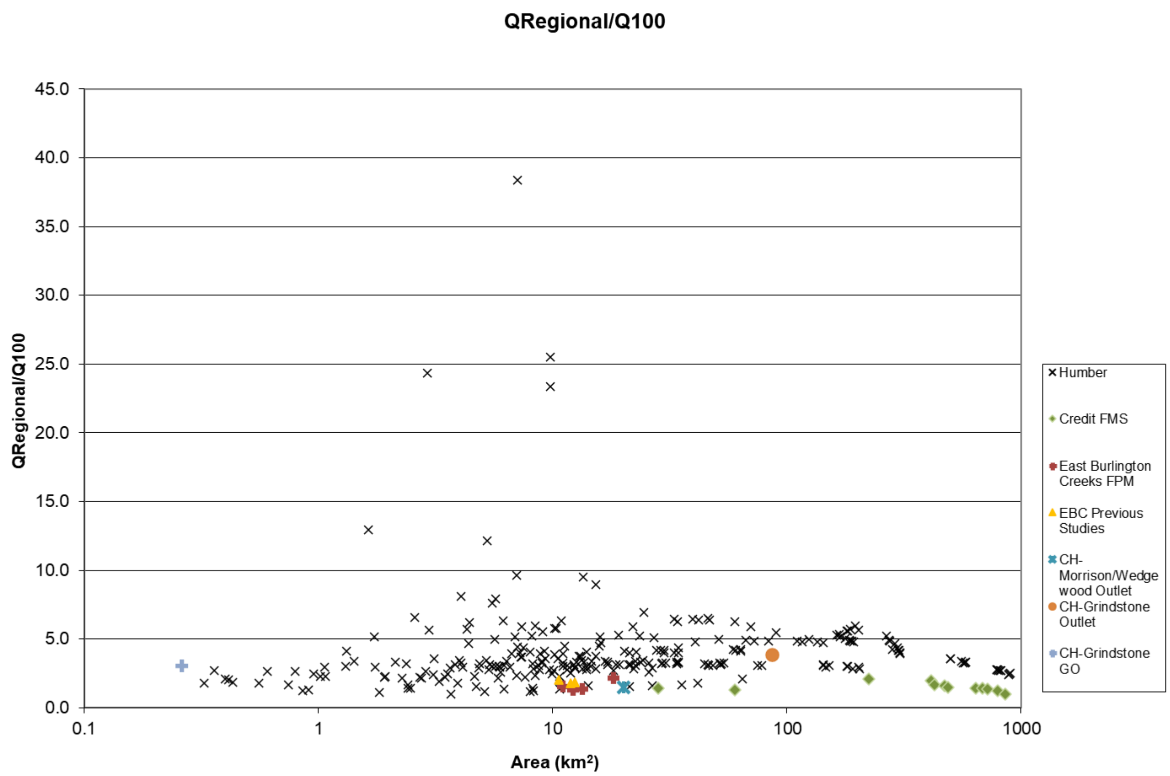


Figure 3.3. Comparison of Normalized Regional Storm Peak Flow over 100 Year Peak Flow

Table 3.4. Simulated Unitary Peak Flows from Previous Studies

LAND USE	LOCATION	AREA (ha)	UNITARY PEAK FLOW (m^3/s)		$Q_{Regional}/Q_{100}$
			100-YEAR	REGIONAL	
Rural	North Waterdown	467	0.023	0.090	3.91
Rural	Sixteen Mile Creek	444	0.019	0.075	3.95
Rural+Urban	Red Hill Creek	6,800	0.026	0.069	2.65
Rural	Stoney (Escarp.)	1,873	0.022	0.073	3.32
Rural	Battlefield (Escarp.)	487	0.022	0.073	3.32
Rural+Urban	Stoney (Outlet)	3,090	0.020	0.063	3.15
Rural+Urban	Morrison/Wedgewood Diversion Channel outlet	2,008	0.076	0.112	1.48
Rural	Grindstone Outlet	8,593	0.007	0.026	3.88
Rural+Urban	Grindstone Aldershot GO	26	0.047	0.142	3.04

Data from comparable locations, both in terms of location (proximity to the current study area) and land use (i.e. primarily urban) are considered more appropriate for the comparison. The preceding has been reviewed further for each of the four (4) watersheds in subsequent sections.

3.6.2 METHODOLOGY

The calibration/validation has focused on achieving a reasonable correlation between the simulated and observed results. Key metrics have included peak flow, runoff volume, and time to peak. A brief sensitivity analysis has been completed to determine the most sensitive parameters, which informs the subsequent calibration efforts. The primary parameters considered for calibration included CN values, Initial Abstraction (IA), flow length, and time of concentration (for rural catchments that employ the NASHYD routine). Conservation Halton has advised that total imperviousness (TIMP) is not considered an appropriated calibration parameter as it can be obtained from the detailed land use layer. Directly connected imperviousness (XIMP) may be considered as a calibration parameter.

In general, model calibration should target matching hydrograph shape and peak timing, while matching runoff volume within +20% to -10% of observed volumes and peak flows to within +25% to -15 % of observed flows. Analyses should be presented with simulated vs. observed hydrograph plots and numerical analysis summaries for each event. Aggregate statistics for all calibration events should be presented on a scatter plot with a trendline fit indicated to demonstrate the results.

Notwithstanding the preceding, it has been confirmed that there is insufficient watershed specific data to undertake a model calibration. Therefore, the methodology has focused upon a model validation, based on a comparison to available data from adjacent watersheds.

CH staff undertook a review of available monitoring data and generated a suite of suggested validation events (October 15, 2021). Data from the May 25 and Oct 27, 2019 events were suggested, as they have been applied by CH on previous studies. Consideration for more recent events, such as the August 26, 2021 event, was also suggested.

Model validation has been completed on a global basis, based on the simulated 100-year and Regional flow rates for each watershed, normalized by area. Peak flows at key nodes for each of the four (4) watersheds has also been compared to results from previous hydrologic modelling for information purposes and to assess the overall magnitude of change.

Index flood and regional flood frequency approaches were considered but ultimately not advanced as part of the current study.

Further details on model validation are provided in subsequent sections with respect to each of the four (4) watersheds.

3.7 EVENT RAINFALL

3.7.1 IDF PARAMETERS

City of Burlington IDF Parameters

The City of Burlington's previously approved rainfall Intensity-Duration-Frequency (IDF) parameters were adopted in 1999, as an update to the City's original Storm Drainage Manual (1977). These IDF values were based on the Hamilton RBG rain gauge for a period of record from 1964 to 1990 (26 years).

As part of the 2012 Hydrology Update for Tuck Creek (ref. "Tuck Creek Erosion Control Municipal Class Environmental Assessment Final Report", Aquafor Beech, June 2012), an IDF from June of 1998 is referenced. It is unclear what IDF data were applied for other previously hydrologic studies for the subject watersheds.

Environment Canada continuously issues updated IDF values based on an extended period of record as validated data becomes available. As of August 2021, available data for the Hamilton RBG gauge extends to 2017 (54 years). For comparison purposes, the rainfall statistics from the previously approved City IDF dataset (1964 to 1990) to the most current (up to 2017) are presented in Table 3.5.

Table 3.5. Comparison of City of Burlington Rainfall Depths (mm)

DURATION (HOURS)	FREQUENCY (YEARS)	1999 APPROVED (mm)	2017 UPDATE (mm)	DIFFERENCE (mm)
6	5	48.7	49.2	+0.5
6	100	85.9	85.2	-0.7
12	5	55.2	57.6	+2.4
12	100	92.1	98.4	+6.3

The results presented in Table 3.5 indicate that the results are relatively consistent, however the 2017 update generally results in greater rainfall depths, as would be expected.

The City of Burlington recently (2020) updated its rainfall Intensity-Duration-Frequency (IDF) curves by applying a 15% increase to the Hamilton RBG rain gauge. The IDF relationships were adjusted to account for climate change scenarios (ref. Stormwater Management Design Guidelines. City of Burlington, May 2020). The IDF values are summarized in Table 3.6.

Table 3.6. City of Burlington IDF Relationships (2020 – Climate Change Adjusted)

RETURN PERIOD	A	B	C
2 Year	681.52	6	0.780
5 Year	802.04	5	0.764
10 Year	918.28	5	0.763
25 Year	1065.95	5	0.762
50 Year	1172.34	5	0.761
100 Year	1281.34	5	0.761

$I = A/(t_d + B)^c$

Where: I is rainfall intensity (mm/hr); and

t_d is the time duration (minutes)

A comparison between the IDF relationships with and without climate change adjustment is presented in Table 3.7.

Table 3.7. City of Burlington IDF Relationships Comparison (mm/hr)

DURATION	2 YEAR	5 YEAR	10 YEAR	25 YEAR	50 YEAR	100 YEAR
Environment Canada (1964-2017) - Existing						
5 min	96.6	123.3	141.0	163.3	179.9	196.3
10 min	69.5	88.6	101.3	117.2	129.1	140.9
15 min	56.1	71.9	82.3	95.4	105.1	114.8
30 min	35.9	45.6	52.1	60.2	66.2	72.2
1 hr	21.8	27.1	30.7	35.1	38.5	41.8
2 hr	13.7	17.5	20.0	23.1	25.5	27.8
6 hr	5.9	8.2	9.6	11.5	12.8	14.2
12 hr	3.6	4.8	5.6	6.7	7.4	8.2
24 hr	2.1	2.7	3.1	3.7	4.1	4.4
City of Burlington (2020) – Climate Change Adjusted						
5 min	105.0	138.1	158.5	184.4	203.3	222.2
10 min	78.4	101.3	116.3	135.4	149.3	163.2
15 min	63.4	81.3	93.4	108.7	119.9	131.1
30 min	41.6	53.0	60.9	71.0	78.3	85.6
1 hr	26.0	33.0	38.0	44.3	48.9	53.5
2 hr	15.7	20.1	23.1	26.9	29.7	32.5
6 hr	6.8	8.8	10.2	11.9	13.2	14.4
12 hr	4.0	5.2	6.0	7.0	7.8	8.5
24 hr	2.3	3.1	3.6	4.2	4.6	5.0

Town of Oakville IDF Parameters

According to the Town of Oakville Development Engineering Procedures and Guidelines, the Toronto Bloor Street station which has continuous rainfall data for the last 50 years shall be used in Oakville. No climate change adjustment has been incorporated into the IDF relationships. The IDF relationships are summarized in Table 3.8, a comparison to the City of Burlington criteria is presented in Table 3.9.

Table 3.8. Town of Oakville IDF Relationships – Based on AES Toronto (Bloor Street Gauge)

EVENT	A	B	C
2 Year	725	4.8	0.808
5 Year	1170	5.8	0.843
10 Year	1400	5.8	0.848
25 Year	1680	5.6	0.851
50 Year	1960	5.8	0.861
100 Year	2150	5.7	0.861
$I = A/(t_d + B)^c$ Where: I is rainfall intensity (mm/hr); and t_d is the time duration (minutes)			

Table 3.9. Comparison of Rainfall Depths (mm)

DURATION (HOURS)	FREQUENCY (YEARS)	2017 BURLINGTON (mm)	OAKVILLE (mm)	DIFFERENCE (mm)
6	5	49.2	48.5	-0.7
6	100	85.2	80.1	-5.1
12	5	57.6	54.4	-3.2
12	100	98.4	88.8	-9.6

The results presented in Table 3.9 indicate that the City of Burlington IDF values are consistently higher than those for the Town of Oakville.

Summary

For consistency (and for conservativeness given generated rainfall depths), the City of Burlington IDF data referred to previously has been applied for the entire study area, including the portions of Sheldon Creek which lie within the Town of Oakville.

In this study, the Climate Change Adjusted IDF Parameters listed in Table 3.6 have been applied for the Future Conditions modelling and the 2017 Environment Canada IDF values have been applied for the existing conditions modelling. This recognizes the direction in the Provincial Policy Statement 2020 and the 2002 Technical Guide. Section 3.1.3 of the Provincial Policy Statement, 2020 states "Planning authorities shall prepare for the impacts of a changing climate that may increase the risk associated with natural hazards" while on page 50 Section D Flow Computations of the 2002 Technical Guide - River and Stream Systems: Flooding Hazard Limit requires consideration of future land use conditions preferably extending 20 years into the future O.Reg 162/06 (Halton Region Conservation Authority: Regulation of Development, Interference with Wetlands and Alterations to Shorelines and Watercourses) also provides a definition of the 100-year flood which is consistent with the preceding interpretations.

3.7.2 DESIGN STORM DISTRIBUTIONS

According to the City of Burlington Stormwater Management Design Guidelines (City of Burlington, 2020), the rainfall distributions for event-based hydrologic modelling should be compatible and consistent with those which were applied in a subwatershed study (SWS), master drainage plan (MDP), or master environmental servicing plan (MESP) study.

Although 3-hour Chicago design storms were applied in the previous studies for Tuck Creek Watershed, Shoreacres Creek Watershed, and Appleby Creek Watershed, the latest City of Burlington Stormwater Management Guideline (May 2020) describes that the acceptable distributions for the City include 6, 12, and 24-hour duration Chicago Design Storms or SCS Type II Design Storms. According to the guidelines, where an event-based model has been applied to calculate volumetric storage requirements, a 24-hour duration storm event shall be included as part of the verification process.

It was noted during the start-up meeting that the Town of Oakville specifies the 24-hour Chicago distribution for SWM pond design, and that this rainfall distribution was applied in past flood risk mapping studies where rainfall distribution was applied. Conservation Halton has noted that the 24-hour Chicago distribution was applied to past flood mapping studies only where SWM ponds were approved and designed based on the 24-hour Chicago rainfall distribution, such as in the Morrison Wedgewood Flood Risk Mapping and Spill Quantification Study (Morrison Hershfield, 2020). In the Grindstone Flood Hazard Mapping Study (Matrix, 2020), however, the Atmospheric Environment Services (AES) distribution best described the flow frequency data analysis and was therefore applied to support modelling of the flood hazard.

Atmospheric Environment Services (AES) Canada also specifies design storm distributions which have been applied for hydrologic studies in Ontario. Distributions for the 1, 6 and 12-hour durations are available, with different temporal probabilities which affect the peaking characteristics of the distribution.

A brief sensitivity analysis is considered to be required to select the most appropriate distribution and evaluate the results. This is discussed further in subsequent sections with respect to specific watersheds, although the results are generally common across all of the watersheds, given the commonalities in watershed size, shape, and land cover.

The Chicago, SCS, and AES design distributions of various lengths have been tested. The simulated peak flows resulting from the candidate design storm distributions at selected locations are presented in Appendix C. For the purposes of this assessment, no areal reduction factors (ARFs) have been applied as the comparison is focused on the differences in the distributions only, and any differences due to ARFs would be common to all distributions. The results are based on "existing" IDF (i.e. Environment Canada data (1964 to 2017)), as per Table 3.6.

The sensitivity analysis indicates that the 12-hour SCS design storm would generate the largest peak flow rates throughout all four (4) watersheds for the 5-year and 100-year return period events. Also, the SCS distribution generally governs over the Chicago distribution, and in particular over the AES distributions, which generate notably lower peak flows. Based on the preceding, the 12-hour SCS distribution has been selected to determine the 2 to 100-year return period flows for all four (4) watersheds.

CH has noted that as per the MNR's 2002 Technical Guidelines, an areal reduction factor (ARF) should be applied for design storm distributions where the area is larger than 25 km^2, either based on total upstream contributing drainage area or an equivalent circular area. This is consistent with the approach to aerial reduction factors applied for the Regional Storm Event (i.e. Hurricane Hazel). Areal reduction factors are reviewed further in Section 3.7.4.

All design storms have been generated at a time step of 5 minutes for VO6 input.

For the simulation of Design Storm Events, SCS Curve Number (CN) values will represent the Antecedent Moisture Conditions (AMC) II (normal) and will be used for the 2-year through 100-year storm events, and stormwater management facilities confirmed for inclusion will be represented within the models.

3.7.3 HISTORIC STORM EVENTS

Regional Storm Event

For the Regional Storm event, two scenarios have been considered to determine which yields the most conservative results:

— CN values at AMC II (normal) conditions, and the full 48-hour version of Hurricane Hazel (36-hour pre-wetting period and 12-hour primary storm) applied

— CN values converted to AMC III (saturated) conditions, and the 12-hour version of Hurricane Hazel applied

It should be noted that in previous studies, the 12-hour Regional Storm was applied for the Tuck Creek Watershed and Sheldon Creek Watershed, while the 48-hour Regional Storm was applied for Shoreacres Creek Watershed and Appleby Creek Watershed. The 12-hour Hurricane Hazel Distribution has been obtained from Ontario Regulation 162/06 and presented in Table 3.10.

Table 3.10. Hurricane Hazel Distribution

TIME PERIOD	DEPTH (mm)	PERCENT OF 12 HOUR
First 36 hours	73	
37th hour	6	3
38th hour	4	2
39th hour	6	3
40th hour	13	6
41st hour	17	8
42nd hour	13	6
43rd hour	23	11
44th hour	13	6
45th hour	13	6
46th hour	53	25
47th hour	38	18
48th hour	13	6
Total	**285**	**100**

August 4th, 2014 Event

The August 4th, 2014 storm event hyetograph is illustrated in Figure 3.3 and Table 3.11. The hyetograph represents a storm duration of 6.5 hours and 196 mm of total rainfall. Two (2) distinct intense periods of rainfall can be observed at 30 minutes and again at 180 minutes, with the peak intensity reaching 125 mm/hr (ref. Urban-Area Flood Vulnerability, Prioritization and Mitigation Study. Amec Foster Wheeler, 2017).

Figure 3.4. August 4th, 2014 Storm Event Maximum Cell Hyetograph (Source: Conservation Halton, 2015)

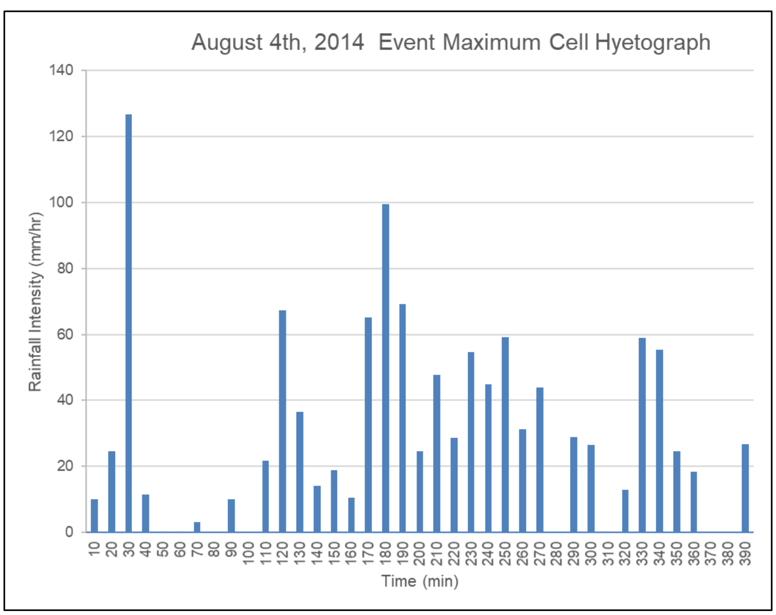


Table 3.11. August 4, 2014 Max Cell Rainfall Distribution

DURATION (min)	INTENSITY (mm/hr)
10	9.95
20	24.66
30	126.75
40	11.49
50	0.11
60	0.29
70	3.06
80	0.00
90	10.05
100	0.00
110	21.84
120	67.37
130	36.48
140	14.14
150	18.84
160	10.59
170	65.05
180	99.51
190	69.19
200	24.62
210	47.73
220	28.62
230	54.66
240	44.98
250	59.09
260	31.22
270	43.87
280	0.00
290	28.99
300	26.45
310	0.00
320	12.85
330	58.92
340	55.36
350	24.56
360	18.48
370	0.00
380	0.00
390	26.84

3.7.4 AREAL REDUCTION FACTORS

As per the MNR's 2002 Technical Guide (River & Stream Systems: Flood Hazard Limit) an areal reduction factor should be calculated for watersheds with areas greater than 25 km^2. Area calculations may be based on either upstream contributions or the equivalent circular area. For the current study, the equivalent circular area has been proposed. Straight line lengths based on contributing watershed areas to points of interest have been measured and applied to determine the associated circular area and need for the reduction factors. This is discussed in greater detail for specific watersheds in subsequent sections.

As per the MNR's 2002 Technical Guide, different reduction factors are specified for design events as compared to the Regional Event (Figure D-6 applies for design events as compared to Figure D-3 for Hurricane Hazel). However based on a review by WSP, the differences for the subject drainage areas are considered extremely minor. As such, and for overall consistency, the same reduction factors has been applied for design storm events as well as the Regional Storm Event. This approach was confirmed with CH staff (ref. e-mail Jin-Senior, November 9, 2021).

The uniform distribution of the August 14, 2014 event (as per Table 3.7) has been applied. Areal reduction factors have not been applied for the simulation of this event, as described further in subsequent sections.

In general, where spills are noted, the appropriate areal reduction factor is applied to the spill hydrograph at its point of occurrence. Additional drainage areas associated with the spill are not subsequently applied when addressing the areal reduction factor to be applied in the receiving watershed.

Reduction values presented in Table 3.11 have been obtained from Conservation Halton Guidelines for Stormwater Management Engineering Submissions, Version 1.0 (November 2021). Conservation Halton have confirmed that the values are consistent with Table 2 from Ontario Regulation 162/06 and the MNR's 2002 Natural Hazards Technical Guide – Flood Hazard. In general, the hourly rainfall values are multiplied by the percentage shown for circular drainage areas larger than 25 km^2. This may require multiple model runs for the various versions of the areal reduction factors, with results from identified flow nodes extracted for the applicable reduction factor accordingly.

Table 3.12. Areal Reduction Factors

CIRCULAR DRAINAGE AREA (km^2)	TOTAL PERCENTAGE OF RAINFALL
0-25	100.0
26-45	99.2
46-65	98.2
66-90	97.1
91-115	96.3
116-140	95.4
141-165	94.8
166-195	94.2
196-220	93.5
221-245	92.7
246-270	92.0
271-450	89.4
451-575	86.7
576-700	84.0
701-850	82.4
851-1000	80.8

4 TUCK CREEK

4.1 MODEL DEVELOPMENT

4.1.1 SUBCATCHMENT BOUNDARIES

The Tuck Creek watershed has a total drainage area of approximately 11.05 km^2. Subcatchment boundaries for Tuck Creek have been developed using the approach summarized in Section 3.1.2. Statistics are presented in Table 4.1. The boundaries are presented graphically in Drawing 6a (attached).

Table 4.1. Proposed Subcatchment Boundaries for Tuck Creek

WATERSHED	TOTAL NUMBER OF SUBCATCHMENTS	MINIMUM SUBCATCHMENT SIZE (ha)	MAXIMUM SUBCATCHMENT SIZE (ha)	AVERAGE SUBCATCHMENT SIZE (ha)	STANDARD DEVIATION (ha)
Tuck Creek	49	1.26	70.33	22.53	+/-15.99

Contributing drainage areas at key locations have been compared with the 2012 Tuck Creek Erosion Control Class EA study. The comparison is summarized in Table 4.2. A comparison to previous drainage boundaries is presented in Drawing 6b (attached).

Table 4.2. Comparison of Drainage Areas with Previous Study for Tuck Creek

LOCATION	2012 TUCK CREEK EROSION CONTROL CLASS EA		2023 EAST BURLINGTON CREEKS FPM		DRAINAGE AREA DIFFERENCES (2023 VS. 2012)	
	NODE	DRAINAGE AREA (ha)	NODE	DRAINAGE AREA (ha)	ABSOLUTE (ha)	%
West Branch @ Headon Forest Dr.	T1	187	TUJ0100	207	+19.4	+10.3%
East Branch @ Headon Forest Dr.	T2	206	TUJ0180	232	+26.4	+12.9%
Confluence of East and West Branches	T3	495	TUJ0210	464	-30.6	-6.2%
Upper Middle Rd	T4	593	TUJ0260	617	+23.7	+4.0%
South of Upper Middle Rd	T5	605	TUJ0270	633	+28.3	+4.7%
Palmer Rd	T6	671	TUJ0280	656	-15.3	-2.3%
QEW	T7	733	TUJ0330	744	+11.0	+1.5%
CNR Oakville	T8	843	TUJ0370	862	+18.4	+2.2%
New St.	T9	993	TUJ0430	1,025	+31.6	+3.2%
Lake Ontario Outfall	T10	1,058	TUJ0490	1,105	+47.0	+4.4%

In general, the drainage areas are comparable to the previous study, with the differences less than 4%. The differences in drainage areas are relatively larger at Headon Forest Drive, the confluence of East and West Branches, and South of Upper Middle Road. It is also notable that previous studies assessed each watershed individually unlike the current study which assesses all four (4) watersheds together; differences are evident on the overall boundaries as per Drawing 6b.

The reduction in drainage area at the confluence of the east and west branches (T3) is primarily attributable to the fact that previous studies included the area of subcatchment TU0200 as contributing to this junction node. The difference reported at Palmer Road (T6) may also reflect differences in how subcatchment boundaries have been delineated, as the 2012 study bounded this area at Mainway rather than Palmer Road.

4.1.2 SUBCATCHMENT PARAMETERIZATION

Based on the subcatchment delineation, subcatchment parameterization has been established following the approach described in Section 3.4. A summary of the uncalibrated subcatchment parameters for Tuck Creek is included in Appendix E.

Subcatchment Slope

The surface slopes within the Tuck Creek Watershed tend to be moderate between 1 and 3%. The undeveloped areas north of Highway 407 are slightly steeper with the average slope greater than 4%.

Impervious Coverage

The land use conditions north of Dundas Street are primarily agricultural lands, open space, and forest, with rural residential areas distributed along major roads. The areas south of Dundas Street are largely developed and the land use conditions are a mix of urban residential and high density residential areas, high impervious areas, institutional areas, industrial areas, commercial areas, parks and open lands, as well as dispersed forests. A comparison to the previous study is presented in Table 4.3.

Table 4.3. Comparison of Modelled Imperviousness to Previous Studies for Tuck Creek

LOCATION	2012 EROSION CONTROL EA	2023 EBC FPM (EXISTING CONDITIONS)	DIFFERENCE
East Branch	13.4%	13.1%	-0.3%
West Branch	23.1%	37.6%	+14.5%
Total	35.0%	49.0%	+14.0%

As evident from Table 4.3, impervious coverage for the east branch is generally comparable, however both the west branch and overall watershed indicate increases on the order of 14%. The rationale for this difference has not been assessed further, however it is considered this may be attributable to changes in industry practice related to typical land use coverage assumptions. Given the highly urbanized nature of the Tuck Creek watershed below Highway 407ETR, the currently proposed values are generally considered more appropriate and reasonable.

Infiltration

The soils within the Tuck Creek Watershed consist largely of Clay Loam (62%), of which 6% is in the rocky phase. The remaining portion of the soils within the watershed consists of loam (21%), sandy loam (11%), and urban built-up areas (3%). Overall, the soils are largely classified as SCS Type 'C' and Type "D" soils, exhibiting low permeability and low infiltration potential with high potential for generating runoff.

SCS Curve Numbers have been applied on the basis of representative values for the pervious land segment. In particular for urbanized areas that utilized the STANDHYD routine, given that impervious coverage is accounted for separately, the CN value represents the solely pervious land segment. As an example, for a residential area, the SCS CN represents the grassed/lawn areas based on the applicable soils.

4.1.3 SWM FACILITIES

No approved quantity control SWM ponds have been identified within the Tuck Creek Watershed.

4.1.4 MAJOR/MINOR SPLIT

Subcatchment TU0470 has a modelled major/minor split at Lakeshore Road. Based on available contour mapping, the City's storm sewer database, and available as-built drawings, minor flow from Subcatchment TU0470 would be conveyed through the 900 mm diameter storm sewer along Walkers Line and discharge directly to Lake Ontario. The maximum capacity of the pipe has been determined to be 0.981 m^3/s using Manning's equation. This value has been applied to split the minor and major components of the hydrograph accordingly. A maximum flow of 0.981 m^3/s would discharge to the Lake. Flows that exceed 0.981 m^3/s will travel west and discharge to Tuck Creek at Lakeshore Road through a combination of sewer and overland flow.

The major/minor split has been represented by DUHYD TUD0010 (NHYD 90100). One inlet with a maximum capture of 0.981 m^3/s has been assumed as the minor flow that would discharge to the Lake and be excluded from the system. Flow which exceeds 0.981 m^3/s is considered as major flow and would contribute to Tuck Creek Node TUJ0470 at Lakeshore Road.

Calculations of the split flows and as-built drawings are included in Appendix E.

4.1.5 AREAL REDUCTION FACTORS

The limits of areal reduction factors (ARFs) for Tuck Watershed are presented on Drawing 7 (attached). ARFs have been calculated consistent with the methodology described in Section 3.7.4. As noted in Section 3.7.4, it has been agreed that the same ARFs are to be applied for Regional Storm event and design storm events. To summarize the findings for the Tuck Creek watershed and ARFs to be applied:

- In general, areas north of Mainway are within the 0 to 25 km^2 circular area and would therefore not require an ARF.
- Areas between Mainway and Fairview Street are within the 26 to 45 km^2 circular area with an ARF of 99.2% to be applied.
- Areas between Fairview Street and New Street are within the 46 to 65 km^2 circular area with an ARF of 98.2% to be applied.
- Areas between New Street and outfall at Lake Ontario is within the 66 to 90 km^2 circular area with an ARF of 97.1% to be applied.

4.2 EXISTING CONDITIONS MODEL RESULTS

4.2.1 UNCALIBRATED MODEL RESULTS

Uncalibrated 100-Year Design Storm and Regional Storm Peak Flow Rates

The VO6 hydrologic model for the Tuck Creek Watershed has been executed for the 12-hour SCS 100-year design storm event (existing 1964 to 2017 IDF), the 12-hour Regional Storm event under the AMC III (saturated) soil conditions, and the 48-hour Regional Storm event under the AMC II (normal) soil conditions. The peak flows at key locations have been summarized and presented in Table 4.4. Applicable areal reduction factors (ARFs) are noted.

The results indicate that the 12-hour Regional Storm under the AMC III soil conditions and the 48 Hour Regional Storm under the AMC II conditions would generate similar peak flow rates. The governing storm event would be the 12-Hour Regional Storm under the AMC III soil conditions except for the nodes along the West Branch, where the Regional Storm and 100-year flows are approximately equal.

