Prepared for:

QUINTE CONSERVATION AUTHORITY

Attention: Mike Smith
Project Manager
Water Control Structures Technologist
E: MSmith@quinteconservation.ca

2061 Old Highway 2
RR#2, Belleville, ON
K8N 4Z2

Selby Creek Flood Hazard Mapping

Final Report

Submitted by:

Aquafor Beech Limited

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Contact:
Rob Amos, MASc, P.Eng
Aquafor Beech Limited

amos.r@aquaforbeech.com
2600 Skymark Avenue
Building 6, Unit 202
Mississauga, ON L4W 5B2
Tel: 416.705.2367

Aquafor Beech Reference: 67369

Selby Creek at Jim Kimmett Blvd

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Quinte Conservation Authority:

Mike Smith – Project Manager / Water Control Structures Technologist

Christine Phillibert – Water Resource Manager

Catherine Sinclair – Regulations Officer

Paul McCoy – Planning & Regulations Manager

Dave Eastcott – Water Resources Technologist

Curtis Vance – GIS / IT Systems Supervisor

Aquafor Beech Limited:

Prepared by:

Alex Verville, BSc.
Geospatial & Mapping Technologist
Aquafor Beech Limited

Yun Xia, M.Eng., EIT
Hydraulic Modelling Technologist
Aquafor Beech Limited

Julie Michel, M.Sc.
Hydraulic Modelling Technologist
Aquafor Beech Limited

Gabriel Dubé, MSc., EIT
Hydrologic Modelling Technologist
Aquafor Beech Limited

Natalie Connell, P.Eng.
Water Resource Engineer
Aquafor Beech Limited

Rob Amos, MASc., P.Eng.,
CEO and Project Manager
Aquafor Beech Limited

EXECUTIVE SUMMARY

The Selby Creek watershed drains an area of 138 km^2 and is located between the Salmon River watershed and the Napanee River watershed, all of which fall under Quinte Conservation's jurisdiction. Selby Creek extends 41 km from its farthest headwaters to its outlet point, passing through the northern outskirts of Napanee and discharging into the Bay of Quinte approximately 5.5 km west of Deseronto. The watershed has an elongated shape and drains in a southwesterly direction.

Aquafor Beech Limited (Aquafor) was retained by Quinte Conservation (QC) to establish updated regulatory floodplain mapping for key reaches along Selby Creek and its tributaries, through detailed hydrologic and hydraulic modelling, and analyses of any flood hazards. The 100-year storm event has been adopted by QC to regulate development within the floodplain and to identify risks to properties and roads within the study area. Quinte Conservation partnered with the Government of Canada (Natural Resources Canada, NRCan) and the Province of Ontario (Ministry of Natural Resources and Forestry) as part of the Flood Hazard Identification and Mapping Program (FHIMP) to develop flood hazard maps for municipalities and territories.

As part of the hydrologic component of the study, Aquafor developed a hydrologic model using the US Army Corps of Engineers HEC-HMS software (Ver. 4.11). Simulations were performed for the 2-year, 5-year, 10-year, 25-year, 50-year, 100-year (regulatory), and 200-year design storms, with the 200-year storm serving as a proxy for evaluating the regulatory storm under the effects of climate change. Junctions were strategically placed throughout the watershed to ensure adequate discretization, as well as to provide flow inputs to the hydraulic model at key locations along the reaches included in the floodplain mapping study. In total, the Selby Creek hydrologic model contains 35 flow nodes (junctions), 36 subbassins, and 28 reaches. The hydrologic modelling approach was consistent with the HEC-HMS User and Technical Reference Manuals (US Army Corps of Engineers, 2000, 2023) and the Technical Guidelines for Flood Hazard Mapping (OMNR, 2002; EWRG, 2017). A LiDAR-derived digital terrain model (DTM) having a 1 m resolution, produced by NRCan, was used as the primary data source for determining subbasin and reach characteristics.

A 1D HEC-RAS hydraulic model (US Army Corps of Engineers HEC-RAS software, version 6.3.1) was subsequently developed for the reaches of interest, comprising the main branch of Selby Creek from Airport Road to the Bay of Quinte, along with the downstream extents of three tributaries. A LiDAR-derived digital terrain model (DTM) having a 1 m resolution, produced by NRCan, was used in conjunction with field survey data to define stream and crossing structure geometries and to establish floodlines. The model was evaluated through a verification exercise and comparison with other studies. In total 48km of reaches, 659 cross-sections and 27 hydraulic structures were modelled. Following a review of the preliminary hydraulic results, a 2D HEC-RAS hydraulic model was also developed to model spill from one area within the project extents. A flood hazard assessment was then undertaken to determine overtopping depth and velocity at road crossings and the associated impacts to road access, as well as to identify potential flood impacts to buildings. The approach used for hydraulic modelling, floodline delineation, and the flood hazard assessment was consistent with the HEC-RAS User's Manual (US Army Corps of Engineers, 2023), the Technical Guide for River & Stream Systems: Flooding Hazard Limit (OMNR, 2002), the Federal Hydrologic and Hydraulic Procedures for Flood Hazard Delineation (NRCan, 2023), and the Technical Guidelines for Flood Hazard Mapping (EWRG, 2017).

A Public Information Centre (PIC) session was held on January 16th, 2024 to provide an overview of the project and methodology, and to present updated regulatory floodplain mapping. No formal comments were received.

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1 INTRODUCTION

The purpose of this study is to establish updated regulatory floodplain mapping for the Selby Creek watershed, through detailed hydrological and hydraulic modelling, and analyses of any flood hazards. The 100-year flood profile can be used by Quinte Conservation to regulate development within the floodplain, to protect developed areas through structural land acquisition measures, and to identify properties at risk within the study area. This report details the methodology and results of the hydrologic and hydraulic components of the floodplain mapping study and provides a summary of the public consultation process.

Quinte Conservation received funding to conduct this study, partnering with the with the Government of Canada (Natural Resources Canada) and the Province of Ontario (Ministry of Natural Resources and Forestry) as part of the Flood Hazard Identification and Mapping Program (FHIMP) to develop flood hazard maps for municipalities and territories.

As part of this study, Aquafor built a hydrologic model to simulate watershed response to rainfall and rain-on-snowmelt events over a range of return periods, producing estimates of peak flood flows at key locations throughout the Selby Creek watershed. The estimated flood flows will subsequently be used for hydraulic modelling, which will in turn be used to simulate water surface elevations and update the regulatory floodplain mapping for Selby Creek. Aquafor developed the hydrologic model using the US Army Corps of Engineers HEC-HMS software (Ver. 4.11), with the goal of simulating real watershed and watercourse responses following major rainfall and snowmelt events.

Key objectives of the hydrologic component of this study are as follows:

- Review all available background information provided by Quinte Conservation;
- Perform a data gap analysis to identify existing model deficiencies;
- Describe the watershed characteristics;
- Develop design storms from rainfall and rain-on snowmelt intensity-duration-frequency (IDF) curves
- Accurately delineate the watershed boundary;
- Build the HEC-HMS model by discretizing the watershed into smaller subbasins, placing junctions, and defining reaches for routing flows;
- Compare results for design storms with different durations and distributions, and select an appropriate set of design storms for subsequent hydrologic modelling, hydraulic modelling, and floodplain mapping; and
- Perform a sensitivity analysis.

As part of the hydraulic component of the study, Aquafor developed a 1D hydraulic model using the US Army Corps of Engineers HEC-RAS software (Ver. 6.3.1) and the Canadian Geodetic Datum of 2013 (CGVD2013) as vertical datum, as opposed to the older CGVD28 vertical datum. The model was developed based on a DTM (created from LiDAR data collected in 2022), topographic field survey data collected by Aquafor, and flows estimated by the hydrologic model. Simulations were performed for the 2-year, 5-year, 10-year, 25-year, 50-year, 100-year (regulatory), and 200-year design storms (also named 0.5, 0.2, 0.1, 0.04, 0.02, 0.01, and 0.005 AEP) with the 200-year storm (0.005 AEP) serving as a proxy for evaluating the regulatory storm under the effects of climate change.

Following a review of the preliminary 1D HEC-RAS model results, one area of spill was identified where event flows exceeded the extents of the 1D model. This area was identified for further mapping, and a 2D HEC-RAS model was developed to analyze flooding conditions surrounding Airport Road.

Key objectives of the hydraulic component of this study are as follows:

- Review all available background information provided by Quinte Conservation;
- Perform a data gap analysis to identify existing model deficiencies and missing road crossing information;
- Identify all watercourses crossing structures along the reaches of interest and complete field surveys;
- Develop a georeferenced 1-D hydraulic model using both HEC-RAS and GeoHECRAS platforms throughout the study area, based upon the LiDAR-derived DTM (Coordinate system NAD83 - UTM Zone 18N, vertical coordinate system CGVD2013);
- Develop a supplementary 2D hydraulic model using HEC-RAS to analyze flood extents resulting from spill beyond the 1D model extents in a select area;
- Incorporate the flood flow estimates from Aquafor's hydrologic model;
- Perform a boundary conditions scenario analysis;
- Generate riverine flood lines for the 50-year, 100-year (regulatory) and 200-year (climate change) return period storms;
- Identify flooding impact and extents;
- Identify flood-susceptible buildings and roadway;
- Identify areas of potential spills;
- Provide the regulated floodplain mapping sheets; and
- Provide the digital flood lines for all events.

2 DESCRIPTION OF THE WATERSHED

2.1 Drainage Network

Selby Creek (also sometimes referred to as Sucker Creek) drains a watershed that has an area of 138 km^2. As illustrated in **Figure 2-1**, the watershed has an elongated shape and is characterized by a main drainage branch (Selby Creek) and two major unnamed tributaries. The main branch runs in southerly direction, converging with the first tributary to the north of the Village of Selby and then through the northern outskirts of the town of Greater Napanee. Selby Creek confluences with the other tributaries further downstream, between Bells Road and Highway 49, and continues through the Tyendinaga Mohawk Territory before discharging into the Bay of Quinte. In total, the main branch extends 41 km from its farthest headwaters to the mouth of the creek.

As shown in **Table 2-1**, the watershed slope at the headwaters of the main branch of Selby Creek is initially relatively high (0.01040 m/m), after which the slope of main branch flattens out gradually, reaching 0.00083 m/m near the mouth of the creek at the Bay of Quinte. Average slopes along the main branch and the three major tributaries are summarized in **Table 2-2**, and an overview of the terrain elevations within the watershed is provided in **Figure 4-1. (Section 4.2).**

The Selby Creek watershed is predominantly rural, comprising forests, cultivated lands and wetlands. Small urban areas are also present, concentrated within the Village of Selby and within the northern part of Greater Napanee.

These areas generally produce more runoff compared to more rural areas, though no stormwater management facilities (SWMF) have been constructed within the study area to attenuate urban runoff.

Table 2-1: Summary of Channel Slope Variation Along the Main Branch of Selby Creek

Chainage - US to DS (m)	Upstream Elevation (m)	Downstream Elevation (m)	Avg Channel Slope (m/m)
0 - 1214	149.52	137.69	0.00974*
1214 - 6144	137.69	136.77	0.00019*
6144 - 18622	136.77	107.57	0.00234
18622 - 19907	107.57	92.64	0.01162**
19907 - 34952	92.64	77.64	0.00100
34952 - 41063	77.64	74.01	0.00059

*Includes headwaters along the longest flow path

**Steep slope

Table 2-2: Summary of Average Channel Slope for Major Reaches

Reach	Length (km)	Avg Channel Slope (m/m)
Selby Creek Main Branch*	41.1	0.00184*
Tributary 1	1.3	0.00319
Tributary 2	7.0	0.00357
Tributary 3	6.7	0.00135

*Includes headwaters along the longest flow path

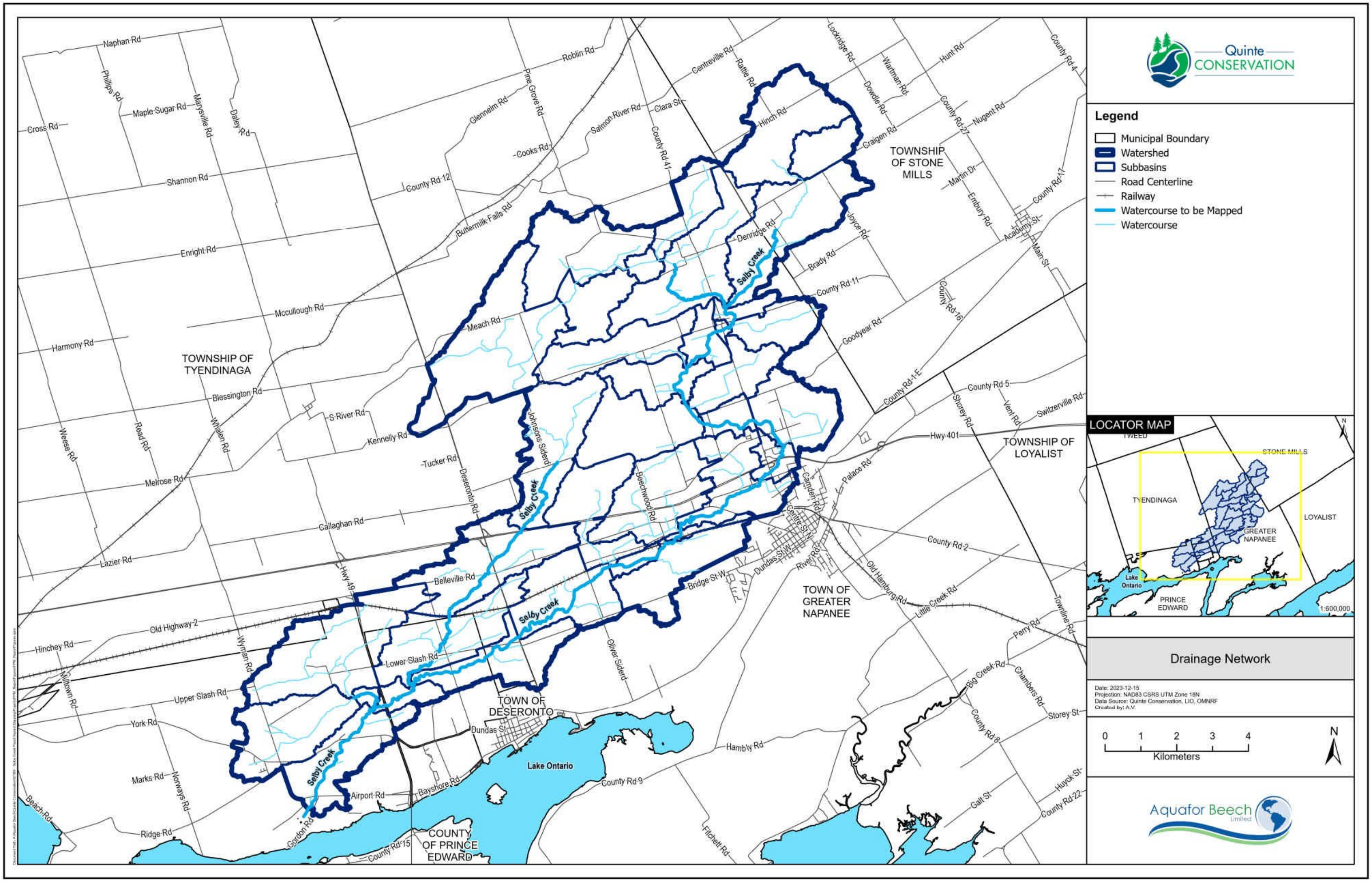


Figure 2-1: Selby Creek Drainage Network and Reaches to be Mapped

2.2 Geological Setting

The bedrock (Paleozoic) geology of the Selby Creek watershed consists of successional formations that are roughly parallel to the Lake Ontario shoreline, as shown in **Figure 2-1.** The north-east extremity of the watershed is occupied by the Gull River Formation (limestone and dolomite), which transitions into the Bobcaygeon Formation (primarily limestone), followed by the Verulam Formation (limestone and shale).

Surficial geology within the study area is shown in **Figure 2-3**. The northern half of Selby Creek watershed is occupied primarily by Paleozoic bedrock, with fine-textured glaciolacustrine deposits dominating the southern half. Pockets of organic deposits are also located within swamps throughout the watershed.

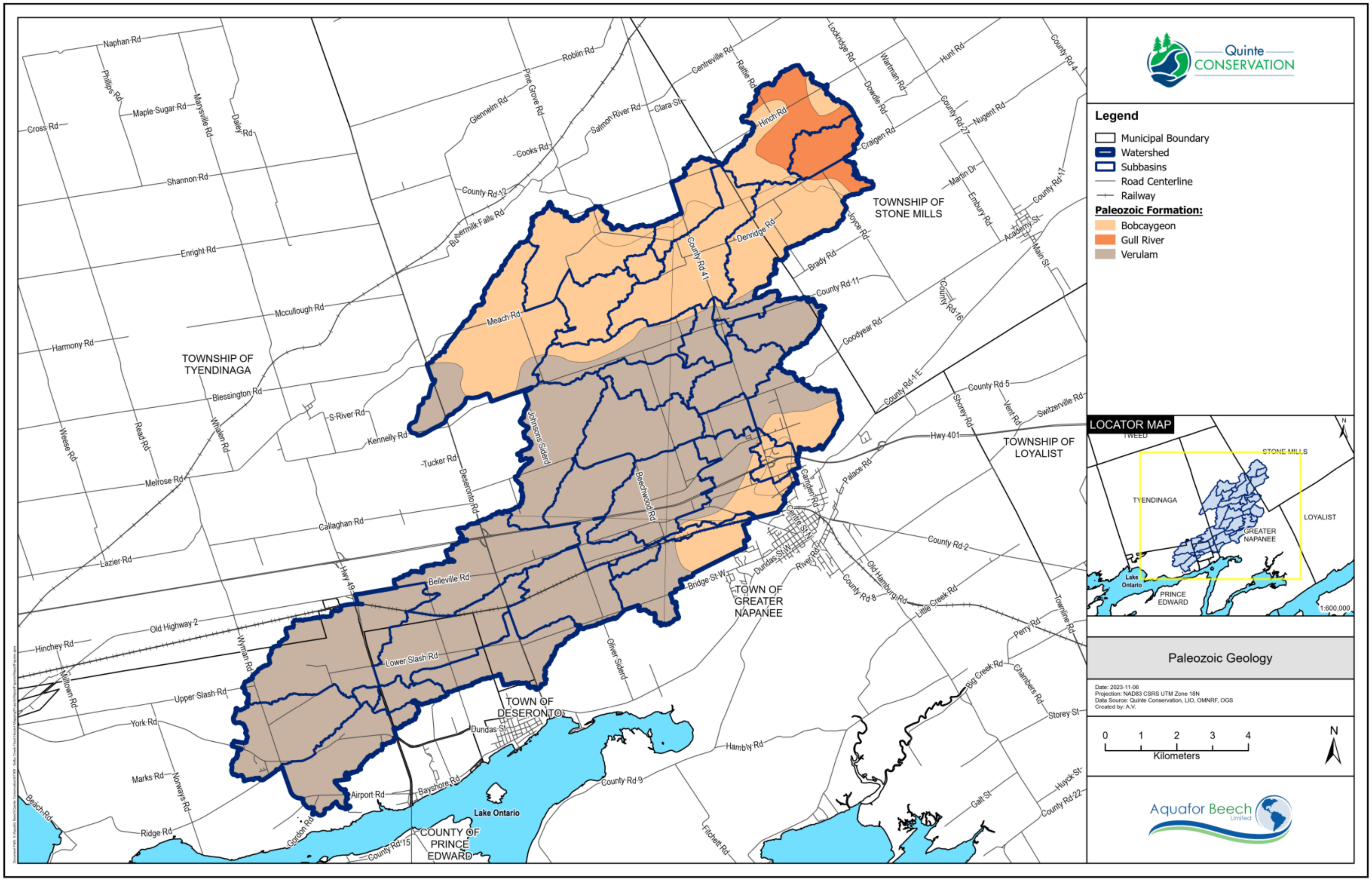


Figure 2-2: Paleozoic Geology within the Selby Creek Watershed

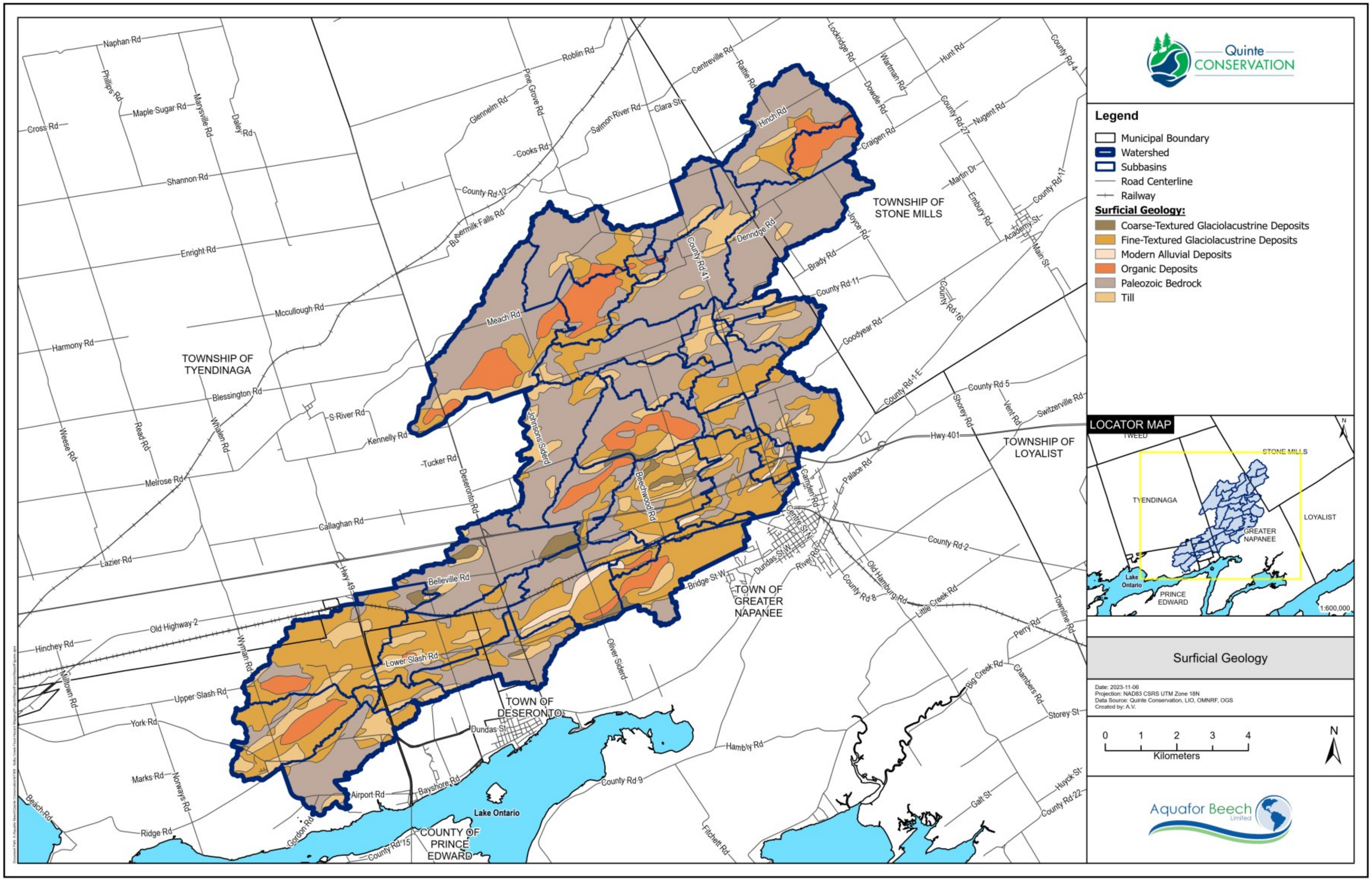


Figure 2-3: Surficial Geology within the Selby Creek Watershed

2.3 Land Use

The Southern Ontario Land Resource Information System (SOLRIS, ver. 3.0) was used as a basis for determining land use within the study area. Aquafor modified the database in order to identify recent residential development (based on aerial imagery) and large commercial impervious areas, so as to improve runoff estimates in the hydrologic model. The file was also modified to differentiate between urban and rural roads.

