LANE CREEK FLOODPLAIN MAPPING

Quinte Conservation

Wellington, ON

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1 Introduction

Lane Creek in the Village of Wellington within Prince Edward County is a large mostly rural watershed that drains predominantly agricultural fields through the Village and into Lake Ontario. Flood risk mapping for the creek was produced in 1979 by Crysler Latham under the old National Flood Damage Reduction Program (FDRP) and is now over 42 years old. Quinte Conservation engaged Jewell Engineering with funding support from the National Disaster Mitigation Program (NDMP) and Prince Edward County to complete an update of the flood risk mapping.

Quinte Conservation also commissioned new topographic survey of the watershed using LiDAR technology to assist with the preparation of a new digital terrain model for the hydraulic modelling. Jewell Engineering supplemented the topographic model with precise GPS based survey of the channel and throughout the downtown area.

The flood risk mapping project was supported by updated hydrologic modelling based on the U.S. Army Corps of Engineers HEC-HMS v4.9 modelling software. Hydraulic modelling was complete using the HEC-RAS v6.0. The advantages of both modelling platforms is that they are publicly available and are both maintained and improved frequently by the U.S. government. The models prepared in this effort will be received by Quinte Conservation and can be used as living models for the analysis of watershed changes for many years into the future.

The flood risk mapping prepared by this project is suitable for use by Quinte Conservation to regulate the Lane Creek system under the Conservation Authorities Act and for protection of the public under their delegated responsibilities for natural hazards under the PPS by agreement with the Ministry of Natural Resources and Forestry and Ministry of Municipal Affairs and Housing. A schematic of the Lane Creek study area is provided in Figure 1-1.

The floodplain mapping was completed based on review and guidance in the following documents.

- Canada Ontario Flood Reduction Program FDRP and MNR (1986)
- Technical Guide – River and Stream Systems Flooding Hazard Limit, Ontario Ministry of Natural Resources, 2002
- Technical Guidelines for Flood Hazard Mapping March 2017, Central Lake Ontario, Credit Valley, Grand River, Ganaraska, Toronto Region and Notawasaga Valley Conservation Authorities
- Federal Flood Mapping Guidelines Series – www.publicsafety.gc.ca/cnt/mrgnc-mngmnt/dsstr-prvntn-mtgtn/ndmp/fldpln-mppng-en.aspx
- HEC-RAS 2D User’s Manual, US Army Corps of Engineers Hydrologic Engineering Center, 2021
- Drainage Management Manual, Ontario Ministry of Transportation, 1997
- Highway Drainage Design Standards, Ontario Ministry of Transportation, 2008
- Lane Creek Village of Wellington Water Management Study, Crysler & Lathem Ltd., 1979

Figure 1-1: Lane Creek Study Area

2 Hydrology

A hydrologic analysis was prepared for several nodes of interest throughout the Lane Creek watershed. Various methodologies were applied and compared to determine representative peak flows at each node. Each methodology was carefully considered prior to selection of the peak flows for use in the hydraulic model, including potential increase in flows due to spring melt conditions.

The detailed hydrologic analysis for Lane Creek is described below.

2.1 Lane Creek Watershed Catchments and Drainage Characteristics

The Lane Creek watershed has a total area of 14.8 km^2 and traverses the community of Wellington before it outlets to Lake Ontario. This section identifies detailed catchment boundaries and the prominent drainage characteristics for the watershed.

2.1.1 Discretized Catchment Areas

Jewell discretized the watershed into several sub-catchments based on confluence points and nodes of interest. The overall watershed including its discretized catchments are shown in Appendix A.

The HEC-HMS model uses catchment specific hydrology inputs to calculate peak runoff rates.

Catchment areas were delineated using topographic information from the following three sources.

1) Jewell survey data using GPS and a total station. Jewell completed a topographic survey within the urban areas of Wellington in the vicinity of Lane Creek. This included creek cross-sections, road centerlines, and overbank areas extending from Millennium Trail southward to Lake Breeze Court, Niles Street, Wharf Street, Wellington Main Street, and ultimately to Lake Ontario.

2) Quinte Conservation had LiDAR flown specifically for this project, and in particular the study area from Figure 1-1. LiDAR details were provided from Quinte Conservation and summarized in Section 3.1. LiDAR station reports are attached in Appendix E.

3) The 2013 Digital Terrain Model for South Central Ontario prepared by Land Information Ontario (LIO). A comparison of LIO data to Jewell survey data at several locations on-site was completed to ensure that the LIO data provides reliable topographic representation. This LiDAR data source was used for the portion of the watershed located outside of the designated study area.

2.1.2 Soils Mapping and Land Cover Usage

A soils map is provided in Appendix B. Soils information was obtained from the Agricultural Atlas published by the *Ontario Ministry of Agriculture, Foods, and Rural Affairs* and from the Southern Ontario Land Resource Information System (SOLRIS) that is also published by the province.

The soils are predominantly loam type soils and classified as Hydrologic Soils Group (HSG) B. The HSG classification for soils is used to identify drainage characteristics for various soil types. An excerpt from Chapter 8 of the *1997 MTO Drainage Management Manual* that describes drainage characteristics for each HSG is provided below. The Lane Creek watershed has 77% HSG B coverage as shown in Appendix B and Table 2-1. A significant portion of HSG C soils is also present within the watershed at 19% coverage.

The hydrologic soil group is used to classify soils into groups of various runoff potential.

The Soil Conservation Service (SCS) classifies bare thoroughly wet soils into four hydrologic soil groups (A, B, C and D). SCS descriptions of the four groups, modified slightly to suit Ontario conditions, are as follows: (Design Chart 1.09)

A: High infiltration and transmission rates when thoroughly wet, eg. deep, well drained to excessively-drained sands and gravels. These soils have a low runoff potential.

B: Moderate infiltration and transmission rates when thoroughly wet, such as moderately deep to deep open textured loam.

C: Slow infiltration and transmission rates when thoroughly wet, eg. fine to moderately fine-textured soils such as silty clay loam.

D: Very slow infiltration and transmission rates when thoroughly wet, eg. clay loams with a high swelling potential. These soils have the highest runoff potential.

In Ontario, soils have been found to lie between the main groups given above, and have therefore been interpolated as AB, BC, CD as appropriate, such as Guelph loam, which is classified as BC.

Figure 2-1: Excerpt from 1997 MTO DMM Describing Hydrologic Soils Group Classifications

The soils data is used to develop curve numbers (CNs) that are a key modelling parameter used in the Soil Conservation Service (now known as the *National Resources Conservation Service*) methodology for estimating the proportion of precipitation that will runoff the lands and the portion that will infiltrate. CNs are a function of soil type, land cover, slope, and land use. The higher the CN – the greater the proportion of precipitation that is expected to runoff the lands. CNs are representative of the pervious portion of the watershed. Jewell followed the guidance in MTO Design Chart 1.09 to determine curve numbers for the discretized catchments.

A review of land coverage for the Lane Creek watershed shows that the land use is predominantly agricultural fields. This is evident in aerial imagery that shows agricultural lands throughout the portions of the watershed located north of Wellington. A summary of land coverage percentage is provided in Table 2-2.

Future land use based on the Official Plan was considered in the floodplain mapping. The urban boundary was imported into RAS Mapper and the land cover was updated to reflect full-build out conditions for the community of Wellington. Impervious values were selected based on the anticipated development areas shown in the Official Plan. Impervious inputs were adjusted in the HMS model for future conditions. The developed portion of the Lane Creek watershed represents approximately 8% of the total area and is condensed at the downstream end of the system. Future development is not

Table 2-1: Summary of Hydrologic Soils Group and Soils Information for Lane Creek Watershed

SOIL CODE	HSG	SOIL NAME	% OF AREA
Acl : B2	B	Ameliasburg Clay Loam	3.4%
Al : B2	B	Ameliasburg Loam	4.3%
Bg : B1	A	Brighton Gravelly Sand	3.7%
Hc : B1	B	Hillier Clay Loam	69.53%
Wc : B0	C	Waupoos Clay	0.03%
Gc: B2	C	Gerow Clay Loam	18.9%
M : A0	D	Muck	0.14%
			100.0%

expected to increase the 100-yr peak flow for the overall system. This is evident based on the timing of the output hydrographs in the hydrologic modelling simulations. With future development, the peak flows are slightly reduced relative to existing conditions. Therefore, existing conditions were applied in the 2022 floodplain maps.

Table 2-2: Lane Creek Watershed Land Cover Summary

Land Use	%
Agriculture / Improved	80.9
Lakes and Wetlands	10.9
Treed	5.0
Impervious	3.3

The image below shows the urban boundary for future development within Wellington. Full build out conditions have been assumed with 60% imperviousness for future urban development areas.

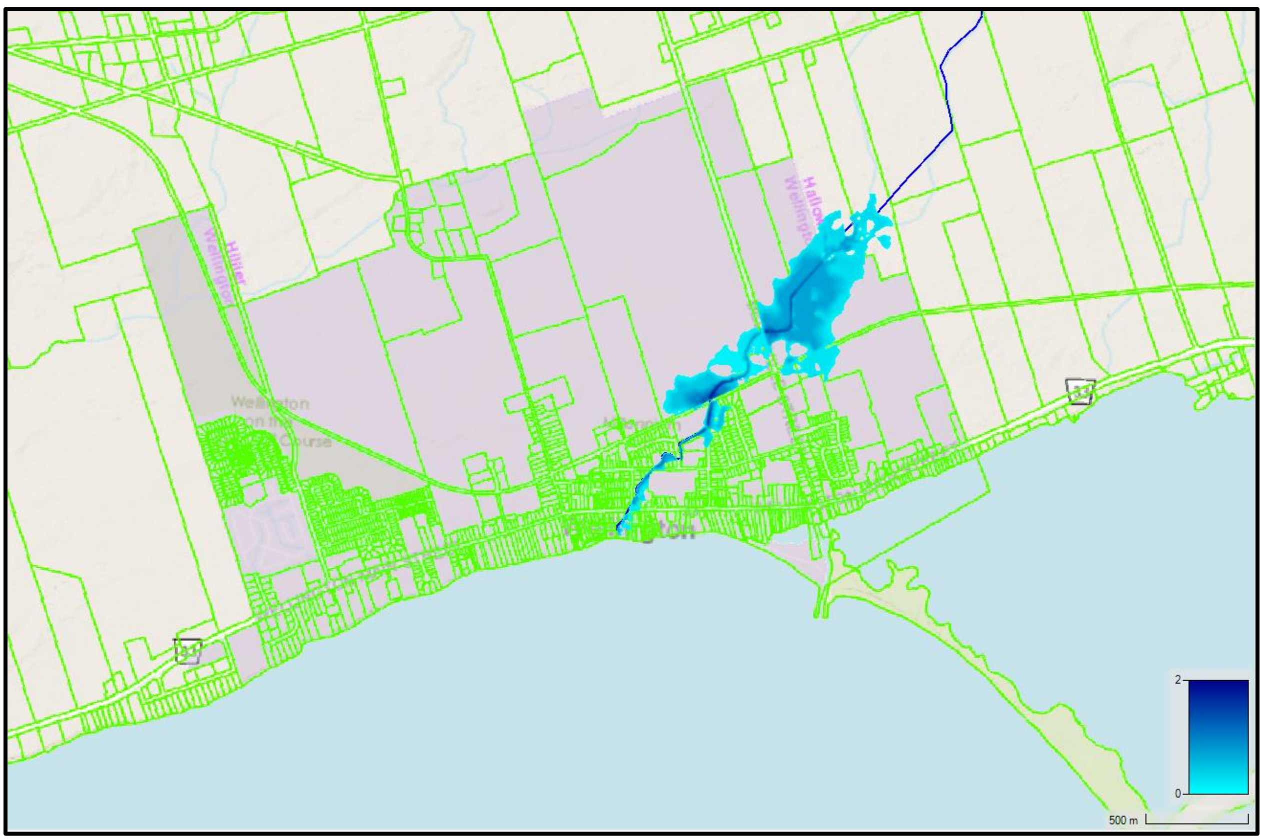


Figure 2-2: Future Full-Build Out Conditions for Community Wellington (Grey Shade)

2.2 Precipitation and Stream Flow Gauge Data

Precipitation data from Environment Canada IDF Curves was applied in the hydrology model. Jewell reviewed the station data from Kingston, Belleville, and Trenton. The Trenton station was selected since it has the longest record of data and is in closest proximity to Lane Creek. An AES distribution was applied consistent with the 1979 floodplain mapping. The rainfall inputs to the HEC-HMS model are provided in Appendix C.

There are no stream flow gauges along Lane Creek. However, Jewell completed an analysis of stream flow gauge data for hydrologically similar watersheds that are within close proximity to Wellington. A statistical analysis of these watersheds, in combination with a transposition of flows, applies in-field stream gauge data to estimate peak flows from the Lane Creek watershed. Further discussion on this method is described in Section 2.6.

Stream flow gauge data is obtained from the Water Survey of Canada (WSOC). Instantaneous flow data is downloaded to obtain peak runoff rates in each given year of record. The hydrologically similar watersheds selected for this analysis are part of the Consecon Creek and Wilton Creek drainage systems. These watersheds were selected due to their similar watershed characteristics, record of stream flow gauge data, and proximity to the Lane Creek watershed. Since the areas contributing to the Consecon Creek and Wilton Creek stream flow gauges are significantly larger than the Lane Creek watershed, a transposition of flows was calculated to account for this area difference.

Stream flow gauge data downloaded from WSOC is provided in Appendix D. The Consecon Creek stream flow gauge has 44 years of streamflow data. The Wilton Creek stream flow gauge has 38 years of data. The HEC-SSP Manual identifies a minimum data record of 30 years in order for reliable return period flows to be calculated. Since both data records were greater than 30 years, their length of record was suitable for this analysis. The stream flow gauge data for each station is provided in Appendix D.

The SCS and Chicago distributions were analyzed since they are recommended distributions for floodplain mapping as described in the MTO Drainage Manual. The AES distribution was also analyzed since it is a common distribution described in the MNR's Technical Guidelines and was applied in the historical mapping. The AES distribution was selected since it produced higher peak flows than the other two distributions. The 6-hour distribution was selected because peak flows from this duration were most consistent with the results from the General Frequency Analysis that applied field data from stream flow gauges. The longer duration events (i.e. 12-hr, 24-hr) were not selected because the back-calculated runoff coefficients in the model increase significantly and result in peak flow values that are too high relative to the calibrated flows from the transposition of measured stream flow data from nearby hydrologically similar watersheds.

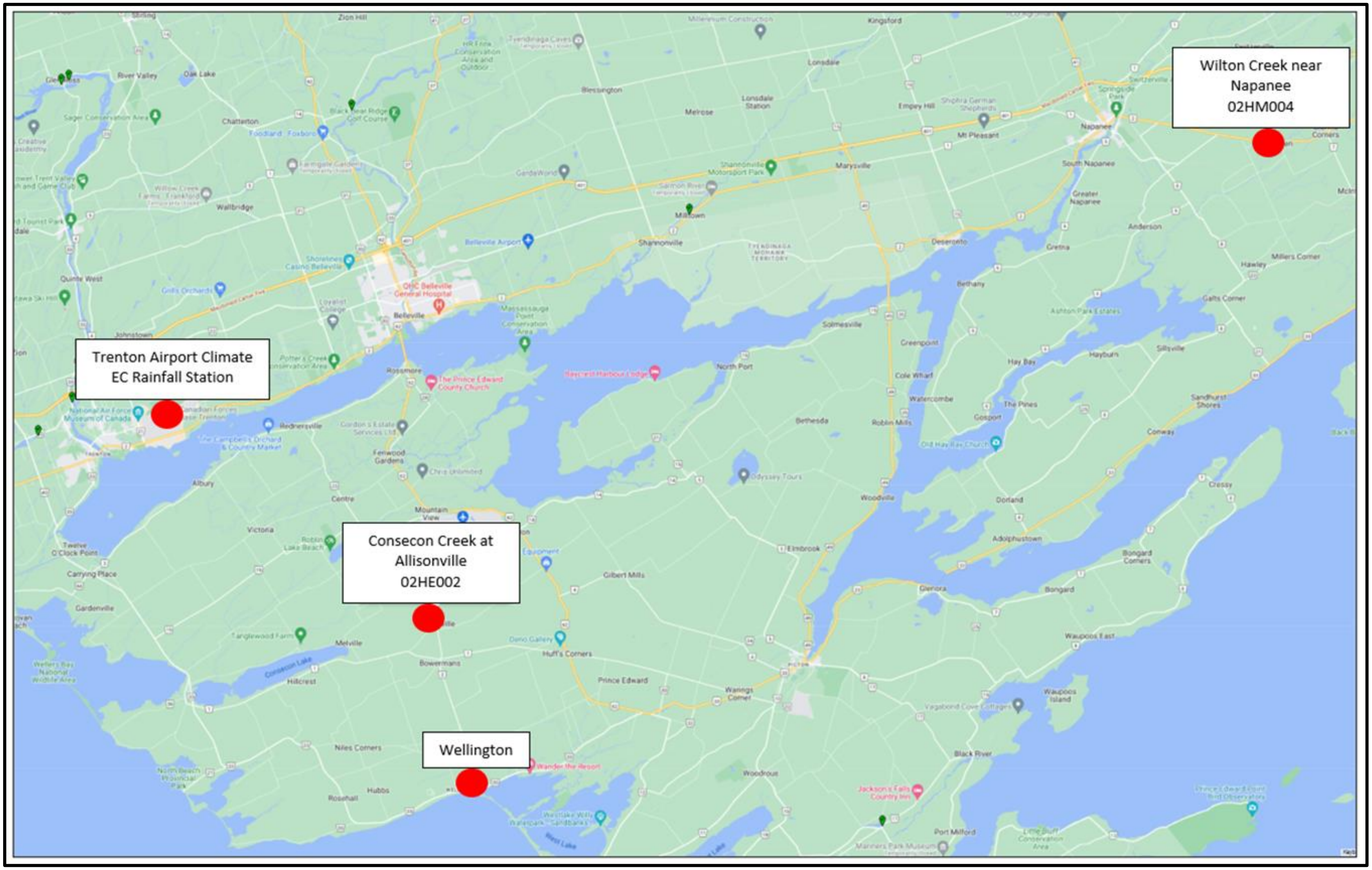


Figure 2-3: Rain and Flow Gauge Locations

2.3 Selected Modelling Programs

Jewell applied four modelling approaches to determine peak flows for Lane Creek.

- General Frequency Analysis (HEC-SSP)
- SCS Curve Number Method (HEC-HMS)
- Index Flood Analysis (OFAT III)
- Multiple Regression Analysis (OFAT III)

The above approaches involve the following three modelling tools.

1) HEC-HMS version 4.9. This hydrologic modeling software is developed by the U.S. Army Corps of Engineers and distributed freely. All modelling programs are simplifications of reality and are limited in their capabilities. While HEC-HMS is a well-established and recommended software program, it is limited by its input parameters and the uncertainty associated in the data sets and calculations used to produce these inputs. The HEC-HMS capabilities and limitations are described in P. 2 – 6 of the *HEC-HMS User's Manual*. The modelling program is acceptable for simulating peak flows for the hydraulic model and the most recent publication has been used for this project.

Parameters applied in HEC-HMS include:

- Precipitation – intensity, duration and frequency as well as distribution
- Catchment area
- Percent imperviousness – runoff volume, time to peak and peak flow increase with percent imperviousness
- Soil conditions – these determine how much and how quickly water will be removed from runoff through infiltration. This may be expressed as a curve number, or by a runoff coefficient or using an infiltration model such as Horton's Infiltration
- Slope – peak flows increase with slope
- Initial abstraction – depth of precipitation input that is subtracted from the model and does not contribute to runoff. This value is different for impervious and pervious areas and is expressed as two values.
- Manning's n – frictional coefficient that affects the time to peak. This value is different for impervious and pervious areas and is expressed as two values.
- Basin lag or time to peak.

2) HEC-SSP version 2.0. This is another software that is publicly available and developed by the U.S. Army Corps of Engineers. This software program is used to perform statistical analyses of hydrologic data obtained by stream flow gauges in the field.

The program has six statistical analysis components (HEC-SSP Statistical Software Pacakage User's Manual, 2016).

1) Flow Frequency Analysis (Bulletin 17)
2) General Frequency Analysis

3) Volume Frequency Analysis
4) Duration Analysis
5) Coincident Frequency Analysis
6) Balanced Hydrograph Analysis

For the purposes of obtaining return period flood flows, the General Frequency Analysis (GFA) component can be employed and is a recommended method in the 2002 MNR guidelines. This statistical component performs a peak flow frequency analysis using various methods. Parameters other than peak flows, such as stage or precipitation data, can also be calculated using a GFA.

In performing a flood frequency analysis, data is provided to the program and the calculated results are output in graphical and tabular formats. Prior to providing input data, a variety of settings are defined by the user. Some notable settings and their descriptions are shown in Table 2-3.

Table 2-3: HEC-SSP Settings and Descriptions

Setting	Description
Log Transform	○ This setting can be selected to have the frequency analysis performed on the logs of the data ○ Log Transform needs to be used to allow for the LogNormal and LogPeason III distributions to be selected ○ If the Log Transform setting is not used, the Normal and Pearson III distributions can be selected
Confidence Limits	○ Confidence limits measure the uncertainty of the computed value for a selected exceedance probability ○ Default settings calculate the 90% confidence interval, with confidence limits of 0.05 and 0.95
Distribution	○ This setting provides the analytical distribution options used to perform the frequency analysis ○ Distribution choices are None, Normal, LogNormal, Pearson III, and LogPearson III
Generalized Skew	○ Computes a skew value for the data ○ Three options that can be selected are Station Skew, Weighted Skew, and Regional Skew ○ The default option is Station Skew, where the skew of the computed curve is based solely on computing a skew from the provided data points

3) <u>Ontario Flow Assessment Tool (OFAT III).</u> OFAT III is developed by the Ministry of Natural Resources to estimate design flows and analyse the hydrology of the contributing drainage area. OFAT III contains two methodologies for determining the return period flows for streams in Ontario. These are Index Flood Method and the Multiple Regression Analysis Method.

The methods are described in various papers and summarized in the OFAT III Users Guide. Both methods are supported by the Province for use in Ontario in the MTO, *Drainage Management Manual, 1997* and the MNRF, *Technical Guide for River and Streams; Flood Hazard Limit, 2002*.

OFAT provides hydrologic characterizations of watersheds for modelling purposes. This would include the slope of the main channel, slope of the land, shape factor and area of lakes/wetlands. The tool also provides land cover characterization to determine the percentages and areas of forested areas, wetlands and lakes, as well as open fields.

2.4 Lane Creek Hydrologic Models

Four modelling approaches were used to determine peak flows for Lane Creek. The peak flow results are summarized in Tables 2-4 and 2-5. The 100-yr peak flow is selected as the regulatory event to delineate the flood hazard limit in accordance with Section 11 of O. Reg. 319/09 that states the following for flood event standards within the jurisdiction of Quinte Conservation:

"The applicable flood event standards used to determine the maximum susceptibility to flooding of lands or areas within the watersheds in the area of jurisdiction of the Authority are the 100 Year Flood Event Standard and the 100-year flood level plus wave uprush, described in Schedule 1. O. Reg. 319/09, s. 11."

Table 2-4 shows the results from the following four modelling methods

- General Frequency Analysis (HEC-SSP)
- SCS Curve Number Method (HEC-HMS)
- Index Flood Analysis (OFAT III)
- Multiple Regression Analysis (OFAT III)

The GFA was prepared based on stream flow gauge data for the nearby watersheds contributing to Consecon Creek and Wilton Creek. The return period peak flows calculated from the GFA were used to derive peak flows for the Lane Creek watershed using a transposition of flows.

The SCS CN method was applied using the HEC-HMS modelling program. With this program, catchment parameters specific to Lane Creek were calculated and supplied as inputs to a basin model with simulated rainfall events. The SCS CN method with a 6-hr, AES rainfall distribution yielded the largest peak flow for the 100-yr return period events in Table 2-4. Since the SCS CN method applies site-specific catchment information and yielded the largest peak outflow, it was selected for the hydraulic model that is used to identify the flood hazard limits for Lane Creek. A discussion of rainfall distributions and duration was provided in Section 2.2.

The Index Flood and Multiple Regression methods were used for comparison purposes. These are reputable methods since input parameters were within their selected range.

Further details on the modelling approaches are described in the following subsections.

Table 2-4: Lane Creek Peak Flow Results at the Community of Wellington (Node A as shown in Appendix A)

Return Period	Lane Creek - Transposition of Flows		Index Flood	Multiple Regression	SCS CN 6-Hr, AES	Crysler and Lathem, 1978
	Consecon Creek	Wilton Creek				
2	5.0	5.1	3.7	3.6	N/A	
5	6.8	6.7	4.6	5.6		
*25	9.2	8.4	6.2	8.6	8.2	7.8
50	10.7	9.3	7.3	10.1	10.2	9.9
100	12.0	9.9	8.2	11.7	**14.0**	12.3

**20-yr shown for Index Flood and Multiple Regression*

2.5 SCS Curve Number (HEC-HMS)

The SCS Curve Number (CN) method uses the land use and hydrologic soils group information to develop the CN as the loss method. This modelling approach is supported by the HEC-HMS program and the loss method is discussed further in the following subsection.

The SCS CN method produced a 100-yr peak flow of 14.0 m^3/s. This is greater than the calibrated flows from the nearby hydrologically similar watersheds of Consecon Creek and Wilton Creek. At 14.0 m^3/s, the peak flow is also higher than the Index Flood and Multiple Regression results.

Further, the 2022 100-yr peak flow of 14.0 m^3/s is 13% greater than the 1979 maximum peak flow of 12.4 m^3/s. The precipitation volume was 76.2mm in the 1979 model, and Jewell applied a rainfall depth of 80.8mm based on the Trenton IDF Curves. This represents a 6% increase in rainfall without including adjustments for climate change or land cover. The climate forecasting tool from the MTO IDF Look-Up was then applied to the 80.8mm rainfall depth to obtain the value of 84.6mm of depth, which was the depth applied in the rainfall distributions for the 2022 event modelling (11% increase relative to 1979). The change in land cover with future full build-out conditions presents no increase in peak flows since the development area is concentrated at the bottom of the Lane Creek watershed. This is evident in a comparison of peak flows and timing of the hydrographs in existing and full build-out conditions. Based on the above information, the 2022 modelling results present a slightly more conservative estimate than the previous modelling work from Crysler and Lathem.

The HEC-HMS model was configured to calculate peak flows at five nodes of interest as shown in Appendix A. The nodes were selected based on confluence points and significant crossing locations. HEC-HMS model schematics for existing and future development conditions are shown in Appendix G. The peak flows at these nodes of interest for each return period event are summarized below.