Table 4.4. Simulated Uncalibrated Design Storm and Regional Storm Peak Flows at Key Locations for Tuck Creek

LOCATION	ARF (%)	NODE	DRAINAGE AREA (ha)	PEAK FLOW RATE (m^3/s)		
				100 YEAR	12 HOUR REGIONAL (AMC III)	48 HOUR REGIONAL (AMC II)
East Branch						
East Trib at Guelph Line	100	TUJ0040	70	4.3	8.6	8.3
West Trib at Guelph Line	100	TUJ0130	80	6.9	10.2	9.9
West Trib at Dundas Street	100	TUJ0140	83	7.1	10.4	10.2
East Trib at Dundas Street	100	TUJ0070	123	7.9	14.7	14.2
Confluence of East and West Trib D/S of Dundas Street	100	TUJ0090	208	15.0	25.4	24.7
Headon Forest Drive	100	TUJ0180	232	15.6	27.9	27.2
U/S of Confluence	100	TUJ0190	244	16.2	29.3	28.5
West Branch						
180 m D/S of Headon Forest Drive	100	TUJ0160	146	18.6	19.1	18.7
U/S of Confluence	100	TUJ0170	207	30.9	27.6	27.2
Main Branch						
Confluence of East and West Branch	100	TUJ0210	464	38.1	55.7	54.5
Headon Road	100	TUJ0240	559	51.9	66.7	65.5
Upper Middle Road	100	TUJ0260	617	60.5	72.9	71.7
Palmer Road	100	TUJ0280	656	59.0	77.4	76.1
Mainway	100	TUJ0300	706	60.1	82.6	81.4

LOCATION	ARF (%)	NODE	DRAINAGE AREA (ha)	PEAK FLOW RATE (m^3/s)		
				100 YEAR	12 HOUR REGIONAL (AMC III)	48 HOUR REGIONAL (AMC II)
CNR - Halton	99.2	TUJ0310	719	59.5	83.4	82.1
QEW	99.2	TUJ0330	744	60.8	86.1	84.8
Harvester Road	99.2	TUJ0350	811	64.4	93.2	91.9
CNR Oakville	99.2	TUJ0370	862	66.9	98.6	97.2
Fairview Street	98.2	TUJ0390	881	66.4	99.5	98.1
Rexway Drive	98.2	TUJ0400	919	67.3	103.5	102.2
New Street	97.1	TUJ0420	1,001	72.4	110.6	109.2
Spruce Avenue	97.1	TUJ0450	1,053	75.3	116.0	114.7
Lakeshore Road	97.1	TUJ0470	1,102	78.4	120.0	118.8
Lake Ontario	97.1	TUJ0490	1,105	78.5	120.2	119.0

Comparison of Simulated Peak Flows with Previous studies

The VO6 hydrologic model has been executed for the 3 Hour Chicago storm event (100-year storm) to compare against the 2012 Tuck Creek Erosion Control Class EA (ref. Table 3.1 from that report). To maintain consistency with the 2012 study, the IDF based on the same 27 years of data recorded at the Atmospheric Environment Royal Botanical Gardens Gauge and the time to peak ratio of 0.33 has been used for the comparison. Also, ARFs have not applied in this assessment for the VO6 flow nodes to ensure a consistent comparison. The results are summarized in Tables 4.5 to 4.6.

Table 4.5. Comparison of Simulated Peak Flows for Tuck Creek at Key Locations (2012 Study)

LOCATION	2012 TUCK CREEK EROSION CONTROL CLASS EA			2023 EAST BURLINGTON CREEKS FPM - UNCALIBRATED		
	NODE	3 HOUR CHICAGO 100 YEAR (m^3/s)	12 HOUR REGIONAL (m^3/s)	NODE	3 HOUR CHICAGO 100 YEAR (m^3/s)	12 HOUR REGIONAL (m^3/s)
West Branch at Headon Forest Dr.	T1	17.7	26.5	TUJ0170	22.2	27.6
East Branch at Headon Forest Dr.	T2	13.8	28.7	TUJ0180	8.1	27.9
Confluence of East and West Branches	T3	39.6	69.0	TUJ0210	25.0	55.7
Upper Middle Rd	T4	39.1	79.7	TUJ0260	35.1	72.9
South of Upper Middle Rd	T5	39.5	81.3	TUJ0270	34.1	74.9
Palmer Rd	T6	41.4	88.7	TUJ0280	35.0	77.4
QEW	T7	44.4	96.0	TUJ0330	38.5	86.9
CNR Oakville	T8	50.5	108.4	TUJ0370	42.6	99.4
New St.	T9	61.3	125.4	TUJ0430	53.2	116.8
Lake Ontario	T10	64.3	132.1	TUJ0490	55.3	124.0

1. For the purposes of this comparison, areal reduction factors and rainfall distributions from the previous study have been maintained. Values in this table may not be consistent with values in other sections of this report.

The comparison indicates that the simulated peak flow rates generated from the current VO6 model are overall lower than the peak flow rates generated from the 2012 SWMHYMO model. The greatest differences are indicated in the upper reaches (nodes T1, T2, and T3) for the 100-year storm event, which likely reflects the sensitivity of smaller drainage areas and the more intense storm event. A larger difference is noted at T3 in particular which may reflect differences in modelling assumptions or drainage areas to this specific location. Differences further downstream (Upper Middle Road and downstream) are much less, with differences of 15% or less typically for the 100-year storm and 13% less for the Regional Storm. The updated modelling does however consistently generate lower peak flows as noted previously.

The differences in the simulated peak flows are considered attributable to the different modelling platforms, parameterization methodology, and minor differences in contributing drainage areas. Overall, the updated VO model is considered comparable to the previous study.

Table 4.6. Differences in Simulated Peak Flows for Tuck Creek at Key Locations (2023 Study vs. 2012 Study)

LOCATION	ABSOLUTE DIFFERENCE (2023 VS. 2012)			PERCENT DIFFERENCE (2023 VS. 2012)		
	NODE	3 HOUR CHICAGO 100 YEAR PEAK FLOW (m^3/s)	12 HOUR REGIONAL PEAK FLOW (m^3/s)	NODE	3 HOUR 3 HOUR CHICAGO 100 YEAR PEAK FLOW	12 HOUR REGIONAL PEAK FLOW
West Branch at Headon Forest Dr.	T1	+4.5	+1.1	TUJ0170	+25.4%	+4.2%
East Branch at Headon Forest Dr.	T2	-5.7	-0.8	TUJ0180	-41.3%	-2.8%
Confluence of East and West Branches	T3	-14.6	-13.3	TUJ0210	-36.9%	-19.3%
Upper Middle Rd	T4	-4.0	-6.8	TUJ0260	-10.2%	-8.5%
South of Upper Middle Rd	T5	-5.4	-6.4	TUJ0270	-13.7%	-7.9%
Palmer Rd	T6	-6.4	-11.3	TUJ0280	-15.5%	-12.7%
QEW	T7	-5.9	-9.1	TUJ0330	-13.3%	-9.5%
CNR Oakville	T8	-7.9	-9.0	TUJ0370	-15.6%	-8.3%
New St.	T9	-8.4	-8.6	TUJ0430	-13.2%	-6.9%
Lake Ontario	T10	-9	-8.1	TUJ0490	-14.0%	-6.1%

4.2.2 MODEL VALIDATION AGAINST AREA MONITORING DATA

In absence of any potential calibration data for the Tuck Creek watershed directly, the VO6 model has been validated using available data from the Fourteen Mile Creek, Hager-Rambo, and Morrison-Wedgewood watersheds, as described previously. Three (3) to four (4) different candidate storm events have been selected for model validation purposes from each of the three (3) watersheds. The VO6 modelling has been executed using the available rainfall for each of the selected events, including 14MC for the 14MC flow gauge, Tyandaga Reservoir and Burlington Fire Station 1 for the Hager-Rambo flow gauge, McCraney Reservoir for the Morrison/Wedgewood flow gauge, and Elizabeth Garden for the Sheldon Creek flow gauge. Peak flows at the watershed outlet (i.e. Lake Ontario) have then been extracted and normalized by area to enable a comparison between the datasets. Table 4.7 summarizes the comparisons between unitary flows for selected events.

Table 4.7. Comparison of Observed and Simulated Unitary Peak Flows – Tuck Creek Model Validation

EVENT	TOTAL RAINFALL DEPTH (mm)	MAX HOURLY RAINFALL INTENSITY (mm/hr)	COMPARISON WATERSHED		TUCK CREEK AT LAKESHORE	
			PEAK FLOW RATE (m^3/s)	UNITARY PEAK FLOW RATE ($m^3/s/ha$)	PEAK FLOW RATE (m^3/s)	UNITARY PEAK FLOW RATE ($m^3/s/ha$)
Fourteen Mile Creek (Drainage Area = 24.5 km^2) Tuck Creek at Lakeshore (Drainage Area = 11.05 km^2)						
2005-07-26	53.5	165.0	9.4	0.004	71.5	0.065
2008-08-05	60.9	64.0	10.2	0.004	43.9	0.040
2009-06-25	22.9	41.6	3.9	0.002	17.4	0.016
2013-06-22	19.8	38.8	3.9	0.002	15.6	0.014
Hager-Rambo at QEW (Drainage Area = 16.13 km^2) Tuck Creek at Lakeshore (Drainage Area = 11.05 km^2)						
2019-05-25	32.6	65.6	22.6	0.014	20.8	0.019
2019-10-27	50.6	30.4	10.4	0.006	21.6	0.020
2021-08-26	24.0	34.4	18.6	0.012	14.5	0.013
2021-10-15	27.2	33.6	8.7	0.005	16.1	0.015
Morrison/Wedgewood Outlet (Drainage Area = 20.08 km^2) Tuck Creek at Lakeshore (Drainage Area = 11.05 km^2)						
2019-05-25	33.4	43.2	20.1	0.010	11.3	0.010
2019-10-27	35.0	14.4	13.1	0.007	12.0	0.011
2021-10-15	24.6	28.0	16.6	0.008	15.6	0.014

1. Note initial model validation was completed at an earlier stage of the project and as such presented results may differ slightly from those from the final modelling.

The results indicate that the simulated unitary peak flows within Tuck Creek Watershed are typically an order of magnitude higher compared with the observed unitary flows at the Fourteen Mile Creek flow gauge. A review of the available flow monitoring data for the Fourteen Mile Creek gauge indicates that this location generated notably lower runoff volumes than the simulated results for Tuck Creek, despite having double the watershed area. The Fourteen Mile Creek flow gauge results also indicate a poor correlation to the available rainfall, and an inconsistent watershed response relative to the storm events. Other factors may also have resulted in the difference, including differences in land use and soil conditions within the two watersheds, as well as the potential impact of stormwater management facilities in the Fourteen Mile Creek watershed.

Based on a cursory review, the simulated results for Tuck Creek for the selected events (from Fourteen Mile Creek) also appear reasonable. The July 26, 2005 storm event rainfall intensity would exceed a 100-year storm, and thus compares reasonably to 100-year peak flow using conventional design storms. The August 5, 2008 storm event has a rainfall intensity roughly equivalent to a 25-year storm event. The other two storm events are more nominal (less than a 2-year storm).

The validation comparison indicates that simulated peak flows from Tuck Creek are slightly higher but are comparable to the monitoring data at both the Hager-Rambo and Morrison/Wedgewood gauges. Peak flow responses tend to be similar during events with higher rainfall intensity. Sample comparison hydrographs are included in Appendix D.

Based on the preceding, the uncalibrated model results for Tuck Creek are considered valid. Notwithstanding, additional model comparisons have been undertaken, as described in Section 4.2.3.

4.2.3 ADDITIONAL MODEL COMPARISONS

Comparison of Unitary Peak Flows with Previous Studies

The simulated uncalibrated 100-year and Regional Storm unitary peak flows have been compared with various previous studies across Southern Ontario based on WSP's database of previous watershed and hydrologic studies, as well as data for Morrison/Wedgwood and Grindstone provided by CH for use in this study. Reference is made to Figures 3.2 and 3.3 and Tables 3.4 presented previously. For the Tuck Creek watershed, simulated unitary flows of 0.071 and 0.109 m^3/s/ha result for the 100-year and Regional Storm Events; and a $Q_{Regional}/Q_{100}$ ratio of 1.53.

The comparison indicates that the simulated 100-year peak flow is generally higher than other studies in the Hamilton area (Waterdown, Red Hill Creek, Stoney and Battlefield Creeks) but is generally comparable to the results for the Morrison-Wedgewood Diversion Channel, which reflects a 24-hour Chicago Storm Event with SWM (rate of 0.094 m^3/s/ha was indicated for the no SWM scenario, which would be more comparable to Tuck Creek given the lack of SWM for this watershed). Similar findings are noted for the Regional Storm Event, however the simulated results for Tuck Creek are somewhat closer to the other study results. The ratio of the Regional Storm peak flow to the 100-year peak flow is lower than the majority of the other study results with the exception of the Morrison-Wedgewood Diversion Channel, owing to the elevated simulated 100-year storm peak flow.

Based on the graphic presentation, the simulated results do not appear to exceed the common range of results, however they are towards the upper end of typical results. The results for the ratio of the Regional to 100-year storm also indicate that the simulated results are at the lower end of the ratio, indicating a relatively lesser difference between the two storm events as compared to other studies.

In summary, the uncalibrated hydrologic model for the Tuck Creek Watershed is generally consistent with the statistics and metrics from the nearby Morrison-Wedgewood diversion channel. Greater differences are indicated for the other subject watersheds, however differences may result based on the degree of urbanization and SWM controls, as well as hydrologic modelling techniques and differences (for instance some of the simulated 100-year peak flows were developed on the basis of continuous simulation rather than design storm events). Overall, the unitary flows and ratios are within the range of reasonable values.

August 4th 2014 Event

In addition to the preceding model validations, the uncalibrated hydrologic model for the Tuck Creek Watershed has also been executed for the August 4th, 2014 storm event. This storm event has been run for model validation purposes only but is not a regulatory storm event. The rainfall distribution presented previously in Table 3.10 has been conservatively applied for all subcatchments (no spatial or temporal variation) without any reduction factors. The simulation results and comparison with both the 100-year storm event (12-hour SCS) the 12-hour Regional Storm (AMC-III) peak flows (with areal reduction factors) are presented in Table 4.8.

The comparison indicates that the Regional Storm would largely govern over the August 4th 2014 storm event and the 100-year storm event within Tuck Creek Watershed. However, the August 4th 2014 storm event would generate slightly higher peak flow between Fairview Street and Lake Ontario. If areal reduction factors were applied to this storm as well however, the results would be more closely comparable. The 100-year governs for the lower portion of the West Branch of Tuck Creek, which may reflect localized hydrograph timing effects.

Table 4.8. Simulated August 4th 2014 Storm and Regional Storm Peak Flows for Tuck Creek

LOCATION	ARF (%)	NODE	DRAINAGE AREA (ha)	PEAK FLOW RATE (m^3/s)		
				100 YEAR	12 HOUR REGIONAL (AMC III)	AUGUST 4TH 2014 (ARF=100%)
East Branch						
East Trib at Guelph Line	100	TUJ0040	70	4.3	8.6	7.2
West Trib at Guelph Line	100	TUJ0130	83	6.9	10.2	9.0
West Trib at Dundas Street	100	TUJ0140	80	7.1	10.4	9.3
East Trib at Dundas Street	100	TUJ0070	123	7.9	14.7	12.8
Confluence of East and West Trib D/S of Dundas Street	100	TUJ0090	208	15.0	25.4	22.4
Headon Forest Drive	100	TUJ0180	232	15.6	27.9	24.8
U/S of Confluence	100	TUJ0190	244	16.2	29.3	26.2
West Branch						
180 m D/S of Headon Forest Drive	100	TUJ0160	146	18.6	19.1	18.1
U/S of Confluence	100	TUJ0170	207	30.9	27.6	28.5
Main Branch						
Confluence of East and West Branch	100	TUJ0210	464	38.1	55.7	52.3
Headon Road	100	TUJ0240	559	51.9	66.7	63.9
Upper Middle Road	100	TUJ0260	617	60.5	72.9	69.9
Palmer Road	100	TUJ0280	656	59.0	77.4	74.5
Mainway	100	TUJ0300	706	60.1	82.6	79.7
CNR - Halton	99.2	TUJ0310	719	59.5	83.4	81.2
QEW	99.2	TUJ0330	744	60.8	86.1	84.1
Harvester Road	99.2	TUJ0350	811	64.4	93.2	91.6
CNR Oakville	98.2	TUJ0370	862	66.9	97.5	97.3
Fairview Street	98.2	TUJ0390	881	66.4	99.5	99.7
Rexway Drive	98.2	TUJ0400	919	67.3	103.5	103.8
New Street	97.1	TUJ0420	1,001	72.4	110.6	113.3
Spruce Avenue	97.1	TUJ0450	1,053	75.3	116.0	118.8
Lakeshore Road	97.1	TUJ0470	1,102	78.4	120.0	122.8
Lake Ontario	97.1	TUJ0490	1,105	78.5	120.2	123.0

4.3 FUTURE CONDITIONS MODEL RESULTS

4.3.1 MODEL UPDATES

As noted previously (Section 3.4.2) no future land use changes are expected for the Tuck Creek watershed (as also evident from Drawings 4a and 4b). As such, under future conditions, the only applied difference is the application of the future rainfall IDF (current City of Burlington IDF which incorporates an adjustment to account for climate change, as discussed in Section 3.7). Model results are noted in subsequent sections accordingly.

4.3.2 MODEL RESULTS

Simulated results for future conditions for Tuck Creek are presented in Table 4.9. Note that the presented flows do not include spills, if applicable; this is reviewed further in Section 4.3.3.

While existing and future data are presented in this report, calculated differences in results should not be interpreted as reassessing or demonstrating the impacts of future development. The climate change adjusted IDF has only been applied to define the future 1:100-year flow data and has not been applied to existing conditions. Also, to support model calibration, the existing conditions modeling represents the current watershed condition. This may include centralized SWM controls that are designed to provide attenuation for a future development condition identified in the Official Plan but where the proposed development is not fully built out. In these areas, the existing conditions model assumes existing land uses where development has not yet occurred and may therefore predict existing condition flow rates less than pre-development conditions.

Under future conditions, only the 100-year storm flows are changed. Based on these results, there is one (1) location where the 100-year becomes the Regulatory Event, namely the upper portion of the west branch (TUJ0160 as per Table 4.9).

The peak flows presented in Table 4.9 have been applied as the basis for the flood hazard mapping (notwithstanding any potential additional spill flows, as described in Section 4.3.3). Flood hazard mapping is described further in the separate hydraulic modelling report.

Table 4.9. Simulated Design Storm and Regional Storm Peak Flows at Key Locations for Tuck Creek – Future Conditions

LOCATION	ARF (%)	NODE	DRAINAGE AREA (ha)	PEAK FLOW RATE (m^3/s)		
				100 YEAR	12 HOUR REGIONAL (AMC III)	48 HOUR REGIONAL (AMC II)
East Branch						
East Trib at Guelph Line	100	TUJ0040	70	4.6	8.6	8.3
West Trib at Guelph Line	100	TUJ0130	80	7.4	10.2	9.9
East Trib at Dundas Street	100	TUJ0140	83	7.5	10.4	10.2
East Trib at Dundas Street	100	TUJ0070	123	8.5	14.7	14.2
Confluence of East and West Trib D/S of Dundas Street	100	TUJ0090	208	16.1	25.4	24.7
Headon Forest Drive	100	TUJ0180	232	16.6	27.9	27.2
U/S of Confluence	100	TUJ0190	244	17.2	29.3	28.5
West Branch						
180 m D/S of Headon Forest Drive	100	TUJ0160	146	19.6	19.1	18.7
U/S of Confluence	100	TUJ0170	207	32.5	27.6	27.2
Main Branch						
Confluence of East and West Branch	100	TUJ0210	464	40.7	55.7	54.5
Headon Road	100	TUJ0240	559	55.0	66.7	65.5
Upper Middle Road	100	TUJ0260	617	64.0	72.9	71.7
Palmer Road	100	TUJ0280	656	63.4	77.4	76.1
Mainway	100	TUJ0300	706	65.2	82.6	81.4
CNR - Halton	99.2	TUJ0310	719	64.2	83.4	82.1
QEW	99.2	TUJ0330	744	66.0	86.1	84.8
Harvester Road	99.2	TUJ0350	811	69.4	93.2	91.9
CNR Oakville	98.2	TUJ0370	862	72.2	97.5	97.2
Fairview Street	98.2	TUJ0390	881	71.8	99.5	98.1
Rexway Drive	98.2	TUJ0400	919	72.3	103.5	102.2
New Street	97.1	TUJ0420	1,001	76.9	110.6	109.2
Spruce Avenue	97.1	TUJ0450	1,053	80.1	116.0	114.7
Lakeshore Road	97.1	TUJ0470	1,102	83.2	120.0	118.8
Lake Ontario	97.1	TUJ0490	1,105	83.3	120.2	119.0

4.3.3 SPILL ASSESSMENT

The simulated future condition flows have been applied to the hydraulic modelling (both 1D and 2D) as described in the separate Hydraulics Report. As noted in that assessment, numerous spills have been identified. In order to apply the "balanced approach" proposed by CH (refer to CH's technical memorandum of May 19, 2022 as included within the Hydraulics Report), integration between the 2D hydraulic modelling and hydrologic modelling is necessary. Spill flows are to be included within the watershed receiving the spill. Iteration between the 2D hydraulic modelling and hydrologic modelling has been required to identify inter-watershed spills specifically that meet the threshold for inclusion, and to "balance" flows such that there is reasonable agreement between hydrologic and hydraulic modelling results.

For the Tuck Creek watershed, a spill flow has been identified from Shoreacres Creek to Tuck Creek at the QEW. An external flow hydrograph has been included in the hydrologic modelling accordingly at this location (to ADDHYD TUJ0330) using a READHYD command within Visual Otthymo. This would therefore affect peak flows for all locations downstream of this point.

The simulated inter-watershed spill flows and updated main branch flows (including spills) are presented in Table 4.10 along with the differences to the base results (from Table 4.9).

Table 4.10. Simulated 12H Regional Storm Peak Flows at Key Locations for Tuck Creek – Future Conditions with Inter-Watershed Spill Flows Included

LOCATION	ARF (%)	NODE	PEAK FLOW RATE (m^3/s)		
			BASE	WITH SPILL FLOW	DIFFERENCE
Spill Flows (from Shoreacres Creek)					
QEW	N/A	947 (TO TU0330)	0	18.7	+18.7
Main Branch (with Spill Flows)					
CNR - Halton	99.2	TUJ0310	83.4	83.4	0
QEW	99.2	TUJ0330	86.1	88.1	+2.0
Harvester Road	99.2	TUJ0350	93.2	94.7	+1.5
CNR Oakville	98.2	TUJ0370	97.5	98.8	+1.3
Fairview Street	98.2	TUJ0390	99.5	100.6	+1.1
Rexway Drive	98.2	TUJ0400	103.5	104.0	+0.5
New Street	97.1	TUJ0420	110.6	110.7	+0.1
Spruce Avenue	97.1	TUJ0450	116.0	116.0	0
Lakeshore Road	97.1	TUJ0470	120.0	120.0	0
Lake Ontario	97.1	TUJ0490	120.2	120.2	0

The results indicate that the magnitude of the spill flow is reduced when combined with the flows with the Tuck Creek; this appears attributable to a difference in hydrograph timing. Peak flows within Tuck Creek are increased by up to 2.0 m^3/s (approximately 2%), with differences decreasing further downstream.

The presented peak flows (with spills) are consistent with those applied for both the 1D and 2D hydraulic modelling and flood hazard mapping.

In addition to the preceding, it should be noted that additional spills have been indicated from Tuck Creek westerly to the adjacent watershed (Roseland Creek). Although not included as part of the hydrologic modelling (spill flows are as identified by the 2D hydraulic modelling; refer to companion Hydraulics Report accordingly) these additional watershed spill flows should be considered as part of any future hydrologic modelling of the Roseland Creek system. Simulated spill flows for the 100-year and Regional Storm Event under future conditions are presented in Table 4.11.

Table 4.11. Simulated External Spill Peak Flows from Tuck Creek to Roseland Creek - Future Conditions

LOCATION	PEAK FLOW RATE (m^3/s)	
	100 YEAR	12 HOUR REGIONAL (AMC III)
Main Branch		
At CNR - Halton	1.8	6.9
At QEW	8.8	24.9

5 SHOREACRES CREEK

5.1 MODEL DEVELOPMENT

5.1.1 SUBCATCHMENT BOUNDARIES

The Shoreacres Creek watershed has a total drainage area of approximately 12.40 km^2. Subcatchment boundaries for Shoreacres Creek have been developed using the approach summarized in Section 3.1.2. Statistics are presented in Table 5.1. The boundaries are presented graphically in Drawing 8a (attached).

Table 5.1. Proposed Subcatchment Boundaries for Shoreacres Creek

WATERSHED	TOTAL NUMBER OF SUBCATCHMENTS	MINIMUM SUBCATCHMENT SIZE (ha)	MAXIMUM SUBCATCHMENT SIZE (ha)	AVERAGE SUBCATCHMENT SIZE (ha)	STANDARD DEVIATION (ha)
Shoreacres Creek	58	4.5	85.6	23.8	+/-17.1

Contributing drainage areas at key locations have been compared with the 1997 Shoreacres Creek Floodline Mapping Update study. The comparison is summarized in Table 5.2.

Table 5.2. Comparison of Drainage Areas with Previous Study for Shoreacres Creek

LOCATION	1997 SHOREACRES FLOODLINE MAPPING STUDY		2023 EAST BURLINGTON CREEKS FPM		DRAINAGE AREA DIFFERENCES (2023 VS. 1997)	
	NODE	DRAINAGE AREA (ha)	NODE	DRAINAGE AREA (ha)	ABSOLUTE (ha)	%
East Branch						
Highway 407	-	90	SAJ0260	98	8	+8.6%
Dundas Street	-	140	SAJ0290	148	8	+6.0%
Walkers Line	-	170	SAJ0300	172	2	+1.0%
West Branch						
East Tributary - Highway 407	-	90	SAJ0110	99	9	+10.1%
Confluence - East & West Tributary	-	260	SAJ0120	271	11	+4.0%
Walkers Line	-	310	SAJ0160	326	16	+5.3%
Main Branch						
Confluence	-	530	SAJ0330	560	30	+5.6%
Upper Middle Road	-	560	SAJ0340	591	31	+5.5%
CN (Halton)	-	600	SAJ0360	623	23	+3.9%
QEW	-	690	SAJ0390	733	43	+6.2%
Harvester Road	-	820	SAJ0440	935	115	+14.0%
Fairview Street	-	930	SAJ0690	971	41	+4.4%
New Street	-	1,150	SAJ0750	1,202	52	+4.5%
Spruce Avenue	-	1,180	SAJ0770	1,229	49	+4.2%
Lake Ontario	-	1,240	SAJ0800	1,342	102	+8.2%

A graphical comparison of the overall boundaries is presented in Drawing 8b (attached).

In general, the drainage areas are comparable to the previous study, with the average differences typically ranging from 4 to 10%. The differences in drainage areas are relatively larger at East Branch at Highway 407, West Branch at Highway 407, and Harvester Road on Main Branch, which likely reflects the additional drainage area to the north of the QEW.

5.1.2 SUBCATCHMENT PARAMETERIZATION

Based on the subcatchment delineation, subcatchment parameterization has been established following the approach described in Section 3.4. A summary of the uncalibrated subcatchment parameters for Shoreacres Creek is included in Appendix F.

Subcatchment Slope

The surface slopes within the Shoreacres Creek Watershed tend to be moderate between 1 and 3%. The undeveloped areas north of Highway 407 are slightly steeper with the average slope greater than 4%.

Impervious Coverage

The land use conditions north of Dundas Street are primarily agricultural lands, open space, and forest, with rural residential areas distributed along major roads. The areas south of Dundas Street are largely developed and the land use conditions are a mix of urban residential and high density residential areas, high impervious areas, institutional areas, industrial areas, commercial areas, parks and open lands, as well as dispersed forests. A comparison to the previous study is presented in Table 5.3.

Table 5.3. Comparison of Modelled Imperviousness to Previous Studies for Shoreacres Creek

LOCATION	1997 SHOREACRES FLOODLINE MAPPING STUDY	2023 EAST BURLINGTON CREEKS FLOODPLAIN MAPPING (EXISTING CONDITIONS)	DIFFERENCE
East Branch	8.7%	27.2%	+18.5%
West Branch	12.7%	27.5%	+14.8%
Total	22.0%	47.0%	+25.0%

As evident from Table 5.3, impervious coverages from the original modelling vary, between 9 and 22%. The rationale for this difference has not been assessed further, however it is considered this may be attributable to changes in industry practice related to typical land use coverage assumptions. Given the highly urbanized nature of the Shoreacres Creek watershed below Highway 407ETR, the current values are generally considered appropriate and reasonable.

Infiltration

The soils within the Shoreacres Creek Watershed consist largely of Clay Loam (65%), of which 29% is in the rocky phase. The remaining portion of the soils within the watershed consists of loam (17%), sandy loam (15%), and silt loam (3%). Overall, the soils are largely classified as SCS Type 'C' and Type "D" soils, exhibiting low permeability and low infiltration potential with high potential for generating runoff.

SCS Curve Numbers have been applied on the basis of representative values for the pervious land segment. In particular for urbanized areas that utilized the STANDHYD routine, given that impervious coverage is accounted for separately, the CN value represents the solely pervious land segment. As an example, for a residential area, the SCS CN represents the grassed/lawn areas based on the applicable soils.

5.1.3 SWM FACILITIES

Based on the completed "SWM Pond Review" report (refer to Appendix B), a total of three (3) quantity control facilities have been proposed for inclusion in the hydrologic modelling of Shoreacres Creek. These facilities are presented on Drawings 5 and 8a (attached). They include:

- **Pond 512** (included up to 100-year storm event; original design rating curve applied with a modified overflow point)
- **Pond 3302** (included up to 100-year storm event; original design rating curve applied with a modified overflow point)
- **Pond 5513** (included up to 100-year storm event; new rating cure developed and applied)

Refer to Appendix B for further details on the proposed rating curves and details on the quantity control facilities.

5.1.4 MAJOR/MINOR SPLIT

Subcatchment SA0810 has been identified as having a split flow condition (major/minor split) based on the available contour mapping, the City's storm sewer database, and as-built drawings. The minor flows from Subcatchment SA0810 would be conveyed by the sewer system and discharge to Lake Ontario via a 1350 mm diameter pipe at a slope of 0.24% west of Avondale Court. The maximum capacity of the pipe has been determined to be 2.61 m^3/s using Manning's equation. This value has been used as a basis to split the minor and major components of the hydrograph accordingly. A maximum flow of 2.61 m^3/s would discharge to the Lake. Flows that exceed 2.61 m^3/s will travel west and discharge to Main Branch of Shoreacres Creek at Lakeshore Road through a combination of sewer and overland flow.