As shown in **Figure 2-4,** the Selby Creek watershed is predominantly rural, with development mostly concentrated within the Towns of Selby and Greater Napanee. Agricultural and undifferentiated land (which mainly corresponds to agricultural land and undeveloped land) account for 66.7% of the watershed area. Wetlands (including marsh, treed and thicket swamp) cover 18.7% of the watershed. Other land uses include: forests (10.3%); transportation (2.6%); built-up impervious and commercial areas (0.8%); and built-up pervious areas (0.5%), which generally consist of open spaces with little impervious development, such as parks and schools. Other land uses classified as extraction areas, open cliffs, and open water areas account for 0.4% of the total watershed area.

The application of land use and soil hydrological classification for calculating runoff is described in **Sections 4.7** and **4.8,** with CN values and other key parameters shown in **Table 4-2**.

2.4 Hydrological Soil Groups

Soil classification data was provided by Quinte Conservation, having originally been obtained from the MNRF's Soil Survey Complex GIS file. The resulting distribution of hydrologic soil groups is shown in **Figure 2-5.**

The majority of soils within the watershed are poorly drained (Hydrologic Soil Group D), though soils with higher infiltration capacity (Hydrologic Soil Groups B) are common in the upper portions of the watershed. In the Selby Creek watershed, Group D occupies 52% of the total area, followed by Groups B (43%), C (4%), and A (1%).

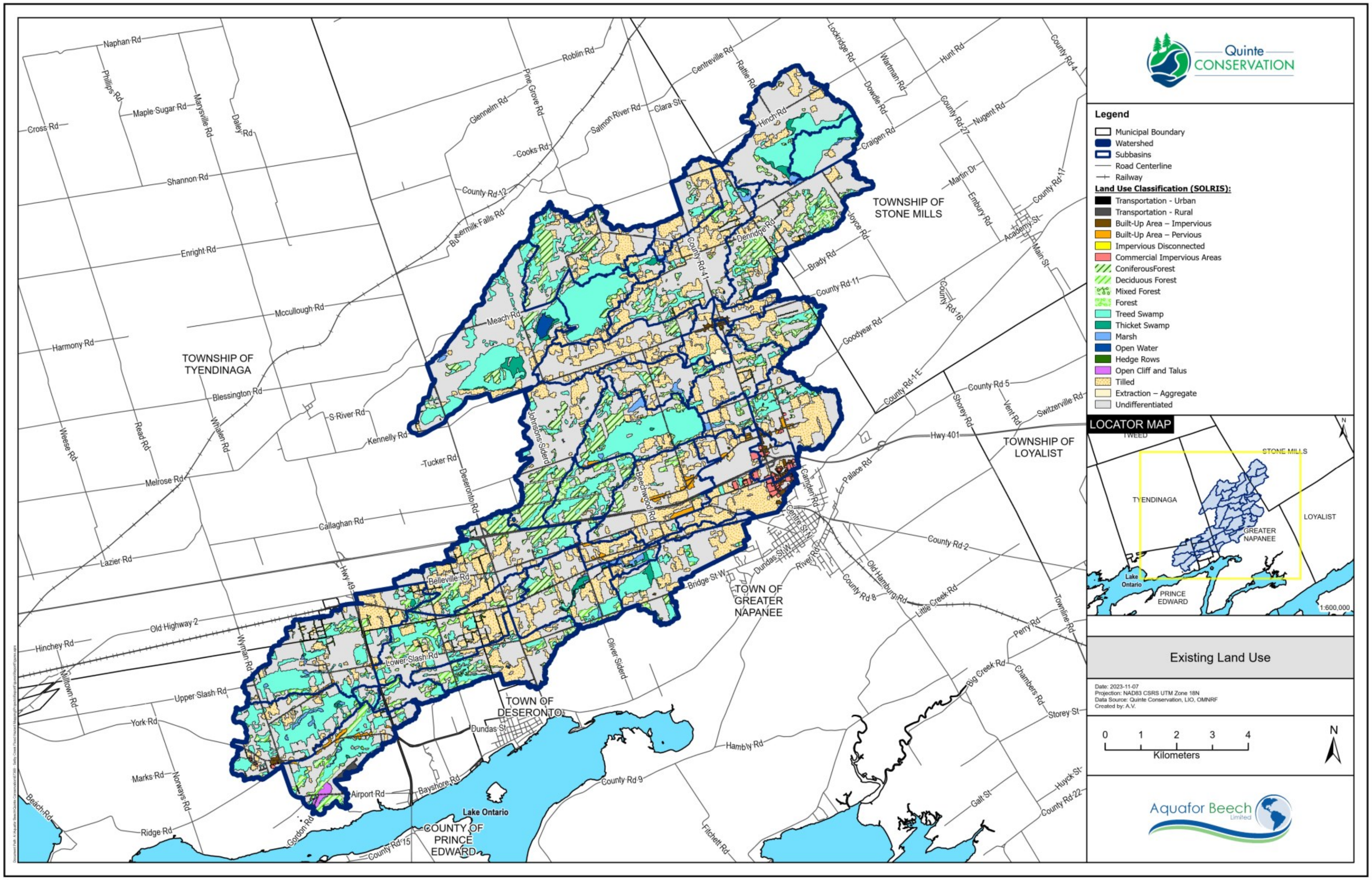


Figure 2-4: Existing Land Use within the Selby Creek Watershed

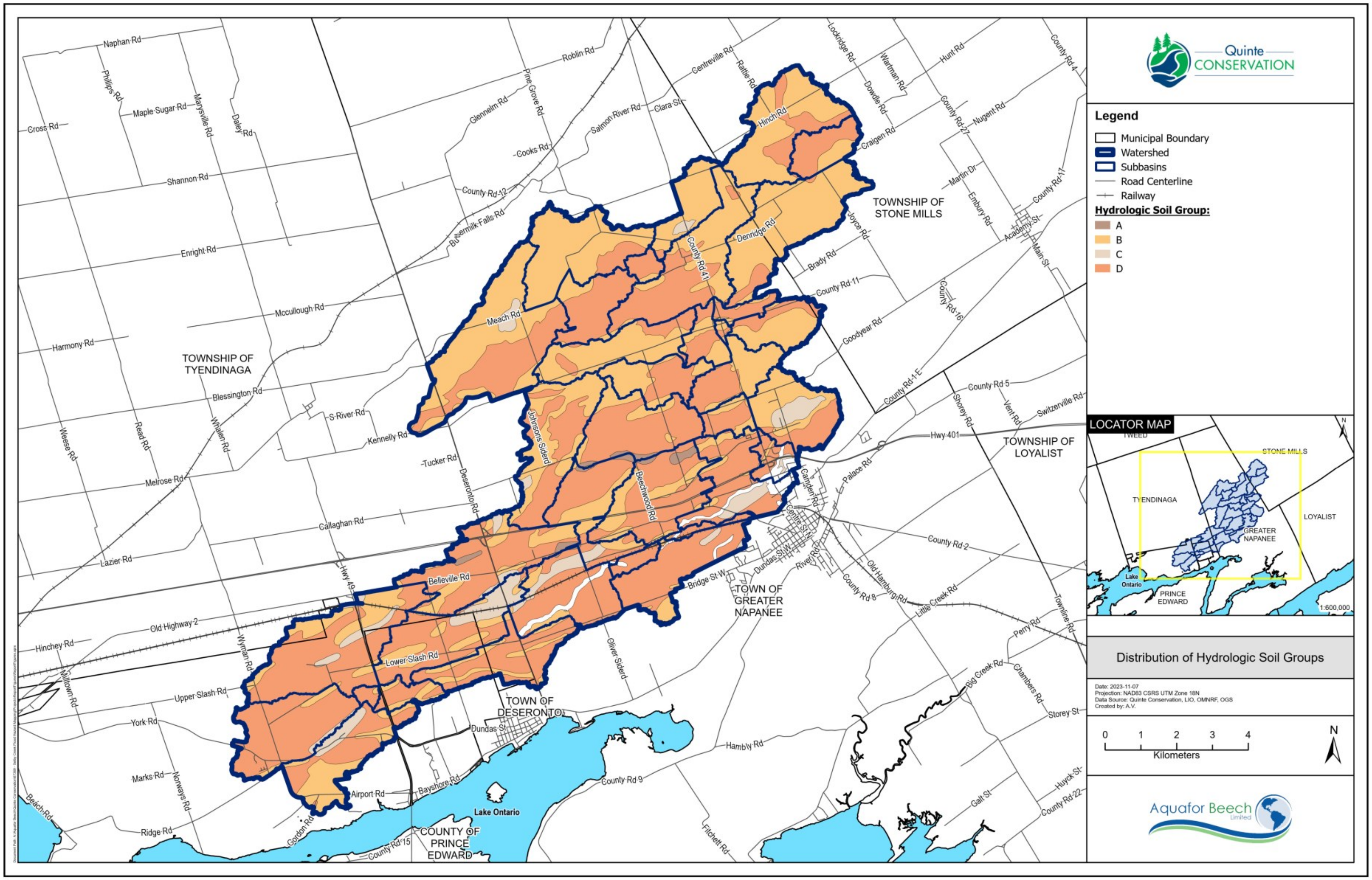


Figure 2-5: Distribution of Hydrologic Soil Groups within the Selby Creek Watershed

3 PREVIOUS STUDIES

Previous hydrologic/hydraulic studies that included the Selby Creek watershed are described below.

Marysville and Selby Creeks Floodline Mapping Study Report (Crysler & Lathem Ltd., 1981)
This study was undertaken on behalf of the Napanee Region Conservation Authority to provide the Napanee-Salmon Watershed Studies Phase III, including floodplain mapping of Selby Creek and Marysville Creek (Part C of the complete study). A regional analysis was completed to estimate flood flows during 24-hour snowmelt events. A simulation of design storms using the US Soil Conservation Service TR-20 model was also undertaken, in which the SCS curve number (CN) infiltration loss method was applied. IDF curves were obtained for the Belleville and Kingston Airport stations managed by Environment Canada, from which average rainfall depths for the 2-year to 100-year return period were calculated to develop the design storms for the watershed. The SCS Type II distribution with a 12-hour duration was used, and an areal reduction factor of 0.95 was applied based on the Design of Small Dams approach.

A comparison of results from the regional analysis and hydrologic models suggested that the snowmelt events produce the highest flows in the main branch of Selby Creek, through the SCS storms produced the largest flows for tributaries, especially located upstream subbasins. For each subbasin, flows used for regulatory floodplain mapping were selected as the larger of the snowmelt 100-year frequency analysis peaks flows and the 100-year SCS Type II peak flows, in order to adopt a conservation approach. This resulted in a regulatory peak flow of 85.3 m^3/s at the mouth of the creek.

Selby Creek Master Drainage Plan (Water Plan Associates and XCG, 2011)
This study was undertaken on behalf of QC as part of the Bay of Quinte Regional Master Drainage Planning Project Program. The study area was located within the Town of Greater Napanee and did not cover the entirety of the Selby Creek watershed, especially near the watershed outlet. As mentioned in the report, the hydrologic analysis was undertaken with a higher level of detail than the 1981 Floodplain Mapping study (Crysler & Lathem Ltd.). A HEC-HMS model was developed and was used to only simulate for 100-year rainfall events, with no consideration given to snowmelt events. Three conditions were modeled: pre-development conditions; future-conditions without stormwater quantity controls; and future conditions with stormwater management (SWM) measures in place. The design storm used to represent the regulatory storm was the 100-year SCS Type II, 12-hour duration. The CN method under the average Antecedent Moisture Condition (AMC II) was used to simulate infiltration losses. Peak flows for Selby Creek at Highway 401 were estimated to be 41.7m^3/s under existing conditions and 43.6 m^3/s under future conditions with SWM facilities, compared to 43.5 m^3/s from the 1981 floodplain mapping study.

4 HYDROLOGIC MODEL

4.1 Model Selection and Setup

The hydrologic model was created using the US Army Corps of Engineers HEC-HMS software (Ver. 4.11). HEC-HMS was selected because it is publicly available and is widely used for floodplain mapping studies, incorporating a variety of loss, transform, and routing methods.

The model was set up using the NAD83 (CSRS) UTM Zone 18N horizontal coordinate system and the CGVD2013 vertical datum. All associated GIS files used the same coordinate system and vertical datum.

4.2 Digital Elevation Model

Quinte Conservation provided a LiDAR-derived digital terrain model (DTM) that was produced by Natural Resources Canada (NRCan) based on LiDAR data collected in 2022. The DTM has a 1 m horizontal resolution and is referenced to the CGVD2013 vertical datum. It was used in the hydrologic study for discretizing the watershed into smaller subbasins, tracing longest flow paths, determining reach and longest flow path slopes, and extracting cross-section elevations used for channel routing. Local relief throughout the watershed, based on the DTM, is illustrated in **Figure 4-1** below.

4.3 Timestep

The model computation time step should be less than 1/5 of the smallest subbasin time to peak (lag time). This represents a time step of 5 mins for the study area, which was selected for the model's control specifications.

The time interval for Muskingum-Cunge routing is dependent on reach index flow, which was selected as the average between baseflow (assumed to be negligeable for the study areas) and peak flow within each reach, as per the HEC-HMS Technical Reference Manual and User Manual (US Army Corps of Engineers, 2000).

4.4 Subbasin Discretization

The Selby Creek watershed was subdivided into 36 subbasins having relatively uniform land use, topography, and soil texture. Discretization was performed using the DTM and the watercourse network GIS shapefile. The discretized subbasins are shown in **Figure 4-2** and the model structure is shown in **Figure 4-3** below. Subbasin properties and model inputs are shown in **Table A.1** (**Appendix A**).

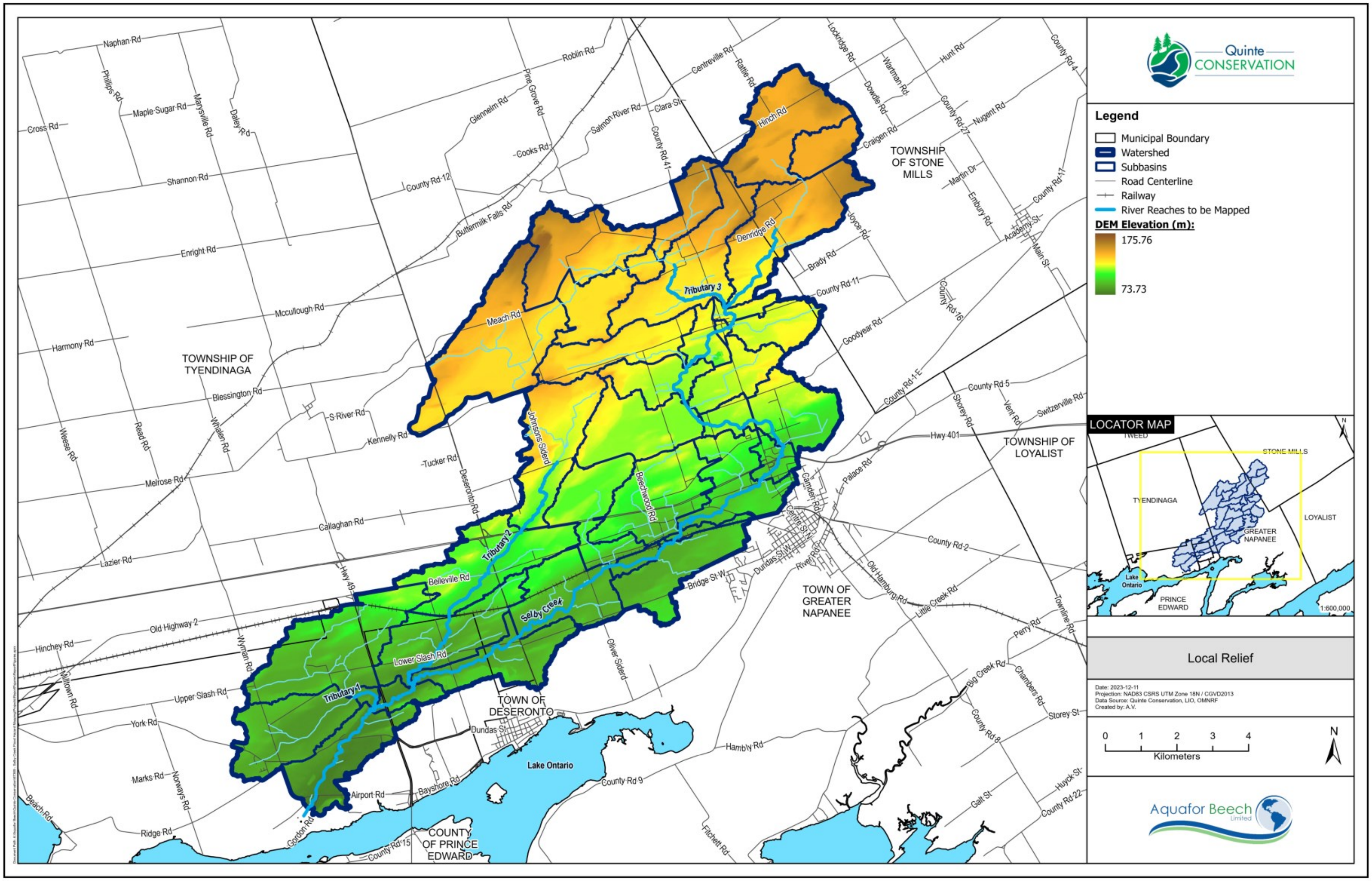


Figure 4-1: Local Relief within the Selby Creek Watershed, as per the LiDAR-derived DTM