Table 2-5: Peak Flow at Each Node of Interest for Each Return Period Event

Node	Return Period (yr)				
	2	5	25	50	100
E	0.3	1.3	3.8	5.2	7.4
D	0.5	2.2	6.3	8.5	11.4
C	0.7	2.9	8.2	10.4	13.9
B	0.7	2.9	8.3	10.4	14.0
A	0.7	2.9	8.2	10.4	14.0

2.5.1 Loss Method

Jewell selected the curve number loss method since it accounts for both land cover and hydrologic soils group information. It was also selected because of the reputable sources available for this information in SOLRIS and OMAFRA. Both data sources are published by the province. Another common loss method that was investigated was the Green-Ampt method. However, it was not selected since this method was found to be extremely sensitive to the soil types and hydraulic conductivity values. Since no boreholes or test pits were included as part of the project, this loss method was not selected.

AMC II per Chapter 8 of the MTO Drainage Manual was applied for antecedent moisture conditions (AMC). This represents 'average' soil conditions. Saturated soil conditions (AMC III) were not selected because this condition, combined with the 100-yr rainfall event, would produce a peak flow beyond the 100-yr return period frequency. Saturated conditions were also not selected because the General Frequency Analysis addressed spring melt conditions since the instantaneous annual peaks in the flow gauge data sets consistently occurred during the spring snow melt season.

AMC III conditions are only required for the last 12 hours of the Hazel storm or when there is other reason to believe saturated conditions are necessary. The selected peak flow from the HEC-HMS model that was applied in the HEC-RAS model was higher than the General Frequency Analysis (GFA). Since the GFA generally represents saturated conditions associated with spring runoff, this condition has been addressed and there are no concerns with the use of AMC II in the HMS model.

2.5.2 Lag Time

Jewell applied the SCS lag time method to determine time of concentration and lag time values. This method was selected since it is recommended for watersheds up to 24 km^2 and the Lane Creek watershed is 14.8 km^2. It was also selected because it accounts for land cover and soil types by incorporating the curve number value to estimate a retardance factor. The SCS lag time method is described in the *Hydrology National Engineering Handbook* published by the United States Department of Agriculture and the Natural Resources Conservation Service.

For existing and future development areas, the Bransby-Williams method was applied since these drainage areas are relatively small and the runoff coefficients are expected to be higher than 0.4.

2.5.3 Channel Routing

Channel routing was completed using the Muskingum-Cunge method. This method is applicable for reaches with small slopes (majority of Lane Creek has a watershed slope of approximately 0.2%) and allows the user to input a cross-section to represent the ground surface data for the channel and overbank areas. Cross-sections were obtained from the terrain data and then simplified into an eight-point cross-section (see Appendix E). The Muskingum-Cunge method was also selected since it incorporates Manning's n values to represent expected roughness for the channel and overbank areas. The applied Manning's n values are based on the design charts in the *MTO Drainage Manual*.

2.5.4 Hydrology Input Summary

A hydrology input summary is provided below for existing land cover and full development conditions based on the zoning identified in the Official Plan. This summarizes the area, curve number, and lag time applied for each sub-catchment.

Table 2-6: Hydrology Input Summary for Lane Creek Catchments Applied in HEC-HMS Model (Existing Conditions)

Catchment	Area (km^2)	Watershed Length (m)	Watershed Slope (%)	% Imp.	CN	Lag Time (min)
500	6.51	2700	0.7	0	66.8	183
400	4.37	2990	0.6	0	68.5	221
300	3.09	2220	0.4	1	69.4	213
200	0.58	490	0.9	3	71.5	92
100	0.25	930	1.1	60	61.7	12

Table 2-7: Hydrology Input Summary for Lane Creek Catchments Applied in HEC-HMS Model (Future Developed Conditions)

Catchment	Area (km^2)	Watershed Length (m)	Watershed Slope (%)	% Imp.	CN	Lag Time (min)
500	6.51	2700	0.7	0	66.8	183
400	4.37	2990	0.6	0	68.5	221
300B	2.58	2220	0.4	1	69.3	213
300A	0.52	780	1	60	61.0	19
200	0.58	490	0.9	60	62.5	92
100	0.25	930	1.1	60	61.7	12

2.5.5 Time Step

The time step for the HEC-HMS model is 10 minutes based on the recommendations in the *MTO Drainage Manual* and the time steps associated with the AES, SCS, and Chicago distributions. This time step was also selected since it produced relatively smooth hydrographs and no sharp peaks that tend to overestimate peak flows.

The time step for the HEC-RAS model described in Section 3 is much smaller due to unsteady flow conditions and the large number of grid cells used to capture detailed water surface elevations for the creek and overbank areas. A time step of 5 seconds and output intervals of 10 minutes was applied in the HEC-RAS model to minimize error while limiting computation intervals to a maximum of 20 iterations. The computation log file in the HEC-RAS model shows a percentage error of 0.03%. This error is minimal and deemed acceptable for this project.

2.6 General Frequency Analysis (HEC-SSP)

A general frequency analysis (GFA) was used to incorporate stream flow gauge data into the hydrology results. Lane Creek does not have a stream flow gauge. Therefore, a GFA was applied on stream flow gauge results for hydrologically similar watersheds in close proximity to the Lane Creek watershed. Two similar watersheds were selected due to their similar watershed characteristics, length of data records, and proximity to Lane Creek. These are the watersheds for Consecon Creek and Wilton Creek that drain to the Water Survey of Canada (WSOC) stream flow gauges identified below.

Consecon Creek: ***Station Number 02HE002***

Wilton Creek: ***Station Number 02HM004***

The GFA method calculates return period flows using HEC-SSP and a Log Pearson Type III distribution. In Ontario, the Log Pearson Type III distribution is used when the coefficient of skew is negative (Floodplain Management in Ontario Technical Guidelines, Ministry of Natural Resources).

WSOC maximum annual instantaneous peak flow data was supplied to HEC-SSP. As mentioned in Section 2.2, the Consecon Creek stream flow gauge has 44 years of streamflow data. The Wilton Creek stream flow gauge has 38 years of data. Both gauges have a data record greater than the 30-yr minimum requirement to calculate an extreme event such as the 100-yr flood (MNR Technical Guidelines).

Section 2.1.2 of this report discussed the soils mapping and land cover usage for the Lane Creek watershed. It shows that the soils for the Lane Creek watershed are predominantly HSG B and that the land cover is predominantly agricultural lands. Tables 2-8 and 2-9 show that this is also the case for the Consecon Creek watershed contributing to the stream flow gauge 02HE002. Tables 2-10 and 2-11 also show that this is the case for the Wilton Creek watershed contributing to stream flow gauge 02HM004. Due to the soil and land cover similarities, as well as the proximity to Lane Creek, the WSOC stream flow gauges at Consecon Creek and Wilton Creek were considered hydrologically similar watersheds.

While these watersheds are similar, the input parameters are not specific to the Lane Creek watershed. Therefore, the GFA was used for comparison purposes. The watershed areas contributing to Consecon Creek and Wilton Creek are significantly greater than the catchment areas for Lane Creek. To accommodate this area discrepancy, a transposition of flows was completed using the equation provided from MTO drainage publications (see excerpt below).

Transposition and interpolation of data from a stream gauge can be done based on the Modified Index Flood method as follows:

$$Q2 = Q1\ [A2 / A1]^{0.75}$$

Where:
Q1 = Known peak discharge
Q2 = Unknown peak discharge
A1 = Known basin area
A2 = Unknown basin area

Figure 2-4: Excerpt from MTO Online Drainage Manual

A benefit of the GFA is that it gives a reasonable expectation for the 100-yr peak flow for the Lane Creek watershed based on measured data. In a review of the data records for Consecon Creek and Wilton Creek, the annual instantaneous peak flows consistently occur during common snow-melt and freeze-thaw times of year. This strongly suggests that the 100-yr peak flow for Lane Creek would occur from a snow melt condition rather than a single rainfall event.

The transposed results for the Lane Creek 100-yr peak flow previously shown in Table 2-4 are less than the HMS peak flows used to produce the 2022 regulatory floodplain maps. Therefore, the flood hazard limit covers potential extents of flooding that would occur during a freeze-thaw event.

Table 2-8: Land Cover for Consecon Creek Watershed Contributing to WSOC Gauge 02HE002

Land Use	%
Agriculture / Improved	55.8
Lakes and Wetlands	29.4
Treed	13.4
Impervious	1.5

Table 2-9: Hydrologic Soils Group Summary for Consecon Creek Watershed Contributing to WSOC Gauge 02HE002

HSG	Coverage
B	78%
C	7%
D	15%

Table 2-10: Land Cover for Wilton Creek Watershed Contributing to WSOC Gauge 02HM004

Land Use	%
Agricultural / Improved	63.2
Lakes and Wetlands	12.4
Treed	20.0
Impervious	**4.4**

Table 2-11: Hydrologic Soils Group Summary for Wilton Creek Watershed Contributing to WSOC Gauge 02HM004

HSG	Coverage
B	92%
D	8%

2.7 Ontario Flow Assessment Tool

The Ontario Flow Assessment Tool (OFAT III) provides two hydrology methods that are suitable for Lane Creek peak flow estimates. These are the Modified Index Flood Analysis and the Multiple Regression Analysis. Each are described in the following subsections.

2.7.1 Modified Index Flood Analysis

Jewell employed the Ontario Flow Assessment Tool (OFAT III) developed by the Ministry of Natural Resources to estimate design flows and analyse the hydrology of the contributing drainage area. OFAT III contains two methodologies for determining the return period flows for streams in Ontario. These are Index Flood Method (this section) and the Multiple Regression Analysis Method (next section).

The Index Flood method relates the annual peak instantaneous flow determined for 247 stream gauges across Ontario to drainage area. Twelve regions across the province were identified as having similar characteristics and a regression curve was developed for each region. See Figure 2-5.

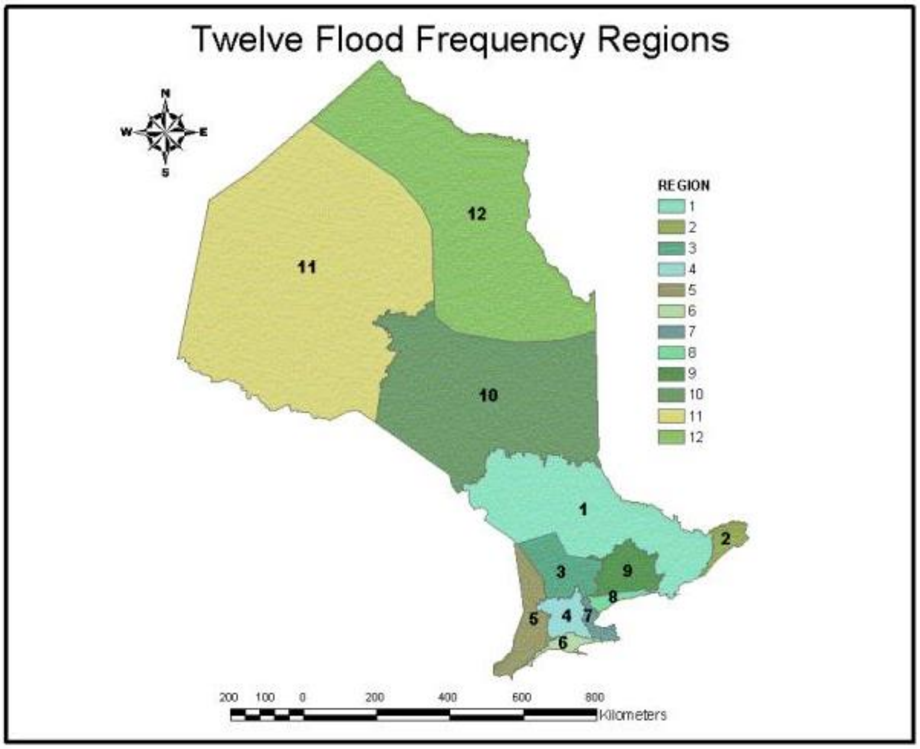


Figure 2-5: Index Flood Regions (from OFAT III)

The 2-yr flows are resolved directly from the equation using the constant and exponent from Table 2-12. OFAT III determines the region based on location of the catchment and selects the appropriate constants. Other return period flows may be derived from the 2-yr flow by multiplying with the factors provided in Table 2-13.

Equation 1: Index Flood Method

$$Q_2 = CA^n$$

Where:

Q_2 = 2 year return period (3 parameter Log Normal) flood
A = Drainage Area (km^2)
C = constant
n = exponent (slope of the line)

Table 2-12: Table of Constant (C) and Exponent (n) for use in the Modified Index Flood Equation

Region	Constant (C)	Exponent n
1(a)	0.22 (A < 60 km^2)	1.000
1 (b)	0.73 (A > 60 km^2)	0.707
2	0.51	0.896
3	0.20	0.957
4	0.71	0.842
5	0.45	0.775
6	0.41	0.806
7	1.13	0.696
8	0.73	0.785
9	0.40	0.810
10	0.28	0.849
11	0.38	0.706
12	0.59	0.765

Table 2-13: Ratio of Various Flood Frequencies to Q_2

Region	$Q_{1.25}/Q_2$	Q_2/Q_2	Q_5/Q_2	Q_{10}/Q_2	Q_{20}/Q_2	Q_{50}/Q_2	Q_{100}/Q_2	Q_{200}/Q_2	Q_{500}/Q_2
1	0.95	1.00	1.24	1.43	1.62	1.86	2.04	2.23	2.48
2	0.94	1.00	1.29	1.52	1.74	2.04	2.25	2.45	2.72
3	0.93	1.00	1.33	1.62	1.89	2.25	2.54	2.82	3.19
4	0.93	1.00	1.32	1.57	1.80	2.13	2.37	2.60	2.92
5	0.94	1.00	1.27	1.50	1.74	2.06	2.34	2.62	2.96
6	0.91	1.00	1.43	1.78	2.13	2.60	2.96	3.33	3.84
7	0.94	1.00	1.27	1.47	1.66	1.90	2.07	2.24	2.47
8	0.92	1.00	1.43	1.85	2.30	2.96	3.46	4.00	4.77
9	0.94	1.00	1.27	1.50	1.72	2.02	2.26	2.49	2.80
10	0.95	1.00	1.20	1.35	1.48	1.64	1.77	1.90	2.07
11	0.93	1.00	1.33	1.62	1.90	2.32	2.67	3.05	3.55
12	0.94	1.00	1.22	1.38	1.52	1.68	1.80	1.90	2.05

Table 2-14: Limitation of Application of Index Flood Method based on Drainage Area

Region	Minimum (km^2)	Maximum (km^2)
1	0.11	9270
2	76.1	3816
3	86.0	3960
4	2.5	5910
5	14.2	4300
6	5.2	697
7	63.5	293
8	4.9	800
9	24.3	1520
10	18.6	11900
11	0.7	24200
12	4250	94300

The parameters for the Lane Creek watershed at Node A were within the allowable range for use of the Index Flood method. Peak flows from the Index Flood method were shown in Table 2-4.

2.7.2 Multiple Regression Analysis

The multiple regression methodology compares watershed characteristics of the watershed under study with those of other watersheds within a similar region. The province was broken into four regions of similar response to weightings of watershed characteristics to flow. The Figure 2-6 image shows the regions. OFAT III determines the region based on location of the catchment and selects the appropriate constants.

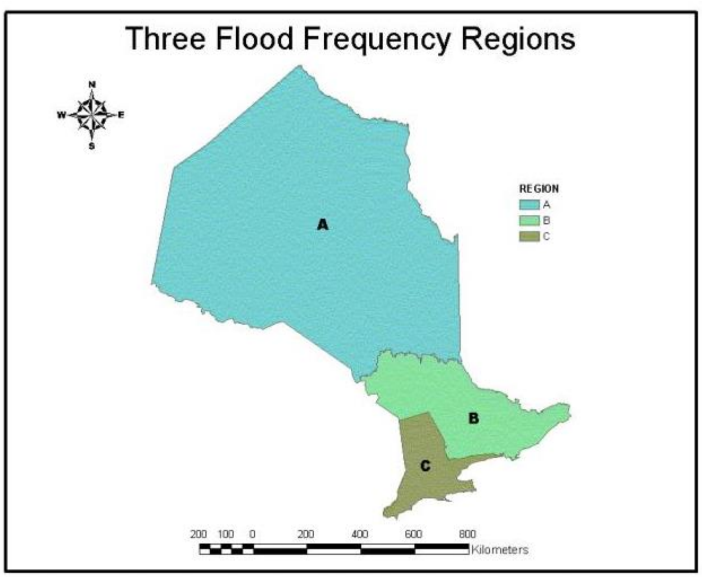


Figure 2-6: Regions of Similar Response for Multiple Regression Method

The characteristic values are entered into Equation 2 using constants provided by either the all Ontario Values or those specifically derived for the region. The coefficients for Eastern Ontario (Region B) are provided below as well as those for All Ontario (Tables 2-15 and 2-16).

The multiple regression method has been tested and verified for use within parameter limitations given in Table 2-17. The method should not be applied if any of the drainage area parameters lie outside of these limitations.

Equation 2: Multiple Regression Method

$$\text{Log(QT)} = a_0 + a_1\text{Log(DA)} + a_2(\text{BFI})^{1/2} + a_3(\text{SLP})^{1/3} + a_4(\text{ACLS})^{1/2} + a_5(\text{SLP}) + a_6\text{Log(MAR)} + a_7(\text{MAR}) + a_8\text{Log(ACLS+1)} + a_9(\text{MAP}) + a_{10}(\text{SF})$$

Where:

DA = Drainage Area (km2)
SLP = Mean Channel Slope (m/km)
ACLS = Index of Area Controlled by Water & Wetland (%)
SF = Shape Factor (dimensionless) (=LNTH^2/DA, where LNTH = length of main channel (km) and DA = drainage area (km2))
BFI = Base Flow Index (dimensionless)
MAR = Mean annual Runoff (mm)
MAP = Mean Annual Precipitation (mm)

Table 2-15: Multiple Regression Coefficients for Region B (Eastern Ontario)

Flow (cms)	a_0	a_1	a_3	a_4	a_{10}	SF	R^2
Q_2	0.2143	0.7464	-0.2172	-0.0194	-0.0077	0.14	0.91
Q_5	0.2746	0.7443	-0.1961	-0.0198	No data	0.14	0.89
Q_{10}	0.3795	0.7217	-0.1799	-0.0202	No data	0.15	0.87
Q_{20}	0.2311	0.7461	No data	-0.0197	-0.0081	0.15	0.87
Q_{50}	0.3659	0.6989	No data	-0.0275	No data	0.15	0.85
Q_{100}	0.4471	0.6839	No data	-0.0276	No data	0.16	0.83

Table 2-16: Multiple Regression Coefficients for All Ontario

Flow (cms)	a_0	a_1	a_3	a_4	a_{10}	SF	R^2
Q_2	0.2143	0.7464	-0.2172	-0.0194	-0.0077	0.14	0.91
Q_5	0.2746	0.7443	-0.1961	-0.0198	No data	0.14	0.89
Q_{10}	0.3795	0.7217	-0.1799	-0.0202	No data	0.15	0.87
Q_{20}	0.2311	0.7461	No data	-0.0197	-0.0081	0.15	0.87
Q_{50}	0.3659	0.6989	No data	-0.0275	No data	0.15	0.85
Q_{100}	0.4471	0.6839	No data	-0.0276	No data	0.16	0.83

Table 2-17: Multiple Regression Parameter Limitations for Region B

Variable	Q2-Q20 Minimum	Q2-Q20 Maximum	Q50-Q100 Minimum	Q50-Q100 Maximum
DA	13.9	3810.0	13.9	4770.0
BFI	0.26	0.82	0.26	0.90
SLP	0.14	5.77	0.02	5.77
ACLS	0.0	97.0	0.0	100.0
SHP	1.41	42.14	1.38	42.14

The parameters for the Lane Creek watershed at Node A were within the allowable range for use of the Multiple Regression method. Peak flows from the Multiple Regression method were provided in Table 2-4.

2.8 Climate Change

With potential impacts due to climate change, there is concern for increased frequency and intensities of severe rainfall events. Therefore, Jewell has considered potential impacts due to climate change in the floodplain analysis.

Potential climate change impacts on peak flows are inherently difficult to quantify due to the earth's extremely complex global atmospheric and hydrologic systems. The MTO IDF Look-Up Tool is supported by the province and offers projected rainfall data for future years. Jewell selected rainfall data for 50 years into the future; the year 2072. This projected rainfall data increases the rainfall depth for the 100-yr event from 80.8mm to 84.6mm as shown in Appendix C.

The stream flow gauge data from Section 2.2 strongly suggests that the statistical 100-yr flood event will occur during a freeze-thaw/snow-melt condition. These events produce high peak flows due to a large volume of stored water content that is released when warmer temperatures occur. With warmer seasonal temperatures generally expected due to climate change, it is reasonable to expect less stored water content during the winter months, since the period of below-freezing temperatures would be shortened with higher average temperatures. With less stored water content, it is possible that the statistical 100-yr peak flow would not increase even with increased rainfall depths for single event conditions. However, for conservatism, the 2072 rainfall data was included in the HEC-RAS model to account for potential impacts on flood hazard limits with increased rainfall depths.

3 Hydraulics

The hydraulic analysis was prepared using HEC-RAS version 6. The hydrology results from the HEC-HMS model were applied in the HEC-RAS model to delineate the Lane Creek flood line. This section describes the topography, cross-sections, storage impacts, and bridge/culvert crossing analysis. It also discusses model sensitivity and spill areas.

3.1 Topography, Cross-Sections, and Geometry for 2-Dimensional Modelling

The topography of the floodplain is characterised by a well-defined channel and urban overbank areas for locations south of the Millennium Trail. Upstream of the Millennium Trail, the creek is less defined and has existing rural land use in the overbank areas. The topography within the floodplain was measured in great detail. Quinte Conservation had LiDAR flown specifically for this project, and in particular the study area from Figure 2-1. LiDAR details provided from Quinte Conservation are summarized below. LiDAR station reports are attached in Appendix E.

- Ground Control:
  - The ASPRS control was set to meet Federal Guidelines: 7 NVA (Non-Vegetated Vertical Accuracy) and 2 VVA (Vertical Vegetated Accuracy) ground control points for each site were specified in the contract. However, 6 NVA and 6 VVA points were provided for the Lane Creek site. Three other sites were flown the same day (Weller's Bay, Meyer's Pier and Madoc) totalling 32 NVAs and 32 VVAs for the entire mission across all four sites. Quinte Conservation has photos of the control points in their records.

- Calibration:
  - Airborne Imaging performs a complete calibration on every LiDAR acquisition flight. The data is first produced with its predetermined boresight values and then the calibration is refined by applying corrections to the attitude of the aircraft (roll, pitch and heading) and fluctuations if necessary. To statistically quantify the accuracy, the LiDAR elevations are compared with independently surveyed ground points. A GPS mounted truck collects data while driving on an open road. The kinematic positions on the road are post-processed from a nearby base station to provide ground truth points.

- DEM/DTM Resolution:
  - DEM/DTM resolution is 1 metre.

Additionally, Jewell completed detailed survey cross-sections of the creek. Jewell also completed detailed GPS survey for certain areas of interest such as all culvert crossings within the study area as well as the Midtown Brewery parking lot and a portion of Wharf Street south of Main Street.

The topographic data was compiled into a terrain layer located within the RAS mapper component of the HEC-RAS model. This combined terrain layer was strategically imported to ensure each portion of the study area is represented by the best available topographic data source.

Historically, 1-dimensional hydraulic models have been used for floodplain mapping. This type of model requires cross-section data to be set up by the user to represent the geometry data applied in the hydraulic model calculations. With recent advancements in the HEC-RAS modelling software that is developed by the U.S. Army Corps of Engineers and distributed freely, 2-dimensional modelling presents an alternative that can provide added benefit depending on the creek of interest.

A 2-dimensional model was selected for Lane Creek for the following reasons:

- To simulate the flow in the overbank areas that are located within the existing Wellington urban centre, including several buildings located within the regulatory flood limits.
- To accommodate the unusual geometry near Wharf Street where Lane Creek flows underneath an existing building. This geometry would produce a flow direction that is not parallel to the creek system.
- To assess the significant storage areas upstream of Belleville Street and the Millennium Trail.
- The detailed topographic data supported the use of a two-dimensional model.

The detailed topographic data was used to develop a computational mesh that ultimately controls the movement of water through Lane Creek and the surrounding overbank areas. For each computation cell, an elevation-volume relationship is calculated to produce a single water surface elevation.

The Lane Creek model is comprised of 67,000 grid cells, with refinement areas applied for the channel and specific areas of interest, such as road crossings within the Wellington urban core. The purpose of the refinement areas is to ensure the movement of water follows the direction of the creek by aligning the cells to the channel banks, and to identify the flow depth and direction in urban overbank areas. An example of the grid applied in the model is shown in Figure 3-1.

3.2 Internal and External Boundary Conditions

There are two boundary conditions for the 2D model. The inflow boundary condition is a flow hydrograph supplied from HEC-HMS. The outflow boundary condition can be a stage hydrograph or normal depth. Normal depth was selected due to the drop in watercourse profile at the bottom end of the channel as well as the elevations of the creek and crossings relative to the Lake Ontario water surface elevations (WSELs).

The 2D unsteady flow model received its flow data from an inflow hydrograph where the incoming flows change with time. The inflow hydrograph was obtained by the tabular output in the HEC-HMS model. Node A produced the largest peak flow and includes the entire Lane Creek watershed. Therefore, the hydrograph at Node A was selected as the inflow hydrograph for the model's inflow boundary condition. A schematic of the inflow hydrograph with a peak of 14.01 m^3/s is provided in Figure 3-2.

The HEC-RAS 2D User's Manual describes that two-dimensional unsteady flow models are most commonly computed using Diffusion Wave equations (DWE). Since the Lane Creek system has several bridge/culvert crossings, smaller grid cells and computational time steps were selected to accommodate this method. A maximum Courant Number of 3.0 was set in the unsteady flow model run configuration for the DWE calculations as per Section 6 of the HEC-RAS 2D User's Manual.

Figure 3-1: Example of Computational Mesh near Lake Breeze Ct. and Maple Ave with Refinement Regions for Channel and Overbank Areas

MNFR guidelines indicate the regulatory floodplain can be delineated by superimposing the regulatory flow with the 2-yr lake level, as well as the 2-yr flow with the 100-yr lake level. With this guidance the downstream boundary condition was reviewed to determine potential impacts from the Lake Ontario water level on the Lane Creek flood mapping.

The Lane Creek channel outlets to a rock beach that connects to Lake Ontario. The most downstream point of the channel has an invert of 75.91m, at which point it connects to the rock beach. This downstream limit of Lane Creek is above the 100-yr WSEL applied for Lake Ontario of 75.70m. Therefore, the 2-yr lake level was not superimposed to the regulatory flow since it would present no increase in WSELs or floodplain mapping extents.