The major/minor split has been represented by DUHYD SAD0010 (NHYD 90010). One inlet with a maximum capture of 2.61 m^3/s has been assumed as the minor flow that would discharge to the Lake and be excluded from the system. Flow which exceeds 2.61 m^3/s is considered as major flow and would contribute to Shoreacres Creek Node SAJ0790 at Lakeshore Road.

Calculations of the split flows and as-built drawings are included in Appendix F.

5.1.5 AREAL REDUCTION FACTORS

The areal reduction factors (ARFs) applied to the Shoreacres Watershed are presented on Drawing 9 (attached). ARFs have been calculated consistent with the methodology described in Section 3.7.4. As noted in Section 3.7.4, it has been agreed that the same ARFs are to be applied for the Regional Storm event and design storm events. To summarize the findings for the Shoreacres Creek watershed:

— In general, areas north of Upper Middle Road are within the 0 to 25 km^2 circular area and would therefore not require an ARF.

— Areas between Upper Middle Road and QEW are within the 26 to 45 km^2 circular area and an ARF of 99.2% has been applied.

— Areas between QEW and Fairview Street are within the 46 to 65 km^2 circular area and an ARF of 98.2% has been applied.

— Areas between Fairview and Spruce Avenue is within the 66 to 90 km^2 circular area and an ARF of 97.1% has been applied.

— Areas between Spruce Avenue and outfall at Lake Ontario is within the 91 to 115 km^2 circular area and an ARF of 96.3% has been applied.

5.2 EXISTING CONDITIONS MODEL RESULTS

5.2.1 UNCALIBRATED MODEL RESULTS

Uncalibrated 100-Year Design Storm and Regional Storm Peak Flow Rates

The VO6 hydrologic model for Shoreacres Creek Watershed has been executed for the 12-hour SCS 100-year design storm event (existing 1964 to 2017 IDF), the 12-hour Regional Storm event under the AMC III (saturated) soil conditions, and the 48-hour Regional Storm event under the AMC II (normal) soil conditions. The peak flows at key locations have been summarized and presented in Table 5.4. Applicable areal reduction factors (ARFs) are noted in the table.

The results indicate that the 12-hour Regional Storm under the AMC III soil conditions and the 48 Hour Regional Storm under the AMC II conditions would generate similar peak flow rates. The governing storm event would be the 12-Hour Regional Storm in all cases.

Table 5.4. Simulated Uncalibrated Design Storm and Regional Storm Peak Flows at Key Locations for Shoreacres Creek

LOCATION	ARF (%)	NODE	DRAINAGE AREA (ha)	PEAK FLOW RATE (m^3/s)		
				100 YEAR	12 HOUR REGIONAL (AMC III)	48 HOUR REGIONAL (AMC II)
East Branch						
U/S of Highway 407	100	SAJ0260	98	5.1	10.7	10.3
U/S of Dundas St	100	SAJ0290	148	5.6	15.9	15.6
Walkers Line	100	SAJ0300	172	7.3	18.2	17.8
West Branch						
U/S of Highway 407-East Tributary	100	SAJ0110	99	6.0	11.4	11.1
U/S of Highway 407-West Tributary	100	SAJ0050	144	9.4	16.5	16.0
Confluence point of Tributaries	100	SAJ0120	271	16.3	30.9	30.0
Walkers Line	100	SAJ0160	326	18.4	36.7	35.8
Main Branch						
Confluence point of East & West Branches	100	SAJ0330	560	28.6	60.9	59.7
D/S of Upper Middle Rd	100	SAJ0340	591	29.3	64.0	62.8
CNR-Halton	99.2	SAJ0360	623	27.5	65.4	64.2
Mainway	99.2	SAJ0370	635	27.8	66.4	65.1

LOCATION	ARF (%)	NODE	DRAINAGE AREA (ha)	PEAK FLOW RATE (m^3/s)		
				100 YEAR	12 HOUR REGIONAL (AMC III)	48 HOUR REGIONAL (AMC II)
QEW	99.2	SAJ0390	733	36.9	76.1	74.9
Harvester Rd	98.2	SAJ0440	935	63.9	95.3	94.0
CNR-Oakville	98.2	SAJ0680	958	62.8	97.2	95.9
Fairview St	98.2	SAJ0690	971	65.3	98.4	97.1
New St	97.1	SAJ0750	1,202	92.9	119.4	118.0
Spruce Avenue	97.1	SAJ0770	1,229	91.1	121.3	119.9
Lakeshore Rd	96.3	SAJ0790	1,285	95.2	127.9	126.3
Lake Ontario	96.3	SAJ0800	1,342	95.6	128.2	126.6

Comparison of Simulated Peak Flows with Previous studies

The VO6 hydrologic model has been executed to compare against the results of the 1997 Shoreacres Creek Floodline Mapping Update study. To maintain consistency with the 1997 study, the same IDF (based on 27 years of data recorded at the Atmospheric Environment Royal Botanical Gardens Gauge) and the same time to peak ratio of 0.46 has been used for the comparison. Also, the ARFs from the 1997 study (100-year and Regional Storm) have been applied in this assessment to maintain consistency with the previous study. Results are presented in Table 5.5, associated differences are presented in Table 5.6.

The 100-year peak flow rates generated from the VO6 model are between 5 and 58% lower than the peak flow rates generated from the 1997 GAWSER model upstream of the QEW. Conversely, the simulated 100-year flows between the QEW and the outlet of Lake Ontario are relatively higher (16-42%) compared to the previous study.

The current model results also indicate some fluctuation for the section between Upper Middle Road and Lake Ontario, with increases and decreases in peak flows despite continually increasing drainage area (as opposed to the GAWSER model which indicates uniform increases). This may reflect a greater sensitivity to routing elements in the modelling, particularly for a short duration storm event. Based on WSP's review, changes in channel geometry, including differences in elevations or flat overbanks may result in different degrees of attenuation for relatively minor differences in peak flows. This could potentially be addressed by further simplifying routing channel cross-sections (reduced number of points) in these areas as part of future study.

By contrast the simulated Regional Storm flows are consistently higher than the peak flow rate generated from the 1997 GAWSER model. The differences are relatively larger along the west branch (19 to 37%) than further downstream on the main branch, where increases are generally consistently in the range of 11 to 21%.

The differences in the simulated peak flows are considered attributable to the different modelling platforms, parameterization methodology, minor differences in contributing drainage areas as well as inclusion of select parking lot and rooftop storage and an on-line private pond in the older hydrologic modelling.

Table 5.5. Comparison of Simulated Peak Flows for Shoreacres Creek at Key Locations (1997 Study)

LOCATION	1997 SHOREACRES CREEK FLOODLINE MAPPING UPDATE			2023 EAST BURLINGTON CREEKS FPM		
	ARF (%)	3 HOUR CHICAGO 100 YEAR (m^3/s)	48 HOUR REGIONAL (m^3/s)	NODE	3 HOUR CHICAGO 100 YEAR (m^3/s)	48 HOUR REGIONAL (m^3/s)
East Branch						
Highway 407	100	4.6	8.2	SAJ0260	3.0	10.3
Dundas Street	100	7.6	14	SAJ0290	3.2	15.6
Walkers Line	100	8.7	16	SAJ0300	6.2	17.8
West Branch						
East Tributary - Highway 407	100	4.4	8.1	SAJ0110	3.6	11.1
Confluence - East & West Tributary	100	14	25	SAJ0120	6.7	30.0
Walkers Line	100	18	30	SAJ0160	11.2	35.8
Main Branch						
Confluence	100	28	52	SAJ0330	18.7	59.7
Upper Middle Road	100	29	55	SAJ0340	20.1	62.8
CNR (Halton)	99.2	29	57	SAJ0360	18.1	64.2
QEW	99.2	31	66	SAJ0390	29.5	74.9
Harvester Road	98.2	35	78	SAJ0440	49.7	94.0
Fairview Street	98.2	38	88	SAJ0690	49.1	97.1
New Street	97.1	57	107	SAJ0750	70.0	118.0
Spruce Avenue	97.1	57	108	SAJ0770	67.9	119.9
Lake Ontario	97.1	61	114	SAJ0800	70.5	126.6

1. For the purposes of this comparison, areal reduction factors and rainfall distributions from the previous study have been maintained. Values in this table may not be consistent with values presented in other sections of the report.

Table 5.6. Differences in Simulated Peak Flows for Shoreacres Creek at Key Locations (2023 Study vs. 1997 Study)

LOCATION	ABSOLUTE DIFFERENCE (2023 VS.1997)			PERCENT DIFFERENCE (2023 VS.1997)		
	ARF	3 HOUR CHICAGO 100 YEAR PEAK FLOW (m^3/s)	48 HOUR REGIONAL PEAK FLOW (m^3/s)	NODE	3 HOUR CHICAGO 100 YEAR PEAK FLOW (m^3/s)	48 HOUR REGIONAL PEAK FLOW (m^3/s)
East Branch						
Highway 407	1	-1.6	+2.1	SAJ0260	-34.8%	+25.6%
Dundas Street	1	-4.4	+1.6	SAJ0290	-57.9%	+11.4%
Walkers Line	1	-2.5	+1.8	SAJ0300	-28.7%	+11.3%
West Branch						
East Tributary -Highway 407	1	-0.8	+3.0	SAJ0110	-18.2%	+37.0%
Confluence - East & West Tributary	1	-7.3	+5.0	SAJ0120	-52.1%	+20.0%
Walkers Line	1	-6.8	+5.8	SAJ0160	-37.8%	+19.3%
Main Branch						
Confluence	1	-9.3	+7.7	SAJ0330	-33.2%	+14.8%
Upper Middle Road	1	-8.9	+7.8	SAJ0340	-30.7%	+14.2%
CNR (Halton)	0.992	-10.9	+7.2	SAJ0360	-37.6%	+12.6%
QEW	0.992	-1.5	+8.9	SAJ0390	-4.8%	+13.5%
Harvester Road	0.982	+14.7	+16.0	SAJ0440	+42.0%	+20.5%
Fairview Street	0.982	+11.1	+9.1	SAJ0690	+29.2%	+10.3%
New Street	0.971	+13.0	+11.0	SAJ0750	+22.8%	+10.3%
Spruce Avenue	0.971	+10.9	+11.9	SAJ0770	+19.1%	+11.0%
Lake Ontario	0.971	+9.5	+12.6	SAJ0800	+15.6%	+11.1%

5.2.2 MODEL VALIDATION AGAINST AREA MONITORING DATA

In absence of any potential calibration data for the Shoreacres Creek watershed directly, the VO6 model has been validated using available data from the Fourteen Mile Creek, Hager-Rambo, and Morrison-Wedgewood watersheds, as described previously. Three (3) to four (4) different candidate storm events have been selected for model validation purposes from each of the three (3) watersheds. The VO6 modelling has been executed using the observed rainfall for selected events, including 14MC for the 14MC flow gauge, Tyandaga Reservoir and Burlington Fire Station 1 for the Hager-Rambo flow gauge, McCraney Reservoir for the Morrison/Wedgewood flow gauge, and Elizabeth Garden for the Sheldon Creek flow gauge. Peak flows have then been extracted and normalized by area to enable a comparison between the datasets. Table 5.7 summarizes comparisons between unitary flows for selected events.

Table 5.7. Comparison of Observed and Simulated Unitary Peak Flows – Shoreacres Creek Model Validation

EVENT	TOTAL RAINFALL DEPTH (mm)	MAX HOURLY RAINFALL INTENSITY (mm/hr)	COMPARISON WATERSHED		SHOREACRES CREEK AT LAKESHORE	
			PEAK FLOW RATE (m^3/s)	UNITARY PEAK FLOW RATE ($m^3/s/ha$)	PEAK FLOW RATE (m^3/s)	UNITARY PEAK FLOW RATE ($m^3/s/ha$)
Fourteen Mile Creek (Drainage Area = 24.5 km^2) Shoreacres Creek at Lakeshore (Drainage Area = 13.42 km^2)						
2005-07-26	53.5	165.0	9.4	0.004	88.2	0.066
2008-08-05	60.9	64.0	10.2	0.004	52.9	0.039
2009-06-25	22.9	41.6	3.9	0.002	21.4	0.016
2013-06-22	19.8	38.8	3.9	0.002	19.2	0.014
Hager-Rambo at QEW (Drainage Area = 16.13 km^2) Shoreacres Creek at Lakeshore (Drainage Area = 13.42 km^2)						
2019-05-25	32.6	65.6	22.6	0.014	24.2	0.018
2019-10-27	50.6	30.4	10.4	0.006	24.3	0.018
2021-08-26	24.0	34.4	18.6	0.012	19.0	0.014
2021-10-15	27.2	33.6	8.7	0.005	20.5	0.015
Morrison/Wedgewood Outlet (Drainage Area = 20.08 km^2) Shoreacres Creek at Lakeshore (Drainage Area = 13.42 km^2)						
2019-05-25	33.4	43.2	20.1	0.010	14.9	0.011
2019-10-27	35.0	14.4	13.1	0.007	13.6	0.010
2021-10-15	24.6	28.0	16.6	0.008	20.0	0.015

1. Note initial model validation was completed at an earlier stage of the project and as such presented results may differ slightly from those from the final modelling.

The results indicate that the simulated unitary peak flows within Shoreacres Creek Watershed are typically an order of magnitude higher compared with the observed unitary flows at the Fourteen Mile Creek flow gauge. A review of the available flow monitoring data for the Fourteen Mile Creek gauge indicates that this location generated notably lower runoff volumes than the simulated results for Shoreacres Creek, despite having double the watershed area. The Fourteen Mile Creek flow gauge results also indicate a poor correlation to the available rainfall, and an inconsistent watershed response relative to the storm events. Other factors may also have resulted in the difference, including differences in land use and soil conditions within the two watersheds, as well as the potential impact of stormwater management facilities in the Fourteen Mile Creek watershed.

Based on a cursory review, the simulated results for Shoreacres Creek for the selected events (from Fourteen Mile Creek) also appear reasonable. The July 26, 2005 storm event rainfall intensity would exceed a 100-year storm, and thus compares reasonably to 100-year peak flow using conventional design storms. The August 5, 2008 storm event has a rainfall intensity roughly equivalent to a 25-year storm event. The other two storm events are more nominal (less than a 2-year storm).

The validation comparison indicates that simulated peak flows from Shoreacres Creek are slightly higher but are comparable to the monitoring data at both the Hager-Rambo and Morrison/Wedgewood gauges. Peak flow responses tend to be similar during events with higher rainfall intensity. Based on the preceding, the uncalibrated model results for Shoreacres Creek are considered valid. Notwithstanding, additional model comparisons have been undertaken, as described in Section 5.2.3.

5.2.3 ADDITIONAL MODEL COMPARISON

Comparison of Unitary Peak Flows with Previous Studies

The simulated uncalibrated 100-year and Regional Storm unitary peak flows have been compared with various previous studies across Southern Ontario based on WSP's database of previous watershed and hydrologic studies, as well as data for Morrison/Wedgwood and Grindstone provided by CH for use in this study. Reference is made to Figures 3.2 and 3.3 and Tables 3.4 presented previously. For the Shoreacres Creek watershed, unitary flows of 0.071 and 0.096 $m^3/s/ha$ result for the 1:100 year and Regional Storm rainfall scenarios, respectively; with a resulting $Q_{Regional}/Q_{100}$ ratio of 1.35.

The comparison indicates that the simulated 100-year peak flow is generally higher than other studies in the Hamilton area (Waterdown, Red Hill Creek, Stoney and Battlefield Creeks) but is generally comparable to the results for the Morrison-Wedgewood Diversion Channel, which reflects a 24-hour Chicago Storm Event with SWM (rate of 0.094 $m^3/s/ha$ was indicated for the no SWM scenario, which would be more comparable to Shoreacres Creek given typical industry practices in place during the development of this watershed). Similar findings are noted for the Regional Storm Event, however the simulated results for Shoreacres Creek are somewhat closer to the other study results. The ratio of the Regional Storm peak flow to the 100-year peak flow is lower than the majority of the other study results with the exception of the Morrison-Wedgewood Diversion Channel, owing to the elevated simulated 100-year storm peak flow.

Based on the graphic presentation, the simulated results do not appear to exceed the common range of results, however they are towards the upper end of typical results. The results for the ratio of the Regional to 100-year storm also indicate that the simulated results are at the lower end of the ratio, indicating a relatively lesser difference between the two storm events as compared to other studies.

In summary, the uncalibrated hydrologic model for the Shoreacres Creek Watershed is generally consistent with the statistics and metrics from the nearby Morrison-Wedgewood diversion channel. Greater differences are indicated for the other subject watersheds, however differences may result based on the degree of urbanization and SWM controls, as well as hydrologic modelling techniques and differences (for instance some of the simulated 100-year peak flows were developed on the basis of continuous simulation rather than design storm events). Overall, the unitary flows and ratios are within the range of reasonable values.

August 4th 2014 Event

In addition to the preceding model validations, the uncalibrated hydrologic model for the Shoreacres Creek Watershed has also been executed for the August 4th, 2014 storm event. This storm event has been run for model validation purposes only but is not a regulatory storm event. The rainfall distribution presented previously in Table 3.10 has been conservatively applied for all subcatchments (no spatial or temporal variation) without any reduction factors. The simulation results and comparison with both the 100-year storm event (12-hour SCS) the 12-hour Regional Storm (AMC-III) peak flows (with areal reduction factors) are presented in Table 5.8.

The comparison indicates that the Regional Storm would largely govern over the August 4th 2014 storm event and the 100-year storm event within Shoreacres Creek Watershed. However, the August 4th 2014 storm event would generate slightly higher peak flow between New Street and Lake Ontario. If areal reduction factors were applied to this storm as well however, the results would be more closely comparable.

Table 5.8. Simulated August 4th, 2014 Storm and Regional Storm Peak Flows for Shoreacres Creek

LOCATION	ARF (%)	NODE	DRAINAGE AREA (ha)	PEAK FLOW RATE (m^3/s)		
				100 YEAR	12 HOUR REGIONAL (AMC III)	AUGUST 4TH 2014 (ARF=100%)
East Branch						
U/S of Highway 407	100	SAJ0260	98	5.1	10.7	9.6
U/S of Dundas St	100	SAJ0290	148	5.6	15.9	15.5
Walkers Line	100	SAJ0300	172	7.3	18.2	17.0
West Branch						
U/S of Highway 407-East Tributary	100	SAJ0110	99	6.0	11.4	10.2
U/S of Highway 407-West Tributary	100	SAJ0050	144	9.4	16.5	15.0
Confluence point of Tributaries	100	SAJ0120	271	16.3	30.9	28.3
Walkers Line	100	SAJ0160	326	18.4	36.7	34.2
Main Branch						
Confluence point of East & West Branches	100	SAJ0330	560	28.6	60.9	57.5
D/S of Upper Middle Rd	100	SAJ0340	591	29.3	64.0	60.6
CNR-Halton	99.2	SAJ0360	623	27.5	65.4	61.8
Mainway	99.2	SAJ0370	635	27.8	66.4	62.8
QEW	99.2	SAJ0390	733	36.9	76.1	73.3
Harvester Rd	98.2	SAJ0440	935	63.9	95.3	94.5
CNR-Oakville	98.2	SAJ0680	958	62.8	97.2	95.9
Fairview St	98.2	SAJ0690	971	65.3	98.4	97.1
New St	97.1	SAJ0750	1,202	92.9	119.4	122.1
Spruce Avenue	97.1	SAJ0770	1,229	91.1	121.3	124.4
Lakeshore Rd	96.3	SAJ0790	1,285	95.2	127.9	133.3
Lake Ontario	96.3	SAJ0800	1,342	95.6	128.2	133.9

5.3 FUTURE CONDITIONS MODEL RESULTS

5.3.1 MODEL UPDATES

The subcatchment total and directly connected impervious coverage have been updated by adding identified additional areas (as per Section 3.4.2 and Drawings 4a and 4b) into the base land use mapping layer; values have been developed consistent with the approach for existing land use as per CH's Table of standard values (Table 7) corresponding to each land use type. The land cover types in the Urban Burlington Land Cover layer have been categorized into the groups outlined in CH's standard parameter Table 7. Proposed total impervious coverage values and direct impervious coverage values corresponding to each land use type are included in Appendix A. A comparison of the overall changes is presented in Table 5.9.

Table 5.9. Comparison of Impervious Coverage between Existing and Future Conditions

LOCATION	1997 SHOREACRES FLOODLINE MAPPING STUDY	2023 EAST BURLINGTON CREEKS FLOODPLAIN MAPPING (EXISTING CONDITIONS)	2023 EAST BURLINGTON CREEKS FLOODPLAIN MAPPING (FUTURE CONDITIONS)	DIFFERENCE FROM EXISTING CONDITIONS
East Branch	8.7%	27.2%	33.3%	+6.1%
West Branch	12.7%	27.5%	28.3%	+0.8%
Total	22.0%	47.0%	52.3%	+5.3%

The comparison indicates there would be an approximately 5% increase in impervious coverage throughout the watershed due to the expected future land use changes, with a notable increase of approximately 6% occurring along the east branch.

In addition, under future conditions the future rainfall IDF has been applied (current City of Burlington IDF which incorporates an adjustment to account for climate change, as discussed in Section 3.7). This would be expected to further increase the simulated peak flows for the 100-year storm event. Model results are noted in subsequent sections accordingly.

5.3.2 MODEL RESULTS

Simulated peak flows under future conditions are presented in Table 5.10. Note that the presented flows do not include spills, if applicable; this is reviewed further in Section 5.3.3.

While existing and future data are presented in this report, calculated differences in results should not be interpreted as reassessing or demonstrating the impacts of future development. The climate change adjusted IDF has only been applied to define the future 1:100-year flow data and has not been applied to existing conditions. Also, to support model calibration, the existing conditions modeling represents the current watershed condition. This may include centralized SWM controls that are designed to provide attenuation for a future development condition identified in the Official Plan but where the proposed development is not fully built out. In these areas, the existing conditions model assumes existing land uses where development has not yet occurred and may therefore predict existing condition flow rates less than pre-development conditions.

Table 5.10. Simulated Design Storm and Regional Storm Peak Flows at Key Locations for Shoreacres Creek – Future Conditions (Without Spills)

LOCATION	ARF (%)	NODE	DRAINAGE AREA (ha)	PEAK FLOW RATE (m^3/s)	
				100 YEAR	12 HOUR REGIONAL (AMC III)
East Branch					
U/S of Highway 407	100	SAJ0260	98	5.5	10.7
U/S of Dundas St	100	SAJ0290	148	6.1	15.9
Walkers Line	100	SAJ0300	172	8.9	18.1 (18.2)[1]
West Branch					
U/S of Highway 407-East Tributary	100	SAJ0110	99	6.4	11.4
U/S of Highway 407-West Tributary	100	SAJ0050	144	10.0	16.5
Confluence point of Tributaries	100	SAJ0120	271	17.3	30.8
Walkers Line	100	SAJ0160	326	19.4	36.6
Main Branch					
Confluence point of East & West Branches	100	SAJ0330	560	31.3	60.8
D/S of Upper Middle Rd	100	SAJ0340	591	32.5	63.9
CNR-Halton	99.2	SAJ0360	623	30.1	65.3
Mainway	99.2	SAJ0370	635	30.4	66.2
QEW	99.2	SAJ0390	733	41.1	76.1
Harvester Rd	98.2	SAJ0440	935	73.1	95.3
CNR-Oakville	98.2	SAJ0680	958	71.5	97.2
Fairview St	98.2	SAJ0690	971	74.4	98.4
New St	97.1	SAJ0750	1,202	103.8	119.5
Spruce Avenue	97.1	SAJ0770	1,229	101.9	121.5
Lakeshore Rd	96.3	SAJ0790	1,285	106.2	128.1
Lake Ontario	96.3	SAJ0800	1,342	107.0	128.4

For the Regional Storm Event, minor decreases in flow are indicated at a few locations which may reflect hydrograph timing effects. The existing conditions peak flow should be used in these cases. Minor increases are noted at the most downstream portion of the main branch, from the CNR downstream. The 12-hour Regional Storm remains the Regulatory Storm Event in all cases.

The peak flows presented in Table 5.10 (with the exception of the locations noted, where the existing condition flow should be applied) have been applied as the basis for the flood hazard mapping, as described further in the separate hydraulic modelling report.

5.3.3 ASSESSMENT OF SPILLS

The simulated future condition flows have been applied to the hydraulic modelling (both 1D and 2D) as described in the separate Hydraulics Report. As noted in that assessment, numerous spills have been identified. In order to apply the "balanced approach" proposed by CH (refer to CH's technical memorandum of May 19, 2022 as included within the Hydraulics Report), integration between the 2D hydraulic modelling and hydrologic modelling is necessary. Spill flows are included within the watershed receiving the spill. Iteration between the 2D hydraulic modelling and hydrologic modelling have been required to identify inter-watershed spills specifically that meet the threshold for inclusion, and to "balance" flows such that there is reasonable agreement between hydrologic and hydraulic modelling results.

For the Shoreacres Creek watershed, an inter-watershed spill has been identified from Appleby Creek at the QEW. The estimated magnitude of the spill and resulting change in nodal peak flows is presented in Table 5.11, along with the difference in peak flows as compared to the without spill scenario (Table 5.10).

Table 5.11. Simulated Design Storm and Regional Storm Peak Flows at Key Locations for Shoreacres Creek with Inter-Watershed Spill Flows – Future Conditions

LOCATION	ARF (%)	NODE	PEAK FLOW RATE (m^3/s)		DIFFERENCE (m^3/s)	
			100 YEAR	12 HOUR REGIONAL (AMC III)	100 YEAR	12 HOUR REGIONAL (AMC III)
Spill Flows (from Appleby Creek)						
QEW	N/A	410 (to SAJ0400)	4.6	5.3	+4.6	+5.3
Main Branch						
QEW	99.2	SAJ0390	41.1	76.1	0	0
Harvester Rd	98.2	SAJ0440	73.1	99.8	0	+4.5
CNR-Oakville	98.2	SAJ0680	71.5	101.9	0	+4.7
Fairview St	98.2	SAJ0690	74.4	103.1	0	+4.7
New St	97.1	SAJ0750	103.8	124.5	0	+5.0
Spruce Avenue	97.1	SAJ0770	101.9	126.5	0	+4.0
Lakeshore Rd	96.3	SAJ0790	106.2	133.0	0	+4.9
Lake Ontario	96.3	SAJ0800	107.0	133.4	0	+5.0

The results indicate that for the 100-year storm event, hydrograph timing is such that there is no increase in peak flows. This likely reflects the more peaked nature of the 100-year storm event. Conversely for the Regional Storm Event, the simulated spill flow results in a relatively consistent increase in peak flows at all downstream nodes.

In addition to inter-watershed spills, a number of intra-watershed spills have been identified (spills between different watercourse branches within the same overall watershed system). These flows are not corrected within the 2D modelling, to avoid double-counting flows (given that the flows would be expected to generally re-combine further downstream). However, in order to ensure consistency between 1D and 2D hydraulic modelling, the spill flow has been incorporated into a separate hydrologic modelling scenario to consider the resulting increased flow to the branch receiving the spill. These results are presented in Table 5.12.

Two (2) different intra-watershed spills are indicated within the upper reaches of Shoreacres Creek Within the East Branch, a spill flow is indicated from the West Branch along Dundas Street, with a more notable and consistent increase indicated for the Regional Storm. The simulated increase for the 100-year storm event appears to be eliminated due to timing effects at Walkers Line. The other spill is indicated on the west branch (west tributary) from the east tributary, which results in increases in both the 100-year and Regional Storm flows downstream.

Table 5.12. Simulated Design Storm and Regional Storm Peak Flows at Key Locations for Shoreacres Creek with Intra-Watershed Spill Flows – Future Conditions

LOCATION	ARF (%)	NODE	PEAK FLOW RATE (m^3/s)		DIFFERENCE (m^3/s)	
			100 YEAR	12 HOUR REGIONAL (AMC III)	100 YEAR	12 HOUR REGIONAL (AMC III)
East Branch						
Spill from West Branch	100	410	1.1	5.6	+1.1	+5.6
U/S of Dundas St	100	SAJ0290	7.1	21.0	+1.0	+5.1
Walkers Line	100	SAJ0300	8.9	22.4	0	+4.3
West Branch – West Tributary						
Spill from East Tributary	100	411	2.4	3.7	+2.4	+3.7
U/S of Highway 407-West Tributary	100	SAJ0050	11.6	19.7	+1.6	+3.2

6 APPLEBY CREEK

6.1 MODEL DEVELOPMENT

6.1.1 SUBCATCHMENT BOUNDARIES

The Appleby Creek watershed has a total drainage area of approximately 12.23 km^2. Subcatchment boundaries for Appleby Creek have been developed using the approach summarized in Section 3.1.2. Statistics are presented in Table 6.1. The boundaries are presented graphically in Drawing 10a (attached).

Table 6.1. Proposed Subcatchment Boundaries for Appleby Creek

WATERSHED	TOTAL NUMBER OF SUBCATCHMENTS	MINIMUM SUBCATCHMENT SIZE (ha)	MAXIMUM SUBCATCHMENT SIZE (ha)	AVERAGE SUBCATCHMENT SIZE (ha)	STANDARD DEVIATION (ha)
Appleby Creek	46	1.12	71.04	26.02	+/-18.33

Contributing drainage areas at key locations have been compared with the 1997 Appleby Creek Floodline Mapping Update. The results are summarized in Table 6.2. A graphical comparison of the overall boundaries is presented in Drawing 10b (attached).

In general, the drainage areas obtained from the updated drainage area boundaries are comparable to the previous study, with an average difference of only 3%. The updated modelling indicates a slightly larger contributing drainage area than the previously modelling.