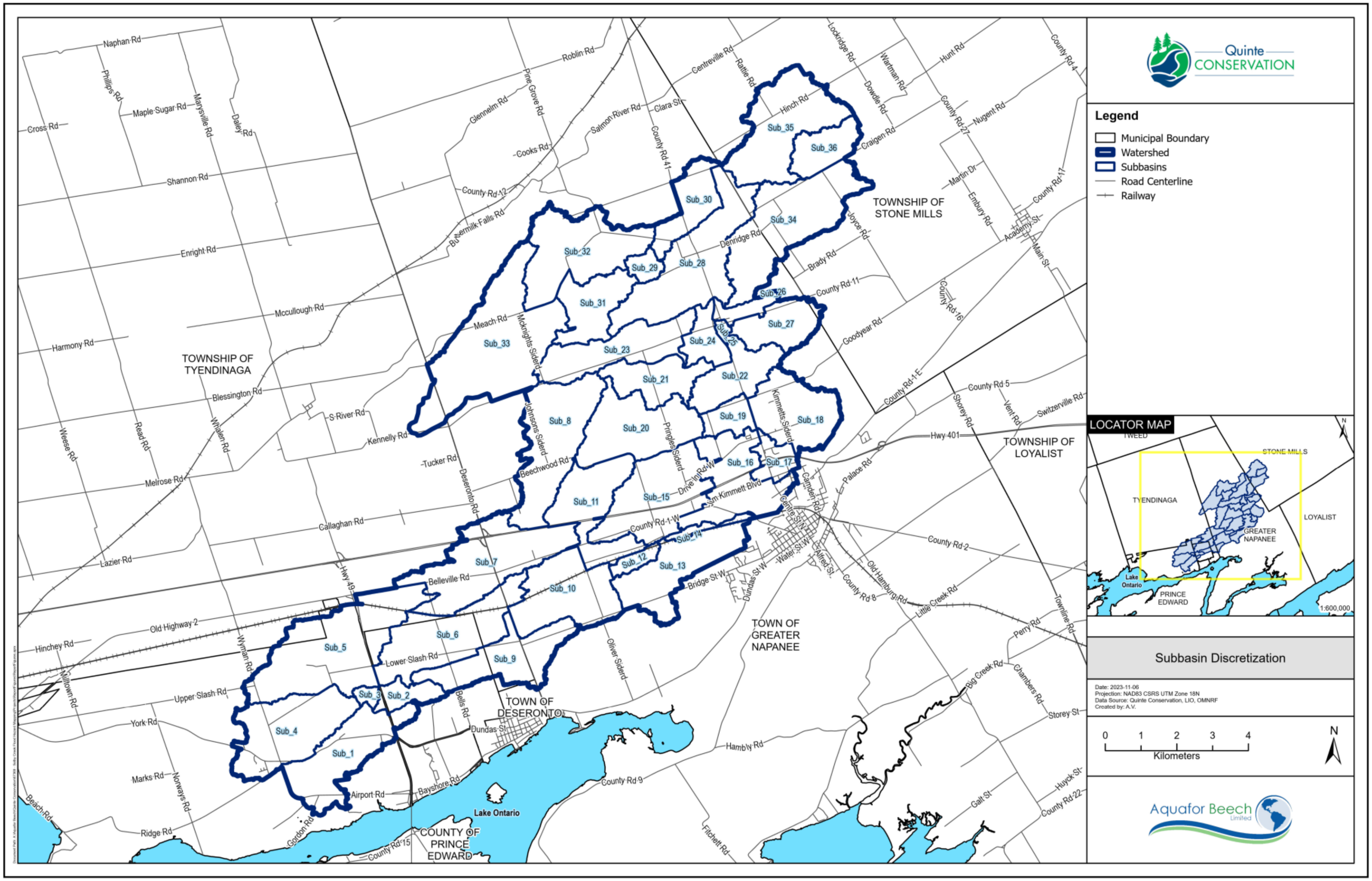


Figure 4-2: Delineated Subbasins in the HEC-HMS Model

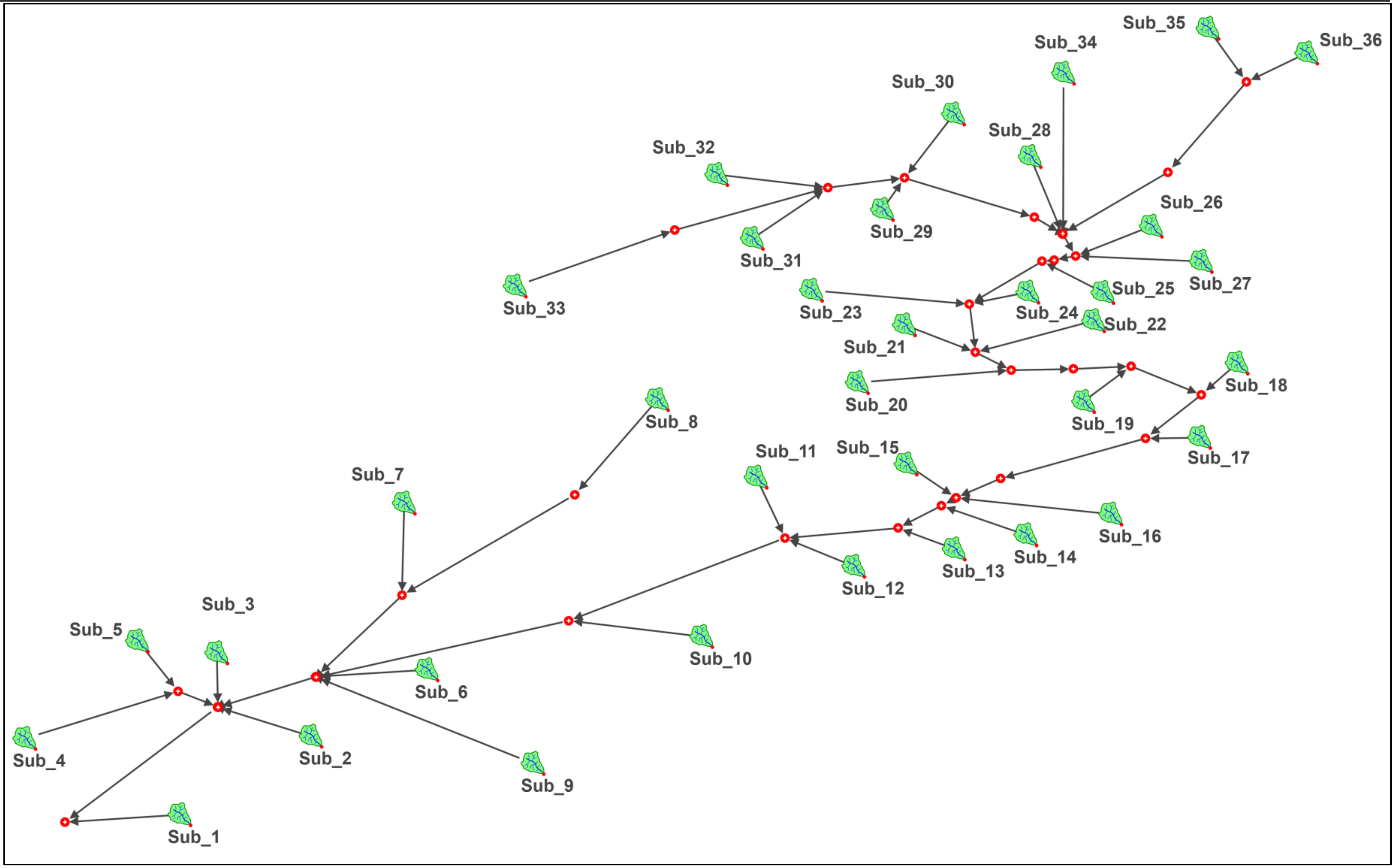


Figure 4-3: Schematic of the Model Structure

4.5 Reach Routing

Reaches were defined using the watercourse network GIS file managed by the QC. Stream slopes were determined using the LiDAR-derived DTM.

A review of the DTM revealed that most reaches are characterised by wide floodplains. In addition, some of the reaches are characterized by low slopes (≤0.002 m/m). Based on these characteristics, the Muskingum-Cunge routing method was selected and was applied using representative 8-point cross-sections, which were defined for each reach using the DTM. This approach accounts for overbank flow and is appropriate for modelling flow in reaches with low slopes (US Army Corps of Engineers, 2000, 2023). Standard values of Manning's n for natural channels and overbanks were applied to the reaches (**Table 4-1**).

The locations of the reaches and the junctions connecting them are shown in **Figure 4-4.** A summary of reach parameters is provided in **Table A.2** (**Appendix A**). The cross-section used for routing flows along Reach 1 is shown as an example in **Figure A.1** (**Appendix A**).

Table 4-1: Standard Manning Roughness Coefficients for Open Channels (EWRG et al., 2017)

Land Cover	Standard 'n' Value
Natural Channel	0.035
Woods (Overbank)	0.07
Meadows (Overbank)	0.055
Wetland (Overbank)*	0.06 and 0.065
Lawns (Overbank)	0.05

*Defined by Aquafor to be equivalent to overbank meadows

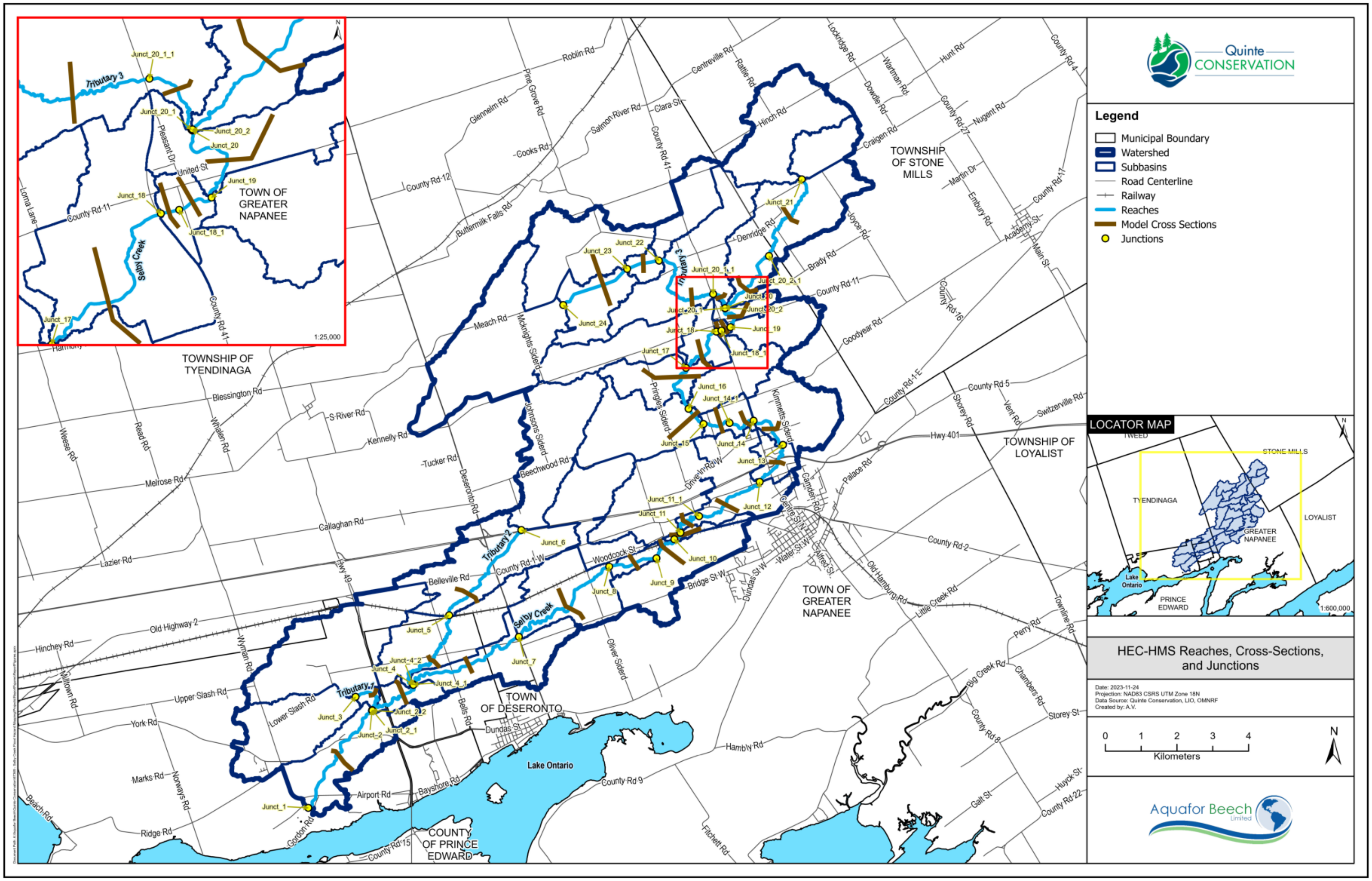


Figure 4-4: HEC-HMS Reaches, Cross-Sections, and Junctions

4.6 Transform Method and Lag Time

The SCS unit hydrograph transform method was selected, as per standard practice, and the standard shape factor of 484 was used in all subbasins. Lag time, t_{lag} was calculated from time of concentration, t_c, using the equation:

$$t_{lag} = 0.6t_c$$

Time of concentration was calculated using 'C' runoff coefficients, which were defined for each land use based on either standard values (for homogeneous land use types) or sampled values (for residential areas and transportation corridors), as described in **Section 4.8** below. For subbasins having a composite 'C' value greater than 0.4, the Bransby-Williams equation was used to calculate time of concentration:

$$t_c = 0.057 \cdot L \cdot S_w{}^{-0.2} \cdot A^{-0.1}$$

where L [m] is length of the watershed's longest flowpath, S_w [%] is slope of the watershed's longest flowpath, and A [ha] is watershed area.

For samples having a composite 'C' value less than 0.4, the Airport equation (MTO, 1997) was used to calculate time of concentration:

$$t_c = 3.26 \cdot (1.1 - C) \cdot L^{0.5} \cdot S_w{}^{-0.33}$$

4.7 Infiltration Loss

The SCS curve number (CN) method for infiltration loss was adopted for estimating runoff, from which precipitation excess (equivalent to runoff in this case) can be calculated. HEC-HMS calculates precipitation excess, P_e [mm], as follows:

$$P_e = \frac{(P - I_a)^2}{P - I_a + \frac{(25400 - 254\,CN)}{CN}}$$

where *P* [mm] is precipitation and I_a [mm] is initial abstraction. Values of CN under average antecedent moisture conditions (AMC II) and initial abstraction corresponding to each land use type are shown in **Table 4-2** in **Section 4.8** below.

4.8 Land Use Parameters

Standard values of initial abstraction, CN, directly connected imperviousness, and 'C' runoff coefficients are defined for homogeneous land use types. However, parameters for land uses containing a mix of impervious and pervious surfaces – namely, residential areas and transportation corridors – should be defined locally for the study area to increase model accuracy. Representative samples were therefore collected to estimate impervious fractions for urban residential areas, urban transportation corridors, and rural transportation corridors; three samples were collected for each of these land use types. The area occupied by directly and indirectly connected impervious surfaces was measured for each sample using areal imagery and building polygons obtained from Google Earth, and was divided by total sample area to determine impervious fractions.

Average values of initial abstraction, CN, directly connected imperviousness, and 'C' runoff coefficients were then calculated based on the sampled values. Average CN values for sampled land uses were calculated by considering the fractions occupied by porous areas (lawns) and indirectly connected impervious areas. Directly connected impervious areas were specified as a separate parameter within the model and were not included in the calculation of composite CN values. Parameters for all land use types are recorded in **Table 4-2**. These values were subsequently used to calculated composite (weighted average) values for each subbasin.

Table 4-2: Standard Land Use Hydrologic Parameters (based on EWRG et al., 2017)

Land use	CN under AMC II				I_a (mm)	'C' Coefficient	% Connected Impervious
	Hydrologic Soil Group						
	A	B	C	D			
Woods	32	60	73	79	10	0.3	0
Meadows	38	65	76	81	8	0.35	0
Cultivated	62	74	82	86	7	0.45	0
Lawns	49	69	79	84	5	0.15	0
Commercial Impervious	–	–	–	–	2	0.95	100
Open Water	100	100	100	100	0	0.95	0
Gravel	76(1)	85(1)	89(1)	91(1)	4(2)	0.5(3)	0
Bedrock / Alvar	80(2)	80(2)	80(2)	80(2)	5(2)	0.5(2)	0
Wetlands	100(2)	100(2)	100(2)	100(2)	25(4)	0.05	0
Built Up – Impervious(5)	54	72	80.9	85.4	4.39	0.31	0
Built Up – Pervious (Lawns)	49	69	79	84	5	0.15	0
Transportation - Urban(5)	49	69	79	84	3.93	0.43	35.58
Transportation - Rural(5)	66.4	79.3	85.8	89	3.93	0.43	0

(1) From TR-55 Report (USDA, 1986)
(2) Assumed
(3) From the MTO Drainage Manual (MTO, 1997)
(4) From Watt et al. (1989)
(5) Determined from sampling

4.9 Rainfall and Rain-on-Snowmelt Design Storm

For the Selby Creek watershed, the regulatory event is defined as the 100-year storm, with the 200-year storm serving as a proxy for evaluating the regulatory storm under the effects of climate change, per MNRF (2023) guidelines. Both the rainfall and rain-on-snowmelt design storms were evaluated, as described in the sections below.

4.9.1 *Rainfall Event*

- IDF Curves and Equivalent Rainfall Depths for the 2-year to 100-year Storm Events

Average equivalent rainfall depths, derived from IDF curves, were obtained for Environment Canada's Belleville and Kingston Airport climate stations (Stations # 6150689 and # 6104142, respectively). Rainfall depths from these stations are provided in the **Appendix B**. Since the Selby Creek watershed is located about halfway between Belleville and Kingston, the equivalent rainfall depths for the watershed were determined by calculating the average of the depths for the two stations. These average depths are recorded in **Table 4-3**.

Table 4-3. Average of the Total Rainfall Depth from the Belleville and Kingston Stations

Average Total Rainfall (mm)						
Duration	**2-yr**	**5-yr**	**10-yr**	**25-yr**	**50-yr**	**100-yr**
5min	7.35	9.85	11.5	13.6	15.2	16.75
10min	10.85	14.05	16.2	18.85	20.8	22.75
15min	13.55	17.4	19.9	23.1	25.5	27.8
30min	17.95	22.95	26.3	30.5	33.65	36.75
1h	22.45	29.4	34.05	39.95	44.35	48.65
2h	26.95	36.35	42.55	50.4	56.25	62.05
6h	36.5	48.95	57.2	67.6	75.35	83
12h	**42.85**	**57.15**	**66.7**	**78.65**	**87.6**	**96.45**
24h	48.75	63.05	72.6	84.55	93.45	102.3

- Distribution and Duration

Various design storm distributions (AES and SCS) and durations (6-hour, 12-hour and 24-hour) were considered and evaluated through preliminary hydrologic modelling. Ultimately, the AES (30%) distribution with a 12-hour duration was deemed to be the most appropriate for representing the summer rainfall events within the Selby Creek watershed. Additional details regarding the selection of the design storm distribution and duration are provided in **Section 4.10**.

Since IDF curves were only defined up to the 100-year storms, it was necessary to determine the equivalent rainfall depth for the 200-year return period through extrapolation. This was achieved by fitting a 2nd order polynomial distribution to logarithms of the 10-year to 100-year storms and corresponding rainfall depths. The fitted curve for the 12-hour duration is shown in **Figure 4-5**, from which a rainfall depth of 107.7 mm was determined.

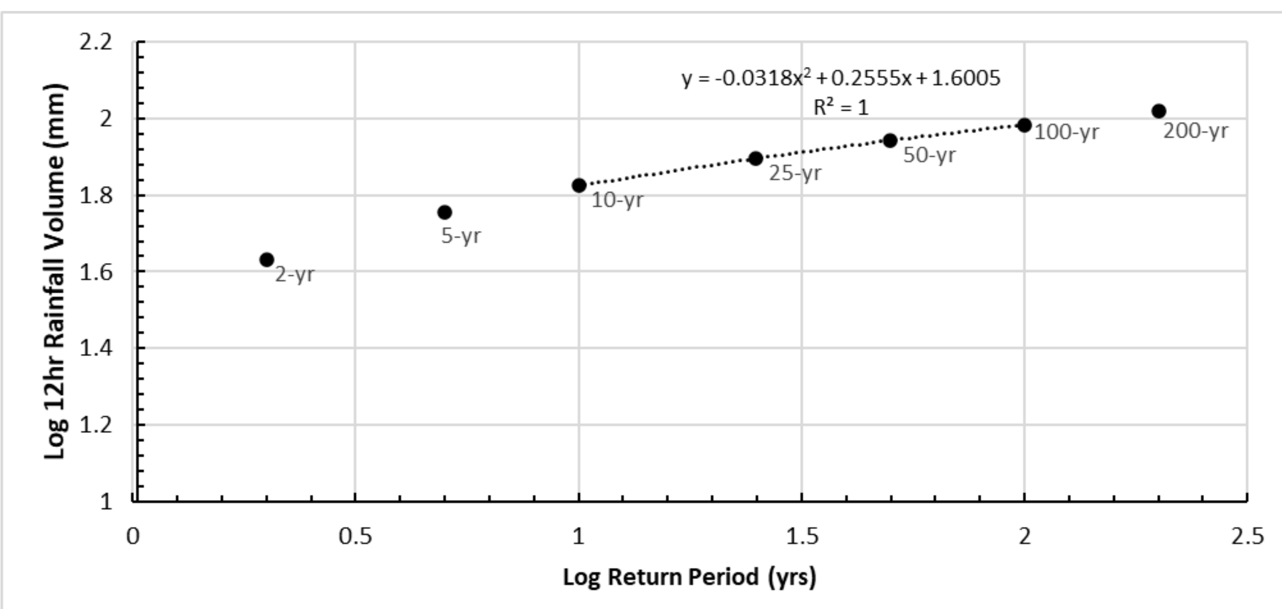


Figure 4-5. Log-Log Plot of 12-hr Storm Rainfall Depths vs. Return Period with Line of Best Fit

- Areal Reduction Factor

Areal Reduction Factors (ARF) are used in many hydrologic designs to transform a point precipitation estimate of a given duration and frequency to a corresponding estimate over a watershed area. In order to avoid over-estimating the amount of rainfall that occurs throughout the Selby Creek watershed, ARFs were determined using the circular area method. The circular area for the Selby Creek watershed was calculated to be 485 km^2. This corresponds to an areal reduction factor of 0.882, based on World Meteorological Organization (WMO) relationships developed for 12-hour storm distributions (**Figure 4-6**). The resulting rainfall depths (**Table 4-4)** were used to create 12-hour AES design storms for the 2-year, 5-year, 10-year, 25-year, 50-year, 100-year, and 200-year rainfall storm events. The final rainfall distribution data for each of these design storms is provided in **Appendix C**.