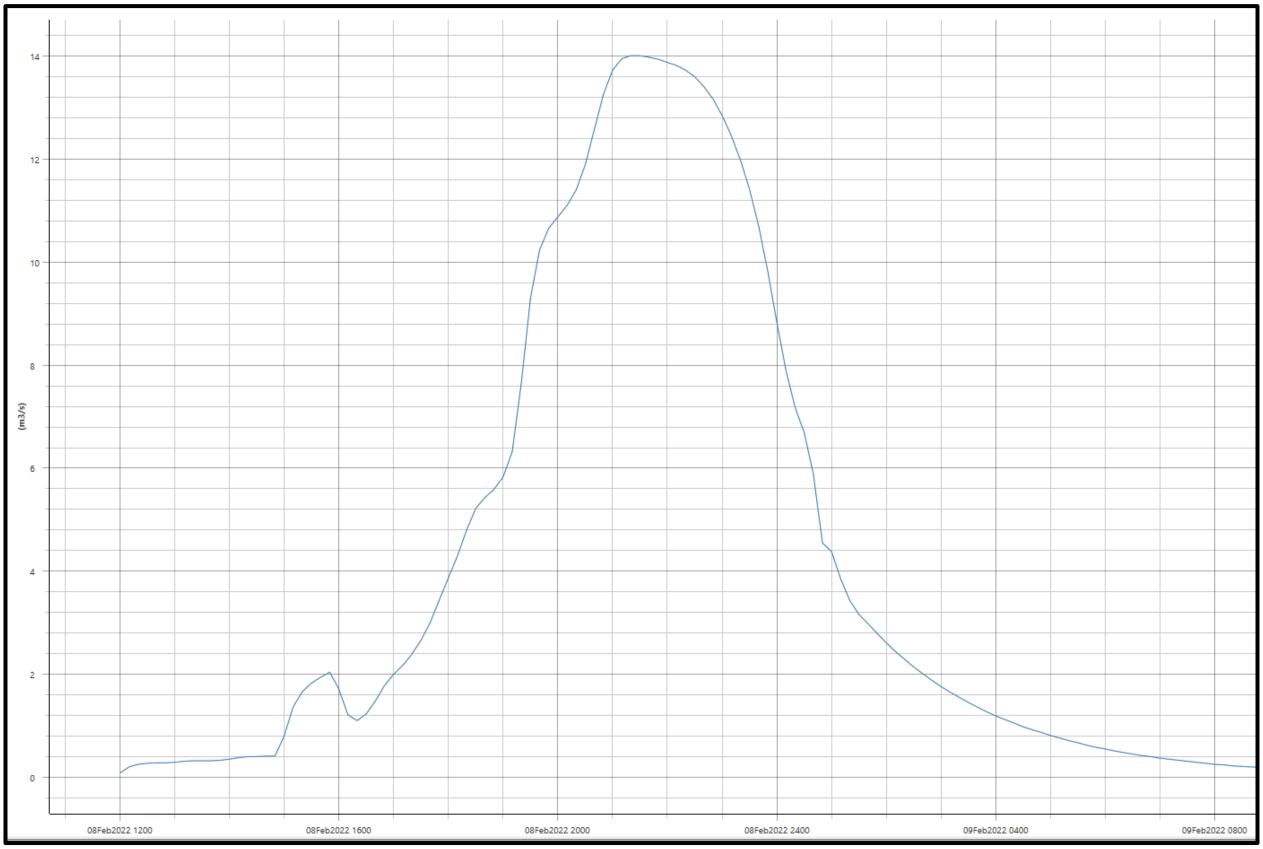


Figure 3-2: Inflow Hydrograph Supplied to HEC-RAS Model

Similarly, the 100-yr lake level was not superimposed with the 2-yr flow. This is because none of the culvert crossings or Lane Creek channel locations are impacted by the 100-yr lake level. The most downstream crossing is Main Street, which has a road sag elevation of 79.85m (>3m above 100-yr WSEL) and a culvert invert of 78.15 (>2m above 100-yr WSEL). This scenario would have narrower flood extents relative to the regulatory floodplain map. A watercourse profile showing the steep drop off at the downstream limit of Lane Creek is shown below.

Figure 3-3: Lane Creek Watercourse Profile (Red) with WSEL (Blue)

3.3 Storage Impacts

The culvert and bridge crossings within the Lane Creek study area are described in Section 3.4. These crossings are included in the HEC-RAS model through the use of the 2D storage area connection component of the geometry editor.

Information for the crossings include the bridge/culvert sizes, lengths, embedment depths, roughness coefficients, and upstream and downstream inverts. The road profile simulates a weir in combination with the culvert openings.

There are two storage areas of interest in the Lane Creek study area. Storage Area #1 is located upstream of Belleville Street. Storage Area #2 is located upstream of the Millennium Trail. These low-lying storage areas are illustrated by the wide floodplain extents upstream of these crossings. The backwater affects from the existing crossings contribute to the storage areas and should be considered in any future crossing replacements.

Storage Area #1 has approximately 80,000 m^3 of volume within the floodplain area upstream of Belleville Street. This storage area consists of shallow ponding with an average depth of approximately 0.5m.

Storage Area #2 has approximately 20,000 m^3 of volume within the floodplain area between the Millennium Trail and Belleville Street.

A third storage area within the Lane Creek watershed is located within Sub-Catchment 105 from the catchment drawing in Appendix A. This storage area is outside of the Lane Creek study area and is therefore not shown in the floodplain mapping. However, this wetland area provides storage within upper portion of the watershed. This storage area volume was conservatively not included in the hydrology or hydraulic computations.

3.4 Culvert and Bridge Crossings

The hydraulic model simulates culvert and weir flow at each crossing within the Lane Creek study area. This section summarizes the existing crossing configurations, stage-discharge curves, and the maximum 100-yr water surface elevations at each road crossing. Embedment depths at the culverts were included in the modelling and were based on site inspection and survey data. The purpose of this section is to address the impacts of the existing infrastructure on the overall floodplain delineation provided in Section 4.0. There are no designated flow control structures within the study area.

The table below summarizes the maximum depth in the regulatory event at each road crossing. It also summarizes the depth*velocity product and compares it to the maximum limit of 0.8 m^3/s per MTO's *Highway Drainage Design Standards*. The product limit is exceeded at Niles Street and should be considered in future replacement options. Similarly, the maximum recommended depth in MTO design standards is 0.3m. The depth limit is exceeded at Maple Street and Niles Street and should also be considered in future replacement options.

Table 3-1: Comparison of 100-Yr Maximum Crossing Depths and Velocities vs. Recommended Limits from MTO Design Standards

Crossing	100-Yr WSEL (m)	Road Sag (m)	Maximum		Product	Limit	
			Depth (m)	Velocity (m/s)	Depth*Velocity	Depth = 0.3m	D*V = 0.8 m^3/s
Belleville Street	85.55	85.35	0.20	0.42	0.08	✓	✓
Millennium Trail	84.37	84.22	0.15	0.81	0.12	✓	✓
Maple Street	83.22	82.84	0.38	0.72	0.27	x	✓
Lake Breeze Ct.	82.60	82.20	0.00	0.00	0.00	✓	✓
Niles Street	81.53	80.94	0.59	1.88	1.11	x	x
Main Street	79.99	79.75	0.24	0.50	0.12	✓	✓

3.4.1 Belleville Street

The Belleville Street crossing is the first road crossing within the Lane Creek study area and consists of two 1.6m diameter culverts (see Figure 3-4). A summary of the Belleville Street crossing is provided in Table 3-2. The current culvert crossing affects backwater conditions upstream of Belleville Street that contributes to the large storage area and wide flood hazard limits that extend onto existing agricultural lands. These lands may have an urban land use in the future (see Appendix B) and potential impacts due to the existing flood hazard should be considered prior to urban development in this area.

Figure 3-4: Image of Belleville Street Crossing Looking West

Table 3-2: Belleville Street Crossing Summary

No. of Culverts	Diameter (m)	Span (m)	Rise (m)
2	1.6	n/a	n/a
Left Culvert		Right Culvert	
Upstream Invert	Downstream Invert	Upstream Invert	Downstream Invert
83.49	83.41	83.44	83.35
Road Sag Elevation (m) = 85.35			
100-Yr Water Surface Elevation (m) = 85.55			
Maximum Relief Flow Depth (m)		Recommended Limit = 0.3 m	
0.2		✓	
Depth*Velocity Calculated (m^3/s)		Recommended Limit = 0.8 m^3/s	
0.08		✓	
$Q_{cap\ culvert}$ @ Road Sag (m^3/s)		Maximum Weir Flow (m^3/s)	
4.5		7.1	

**Culverts labelled left and right looking in the downstream direction.*

The schematic in Figure 3-5 illustrates the extents of the wide flood hazard limit upstream of Belleville Street. The blue shaded areas represent the flood depth, with darker shading corresponding to greater depths up to 2m. The lighter shaded areas represent shallower flow depths. The outline of Lane Creek can be seen in the schematic and particle tracing has been included in the image to show the general movement of water during the peak inundation period in the regulatory flood event.

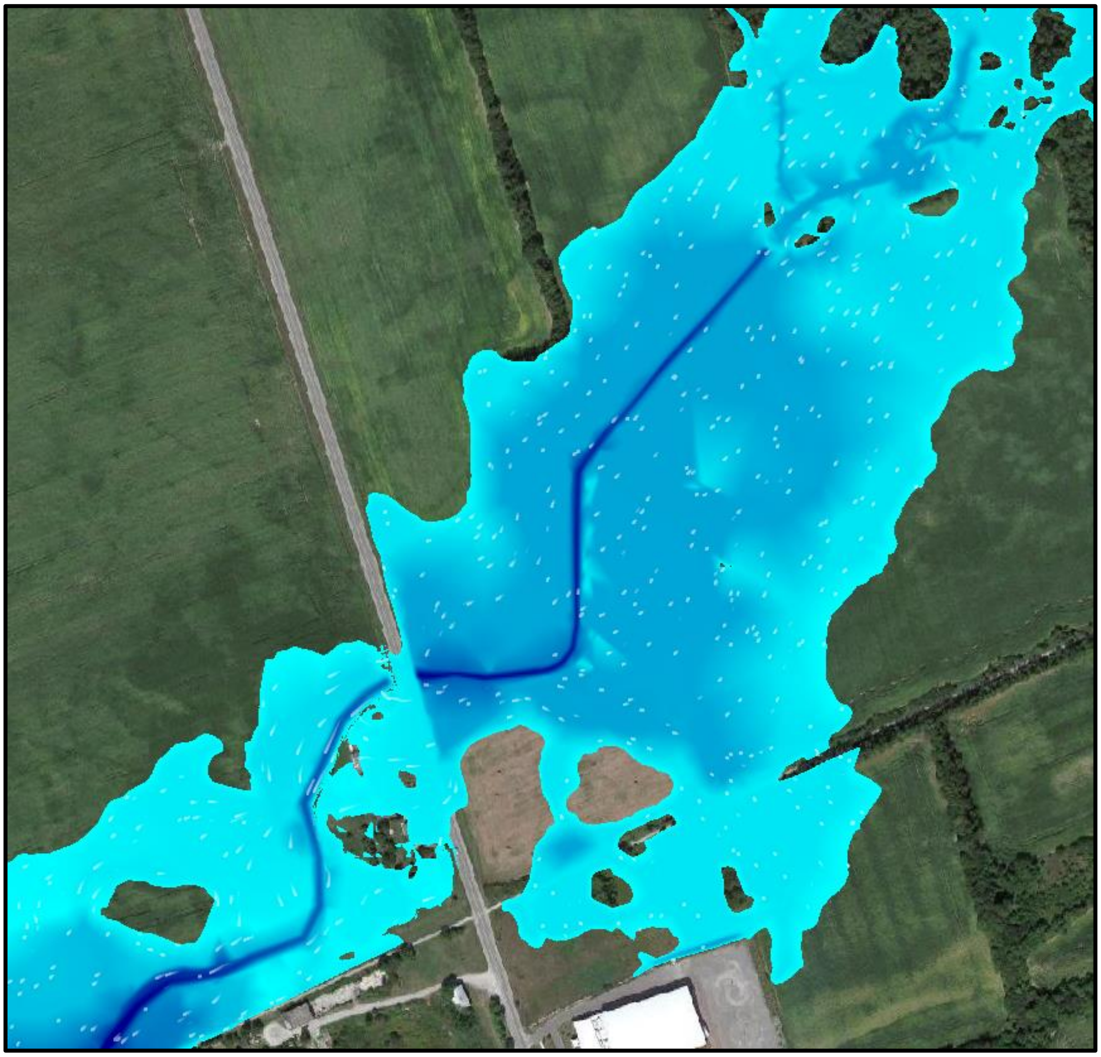


Figure 3-5: Schematic of Inundation Area Upstream of Belleville Street Crossing

The culvert and weir flow for the crossing was simulated using a 2D storage area connection in the hydraulic model. In the regulatory storm event, conveyance occurs through the culvert as well as overtop of the road. Flow over the road is simulated as weir flow, and the *2008 MTO Highway Drainage Design Standards* recommends limiting the depth of overtopping to 0.3m during the 100-yr return period event. The intent is to ensure access is maintained for emergency service vehicles during the regulatory storm event.

The stage-discharge relationship in Figure 3-6 summarizes the culvert and weir flow as well as the maximum headwater elevation at the crossing. The maximum headwater elevation is 85.55m and presents an overtopping depth of approximately 0.20m in the 100-yr storm. A comparison of maximum 100-yr water surface elevation (WSEL) to the Belleville Street road profile is provided in Figure 3-7.

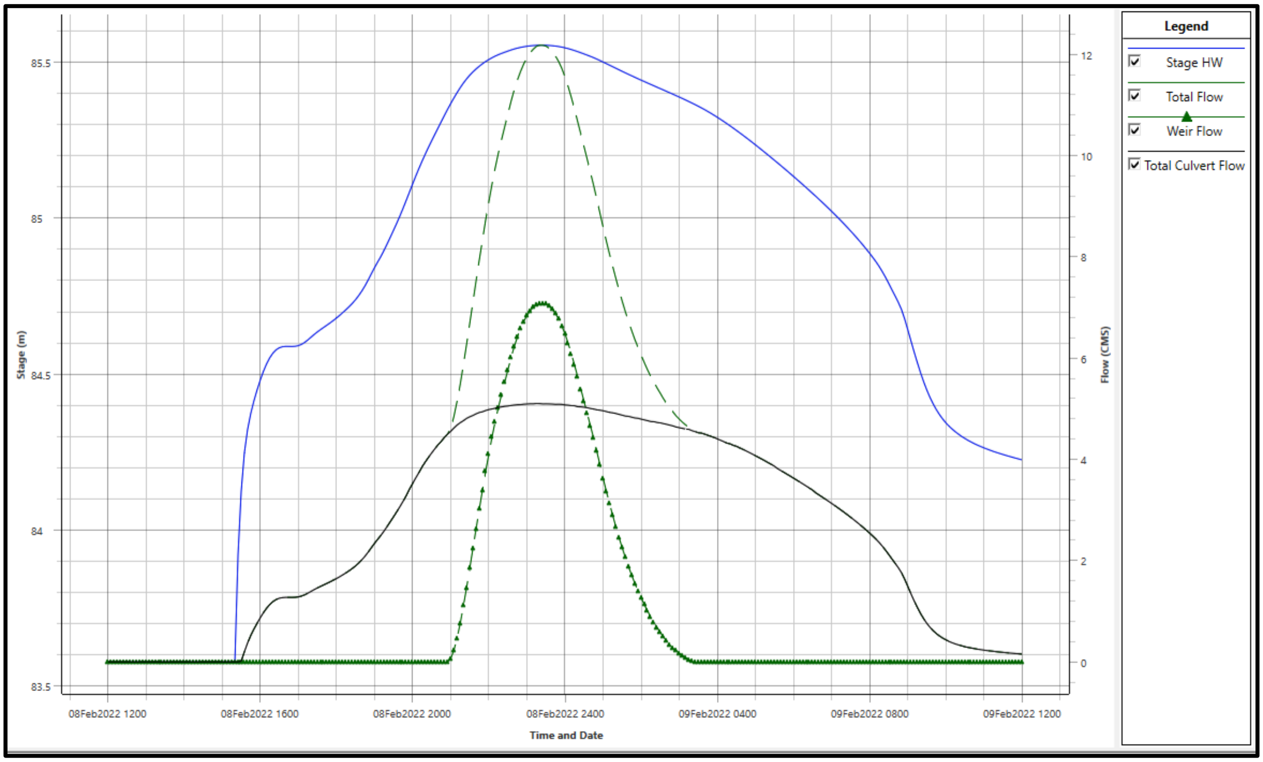


Figure 3-6: Stage-Discharge Relationship for Belleville Street Crossing

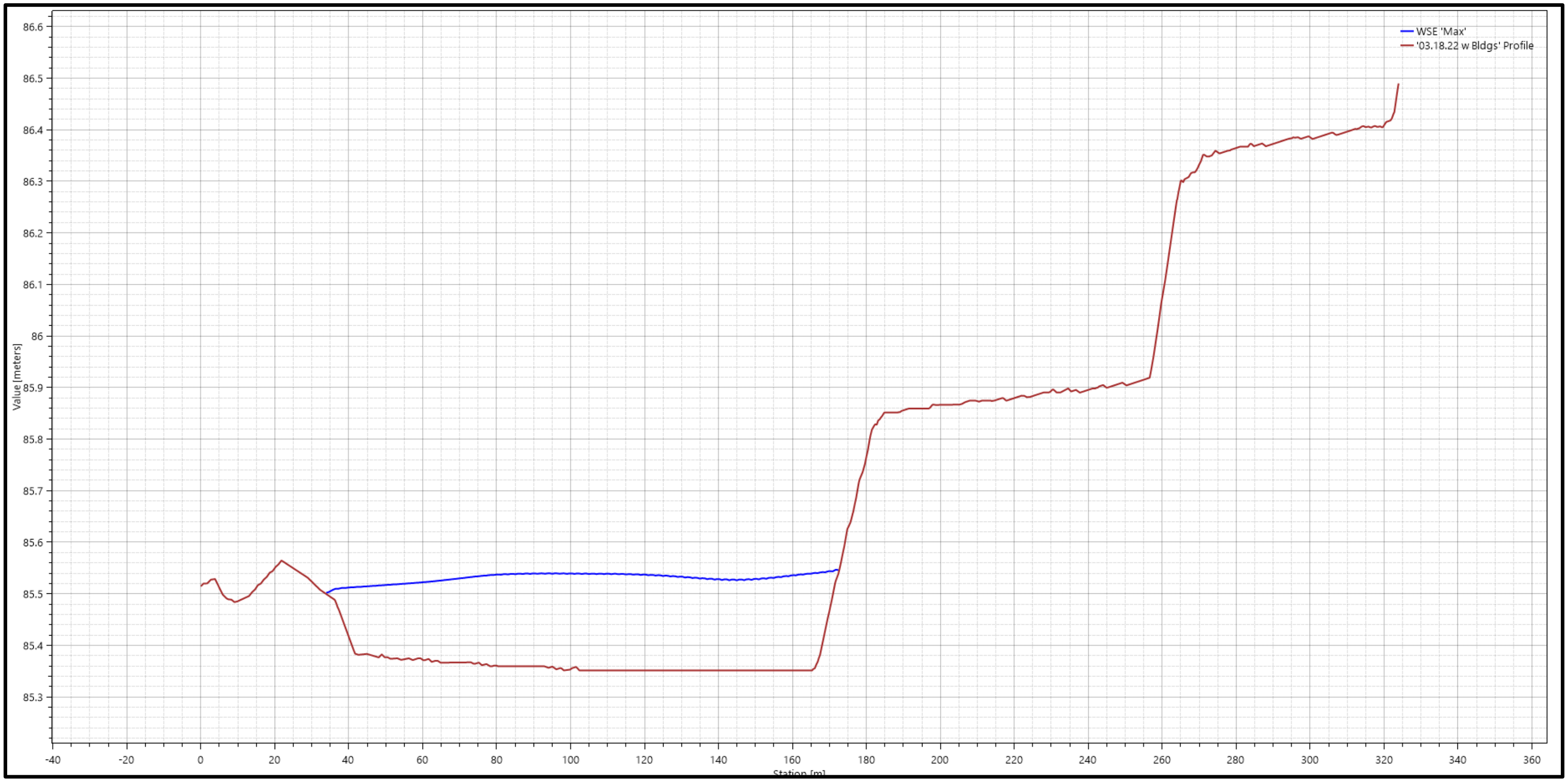


Figure 3-7: Comparison of Maximum 100-Yr WSEL to Existing Belleville Street Road Profile

3.4.2 Millennium Trail

The Millennium Trail crossing is the next crossing following Belleville Street and creates the other significant storage area within the Lane Creek study area (Storage Area #2). Similar to Belleville Street, the current culvert crossing affects backwater conditions upstream and contributes to the storage area between Belleville Street and the Millennium Trail that extends onto existing agricultural lands. These lands are also expected to have an urban land use in the future (see Appendix B) and potential impacts due to the existing flood hazard will need to be considered prior to development of these lands.

The schematic in Figure 3-8 illustrates the extents of this storage area and a crossing summary is provided in Table 3-3. In aerial imagery, there appears to be a low-lying area northwest of the crossing that has been susceptible to stagnant water in past years. This is consistent with the topographic data as well as the flood extents shown in Figure 3-8.

The stage-discharge relationship in Figure 3-9 summarizes the culvert and weir flow as well as the maximum headwater elevation at the crossing. This figure shows that there is minimal weir flow and overtopping of the Millennium Trail since the culvert conveys almost the entire contributing flow.

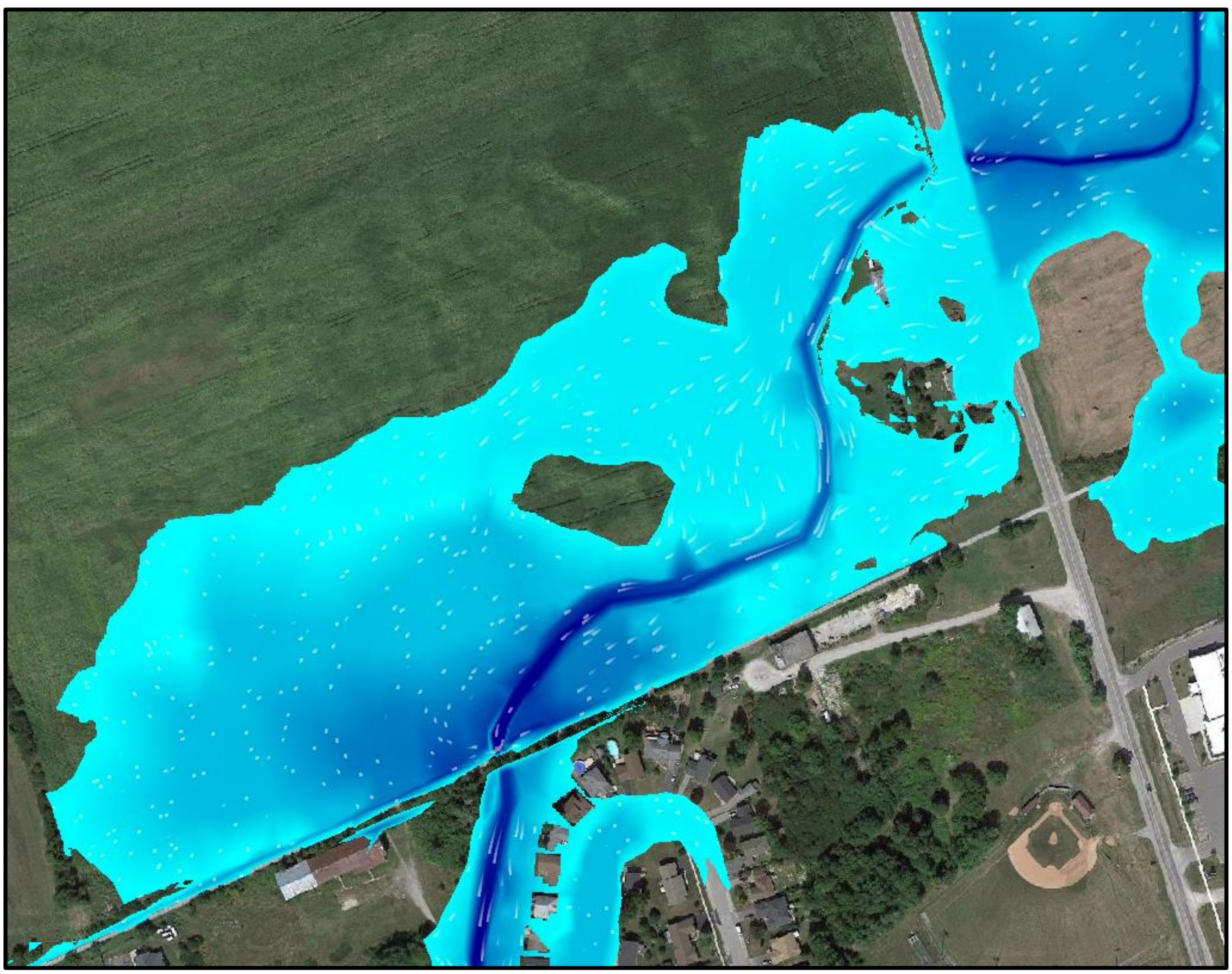


Figure 3-8: Schematic of Inundation Area Upstream of the Millennium Trail

Table 3-3: Millennium Trail Crossing Summary

No. of Culverts	Diameter (m)	Span (m)	Rise (m)
1	n/a	3.5	2
Upstream Invert		Downstream Invert	
82.15		82.0	
Road Sag Elevation (m) = 84.27			
100-Yr Water Surface Elevation (m) = 84.36			
Maximum Relief Flow Depth (m)		Recommended Limit = 0.3 m	
0.15		✓	
Depth*Velocity Calculated (m^3/s)		Recommended Limit = 0.8 m^3/s	
0.12		✓	
$Q_{cap\ culvert}$ @ Road Sag (m^3/s)		Maximum Weir Flow (m^3/s)	
9.58		0.15	

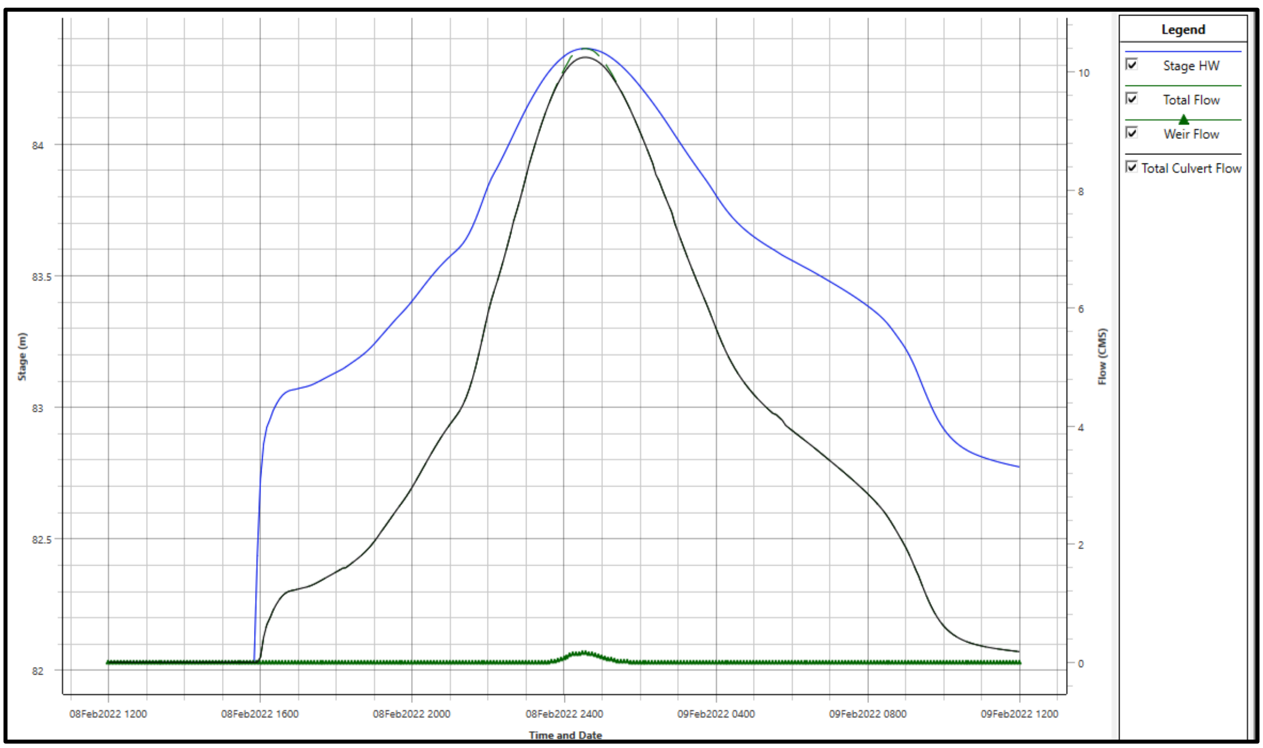


Figure 3-9: Stage-Discharge Relationship for Millennium Trail Crossing

3.4.3 Maple Avenue

The Maple Avenue crossing is approximately 130m downstream of the Millennium Trail and has a concrete rectangular opening with a 3.5m span by 1.5 rise (see Figure 3-10 and Table 3-4). There are existing homes located east of Lane Creek, along Westwind Crescent, that are located within the 100-yr floodplain (see Figure 3-11). In the 100-yr storm overtopping would occur south of the crossing and then ultimately drain back into Lane Creek. A comparison of maximum 100-yr WSEL to the Maple Avenue road profile is provided in Figure 3-13. This figure shows a maximum of 0.38m depth of overtopping of the roadway.