Table 6.2. Comparison of Drainage Areas with Previous Study for Appleby Creek

LOCATION	1997 APPLEBY CREEK EROSION CONTROL CLASS EA		2023 EAST BURLINGTON CREEKS FPM		DRAINAGE AREA DIFFERENCES (2023 VS. 2012)	
	NODE	DRAINAGE AREA (ha)	NODE	DRAINAGE AREA (ha)	ABSOLUTE (ha)	%
East Branch						
Prop. Hwy 407	-	70	AP0070	71	+1	+1.4%
Prop. Hwy 407-West Trib	-	200	APJ0060	199	-1	-0.5%
Confluence - East & West Tribs	-	270	APJ0070	271	+1	+0.4%
Dundas Street	-	330	APJ0100	321	-9	-2.7%
Millcroft Park Drive	-	430	APJ0130	441	+11	+2.6%
CN (Halton)	-	470	APJ0150	480	+10	+2.1%
Appleby Line	-	520	APJ0160	533	+13	+2.5%
Confluence - West Branch	-	620	APJ0220	638	+18	+2.9%

LOCATION	1997 APPLEBY CREEK EROSION CONTROL CLASS EA		2023 EAST BURLINGTON CREEKS FPM		DRAINAGE AREA DIFFERENCES (2023 VS. 2012)	
	NODE	DRAINAGE AREA (ha)	NODE	DRAINAGE AREA (ha)	ABSOLUTE (ha)	%
West Branch						
Millcroft Park Drive	-	100	APJ0310	99	-1	-1.0%
Upper Middle Road	-	120	APJ0320	120	0	0%
CN (Halton)	-	140	APJ0330	146	+6	+4.3%
Appleby Line	-	180	APJ0350	196	+16	+8.9%
Confluence - East Branch	-	240	APJ0380	237	-3	-1.3%
Main Branch						
Confluence - East & West Branch	-	860	APJ0230	875	+15	+1.7%
CN (Oakville)	-	900	APJ0240	920	+20	+2.2%
Pinedale Avenue	-	1,010	APJ0530	1,041	+31	+3.1%
New Street	-	1,100	APJ0560	1,126	+26	+2.4%
Spruce Avenue	-	1,130	APJ0590	1,163	+33	+2.9%
Lake Ontario	-	1,190	APJ0560	1,223	+33	+2.8%

6.1.2 PARAMETERIZATION

Based on the subcatchment delineation, subcatchment parameterization has been established following the approach described in Section 3.4. A summary of the uncalibrated subcatchment parameters for Appleby Creek is included in Appendix G.

Subcatchment Slope

The surface slopes within the Appleby Creek Watershed tend to be moderate between 1 and 3%. The undeveloped areas north of Highway 407 are slightly steeper with the average slope greater than 3.9%.

Impervious Coverage

The land use conditions north of Highway 407 are primarily agricultural lands, open space, and forest, with rural residential areas distributed along major roads. The areas south of Highway 407 are largely developed and the land use conditions are a mix of urban residential and high density residential areas, high impervious areas, institutional areas, industrial areas, commercial areas, parks and open lands, as well as dispersed forests. A comparison to the previous study is presented in Table 6.3.

Table 6.3. Comparison of Modelled Imperviousness to Previous Studies for Appleby Creek

LOCATION	1997 APPLEBY CREEK EROSION CONTROL CLASS EA	2023 EAST BURLINGTON CREEKS FLOODPLAIN MAPPING (EXISTING CONDITIONS)	DIFFERENCE
East Branch	40.8%	58.2%	+17.4%
West Branch	12.0%	37.2%	+25.2%
Total	25.3%	50.6%	+25.3%

As evident from Table 6.3, representations of impervious coverages have increased, by between 17 and 25%. The rationale for this difference has not been assessed further, however it is considered this may be attributable to changes in industry practice related to typical land use coverage assumptions. Given the highly urbanized nature of the Appleby Creek watershed below Highway 407ETR, the current values are generally considered appropriate and reasonable.

Infiltration

The soils consist predominantly of Clay Loam (81%), of which one-fifth is in the rocky phase. The remaining portion of the soils within the watershed consists of loam (4%), sandy loam (2%), and urban built-up areas (13%). Overall, the soils are largely classified as SCS Type 'C' and Type "D" soils, exhibiting low permeability and low infiltration potential with high potential for generating runoff.

SCS Curve Numbers have been applied on the basis of representative values for the pervious land segment. In particular for urbanized areas that utilized the STANDHYD routine, given that impervious coverage is accounted for separately, the CN value represents the solely pervious land segment. As an example, for a residential area, the SCS CN represents the grassed/lawn areas based on the applicable soils.

6.1.3 SWM FACILITIES

Based on the completed "SWM Pond Review" report (refer to Appendix B), a total of one (1) quantity control facility has been proposed for inclusion in the hydrologic modelling of Appleby Creek. The facility is presented on Drawings 5 and 10a (attached). The subject quantity control facility is:

- **Pond 513** (included up to 100-year storm event; original design rating curve applied with a modified overflow point)

Refer to Appendix B for further details on the proposed rating curves and details on quantity control facilities.

6.1.4 MAJOR/MINOR SPLIT

In addition to Subcatchment AP0080 (contributing to SWM Pond 513), subcatchments AP0150 and AP0600 have been identified with a split flow based on the available contour mapping, the City's storm sewer data base and as-built drawings. Further details are included in Appendix G (calculations and drawings).

Within Subcatchment AP0150 (east branch near the CNR tracks), a portion of the flow would discharge to Appleby Creek downstream of CNR between Ironstone Drive and Corporate Drive through the 675 mm diameter storm sewer at the underpass. There are approximately 18 catch basins that connect to the 675 mm diameter sewer at the underpass. Assuming each catch basin has an average inlet capacity of 0.06 m^3/s (as per the available inlet capacity curves for standard on-grade (non-sag point) catch basins in the MTO Drainage Management Manual, 1997), the total maximum captured flow would be 1.08 m^3/s which would then discharge to Appleby Creek between Ironstone Drive and Corporate Drive. The flows exceeding 1.08 m^3/s would discharge to Appleby Creek at Upper Middle Road west of the tracks through a combination of storm sewer and overland flow. This split flow is represented by DUHYD APD0020 (NHYD90020). Eighteen (18) inlets with a combined total maximum capture of 1.08 m^3/s have been assumed as the minor flow that would contribute to the 675 mm diameter storm sewer at the underpass and discharge to Appleby Creek Node APJ0150 downstream of Upper Middle Road. Flow which exceeds 1.08 m^3/s is considered as major flow and would contribute to Appleby Creek Node APJ0140 at Upper Middle Road.

Within Subcatchment AP0600 (main branch upstream of Lakeshore Road), minor flow from approximately 1.3 ha of the residential area at Bayfield Crescent would contribute to the storm sewer along Spruce Avenue through a 300 mm diameter storm sewer at a slope of 1.3%. The majority of the flow would travel south along Linwood Crescent and discharge to Appleby Creek east of Appleby Line through a combination of sewer and overland flow. There are approximately 6 catch basins that connect to the 300 mm sewer. Assuming each catch basin has an average inlet capacity of 0.06 m^3/s (as per the MTO Drainage Management Manual for on-grade catchbasins), the total maximum captured flow would be 0.36 m^3/s which would discharge to Appleby Creek at Spruce Avenue through the trunk storm sewer. The flows exceeding 0.36 m^3/s would discharge to Appleby Creek between Spruce Avenue and Lakeshore Road through a combination of storm sewer and overland flow. This split flow is represented by DUHYD APD0030 (NHYD90030). Six (6) inlets with a total maximum capture of 0.36 m^3/s have been assumed as the minor flow that would discharge to the storm sewer connecting at Spruce Avenue. Flow which exceeds 0.36 m^3/s is considered as major flow and would contribute to Appleby Creek Node APJ0600 at Lakeshore Road.

Within Subcatchment AP0630 (main branch downstream of Lakeshore Road), minor flow from approximately 1.93 ha of the residential area at Appleby Place would discharge to the Main Branch at Lakeshore Road via a 825 mm diameter pipe at a slope of 0.35%. The maximum capacity of the pipe has been determined to be 0.85 m^3/s using Manning's equation. This value has been used in order to split the minor and major components of the hydrograph accordingly. Flows exceeding 0.85 m^3/s would discharge directly to Lake Ontario. This split flow is represented by DUHYD APD0040 (NHYD90040). One inlet with a maximum capture of 0.85 m^3/s has been assumed as the minor flow that would discharge to Appleby Creek Node APJ0600 at Lakeshore Road. Flow which exceeds 0.85 m^3/s is considered as major flow and would discharge to the lake and be excluded from the system.

6.1.5 AREAL REDUCTION FACTORS

The limits of areal reduction factors (ARFs) for Appleby Watershed are presented on Drawing 11. ARFs have been calculated consistent with the methodology described in Section 3.7.4. As noted in Section 3.7.4, it has been agreed that the same ARFs are to be applied for Regional Storm event and design storm events. To summarize the findings for the Appleby Creek watershed:

In general, areas north of Upper Middle Road and the entire West Branch are within the 25 km^2 circular area and would therefore not require an ARF.

- Areas between Upper Middle Road and QEW for the East Branch are within the 45 km^2 circular area and an ARF of 99.2% has been applied.
- Areas between the QEW and Fairview Street are within the 65 km^2 circular area and an ARF of 98.2% has been applied.
- Areas between Fairview Street and New Street are within the 90 km^2 circular area and an ARF of 97.1% has been applied.
- Areas between New Street and Lake Ontario are within the 115 km^2 circular area and an ARF of 96.3% has been applied.

6.2 EXISTING CONDITIONS MODEL RESULTS

6.2.1 UNCALIBRATED MODEL RESULTS

Uncalibrated 100-Year Design Storm and Regional Storm Peak Flow Rates

The VO6 hydrologic model for Appleby Creek has been executed for the 12-hour SCS 100-year design storm event (existing 1964 to 2017 IDF), the 12-hour Regional Storm event under the AMC III (saturated) soil conditions, and the 48-hour Regional Storm event under the AMC II (normal) soil conditions. The peak flows at key locations have been summarized and presented in Table 6.4.

The results indicate that the 12-hour Regional Storm under the AMC III soil conditions and the 48 Hour Regional Storm under the AMC II conditions would generate similar peak flow rates. The governing storm event would be the 12-Hour Regional Storm under the AMC III soil conditions except for the nodes upstream of CNR at the West Branch.

Comparison of Simulated Peak Flows with Previous studies

The VO6 hydrologic model has been executed for the 3 Hour Chicago storm events and Regional Storm event to compare against the results presented in the 1997 Appleby Creek Floodline Mapping Update. To maintain consistency with the 2012 study, the IDF based on 27 years of data recorded at the Atmospheric Environment Royal Botanical Gardens Gauge and the time to peak ratio of 0.46 has been used for the comparison. Also, the ARFs from the 1997 study have been applied in this assessment to maintain consistency with the previous study. The comparison is presented in Tables 6.5 to 6.6.

The comparison indicates that the simulated 100-year peak flow rate generated from the VO6 model are generally lower than the peak flow rate generated from the 1997 GAWSER model along the upstream portions of the East Branch to Dundas Street (-11 to -39%), downstream portions of the West Branch from Appleby Line to the Confluence (-6 to -12%). Conversely 100-year storm peak flows from the VO6 model are greater along the downstream sections of the East Branch (downstream of Dundas Street; 5 to 17% higher), upper sections of the West Branch (to CN Halton; 10 to 17% higher), and most downstream sections of the Main Branch (between 2 and 16% higher).

The simulated Regional Storm flows are consistently higher than the peak flow rate generated from the 1997 GAWSER model. Differences on the East Branch range from 19 to 33% higher, whereas differences on the West Branch are slightly less (2% to 15% higher). Differences on the main branch are relatively consistent (15 to 18% higher).

The differences in the simulated peak flows are considered attributable to the modelling platform (GAWSER vs VO), parameterization methodology, and the minor differences in calculated drainage areas. Overall, the updated VO model is considered reasonably comparable to the previous study.

Table 6.4. Simulated Uncalibrated Peak Flows at Key Locations for Appleby Creek

LOCATION	ARF (%)	NODE	DRAINAGE AREA (ha)	PEAK FLOW RATE (m^3/s)		
				100 YEAR	12 HOUR REGIONAL (AMC III)	48 HOUR REGIONAL (AMC II)
East Branch						
D/S of Highway 407	100	APJ0070	271	17.9	31.7	31.0
Dundas Street	100	APJ0100	321	18.3	36.9	36.1
Millcroft Park Drive	100	APJ0130	441	27.3	49.3	48.5
Upper Middle Road	100	APJ0140	469	30.6	52.2	51.3
CNR	99.2	APJ0150	481	31.8	52.8	52.0
Appleby Line	99.2	APJ0160	533	35.1	58.3	57.5
Mainway	99.2	APJ0180	568	40.5	62.0	61.1
QEW	98.2	APJ0210	623	43.1	66.7	65.8
U/S of Confluence	98.2	APJ0220	638	44.0	68.1	67.1
West Branch						
Millcroft Park Drive	100	APJ0310	99	20.0	13.6	13.5
Upper Middle Road	100	APJ0320	120	21.7	16.4	16.4
CNR	100	APJ0330	146	22.7	19.5	19.4
Mainway	100	APJ0340	177	23.9	22.9	22.8
Appleby Line	100	APJ0350	196	24.1	24.9	24.8
QEW	100	APJ0370	229	26.0	28.7	28.5
U/S of Confluence	100	APJ0380	237	26.5	29.6	29.4
Main Branch						
Confluence of East and West Branch	98.2	APJ0230	875	68.5	94.2	93.1
CNR Oakville	98.2	APJ0240	920	71.7	98.8	97.7
D/S of Fairview Street	97.1	APJ0520	988	75.6	104.5	103.6
Pinedale Avenue	97.1	APJ0530	1,041	80.5	109.9	108.7
New Street	97.1	APJ0560	1,126	90.1	118.6	117.4
Spruce Avenue	96.3	APJ0590	1,163	93.8	121.6	120.4
Lakeshore Road	96.3	APJ0600	1,216	96.1	126.5	125.4
Lake Ontario	96.3	APJ0610	1,223	96.8	127.3	126.1

Table 6.5. Comparison of Simulated Peak Flows for Appleby Creek at Key Locations (1997 Study)

LOCATION	1997 APPLEBY CREEK FLOODLINE MAPPING UPDATE			2023 EAST BURLINGTON CREEKS FPM - UNCALIBRATED		
	ARF	3 HOUR CHICAGO 100 YEAR PEAK FLOW (m^3/s)	12 HOUR REGIONAL PEAK FLOW (m^3/s)	NODE	3 HOUR CHICAGO 100 YEAR PEAK FLOW (m^3/s)	12 HOUR REGIONAL PEAK FLOW (m^3/s)
East Branch						
Prop. Hwy 407	1	3.5	6.3	AP0070	3.1	8.2
Prop. Hwy 407-West Trib	1	9.8	18	APJ0060	7.4	23.6
Confluence - East & West Tribs	1	13	25	APJ0070	10.5	31.7
Dundas Street	1	17	31	APJ0100	10.3	36.9
Millcroft Park Drive	1	19	37	APJ0130	19.9	49.3
CN (Halton)	0.992	22	43	APJ0150	24.3	52.8
Appleby Line	0.992	23	48	APJ0160	26.3	58.3
Confluence - West Branch	0.982	27	56	APJ0220	31.7	68.1
West Branch						
Millcroft Park Drive	1	14	13	APJ0310	16.2	13.6
Upper Middle Road	1	15	15	APJ0320	17.5	16.4
CN (Halton)	1	16	17	APJ0330	17.6	19.5
Appleby Line	1	19	22	APJ0350	17.8	24.9
Confluence - East Branch	1	22	29	APJ0380	19.4	29.6
Main Branch						
Confluence - East & West Branch	0.982	48	81	APJ0230	48.7	94.2
CN (Oakville)	0.982	50	85	APJ0240	51.0	98.8
Pinedale Avenue	0.971	54	93	APJ0530	56.0	109.9
New Street	0.971	57	101	APJ0560	64.7	118.6
Spruce Avenue	0.971	59	106	APJ0590	68.0	122.7
Lake Ontario	0.963	61	109	APJ0610	70.9	127.3

1. For the purposes of this comparison, areal reduction factors and rainfall distributions from the previous study have been maintained. Values in this table may not be consistent with values presented in other sections of the report.

Table 6.6. differences in Simulated Peak Flow for Appleby Creek at Key Locations (2023 Study vs. 1997 Study)

LOCATION	ABSOLUTE DIFFERENCE (2023 VS. 1997)			PERCENT DIFFERENCE (2023 VS. 1997)		
	ARF	3 HOUR CHICAGO 100 YEAR PEAK FLOW (m^3/s)	12 HOUR REGIONAL PEAK FLOW (m^3/s)	NODE	3 HOUR CHICAGO 100 YEAR PEAK FLOW	12 HOUR REGIONAL PEAK FLOW (m^3/s)
East Branch						
Prop. Hwy 407	1	-0.4	+1.9	AP0070	-11.4%	+30.2%
Prop. Hwy 407-West Trib	1	-2.4	+5.6	APJ0060	-24.5%	+31.1%
Confluence - East & West Tribs	1	-2.5	+6.7	APJ0070	-19.2%	+26.8%
Dundas Street	1	-6.7	+5.9	APJ0100	-39.4%	+19.0%
Millcroft Park Drive	1	+0.9	+12.3	APJ0130	+4.7%	+33.2%
CN (Halton)	0.992	+2.3	+9.8	APJ0150	+10.5%	+22.8%
Appleby Line	0.992	+3.3	+10.3	APJ0160	+14.3%	+21.5%
Confluence - West Branch	0.982	+4.7	+12.1	APJ0220	+17.4%	+21.6%
West Branch						
Millcroft Park Drive	1	+2.2	+0.6	APJ0310	+15.7%	+4.6%
Upper Middle Road	1	+2.5	+1.4	APJ0320	+16.7%	+9.3%
CN (Halton)	1	+1.6	+2.5	APJ0330	+10.0%	+14.7%
Appleby Line	1	-1.2	+2.9	APJ0350	- 6.3%	+13.2%
Confluence - East Branch	1	-2.6	+0.6	APJ0380	-11.8%	+2.1%
Main Branch						
Confluence - East & West Branch	0.982	+0.7	+13.2	APJ0230	+1.5%	+16.3%
CN (Oakville)	0.982	+1.0	+13.8	APJ0240	+2.0%	+16.2%
Pinedale Avenue	0.971	+2.0	+16.9	APJ0530	+3.7%	+18.2%
New Street	0.971	+7.7	+17.6	APJ0560	+13.5%	+17.4%
Spruce Avenue	0.971	+9.0	+15.6	APJ0590	+15.3%	+14.7%
Lake Ontario	0.963	+9.9	+18.3	APJ0610	+16.2%	+16.8%

6.2.2 MODEL VALIDATION AGAINST AREA MONITORING DATA

Comparison of Simulated Peak Flows with Observed flows

In absence of any potential calibration data for the Appleby Creek watershed directly, the VO6 model has been validated using available data from the Fourteen Mile Creek, Hager-Rambo, and Morrison-Wedgewood watersheds, as described previously. Three (3) to four (4) different candidate storm events have been selected for model validation purposes from each of the three (3) watersheds. The VO6 modelling has been executed using the available rainfall for each of the selected events, including 14MC for the 14MC flow gauge, Tyandaga Reservoir and Burlington Fire Station 1 for the Hager-Rambo flow gauge, McCraney Reservoir for the Morrison/Wedgewood flow gauge, and Elizabeth Garden for the Sheldon Creek flow gauge. Peak flows at the watershed outlet (i.e. Lake Ontario) have then been extracted and normalized by area to enable a comparison between the datasets. Table 6.7 summarizes comparisons between unitary flows for selected events.

Table 6.7. Comparison of Observed and Simulated Unitary Peak Flows – Appleby Creek Model Validation

EVENT	TOTAL RAINFALL DEPTH (mm)	MAX HOURLY RAINFALL INTENSITY (mm/hr)	COMPARISON WATERSHED		APPLEBY CREEK AT LAKESHORE	
			PEAK FLOW RATE (m^3/s)	UNITARY PEAK FLOW RATE ($m^3/s/ha$)	PEAK FLOW RATE (m^3/s)	UNITARY PEAK FLOW RATE ($m^3/s/ha$)
Fourteen Mile Creek (Drainage Area = 24.5 km^2) Appleby Creek at Lakeshore (Drainage Area = 12.23 km^2)						
2005-07-26	53.5	165.0	9.4	0.004	83.4	0.068
2008-08-05	60.9	64.0	10.2	0.004	54.3	0.044
2009-06-25	22.9	41.6	3.9	0.002	18.8	0.015
2013-06-22	19.8	38.8	3.9	0.002	16.5	0.013
Hager-Rambo at QEW (Drainage Area = 16.13 km^2) Appleby Creek at Lakeshore (Drainage Area = 12.23 km^2)						
2019-05-25	32.6	65.6	22.6	0.014	21.0	0.017
2019-10-27	50.6	30.4	10.4	0.006	26.1	0.021
2021-08-26	24.0	34.4	18.6	0.012	17.2	0.014
2021-10-15	27.2	33.6	8.7	0.005	19.9	0.016
Morrison/Wedgewood Outlet (Drainage Area = 20.08 km^2) Appleby Creek at Lakeshore (Drainage Area = 12.23 km^2)						
2019-05-25	33.4	43.2	20.1	0.010	12.8	0.010
2019-10-27	35.0	14.4	13.1	0.007	14.4	0.012
2021-10-15	24.6	28.0	16.6	0.008	18.7	0.015

1. Note initial model validation was completed at an earlier stage of the project and as such presented results may differ slightly from those from the final modelling.

The results indicate that the simulated unitary peak flows within Appleby Creek Watershed are typically an order of magnitude higher compared with the observed unitary flows at the Fourteen Mile Creek flow gauge. A review of the available flow monitoring data for the Fourteen Mile Creek gauge indicates that this location generated notably lower runoff volumes than the simulated results for Appleby Creek, despite having double the watershed area. The Fourteen Mile Creek flow gauge results also indicate a poor correlation to the available rainfall, and an inconsistent watershed response relative to the storm events. Other factors may also have resulted in the difference, including differences in land use and soil conditions within the two watersheds, as well as the potential impact of stormwater management facilities in the Fourteen Mile Creek watershed.

Based on a cursory review, the simulated results for Appleby Creek for the selected events (from Fourteen Mile Creek) also appear reasonable. The July 26, 2005 storm event rainfall intensity would exceed a 100-year storm, and thus compares reasonably to 100-year peak flow using conventional design storms. The August 5, 2008 storm event has a rainfall intensity roughly equivalent to a 25-year storm event. The other two storm events are more nominal (less than a 2-year storm).

The validation comparison indicates that simulated peak flows from Appleby Creek are slightly higher but are comparable to the monitoring data at both the Hager-Rambo and Morrison/Wedgewood gauges. Peak flow responses tend to be similar during events with higher rainfall intensity.

Based on the preceding, the uncalibrated model results for Appleby Creek are considered valid. Notwithstanding, additional model comparisons have been undertaken, as described in Section 6.2.3.

6.2.3 ADDITIONAL MODEL COMPARISONS

Comparison of Unitary Peak Flows with Previous Studies

The simulated uncalibrated 100-year and Regional Storm unitary peak flows have been compared with various previous studies across Southern Ontario based on WSP's database of previous watershed and hydrologic studies, as well as data for Morrison/Wedgwood and Grindstone provided by CH for use in this study. Reference is made to Figures 3.2 and 3.3 and Tables 3.4 presented previously. For the Appleby Creek watershed, unitary flows of 0.080 and 0.104 m3/s/ha result; and a $Q_{Regional}/Q_{100}$ ratio of 1.31.

The comparison indicates that the simulated 100-year peak flow is generally higher than other studies in the Hamilton area (Waterdown, Red Hill Creek, Stoney and Battlefield Creeks) but is generally comparable to the results for the Morrison-Wedgewood Diversion Channel, which reflects a 24-hour Chicago Storm Event with SWM (rate of 0.094 m3/s/ha was indicated for the no SWM scenario, which would be more comparable to Appleby Creek given the lack of SWM for this watershed). Similar findings are noted for the Regional Storm Event, however the simulated results for Appleby Creek are somewhat closer to the other study results. The ratio of the Regional Storm peak flow to the 100-year peak flow is lower than the majority of the other study results with the exception of the Morrison-Wedgewood Diversion Channel, owing to the elevated simulated 100-year storm peak flow.

Based on the graphic presentation, the simulated results do not appear to exceed the common range of results, however they are towards the upper end of typical results. The results for the ratio of the Regional to 100-year storm also indicate that the simulated results are at the lower end of the ratio, indicating a relatively lesser difference between the two storm events as compared to other studies.

In summary, the uncalibrated hydrologic model for the Appleby Creek Watershed is generally consistent with the statistics and metrics from the nearby Morrison-Wedgewood diversion channel. Greater differences are indicated for the other subject watersheds, however differences may result based on the degree of urbanization and SWM controls, as well as hydrologic modelling techniques and differences (for instance some of the simulated 100-year peak flows were developed on the basis of continuous simulation rather than design storm events). Overall, the unitary flows and ratios are within the range of reasonable values.

August 4th 2014 Event

In addition to the preceding model validations, the uncalibrated hydrologic model for the Appleby Creek Watershed has also been executed for the August 4th, 2014 storm event. This storm event has been run for model validation purposes only but is not a regulatory storm event. The rainfall distribution presented previously in Table 3.10 has been conservatively applied for all subcatchments (no spatial or temporal variation) without any reduction factors. The simulation results and comparison with both the 100-year storm event (12-hour SCS) the 12-hour Regional Storm (AMC-III) peak flows (with areal reduction factors) are presented in Table 6.8.

Table 6.8. Simulated August 4th Storm and Regional Storm Peak Flows for Appleby Creek

LOCATION	ARF (%)	NODE	DRAINAGE AREA (ha)	PEAK FLOW RATE (m^3/s)		
				100 YEAR	12 HOUR REGIONAL (AMC III)	AUGUST 4TH 2014 (ARF=100%)
East Branch						
D/S of Highway 407	100	APJ0070	271	17.9	31.7	29.0
Dundas Street	100	APJ0100	321	18.3	36.9	34.4
Millcroft Park Drive	100	APJ0130	441	27.3	49.3	47.7
Upper Middle Road	100	APJ0140	469	30.6	52.2	50.4
CNR	99.2	APJ0150	481	31.8	52.8	51.5
Appleby Line	99.2	APJ0160	533	35.1	58.3	57.4
Mainway	99.2	APJ0180	568	40.5	62.0	61.4
QEW	98.2	APJ0210	623	43.1	66.7	67.3
U/S of Confluence	98.2	APJ0220	638	44.0	68.1	68.7
West Branch						
Millcroft Park Drive	100	APJ0310	99	20.0	13.6	17.0
Upper Middle Road	100	APJ0320	120	21.7	16.4	20.0
CNR	100	APJ0330	146	22.7	19.5	21.8
Mainway	100	APJ0340	177	23.9	22.9	24.4
Appleby Line	100	APJ0350	196	24.1	24.9	26.1
QEW	100	APJ0370	229	26.0	28.7	29.8
U/S of Confluence	100	APJ0380	237	26.5	29.6	30.2
Main Branch						
Confluence of East and West Branch	98.2	APJ0230	875	68.5	94.2	97.0
CNR Oakville	98.2	APJ0240	920	71.7	98.8	102.0
D/S of Fairview Street	97.1	APJ0520	988	75.6	104.5	109.2
Pinedale Avenue	97.1	APJ0530	1,041	80.5	109.9	114.9
New Street	97.1	APJ0560	1,126	90.1	118.6	124.4
Spruce Avenue	96.3	APJ0590	1,163	93.8	121.6	128.9
Lakeshore Road	96.3	APJ0600	1,216	96.1	126.5	134.0
Lake Ontario	96.3	APJ0610	1,223	96.8	127.3	134.7

The comparison indicates that the August 4th 2014 storm event would largely govern over the Regional Storm and the 100-year storm event within Appleby Creek Watershed. However, the Regional Storm would generate slightly higher peak flow along the East Branch. If areal reduction factors were applied to this storm as well however, the results would be more closely comparable for the Main Branch in particular. The 100-year storm event also is noted to govern for the upper reaches of the West Branch.

6.3 FUTURE CONDITIONS MODEL RESULTS

6.3.1 MODEL UPDATES

The subcatchment total and directly connected impervious coverage have been updated by adding identified additional areas (as per Section 3.4.2 and Drawings 4a and 4b) into the base land use mapping layer; values have been developed consistent with the approach for existing land use as per CH's Table of standard values (Table 7) corresponding to each land use type. The land cover types in the Urban Burlington Land Cover layer have been categorized into the groups outlined in CH's standard parameter Table 7. Proposed total impervious coverage values and direct impervious coverage values corresponding to each land use type are included in Appendix A. A comparison of the overall changes is presented in Table 6.9.

Table 6.9. Comparison of Impervious Coverage between Existing and Future Conditions

LOCATION	1997 APPLEBY CREEK EROSION CONTROL CLASS EA	2023 EAST BURLINGTON CREEKS FLOODPLAIN MAPPING (EXISTING CONDITIONS)	2023 EAST BURLINGTON CREEKS FLOODPLAIN MAPPING (FUTURE CONDITIONS)	DIFFERENCE FROM EXISTING CONDITIONS
West Branch	40.8%	58.2%	60.5%	+2.3%
East Branch	12.0%	37.2%	39.4%	+2.2%
Total	25.3%	50.6%	51.9%	+1.3%

The comparison indicates that the overall increases in imperviousness throughout the watershed are 1% due to the future land use changes. The changes are considered marginal. The east and west branches would have nominally higher increases of 2% as the majority of the expected development occurs upstream of the confluence of the two tributaries.

In addition, under future conditions the future rainfall IDF has been applied (current City of Burlington IDF which incorporates an adjustment to account for climate change, as discussed in Section 3.7). This would be expected to further increase the simulated peak flows for the 100-year storm event. Model results are noted in subsequent sections accordingly.

6.3.2 MODEL RESULTS

Simulated peak flows under future conditions are presented in Table 6.10 (without spills). Note that the presented flows do not include spills, if applicable; this is reviewed further in Section 6.3.3.

While existing and future data are presented in this report, calculated differences in results should not be interpreted as reassessing or demonstrating the impacts of future development. The climate change adjusted IDF has only been applied to define the future 1:100-year flow data and has not been applied to existing conditions. Also, to support model calibration, the existing conditions modeling represents the current watershed condition. This may include centralized SWM controls that are designed to provide attenuation for a future development condition identified in the Official Plan but where the proposed development is not fully built out. In these areas, the existing conditions model assumes existing land uses where development has not yet occurred and may therefore predict existing condition flow rates less than pre-development conditions.