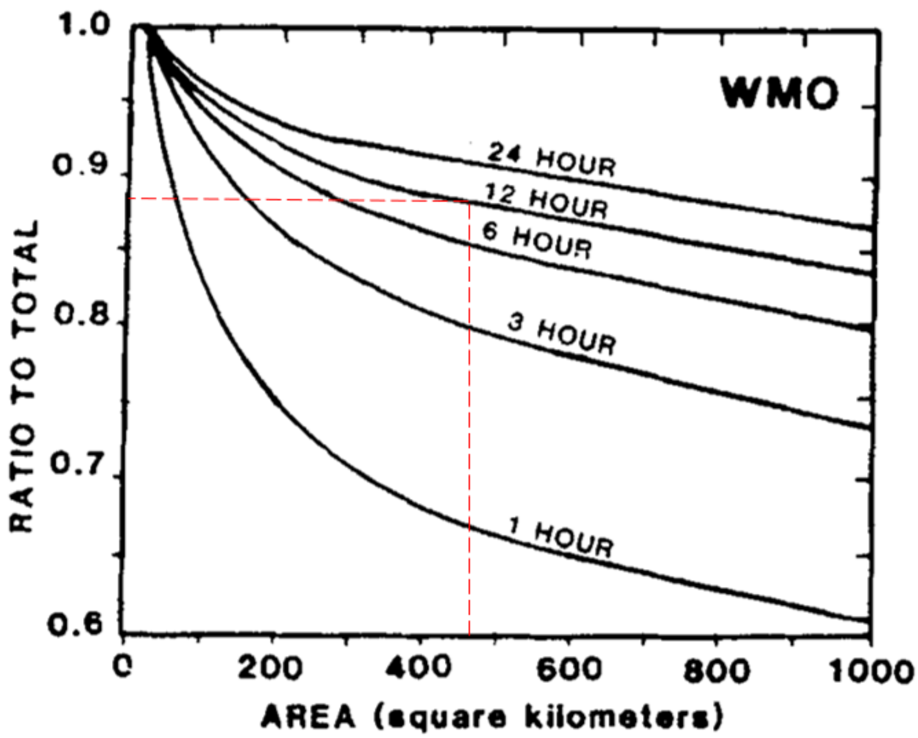


Figure 4-6: WMO Areal Reduction Factors

Table 4-4. Total Rainfall Depth for the 12-hr Duration

Total Rainfall (mm) for 12-hr Duration		
Return Period	Without ARF	With ARF (0.882)
2-yr	42.85	37.79
5-yr	57.15	50.41
10-yr	66.70	58.83
25-yr	78.65	69.37
50-yr	87.60	77.26
100-yr	96.45	85.07

200-yr (extrapolated)	107.70	94.99

4.9.2 *Rain-on-Snowmelt Event*

The rain-on-snowmelt distributions were developed using a method that has previously been used by the Upper Thames River Conservation Authority (UTRCA, 2004). First, rain-on-snowmelt IDF curves for 1- to 10-day durations at the Kingston Airport station and the Belleville station were retrieved from Environment Canada's database. These IDF curves were derived using Snowmelt Model 4 (Southern Ontario model). Average equivalent precipitation depth was determined from these two stations in order to establish a single set of IDF curves for the 1- to 10-day durations. Precipitation for each duration under the 200-year event was extrapolated by fitting a 2nd order polynomial distribution to logarithms of the 10-year to 100-year rain-on-snowmelt storm events and corresponding precipitation depths; the fitted curve for the 1-day duration is shown as an example in **Figure 4-7**. Equivalent precipitation depths for the individual stations, along with average depths, are recorded in **Table 4-5**.

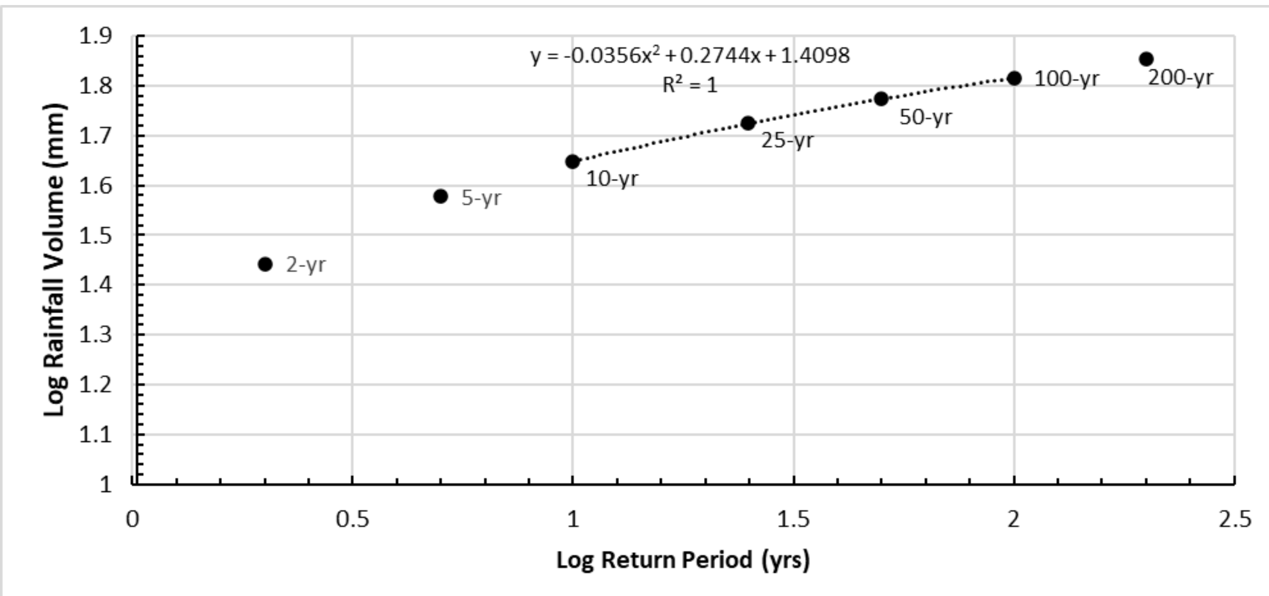


Figure 4-7. Log-Log Plot of 1-Day Rain-on-Snowmelt Precipitation Depths vs. Return Period, with the Fitted Polynomial Distribution

Next, precipitation depth for the 10-day duration was broken down into 1-day increments, starting with the precipitation depth for the 1-day duration and then calculating incremental increases in depth for the 2-day to 10-day durations. The resulting precipitation incremental depths were subsequently assigned ranks from 1 to 10, with Rank 1 representing the peak day (i.e., the day with the highest precipitation) and Rank 10 representing the day when the smallest amount of precipitation occurred. An example of this process is shown in **Table D1** (**Appendix D**) for the 100-year storm events. Daily precipitation was then ordered based on rank, per **Table D2**.

For the peak day, precipitation was defined in 2-hour increments based on the winter rainfall distribution published by Brater, Sangal and Sherrill (1974), which is shown in **Table D3**. In order to avoid over-estimating the amount of rainfall that occurs throughout the watershed, an areal reduction factor was applied to the peak day rainfall depths. As for the rainfall design storm, the circular area of the Selby Creek watershed was used, which corresponds to an areal reduction factor of 0.907 per the WMO relationship defined for 24-hour rainfall events (**Figure 4-6**: WMO Areal Reduction Factors).

For each of the remaining days constituting the 10-day event, precipitation was defined in 2-hour depth increments based on a modified sinusoidal distribution representing the distribution of precipitation intensity. Modifications to this function consisted of added a value of 1 to the sine function and shifting it right by 6 hours, such that the value of the final function value was 0 at the beginning and end of each day (i.e., at t = 00:00). The fraction of precipitation occurring over each 2-hour interval was determined by calculating the integral over the period and dividing the result by the integral over an entire day (**Table D4**). For each day, precipitation depth over each time interval could then be calculated by multiplying the fraction of precipitation by total daily precipitation. No areal reduction factors were applied to non-peak day precipitation depths. The rain-on-snowmelt design storm patterns accounting for the areal reduction factor was used in the hydrologic model and detail data is provided in **Table D5** of the **Appendix D**.

Table 4-5. Equivalent Rain-on-Snowmelt Precipitation Depths for the 1-day to 10-day Durations

Return Period	Storm Duration									
	1-day	2-day	3-day	4-day	5-day	6-day	7-day	8-day	9-day	10-day
Kingston Airport Station (mm)										
2-yr	29.5	38.2	43.5	48.8	53.9	59.1	64.4	69.5	73.5	76.8
5-yr	40.4	50.5	56.5	62.5	69.0	76.3	83.3	89.6	95.7	100.9
10-yr	47.6	58.6	65.2	71.5	78.9	87.6	95.7	102.9	110.4	116.9
25-yr	56.8	68.9	76.1	83.0	91.5	102.0	111.4	119.8	128.9	137.1
50-yr	63.5	76.5	84.1	91.5	100.9	112.6	123.1	132.3	142.7	152.1
100-yr	70.3	84.1	92.2	99.9	110.1	123.2	134.7	144.7	156.4	166.9
Belleville Station (mm)										
2-yr	25.9	35.0	41.0	45.9	50.8	55.4	59.6	63.5	66.8	69.5
5-yr	35.2	46.0	53.8	60.2	67.3	73.4	79.8	85.2	90.4	95.0
10-yr	41.4	53.3	62.3	69.7	78.2	85.4	93.2	99.6	106.0	111.9
25-yr	49.2	62.6	73.1	81.6	91.9	100.5	110.1	117.8	125.7	133.3
50-yr	55.0	69.4	81.0	90.5	102.2	111.7	122.6	131.3	140.4	149.1
100-yr	60.8	76.2	88.9	99.3	112.3	122.8	135.0	144.7	154.9	164.9
Average (mm)										
2-yr	27.7	36.6	42.2	47.4	52.3	57.2	62.0	66.5	70.1	73.1
5-yr	37.8	48.2	55.2	61.4	68.1	74.8	81.5	87.4	93.0	98.0
10-yr	44.5	56.0	63.7	70.6	78.5	86.5	94.4	101.3	108.2	114.4
25-yr	53.0	65.7	74.6	82.3	91.7	101.2	110.8	118.8	127.3	135.2
50-yr	59.3	73.0	82.6	91.0	101.5	112.1	122.9	131.8	141.5	150.6
100-yr	65.5	80.1	90.5	99.6	111.2	123.0	134.9	144.7	155.7	165.9
200-yr (extrapolated)	71.2	86.8	97.9	107.6	120.2	133.0	146.0	156.6	168.7	180.0

4.10 Design Storm Discussion for Regulatory Event

The hydrologic model was run to simulate different design storm durations and distributions for summer rainfall storms, as well as to compare them with rain-on-snowmelt events. Peak flow results at Highway 401 and watershed outlet have been compared for the 2-year and 100-year (regulatory) storm events as shown in **Table 4-6** and **Table 4-7**, respectively. ARFs were applied to all design storms.

Table 4-6. Estimated Peak Flow at Highway 401 (m^3/s) – HEC-HMS Junction 12

Return Period	SCS 6-hr	SCS 12-hr	AES 12-hr	SCS 24-hr	Rain-on-Snow
2-yr	5.7	8.2	8.4	9.6	13.2
100-yr	29.0	36.5	38.7	39.25	44.4

Table 4-7. Estimated Peak Flow at Selby Creek Watershed Outlet (m^3/s) – HEC-HMS Junction 1

Return Period	SCS 6-hr	SCS 12-hr	AES 12-hr	SCS 24-hr	Rain-on-Snow
2-yr	10.9	15.3	16.3	18.4	25.5
100-yr	53.9	69.1	71.0	72.5	84.9

The purpose of this study is to develop regulatory floodplain mapping for key reaches of the Selby Creek watershed network. Based on the watershed characteristics, previous studies, input from Quinte Conservation, and the comparison of the peak flows for the different design storms, it was determined that the AES 12-hr duration is the design storm that best represents summer rainfall events. However, the 10-day rain-on-snowmelt events produced higher peak flows, and Quinte Conservation has indicated that annual peak flows usually occur spring snowmelt period. As such, it was decided that all subsequent hydrologic modelling, hydraulic modelling, and floodplain mapping would be conducted using the rain-on-snowmelt events for the 2-year to 200-year return periods, with the 100-year rain-on-snowmelt event serving as the regulatory event.

4.11 Peak Flow Results

The hydrologic model was run for the 2-year, 5-year, 10-year, 25-year, 50-year, 100-year, and 200-year rain-on-snowmelt events. Results are presented at key junctions in
Table 4-8 below; peak flows for all junctions are recorded in **Appendix E**. The 200-year event, which serves as a proxy for evaluating the 100-year (regulatory) event under the effects of climate change (MNRF, 2023), produces substantially higher flows than the 100-year event. Based on this analysis, it is anticipated that regulatory peak flows at the downstream extents of Selby Creek would increase from 84.9m^3/s to 100.15 m^3/s due to climate change, which represents a 18% increase.

Table 4-8: Peak Flows (m^3/s) at Key Locations throughout the Selby Creek Watershed

Junction ID	Location	2-yr	5-yr	10-yr	25-yr	50-yr	100-yr	200-yr
Junct_20	Confluence of Reach 28-DS and Reach 34-DS	9.73	15.11	18.77	23.45	26.88	30.32	33.49

Junct_13	Upstream of Napanee	13.07	20.36	25.55	32.73	38.38	44.1	49.47
Junct_12	Downstream of Napanee	13.22	20.52	25.79	32.97	38.65	44.43	49.83
Junct_4	Confluence of Reach 6 and Reach 9	18.9	30.16	39.2	51.84	61.15	70.67	79.64
Junct_2	Confluence of Reach 3 and Reach 2	24.59	38.24	48.14	61.67	72.35	83.38	93.72
Junct_1	Watershed Outlet	25.52	38.78	49.15	62.98	73.76	84.93	95.94

5 HYDROLOGIC MODEL EVALUATION

5.1 Comparison with Previous Studies

Results from Aquafor's model were compared, at two locations, against results from the 1981 Floodplain Mapping Study (at the most downstream junction) of the watershed and against the Master Drainage Plan (2011) (at Highway 401).

As shown in **Table 5-1** and **Table 5-2,** the 100-year peak flows from Aquafor's model were similar to those predicted by the 1981 Floodplain Mapping Study and the 2011 Master Drainage Plan. In fact, the difference between the flows estimated by both studies was less than 5 m^3/s at both the Highway 401 junction and at the watershed outlet.

Table 5-1. Peak Flow Comparison (m^3/s) for the 2-year and 100-year Storms at Highway 401 (Junction 12)

Return Period	Aquafor Rain-on-Snowmelt	FPM Study (1981)	MDP Study – pre-construction (2011)	MDP Study – post-construction with SWM (2011)
2-yr	13.2	/	/	/
100-yr	44.4	43.5	41.7	43.6

Table 5-2. Peak Flow Comparison (m^3/s) for the 2-year and 100-year Storms at Watershed Outlet (Junction 1)

Return Period	Aquafor Rain-on-Snowmelt	FPM Study (1981)
2-yr	25.5	/
100-yr	84.9	85.3

5.2 Regional Flood Frequency Analysis

As a point of comparison, a Regional Flood Frequency Analysis was performed using the Modified Index Flood Method, as described in the MTO Drainage Management Manual (MTO, 1997). This manual provides design charts that can be used to calculate an estimated 25-year Index Flood, Q_{25} [m^3/s], using the equation:

$$Q_{25} = C_{25} \times A^{0.75}$$

where C_{25} is a watershed class coefficient for the 25-year Index Flood and A [km^2] is watershed area.

The Selby Creek watershed has an area of 137.97 km^2. It is a Shield Type type basin because it is characterized by shallow soils with outcrops of bedrock, and has a large storage fraction (18.7% occupied by wetlands and open water). From the MTO design charts, it was determined that the base watershed class is 4.78 based on the storage fraction (wetlands and lakes), corresponding to a C_{25} value of 0.59.

Therefore, the 25-year Modified Index Flood value is:

$$Q_{25} = 0.59 \times 137.97^{0.75} = 23.8 \text{ m}^3/\text{s}$$

This estimate is much lower than the modelled peak flow of 63.0 m^3/s at the watershed outlet under the 25-year rain-on-snowmelt event. However, it should be noted that index flows rely only on generalized empirical relationships and do not account for certain factors such as IDF data, basin shape, reach cross-sectional geometry, and Manning's roughness. Furthermore, although the Selby Creek watershed is best described as a Shield Type type basin due to its large storage fraction and shallow soils, much of the surficial soil is classified as clay rather than loam, which would increase runoff. Additionally, the majority of the watershed is occupied by agricultural land (66.7%), whereas Shield Type basins are typically dominated by forests; this would be expected to increase peak flow due to lower interception storage and lower lag times (compared to forested areas).

5.3 Sensitivity Analysis

A sensitivity analysis was conducted to evaluate the effects of varying CN values, initial abstraction, impervious fraction, lag time, and Manning's "n" on 100-year peak flows at the watershed outlet. Sensitivity was evaluated by applying a range of factors for each parameter.

The results of the sensitivity analysis are recorded in **Table 5-3** to **Table 5-7.** It was found that the model had a relatively high degree of sensitivity to antecedent moisture conditions, which impacted peak flows at the downstream extents of Selby Creek by up to 27.2% compared to baseline conditions. The model had a moderate degree of sensitivity to Manning's "n", with peak flows changing by up to 13.1%, but had a low sensitivity to lag time, initial abstraction, and impervious fraction.

Table 5-3: Impact of Varying CN on Peak Flows at the Watershed Outlets under the 100-year Rain-on-Snowmelt Event

Multiplication Factor	Peak Flow (m^3/s)	Peak Flow Change (%)
0.8	61.87	-27.2
0.9	73.46	-13.5
1.0 (baseline)	84.93	0.0
1.1	95.06	11.9
1.2	100.08	17.8

Table 5-4: Impact of Varying Initial Abstraction on Peak Flows at the Watershed Outlets under the 100-year Rain-on-Snowmelt Event

Multiplication Factor	Peak Flow (m^3/s)	Peak Flow Change (%)
0.8	85.34	0.5
0.9	85.14	0.2
1.0 (baseline)	84.93	0.0
1.1	84.72	-0.2
1.2	84.50	-0.5

Table 5-5: Impact of Varying Impervious Fraction on Peak Flows at the Watershed Outlets under the 100-year Rain-on-Snowmelt Event

Multiplication Factor	Peak Flow (m^3/s)	Peak Flow Change (%)
0.8	84.91	0.0
0.9	84.82	-0.1
1.0 (baseline)	84.93	0.0
1.1	84.95	0.0
1.2	84.95	0.0

Table 5-6: Impact of Varying Lag Time on Peak Flows at the Watershed Outlets under the 100-year Rain-on-Snowmelt Event

Multiplication Factor	Peak Flow (m^3/s)	Peak Flow Change (%)
0.8	86.11	1.4
0.9	85.44	0.6
1.0 (baseline)	84.93	0
1.1	84.48	-1
1.2	84.04	-1

Table 5-7: Impact of Varying Manning's "n" on Peak Flows at the Watershed Outlets under the 100-year Rain-on-Snowmelt Event

Multiplication Factor	Peak Flow (m^3/s)	Peak Flow Change (%)
0.8	96.02	13.1
0.9	89.34	5.2
1.0 (baseline)	84.93	0.0
1.1	80.93	-4.7
1.2	78.12	-8.0

6 BACKGROUND REVIEW AND SITE RECONNAISSANCE FOR HYDRAULIC MODEL

6.1 Background Data Review

At onset of the study, Aquafor collected and compiled all pertinent background information from Quinte Conservation, including:

- A high-resolution DTM that was derived from LiDAR data collected in 2022;
- The Marysville and Selby Creeks Floodline Mapping Study Report (Crysler & Lathem Ltd., 1981);
- The Selby Creek Master Drainage Plan (Water Plan Associates and XCG, 2011); and
- GIS data layers for existing floodlines, land use, soils, watercourse centrelines, waterbodies, and wetlands.

Additional GIS data (e.g., road network and building polygons) was retrieved from open-source platforms.

6.2 Coordinate System and Vertical Datum

As per the FHIMP program, the floodplain mapping and associated models have to be developed in a specific geospatial reference system. For consistency, all topography data such as the DTM LiDAR derived, Aquafor GPS survey, where in the same coordinate system and vertical datum as follows:

- Horizontal Datum: North American Datum of 1983 (NAD83), Canadian Spatial Reference System (CSRS)
- Projection: Universal Transverse Mercator, Zone 18 North (UTM 18N)
- Vertical Datum: Canadian Geodetic Vertical Datum of 2013 (CGVD2013)

For elevations that were provided in CGVD28 (i.e., Bay of Quinte water level), Aquafor converted the elevations to CGVD2013. Per the NRCan Passive Control Network Tool, elevations referenced to CGVD2013 are 0.354m lower than those referenced to CGVD28 at the 67U113 benchmark station, which was used for converting between vertical datums within the study area.