Figure 3-10: Image of Maple Avenue Crossing Looking West

Table 3-4: Maple Avenue Crossing Summary

No. of Culverts	Diameter (m)	Span (m)	Rise (m)
1	n/a	3.5	1.5
Upstream Invert		Downstream Invert	
81.24		81.2	
Road Sag Elevation (m) = 82.84			
100-Yr Water Surface Elevation (m) = 83.22			
Maximum Relief Flow Depth (m)		Recommended Limit = 0.3 m	
0.38		✗	
Depth*Velocity Calculated (m^3/s)		Recommended Limit = 0.8 m^3/s	
0.27		✓	
$Q_{cap\ culvert}$ @ Road Sag (m^3/s)		Maximum Weir Flow (m^3/s)	
5.07		4.79	

Figure 3-11: Schematic of Inundation Area Upstream of Maple Avenue

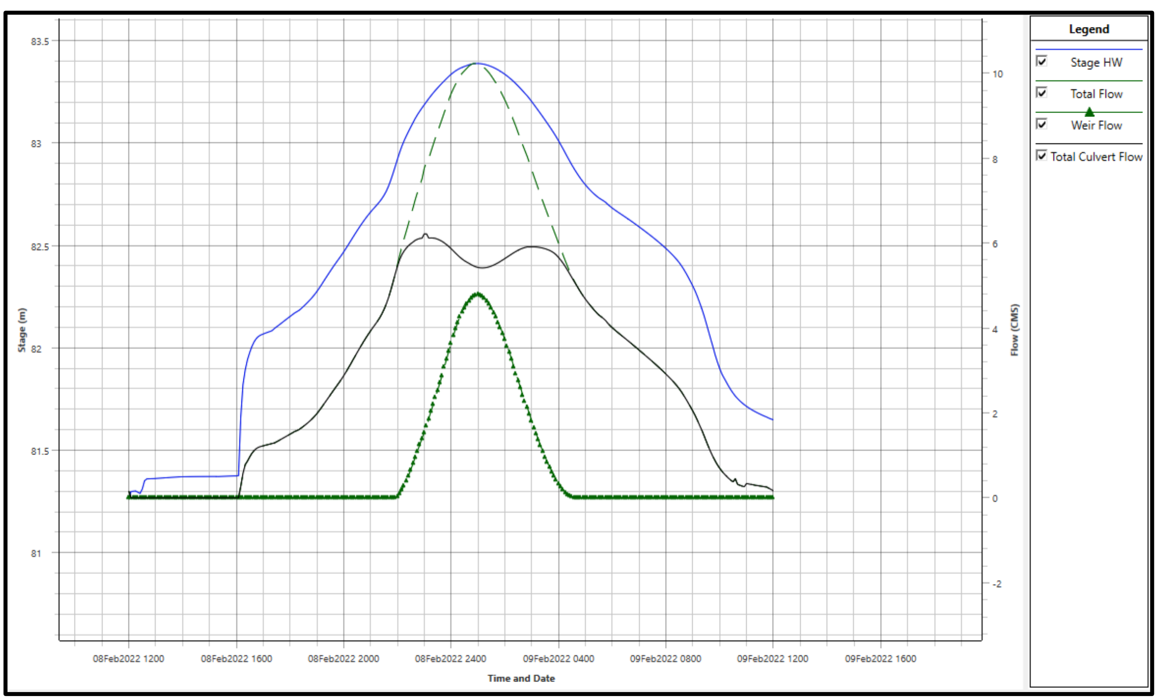


Figure 3-12: Stage-Discharge Relationship for Maple Avenue Crossing

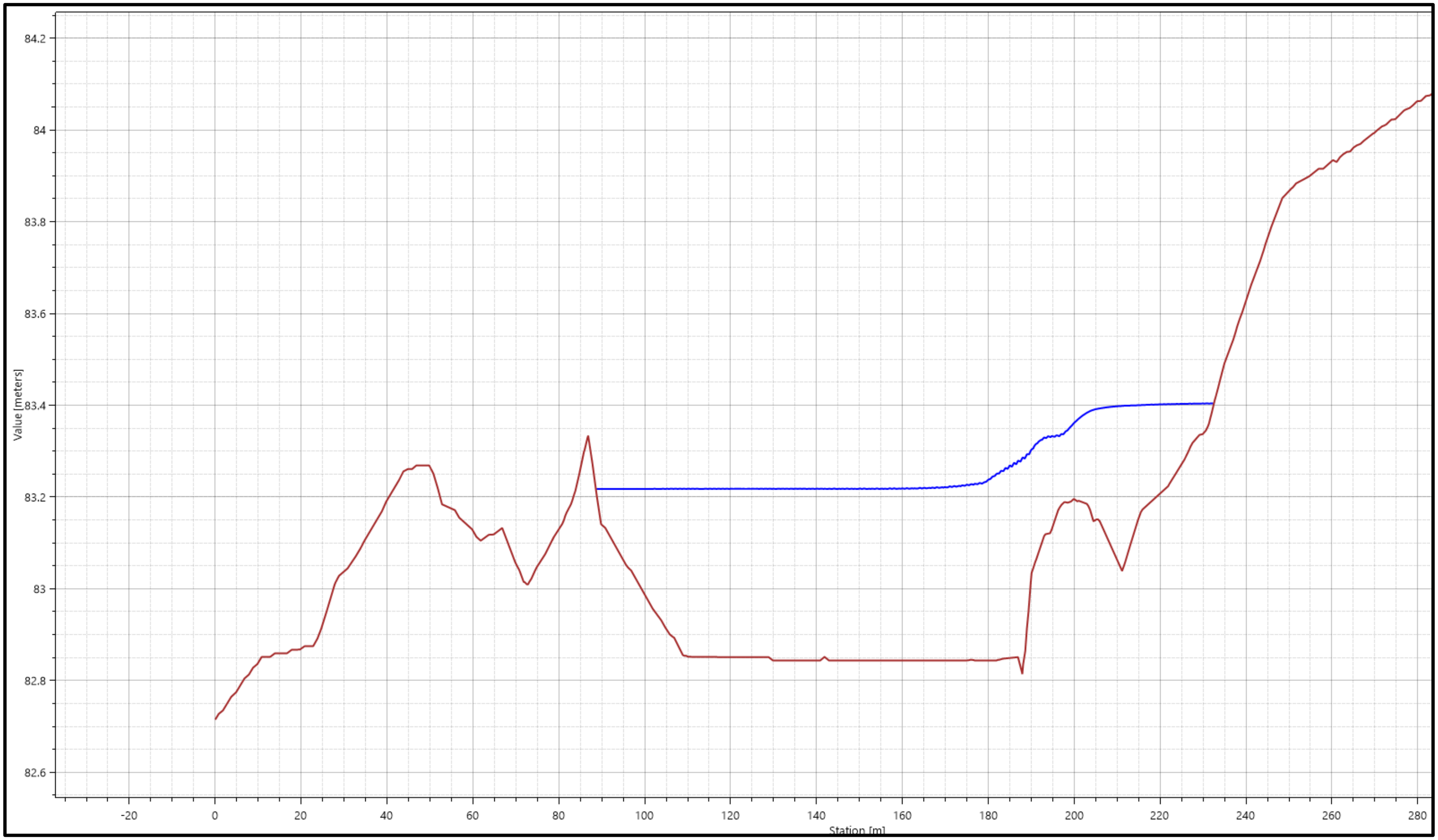


Figure 3-13: Comparison of Maximum 100-Yr WSEL to Existing Maple Avenue Road Profile

3.4.4 Lake Breeze Court

The Lake Breeze Court crossing is located immediately west of the former Wellington community centre. It is comprised of two, 2.75m span x 1.8m rise CSP arch culverts (see Figure 3-14 and Table 3-5). There is minimal ponding upstream of these culverts, and Figure 3-15 shows that the floodplain extents are limited to the channel dimensions immediately upstream of the culvert. It is expected that this is due to the large culvert sizes in combination with channelization of the creek in this location that would have occurred to accommodate the Wellington Legion Manor at 68 Maple Street. The historical floodplain map from 1979 shows the creek used to traverse the property at 68 Maple Street, and that a re-alignment of the channel has been completed since the 1979 study.

The stage-discharge relationship in Figure 3-16 shows the culverts convey the entire 100-yr contributing flow and that no road overtopping is expected along Lake Breeze Court in the 100-yr event.

Figure 3-14: Image of Lake Breeze Court Culverts Looking West

Table 3-5: Lake Breeze Court Crossing Summary

No. of Culverts	Diameter (m)	Span (m)	Rise (m)
2	n/a	2.75	1.8
Left Culvert		Right Culvert	
Upstream Invert	Downstream Invert	Upstream Invert	Downstream Invert
80.40	80.25	80.40	80.25
Road Sag Elevation (m) = 82.20			
100-Yr Water Surface Elevation (m) = 82.0			
Maximum Relief Flow Depth (m)		Recommended Limit = 0.3 m	
0		✓	
Depth*Velocity Calculated (m^3/s)		Recommended Limit = 0.8 m^3/s	
0		✓	
$Q_{cap\ culvert}$ @ Road Sag (m^3/s)		Maximum Weir Flow (m^3/s)	
10.18		n/a	

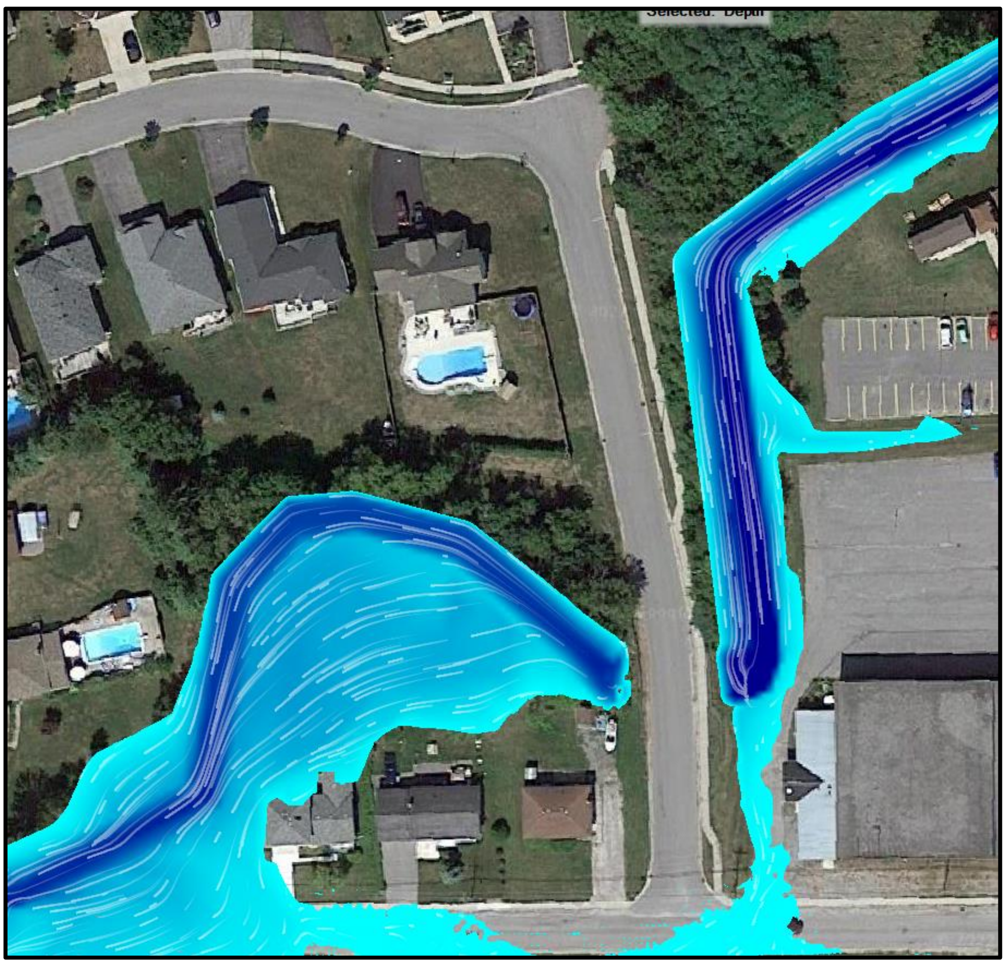


Figure 3-15: Schematic of Inundation Area Upstream of Lake Breeze Court

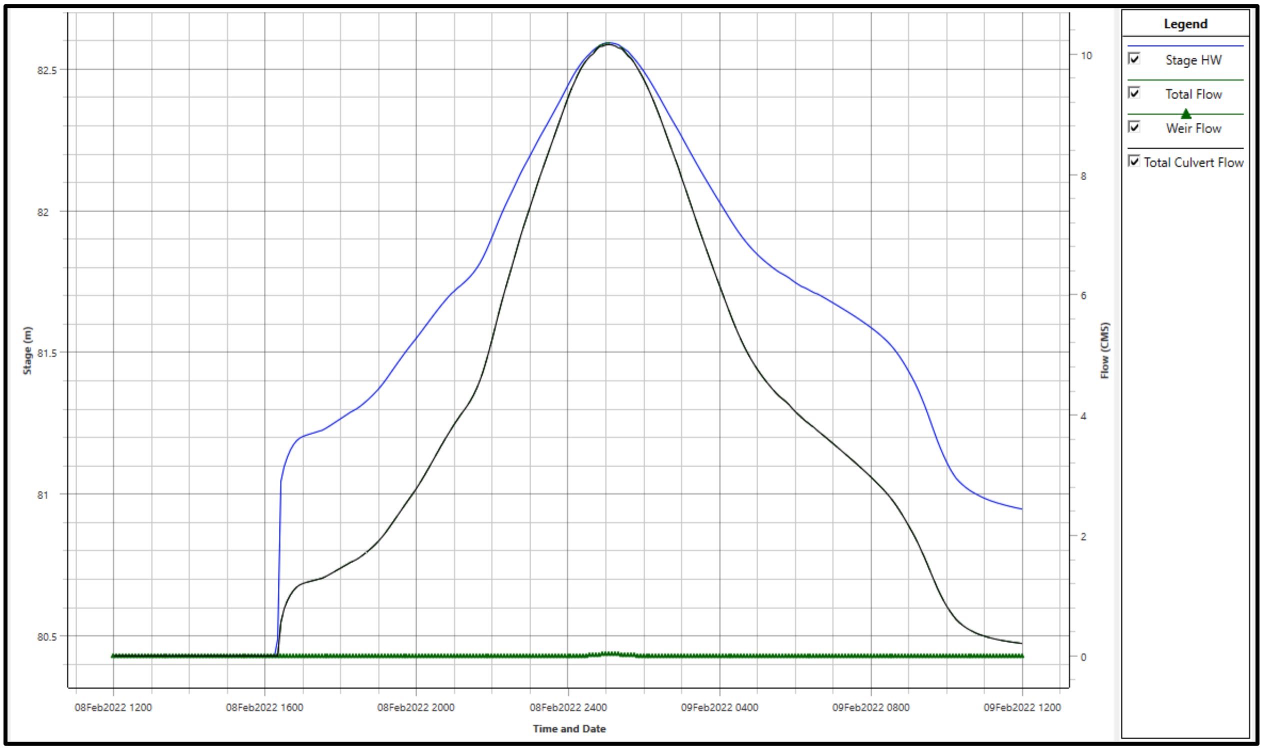


Figure 3-16: Stage-Discharge Relationship for Lake Breeze Court Culvert Crossing

3.4.5 Niles Street

After Lake Breeze Court, Lane Creek drains through an existing 2.6m span by 1m rise concrete rectangular opening at Niles Street (see Table 3-6). There is a low-lying area adjacent to Lane Creek northeast of the Niles Street crossing. As a result, the flood limit extends towards an existing house along Niles Street and there is a spill over the road just east of the existing concrete structure (see Figure 3-17). A comparison of maximum 100-yr WSEL to the Niles Street road profile is provided in Figure 3-19. This figure shows a maximum of 0.59m depth of overtopping of the roadway.

Table 3-6: Niles Street Crossing Summary

No. of Culverts	Diameter (m)	Span (m)	Rise (m)
1	n/a	2.6	1
Upstream Invert		Downstream Invert	
79.61		79.6	
Road Sag Elevation (m) = 80.94			
100-Yr Water Surface Elevation (m) = 81.53			
Maximum Relief Flow Depth (m)		Recommended Limit = 0.3 m	
0.59		x	
Depth*Velocity Calculated (m^3/s)		Recommended Limit = 0.8 m^3/s	
1.11		x	
$Q_{cap\ culvert}$ @ Road Sag (m^3/s)		Maximum Weir Flow (m^3/s)	
3.05		7.35	

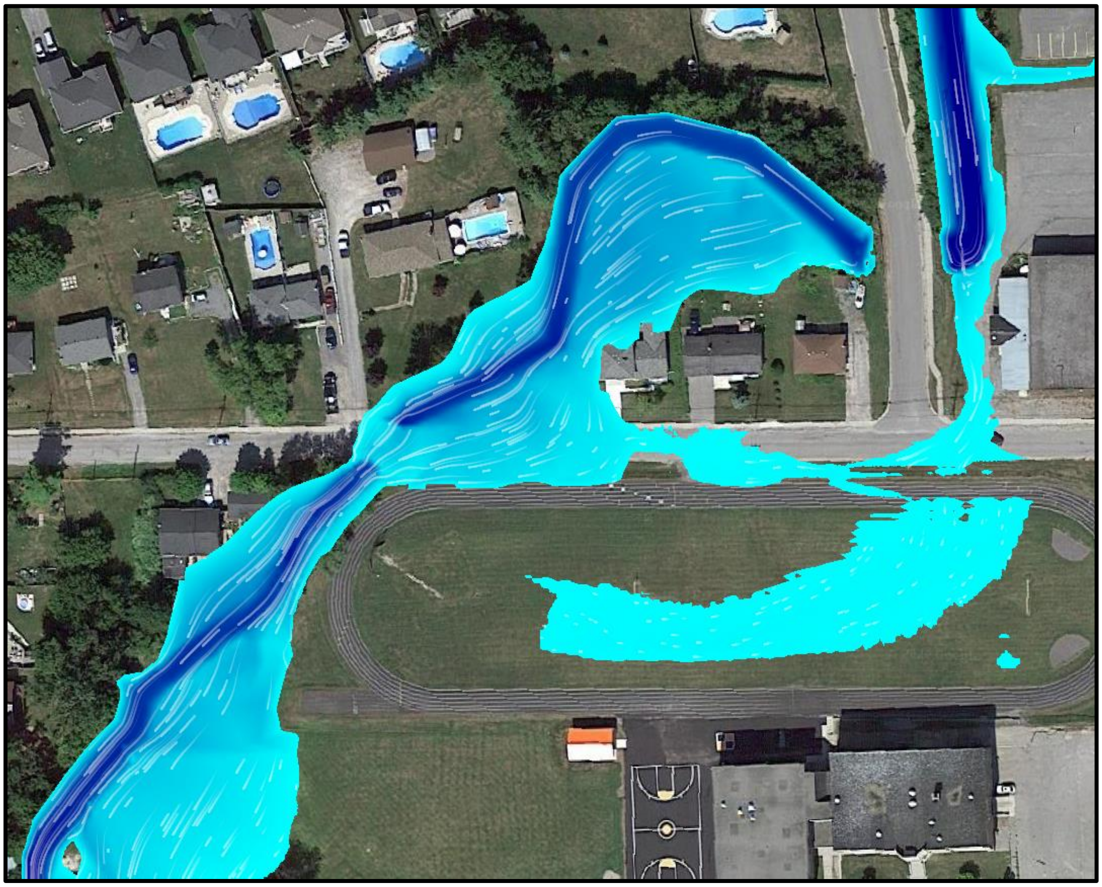


Figure 3-17: Schematic of Inundation Area Upstream of Niles Street

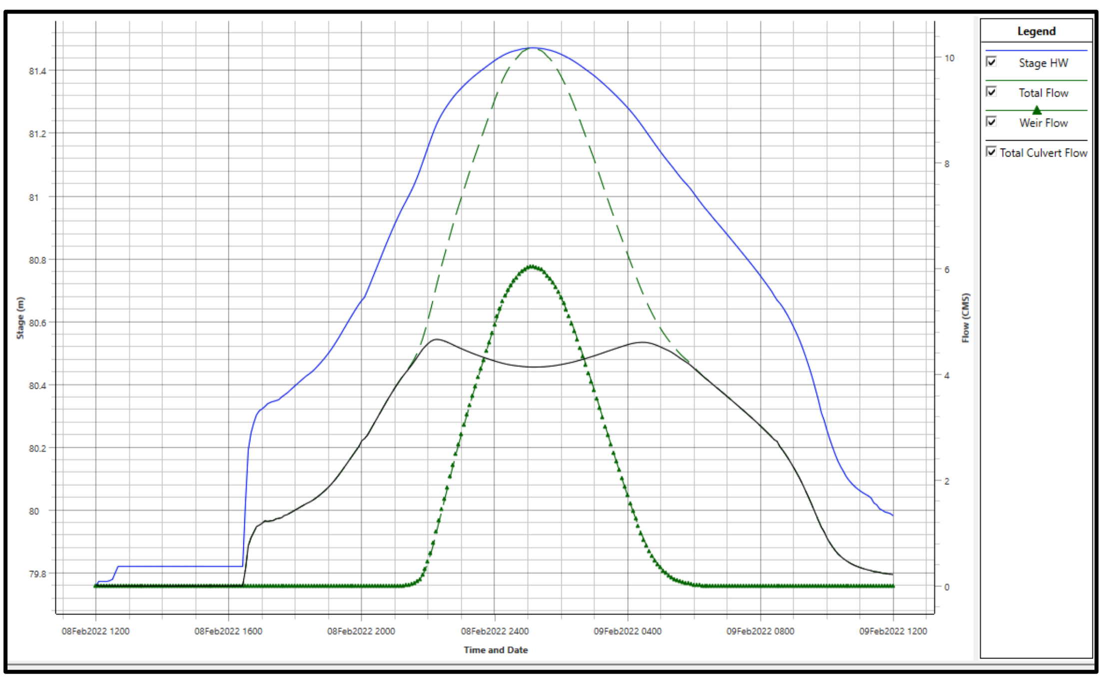


Figure 3-18: Stage-Discharge Relationship for Niles Street Crossing

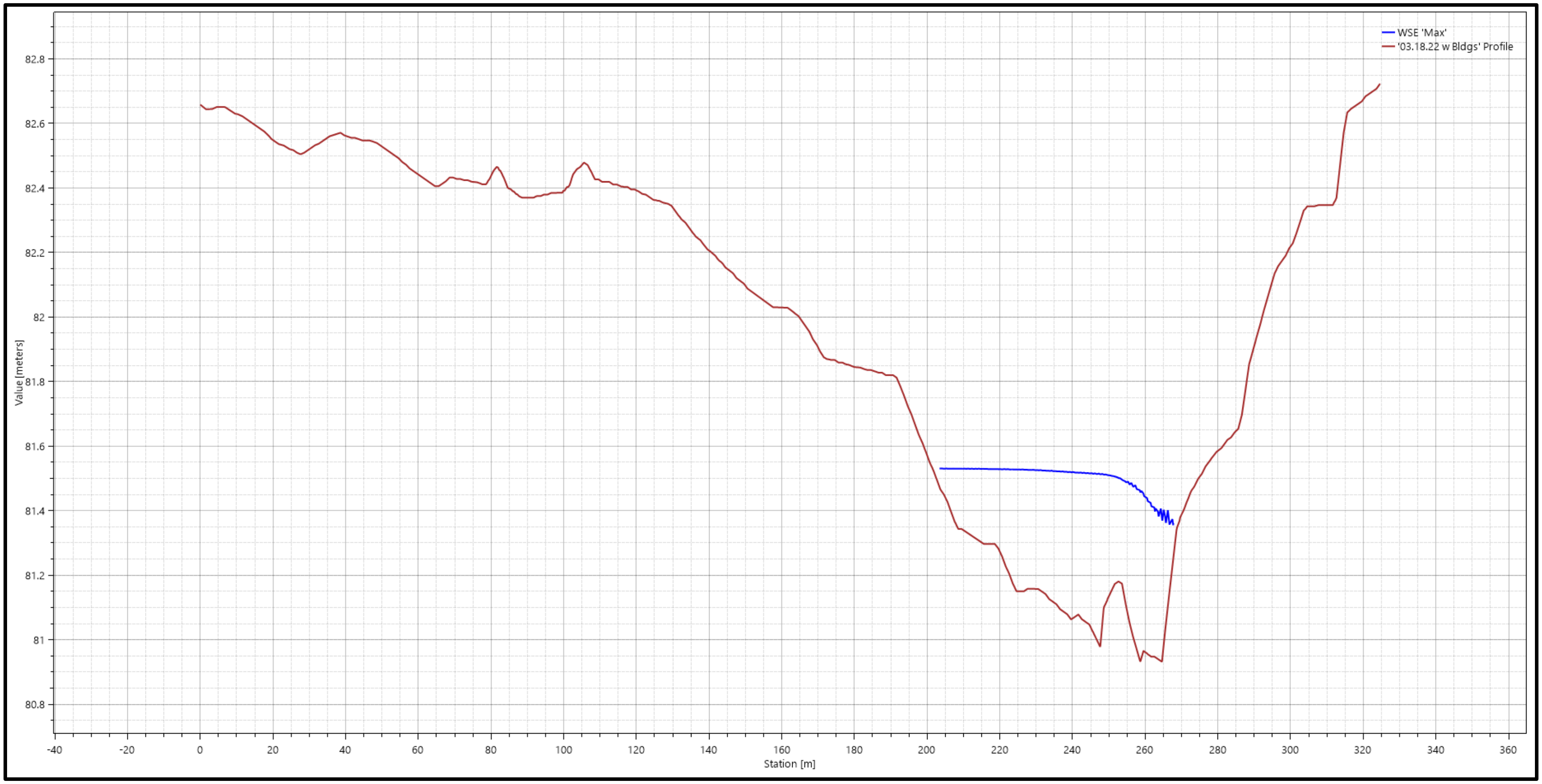


Figure 3-19: Comparison of Maximum 100-Yr WSEL to Existing Niles Street Road Profile

3.4.6 Drake Motor Inn

Lane Creek drains south of Niles Street towards a concrete rectangular opening that drains under an existing building (see Figure 3-20). The concrete structure has a 3.6m span x 1.1m rise.