Table 6.10. Simulated Future Conditions Peak Flows at Key Locations for Appleby Creek

LOCATION	ARF (%)	NODE	DRAINAGE AREA (ha)	PEAK FLOW RATE (m^3/s)		
				100 YEAR	12 HOUR REGIONAL (AMC III)	48 HOUR REGIONAL (AMC II)
East Branch						
D/S of Highway 407	100	APJ0070	271	19.1	31.7	31.0
Dundas Street	100	APJ0100	321	19.9	36.9	36.1
Millcroft Park Drive	100	APJ0130	441	29.4	49.3	48.5
Upper Middle Road	100	APJ0140	469	32.9	52.2	51.3
CNR	99.2	APJ0150	481	34.0	52.8	52.0
Appleby Line	99.2	APJ0160	533	37.5	58.3	57.4
Mainway	99.2	APJ0180	568	43.6	62.0	61.0
QEW	98.2	APJ0210	623	47.1	66.7	65.7
U/S of Confluence	98.2	APJ0220	638	47.9	68.0	67.0
West Branch						
Millcroft Park Drive	100	APJ0310	99	21.0	13.6	13.5
Upper Middle Road	100	APJ0320	120	22.8	16.4	16.4
CNR	100	APJ0330	146	24.0	19.5	19.4
Mainway	100	APJ0340	177	25.1	22.9	22.8
Appleby Line	100	APJ0350	196	25.4	24.9	24.7
QEW	100	APJ0370	229	27.4	28.7	28.5
U/S of Confluence	100	APJ0380	237	27.8	29.5	29.4
Main Branch						
Confluence of East and West Branch	98.2	APJ0230	875	73.7	94.1	93.0
CNR Oakville	98.2	APJ0240	920	77.6	98.7	97.6
D/S of Fairview Street	97.1	APJ0520	988	83.6	104.4	103.3
Pinedale Avenue	97.1	APJ0530	1,041	90.3	109.8	108.6
New Street	97.1	APJ0560	1,126	100.6	118.5	117.3
Spruce Avenue	96.3	APJ0590	1,163	104.3	121.4	120.7
Lakeshore Road	96.3	APJ0600	1,216	106.4	126.4	125.8
Lake Ontario	96.3	APJ0610	1,223	107.0	127.4	126.6

For simulation of the Regional Storm Event minor decreases in flow are indicated in three (3) locations, which may reflect hydrograph timing effects. The 12-hour Regional Storm remains the Regulatory Storm Event in most locations other than the upper portion of the West Branch, where the 100-year event would govern.

The peak flows presented in Table 6.10 have been applied as the basis for the flood hazard mapping (notwithstanding any potential additional spill flows, as described in Section 6.3.3). Flood hazard mapping is described further in the separate hydraulic modelling report.

6.3.3 ASSESSMENT OF SPILLS

The simulated future condition flows have been applied to the hydraulic modelling (both 1D and 2D) as described in the separate Hydraulics Report. As noted in that assessment, numerous spills have been identified. In order to apply the "balanced approach" proposed by CH (refer to CH's technical memorandum of May 19, 2022 as included within the Hydraulics Report), integration between the 2D hydraulic modelling and hydrologic modelling is necessary. Spill flows are to be included within the watershed receiving the spill. Iteration between the 2D hydraulic modelling and hydrologic modelling has been required to identify inter-watershed spills specifically that meet the threshold for inclusion, and to "balance" flows such that there is reasonable agreement between hydrologic and hydraulic modelling results.

For the Appleby Creek watershed, two (2) intra-watershed spills had been identified from adjacent branches of Appleby to the west branch. The estimated magnitude of the spills is approximately 5.4 m^3/s and 1.1 m^3/s, and have therefore been added into the VO modelling at their appropriate locations. Upon review of the updated modelling results, it was determined that with the inclusion of the spill flows, there was no change to the peak flows throughout the west branch. This is due to the peak of the spill flow hydrographs not coinciding with the peak of the west branch, therefore negating the effect of the intra-basin spills on the Regional Storm flows. Through discussions with CH it was agreed that for this reason, the "base" conditions model should be carried forward as the source of input into the hydraulic modelling, as there are no spills which meet the criteria for inclusion or that would impact the regulatory results.

7 SHELDON CREEK

7.1 MODEL DEVELOPMENT

7.1.1 SUBCATCHMENT BOUNDARIES

The Sheldon Creek watershed has a total drainage area of approximately 18.23 km^2 including all split flow drainage areas (approximately 17.48 km^2 excluding these areas). Subcatchment boundaries for Sheldon Creek have been developed using the approach summarized in Section 3.1.2. Statistics are presented in Table 7.1. The boundaries are presented graphically in Drawing 12a (attached).

Table 7.1. Proposed Subcatchment Boundaries for Sheldon Creek

WATERSHED	TOTAL NUMBER OF SUBCATCHMENTS	MINIMUM SUBCATCHMENT SIZE (ha)	MAXIMUM SUBCATCHMENT SIZE (ha)	AVERAGE SUBCATCHMENT SIZE (ha)	STANDARD DEVIATION (ha)
Sheldon Creek	67	0.48	90.19	27.21	22.79

Contributing drainage areas at key locations have been compared with the 2019 Sheldon Creek Watershed Hydrologic and Hydraulic Study. The comparison is summarized in Table 7.2. A graphical comparison of the boundaries is presented in Drawing 12b (attached).

As discussed in the following section, a split flow would occur on the East Branch at Mainway. In the previous (2019) study, this area was diverted entirely towards Bronte Creek, which is the reason for the difference in drainage area at this location. Further, there are additional identified drainage areas between Fairview Street and New Street along the Main Branch which tend to increase the total identified contributing drainage area. When considering these primary differences, the drainage areas resulted from the updated subcatchment boundaries are comparable to the previous study, with the difference of only 2% when excluding split areas at Lake Ontario.

Table 7.2. Comparison of Drainage Areas with Previous Study for Sheldon Creek

LOCATION	2019 SHELDON CREEK HYDROLOGY		2023 EAST BURLINGTON CREEKS FPM		DRAINAGE AREA DIFFERENCES (2023 VS. 2019)	
	NODE	DRAINAGE AREA (ha)	NODE	DRAINAGE AREA (ha)	ABSOLUTE (ha)	%
West Branch - West Tributary						
North Of Dundas St	162.1	225	SDSU804	220	-5	-2.2%
Appleby Line	110.2	279	SDJ0060	273	-6	-2.2%
West Branch - East Tributary						
Dundas St	194.1	194	SDJ0140	186	-9	-4.4%
North of CNR-Halton	141.1	276	SDJ0250	271	-5	-1.7%
West Branch						
Confluence Point / South of Upper Middle Rd	108.2	705	SDJ0350	708	+3	+0.4%
Mainway	108.1	747	SDJ0390	733	-14	-1.9%
QEW	106.1	879	SDJ0430	860	-19	-2.2%
East Branch						
Upper Middle Rd	208.1	114	SDJ0640	111	-2	-1.8%
Mainway	207.1	172	SDJ0660	209 (148)	+37 (-24)	+22% (-14%)
QEW	206.1	212	SDJ0670	260 (198)	+48 (-14)	+23% (-7%)
Main Branch						
Great Lakes Blvd	101.2	1,605	SDJ0840	1,678 (1,617)	+73 (+12)	+4.5% (+0.7%)
Lake Ontario	101.1	1,737	SDJ0910	1,823 (1,748)	+86 (+11)	+5.0% (+0.6%)

Note: numbers in brackets represent areas excluding the DUHYD subcatchments.

7.1.2 SUBCATCHMENT PARAMETERIZATION

Based on the subcatchment delineation, subcatchment parameterization has been established following the approach described in Section 3.4. A summary of the uncalibrated subcatchment parameters for Sheldon Creek is included in Appendix H.

Subcatchment Slope

The surface slopes within the Sheldon Creek Watershed tend to be moderate between 1 and 4%. The undeveloped areas north of Highway 407 are slightly steeper with the average slope greater than 3%.

Impervious Coverage

The land use conditions north of Dundas Street are primarily agricultural lands, open space, and forest, with rural residential areas distributed along major roads. The areas south of Dundas Street are largely developed and the land use conditions are a mix of urban residential and high-density residential areas, high impervious areas, institutional areas, industrial areas, commercial areas, parks and open lands, as well as dispersed forests. Between

Upper Middle Road and Mainway west of Burloak Drive, the land use is primarily Agricultural and open space, and woodlot. A comparison to the previous study is presented in Table 7.3.

Table 7.3. Comparison of Modelled Imperviousness to Previous Studies for Sheldon Creek

LOCATION	2019 SHELDON CREEK HYDROLOGY	2023 EAST BURLINGTON CREEKS FLOODPLAIN MAPPING	DIFFERENCE
East Branch	52.0%	51.0%	-1.0%
West Branch	43.0%	48.9%	+5.9%
Total	45.4%	49.1%	+3.7%

As evident from Table 7.3, impervious coverages are reasonably consistent between the two (2) studies, with differences ranging from -1.0% to +5.9%. This generally reflects the more recent vintage of the hydrologic modelling in this case as compared to the other three (3) watersheds.

Infiltration

The soils consist predominantly of Clay Loam (76%), of which 13% is in the rocky phase. The remaining portion of the soils within the watershed consists of loam (1%), sandy loam (2%), and urban built-up areas (22%). Overall, the soils are largely classified as SCS Type 'C' and Type "D" soils, exhibiting low permeability and low infiltration potential with high potential for generating runoff.

SCS Curve Numbers have been applied on the basis of representative values for the pervious land segment. In particular for urbanized areas that utilized the STANDHYD routine, given that impervious coverage is accounted for separately, the CN value represents the solely pervious land segment. As an example, for a residential area, the SCS CN represents the grassed/lawn areas based on the applicable soils.

7.1.3 SWM FACILITIES

Based on the completed "SWM Pond Review" report (refer to Appendix B), a total of five (5) quantity control facilities have been proposed for inclusion in the hydrologic modelling of Shoreacres Creek. These facilities are presented on Drawings 5 and 12a (attached). They include:

- **Pond 804** (included up to 100-year storm event; new rating curve developed and applied)
- **Pond 823** (included up to the Regional Storm Event; design rating curve applied)
- **Pond 824** (included up to 100-year storm event; design rating curve applied)
- **Pond 825** (included up to 100-year storm event; design rating curve applied)
- **Pond 826** (included up to 100-year storm event; design rating curve applied)

Note that Pond 808 (located near Upper Middle Road and Sutton Drive) has not been recommended for inclusion in the hydrologic modelling for the 100-year storm event as it was indicated as over-capacity for this storm, with the spill expected to be re-directed to other receivers based on the grades on Upper Middle Road (drains westerly). The function could however be included for less formative storm events (i.e. 2 to 50-year storm events) but has not been assessed for performance or included in the current hydrologic modelling under these events. This should be considered as part of future study.

Refer to Appendix B for further details on the proposed rating curves and details on the quantity control facilities.

7.1.4 MAJOR/MINOR SPLIT

Subcatchment SD0660 (Mainway) has been identified as having a split flow condition based on the City's storm sewer database, available topographic data, and field visits conducted by CH. Minor flow would discharge to Sheldon Creek through a 675 mm diameter storm sewer with 1.4% slope. The maximum captured flow that would travel westly via the 675mm sewer and discharge to Sheldon Creek at Node SDJ0660 has been determined to be 1 m^3/s based on Manning's equation. Flow which exceeds 1 m^3/s is considered as major flow and would contribute to the Bronte Creek watershed.

Within subcatchment SD0900, minor flow from approximately 13.56 ha of the residential area south of Rebecca Street would contribute to the storm sewer along Chalmers Street through a 1200 mm diameter storm sewer at a slope of 0.06% and would discharge to Sheldon Creek. The majority of the flow would travel south along Mohawk Rd and discharge through a combination of sewer and overland flow. The total maximum captured flow of 0.96 m^3/s has been assumed as the minor flow based on a Manning's Equation calculation and would discharge to Node SDJ0890. Flow which exceeds 0.96 m^3/s is considered as major flow and would discharge to Bronte Creek.

Calculations of spilt flows are included in Appendix H along with relevant drawings.

7.1.5 AREAL REDUCTION FACTORS

The limits of areal reduction factors (ARFs) for the Sheldon Watershed are presented on Drawing 13 (attached). ARFs have been calculated consistent with the methodology described in Section 3.7.4. As noted in Section 3.7.4, it has been agreed that the same ARFs are to be applied for Regional Storm event and design storm events. To summarize the findings for the Sheldon Creek watershed:

- In general, areas north of Upper Middle Road are within the 0 to 25 km^2 circular area and would therefore not require an ARF.
- Areas between Upper Middle Road and Harvester Road are within the 26 to 45 km^2 circular area, an ARF of 99.2% has been applied.
- Areas between Harvester Road and New Street are within the 46 to 65 km^2 circular area, an ARF of 98.2% has been applied.
- Areas between New Street and outfall at Lake Ontario is within the 66 to 90 km^2 circular area, an ARF of 97.1% has been applied.
- On the East Branch, areas north of Rebecca Street are within the 0 to 25 km^2 circular area and would therefore not require an ARF.
- On the East Branch, areas between Rebecca Street and the confluence with Main Branch are within the 26 to 45 km^2 circular area and an ARF of 99.2% has been applied.

7.2 EXISTING CONDITIONS MODEL RESULTS

7.2.1 UNCALIBRATED MODEL RESULTS

Uncalibrated 100-Year Design Storm and Regional Storm Peak Flow Rates

The VO6 hydrologic model for Sheldon Creek Watershed has been executed for the 100-year storm event, the 12-hour Regional Storm event under the AMC III (saturated) soil conditions, and the 48-hour Regional Storm event under the AMC II (normal) soil conditions. The peak flows at key locations have been summarized and presented in Table 7.4.

The results indicate that the 12-hour Regional Storm under the AMC III soil conditions and the 48 Hour Regional Storm under the AMC II conditions would generate similar peak flow rates. The governing storm event would be the 12-Hour Regional Storm under the AMC III soil conditions throughout the entire watershed.

Comparison of Simulated Peak Flows with Previous Studies

The VO6 hydrologic model has been executed to compare against the results of the 2019 Sheldon Creek Watershed Hydrologic and Hydraulic Study. Only Regional Storm peak flows have been compared since the 2019 Sheldon Creek analyzed frequency flows from continuous simulation rather than design storm event peak flows, which may not result in an appropriate comparison. The results are presented in Table 7.5.

The comparison indicates that the simulated Regional Storm peak flow rate generated from the VO6 model are consistently 14 to 37% higher than the peak flow rate generated from the 2019 HSP-F model across the entire watershed, with the exception of East Branch at Mainway, where the simulated flow rate is slightly less (-11%). The differences in the simulated peak flows are considered attributable to the different modelling platforms, parameterization methodology, and differences in contributing areas.

Table 7.4. Simulated Uncalibrated Design Storm and Regional Storm Peak Flows at Key Locations for Sheldon Creek

LOCATION	ARF (%)	NODE	DRAINAGE AREA (ha)	PEAK FLOW RATE (m^3/s)		
				100 YEAR	12 HOUR REGIONAL (AMC III)	48 HOUR REGIONAL (AMC II)
West Branch - West Tributary						
North Of Dundas St	100	SDC0040	220	9.4	26.8	26.4
Appleby Line	100	SDC0050	273	12.4	33.2	32.8
North of Upper Middle Rd	100	SDJ0090	345	17.0	38.4	38.0
West Branch - East Tributary						
South Of Dundas St	100	SDJ0140	186	30.6	29.5	29.1
North of CNR-Halton	100	SDJ0250	271	34.5	33.0	32.6
Dryden Avenue	100	SDJ0260	300	33.9	36.1	35.7
North of Upper Middle Rd	100	SDJ0300	357	35.2	41.8	41.4
West Branch						
Confluence Point	100	SDJ0350	708	54.8	84.4	83.5
Mainway	99.2	SDJ0390	733	49.1	85.1	84.2
QEW	99.2	SDJ0430	860	56.1	98.6	97.6
South of New Street	97.1	SDJ0470	1,060	61.8	116.8	115.6
Spruce Avenue	97.1	SDJ0480	1,116	62.6	122.4	121.2
Burloak Drive	97.1	SDJ0490	1,133	63.0	124.1	122.8
East Branch						
Upper Middle Rd	100	SDJ0640	111	20.6	15.3	15.3
Mainway	100	SDJ0650	140	19.1	18.2	18.1
QEW	100	SDJ0670	260	23.9	26.1	25.9
CNR Oakville	100	SDJ0730	343	27.5	35.3	35.0
South of Rebecca Street	100	SDJ0800	454	28.2	46.7	46.3
Main Branch						
Confluence Point	97.1	SDJ0830	1,631	88.9	172.0	170.4
Great Lakes Blvd	97.1	SDJ0840	1,678	90.1	176.8	175.2
South of Lakeshore Road West	96.3	SDJ0890	1,816	94.5	188.6	186.6
Lake Ontario	96.3	SDJ0910	1,823	94.7	189.1	187.1

Table 7.5. Comparison of Simulated Peak Flows for Sheldon Creek at Key Locations (2019 Study)

LOCATION	2019 SHELDON CREEK WATERSHED HYDROLOGIC AND HYDRAULIC UPDATE STUDY		2023 EAST BURLINGTON CREEKS FPM		DIFFERENCE	
	NODE	48 HOUR REGIONAL PEAK FLOW (m^3/s)	NODE	48 HOUR REGIONAL PEAK FLOW (m^3/s)	(m^3/s)	(%)
West Branch - West Tributary						
North Of Dundas St	162.1	21.9	SDC0040	26.4	+4.5	+21%
Appleby Line	110.2	28.8	SDC0050	32.8	+4.0	+14%
West Branch - East Tributary						
Dundas St	194.1	21.2	SDJ0140	29.1	+7.9	+37%
North of CNR-Halton	141.1	28.0	SDJ0250	32.6	+4.6	+16%
West Branch						
Confluence Point / South of Upper Middle Rd	108.2	67.5	SDJ0350	83.5	+16.0	+24%
Mainway	108.1	70.4	SDJ0390	84.2	+13.8	+20%
QEW	106.1	81.5	SDJ0430	97.6	+16.1	+20%
New Street	104.1	97.7	SDJ0470	115.6	+17.9	+18%
East Branch						
Upper Middle Rd	208.1	13.6	SDJ0640	15.3	+1.7	+13%
Mainway	207.1	20.3	SDJ0650	18.1	-2.2	-11%
QEW	206.1	24.6	SDJ0670	25.9	+1.3	+5%
Main Branch						
Great Lakes Blvd	101.2	144.0	SDJ0840	175.2	+31.2	+22%
Lake Ontario	101.1	147.0	SDJ0910	187.1	+40.1	+27%

1. For the purposes of this comparison, areal reduction factors and rainfall distributions from the previous study have been maintained. Values in this table may not be consistent with values presented in other sections of the report.

7.2.2 MODEL VALIDATION AGAINST AREA MONITORING DATA

Model Validation against Comparable Watersheds

In the absence of extensive calibration data for the Sheldon Creek watershed, the VO6 model has been validated using available data from the Fourteen Mile Creek, Hager-Rambo, and Morrison-Wedgewood watersheds, as described previously. Three (3) to four (4) different candidate storm events have been selected for model validation purposes from each of the three (3) watersheds. The VO6 modelling has been executed using the available rainfall for each of the selected events, including 14MC for the 14MC flow gauge, Tyandaga Reservoir and Burlington Fire Station 1 for the Hager-Rambo flow gauge, McCraney Reservoir for the Morrison/Wedgewood flow gauge, and Elizabeth Garden for the Sheldon Creek flow gauge. Peak flows at the watershed outlet (i.e. Lake Ontario) have then been extracted and normalized by area to enable a comparison between the datasets. Table 7.6 summarizes comparisons between unitary flows for selected events.

Table 7.6. Comparison of Observed and Simulated Unitary Peak Flows – Sheldon Creek Model Validation

EVENT	TOTAL RAINFALL DEPTH (mm)	MAX HOURLY RAINFALL INTENSITY (mm/hr)	COMPARISON WATERSHED		SHELDON CREEK AT LAKESHORE	
			PEAK FLOW RATE (m^3/s)	UNITARY PEAK FLOW RATE ($m^3/s/ha$)	PEAK FLOW RATE (m^3/s)	UNITARY PEAK FLOW RATE ($m^3/s/ha$)
Fourteen Mile Creek (Drainage Area = 24.5 km^2) Sheldon Creek at Lakeshore (Drainage Area = 18.24 km^2)						
2005-07-26	53.5	165.0	9.4	0.004	67.2	0.036
2008-08-05	60.9	64.0	10.2	0.004	56.6	0.031
2009-06-25	22.9	41.6	3.9	0.002	15.7	0.008
2013-06-22	19.8	38.8	3.9	0.002	13.7	0.007
Hager-Rambo at QEW (Drainage Area = 16.13 km^2) Sheldon Creek at Lakeshore (Drainage Area = 18.24 km^2)						
2019-05-25	32.6	65.6	22.6	0.014	20.4	0.011
2019-10-27	50.6	30.4	18.6	0.012	28.6	0.015
2021-08-26	24.0	34.4	10.4	0.006	15.0	0.008
2021-10-15	27.2	33.6	8.7	0.005	17.2	0.009
Morrison/Wedgewood Outlet (Drainage Area = 20.08 km^2) Sheldon Creek at Lakeshore (Drainage Area = 18.24 km^2)						
2019-05-25	33.4	43.2	20.1	0.010	12.7	0.007
2019-10-27	35.0	14.4	13.1	0.007	17.1	0.009
2021-10-15	24.6	28.0	16.6	0.008	16.3	0.009

1. Note initial model validation was completed at an earlier stage of the project and as such presented results may differ slightly from those from the final modelling.

The results indicate that the simulated unitary peak flows within Sheldon Creek Watershed are typically an order of magnitude higher compared with the observed unitary flows at the Fourteen Mile Creek flow gauge. A review of the available flow monitoring data for the Fourteen Mile Creek gauge indicates that this location generated notably lower runoff volumes than the simulated results for Sheldon Creek, despite having double the watershed area. The Fourteen Mile Creek flow gauge results also indicate a poor correlation to the available rainfall, and an inconsistent watershed response relative to the storm events. Other factors may also have resulted in the difference, including differences in land use and soil conditions within the two watersheds, as well as the potential impact of stormwater management facilities in the Fourteen Mile Creek watershed.

Based on a cursory review, the simulated results for Sheldon Creek for the selected events (from Fourteen Mile Creek) also appear reasonable. The July 26, 2005 storm event rainfall intensity would exceed a 100-year storm, and thus compares reasonably to 100-year peak flow using conventional design storms. The August 5, 2008 storm event has a rainfall intensity roughly equivalent to a 25-year storm event. The other two storm events are more nominal (less than a 2-year storm).

The validation comparison indicates that simulated peak flows from Sheldon Creek are slightly higher but are comparable to the monitoring data at both the Hager-Rambo and Morrison/Wedgewood gauges. Peak flow responses tend to be similar during events with higher rainfall intensity.

Direct Model Validation

The storm event of October 15, 2021 was monitored at CH's recently installed gauge located in Shell Park; the data has been provided for use in the current validation. Figure 7.1 presents the water level response during the event (a rating curve has not been established or provided). The total rainfall was 22.85 mm over 5 hours, with the average intensity of 4.57 mm/hr and peak intensity of 43.7 mm/hr.

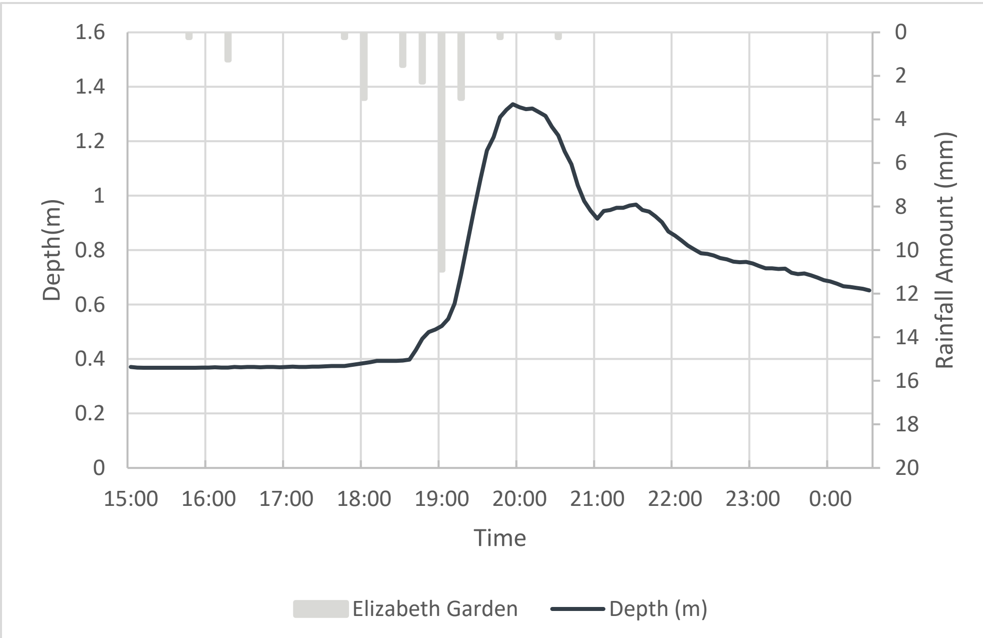


Figure 7.1. Observed Water Level at Sheldon Creek (Oct 15, 2021)

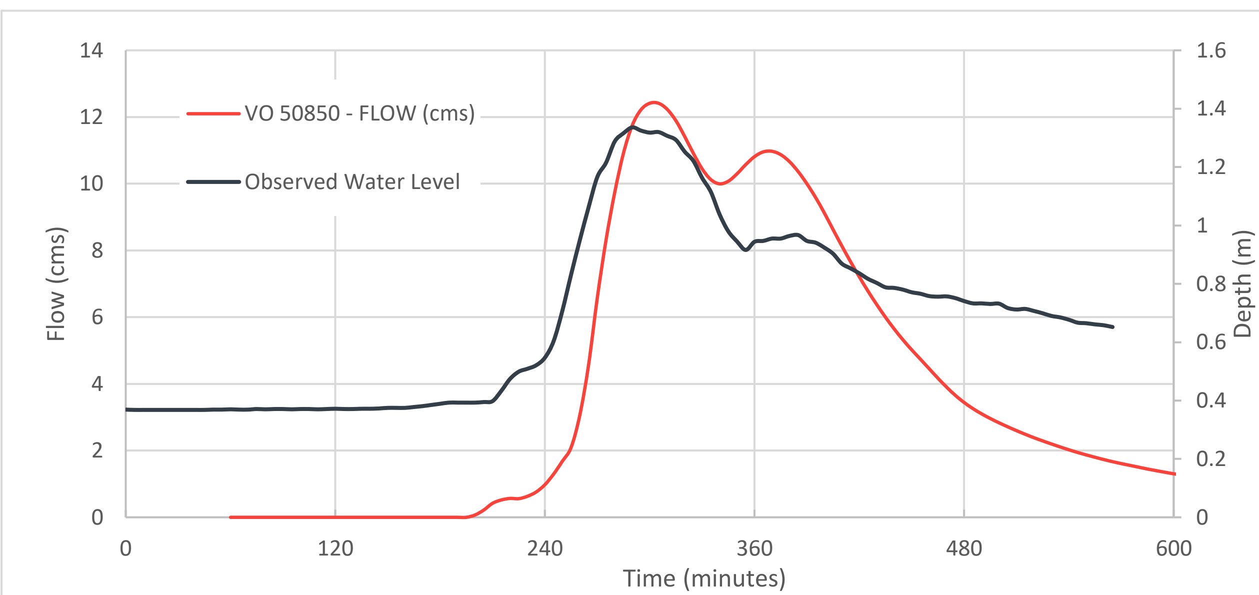


Figure 7.2. Observed Water Level and Simulated Flow at Sheldon Creek (Oct 15, 2021)

Figure 7.2 compares the responses of water level and simulated flow during the event of Oct 15, 2021. The simulated hydrograph response has been shifted by 1 hour to better align with the observed peak flow response as it is unknown whether or not the observed water level and rainfall data were provided in a consistent time format (i.e. correction for daylight savings time or not). The secondary peak in the observed response is generally larger than that from the modelling results, however this may be attributable to spatial differences in the rainfall event.

Based on the preceding, the uncalibrated model results for Sheldon Creek are considered valid. Notwithstanding, additional model comparisons have been undertaken, as described in Section 7.2.3.

7.2.3 ADDITIONAL MODEL COMPARISONS

Comparison of Unitary Peak Flows with Previous Studies

The simulated uncalibrated 100-year and Regional Storm unitary peak flows have been compared with various previous studies across Southern Ontario based on WSP's database of previous watershed and hydrologic studies, as well as data for Morrison/Wedgwood and Grindstone provided by CH for use in this study. Reference is made to Figures 3.2 and 3.3 and Tables 3.4 presented previously. For the Sheldon Creek watershed, 100-year and Regional Storm unitary flows of 0.048 and 0.104 $m^3/s/ha$ result; and a $Q_{Regional}/Q_{100}$ ratio of 1.87.

The comparison indicates that the simulated 100-year peak flow is generally higher than other studies in the Hamilton area (Waterdown, Red Hill Creek, Stoney and Battlefield Creeks) but is generally comparable to the results for the Morrison-Wedgewood Diversion Channel, which reflects a 24-hour Chicago Storm Event with SWM (rate of 0.094 $m^3/s/ha$ was indicated for the no SWM scenario, which would be more comparable to Sheldon Creek given the lack of SWM for this watershed). Similar findings are noted for the Regional Storm Event, however the simulated results for Sheldon Creek are somewhat closer to the other study results. The ratio of the Regional Storm peak flow to the 100-year peak flow is lower than the majority of the other study results with the exception of the Morrison-Wedgewood Diversion Channel, owing to the elevated simulated 100-year storm peak flow.

Based on the graphic presentation, the simulated results do not appear to exceed the common range of results, however they are towards the upper end of typical results. The results for the ratio of the Regional to 100-year storm also indicate that the simulated results are at the lower end of the ratio, indicating a relatively lesser difference between the two storm events as compared to other studies.

In summary, the uncalibrated hydrologic model for the Sheldon Creek Watershed is generally consistent with the statistics and metrics from the nearby Morrison-Wedgewood diversion channel. Greater differences are indicated for the other subject watersheds, however differences may result based on the degree of urbanization and SWM controls, as well as hydrologic modelling techniques and differences (for instance some of the simulated 100-year peak flows were developed on the basis of continuous simulation rather than design storm events). Overall, the unitary flows and ratios are within the range of reasonable values.