6.3 LiDAR-Derived DTM

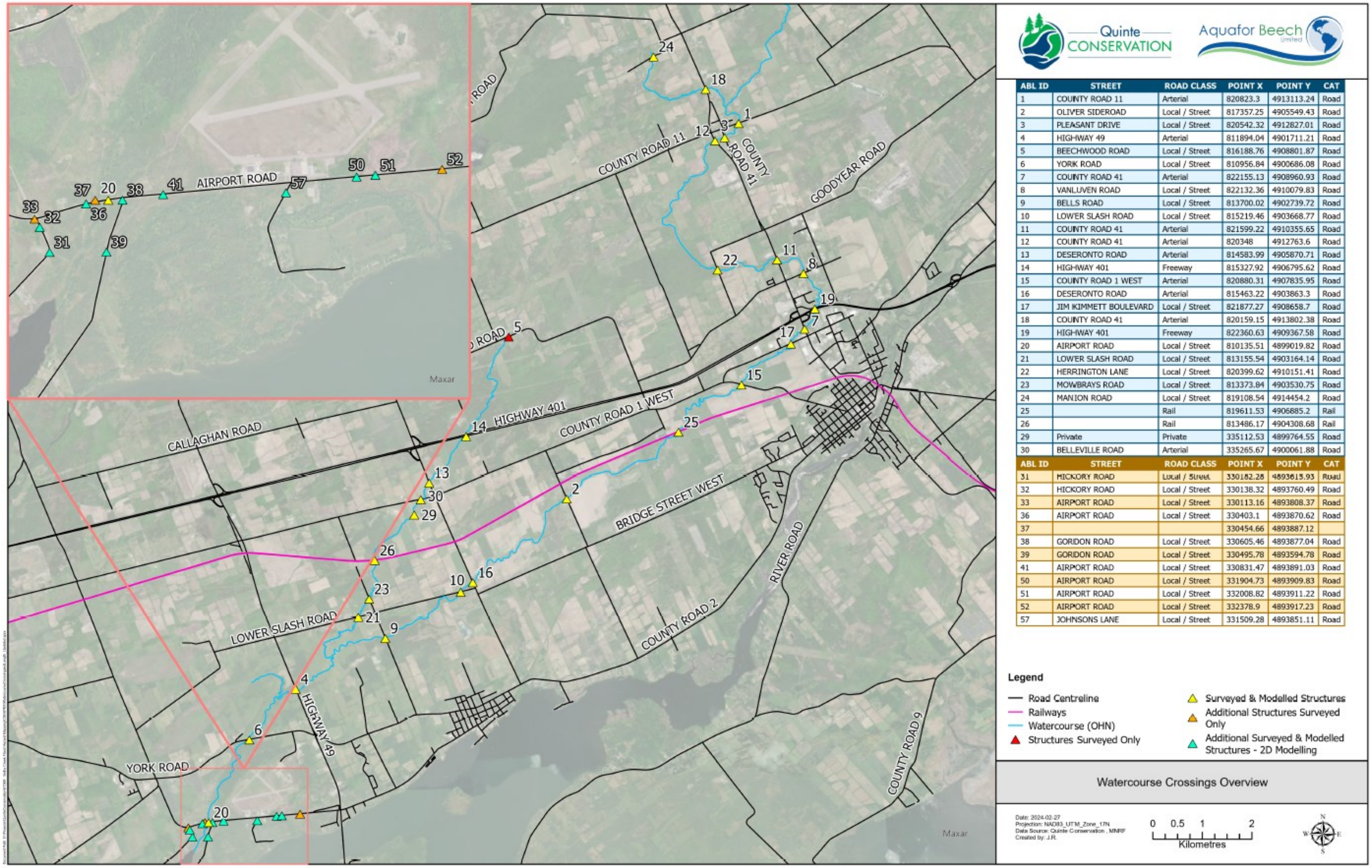
Quinte Conservation provided a digital terrain model (DTM) with a 1 m horizontal resolution that was produced by Natural Resources Canada (NRCan) based on LiDAR data collected in 2022. The DTM was the primary elevation data source for defining the geometries of model components and delineating flood extents. It is referenced to the NAD83 (CSRS) UTM Zone 18N horizontal coordinate system and the CGVD2013 vertical datum.

6.4 Structure Inventory and Survey

Crossing structures within the 1D and 2D study areas were identified and indexed by Aquafor based upon the preliminary review of GIS mapping and aerial imagery. The centrelines of the reaches included in the study were plotted on a map, and public crossings were identified as structures to be included within the 1D hydraulic model. Working with Quinte Conservation, a total of 28 hydraulically significant crossing structures were identified to be surveyed for the extents of the 1D model. The topographic survey was completed for all 28 structures, though only 27 were ultimately included in the hydraulic model, since one of the structures (ABL_ID 5) was located outside of the model bounds. The survey included structure type and material, location and size of openings and headwalls/wingwalls, heights, depth of embedment, culvert entrance types, invert and obvert elevations, etc.

Following the 1D culvert inventory, several culverts were identified as potentially hydraulically significant to the 2D hydraulic analysis. These additional structures were also identified and indexed by Aquafor using GIS mapping and aerial imagery in combination with the preliminary 1D hydraulic modelling results. A second field survey was completed to characterize these culverts and, where relevant and possible, collect topographic survey data. It should be noted that some of the additional structures are on private lands, and could not be accessed at the time of the field investigation.

A structure tracking sheet and summaries of field inventories at each crossing structure, in the form of structure inventory sheets, are compiled in **Appendix F**. **Figure 6-1** illustrates the general location of each hydraulic structure incorporated into both the 1D and 2D models.



ABL ID	STREET	ROAD CLASS	POINT X	POINT Y	CAT
1	COUNTY ROAD 11	Arterial	820823.3	4913113.24	Road
2	OLIVER SIDEROAD	Local / Street	817357.25	4905549.43	Road
3	PLEASANT DRIVE	Local / Street	820542.32	4912827.01	Road
4	HIGHWAY 49	Arterial	811894.04	4901711.21	Road
5	BEECHWOOD ROAD	Local / Street	816188.76	4908801.87	Road
6	YORK ROAD	Local / Street	810956.84	4900686.08	Road
7	COUNTY ROAD 41	Arterial	822155.13	4908960.93	Road
8	VANLUVEN ROAD	Local / Street	822132.36	4910079.83	Road
9	BELLS ROAD	Local / Street	813700.02	4902739.72	Road
10	LOWER SLASH ROAD	Local / Street	815219.46	4903668.77	Road
11	COUNTY ROAD 41	Arterial	821599.22	4910355.65	Road
12	COUNTY ROAD 41	Arterial	820348	4912763.6	Road
13	DESERONTO ROAD	Arterial	814583.99	4905870.71	Road
14	HIGHWAY 401	Freeway	815327.92	4906795.62	Road
15	COUNTY ROAD 1 WEST	Arterial	820880.31	4907835.95	Road
16	DESERONTO ROAD	Arterial	815463.22	4903863.3	Road
17	JIM KIMMETT BOULEVARD	Local / Street	821877.27	4908658.7	Road
18	COUNTY ROAD 41	Arterial	820159.15	4913802.38	Road
19	HIGHWAY 401	Freeway	822360.63	4909367.58	Road
20	AIRPORT ROAD	Local / Street	810135.51	4899019.82	Road
21	LOWER SLASH ROAD	Local / Street	813155.54	4903164.14	Road
22	HERRINGTON LANE	Local / Street	820399.62	4910151.41	Road
23	MOWBRAYS ROAD	Local / Street	813373.84	4903530.75	Road
24	MANION ROAD	Local / Street	819108.54	4914454.2	Road
25		Rail	819611.53	4906885.2	Rail
26		Rail	813486.17	4904308.68	Rail
29	Private	Private	335112.53	4899764.55	Road
30	BELLEVILLE ROAD	Arterial	335265.67	4900061.88	Road
ABL ID	STREET	ROAD CLASS	POINT X	POINT Y	CAT
31	HICKORY ROAD	Local / Street	330182.28	4893615.93	Road
32	HICKORY ROAD	Local / Street	330138.32	4893760.49	Road
33	AIRPORT ROAD	Local / Street	330113.16	4893808.37	Road
36	AIRPORT ROAD	Local / Street	330403.1	4893870.62	Road
37			330454.66	4893887.12	
38	GORDON ROAD	Local / Street	330605.46	4893877.04	Road
39	GORDON ROAD	Local / Street	330495.78	4893594.78	Road
41	AIRPORT ROAD	Local / Street	330831.47	4893891.03	Road
50	AIRPORT ROAD	Local / Street	331904.73	4893909.83	Road
51	AIRPORT ROAD	Local / Street	332008.82	4893911.22	Road
52	AIRPORT ROAD	Local / Street	332378.9	4893917.23	Road
57	JOHNSONS LANE	Local / Street	331509.28	4893851.11	Road

Figure 6-1. Location of the Watercourse Crossing Structures

6.5 Topographic and Bathymetric Survey

Aquafor identified data gaps within the LiDAR-derived DTM regarding low flow channel characteristics. Therefore, in addition of the detailed hydraulic structure survey, Aquafor undertook a detailed topographic survey of river cross-sections at different locations to complete the hydraulic modelling geometry. In total, 119 stream cross-sections were surveyed throughout the study area, including 4 cross-sections at each crossing structure (2 upstream and 2 downstream) and an additional 7 cross-sections located between structures to better define watercourse bathymetry. The topographic elevation survey of the river cross-sections identifies the locations and elevations of the thalweg, streambed topography, bottom and top of bank, bottom and top of slop, and overbank topography within the floodplain.

7 1D HYDRAULIC MODEL DEVELOPMENT

7.1 Model Software and Platform

The updated Selby Creek 1D hydraulic model was built using a combination of the HEC-RAS software (ver. 6.3.1) and the GeoHECRAS platform, which integrates HEC-RAS modelling directly with various GIS tools and data sources (AutoCAD drawings, elevation rasters, CSV files, ESRI shapefiles, etc.). Preliminary floodplain mapping was also generated using GeoHECRAS from HEC-RAS results and elevation surface data.

7.2 Channel Network and Cross-Sections

The watercourse network to be mapped was determined by Aquafor in primary during the hydrology assessment and then was redefined using the 2022 LiDAR derived DTM using GeoHECRAS software. Within a HEC-RAS hydraulic model, the term “River” refers to a watercourse made of multiple “Reaches”. In total, 47.8 km of river divided on seven (7) reaches with a singular nomenclature are simulated in the hydraulic model as presented in **Table 7-1**.

Table 7-1. HEC-RAS River and Reach Nomenclature

HEC-RAS River Name	HEC-RAS Reach Name	Total Channel Length (m)
Creek	Selby_1	4061
Selby Creek	Selby_2	1778
Selby Creek	Selby_3	24080
Selby Creek	Selby_4	3217
Selby Creek	Selby_T1	1379
Selby Creek	Selby_T2	9927
Selby Creek	Selby_T3	3364
Total Length (m)		**47805**

A base model was generated in GeoHECRAS using the 2022 LiDAR-derived DTM. In addition to the watercourse network, this elevation data was used to define channel cross-sections and overbank locations for the majority of the study area. At select locations where more detailed survey information was obtained through the field investigation, the low flow channel was redefined with detailed topographic survey data (119 cross-sections surveyed) for better representation of the bed channel, top of bank and bottom of bank. A total of 659 cross sections were built to define the channel geometry and floodplain geometry.

Cross-sections were spaced to account for changes in channel geometry, meanders, bridge/culvert/weir structures, and to account for the narrowest sections of the creeks. Additionally, cross-sections were placed close enough to ensure accurate computation of the energy losses. As per standard modeling procedures, cross-sections were extended across the entire floodplain and oriented perpendicular to the anticipated flow lines.

7.3 Hydraulic Structures (Bridges and Culverts)

Hydraulic structures included in this study consisted of bridges and culverts. Four cross-sections (i.e., 2 upstream and 2 downstream of each structure) were coded at each crossing structure to define streambed and floodplain geometry at close proximity of the structure, as well as to account for expansion and contraction of the flow at these structures.

The spacing of these cross-sections was consistent with the HEC-RAS reference manual (US Army Corps of Engineers, 2022), estimated using the recommended flow expansion and contraction. In general, the locations for the upstream cross-sections were selected by assuming a typical flow contraction ratio of 1:1, while the downstream cross-section locations were selected based on expansion ratios that were typically in the range of 2:1 (**Figure 7-1**).

Structure parameters were then coded consistent with the approaches defined in **Figure 7-1**, including the structure material, opening dimensions, invert elevations, skew angles, depth of embedment, etc. Road profiles were mostly defined using LiDAR DEM and cross referenced with background information. In addition, the height of railing was added to the road profile for certain bridges, where the railing or fencing was anticipated to act as a blockage under high flows. In this case, 100% blockage was coded in the model to be conservative.

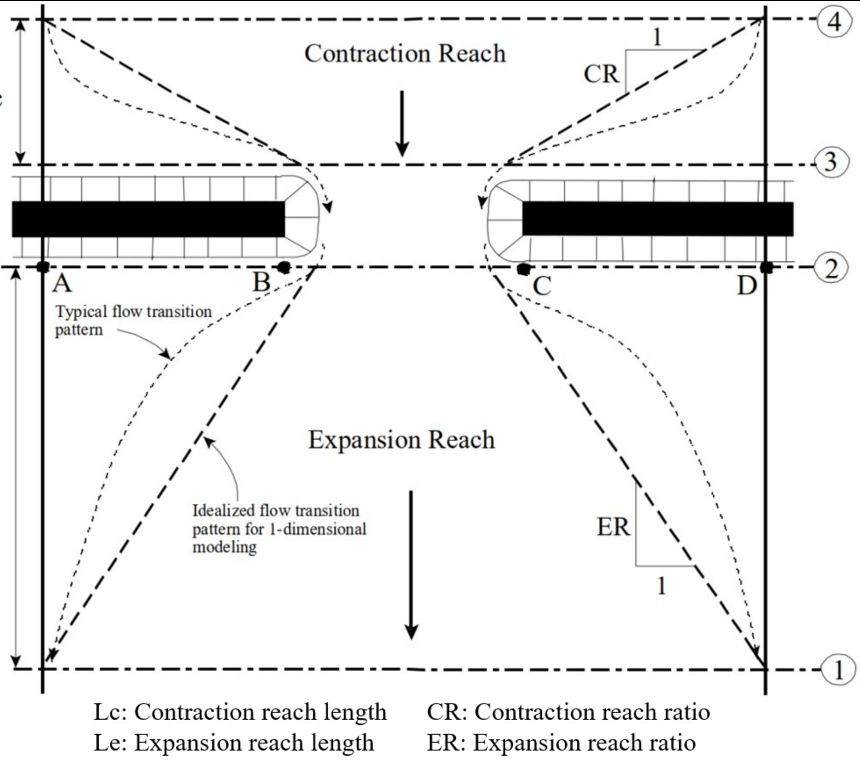


Figure 7-1. Cross-Section Locations at Crossing Structures (US Army Corps of Engineers, 2022)

A coefficient was applied to each culvert crossing to account for the energy loss by the entrance and exit of the culvert. Those entrance and exit loss coefficients depend on the culvert upstream and downstream face characteristics (i.e., projecting, mitered to the slope, wingwalls, etc.,) and were determined based on the field condition (field notes and photos), using values from the HEC-RAS reference manual.

It's important to note that for high flow bridge computation, the pressure and weir flow calculation were chosen when a bridge was under pressure for the 100-year storm. If the coded bridge was partially under pressure (only the upstream water surface elevation reaches the soffit elevation), the discharge coefficient applied was 0.5 while when the bridge was fully submerged (upstream and downstream faces of the bridge submerged), the discharge coefficient applied was 0.8.

7.4 Ineffective Flow Areas and Levees

The ineffective flow area option was applied in the model to restrict the flow area to the width of the structure opening until the structure is overtopped and weir flow begins over the road structure. Ineffective flow areas were coded in accordance with HEC-RAS Hydraulic Reference Manual (USACE, 2016) where stations were placed each side of the structure opening at a 1:1 ratio in upstream and 2:1 ratio in downstream of the distance between the bounding cross-sections and structure faces. The ineffective flow elevations upstream were set to the lowest point of the top-of-road (minimum weir elevation) and downstream to the average between the soffit and minimum top-of-road. Refinements were made after the initial run where surface water elevations were confined by the ineffective flow areas.

Levees have been added to the model at some roads and other topographic high points in order that the estimated water surface elevation remains concentrated in the main streambed until the flood flow reaches the levee elevation and spreads within the floodplain depressions.

7.5 Manning's Roughness Coefficients

Manning's roughness coefficients were assigned to each cross-section based on the SOLRIS land use layer (ver. 3.0), which was retrieved from the Ontario GeoHub database. Selby Creek and its tributaries flow through a mix of land uses within the study area, consisting mostly natural coverage with some urbanization in the communities of Selby and Napanee, which typically have wide range of Manning roughness values. These values were defined as shown in **Table 7-2**, based on the Technical Guidelines for Flood Hazard Mapping (EWRG, 2017) and the MTO Drainage Management Manual (Design Chart 2.01, 1997). A map illustrating the different land uses within the watershed is shown in **Figure 7-2**.

Table 7-2. Manning's Roughness Values used in the HEC-RAS Model

Classification	Manning's Roughness Value
Built-Up Area – Impervious	0.02
Built-Up Area – Pervious	0.05
Commercial Impervious Areas	0.02
Coniferous Forest	0.07
Deciduous Forest	0.075
Extraction – Aggregate	0.011
Forest	0.07
Hedge Rows	0.06
Impervious Disconnected	0.04
Marsh	0.075
Mixed Forest	0.07
Open Cliff and Talus	0.06
Open Water	0.035
Thicket Swamp	0.08
Tilled	0.055
Transportation - Rural	0.013
Transportation - Urban	0.013
Treed Alvar	0.055
Treed Swamp	0.08
Undifferentiated	0.05

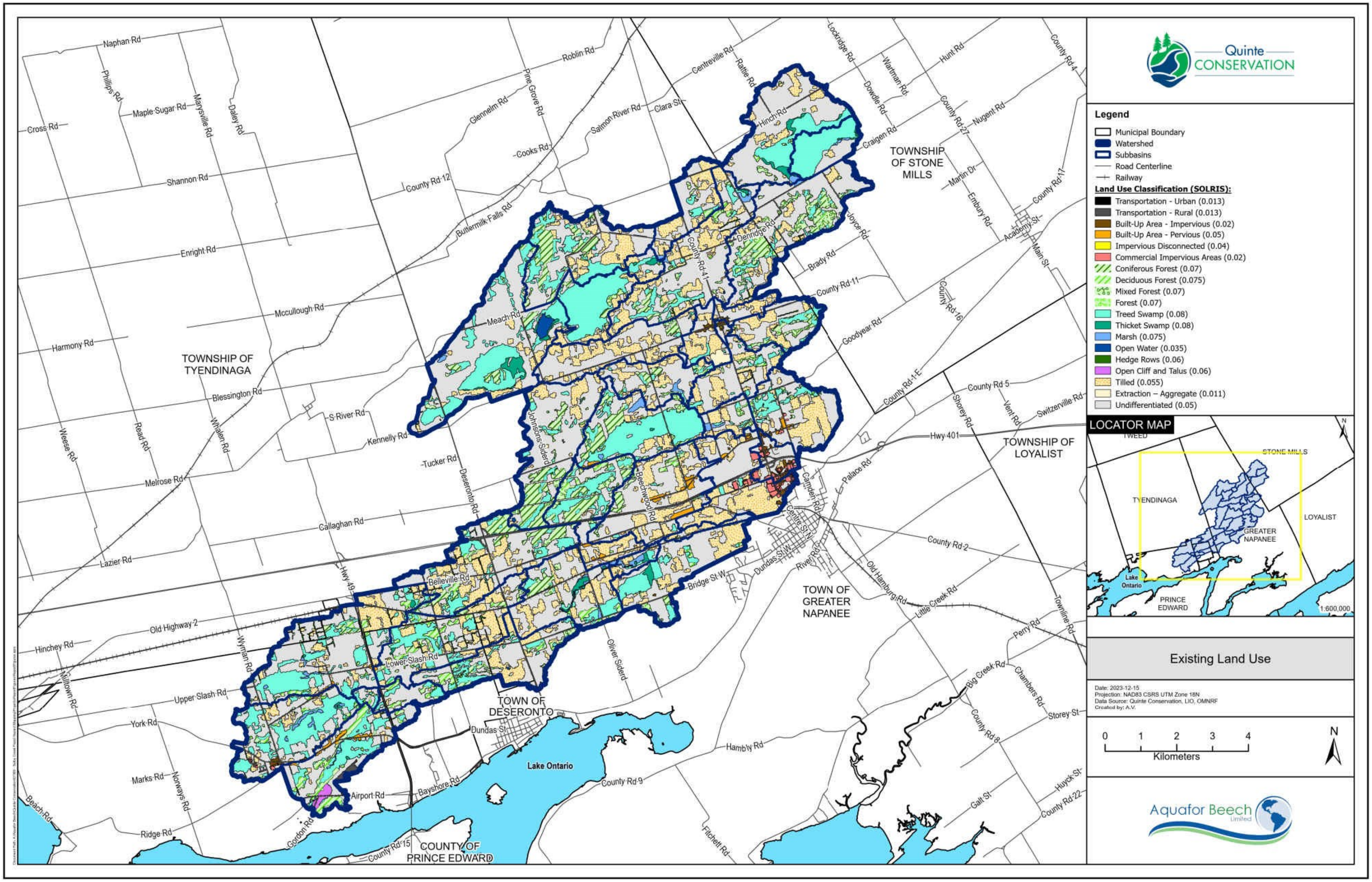


Figure 7-2. Land Use and Manning's N Coefficients within Selby Creek Watershed

7.6 Contraction and Expansion Coefficients

Contraction and expansion coefficients were coded in the model to evaluate transition loss due to changes of flow between cross-sections. These coefficients were applied differently between regular cross-sections and structure cross-sections following the recommended values, as summarized in **Table 7-3**.