The schematic in Figure 3-21 shows that the existing building intersects the creek and creates a flow obstruction that causes flows exceeding the capacity of the culvert to drain east and then south through the Midtown Brewery parking area. For the surplus flows that drain to this parking area, a portion drains back into Lane Creek, while the remaining flows continue southwards where it overtops Main Street as described in the next sub-section.

Figure 3-20: Image of Concrete Culvert Under Existing Drake Motor Inn Building Looking North

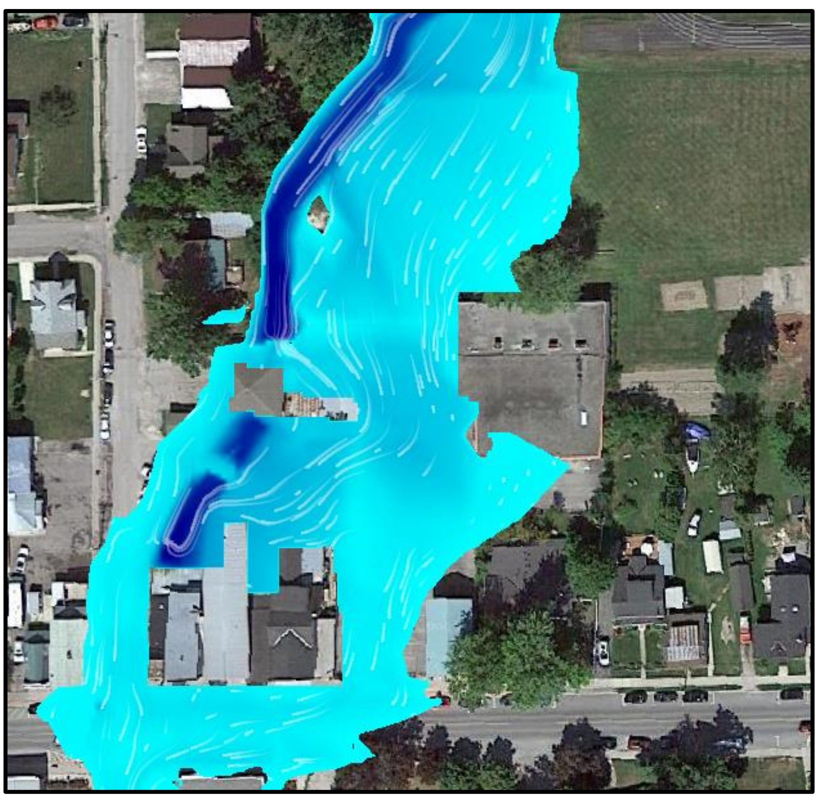


Figure 3-21: Schematic of Inundation Area NE of the Intersection of Main Street and Wharf Street

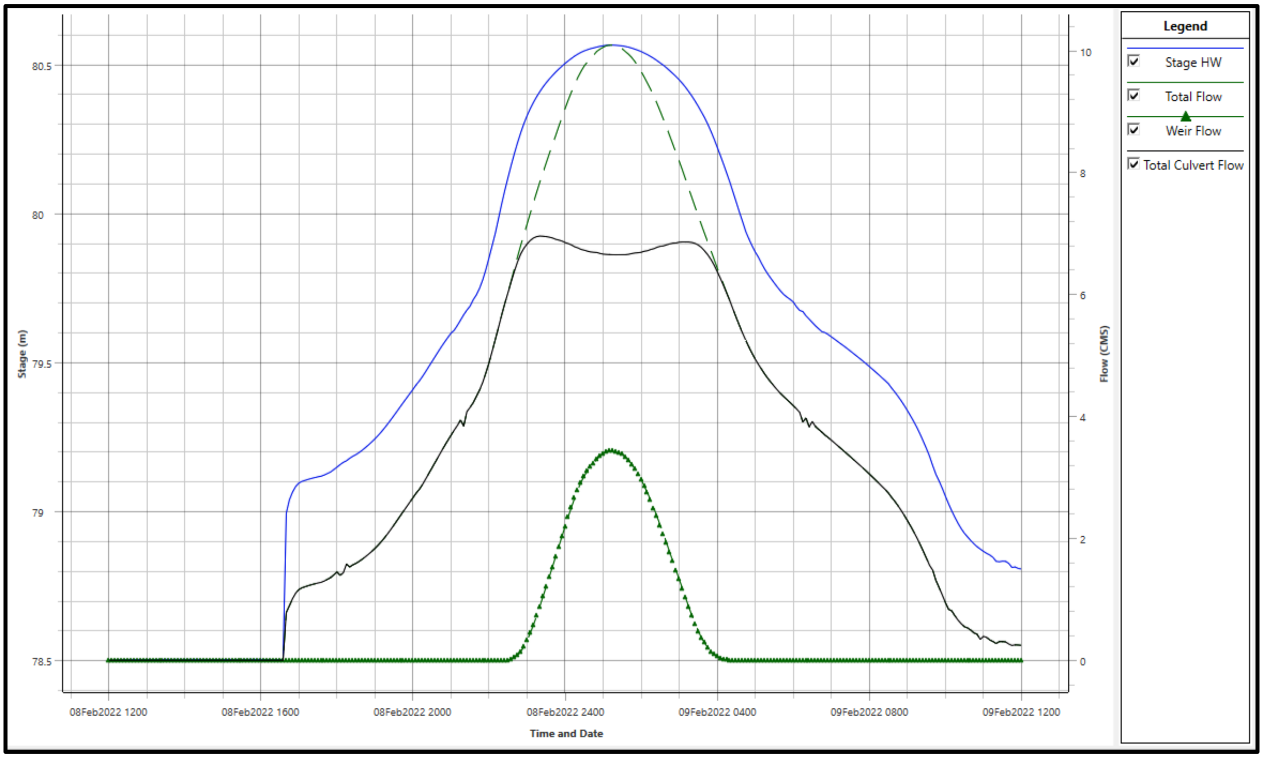


Figure 3-22: Stage-Discharge Relationship for Drake Motor Inn Crossing

3.4.7 Main Street

The Main Street crossing is the final road crossing within the Lane Creek study area and consists of a long, 2.6m span by 1.5m rise concrete structure that flows under the intersection of Main Street and Wharf Street (see Figure 3-23). A crossing summary is provided in Table 3-7.

The schematic in Figure 3-24 illustrates the various flow paths due to the unique building and culvert configurations in the vicinity of this road intersection. The stage-discharge relationship in Figure 3-25 summarizes the culvert and weir flow as well as the maximum headwater elevation at the crossing. The maximum depth of road overtopping is 0.24m, with flows spilling over the road following the roadway southward along Wharf Street, ultimately draining to Lake Ontario at the downstream limit of Lane Creek. A comparison of maximum 100-yr WSEL to the Main Street road profile is provided in Figure 3-26.

Figure 3-23: Image of Main Street Concrete Structure Looking South

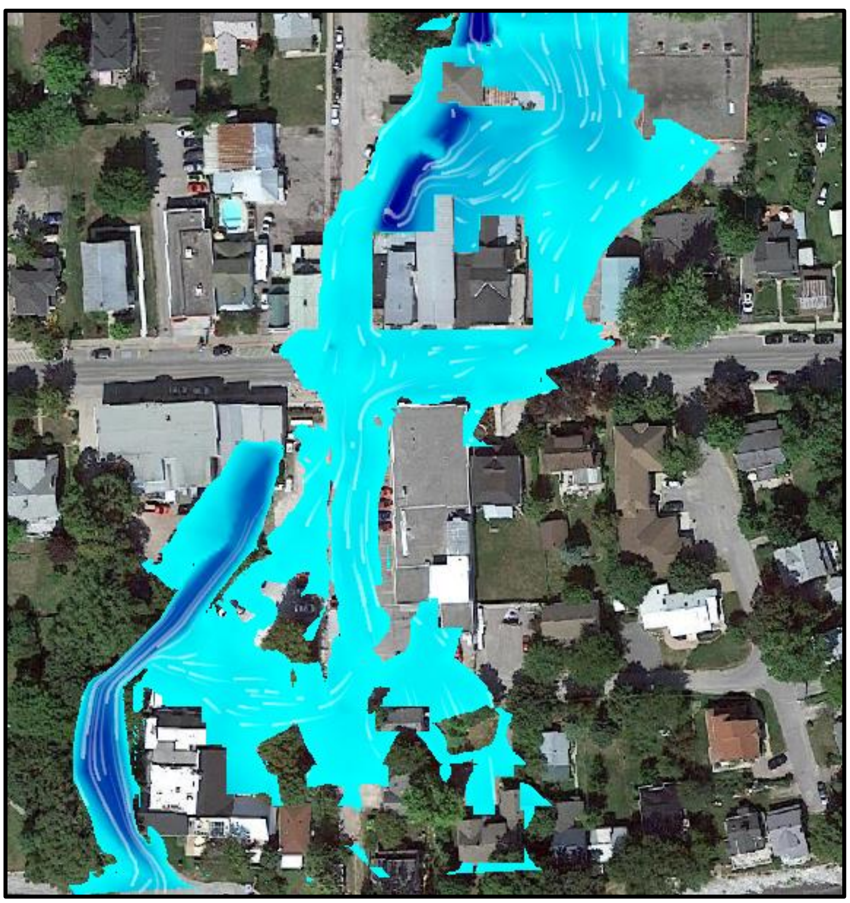


Figure 3-24: Schematic of Inundation Area near Intersection of Main Street and Wharf Street

Table 3-7: Main Street Crossing Summary

No. of Culverts	Diameter (m)	Span (m)	Rise (m)
1	n/a	2.65	1.5
Upstream Invert		Downstream Invert	
78.15		77.85	
Road Sag Elevation (m) = 79.75			
100-Yr Water Surface Elevation (m) = 79.99			
Maximum Relief Flow Depth (m)		Recommended Limit = 0.3 m	
0.24		✓	
Depth*Velocity Calculated (m^3/s)		Recommended Limit = 0.8 m^3/s	
0.12		✓	
$Q_{cap\ culvert}$ @ Road Sag (m^3/s)		Maximum Weir Flow (m^3/s)	
7.05		0.86	

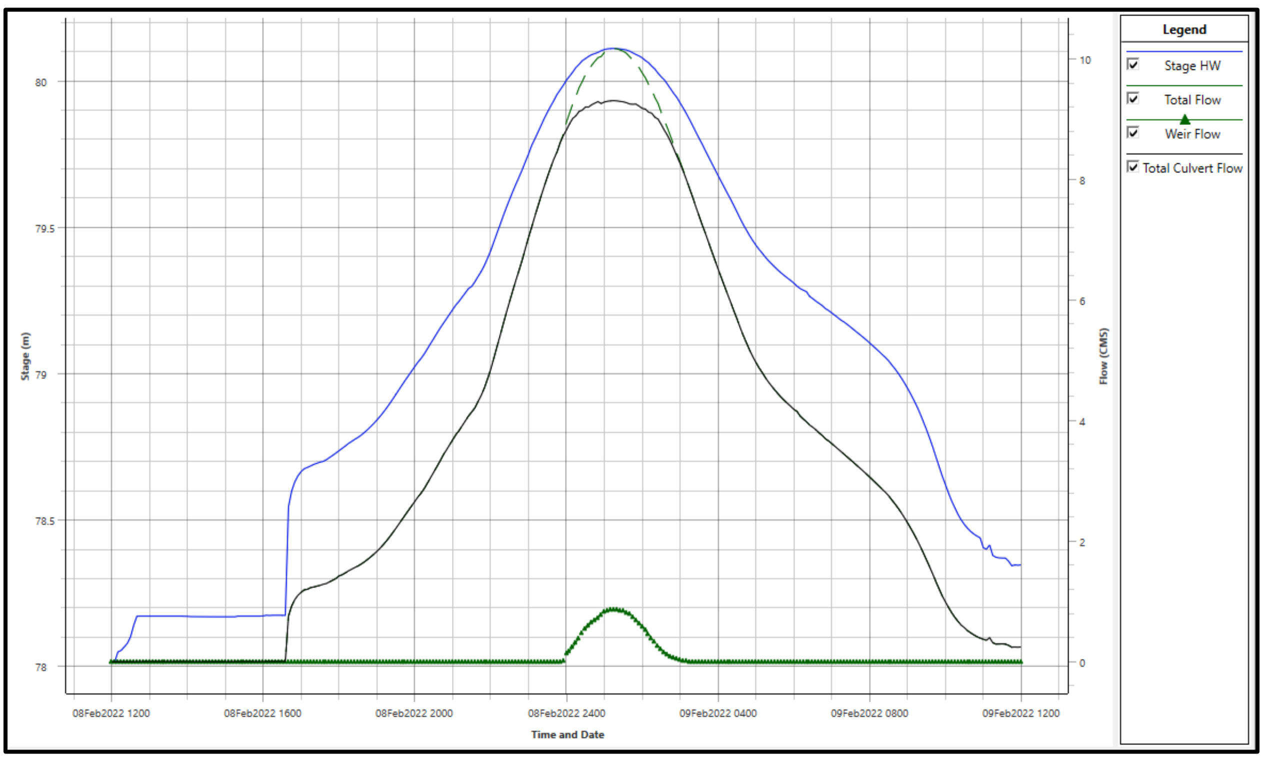


Figure 3-25: Stage-Discharge Relationship for Main Street Crossing

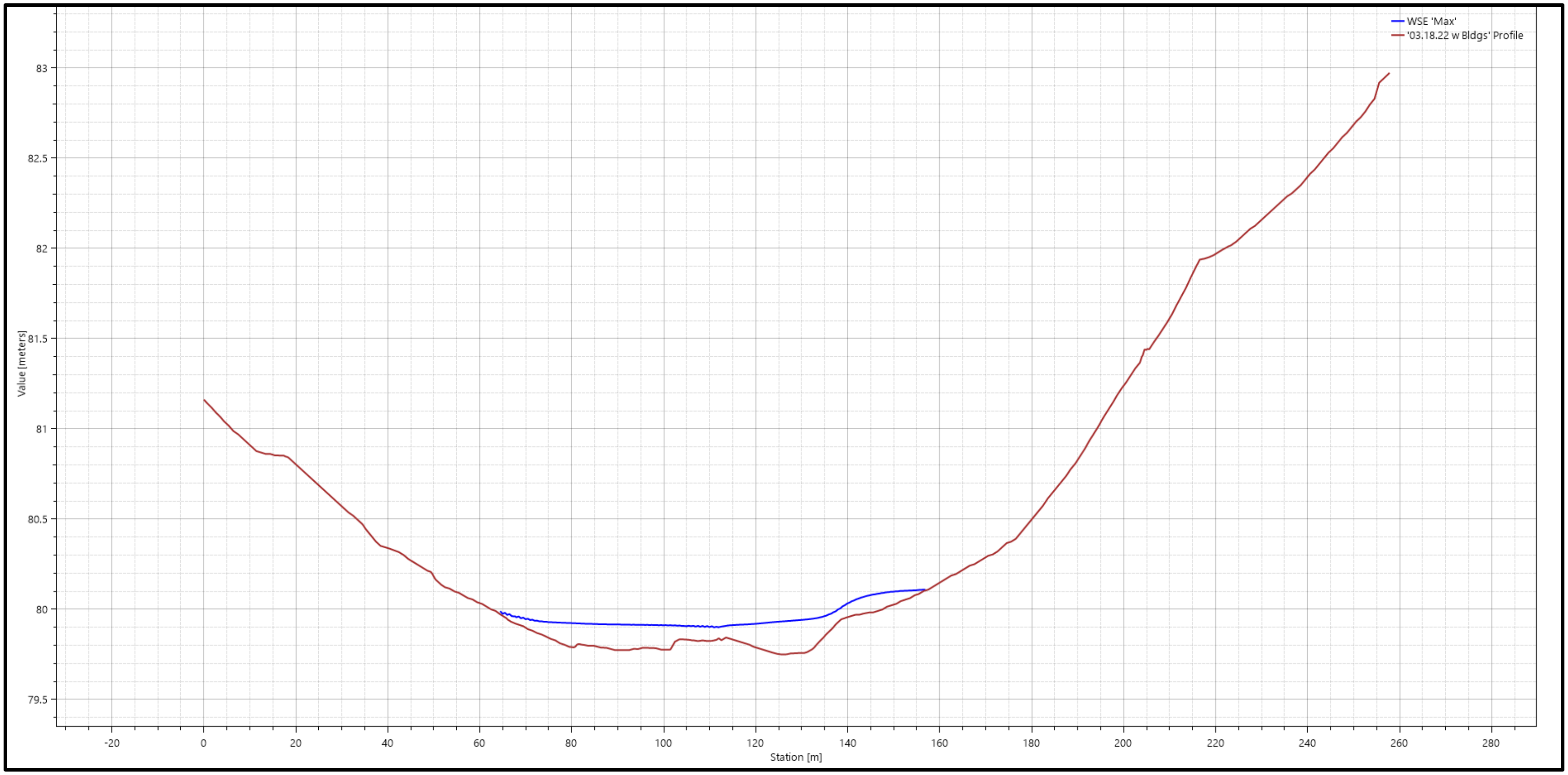


Figure 3-26: Comparison of Maximum 100-Yr WSEL to Existing Main Street Road Profile

3.5 Sensitivity Analysis

A sensitivity analysis is completed to assess potential impacts due to uncertainties in modelling parameters on the resulting flood limits. The sensitivity analysis was performed by varying the values of a specific modeling parameter while keeping the other variables held constant.

The hydrology and hydraulic modelling results are sensitive to the following information:

- Terrain data and drainage area
- Curve number
- Peak flood discharge
- Manning's roughness values

A discussion of model sensitivity to each parameter is provided below.

3.5.1 Terrain Data and Drainage Area

Section 3.1 identified the detailed topographic data applied in the geometry component of the hydraulic modelling computations. The model geometry is a critical component of the hydraulic model, since it controls the direction of flow, the water surface elevations, channel and overbank flow, and the backwater effects from infrastructure. The thorough investigation of ground surface elevations using project specific LiDAR combined with creek and bridge/culvert surveys prepared by Jewell using GPS and a total station provides a high confidence level in terrain data applied in the hydraulic model. A sensitivity analysis of the model to terrain was not required due to this high confidence level and accuracy with the topographic instruments used for the ground surface data. Similarly, a sensitivity analysis was not required for the drainage area since it is a product of the topographic information. The drainage area was also conservatively estimated by including the low-lying wetland area that receives runoff from Sub-Catchment 105 in the northernmost portion of the watershed.

3.5.2 Curve Number

The soils data is used to develop curve numbers (CNs) that are a key modelling parameter used in the Soil Conservation Service (now known as the *National Resources Conservation Service*) methodology for estimating the proportion of precipitation that will runoff the lands and the portion that will infiltrate. Design charts from the MTO Drainage Manual are commonly applied for selecting curve numbers. These design charts recommend CN values based on the hydrologic soils group classification and land cover. The CN values are relatively generic and are best relied upon when no field-related data is available.

The sensitivity analysis for the CN was completed to determine the impact this value has on peak flows.

Figure 3-27 shows a comparison of CN values to the resulting peak flows from the HMS model. As expected, there is a strong correlation between CN and peak flows values. With a 7% increase in CN, there is approximately a 33% increase in peak flow. Similarly, for a 7% decrease in CN, there is approximately a 30% reduction in peak flow.

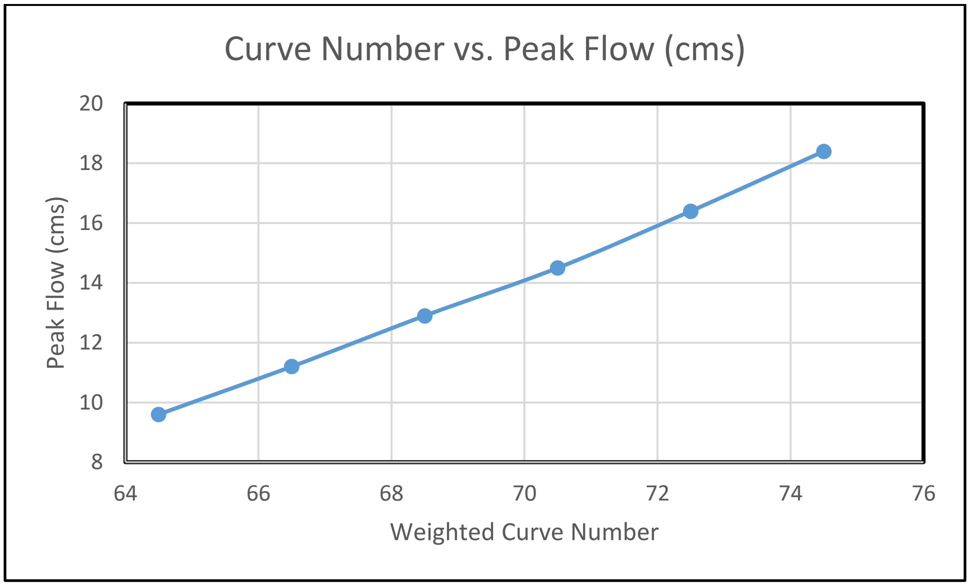


Figure 3-27: Weighted Curve Number vs. Peak Flow

The following sub-section takes this a step further by investigation of the sensitivity of the floodplain delineation to increased or decreased inflow hydrograph values from the hydrology model.

3.5.3 Peak Flood Discharge

The maximum and minimum peak flows from Figure 3-27 were supplied to the HEC-RAS model to determine the sensitivity of the floodplain delineation. Figure 3-28 illustrates the following three flood limits:

Green: minimum HEC-HMS peak flow of 9.6 m^3/s

Blue: selected HEC-HMS peak flow of 14.0 m^3/s

Red: maximum HEC-HMS peak flow of 18.4 m^3/s

This figure shows that the flood line has minimal change within this peak flow range for the flood extents upstream of Belleville Street. There are notable differences in the west flood extents between the Millennium Trail and Belleville Street due to the existing low-lying area. In peak flood discharge well beyond the regulatory peak flow of 14.0 m^3/s, the red line shows a spill would occur east of Lane Creek towards Maple Street.

In the lower peak discharge that is more similar to the findings from the general frequency analysis, the floodplain is noticeably narrowed towards the creek banks between Maple Avenue through to the Lane Creek outlet to Lake Ontario.

The results show that the floodplain delineation is dependent on the selected peak flows. Given the conservatism and level of detail applied in the hydrology analysis, the peak flow of 14.0 m^3/s was selected. Any higher peak flow would be significantly greater than the general frequency results and would expand the flood limits beyond what would reasonably be expected for the 100-yr storm event.

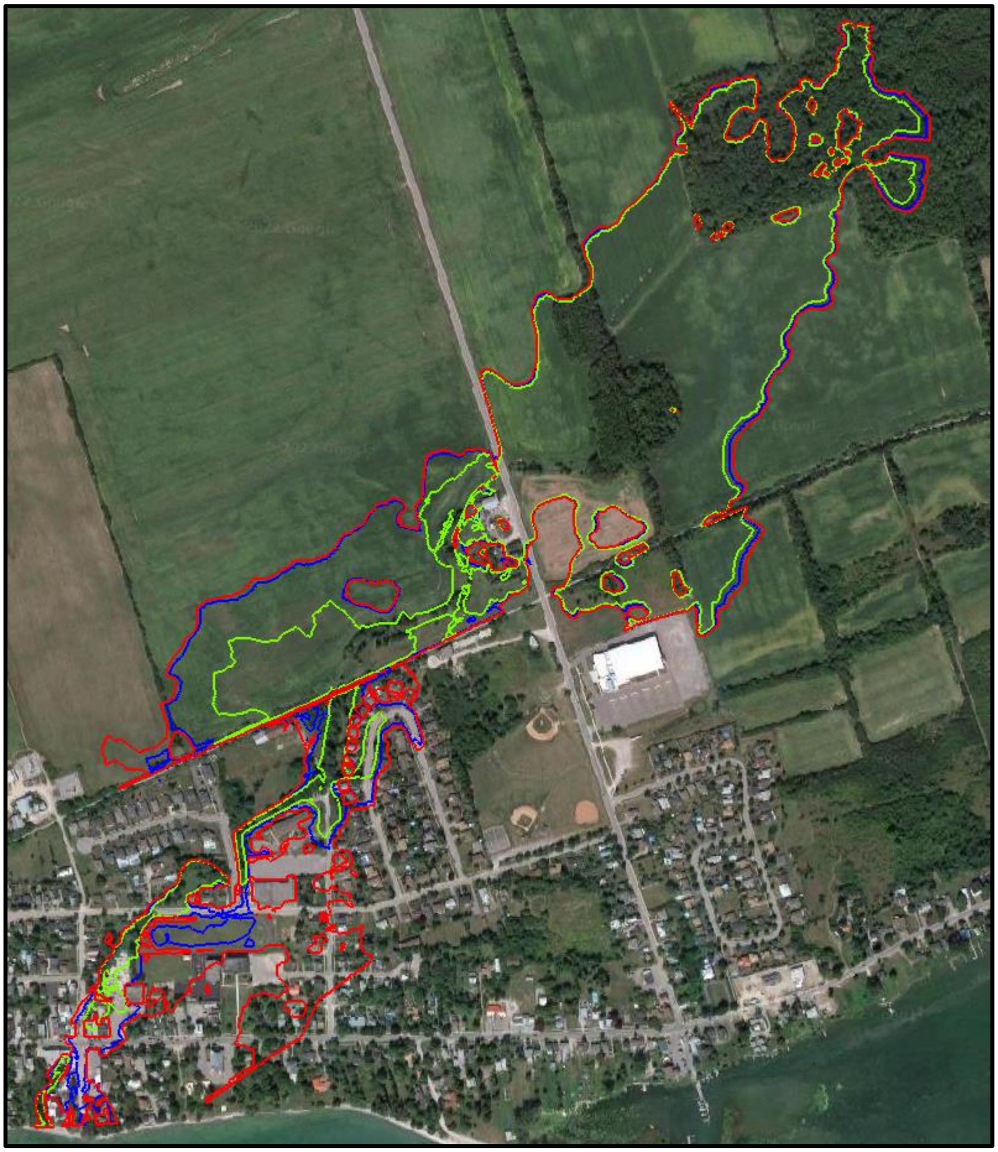


Figure 3-28: Comparison of Flood Limits with Overestimated (Red Line) and Underestimated (Green Line) Peak Flood Discharges Relative to the 2022 Floodplain Limits (Blue Line)

3.5.4 Manning's Roughness Values

The hydraulic model requires inputs for Manning's n values. The *HEC-RAS User's Manual* and *MTO Drainage Manual* provide ranges of roughness coefficient values for varying surface cover such as crop overbank areas, treed areas, and channel bottoms with dense weeds. Mid-range, high, and low Manning's values were tested in different simulations to determine the affect of these values on the floodplain limits. With the exception of Niles Street and Maple Avenue, there is minimal change in flood limits with the adjusted n values. Mid-range values from the HEC RAS User's Manual and from MTO Design Charts were selected and applied in the regulatory floodplain mapping (see Figure 3-29).