August 4th 2014 Event

In addition to the preceding model validations, the uncalibrated hydrologic model for the Sheldon Creek Watershed has also been executed for the August 4th, 2014 storm event. This storm event has been run for model validation purposes only but is not a regulatory storm event. The rainfall distribution presented previously in Table 3.10 has been conservatively applied for all subcatchments (no spatial or temporal variation) without any reduction factors. The simulation results and comparison with both the 100-year storm event (12-hour SCS) the 12-hour Regional Storm (AMC-III) peak flows (with areal reduction factors) are presented in Table 7.7.

Table 7.7. Simulated August 4th 2014 Storm and Regional Storm Peak Flows for Sheldon Creek

LOCATION	ARF (%)	NODE	DRAINAGE AREA (ha)	PEAK FLOW RATE (m^3/s)		
				100 YEAR	12 HOUR REGIONAL (AMC III)	AUGUST 4TH 2014
West Branch- West Tributary						
North Of Dundas St	100	SDC0040	220	9.4	26.8	25.9
Appleby Line	100	SDC0050	273	12.4	33.2	32.5
North of Upper Middle Rd	100	SDJ0090	345	17.0	38.4	37.3
West Branch -East Tributary						
South Of Dundas St	100	SDJ0140	186	30.6	29.5	29.9
North of CNR-Halton	100	SDJ0250	271	34.5	33.0	33.3
Dryden Avenue	100	SDJ0260	300	33.9	36.1	35.9
North of Upper Middle Rd	100	SDJ0300	357	35.2	41.8	41.4
West Branch						
Confluence Point	100	SDJ0350	708	54.8	84.4	82.6
Mainway	99.2	SDJ0390	733	49.1	85.1	84.5
QEW	99.2	SDJ0430	860	56.1	98.6	98.0
South of New Street	97.1	SDJ0470	1,060	61.8	116.8	117.4
Spruce Avenue	97.1	SDJ0480	1,116	62.6	122.4	123.0
Burloak Drive	97.1	SDJ0490	1,133	63.0	124.1	124.7
East Branch						
Upper Middle Rd	100	SDJ0640	111	20.6	15.3	19.0
Mainway	100	SDJ0650	140	19.1	18.2	18.9
QEW	100	SDJ0670	260	23.9	26.1	26.6
CNR Oakville	100	SDJ0730	343	27.5	35.3	33.5
South of Rebecca Street	100	SDJ0800	454	28.2	46.7	43.9
Main Branch						
Confluence Point	97.1	SDJ0830	1.631	88.9	172.0	171.1
Great Lakes Blvd	97.1	SDJ0840	1,678	90.1	176.8	176.2
South of Lakeshore Road West	96.3	SDJ0890	1,816	94.5	188.6	189.7
Lake Ontario	96.3	SDJ0910	1,823	94.7	189.1	190.0

The Regional Storm would largely govern over the August 4th 2014 storm event within Sheldon Creek Watershed. However, the August 4th 2014 storm event would generate slightly higher peak flows on the upper portion of the West Branch (East Tributary) and the East Branch at CNR Oakville, which may be attributable to hydrograph timing factors.

Some results for the August 4, 2014 storm event indicate decreasing peak flows with increasing area (i.e. upper sections of the west and east branches); this may reflect the peaky nature of the storm and the impact of routing elements.

There are three (3) locations where the 100-year storm event would govern over the Regional Storm, these reflect upstream areas with smaller drainage areas which are typically more responsive to a more peaked/intense 100-year storm event than the Regional Storm.

7.3 FUTURE CONDITIONS MODEL RESULTS

7.3.1 MODEL UPDATES

The subcatchment total and directly connected impervious coverage have been updated by adding identified additional areas (as per Section 3.4.2 and Drawings 4a and 4b) into the base land use mapping layer; values have been developed consistent with the approach for existing land use as per CH's Table of standard values (Table 7) corresponding to each land use type. The land cover types in the Urban Burlington Land Cover layer have been categorized into the groups outlined in CH's standard parameter Table 7. Proposed total impervious coverage values and direct impervious coverage values corresponding to each land use type are included in Appendix A. A comparison of overall changes is presented in Table 7.8.

Table 7.8. Comparison of Modelled Imperviousness to Previous Studies for Sheldon Creek

LOCATION	2019 SHELDON CREEK HYDROLOGY	2023 EAST BURLINGTON CREEKS FLOODPLAIN MAPPING (EXISTING CONDITIONS)	2023 EAST BURLINGTON CREEKS FLOODPLAIN MAPPING (FUTURE CONDITIONS)	DIFFERENCE FROM EXISTING CONDITIONS
East Branch	52.0%	51.0%	58.1%	+7.1%
West Branch	43.0%	48.9%	51.9%	+3.0%
Total	45.4%	49.1%	53.3%	+4.2%

The comparison indicates that the overall increase in impervious coverage due to future land use is approximately 4%. The East Branch would have a relatively higher increase of 7% which reflects the expected development between Wyecroft Road and Rebecca Street.

In addition, under future conditions the future rainfall IDF has been applied (current City of Burlington IDF which incorporates an adjustment to account for climate change, as discussed in Section 3.7). This would be expected to further increase the simulated peak flows for the 100-year storm event. Model results are noted in subsequent sections accordingly.

7.3.2 MODEL RESULTS

Simulated peak flows under future conditions are presented in Table 7.9. Note that the presented flows do not include spills, if applicable; this is reviewed further in Section 7.3.3.

Table 7.9. Simulated Design Storm and Regional Storm Peak Flows at Key Locations for Sheldon Creek – Future Conditions

LOCATION	ARF (%)	NODE	DRAINAGE AREA (ha)	PEAK FLOW RATE (m^3/s)		
				100 YEAR	12 HOUR REGIONAL (AMC III)	48 HOUR REGIONAL (AMC II)
West Branch- West Tributary						
North Of Dundas St	100	SDC0040	220	11.1	26.9	26.5
Appleby Line	100	SDC0050	273	13.5	33.3	33.0
North of Upper Middle Rd	100	SDJ0090	345	18.6	38.6	38.2
West Branch -East Tributary						
South Of Dundas St	100	SDJ0140	186	36.6	29.7	29.4
North of CNR-Halton	100	SDJ0250	271	40.9	33.3	32.9
Dryden Avenue	100	SDJ0260	300	39.8	36.4	36.1
North of Upper Middle Rd	100	SDJ0300	357	40.1	42.1	41.7
West Branch						
Confluence Point	100	SDJ0350	708	61,7	84.8	84.0
Mainway	99.2	SDJ0390	733	54.8	85.4	84.6
QEW	99.2	SDJ0430	860	61.9	99.0	98.0
South of New Street	97.1	SDJ0470	1,060	68.1	117.2	116.0
Spruce Avenue	97.1	SDJ0480	1,116	68.7	122.8	121.6
Burloak Drive	97.1	SDJ0490	1,133	69.2	124.5	123.3
East Branch						
Upper Middle Rd	100	SDJ0640	111	21.7	15.3	15.3
Mainway	100	SDJ0650	140	20.0	18.2	18.1
QEW	100	SDJ0670	260	24.7	25.9	25.8
CNR Oakville	100	SDJ0730	343	28.8	35.2	34.9
South of Rebecca Street	100	SDJ0800	454	30.9	46.3	46.1
Main Branch						
Confluence Point	97.1	SDJ0830	1.631	97.6	172.4	171.0
Great Lakes Blvd	97.1	SDJ0840	1,678	98.9	177.3	175.8
South of Lakeshore Road West	96.3	SDJ0890	1,816	103.7	189.1	187.5
Lake Ontario	96.3	SDJ0910	1,823	103.8	189.7	188.0

While existing and future data are presented in this report, calculated differences in results should not be interpreted as reassessing or demonstrating the impacts of future development. The climate change adjusted IDF has only been applied to define the future 1:100-year flow data and has not been applied to existing conditions. Also, to support model calibration, the existing conditions modeling represents the current watershed condition. This may include centralized SWM controls that are designed to provide attenuation for a future development condition identified in the Official Plan but where the proposed development is not fully built out. In these areas, the existing conditions model assumes existing land uses where development has not yet occurred and may therefore predict existing condition flow rates less than pre-development conditions.

The Regional Storm (12-hour) governs in the majority of the identified locations of interest, with the exception of upper sections of the West and East Branches, where the 100-year storm event generates greater peak flows.

The peak flows presented in Table 7.9 have been applied as the basis for the flood hazard mapping (notwithstanding any potential additional spill flows, as described in Section 7.3.3). Flood hazard mapping is described further in the separate hydraulic modelling report.

7.3.3 ASSESSMENT OF SPILLS

The simulated future condition flows have been applied to the hydraulic modelling (both 1D and 2D) as described in the separate Hydraulics Report. As noted in that assessment, numerous spills have been identified. In order to apply the “balanced approach” proposed by CH (refer to CH’s technical memorandum of May 19, 2022 as included within the Hydraulics Report), integration between the 2D hydraulic modelling and hydrologic modelling is necessary. Spill flows are to be included within the watershed receiving the spill. Iteration between the 2D hydraulic modelling and hydrologic modelling has been required to identify inter-watershed spills specifically that meet the threshold for inclusion, and to “balance” flows such that there is reasonable agreement between hydrologic and hydraulic modelling results.

For the Sheldon Creek watershed, an inter-watershed spill has been identified from Appleby Creek along Highway 407. The estimated magnitude of the spill and resulting change in nodal peak flows is presented in Table 7.10, along with the difference in peak flows as compared to the without spill scenario (Table 7.9).

The results indicate that for the 100-year storm event, hydrograph timing is such that increases in peak flows are limited to the immediate downstream area only (north of Dundas Street); further downstream no increases are indicated. This likely reflects the more peaked nature of the 100-year storm event. Conversely for the Regional Storm Event, the simulated spill flow results in a relatively consistent increase in peak flows at all downstream nodes, all the way to the outlet at Lake Ontario.

In addition to inter-watershed spills, a number of intra-watershed spills have been identified (spills between different watercourse branches within the same overall watershed system). These flows are not corrected within the 2D modelling, to avoid double-counting flows (given that the flows would be expected to generally re-combine further downstream). However, in order to ensure consistency between 1D and 2D hydraulic modelling, the spill flow should be incorporated into a separate hydrologic modelling scenario to consider the resulting increased flow to the branch receiving the spill. These results are presented in Table 7.11.

Two (2) different intra-watershed spills are indicated for the Regional Storm Event. The first is along the west branch (from the east tributary to the west tributary) at Upper Middle Road. The other is along the west branch (from the west tributary to the east tributary) in the vicinity of Pond 804 (immediately upstream of Dundas Street). In both cases the resultant additional intra-watershed spill results in a relatively consistent increase in combined peak flow.

Table 7.10. Simulated Design Storm and Regional Storm Peak Flows at Key Locations for Sheldon Creek – Future Conditions with Inter-Watershed Spill Flows

LOCATION	ARF (%)	NODE	PEAK FLOW RATE (m^3/s)		DIFFERENCE (m^3/s)	
			100 YEAR	12 HOUR REGIONAL (AMC III)	100 YEAR	12 HOUR REGIONAL (AMC III)
West Branch- West Tributary						
Spill Flow from Appleby Creek at Highway 407	100	400 (to SDJ0010)	2.0	4.3	+2.0	+4.3
North Of Dundas St	100	SDC0040	12.1	29.2	+1.0	+2.3
Appleby Line	100	SDC0050	13.5	35.2	0	+1.9
North of Upper Middle Rd	100	SDJ0090	18.6	40.4	0	+1.8
West Branch						
Confluence Point	100	SDJ0350	61.7	86.4	0	+1.6
Mainway	99.2	SDJ0390	54.8	87.3	0	+1.9
QEW	99.2	SDJ0430	61.9	100.8	0	+1.8
South of New Street	97.1	SDJ0470	68.1	119.1	0	+1.9
Spruce Avenue	97.1	SDJ0480	68.7	124.7	0	+1.9
Burloak Drive	97.1	SDJ0490	69.2	126.3	0	+1.8
Main Branch						
Confluence Point	97.1	SDJ0830	97.6	174.1	0	+1.7
Great Lakes Blvd	97.1	SDJ0840	98.9	178.9	0	+1.6
South of Lakeshore Road West	96.3	SDJ0890	103.7	190.6	0	+1.5
Lake Ontario	96.3	SDJ0910	103.8	191.1	0	+1.4

Table 7.11. Simulated Regional Storm Peak Flows at Key Locations for Sheldon Creek with Intra and Inter-Watershed Spill Flows - Future Conditions

LOCATION	ARF (%)	NODE	PEAK FLOW RATE (m^3/s)	DIFFERENCE WITH INTRA-WATERSHED SPILLS (m^3/s)
West Branch- West Tributary				
Spill Flow from Appleby Creek at Highway 407	100	407 (to SDJ0010)	4.3	0
North Of Dundas St	100	SDC0040	29.2	0
Appleby Line	100	SDC0050	35.2	0
North of Upper Middle Rd	100	SDJ0090	40.4	0
Intra-Watershed Spill at Upper Middle Road	100	416 (to SDJ0110)	9.0	+9.0
Upstream of Confluence with East Tributary	100	SDJ0110	51.6	+8.1
West Branch -East Tributary				
Intra-Watershed Spill at Pond 804	100	400 (to SDJ0140)	9.1	+9.1
South Of Dundas St	100	SDJ0140	38.8	+9.1
North of CNR-Halton	100	SDJ0250	42.3	+9.0

In addition to the preceding, it should be noted that additional spills have been indicated from Sheldon Creek easterly to the adjacent watershed (Bronte Creek). Although not included as part of the hydrologic modelling (spill flows are as identified by the 2D hydraulic modelling; refer to companion Hydraulics Report accordingly) these additional watershed spill flows should be considered as part of any future hydrologic modelling of the Bronte Creek system. Simulated spill flows for the 100-year and Regional Storm Event under future conditions are presented in Table 7.12. The extents of the simulated spill from the East Branch at Rebecca Street have been included as part of the flood hazard mapping; refer to the Hydraulics Report and associated mapping sheets for further detail.

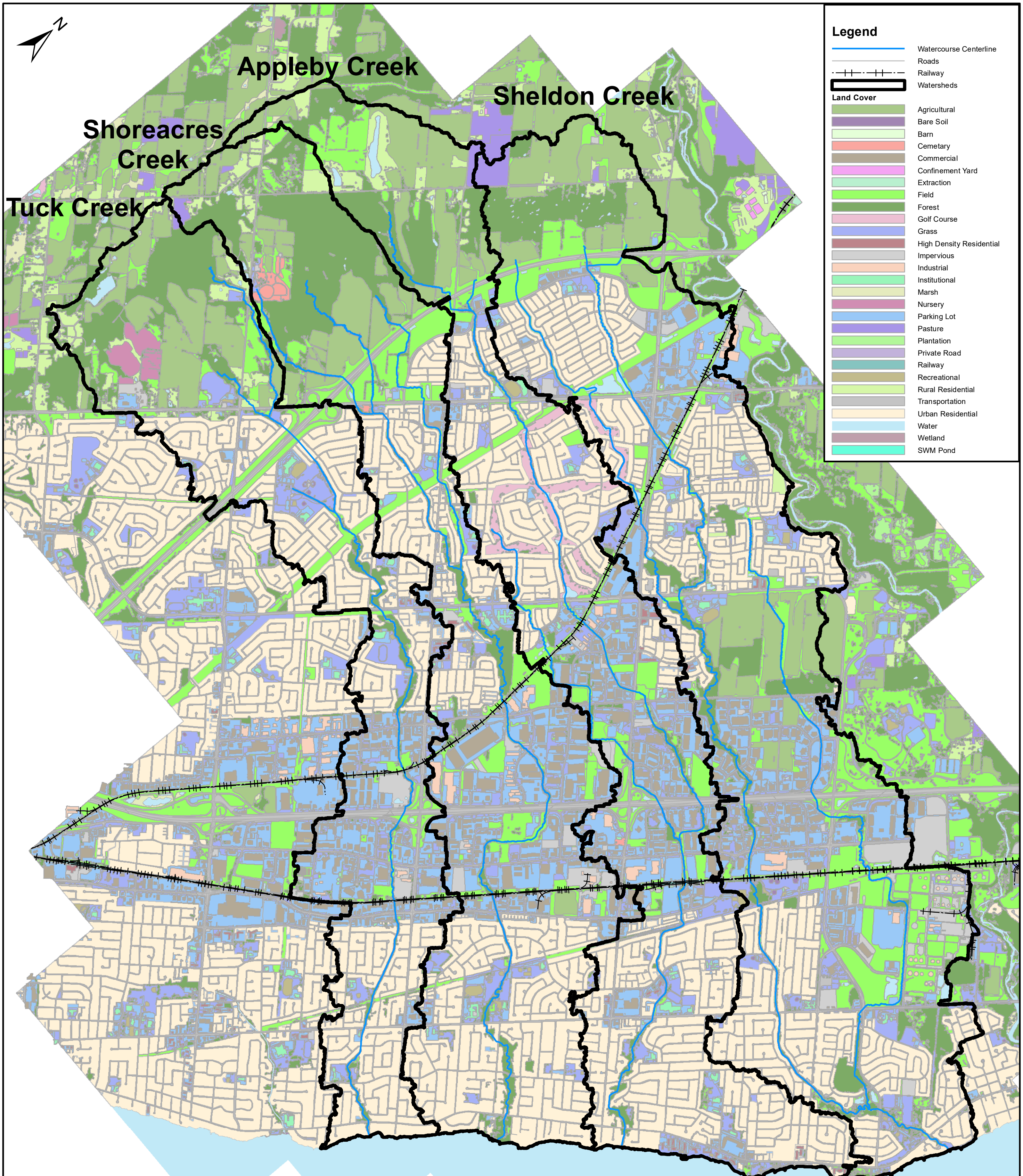
Table 7.12. Simulated External Spill Peak Flows from Sheldon Creek to Bronte Creek - Future Conditions

LOCATION	PEAK FLOW RATE (m^3/s)	
	100 YEAR	12 HOUR REGIONAL (AMC III)
West Branch – East Tributary		
North side of Highway 407	2.5	4.2
East Branch		
At Rebecca Street (near 3361 Rebecca Street)	22.4	30.0

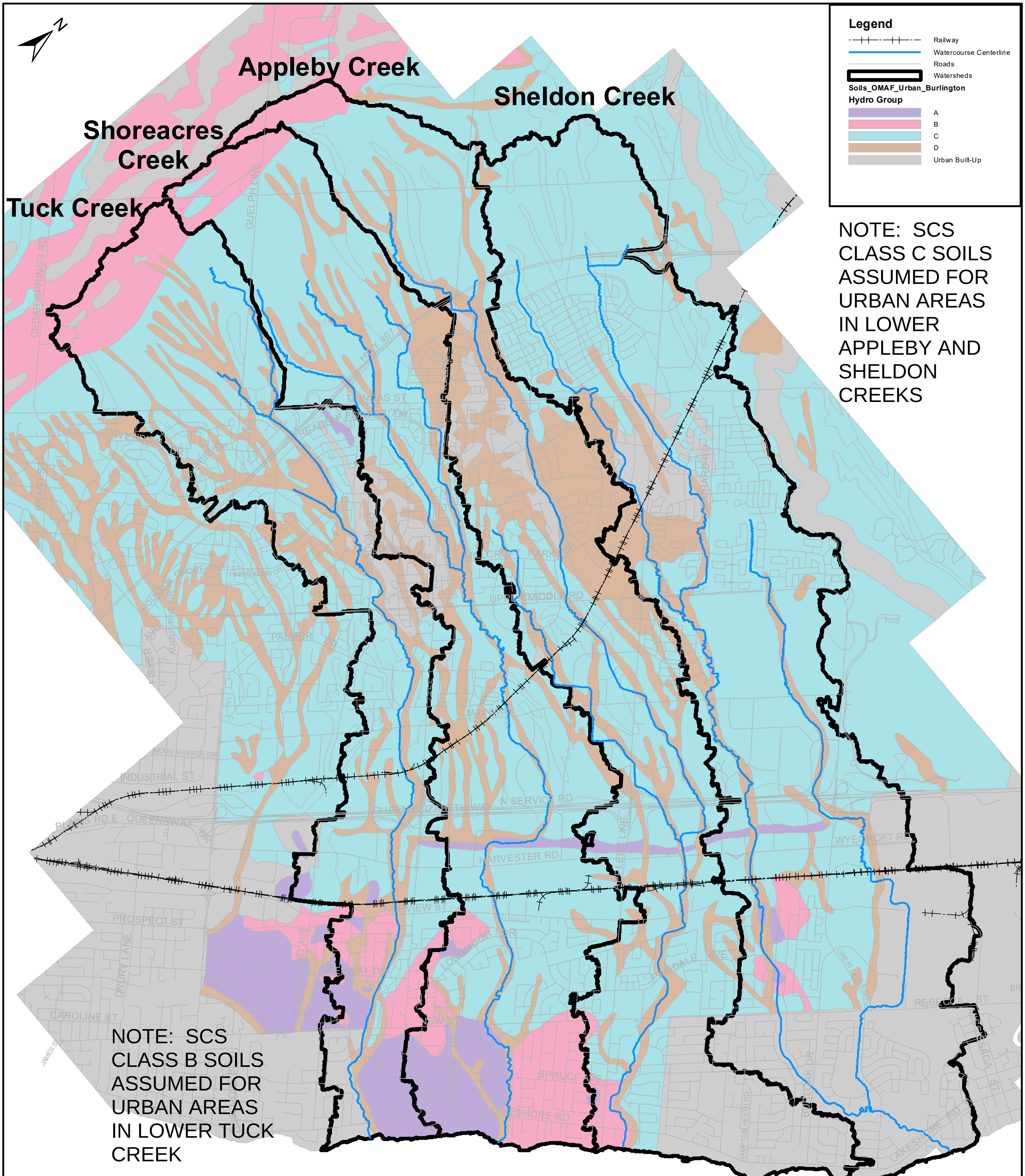
8 SUMMARY AND CONCLUSIONS

New hydrologic models have been developed in Visual Otthymo for each of the four (4) subject watersheds (Tuck, Shoreacres, Appleby and Sheldon Creeks). The new models have been developed using current best practices and standards, including the 2002 Provincial document "Technical Guide – River and Stream Systems: Flooding Hazard Limit" and the specifications of Conservation Halton. The modelling has been developed under both existing and future conditions. The modelling developed for future conditions (which reflects both future land use and a future climate-change adjusted rainfall for the 2-100 year storm events) is to be applied for subsequent hydraulic modelling (other than locations where the existing condition flow governs).

The presented hydrologic modelling results are considered valid and appropriate and have been implemented in the hydraulic modelling (both 1-dimensional (1D) and 2-dimensional (2D)) as described in the separate reporting for that work. As noted, spill interactions between watersheds (as determined iteratively using the 2D hydraulic modelling and the current hydrologic modelling) have been determined and incorporated into the final estimated flows for the future scenario.



East Burlington Creeks Flood Hazard Mapping Update Conservation Halton	Land Covers Existing Condition	wsp	Scale 1:48,000 Project No. WW21011057 Figure No. 1



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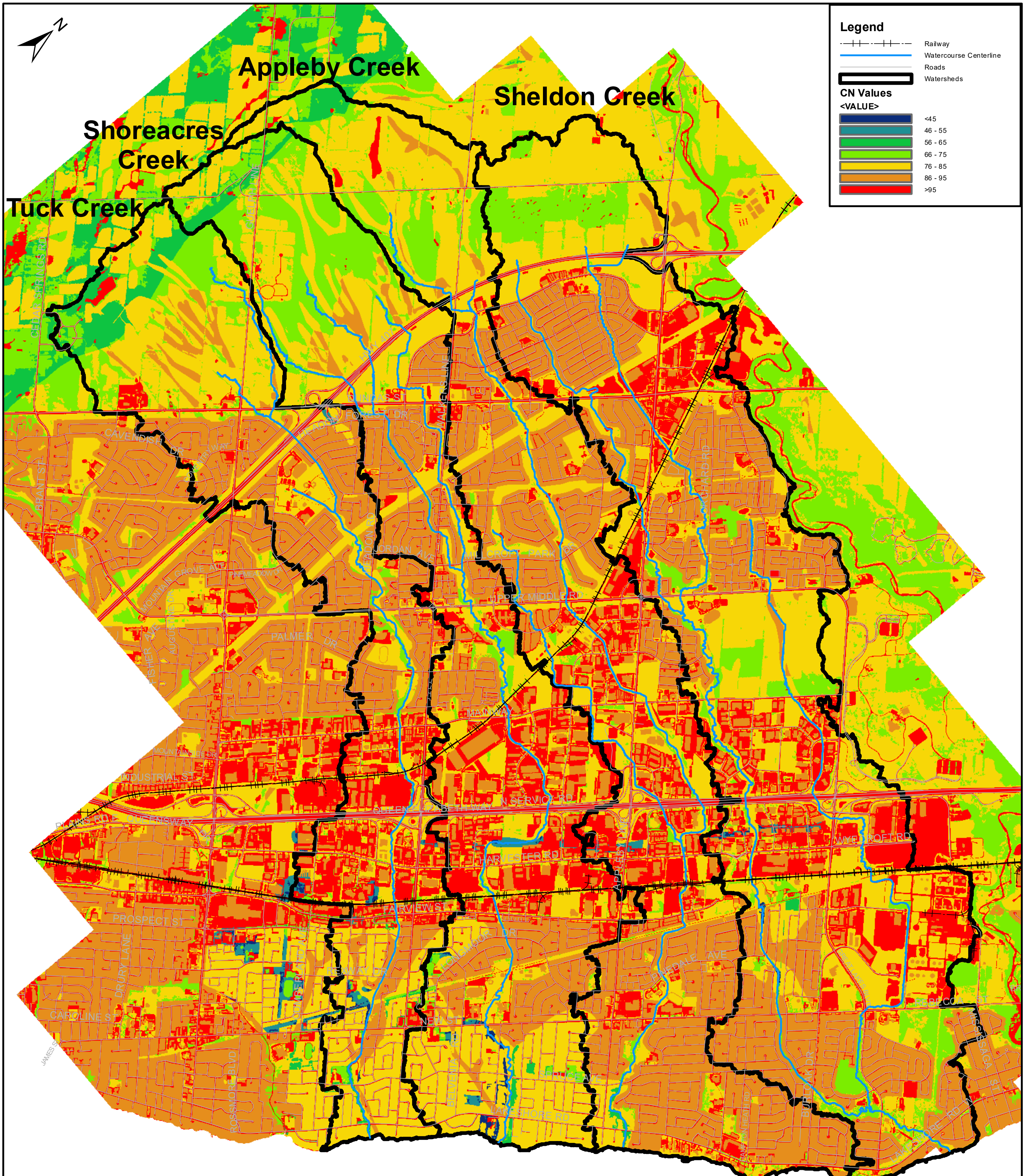
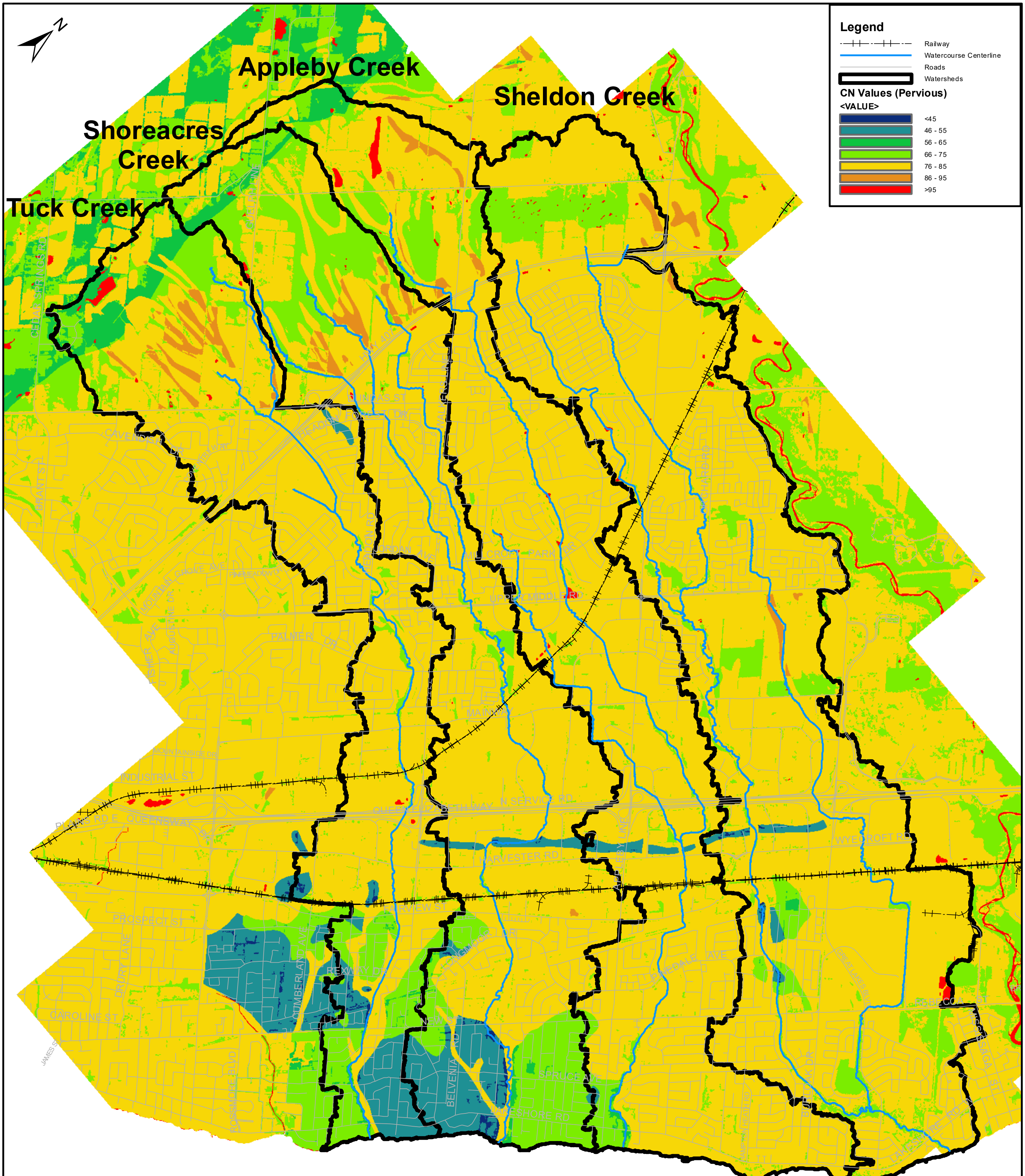