Table 7-3. Contraction and Expansion Loss Coefficients used in the HEC-RAS Model

	Contraction Coefficient	Expansion Coefficient
Regular Cross-sections	0.1	0.3
Structure Cross-sections	0.3	0.5

7.7 Conveyance Obstructions (Buildings)

Buildings located within the study area were modeled as conveyance obstructions when defining areas of the cross-section that are permanently blocked out. Aquafor acquired building footprint data through the Open Street Map 3D building feature layer available on GeoHECRAS. Building height were assigned as per the maximum cross-section elevation option available on GeoHECRAS.

7.8 Modelling of Reaches Within Wetland Areas

In wetland areas where the topography is relatively flat and stream channels are poorly defined, tributaries can become laterally connected during flow events during flood flow events and no longer function as separate channels. Since the main purpose of the hydraulic model is for floodplain mapping, particularly of the Regulatory (Regulatory (100-year) storm event, these tributaries were not discretely modelled in the HEC-RAS modelling. Upstream reaches such as Selby_T3 and Selby_3 would fall entirely within the regional flood extent as a single floodplain. Due to the topography of these wetland areas, the smaller tributaries are typically contained within the backwater condition stemming from the downstream junction or are impacted by the flow from the adjacent main branch within the wetland. As a result, cross-sections were drawn wide enough along the main channel to incorporate the flow area of the smaller adjacent tributaries to best represent the hydraulic function under high flow conditions. However, the flows from the smaller tributaries were incorporated into the total flow passing through these wetland areas by using the flows computed in the hydrologic model at the confluence of the tributaries with the main branch.

7.9 Steady-State Flow Rate and Boundary Conditions

7.9.1 *Flow Rate*

The flows used in the hydraulic model were estimated from the HEC-HMS model developed by Aquafor in December 2023 as part of the Selby Creek Flood Hazard Mapping study. The flow rates used for the hydraulic modelling purposes reflect the computed HEC-HMS peak flows for the 10-day rain-on-snowmelt design storms (refer to the Aquafor hydrologic report, December 2023). The downstream reach flow nodes from the HEC-HMS model were assigned to the upstream reach in the hydraulic HEC-RAS model in order to be conservative and to avoid underestimating flows along the modelled reaches. The locations of all HEC-RAS flow nodes are provided in **Figure 7-3** and associated flow rates for design storm events ranging from the 2-year through 200-year storms based on the hydrologic model results are presented in **Table 7-4**.

Table 7-4. HEC-RAS Flows Entered in the Model for Design Storms from the 2-year to 200-year Events

HEC-HMS Junction	HEC-RAS Location		Peak Flow (m^3/s)						
	River Reach	Cross-section ID	2-year	5-year	10-year	25-year	50-year	100-year	200-year
Junct_1	Selby_1	3965	25.52	38.78	49.15	62.98	73.76	84.93	95.94
Junct_4	Selby_2	5755	18.9	30.16	39.2	51.84	61.15	70.67	79.64
Junct_18	Selby_3	29849	10.14	15.93	19.87	24.94	28.68	32.41	35.84
Junct_17	Selby_3	28959	10.2	16.2	20.28	25.57	29.54	33.56	37.54
Junct_16	Selby_3	26859	11.09	16.97	21.23	26.89	31.57	36.35	40.62
Junct_15	Selby_3	25189	12.13	18.66	23.98	31.03	36.36	41.71	46.52
Junct_14	Selby_3	24593	11.99	18.71	23.96	30.85	36.05	41.31	46.34
Junct_13	Selby_3	22914	13.07	20.36	25.55	32.73	38.38	44.1	49.47
Junct_12	Selby_3	21544	13.22	20.52	25.79	32.97	38.65	44.43	49.83
Junct_11	Selby_3	19991	14.43	22.71	28.85	37.06	43.32	49.84	55.93
Junct_10	Selby_3	16416	14.42	22.7	28.82	37.02	43.29	49.81	55.89
Junct_9	Selby_3	16168	14.41	23.15	29.4	37.84	44.32	51.03	57.3
Junct_8	Selby_3	15288	15.68	25.15	32.01	41.3	48.41	55.76	62.62
Junct_7	Selby_3	13841	16.18	25.98	33.06	42.61	49.96	57.54	64.62
Junct_4_2	Selby_3	9787	16.71	26.85	34.26	44.23	51.9	59.83	67.25
Junct_20_1	Selby_4	33060	6.21	9.52	11.76	14.6	16.73	18.87	20.86
Junct_2_1	Selby_T1	1335	6.63	10.01	12.28	15.12	17.22	19.31	21.22
Junct_6	Selby_T2	9890	3.34	5.26	6.59	8.29	9.56	10.84	12.01
Junct_5	Selby_T2	7031	6.88	10.56	13.15	16.48	18.98	21.51	23.73
Junct_4_1	Selby_T2	3500	9.72	14.8	18.42	23.1	26.6	30.16	33.27
Junct_20_2	Selby_T3	3328	4.34	6.78	8.5	10.78	12.51	14.28	15.92

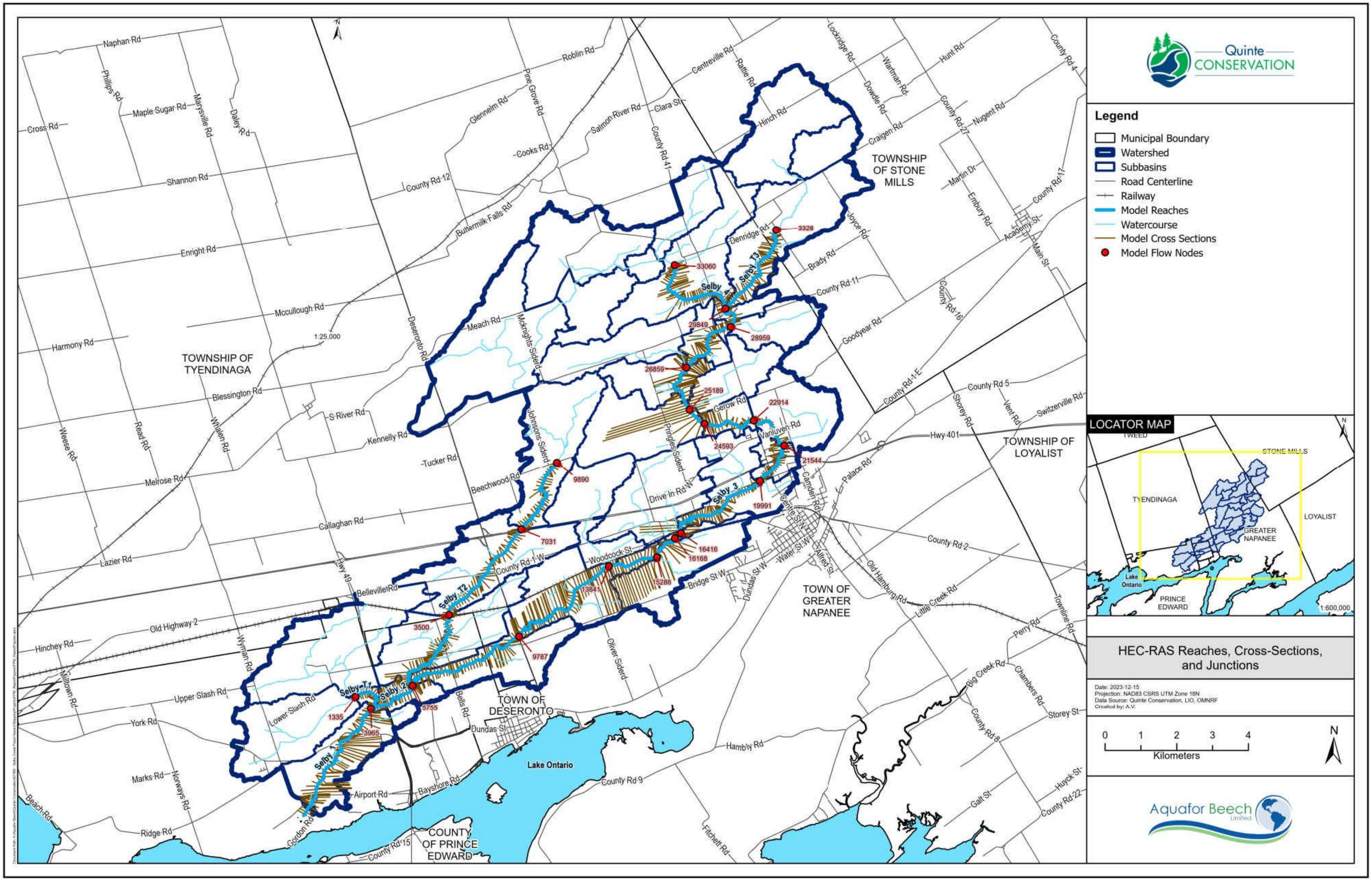


Figure 7-3. HEC-RAS Model Setup and Flow Node Locations

7.9.2 *Scenarios Analysis on Boundary Conditions*

The outlet conditions of the Selby Creek watershed are controlled by the water level at the Bay of Quinte (or Lake Ontario). Several approaches can be selected to define the level of the lake. As defined in the Technical Guidelines for Flood Hazard Mapping (EWRG, 2017), the long-term mean lake level should be considered as the starting water surface elevation. However, the MNRF in the Technical Guide for River and Stream Systems – Flooding Hazard Limit (2002) defines that the flood level of a large Lake should be the higher between the mean annual flood level in the river at the outlet or the mean monthly levels of the connecting lake. Also, for a regulatory flood hazard delineation purpose the common practice and the most conservative approach is to use the 100-year (1% AEP) of the Lake Level.

The mean annual flood level in the creek is unknown as the Selby Creek does not have gauges. However, the mean level of the Bay of Quinte or Lake Ontario is known. Aquafor performed several simulations on the downstream boundary conditions to determine which boundary condition would be best suited for use in the model. Simulations were performed for the six scenarios listed below:

-Normal Depth conditions [1]

-Long-term Mean water level of the Lake Ontario at Kingston [2]

-Annual Max Average water level of the Lake Ontario at Kingston [3]

-Historical Monthly Mean water level of the Lake Ontario as per Great Lakes Water Level data [4]

-1% AEP Lake Ontario water level for the 100-year flood flow [6]

-1% AEP Lake Ontario water level for the 2-year flood flow [6]

For the Long-term Mean [2] and the Annual Maximum Average [3], the recorded water levels used are from Environment Canada at the Kingston station (#02HM008). The long-term mean water level from Kingston is 74.43m (CGVD2013) while the annual max average from Kingston station is 74.86m (CGVD2013).

The Historical Monthly Mean water level as per the Great Lakes Water Lavel [4] are recorded data from Fisheries and Ocean. The year range of the recorded data is from 2012 to 2021. The mean water level of the Lake Ontario as per this data is 74.48m (CGVD2013).

The 1% AEP Lake Ontario water level for the 2-year flood flow [5] is a scenario reflecting the water level of the lake for the 1% AEP (100-year storm) and under the 2-year peak flow of the creek (50% AEP), while the 1% AEP Lake Ontario water level for the 100-year flood flow [6] is a scenario reflecting the water level of the lake for the 1% AEP (100-year storm) and also under the 100-year peak flow of the creek (1% AEP).The 1% AEP Lake Level used in the scenario [5] and [6] is from the Lake Ontario Shoreline Hazard Summary Risk Assessment and Management Plan completed by CLOCA in 2022. In this study, the 100-year flood level considers both static lake level and storm surge. The 1% AEP lake elevation is 75.55m (CGVD2013).

Table 7-5 summarizes the water surface elevations referenced in CGVD2013 vertical datums. It is important to note that the lake elevations referenced to the CGVD2013 datum were used in the hydraulic model to be consistent with the other elevation sources.

Table 7-5. Statistical Water Surface Elevation of the Lake Ontario

Data source	WSE (m) CGVD2013
Long-term Mean at Kingston Station - ECCC since 1963 [2]	74.43
Annual Max Average at Kingston Station - ECCC since 1963 [3]	74.86
Historical Monthly Mean Water Level as per the Great Lakes - Fisheries and Ocean from 2012 to 20210 [4]	74.48
The 1% AEP Lake Ontario Water Level [5] and [6]	75.55

The computed water levels for each tested scenario were compared to evaluate the extent and magnitude of the impact of the downstream boundary condition assumptions. More specifically, the furthest affected cross-sections and the differences in computed water surface elevations (WSE) of all affected cross-sections were identified and summarized in **Table 7-6**. All scenarios were run without the lateral structure (Existing-v2 HEC-RAS scenario) to account for the maximum flow from the downstream main branch.

Comparison of results for the six scenarios suggests that the boundary condition impacts the model results up to maximum upstream distance of 450m (XS # 451) along the Main branch (**Figure 7-4**). The long-term daily mean and the historical monthly mean scenarios provide the least conservative WSE results.

The Normal Depth [1] and the 1% AEP with the 100-year stream flow [5] provide the most conservative WSE results. As per results, the water surface elevation is again governed by normal depth condition from XS #451 (450 m upstream the outlet). However, the 1D HEC-RAS computed water surface elevations near the Watershed outlet were not used to define the regulatory flood lines. Indeed, as a spill area has been identified upstream Airport Road and a 2D HEC-RAS model has been developed to estimate the flood impact. Therefore, the flood lines at peak flow from the 2D modeling are used to define the flood extents from the outlet (XS # 1) to upstream XS # 1259.

Table 7-6. Scenarios Analysis on Boundary Conditions for Selby Creek Downstream Main Channel

Reach	Cross-section ID	Distance from downstream Cross-section (m)	100-year flow (m3/s)	[1] Normal Depth	[2] Long-term Mean WSEL	[3] Annual Maxi-mum Average WSEL	[4] Historical Monthly Mean (Great Lakes)	[5] The 1% AEP Lake Level with 100-year stream flow	2-year flow (m3/s)	[6] The 1% AEP with 2-year stream flow
Selby_1	636	635.26	84.93	76.48	76.48	76.48	76.48	76.48	25.49	76.15
Selby_1	593	591.91	84.93	76.47	76.47	76.47	76.47	76.46	25.49	76.11
Selby_1	518	517.25	84.93	76.32	76.32	76.32	76.32	76.32	25.49	75.92
Selby_1	451	449.59	84.93	76.12	76.12	76.12	76.12	76.12	25.49	75.71
Selby_1	413	411.8	84.93	75.91	75.9	75.9	75.9	75.94	25.49	75.61
Selby_1	350	349.45	84.93	75.8	75.77	75.76	75.77	75.86	25.49	75.59
Selby_1	284	283.48	84.93	75.63	75.54	75.53	75.54	75.72	25.49	75.57
Selby_1	230	228.88	84.93	75.55	75.42	75.41	75.42	75.67	25.49	75.56

Reach	Cross-section ID	Distance from downstream Cross-section (m)	100-year flow (m3/s)	[1]	[2]	[3]	[4]	[5]	2-year flow (m3/s)	[6]
				Normal Depth	Long-term Mean WSEL	Annual Maxi-mum Average WSEL	Historical Monthly Mean (Great Lakes)	The 1% AEP Lake Level with 100-year stream flow		The 1% AEP with 2-year stream flow
Selby_1	151	149.78	84.93	75.47	75.29	75.28	75.29	75.57	25.49	75.56
Selby_1	77	76.4	84.93	75.41	75.13	75.11	75.13	75.56	25.49	75.55
Selby_1	1	0	84.93	**75.36**	**74.57**	**74.86**	**74.57**	**75.55**	25.49	**75.55**

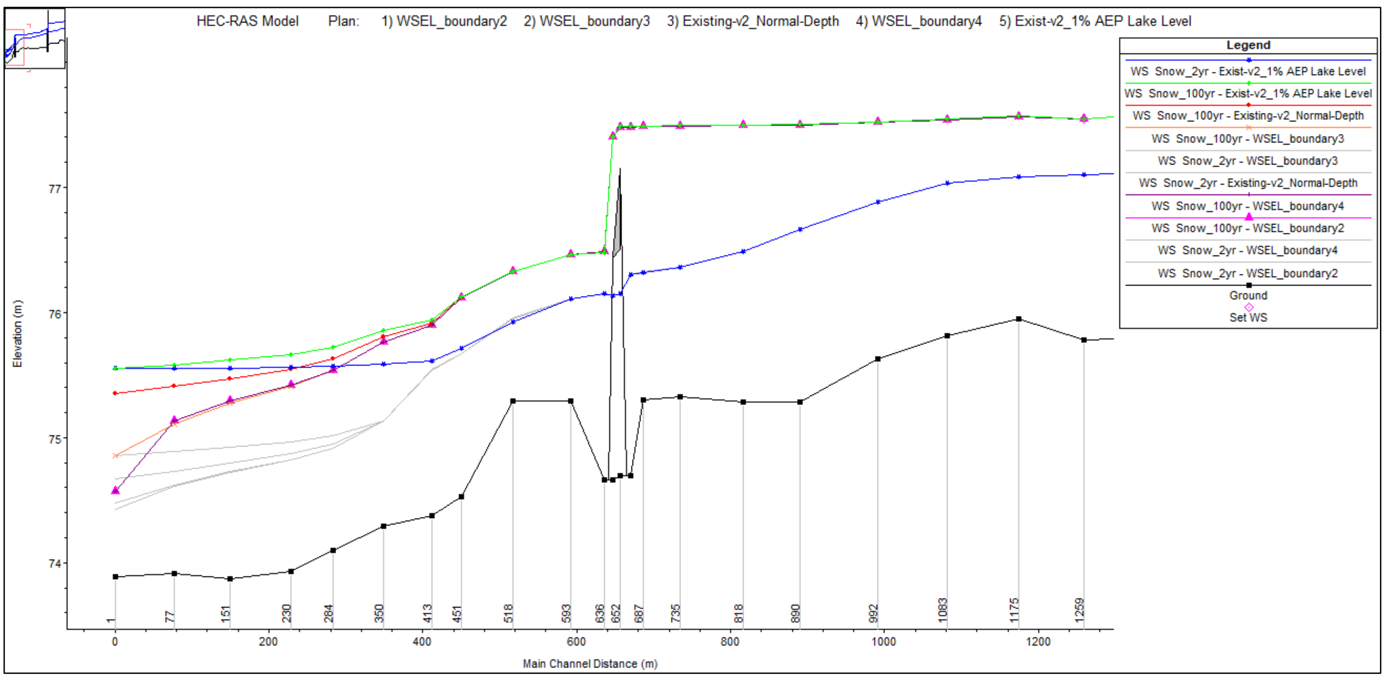


Figure 7-4. Profile Plot Result for the Six Simulated Boundary Conditions Scenarios

7.10 Preliminary 1D Model Results

7.11 Identification of Spill Areas

The preliminary 1D HEC-RAS modelling results identified one potential spill area within the study extents. Floodplain spill areas are defined as areas where flood waters are not physically contained within the valley or stream corridor and flow into surrounding lands. Floodplain spill areas can occur naturally, or can occur as a result of downstream barriers to the passage of flood flows, such as undersized bridges or culverts. One potential spill area was identified within the study limit, upstream Airport Road, near the Watershed Outlet.

At river cross-section #890, floodwaters exceed the floodplain storage and spill out of the Selby Creek watershed into a marshy area to the south-southeast of the spill location, before discharging into the Bay of Quinte, as shown in **Figure 7-5**. Modelling results suggest that floodwaters may begin to spill for flood events as frequently as the 5-year flow event.

To better understand the spill at Airport Road and to provide more accurate flood hazard identification, following discussions with Quinte Conservation staff, Aquafor was directed to prepare a 2D model of the spill area. This modelling is described in subsequent report sections.

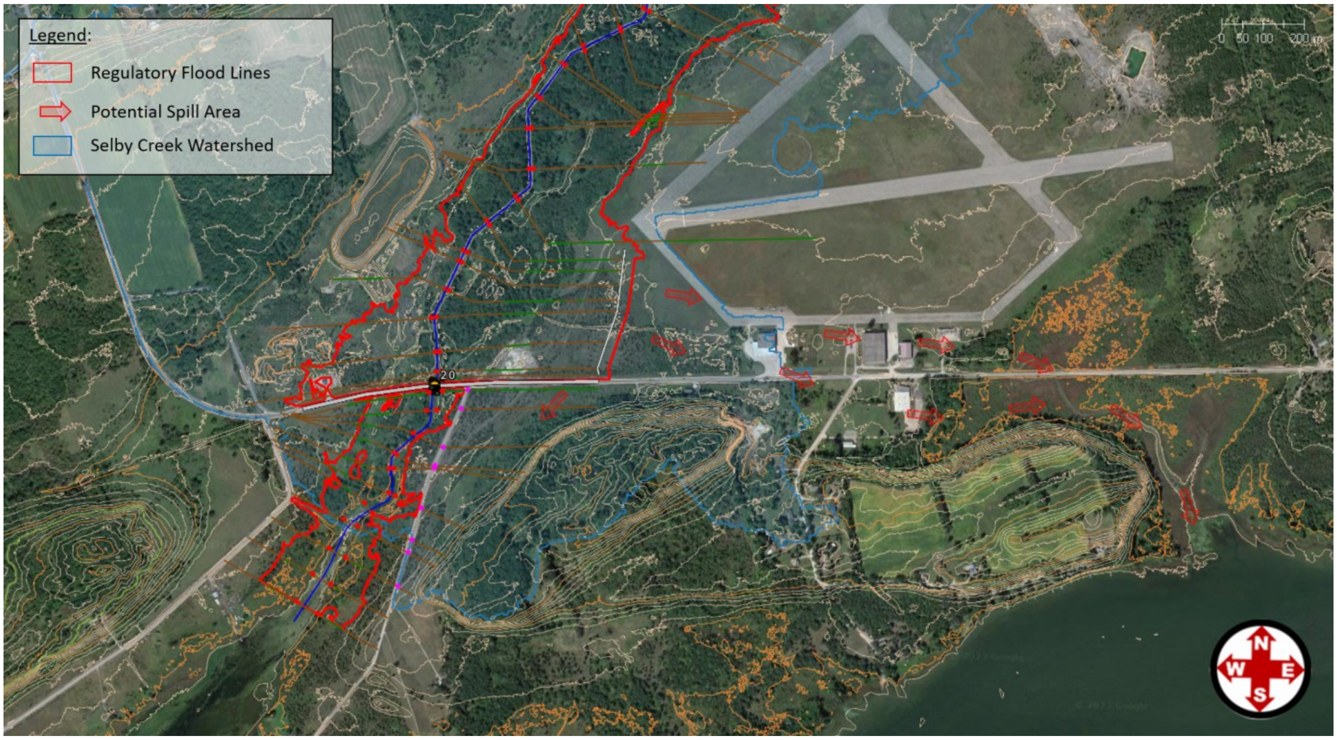


Figure 7-5. Potential Downstream Spill Area in 1D Modelling

8 2D HYDRAULIC MODEL DEVELOPMENT

Detailed 2D hydraulic modelling of the area of Airport Road was built using the HEC-RAS software (ver. 6.4.1). Preliminary floodplain mapping for the 5-yr and greater flow events was also generated using 2D HEC-RAS results and elevation surface data.