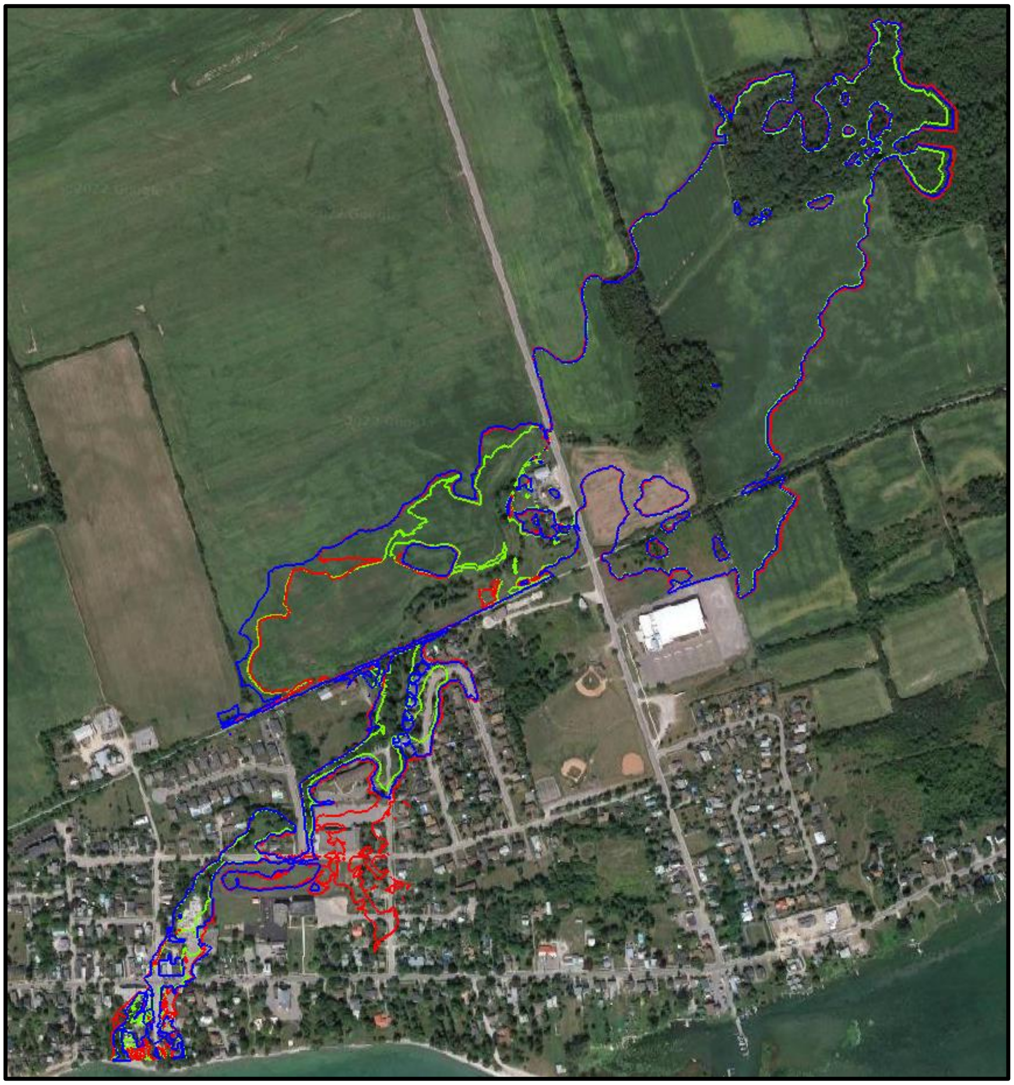


Figure 3-29: Comparison of Flood Limits with High Range (Red Line) and Low Range (Green Line) Manning's Roughness Values Relative to the 2022 Floodplain Limits (Blue Line)

4 Flood Line Delineation

The 2022 floodplain map is provided in Appendix G. After review of the mapping results with Quinte Conservation, Prince Edward County, and local residents during a Public Information Consultation, there are several areas of interest with the updated mapping. A comparison of the 2022 flood limit to the historical 1979 mapping is provided in Figure 4-1.

As described in Section 4, areas of interest include the wide extents upstream of Millennium Trail and Belleville Street. These areas are subject to future urban development, and the current flood hazard limits will limit the developable area. The municipality and local residents expressed interest in engineering solutions to mitigate the existing flood limits to reduce risk to existing property owners as well as to accommodate potential future development lands. The intent of the 2022 flood line is to provide reliable delineation of the existing limits that update the study completed in 1979. Engineered solutions to mitigate the flood limits would be undertaken as a separate study.

Additional areas of interest include where existing infrastructure is located within the 100-yr flood limits, particularly near the intersection of Main Street and Wharf Street.

In a comparison of the 2022 flood limit to the 1979 mapping, the following differences are noted:

1. The 1979 mapping does not extend to Main Street and does not address the flood hazard limits in this high area of interest for the municipality due to the amount of infrastructure that is located within or near the floodplain.

2. Lane Creek has been realigned since 1979 and it currently drains north around the building at 68 Maple Street. The updated creek location has been included in the 2022 mapping.

3. There is an apparent shift in the 1979 north flood line relative to the 2022 limit upstream and downstream of Belleville Street. The 2022 geometry was prepared using georeferenced survey data. It is suspected there was a geolocation error in the 1979 results that shifted the north flood line 10 – 15m north of its intended location. This shift has been addressed in the 2022 mapping through use of the georeferenced survey that is brought into the HEC-RAS mapper tool by importing georeferenced TIF files into the terrain layer.

4. The flood extents are generally wider with the 2022 map. This is expected due to the higher peak flow selected and the inclusion of potential impacts due to climate change.

5. The area immediately north of the Millennium Trail is considerably larger. It is expected that the 2022 results are more reliable due to the detailed survey data applied in the model as well as a review of aerial imagery that suggests there is a low-lying area that closely follow the 2022 flood limit and appears to have been subject to stagnant water in past years.

6. The 1979 study shows an arrow and spill label for the Millennium Trail east of Belleville Street. The 2022 model includes this spill within the geometry data and identifies the limit of spill as part of the floodplain limits.

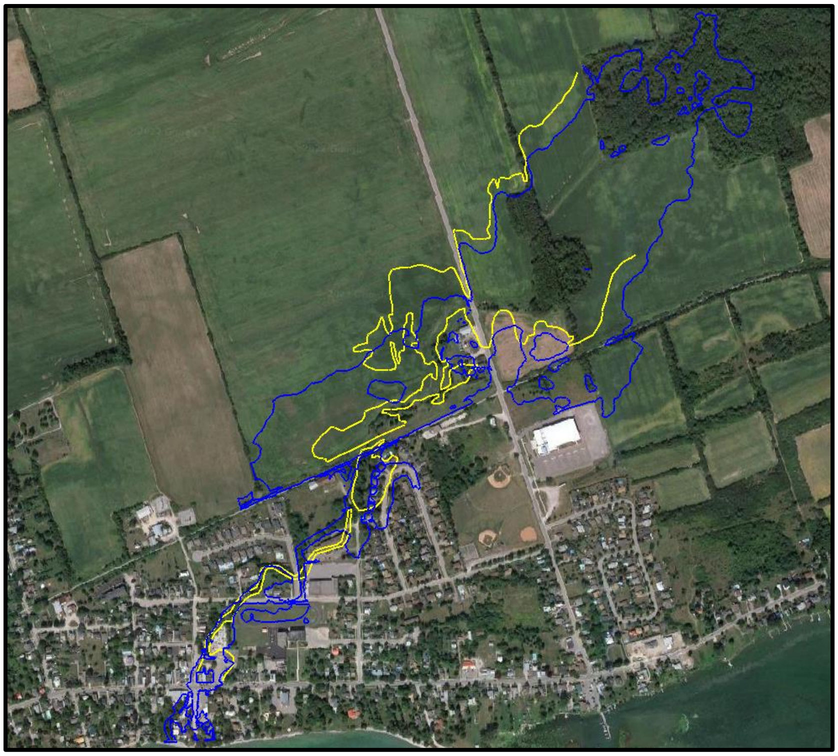


Figure 4-1: Comparison of 2022 Flood Line (Blue) to Historical 1979 Flood Lone (Yellow)

Prepared by:

Elliott Fledderus, P. Eng.
Jewell Engineering Inc.

Bryon Keene, P. Eng.
Jewell Engineering Inc.

Appendix A – Catchment Drawings

JEWELL ENGINEERING

GENERAL NOTES:

- ALL INFORMATION TO BE VERIFIED ON SITE PRIOR TO COMMENCING ANY WORK. ANY DISCREPANCIES ARE TO BE REPORTED TO THE CONSULTANT IMMEDIATELY.
- ALL UTILITY LOCATIONS SHOWN ON THE DRAWINGS ARE APPROXIMATE. THE CONTRACTOR SHALL CONFIRM THE LOCATION ON SITE AND ASSUME ALL LIABILITY FOR DAMAGE TO ALL UTILITIES.
- EXCLUDING THE BENCHMARK AND DESCRIPTION PROVIDED FOR THIS PROJECT, NO OTHER ELEVATIONS ARE TO BE USED AS A REFERENCE ELEVATION FOR ANY PURPOSE.

METRIC NOTE:

- ALL DIMENSIONS SHOWN ARE IN METRES OR MILLIMETRES, UNLESS OTHERWISE NOTED.

GEOMETRIC NOTE:

- ALL SURVEY DATA SHOWN ON THIS DRAWING WAS RECORDED USING REAL-TIME KINETIC (RTK) GPS OBSERVATIONS IN REFERENCE TO UTM 18 NORTH COORDINATE SYSTEM.
- ALL ELEVATIONS ARE IN REFERENCE TO LOCAL DATUM NAD83 - GEODETIC MODEL HT2_0, UNLESS DESCRIBED OTHERWISE.

* * DRAWINGS ARE NOT TO BE SCALED * *

REVISIONS

NO.	DATE	DESCRIPTION	BY

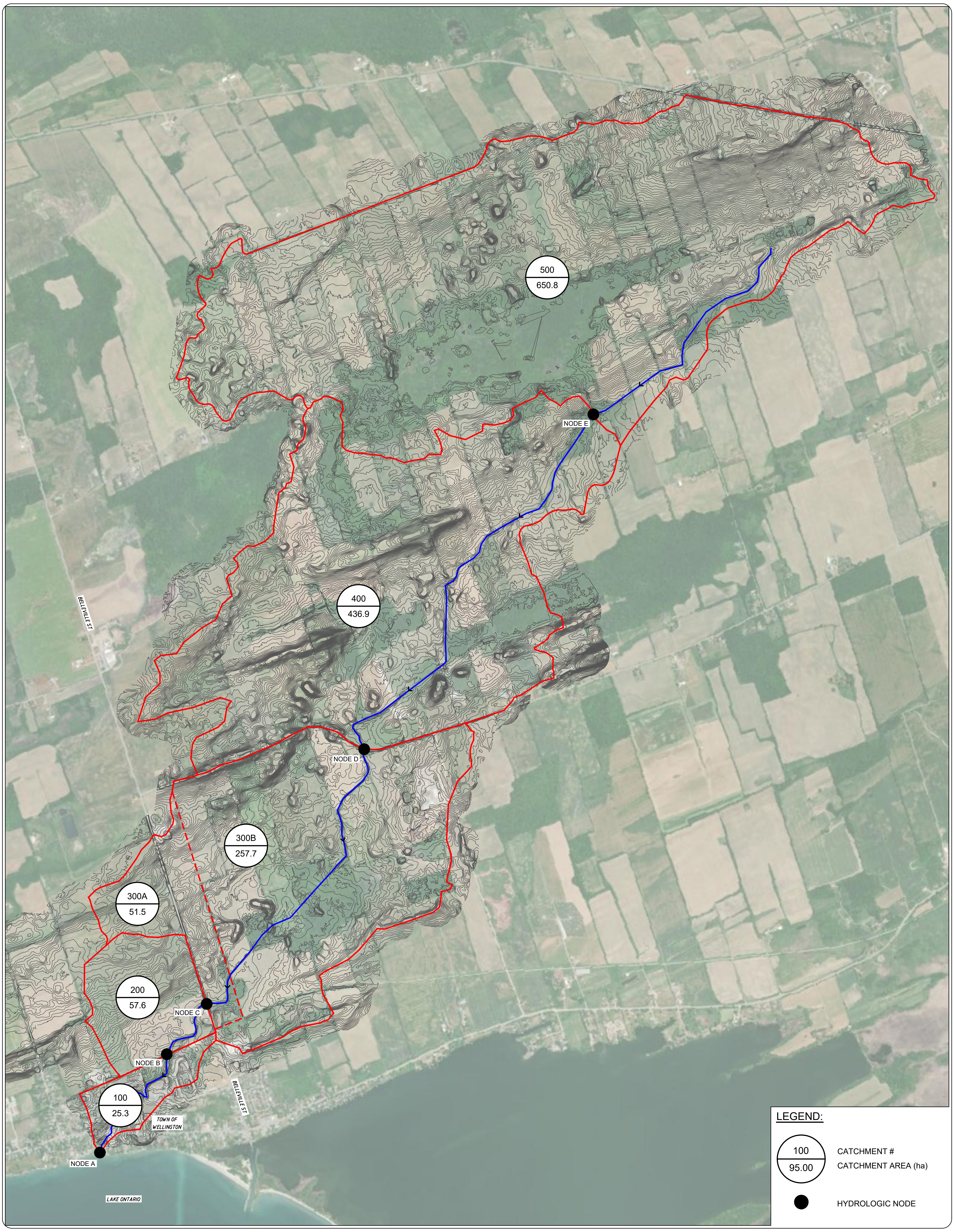
N
W E
S

QUINTE CONSERVATION
LANE CREEK WATERSHED

WELLINGTON, ONTARIO

LANE CREEK EXISTING CONDITION
CATCHMENT DRAWING

DRAWN BY: MB
PROJECT NO: 210-4981
DESIGNED BY: EF
DATE: April 2022
CHECKED BY: EF
CONTRACT NO:
APPROVED BY:
SCALE:
HORIZONTAL - 1:25000
VERTICAL - N/A
DRAWING NO: A-1



JEWELL ENGINEERING

GENERAL NOTES:

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- ALL UTILITY LOCATIONS SHOWN ON THE DRAWINGS ARE APPROXIMATE. THE CONTRACTOR SHALL CONFIRM THE LOCATION ON SITE AND ASSUME ALL LIABILITY FOR DAMAGE TO ALL UTILITIES.
- EXCLUDING THE BENCHMARK AND DESCRIPTION PROVIDED FOR THIS PROJECT, NO OTHER ELEVATIONS ARE TO BE USED AS A REFERENCE ELEVATION FOR ANY PURPOSE.

METRIC NOTE:

- ALL DIMENSIONS SHOWN ARE IN METRES OR MILLIMETRES, UNLESS OTHERWISE NOTED.

GEOMETRIC NOTE:

- ALL SURVEY DATA SHOWN ON THIS DRAWING WAS RECORDED USING REAL-TIME KINETIC (RTK) GPS OBSERVATIONS IN REFERENCE TO UTM 18 NORTH COORDINATE SYSTEM.
- ALL ELEVATIONS ARE IN REFERENCE TO LOCAL DATUM NAD83 - GEODETIC MODEL HT2_0, UNLESS DESCRIBED OTHERWISE.

* * DRAWINGS ARE NOT TO BE SCALED * *

REVISIONS

NO.	DATE	DESCRIPTION	BY

N W E S

QUINTE CONSERVATION
LANE CREEK WATERSHED

WELLINGTON, ONTARIO

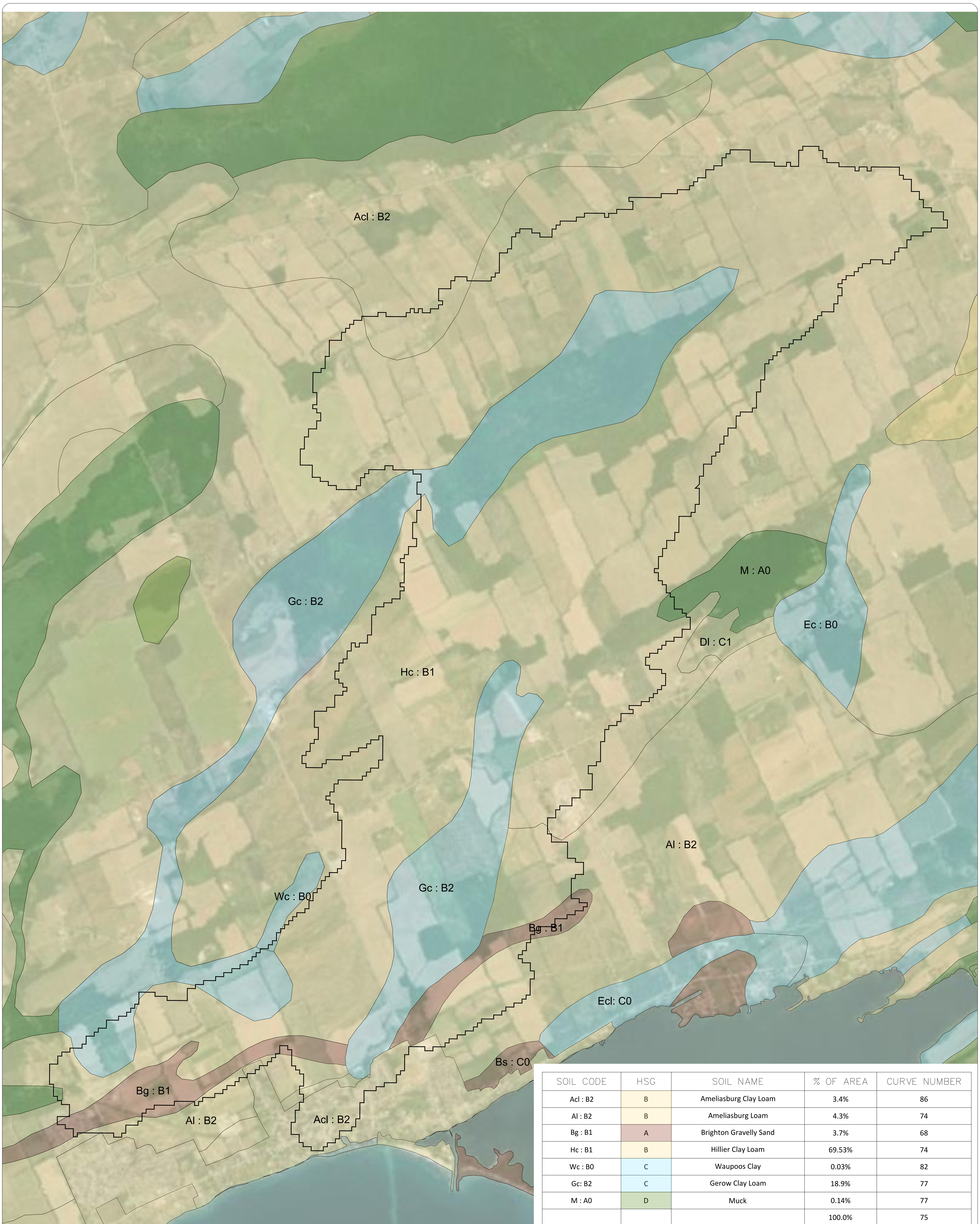
LANE CREEK FUTURE DEVELOPMENT
CATCHMENT DRAWING

DRAWN BY: MB
PROJECT NO: 210-4981
DESIGNED BY: EF
DATE: April 2022
CHECKED BY: EF
CONTRACT NO:
APPROVED BY:

SCALE:
HORIZONTAL - 1:25000
VERTICAL - N/A

DRAWING NO: A-2

Appendix B – Lane Creek Watershed Soils Map and Land Use Schematics



SOIL CODE	HSG	SOIL NAME	% OF AREA	CURVE NUMBER
Acl : B2	B	Ameliasburg Clay Loam	3.4%	86
Al : B2	B	Ameliasburg Loam	4.3%	74
Bg : B1	A	Brighton Gravelly Sand	3.7%	68
Hc : B1	B	Hillier Clay Loam	69.53%	74
Wc : B0	C	Waupoos Clay	0.03%	82
Gc: B2	C	Gerow Clay Loam	18.9%	77
M : A0	D	Muck	0.14%	77
			100.0%	75

JEWELL ENGINEERING

GENERAL NOTES:

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- ALL UTILITY LOCATIONS SHOWN ON THE DRAWINGS ARE APPROXIMATE. THE CONTRACTOR SHALL CONFIRM THE LOCATION ON SITE AND ASSUME ALL LIABILITY FOR DAMAGE TO ALL UTILITIES.
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METRIC NOTE:

- ALL DIMENSIONS SHOWN ARE IN METRES OR MILLIMETRES, UNLESS OTHERWISE NOTED.

GEOMETRIC NOTE:

- ALL SURVEY DATA SHOWN ON THIS DRAWING WAS RECORDED USING REAL-TIME KINETIC (RTK) GPS OBSERVATIONS IN REFERENCE TO UTM 18 NORTH COORDINATE SYSTEM.
- ALL ELEVATIONS ARE IN REFERENCE TO LOCAL DATUM NAD83 - GEODETIC MODEL HT2_0, UNLESS DESCRIBED OTHERWISE.