East Burlington Creeks Flood Hazard Mapping Update Conservation Halton	SCS Curve Number Values (Overall)	wsp	Scale 1:48,000 Project No. WW21011057 Figure No. 3a



East Burlington Creeks Flood Hazard Mapping Update

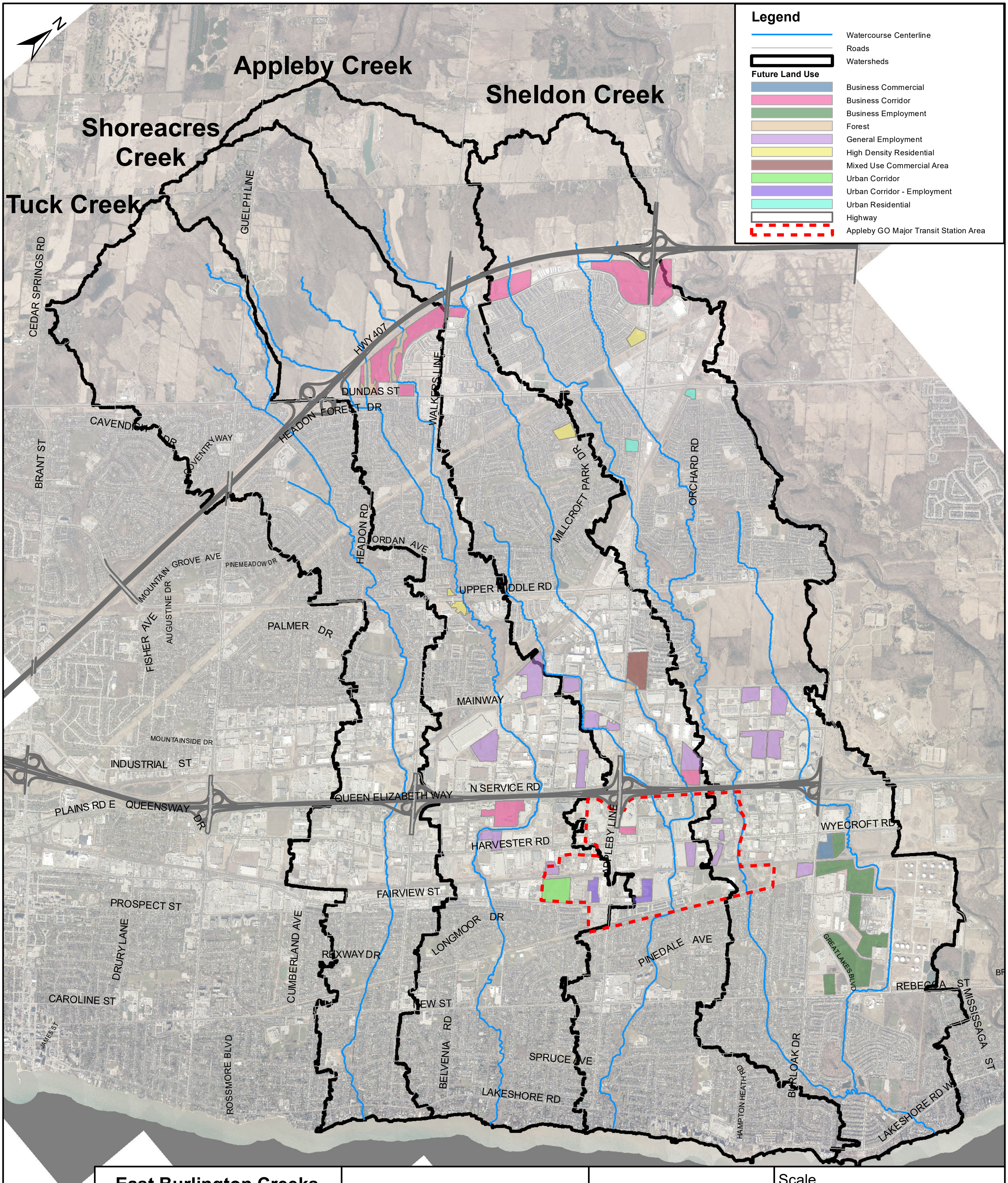
Conservation Halton

SCS Curve Number Values (Pervious Area Only)

Scale **1:48,000**

Project No. **WW21011057**

Figure No. **3b**



East Burlington Creeks Flood Hazard Mapping Update

Conservation Halton

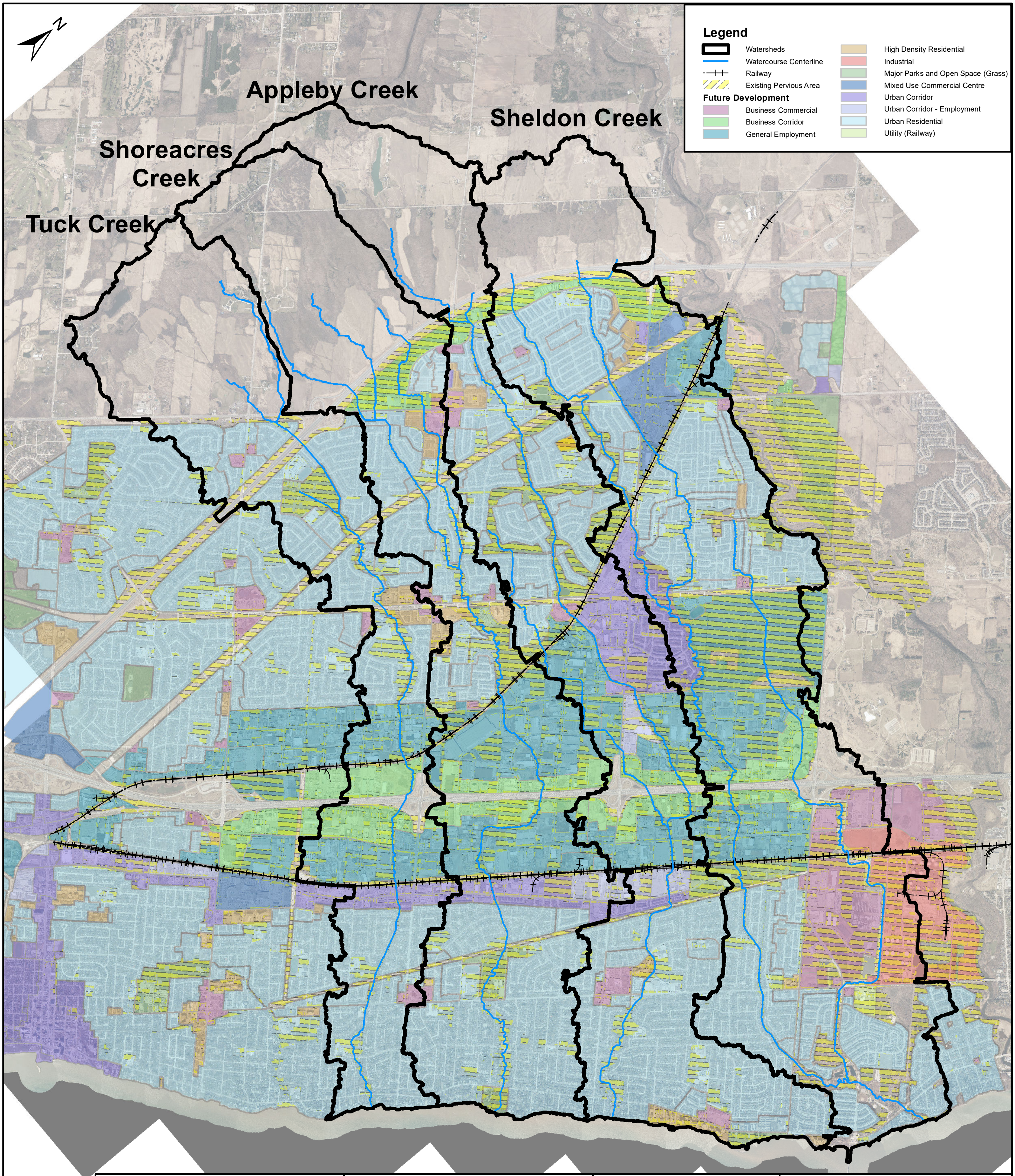
Future Land Use Areas

wsp

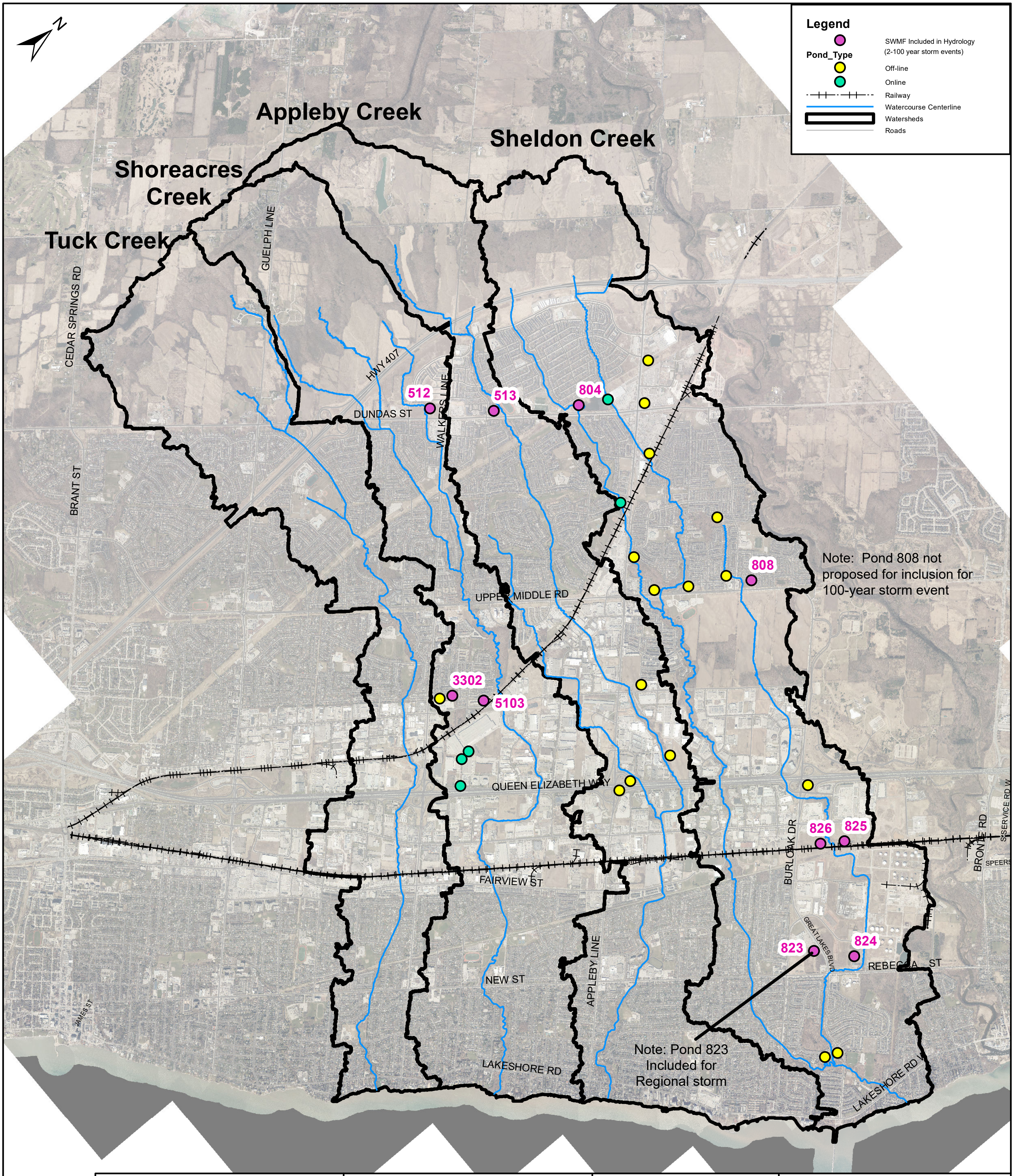
Scale **1:48,000**

Project No. **WW21011057**

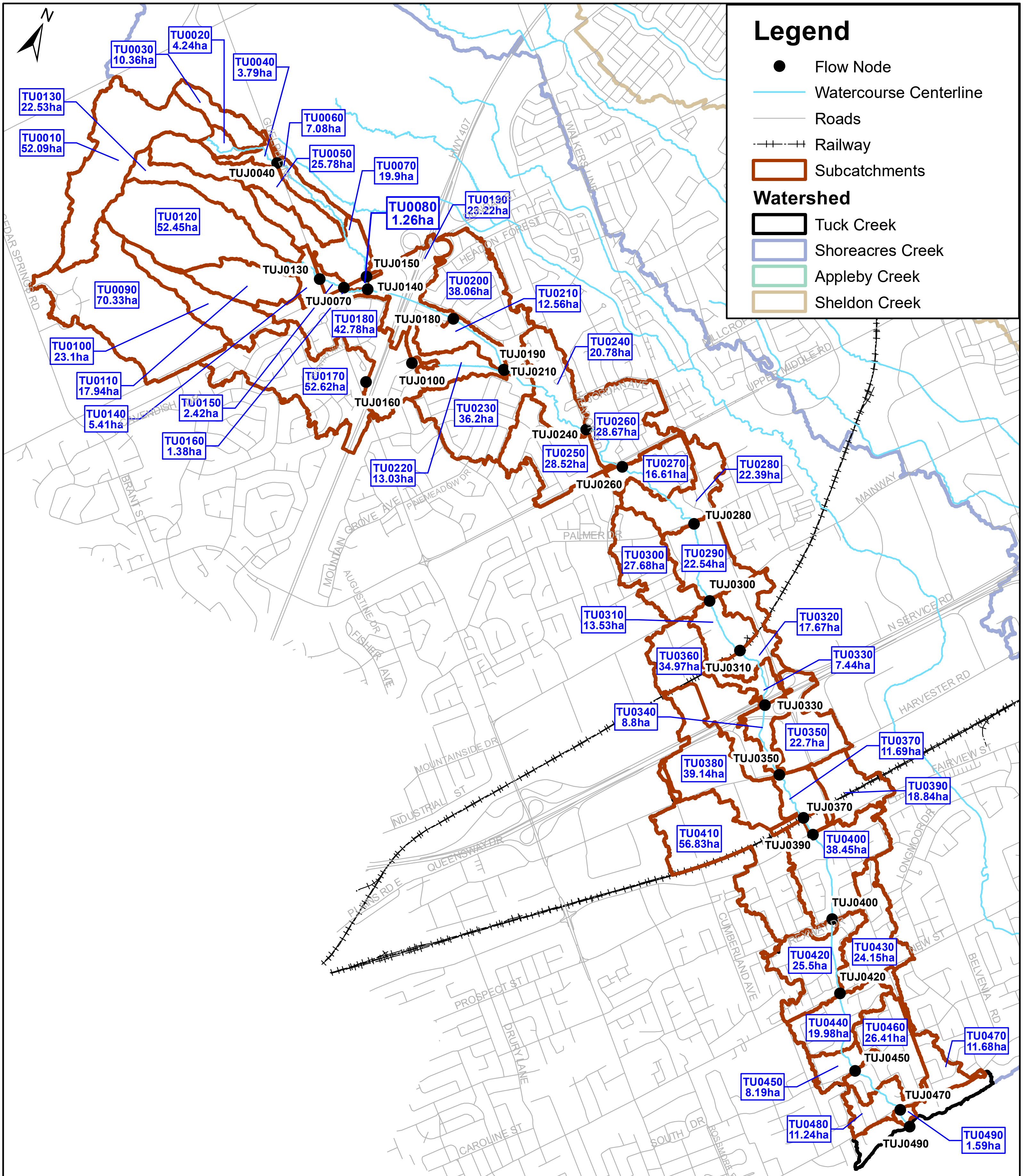
Figure No. **4a**



East Burlington Creeks Flood Hazard Mapping Update Conservation Halton	Overlay of Existing Undeveloped Areas and Future Development Areas	wsp	Scale 1:50,000 Project No. WW21011057 Figure No. 4b

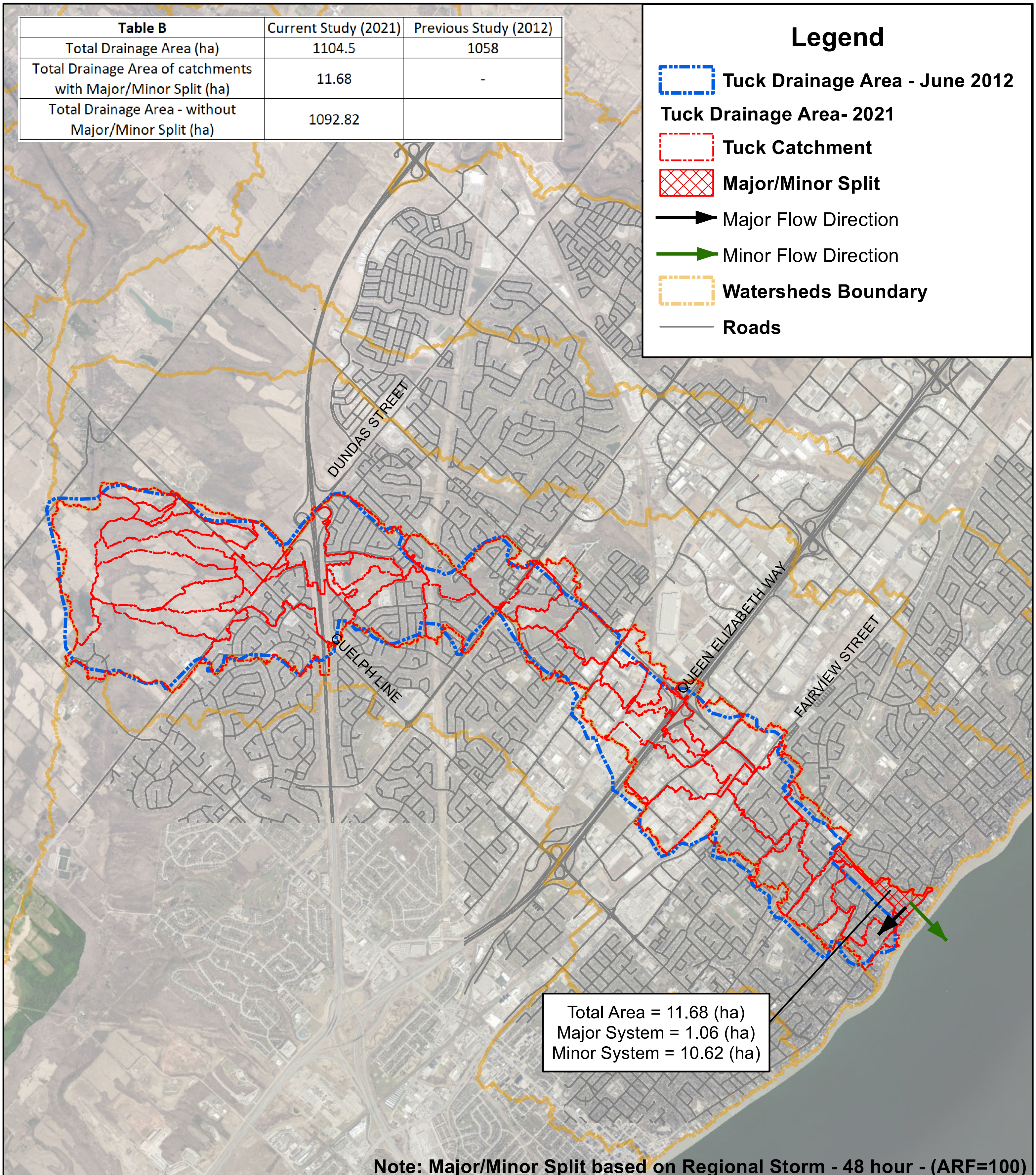


East Burlington Creeks Flood Hazard Mapping Update Conservation Halton	SWM Facilities	wsp	Scale 1:52,000 Project No. WW21011057 Figure No. 5



East Burlington Creeks Flood Hazard Mapping Update Conservation Halton	Subcatchment Boundaries Tuck Creek Watershed		Scale 1:35,000 Project No. WW21011057 Figure No. 6a

Table B	Current Study (2021)	Previous Study (2012)
Total Drainage Area (ha)	1104.5	1058
Total Drainage Area of catchments with Major/Minor Split (ha)	11.68	-
Total Drainage Area - without Major/Minor Split (ha)	1092.82	



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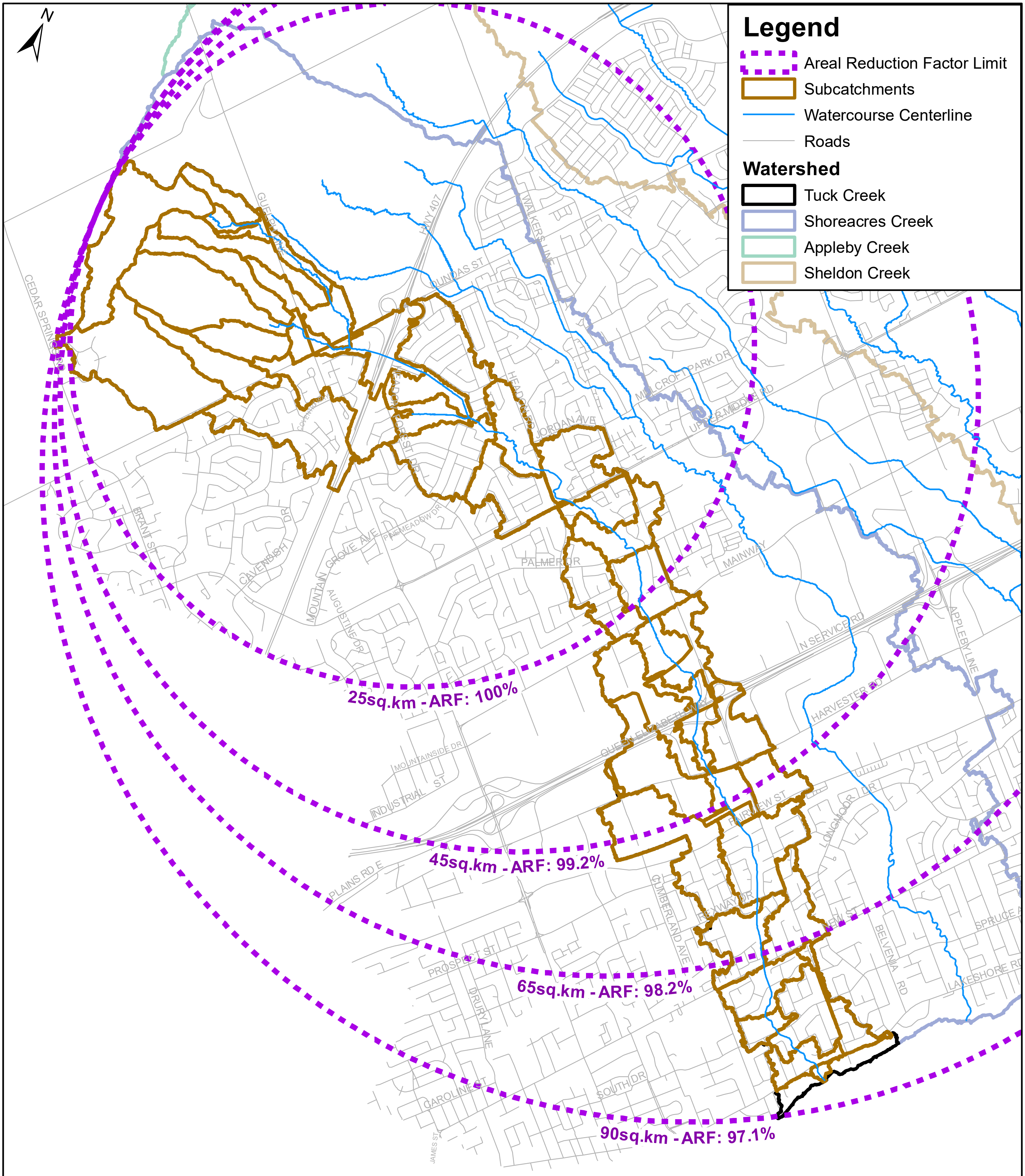
East Burlington Creeks Flood Hazard Mapping Update

Conservation of Halton

Tuck Creek Drainage Area Comparison (2021 & 2012)

Scale	1:50,000
Project No.	WW21011057
Figure No.	6b



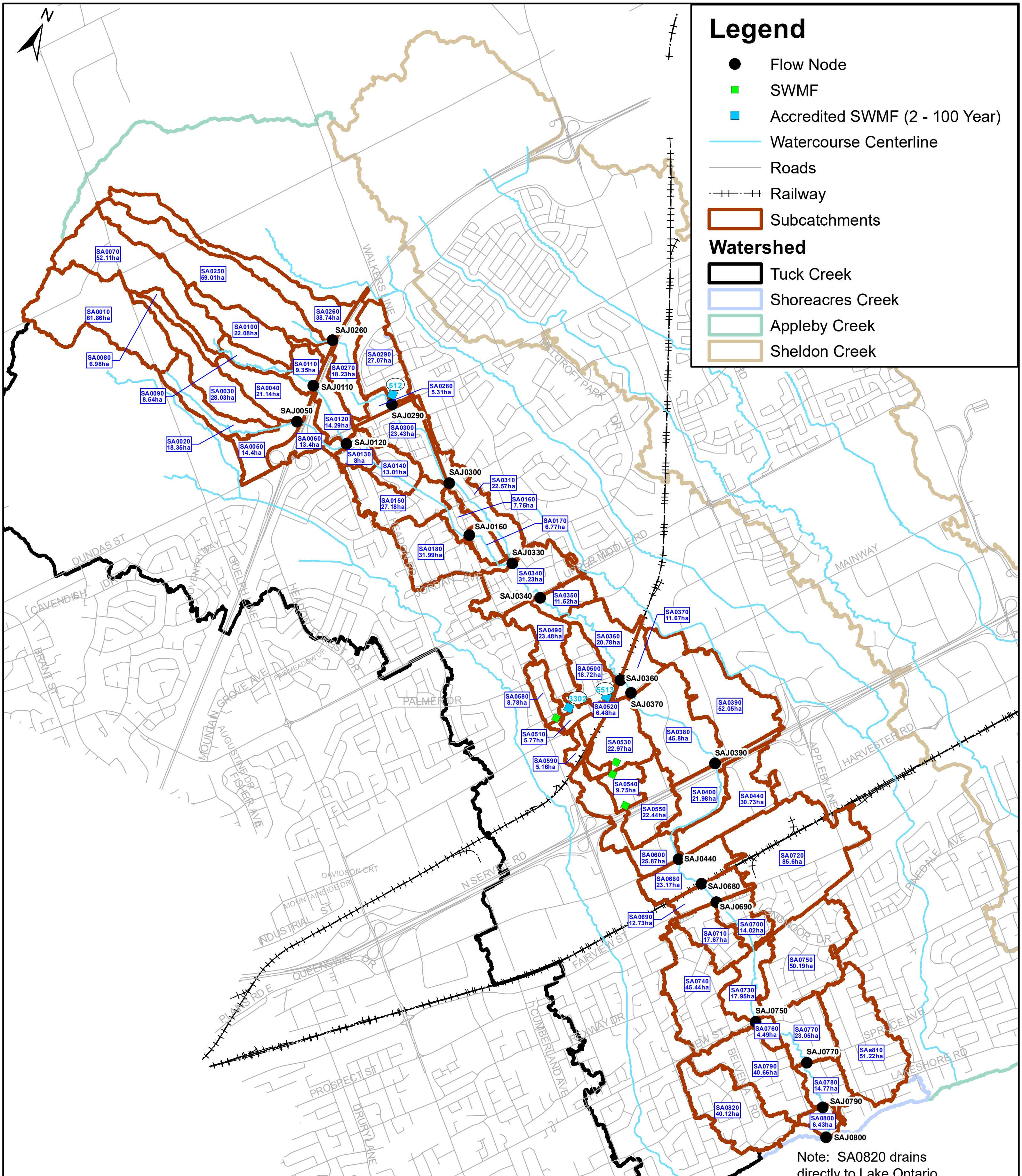
East Burlington Creeks Flood Hazard Mapping Update