8.1 Terrain and Modifications

The LiDAR DTM provided by Quinte Conservation was used to generate the terrain file used in the 2D HEC-RAS modelling. Terrain modifications have been added within the RAS Mapper to accurately represent key hydraulic features within the hydraulic modelling.

8.1.1 *Channel Improvements*

Invert elevations of culvert crossings on Airport Road, Hickory Road, Gordon Road, and Johnsons Lane have been identified as residing below the LiDAR DTM data. Accordingly, to accurately replicate culvert hydraulics within the 2D modelling environment, terrain modifications have been performed along the profile of the existing roadway culverts in the form of a Ground Line Modification. The Lower (Terrain/User) Value option has been selected to provide a nominally lower elevation profile (≤ 1cm) through each culvert crossing.

8.2 2D Flow Area and Perimeter

The 2D modelling perimeter has been configured with the assumption that lower AEP event peak flows may spill east beyond the Selby Creek banks at Airport Road and overtop Airport Road at several locations. Flows are then split downstream of airport road, either returning to Selby Creek or flowing to the watercourse east of Johnsons Lane and eventually to Lake Ontario. Accordingly, the 2D perimeter extends from the intersection of Airport Road and Hickory Road to the west to the intersection of Airport Road and Seros Road to the east. It is bounded by 1D HEC-RAS cross section 1259 to the north, and Lake Ontario to the south. The horizontal extents of the perimeter have been placed such that the split of flows overtopping Airport Road to either the west (Selby Creek) or east (crossing Johnsons Lane) could be adequately assessed. **Figure 8-1** illustrates the 2D modelling perimeter of the study area within the context of the existing 1D model.

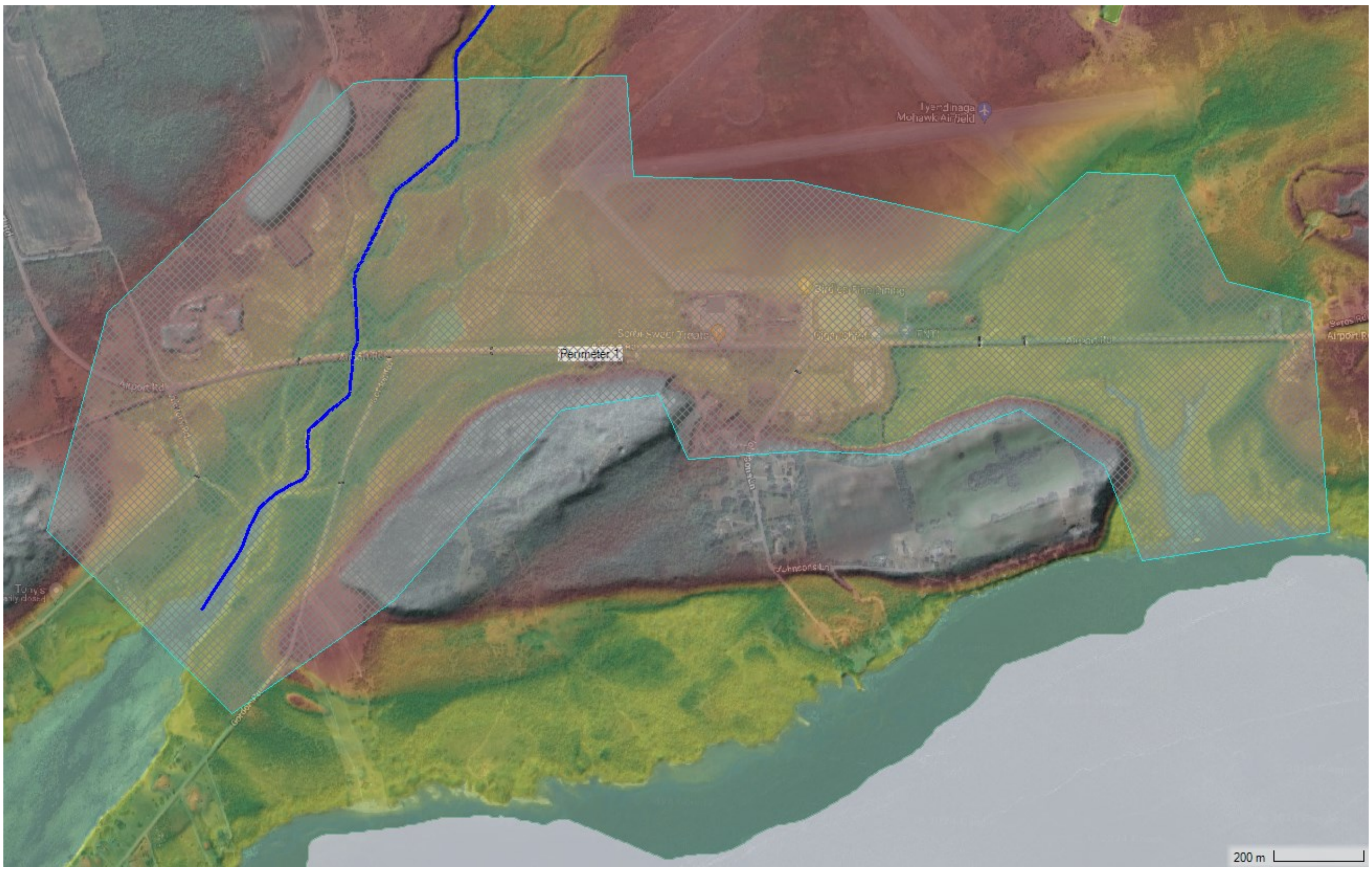


Figure 8-1. 2D Model Perimeter

As noted in the HEC-RAS 2D User's Manual; *"Assigning an appropriate mesh cell size (or sizes) and computational time step (ΔT) is very important to getting accurate answers with 2D flow areas."*. Accordingly, a generic 20 (X) by 20 (Y) rectangular cell mesh distribution has been used as a mesh default, given the relatively flat sloping grades of the study area and anticipated regulatory floodplain extents. Additional mesh detail has been provided via breaklines within the model at reduced spacing/intervals to refine the mesh within the channel banks and roadway right-of-ways. The 2D flow area mesh specifics have been provided in **Table 8-1**.

Table 8-1. 2D Flow Area Mesh Specifics

Number of Cells	Average Face Length	Average Cell Size	Maximum Cell Size	Minimum Cell Size
44,448	7	46	769	1

Details surrounding the selection of an appropriate computational time step have been provided in **Section 8.8**.

8.3 2D Breaklines

Breaklines have been added to the HEC-RAS hydraulic model to refine the 2D mesh and provide a higher level of computational detail in key locations including road and rail centrelines, watercourse corridors and culvert crossings.

For watercourses, breaklines have been strategically placed to identify the centerline of the watercourse between roadway crossings. Breakline attributes for watercourses have been selected to replicate bank full flow widths and channel depths. Breaklines used to refine the mesh within roadway corridors have been coded to identify the centerline of roadway, driving lane width, roadway embankments and parallel ditches.

All breaklines have been enforced within the 2D mesh. A visual representation of the road breaklines and river centerlines has been provided in **Figure 8-2**.

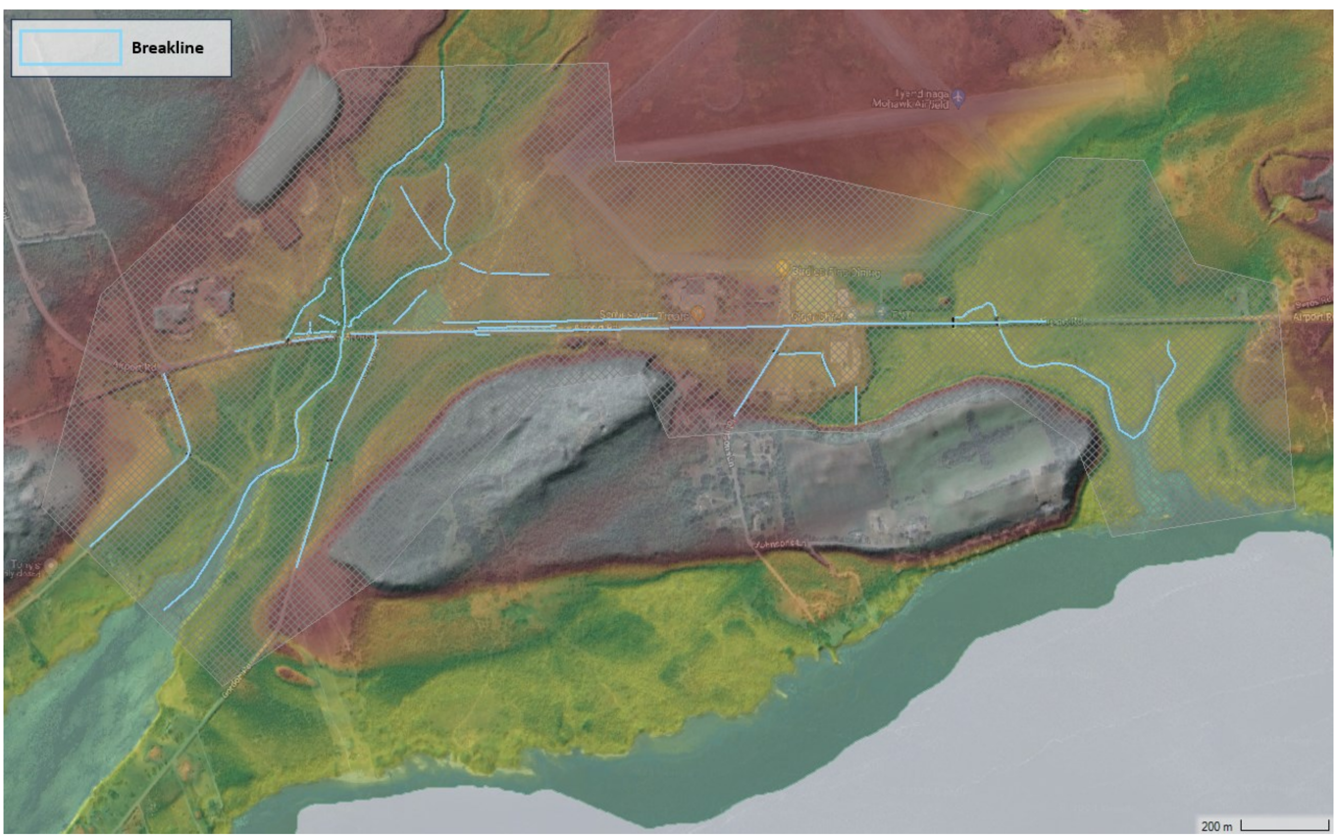


Figure 8-2. 2D Model Breaklines

8.4 2D Computation Points

In addition to the default Computational Points derived by the RAS Mapper, Computational points were added to the 2D Mesh in the direct vicinity of roadway culvert inlets and outlets. These additional computational points provide an enhanced level of mesh detail at key locations in the hydraulic model to aid in computational efficiency and accuracy of roadway culvert crossings. Additional computational points have also been added, where required, to the mesh to retain a maximum of eight (8) cell sides within in overall 2D mesh in accordance with the 2D HEC-RAS Users Manual.

8.5 2D SA/2D Connections

The Airport Road bridge crossing Selby Creek, and nine (9) additional culvert crossings have been added to the 2D HEC-RAS model via SA/2D Connections. Detailed crossing information was then coded, including the structure material, opening dimensions, invert elevations, skew angles, depth of embedment, etc. Road profiles were defined using LiDAR DEM and topographic survey information. Detailed hydraulic inventory sheets of each structure within the project area have been provided in **Appendix F.**

8.6 2D Boundary Conditions

A total of three (3) boundary conditions have been applied to the 2D model including one inflow hydrograph and two downstream boundary conditions. Details surrounding each of the applied boundary are provided below:

- Inflow Hydrograph → The HEC-HMS Junction 1 hydrograph has been conservatively applied at the upstream limits of the 2D study area, at approximately HEC-RAS 1D River Station 1259 as an Internal Flow Hydrograph. To improve computational time, the peak of the hydrograph was extracted from the full event. Hydrographs for the 20%, 10%, 4%, 2%, 1% and 0.5% AEP events were input as scenarios for this boundary condition.

A visual representation of the six event hydrographs has been provided in **Figure 8-3.**

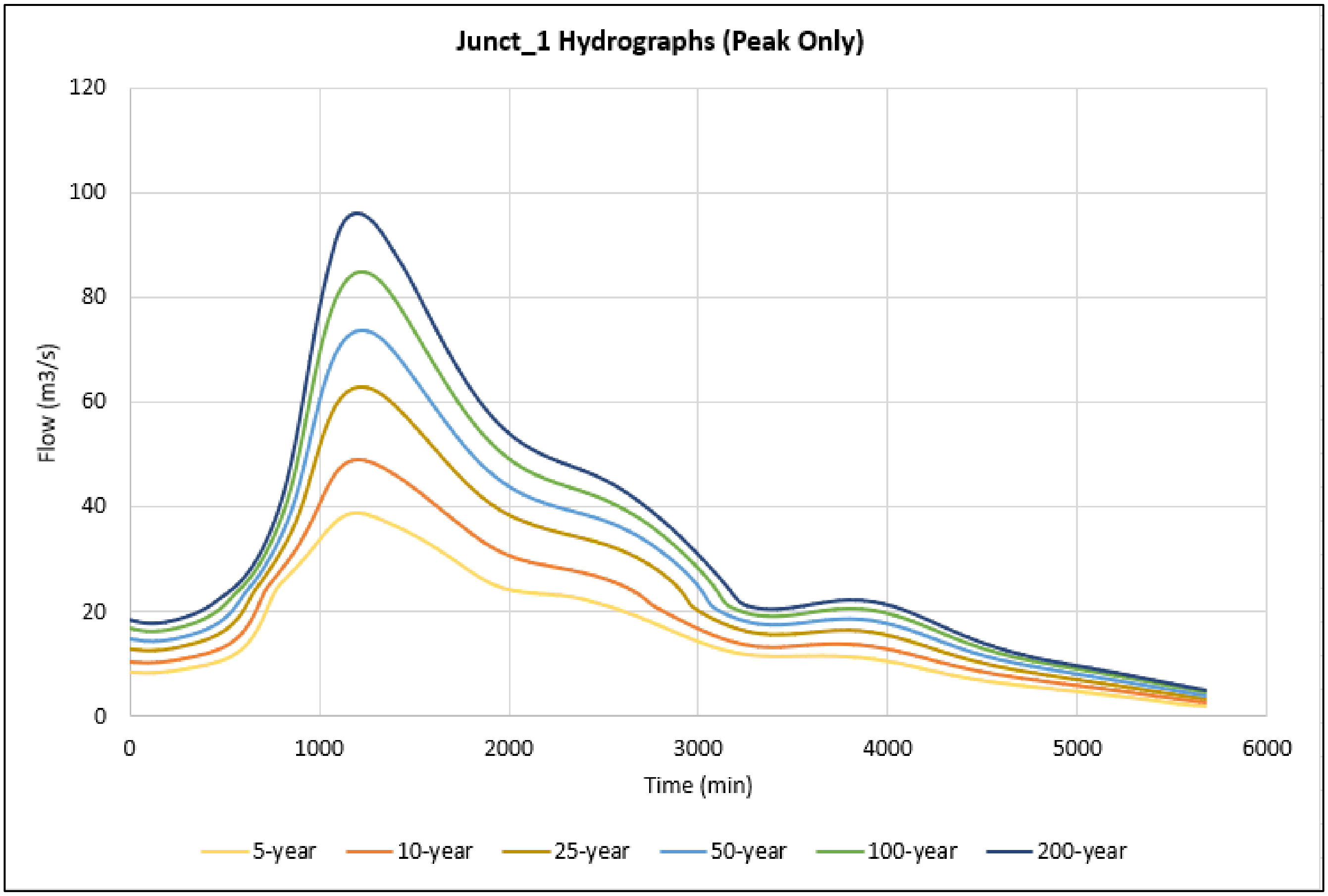


Figure 8-3. Inflow Hydrographs Applied at Upstream Boundary Condition

- Downstream Boundary Condition 1 → Selby Creek Outlet → Normal Depth. The Normal Depth Boundary condition has been selected for the southwestern outlet into Lake Ontario. A normal depth slope of 0.0002 m/m has been set and measured directly from the Selby Creek profile at the downstream limits of the project area. The Normal Depth condition was selected based on the scenario and boundary condition analysis completed for the 1D HEC-RAS model build. We note that the 2D Flow Area Boundary Condition Parameter has been set to "Compute Single Water Surface for Entire BC Line".

- Downstream Boundary Condition 2 → Overflow Outlet → Normal Depth. The Normal Depth Boundary condition has been selected for the southeastern outlet into Lake Ontario. A normal depth slope of 0.0002 m/m has been set and measured directly from the terrain profile at the downstream limits of the project area. The Normal Depth condition was selected based on the scenario and boundary condition analysis completed for the 1D HEC-RAS model build. We note that the 2D Flow Area Boundary Condition Parameter has been set to "Compute Single Water Surface for Entire BC Line".

8.7 2D Computational Settings

To provide an accurate representation of all flow events within the 2D modelling environment, we have selected the following computational settings;

- Computational Interval: 1 min

- Detailed Output Interval: 10min
- 2D Un-Steady State Flow Routing – SWE-ELM
- Advanced Time Step Control – Adjusted Time Step based on Courant.

8.8 Evaluation of 2D Modelling Stability

The Computational Message Window and its associated Computational Log File were reviewed with respect to the 2D model's overall volume accounting. Based on our experience with 2D modelling, we would identify a stable model with having an overall volume accounting error of less than 2% with an optimal result producing a volume accounting error of less than 1%. The above noted thresholds would apply for both the overall simulation as well as each 2D flow area.

The overall regulatory (1% AEP) simulation illustrates that the model gained 67 m^3 of water which equates to a 0.0006 % volume error. Therefore, the overall volume accounting error is quite low and within desirable modelling tolerances.

Model instabilities were observed within the 2D model results, particularly at culvert crossings and locations where flows overtopped a roadway. These instabilities are considered minor in consideration the overall objective of the model. Use of the 2D model beyond spill analysis and floodplain mapping for Selby Creek at Airport Road requires additional detailed topographic survey and input.

9 HYDRAULIC MODEL VERIFICATION

Flood lines were generated based on Aquafor's preliminary hydraulic model results for the 100-year storm event and were plotted alongside the existing Regulatory flood lines (Crysler & Lathem Ltd, 1981) for comparison purposes, as a means of model verification. While we recognize that this comparison does not prove the validity of either model, the comparison is used to understand the differences between the models and to identify areas requiring detailed review when refining the model and flood lines. It was noted that the extents of flooding under the Regulatory event for these two models were not directly comparable due to the following:

- differences in topography (1980s mapping vs. 2022 LiDAR)
- different hydrologic models (US Soil Conservation Service TR-20 model vs. HEC-HMS software, different assumptions and subbasin discretization, etc.)
- different watershed boundaries
- different design storms (larger of the snowmelt 100-year frequency analysis peaks flows and the 100-year SCS Type II vs. 10-day rain-on -snowmelt)
- different hydraulic modelling software (HEC-2 vs. HEC-RAS)
- different hydraulic structure crossing considerations
- differences in the reaches and reach extents mapped between the new and the existing flood lines.

There was an acceptable agreement between the 100-year flood extents from Crysler & Lathem Ltd, 1981 and the preliminary Aquafor HEC-RAS results. Although the HEC-RAS model results were subsequently refined, this comparison exercise was not repeated. Mapping included in this report is based on the final HEC-RAS modelling as refined throughout the project.

10 HYDRAULIC MODEL RESULTS AND FLOODPLAIN MAPPING

The 1D HEC-RAS hydraulic model for Selby Creek was executed using the subcritical flow regime to establish water surface profiles for each of the 2-year through 100-year return period storm and climate change scenario (200-year) flood profiles (also named 0.5, 0.2, 0.1, 0.04, 0.02, 0.01, and 0.005 AEP), with a focus on the 100-year storm, which is considered to be the Regulatory event for this watershed. Model output results for all reaches are presented in **Appendix G**. The 2D HEC-RAS hydraulic model for the identified spill area was executed for the 5-year through 200-year events, as spill in the 1D model was observed under these events.