Conservation Halton

Areal Reduction Factor Tuck Creek Watershed

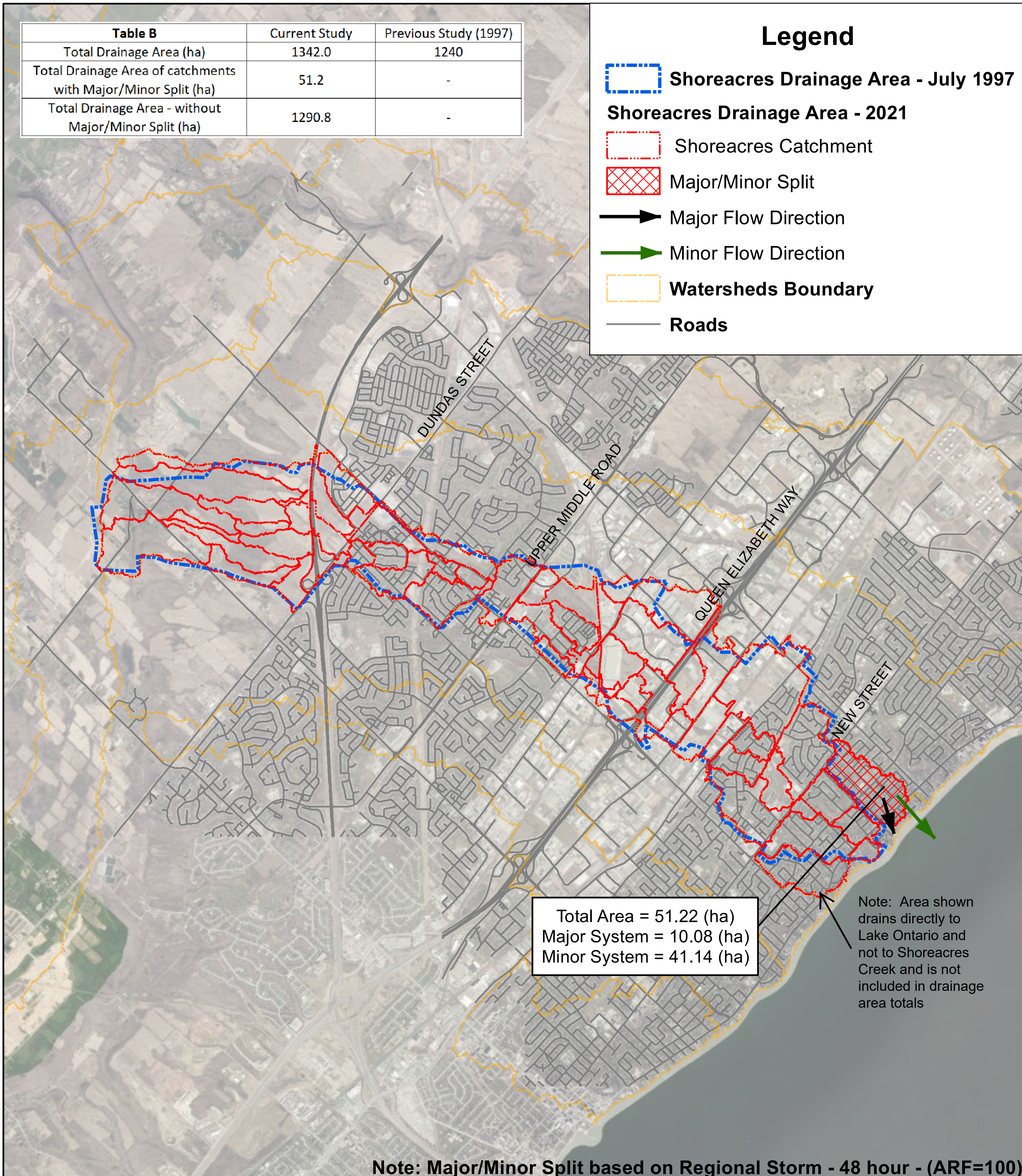
wsp

Scale	1:40,000
Project No.	WW21011057
Figure No.	7

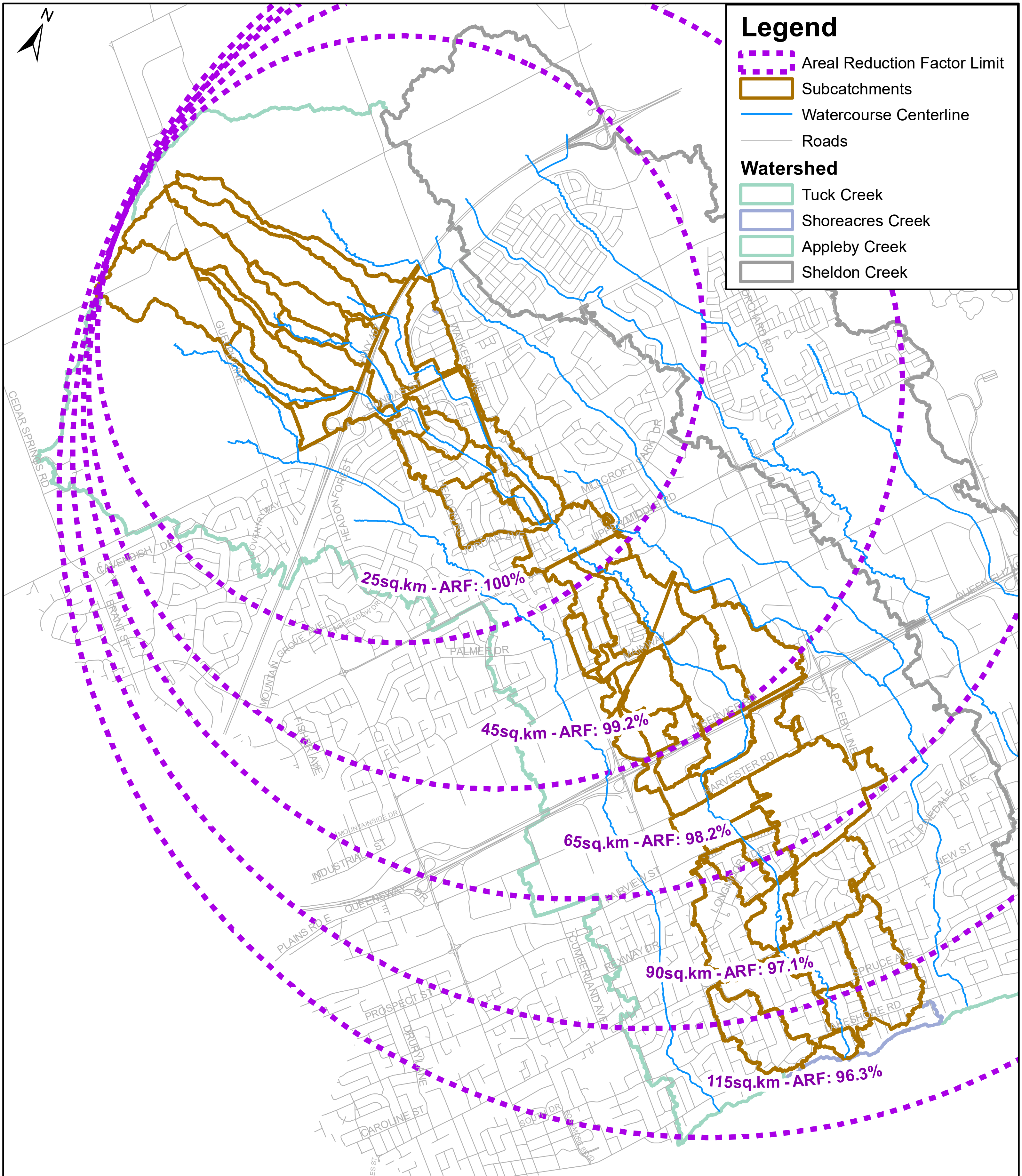


East Burlington Creeks Flood Hazard Mapping Update Conservation Halton	Subcatchment Boundaries Shoreacres Creek Watershed	WSP	Scale 1:42,000 Project No. WW21011057 Figure No. 8a

Table B	Current Study	Previous Study (1997)
Total Drainage Area (ha)	1342.0	1240
Total Drainage Area of catchments with Major/Minor Split (ha)	51.2	-
Total Drainage Area - without Major/Minor Split (ha)	1290.8	-



East Burlington Creeks Flood Hazard Mapping Update **Conservation of Halton**	**Shoreacres Creek Drainage Area Comparison (2021 & 1997)**	Scale	**1:60,000**
		Project No.	**WW21011057**
		Figure No.	**8b**



East Burlington Creeks Flood Hazard Mapping Update

Conservation Halton

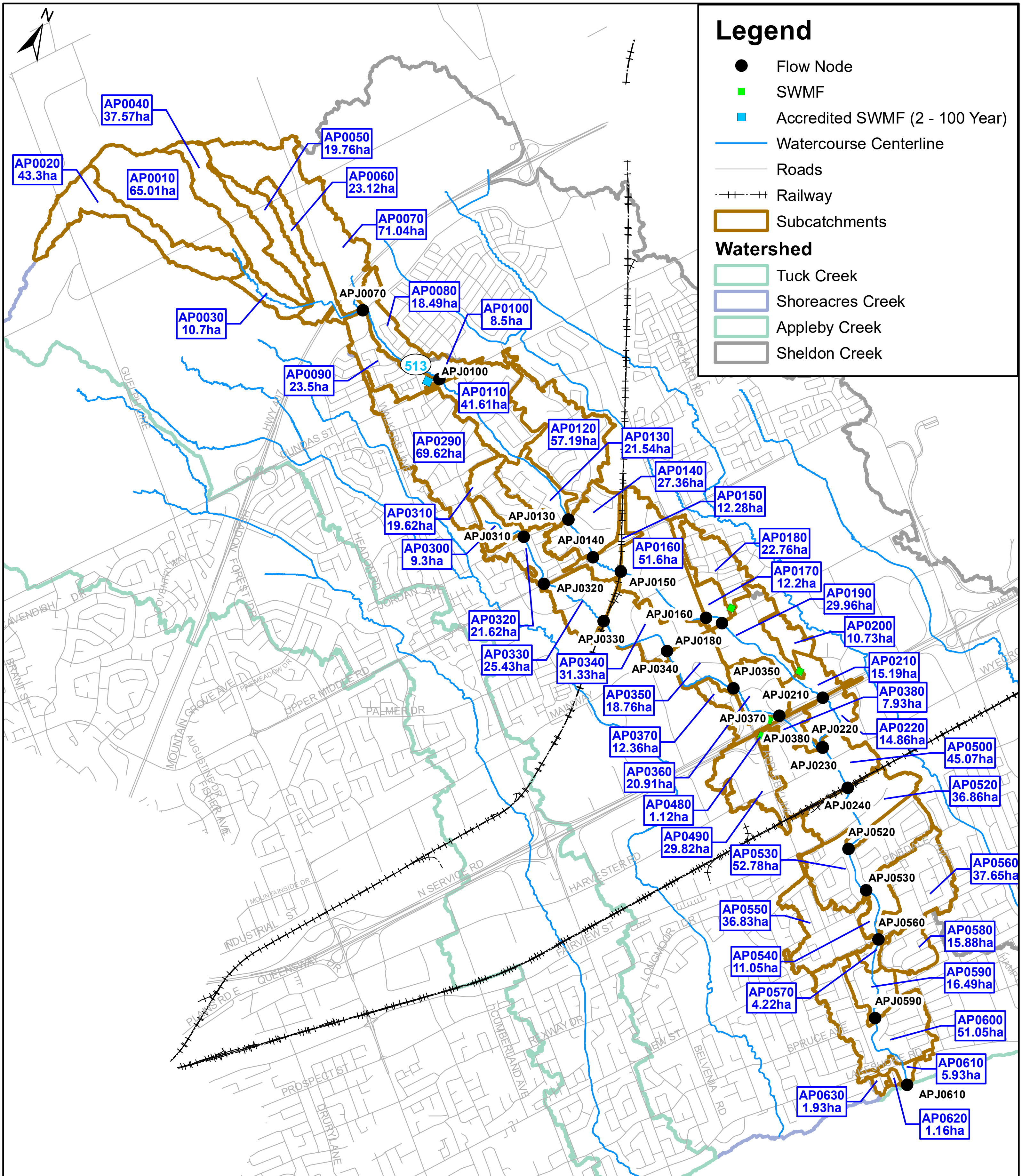
Areal Reduction Factor Shoreacres Creek Watershed

wsp

Scale **1:45,000**

Project No. **WW21011057**

Figure No. **9**



East Burlington Creeks Flood Hazard Mapping Update

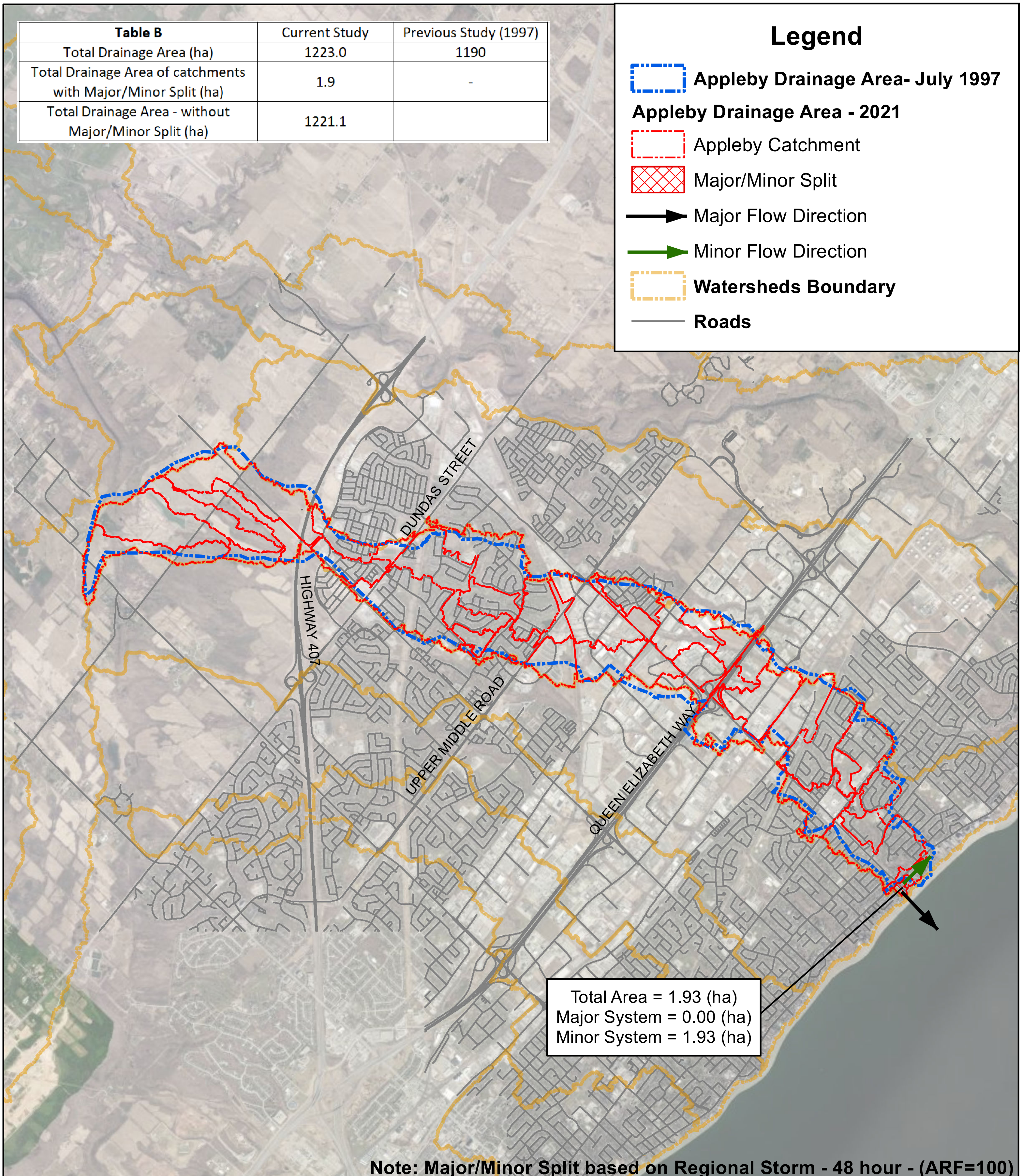
Conservation Halton

Subcatchment Boundaries Appleby Creek Watershed

wsp

Scale	1:43,000
Project No.	WW21011057
Figure No.	10a

Table B	Current Study	Previous Study (1997)
Total Drainage Area (ha)	1223.0	1190
Total Drainage Area of catchments with Major/Minor Split (ha)	1.9	-
Total Drainage Area - without Major/Minor Split (ha)	1221.1	



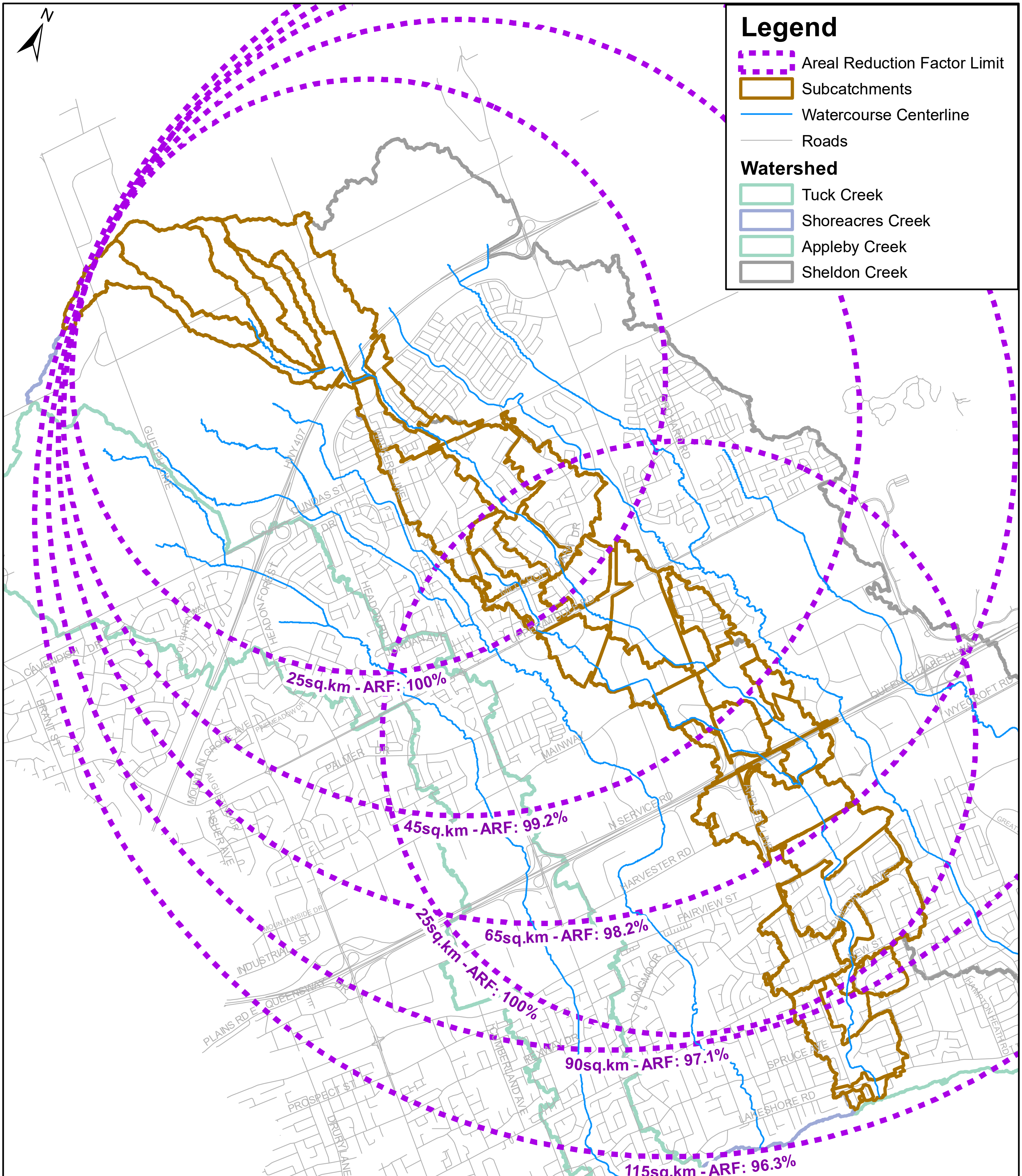
East Burlington Creeks Flood Hazard Mapping Update

Conservation of Halton

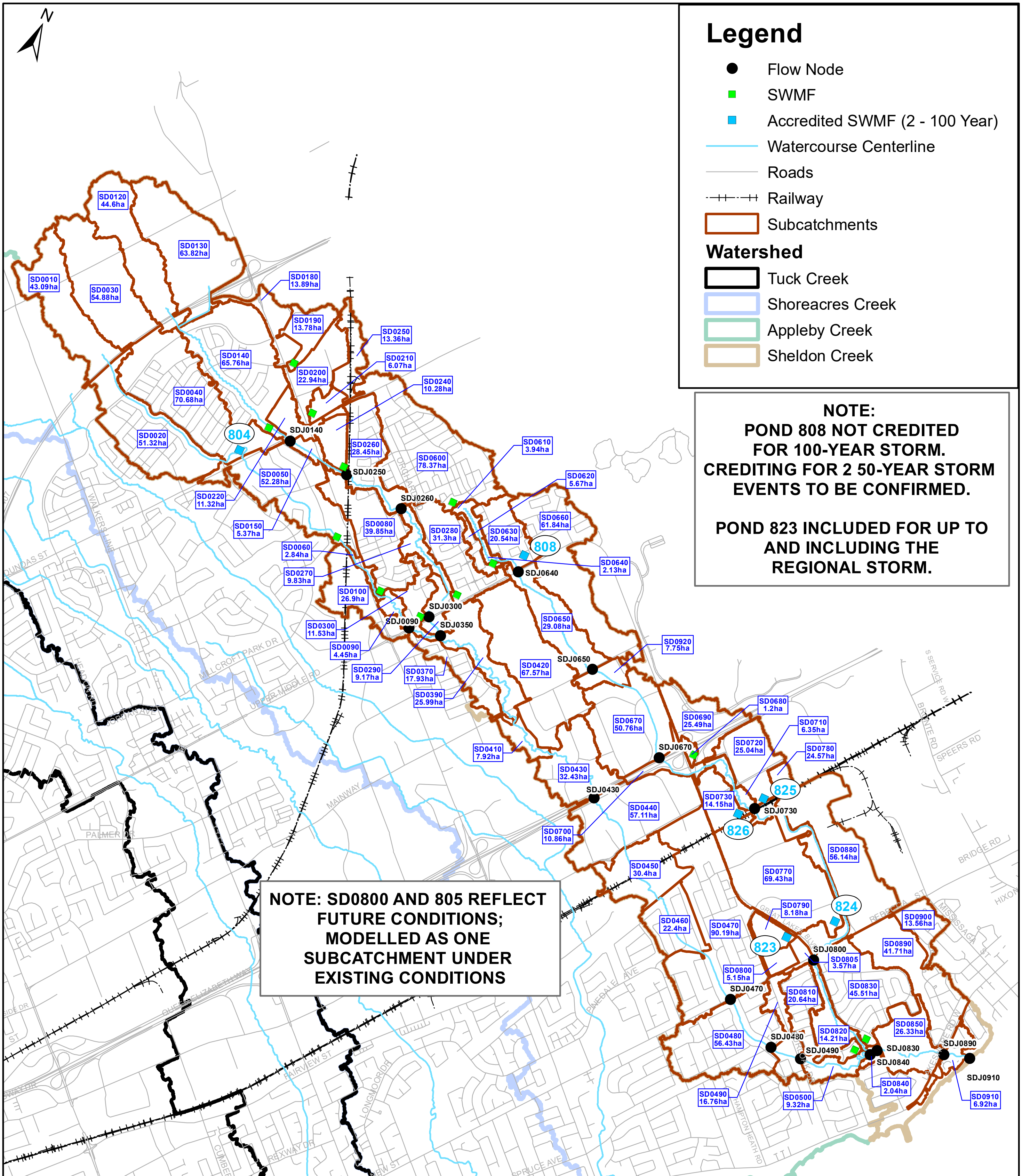
Appleby Creek Drainage Area Comparison (2021 & 1997)

wsp

Scale	**1:60,000**
Project No.	**WW21011057**
Figure No.	**10b**

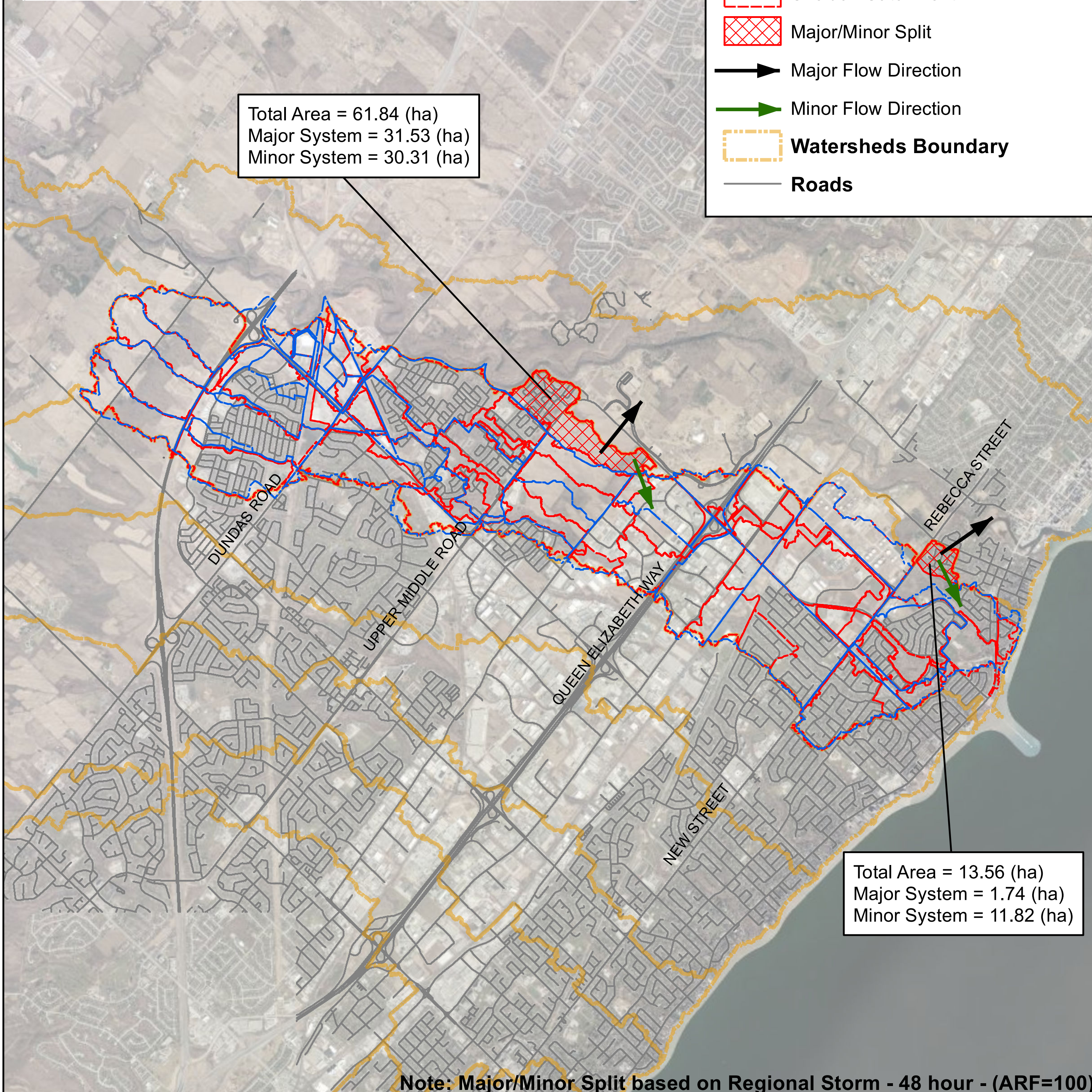


East Burlington Creeks Flood Hazard Mapping Update **Conservation Halton**	**Areal Reduction Factor Appleby Creek Watershed**	wsp	Scale **1:46,000** Project No. **WW21011057** Figure No. **11**



East Burlington Creeks Flood Hazard Mapping Update Conservation Halton	Subcatchment Boundaries Sheldon Creek Watershed	wsp	Scale 1:42,500 Project No. WW21011057 Figure No. 12a

Table B	Current Study	Previous Study
Total Drainage Area (ha)	1824.0	1737
Total Drainage Area of catchments with Major/Minor Split (ha)	75.4	-
Total Drainage Area - without Major/Minor Split (ha)	1748.6	-



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East Burlington Creeks Flood Hazard Mapping Update

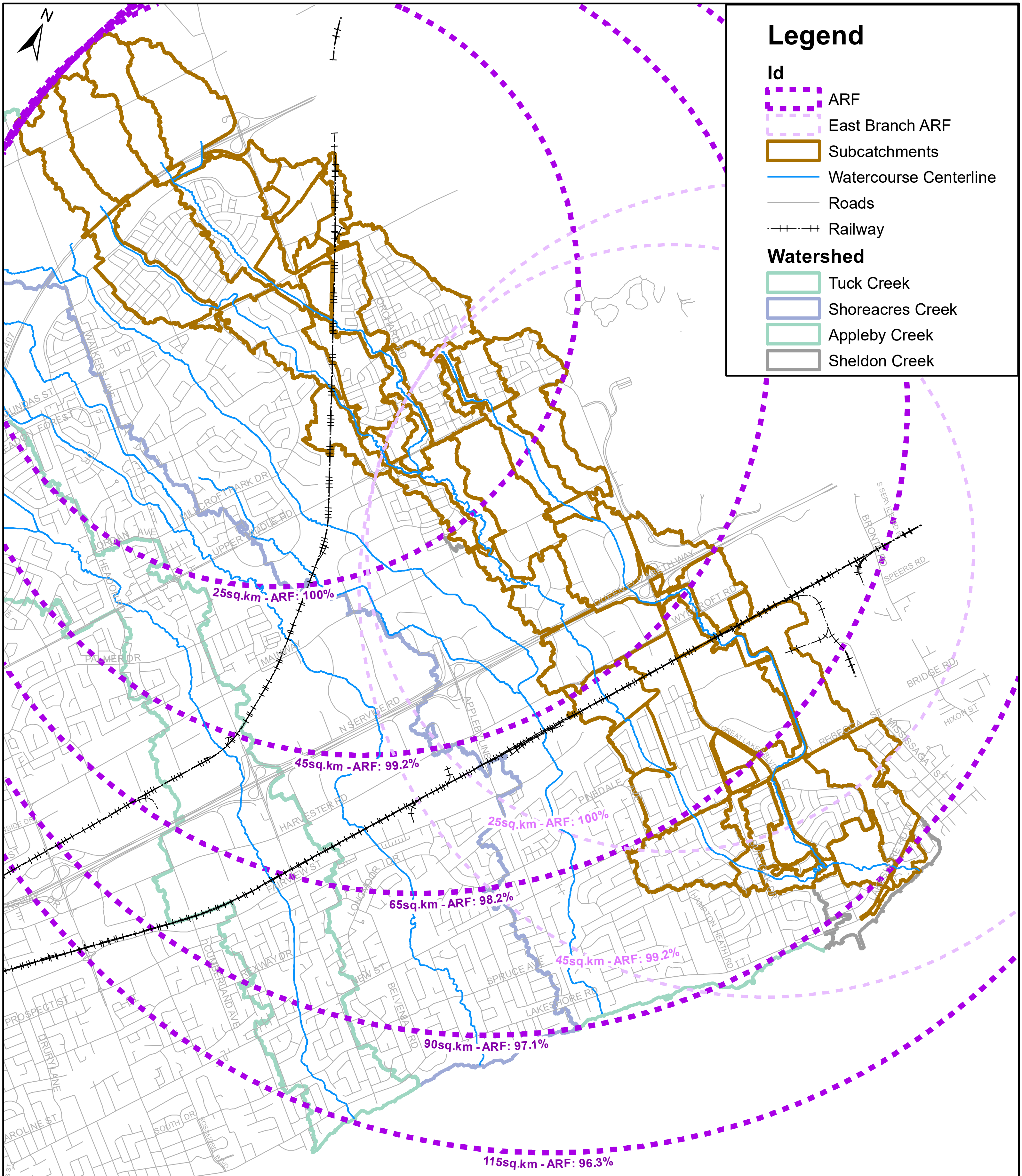
Conservation of Halton

Sheldon Creek Drainage Area Comparison (2021 & 2020)

Scale **1:60,000**

Project No. **WW21011057**

Figure No. **12b**



East Burlington Creeks Flood Hazard Mapping Update **Conservation Halton**	**Areal Reduction Factor Sheldon Creek Watershed**	wsp	Scale **1:45,000** Project No. **WW21011057** Figure No. **13**