10.1 Floodplain Mapping

Regulatory flood lines upstream of HEC-RAS cross section 1259 were generated from the water surface elevation outputs from the 1D HEC-RAS model under the 100-year rain-on-snowmelt event and current land use, as described in the Aquafor's hydrologic modelling report (December 2023). These flood lines were mapped based on the intersection between the predicted water surface elevation and the LiDAR-derived DTM, as well as engineering judgement. Regulatory flood lines downstream of cross section 1259 were generated using the 2D HEC-RAS model using the 100-year event inflow hydrograph. Cross section 1259 was selected as the match point as the model results generally agreed at this location, as flows in all events are contained to the channel and have reached normal flow conditions. Detailed regulatory floodplain mapping (100-year storm event) is included in **Appendix H**, while **Appendix I** provides a set of map sheets with the flood lines for the 50-year, 100-year and 200-year storm events. **Figure 10-1** provides an overview of the regulatory floodplain mapping showing the location of the 32 map sheets. The following are notable components of the flood hazard mapping exercise:

- The 1D and 2D hydraulic models were built using a combination of topographic data (LiDAR and field survey), both referenced to the CGVD2013 vertical datum.
- The 1D hydraulic model was run under subcritical flow for all flow events. The 2-year, 5-year, 10-year and 25-year events were simulated for information purpose only, therefore the results only provide general guidance on the hydraulic parameters for the storms with a lower magnitude than the regulatory storm.
- A spill area analysis was performed on the preliminary 1D hydraulic results, and one (1) spill was identified and would start with the 5-year storm event. This spill area was subsequently modelled using a 2D HEC-RAS model for the 5-year to the 100-year including the 200-year event (Climate Change).
- The 2D hydraulic model was run under unsteady state flow conditions for the 50, 100 and 200-year events to inform regulatory flood mapping. The 5, 10 and 25-year events were run for information purposes only.
- The results of the 1D and 2D floodplain mapping were combined to provide final flood lines.
- The Normal Depth downstream boundary condition was applied for all storm events.
- Smoothing of flood lines and full manual inspection of the 2-year, 10-year, 50-year, 100-year and 200-year flood lines was performed.
- Where the automatically-generated flooding extent were cut off, the flooding boundary was filled in by extending it to an appropriate contour elevation.
- Islands were manually filled and isolated areas of flooding were removed, as deemed appropriate.

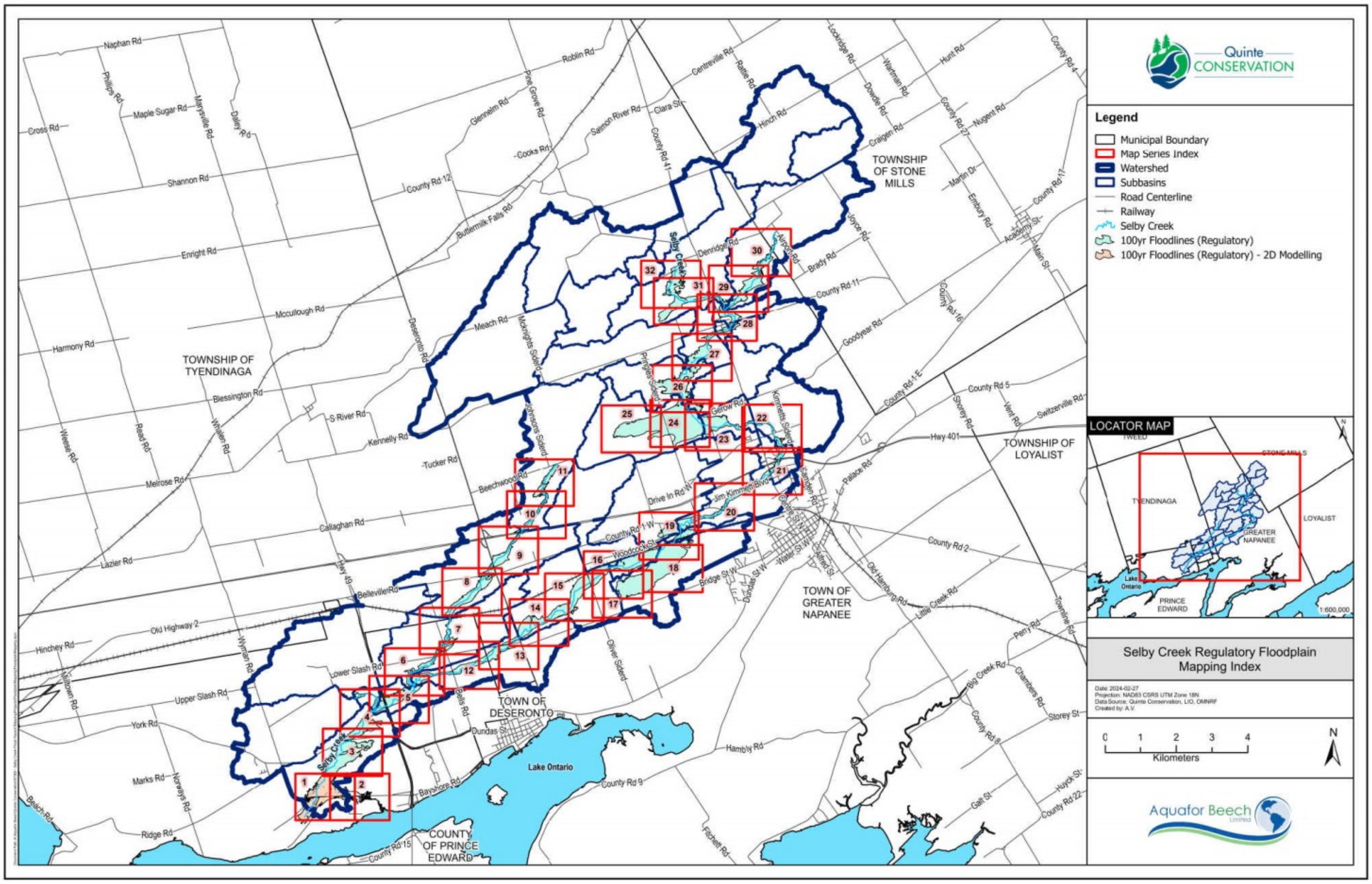


Figure 10-1. Overview of the Regulatory Selby Creek Floodplain Mapping

11 FLOOD HAZARD ASSESSMENT

11.1 Watercourse Crossing Overtopping Analysis

Pedestrian and vehicle access can be limited when a road crossing is overtopped and inundated. As per the Technical Guide – River and Stream System: Flooding Hazard Limit (OMNR, 2002), a road is impassible if the overtopping water depth is equal or greater than 0.3m. A road can be overtopped but only becomes impassable if this threshold is reached or exceeded. The product of the depth and velocity of the water on top of the roadway can also be a criteria that define an impassable road. 0.8m^2/s is safety condition threshold used in this analysis as per appendix 6 of the Technical Guide – River and Stream System: Flooding Hazard Limit (OMNR, 2002). In order to define the road elevation at the crossing location, the final water surface profiles were reviewed and the lowest point elevation of the road (as defined in the existing HEC-RAS models) was selected to calculate the depth of water overtopping the road. The velocity of the cross-section upstream the road crossing has been used to assess the combination of the depth and velocity threshold. The following tables (**Table 11-1 to Table 11-3**) provide detailed results of the analysis and the **Appendix J** supplies the location and the status of the crossings.

This analysis was completed for the road crossings over the Selby Creek system modelled in the 1D HEC-RAS model. Under the Regulatory flood (100-year storm event), flows are expected to exceed the capacity of an estimated 13 crossing structures, causing the road profile to be overtopped while 11 and 14 crossings would be impacted for the 50-year and the 200-year flood events respectively, as summarized in **Table 11-1.** Of these overtopped structures, only one (ABL_22) is estimated to be impassable under the Regulatory event. In order to provide additional insight into the road overtopping conditions, each structure included in the hydraulic model was classified for each storm event using a colour code. **Table 11-2** provides details on the water depth on top the road and it is based on 3 categories:

- Road not overtopped – highlighted in green
- Road overtopped but passable (water depth <0.3m) – highlighted in orange
- Road impassable (water depth ≥ 0.3m) – highlighted in red

Table 11-1. Total of Crossing Structures Overtopped for the 50-yera, 100-year and 200-year flood events

	50-year	100-year	200year
Total of Crossing Structures Overtopped	11	13	14
Total of Crossing Structures	27		
Percentage	40.7%	48.1%	51.9%

Table 11-2. Depth overtopping conditions analysis for all crossing structures listing by storm event - the roads not overtopped (green), the roads overtopped but passable (orange) and the roads impassable (red)

Reach	Structure ID	Upstream XS #	Road top Elev. (m)	Difference between WSE and Top of Road Elevation (m)						
				2-year	5-year	10-year	25-year	50-year	100-year	200-year
Selby_4	24	32983	125.80	-0.34	-0.10	0.10	0.13	0.15	0.17	0.18
Selby_4	18	30579	125.83	-4.68	-4.41	-4.25	-4.06	-3.93	-3.63	-3.55
Selby_3	1	29232	116.81	-0.37	-0.19	-0.06	0.13	0.17	0.20	0.23
Selby_3	3	28716	116.65	-1.15	-0.94	-0.81	-0.65	-0.55	-0.44	-0.35
Selby_3	12	28518	117.86	-2.49	-2.31	-2.21	-2.09	-2.00	-1.92	-1.86
Selby_3	22	24399	111.62	0.51	0.67	0.76	0.86	0.93	0.99	1.05
Selby_3	11	22914	111.60	-1.99	-1.69	-1.51	-1.28	-0.89	-0.75	-0.55
Selby_3	8	22091	105.69	-2.22	-2.02	-1.89	-1.67	-1.50	-1.34	-1.16
Selby_3	19	21047	96.18	-2.87	-2.64	-2.50	-2.33	-2.22	-2.10	-1.97
Selby_3	7	20521	94.42	-1.54	-1.38	-1.28	-1.12	-0.97	-0.80	-0.64
Selby_3	17	19991	93.40	-0.84	-0.63	-0.48	-0.27	-0.11	0.08	0.32
Selby_3	15	18471	92.98	-1.64	-1.42	-1.30	-1.16	-1.07	-0.98	-0.90
Selby_3	25	16168	90.80	-2.44	-2.21	-2.05	-1.83	-1.52	-1.43	-1.31
Selby_3	2	13149	87.79	-0.47	-0.28	-0.16	-0.01	0.12	0.23	0.31
Selby_3	16	9787	87.21	-1.53	-1.26	-1.09	-0.87	-0.71	-0.55	-0.33
Selby_3	10	9457	85.69	-0.52	-0.35	-0.23	-0.09	0.03	0.15	0.57
Selby_3	9	7365	84.32	-1.63	-1.32	-1.12	-0.89	-0.73	-0.30	-0.19
Selby_T2	14	7031	105.89	-1.73	-0.81	-0.04	0.13	0.17	0.22	0.25
Selby_T2	13	5721	101.00	-0.73	-0.30	-0.06	0.14	0.22	0.26	0.30
Selby_T2	30	5259	99.48	-0.65	-0.43	-0.23	0.08	0.14	0.18	0.21
Selby_T2	29	4873	98.45	0.01	0.11	0.15	0.21	0.23	0.25	0.28
Selby_T2	26	3500	98.30	-1.99	-1.25	-0.46	0.13	0.18	0.21	0.23
Selby_T2	23	2364	84.76	-0.77	-0.40	-0.30	-0.04	0.14	0.21	0.27

Reach	Structure ID	Upstream XS #	Road top Elev. (m)	Difference between WSE and Top of Road Elevation (m)						
				2-year	5-year	10-year	25-year	50-year	100-year	200-year
Selby_T2	21	1793	83.06	-1.22	-0.98	-0.80	-0.55	-0.35	-0.35	0.06
Selby_2	4	4530	81.60	-2.75	-2.44	-2.22	-1.92	-1.70	-1.48	-1.27
Selby_1	6	2825	80.84	-3.07	-2.86	-2.63	-2.36	-2.12	-1.85	-1.59
Selby_1 *	20	671	77.16	-0.86	-0.37	-0.23	-0.09	0.01	0.07	0.11

*Analysis performed based on the 1D HEC-RAS Model

Table 11-3 provides details on the product of the water depth and velocity that would occur on top the road and it is based on 3 categories:

- Road not overtopped – highlighted in green
- Road overtopped but passable (water depth x velocity <0.8m^2/s) – highlighted in orange
- Road impassable (water depth x velocity ≥ 0.8m^2/s) – highlighted in red

It is important to note that as per the combination of the depth and velocity threshold (0.8m^2/s), none of a roadway crossing would become impassable, even under the largest flood flow such as the 100-year and the 200-year return period storms. As per results, the most restrictive threshold is the water depth criteria and the associated roadway crossing status is highlighted in **Appendix I**.

Table 11-3. Combination of Depth and Velocity overtopping conditions analysis for all crossing structures listing by storm event - the roads not overtopped (green), the roads overtopped but passable (orange) and the roads impassable (red)

Reach	Structure ID	Upstream XS #	Depth x Velocity (m^2/s)						
			2-year	5-year	10-year	25-year	50-year	100-year	200-year
Selby_4	24	32983			0.01	0.01	0.02	0.02	0.02
Selby_3	1	29232				0.03	0.05	0.06	0.07
Selby_3	22	24399	0.06	0.07	0.09	0.11	0.12	0.14	0.16
Selby_3	17	19991						0.09	0.11
Selby_3	2	13149					0.02	0.04	0.05
Selby_3	10	9457					0.04	0.23	0.74
Selby_T2	14	7031				0.01	0.01	0.01	0.01
Selby_T2	13	5721				0.03	0.05	0.04	0.06
Selby_T2	30	5259				0.02	0.01	0.01	0.01
Selby_T2	29	4873	0.00	0.02	0.02	0.04	0.04	0.05	0.06
Selby_T2	26	3500				0.01	0.02	0.03	0.03
Selby_T2	23	2364					0.03	0.05	0.07
Selby_T2	21	1793							0.03
Selby_1	20	671					0.00	0.01	0.02

The culverts assessed for the supplementary 2D modelling were investigated to assist in analysing spill from Selby Creek only. The detailed overtopping analysis of the culverts included in the 2D HEC-RAS model is beyond the scope of this study, and the are not included in this analysis.

11.2 Flooded Building Assessment

The impact of the flood lines on private and public facilities was analyzed using the building feature layer shapefile (same as the layer used for defining conveyance obstructions) and GIS tools. In total, 4 buildings within the study area are impacted by the regulatory flooding, as they are either fully or partially located within the flooding extents under the 100-year (Regulatory) storm event. The map in **Appendix K** shows the locations of these impacted buildings for the Regulatory event and provides as well their addresses.

12 UNCERTAINTIES, LIMITS AND RECOMMENDATIONS

Aquafor undertook a variety of measures to reduce uncertainty and increase confidence in the HEC-RAS hydraulic model's ability to predict water surface elevations. These included the use of estimated flow rates from the approved HEC-HMS hydrologic model and approved scenarios (10-day rain-on-snowmelt events), the use of appropriate hydraulic parameters based on technical guidelines, reviewing errors, warnings and notes in the model, and completing a visual verification of the preliminary model results screening the regulatory flood lines delineation.

12.1 1D Model Uncertainties

Detailed reach investigations of the low flow channel were not conducted along the entirety of the creek due to the scale of the study area and because of site access and safety conditions. River cross-sections were only surveyed within the channel at certain locations and generally located a few meters upstream and downstream hydraulic structures. Aquafor was able to correct the low flow shape including the bed channel elevation at those specific areas but did not correct the channel bed for the entire study area.

It is noted that the hydraulic model has first and foremost been developed for the purposes of flood hazard mapping. The development of the model was focused on generating water surface elevations for the Regulatory flood event (100-year storm). The hydraulic model results for smaller return period storm events have higher degrees of uncertainty. The hydraulic model and the results presented within this report for storms less than the Regulatory event will provide only general guidance for infrastructure planning and/or flood estimation purposes. Additional detailed studies (i.e., low flow channel corrections, placement of levees, etc.), may be required to ensure adequate accuracy of modelling results for smaller storm events.

12.2 2D Model Uncertainties

It is noted that areas of instability exist within the 2D model. These areas are largely based on road crossings and roadway overtopping locations. The purpose of the additional 2D modelling was to improve floodplain mapping in spill events from Selby Creek at Airport Road. Use of the 2D modelling results beyond the wider floodplain mapping scope will require more detailed topographic survey and model inputs that were not feasible at the scale of the current analysis.

Limitations on the topographic survey completed for the additional culverts may also affect 2D model uncertainties. At the time of the data collection for the additional culverts, Aquafor staff were unable to gain permission to enter private lands off of Airport Road and Johnsons Lane. Suspected culverts on these lands are not included in the modelling.

13 PUBLIC CONSULTATION

A Public Information Centre (PIC) was held on January 16, 2024. The session was held in order to provide an update on the flood hazard mapping study and address any project issues, comments, or concerns from the public. A presentation was given, providing information on the following topics:

- Flood hazards and historic flooding impacts;
- Approaches to reducing flooding impacts;
- Consultant company profile; and
- Overview of study methodology including topographic survey, hydrologic modelling, hydraulic modelling, and floodplain mapping.

Updated floodplain maps were also presented in draft format, including 50 year, 100 year (regulatory), and 200 year (climate change) floodlines. Comment sheets were made available at the PIC for the public to submit comments, concerns, or questions; however, no formal comments were received. PIC materials including a copy of the presentation are provided in **Appendix L.**

14 CONCLUSIONS

Aquafor developed a hydrologic model that was evaluated by comparing the estimated design storm peak flows to those simulated in previous studies and performing a sensitivity analysis. The hydrologic modelling approach was consistent with all pertinent technical guidelines (US Army Corps of Engineers, 2000, 2023; OMNR, 2002; EWRG, 2017). A LiDAR-derived digital terrain model (DTM) having a 1 m resolution, produced by NRCan, was used as the primary data source for determining subbasin and reach characteristics.

A 1-D HEC-RAS hydraulic model was built using the most recent data in possession of Quinte Conservation and is compliant with all pertinent technical guidelines (OMNR, 2002; EWRG, 2017; NRCan, 2023). This study will thus allow the Conservation Authority to recognize in detail the behavior of the riverine system of Selby Creek Watersheds for significant flood events.

The following points are key conclusions drawn from this study:

- The hydrologic model contains 35 flow nodes (junctions), 36 subbassins, and 28 reaches;
- Design storms from rainfall and rain-on snowmelt intensity-duration-frequency (IDF) curves were developed;
- Simulations were performed for the 2-year, 5-year, 10-year, 25-year, 50-year, 100-year (regulatory), and 200-year design storms;
- The hydraulic model includes a total of 7 river reaches and 659 river cross-sections, covering a total length of 48 km;
- The 1D model includes a total of 27 hydraulic structures including bridges and culverts;
- The model includes a total of 20 flow nodes;
- Survey investigations were conducted by Aquafor Beech Limited from July and August of 2023 (1D model development) and February 2024 (2D model development);
- Structure inventory sheets were developed for each of the surveyed hydraulic structures;

- 2022 LiDAR-derived DTM referenced to NAD83 (CSRS) UTM Zone 18N and vertical datum to CGVD2013 was applied to define river cross-sections, stream centerline, overbank locations and generate flood lines;
- Flows inputted in the hydraulic model were retrieved from Aquafor's HEC-HMS hydrologic model for the 2-year to the 100-year rain-on-snowmelt storm events and the 200-year rain-on-snowmelt storm (climate change scenario);
- Manning's roughness values based upon existing land use were assigned in the model according to the MTO Standard Coefficient (MTO Drainage Management Manual - Design Chart 2.01) and Technical Guidelines for Flood Hazard Mapping (EWRG, 2017);
- Steady flow model simulations and subcritical flow regime were simulated to generate 1D model results;
- A spill occurs upstream Airport Road and would start with the 5-year storm event;
- A 2D HEC-RAS Model were developed to analyze the flood impact of the spill;
- The final flood lines are a combination from the 1D model result (upstream XS # 1259) and the 2D model result (downstream XS # 1259), for the 5-year to the 200-year storms, while the 2-year flood line has been developed based on the 1D model only;
- Floodplain mapping for the 100-year storm event (Regulatory event) were defined.

The study culminated with the development of 1-D HEC-RAS hydraulic model for the Selby Creek Watershed, along with a 2D model to provide a more detailed representation of flooding extent within Airport Road Area. Aquafor has confidence in the 1D and 2D hydraulic models to predict water surface elevations with a reasonable degree of accuracy for the Regulatory event (100-year storm) and therefore, in the Regulatory Floodplain Mapping product.

15 REFERENCES

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APPENDIX A – HYDROLOGIC MODEL PARAMETERS

APPENDIX B – RAINFALL DEPTH

APPENDIX C – RAINFALL DESIGN STORM

APPENDIX D – RAIN-ON-SNOWMELT DESIGN STORM

APPENDIX E – HYDROLOGIC MODEL RESULTS

APPENDIX F – STRUCTURE INVENTORY SHEETS

APPENDIX G – HEC-RAS MODEL OUTPUT RESULTS

APPENDIX H – REGULATORY FLOODPLAIN MAPPING

APPENDIX I – 50-YEAR, 100-YEAR AND 200-YEAR FLOOD LINES MAPPING

APPENDIX J – DEPTH OVERTOPPING EXCEEDANCE

APPENDIX K – FLOODED BUILDING LOCATIONS WITHIN THE FLOODPLAIN

APPENDIX L – PIC MATERIALS