* * DRAWINGS ARE NOT TO BE SCALED * *

REVISIONS

NO.	DATE	DESCRIPTION	BY

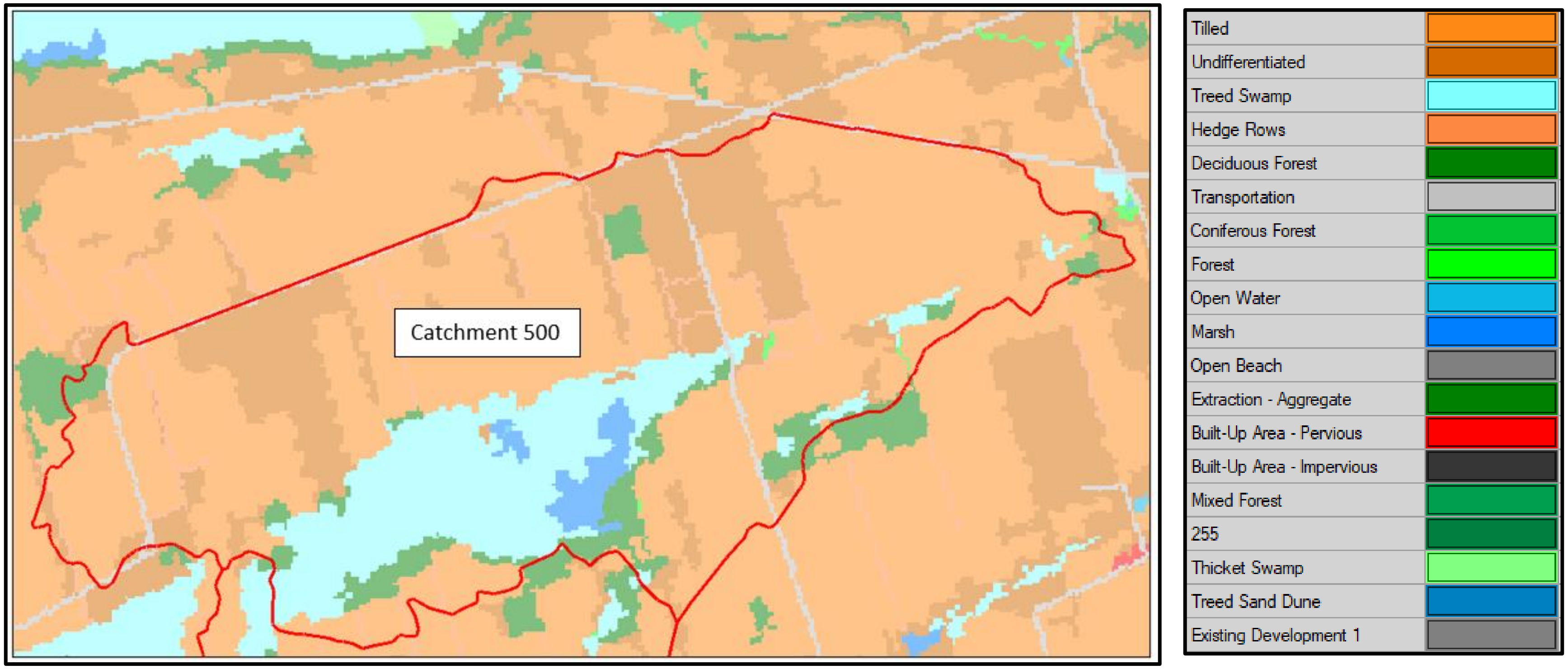
N
W E
S

QUINTE CONSERVATION
LANE CREEK WATERSHED

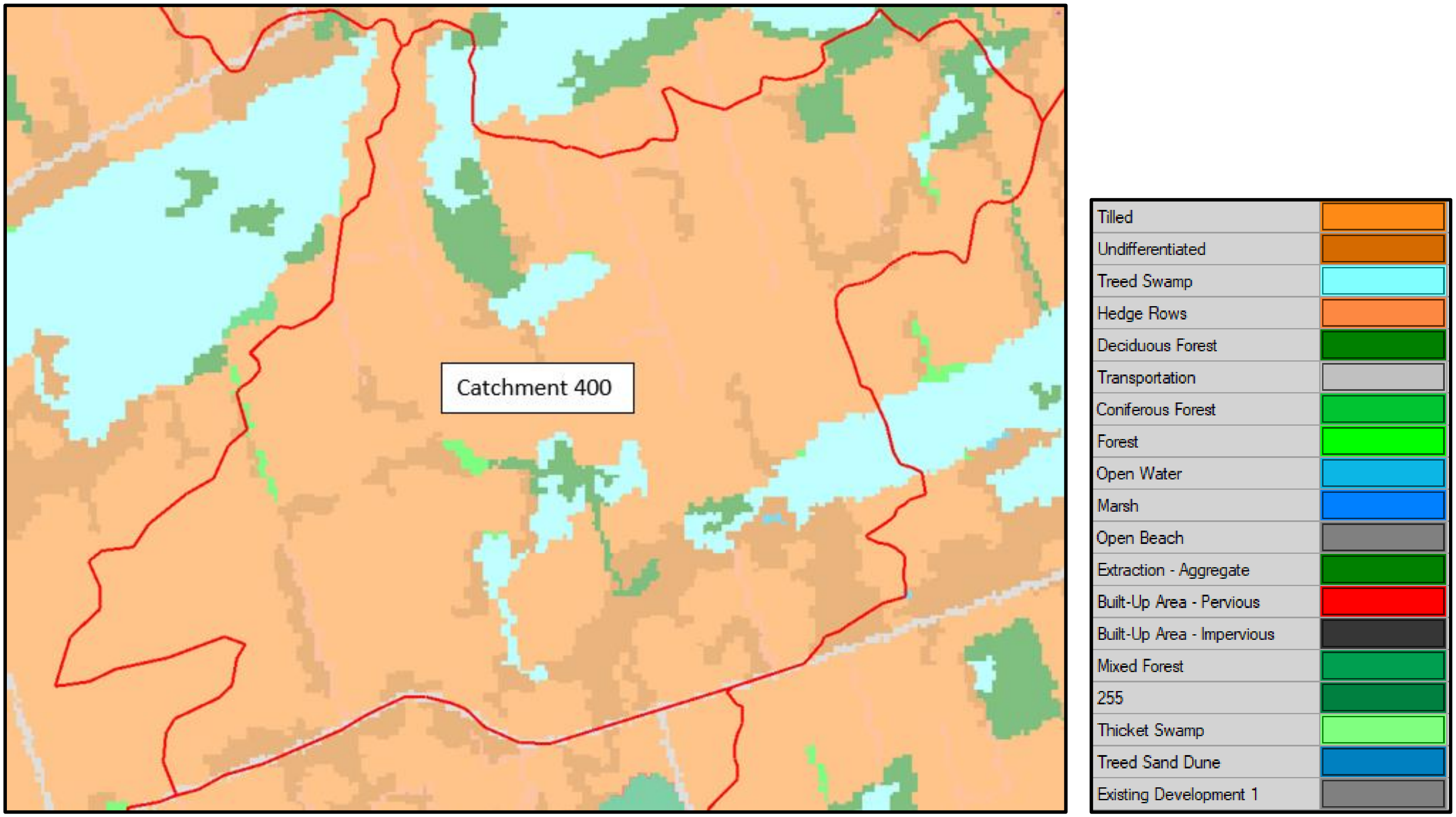
WELLINGTON, ONTARIO

SOILS

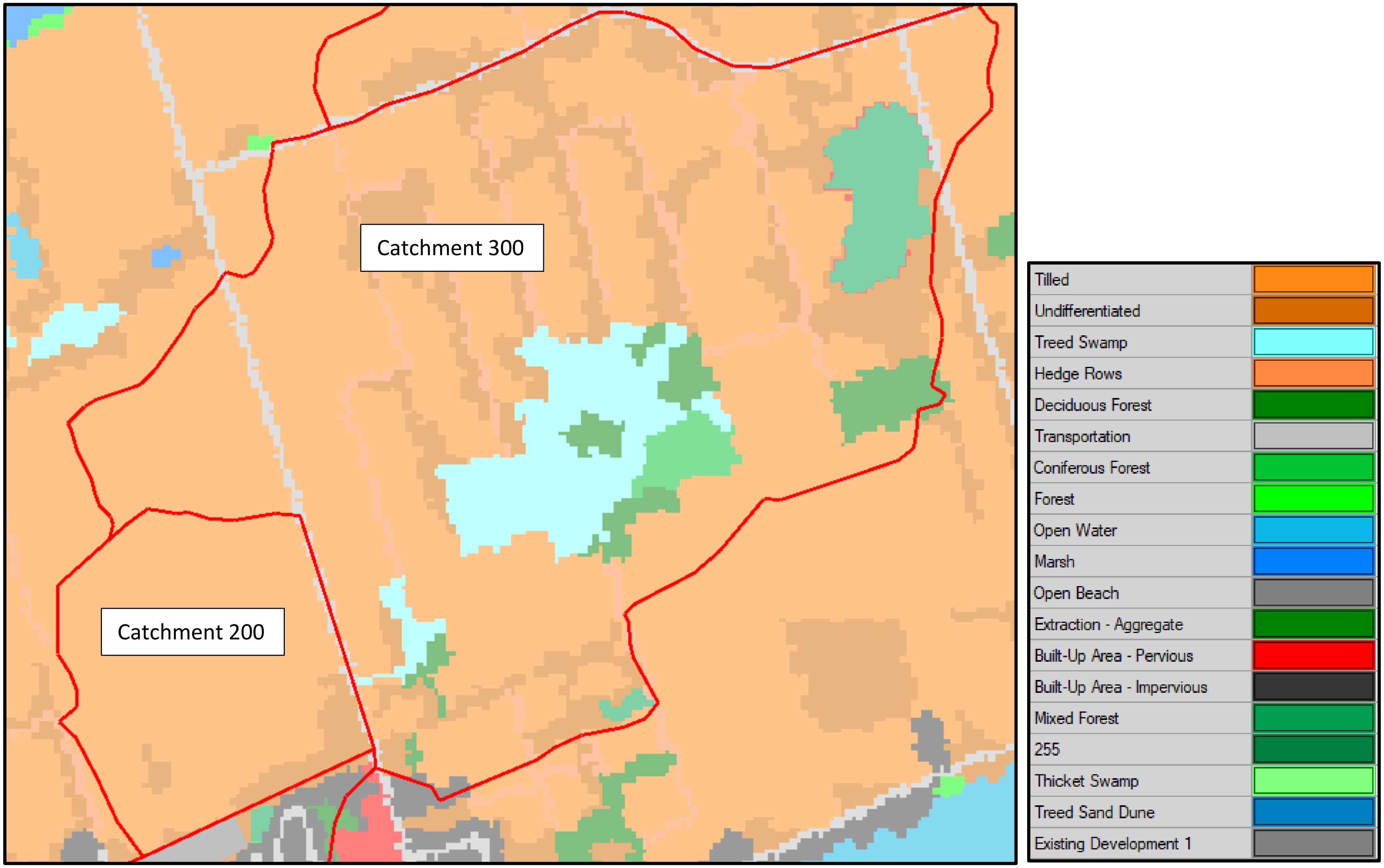
DRAWN BY: MB
PROJECT NO: 210-4981-2
DESIGNED BY:
DATE: November 2021
CHECKED BY:
CONTRACT NO:
APPROVED BY:
SCALE:
HORIZONTAL - 1:12500
VERTICAL - N/A
DRAWING NO:



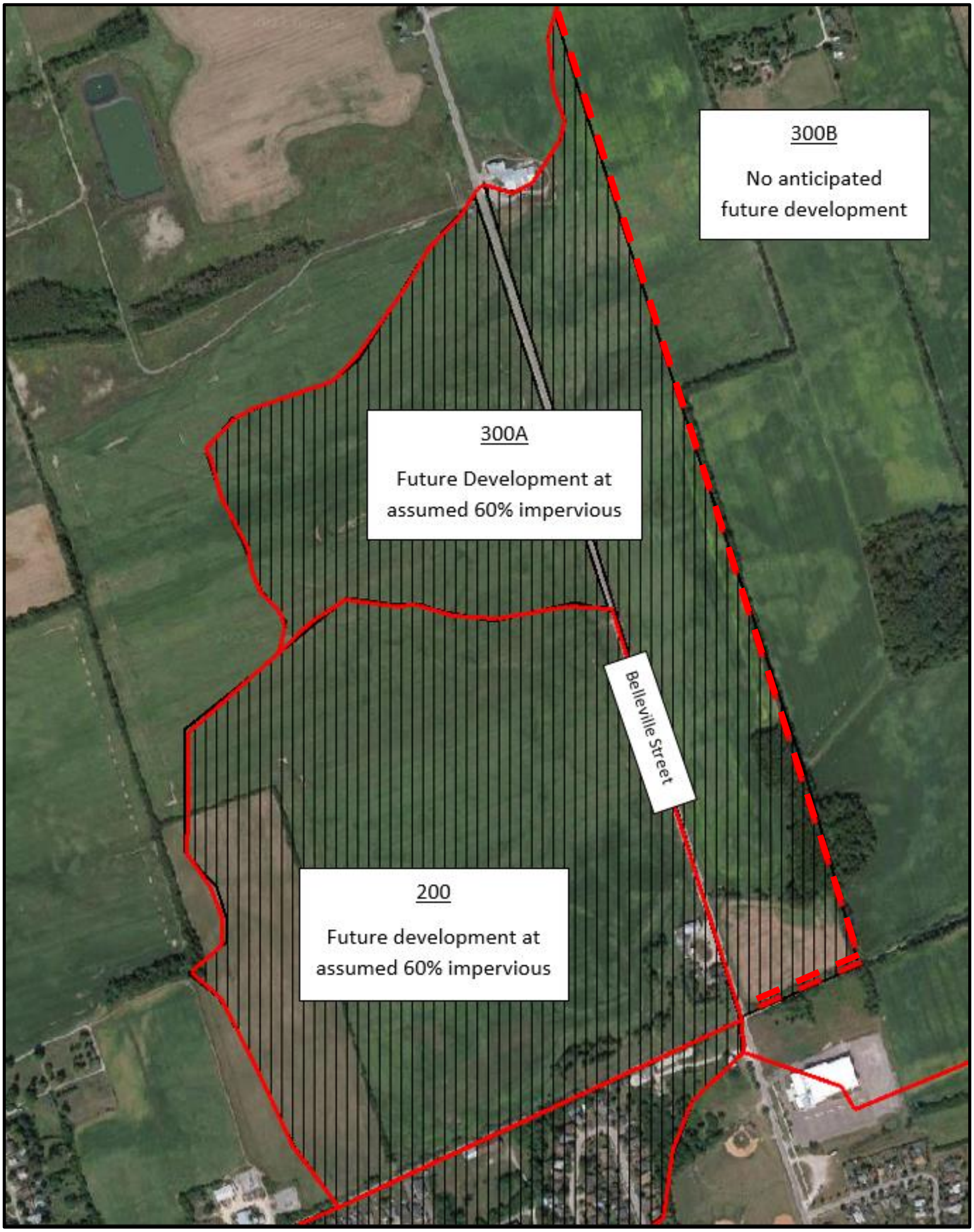
Existing Conditions Land Use Schematic for Catchment 500 Showing Predominantly Agriculture, Treed, and Swamp Coverage– No Anticipated Future Development



Existing Conditions Land Use Schematic for Catchment 400 Showing Predominantly Agriculture, Treed, and Swamp Coverage - No Anticipated Future Development



Existing Conditions Land Use Schematic for Catchments 200 & 300 Showing Predominantly Agriculture, Treed, and Swamp Coverage



Future Development Land Use Schematic for Catchments 200 and 300A

Land Use Schematic for Catchment 100

Appendix C – Lane Creek HEC-HMS Tabular Rainfall Inputs

2-Yr, Trenton IDF Curves		Rainfall depth =		35.3
Hour	%	Incremental Rainfall	%	Accumulated Rainfall
		mm		mm
1	8%	2.8	8%	2.8
2	9%	3.2	17%	6.0
3	11%	3.9	28%	9.9
4	49%	17.3	77%	27.2
5	15%	5.3	92%	32.5
6	8%	2.8	100%	35.3

5-Yr, Trenton IDF Curves		Rainfall depth =		47.5
Hour	%	Incremental Rainfall	%	Accumulated Rainfall
		mm		mm
1	8%	3.8	8%	3.8
2	9%	4.3	17%	8.1
3	11%	5.2	28%	13.3
4	49%	23.3	77%	36.6
5	15%	7.1	92%	43.7
6	8%	3.8	100%	47.5

10-Yr, Trenton IDF Curves		Rainfall depth =		55.5
Hour	%	Incremental Rainfall	%	Accumulated Rainfall
		mm		mm
1	8%	4.4	8%	4.4
2	9%	5.0	17%	9.4
3	11%	6.1	28%	15.5
4	49%	27.2	77%	42.7
5	15%	8.3	92%	51.1
6	8%	4.4	100%	55.5

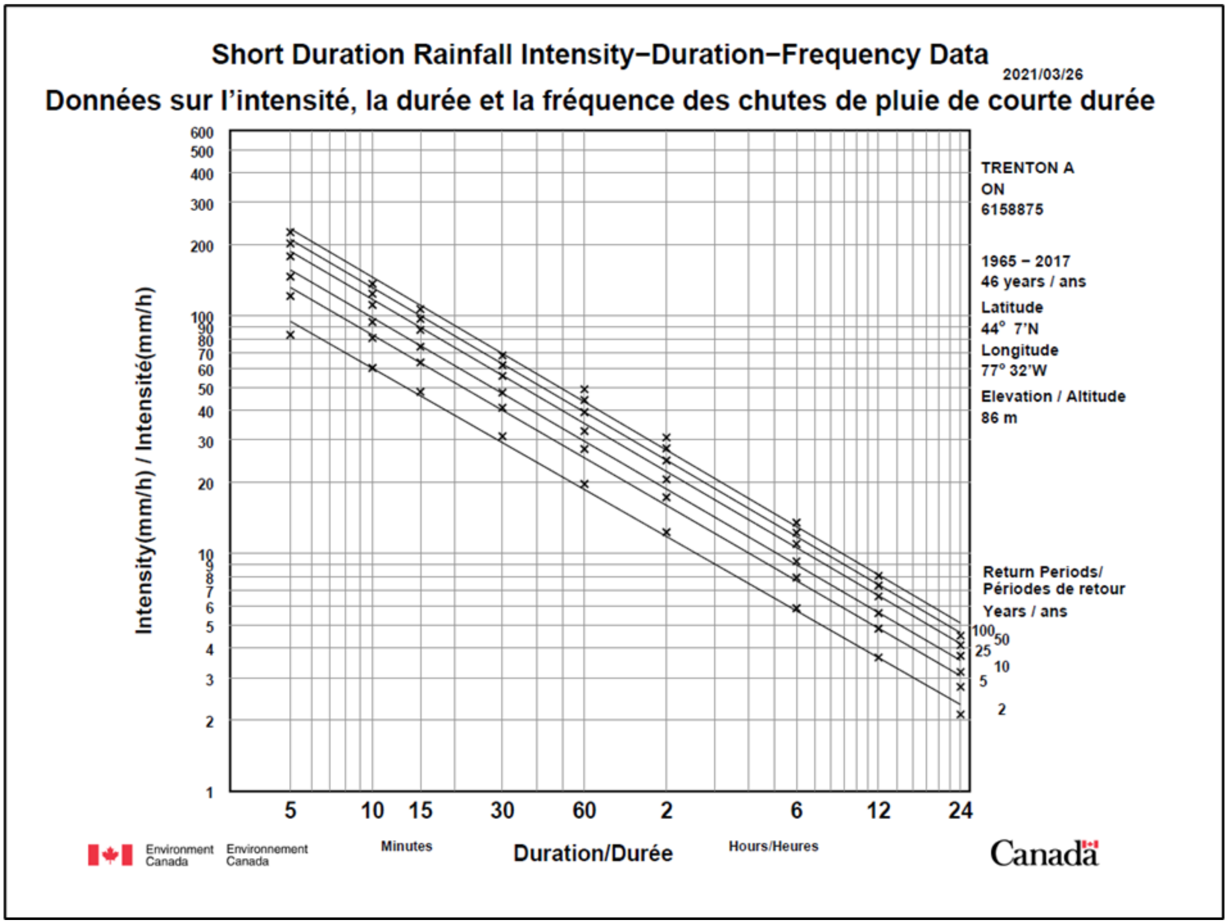
25-Yr, Trenton IDF Curves		Rainfall depth =		65.7
Hour	%	Incremental Rainfall	%	Accumulated Rainfall
		mm		mm
1	8%	5.3	8%	5.3
2	9%	5.9	17%	11.2
3	11%	7.2	28%	18.4
4	49%	32.2	77%	50.6
5	15%	9.9	92%	60.4
6	8%	5.3	100%	65.7

50-Yr, Trenton IDF Curves		Rainfall depth =		73.3
Hour	%	Incremental Rainfall mm	%	Accumulated Rainfall mm
1	8%	5.9	8%	5.9
2	9%	6.6	17%	12.5
3	11%	8.1	28%	20.5
4	49%	35.9	77%	56.4
5	15%	11.0	92%	67.4
6	8%	5.9	100%	73.3

100-Yr, Trenton IDF Curves		Rainfall depth =		80.8
Hour	%	Incremental Rainfall mm	%	Accumulated Rainfall mm
1	8%	6.5	8%	6.5
2	9%	7.3	17%	13.7
3	11%	8.9	28%	22.6
4	49%	39.6	77%	62.2
5	15%	12.1	92%	74.3
6	8%	6.5	100%	80.8

100-Yr, MTO IDF Look-Up 2072		Rainfall depth =		84.6
Hour	%	Incremental Rainfall mm	%	Accumulated Rainfall mm
1	8%	6.8	8%	6.8
2	9%	7.6	17%	14.4
3	11%	9.3	28%	23.7
4	49%	41.5	77%	65.1
5	15%	12.7	92%	77.8
6	8%	6.8	100%	84.6

Trenton Station IDF Curve:



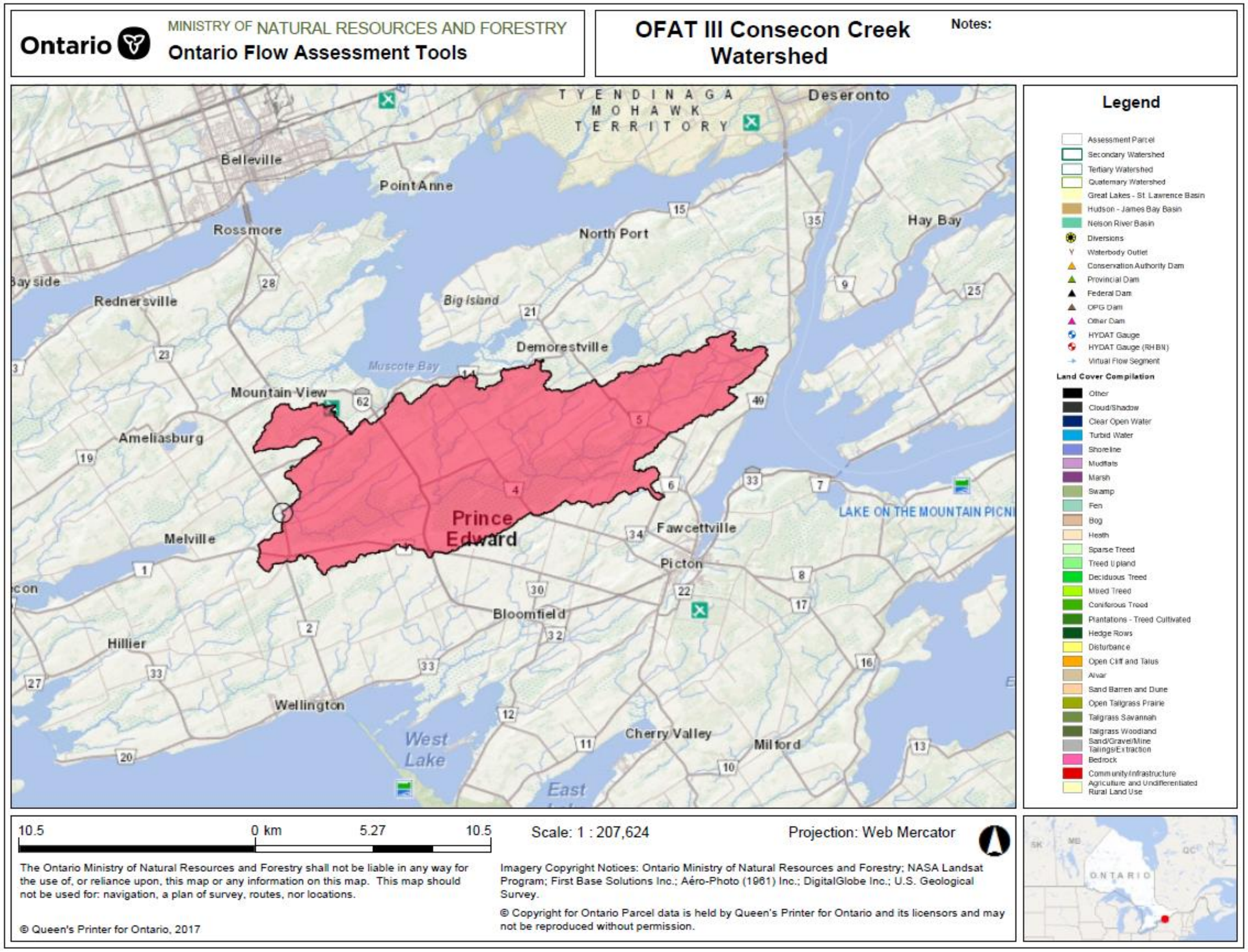
```
**********************************************************************

Table 2a : Return Period Rainfall Amounts (mm)
           Quantité de pluie (mm) par période de retour

**********************************************************************

 Duration/Durée        2       5      10      25      50     100   #Years
                  yr/ans  yr/ans  yr/ans  yr/ans  yr/ans  yr/ans   Années
         5 min       6.9    10.1    12.2    14.8    16.8    18.7       47
        10 min      10.0    13.4    15.7    18.5    20.6    22.7       47
        15 min      12.0    15.9    18.5    21.8    24.2    26.7       47
        30 min      15.5    20.5    23.8    27.9    31.0    34.0       47
         1 h        19.6    27.5    32.7    39.3    44.2    49.0       47
         2 h        24.5    34.4    41.0    49.2    55.4    61.4       47
         6 h        35.3    47.5    55.5    65.7    73.3    80.8       46
        12 h        43.8    57.9    67.2    79.1    87.8    96.5       48
        24 h        50.4    65.9    76.1    89.0    98.6   108.1       48

**********************************************************************

Table 2b :

 Return Period Rainfall Rates (mm/h) - 95% Confidence limits
 Intensité de la pluie (mm/h) par période de retour - Limites de confiance de 95%

**********************************************************************

 Duration/Durée        2       5      10      25      50     100   #Years
                  yr/ans  yr/ans  yr/ans  yr/ans  yr/ans  yr/ans   Années
         5 min      83.1   120.9   146.0   177.7   201.2   224.5       47
                +/- 11.2 +/- 18.9 +/- 25.6 +/- 34.5 +/- 41.3 +/- 48.1  47
        10 min      60.3    80.6    94.0   111.0   123.7   136.2       47
                +/-  6.0 +/- 10.2 +/- 13.7 +/- 18.5 +/- 22.1 +/- 25.8  47
        15 min      47.8    63.6    74.0    87.1    96.9   106.6       47
                +/-  4.7 +/-  7.9 +/- 10.6 +/- 14.3 +/- 17.1 +/- 20.0  47
        30 min      31.1    41.0    47.5    55.8    61.9    68.0       47
                +/-  2.9 +/-  4.9 +/-  6.7 +/-  9.0 +/- 10.8 +/- 12.5  47
         1 h        19.6    27.5    32.7    39.3    44.2    49.0       47
                +/-  2.3 +/-  3.9 +/-  5.3 +/-  7.2 +/-  8.6 +/- 10.0  47
         2 h        12.3    17.2    20.5    24.6    27.7    30.7       47
                +/-  1.5 +/-  2.5 +/-  3.3 +/-  4.5 +/-  5.4 +/-  6.3  47
         6 h         5.9     7.9     9.3    11.0    12.2    13.5       46
                +/-  0.6 +/-  1.0 +/-  1.4 +/-  1.9 +/-  2.2 +/-  2.6  46
        12 h         3.6     4.8     5.6     6.6     7.3     8.0       48
                +/-  0.3 +/-  0.6 +/-  0.8 +/-  1.1 +/-  1.3 +/-  1.5  48
        24 h         2.1     2.7     3.2     3.7     4.1     4.5       48
                +/-  0.2 +/-  0.3 +/-  0.4 +/-  0.6 +/-  0.7 +/-  0.8  48

**********************************************************************
```

```
**********************************************************************************

Table 3 : Interpolation Equation / Équation d'interpolation: R = A*T^B

R = Interpolated Rainfall rate (mm/h)/Intensité interpolée de la pluie (mm/h)
RR = Rainfall rate (mm/h) / Intensité de la pluie (mm/h)
 T = Rainfall duration (h) / Durée de la pluie (h)

**********************************************************************************
```

Statistics/Statistiques	2 yr/ans	5 yr/ans	10 yr/ans	25 yr/ans	50 yr/ans	100 yr/ans
Mean of RR/Moyenne de RR	29.5	40.7	48.1	57.4	64.4	71.2
Std. Dev. /Écart-type (RR)	28.6	40.4	48.3	58.4	65.8	73.2
Std. Error/Erreur-type	4.5	4.2	4.2	4.5	4.9	5.3
Coefficient (A)	18.6	25.2	29.6	35.2	39.3	43.4
Exponent/Exposant (B)	-0.656	-0.664	-0.668	-0.671	-0.673	-0.674
Mean % Error/% erreur moyenne	5.1	5.0	5.2	5.6	6.0	6.2

Appendix D – Streamflow Gauge Data for Hydrologically Similar Watersheds

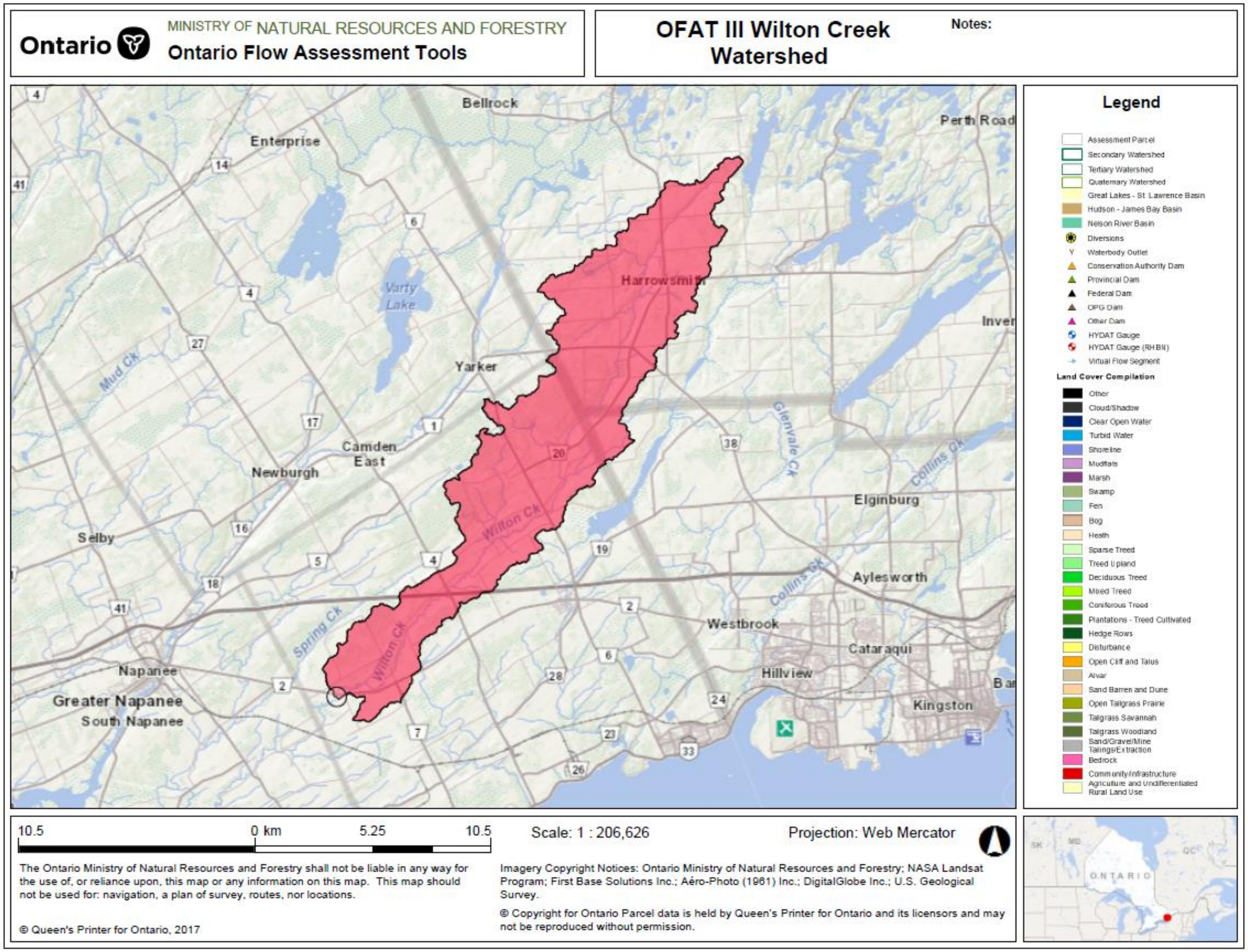
Consecon Creek Watershed to WSOC Stream Flow Gauge 02HE002:



WSOC Consecon Creek Instantaneous Stream Flow Gauge Data Record

Year	Flow (cms)	Flow (cfs)
1970	23	812
1971	27.6	975
1972	27.5	971
1973	21	742
1974	25.1	886
1975	21.4	756
1976	19.7	696
1977	39.1	1381
1978	36.5	1289
1979	23.5	830
1981	43.9	1550
1982	36.1	1275
1984	35.7	1261
1985	18.4	650
1986	31.9	1126
1987	21.4	756
1989	18.7	660
1990	20.7	731
1991	18.9	667
1992	24.5	865
1993	39.3	1388
1994	17.3	611
1995	13.2	466
1996	21.3	752
1997	17.5	618
1998	15.1	533
2001	16.4	579
2002	14.4	509
2003	43.8	1547
2004	22.3	787
2005	27.2	961
2006	15.3	540
2007	16.5	583
2008	23.7	837
2009	14.7	519
2011	24	848
2012	5.8	205
2013	26.6	939
2014	33	1165
2015	13.2	466
2016	11	388
2017	26.3	929
2018	16.3	576
2019	21.5	759
2020	11.6	410

Wilton Creek Watershed to WSOC Stream Flow Gauge 02HM004:

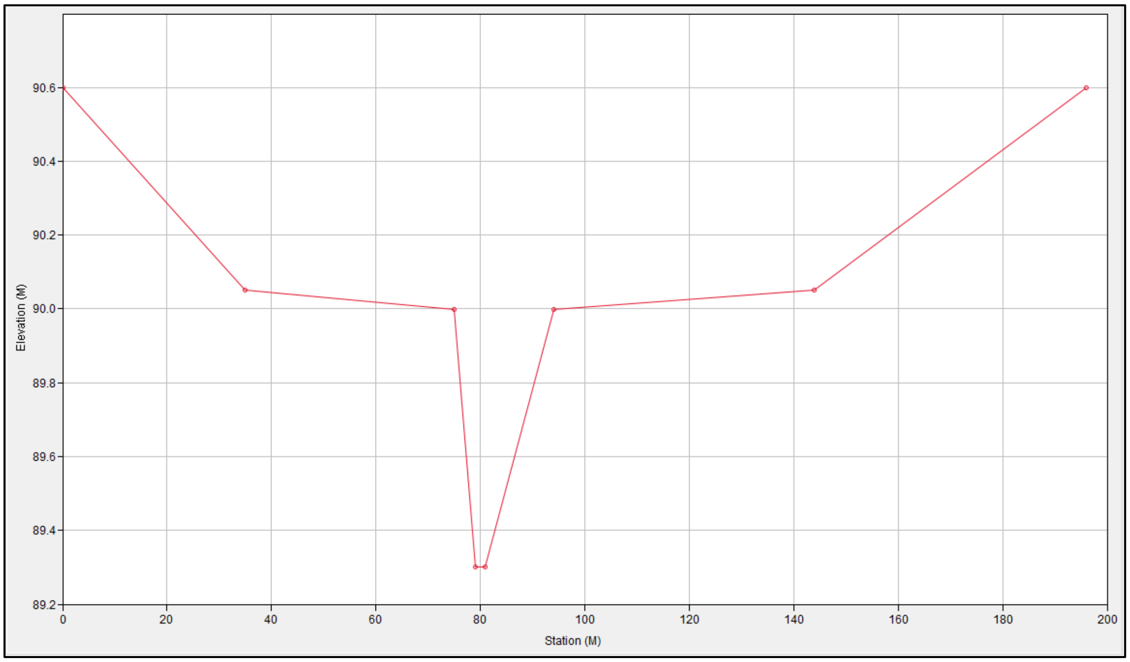


WSOC Wilton Creek Instantaneous Stream Flow Gauge Data Record

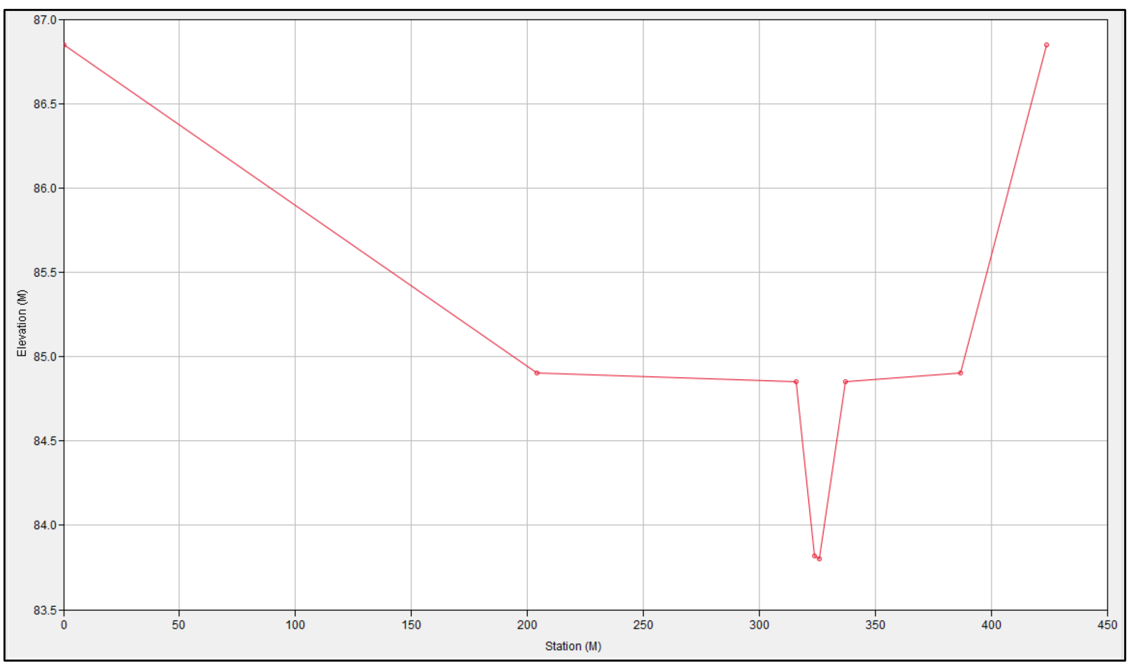
Year	Flow (cms)	Flow (cfs)
1965	12.9	456
1967	18.3	646
1968	18.3	646
1969	27.8	982
1970	22.1	780
1971	20.6	727
1972	23.8	840
1973	16.9	597
1974	27.4	968
1982	26.6	939
1986	32.4	1144
1987	22.4	791
1988	20.8	735
1991	11.9	420
1993	29.2	1031
1994	12.7	448
1995	20.9	738
1996	17.9	632
1997	15	530
1998	10.6	374
2001	10.3	364
2002	20	706
2004	21	742
2005	26.7	943
2006	26.9	950
2007	12.7	448
2008	23.1	816
2009	19.6	692
2010	35.5	1254
2011	32.9	1162
2012	14	494
2013	21.2	749
2014	31.6	1116
2015	14.2	501
2016	13.9	491
2017	28.6	1010
2018	27.2	961
2019	27	953
2020	21.2	749

Appendix E – Channel Routing Cross Sections

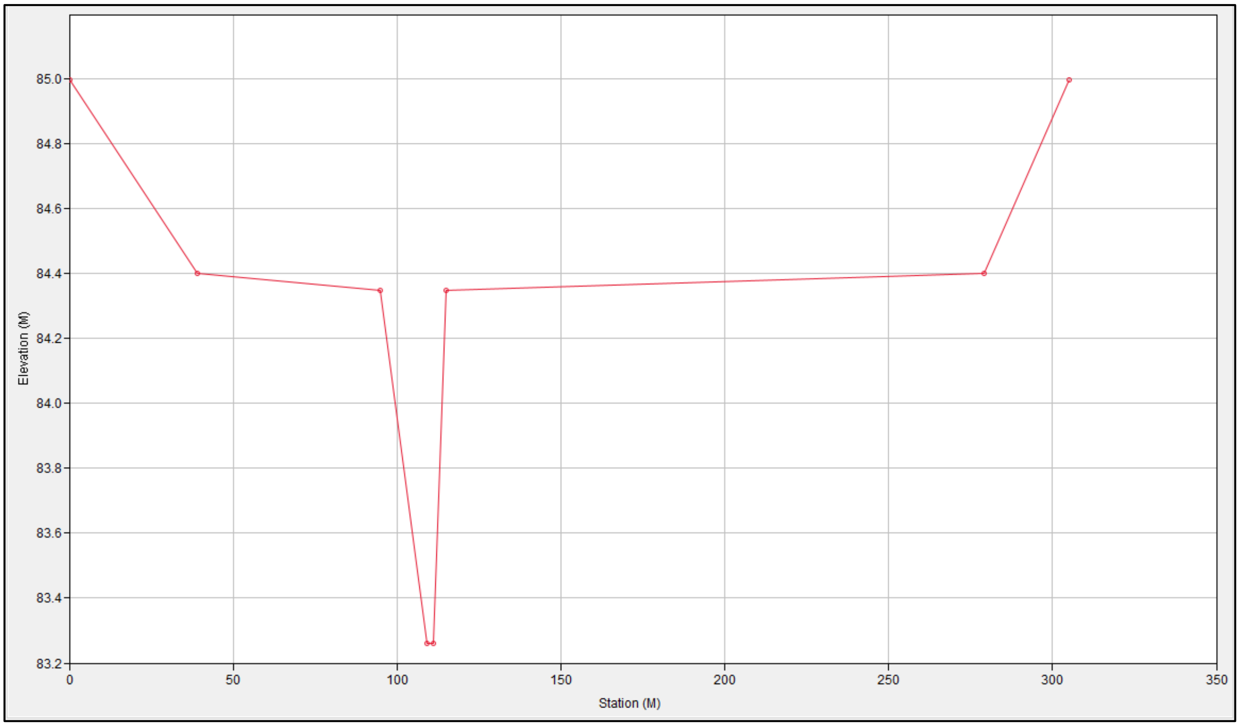
Reach 1:



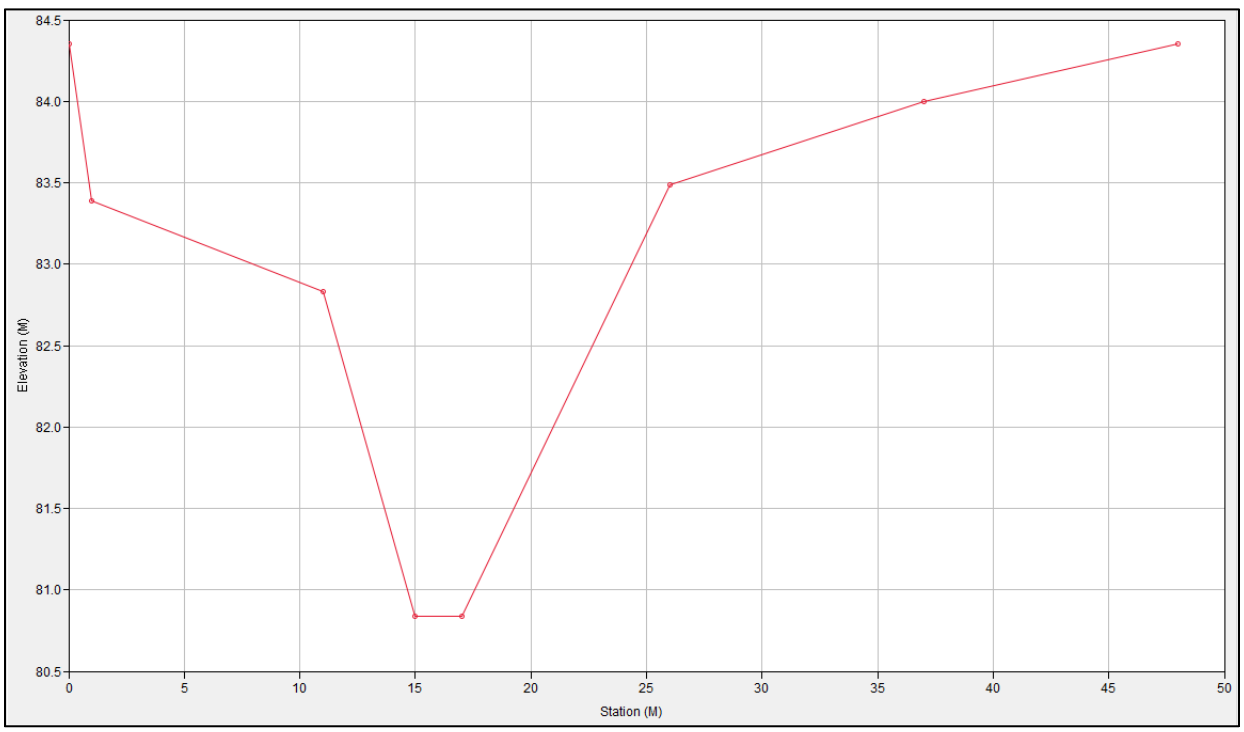
Reach 2:



Reach 3:



Reach 4:



Appendix F – LiDAR Station Reports

LiDAR Project Summary

produced: December 21, 2021

Airborne Imaging

A Clean Harbors Company

2700 - 61 Avenue SE
Calgary, Alberta, Canada
T2C 4V2

Telephone: (403) 215 2960
Fax: (403) 258 3189
www.airborneimaginginc.com

Project Information

Project Name:	Quinte Areas (MeyersPier/WellersBay/LaneCreek/DeerCreek)
Airborne Project Number:	16560
Client:	Quinte Conservation
Project Location:	Wellers Bay / Belleville / Madoc / Wellington, Ontario, Canada
Project Size:	17.2 km²

Acquisition Projects

Project Number	Project Name	Vintage
1875	Quinte Conservation	November 2021

Acquisition Parameters

Date (MM/DD/YY)	Mission	Flying Height (m)	Flying Speed (knots)	Pulse Rate Rep (kHz)	Scan Freq (Hz)	Scan Angle (degree)	Side Lap %	Point Density (pts/m²)	LiDAR System
11/27/21	3821331a	1600	160	1000	100	60	20	8.5	Riegl VQ-1560ii

Geodetic Control

Horizontal Datum:	Nad83 CSRS	**Vertical Datum:**	CGVD2013
		Undulation model:	CGG2013

Note: We established a local geodetic network fixed to the following control:

Station ID	Lat	Long	Ellp Height (m)
CANNET - PCI2	N44° 00' 23.00114"	W77° 08' 30.371821"	67.002
CANNET - CMAD	N44° 28' 55.586031"	W77° 28' 16.432343"	135.057

Calibration Methodology

Airborne Imaging performs a complete calibration on every LiDAR acquisition flight. The data is first produced with its predetermined boresight values and then the calibration refined by applying corrections to the attitude of the aircraft (roll, pitch and heading) and fluctuations if necessary. To statistically quantify the accuracy, we compare the LiDAR elevations with independently surveyed ground points. A GPS mounted truck collects data while driving on an open road. The kinematic positions on the road are post-processed from a nearby base station to provide ground truth points.

Accuracy

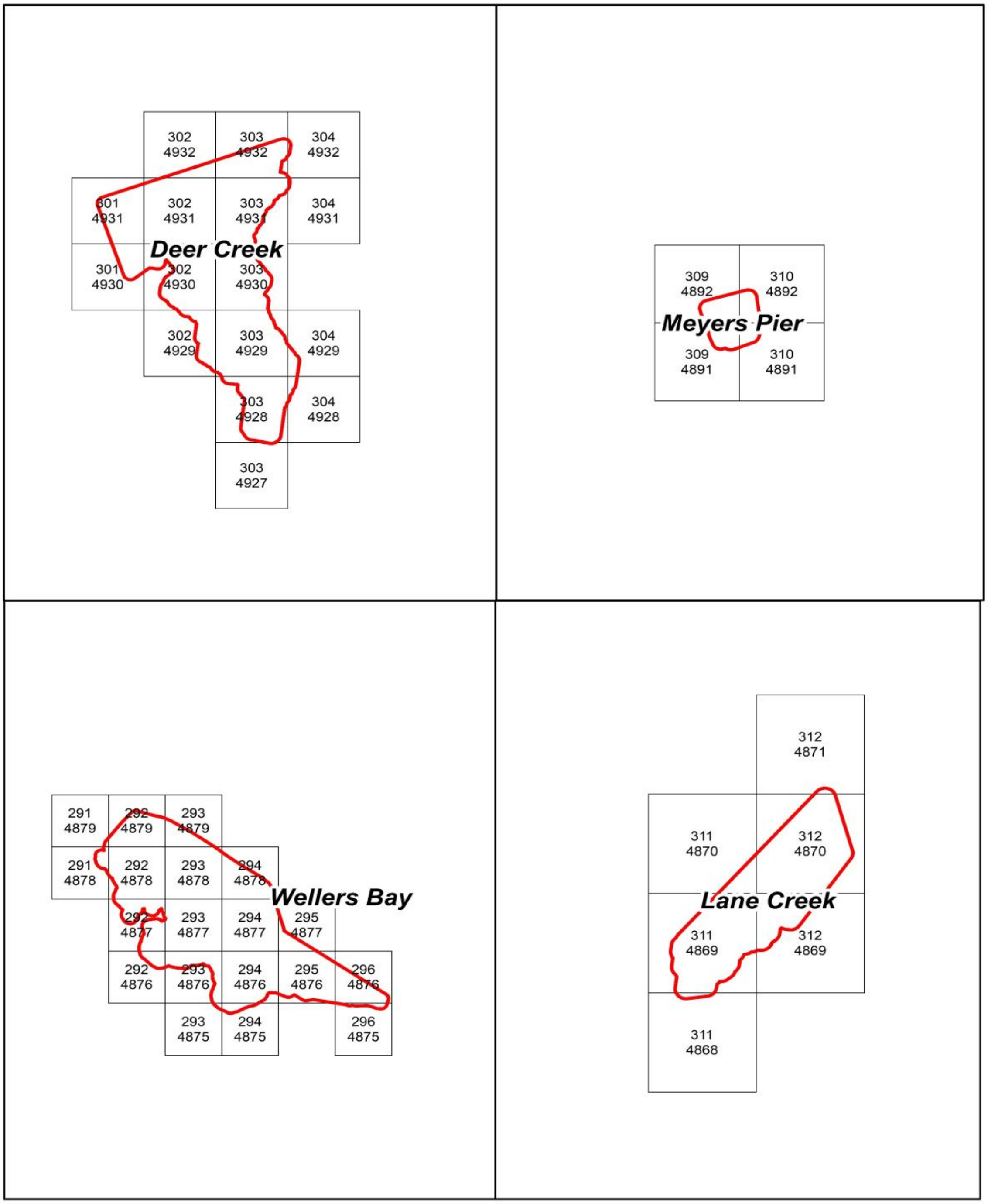
Horizontal Accuracy, 95% or 2σ:	25 cm
Fundamental Vertical Accuracy (on flat hard surfaces), 95% or 2σ:	10 cm

Deliverables

Projection: **UTM 18**

Deliverables Formats

Shapefiles (Project Boundary, Tile Index, Hydro Breaklines)
1m Grids (XYZ ASCII), Bare Earth, Bare Earth Hydroflattened and Full Feature
1m Hillshade Images (Geotiffs), Bare Earth, Bare Earth Hydroflattened and Full Feature
Point Cloud (LAS v1.2)
1m and 50cm Contours (shp)
Metadata (LiDAR Summary, Flightlog, Control Stations, Accuracies)



16560 - Quinte Areas - Quinte Conservation

Station Report - CMAD

Station 1 of 1

Site Identification

Name	Province	NTS map sheet	Unique Number	Provincial Identifier	Network
CANNET-MADO	Ontario	031C06	CMAD		CANNET

Station Coordinates

Coordinates	Reference Frame	Vertical Datum	Geoid	Epoch
geo	NAD83(CSRS)	CGVD2013	CGG2013	2010.0

Latitude	Longitude	h (metres)
N44° 28' 55.586031" ± 0.0015m	W77° 28' 16.432343" ± 0.0008m	135.057 ± 0.0024m
Vφ (mm/y)	Vλ (mm/y)	Vh (mm/y)
-1.87 ± 0.00	1.87 ± 0.00	0.87 ± 0.00
N (metres)	H (metres)	Published date and project ID
-34.283 ± 0.010	169.340	2015-09-24 M15-703

Vertical Data

Use the value of H from the coordinates above.

Station Marker

Marker Type	Inspected in	Established by	Status	Comments
Unknown		Cannet		None

Station Report - CPI2

Station 1 of 1

Site Identification

Name	Province	NTS map sheet	Unique Number	Provincial Identifier	Network
CANNET-PCT2	Ontario	031C03	CPI2		CANNET

Station Coordinates

Coordinates	Reference Frame	Vertical Datum	Geoid	Epoch
geo	NAD83(CSRS)	CGVD2013	CGG2013	2010.0

Latitude	Longitude	h (metres)
N44° 00' 23.00114" ± 0.0009m	W77° 08' 30.371821" ± 0.0004m	67.002 ± 0.0014m
Vφ (mm/y)	Vλ (mm/y)	Vh (mm/y)
-1.73 ± 0.00	1.77 ± 0.00	0.43 ± 0.00
N (metres)	H (metres)	Published date and project ID
-34.791 ± 0.011	101.793	2015-09-24 M15-703

Vertical Data

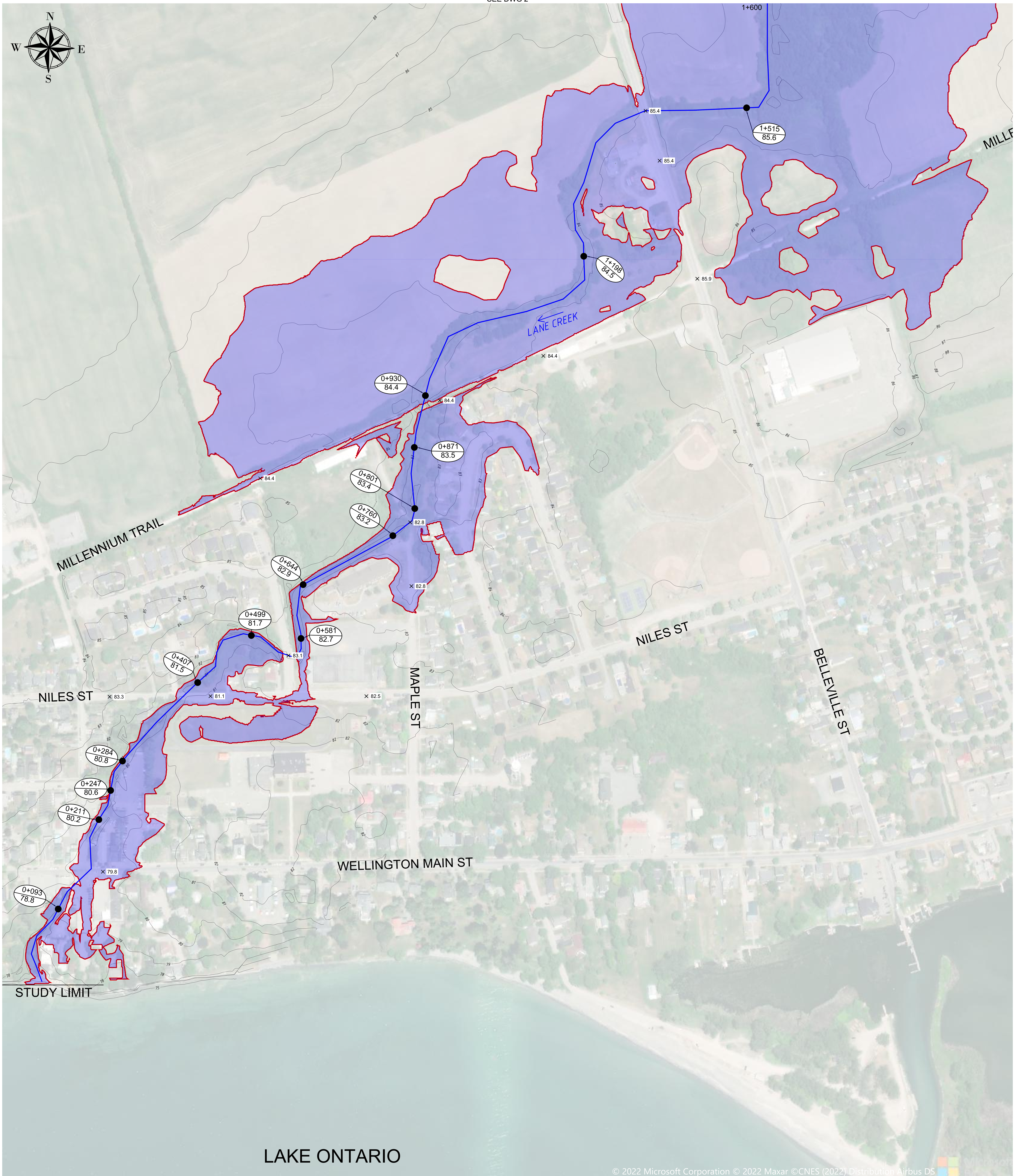
Use the value of H from the coordinates above.

Station Marker

Marker Type	Inspected in	Established by	Status	Comments
Unknown		Cannet		None

Appendix G-1 – Lane Creek 100-Yr Regulatory Floodplain Mapping

SEE DWG 2



CROSSING NAME	100-YR WSEL (m)
BELLEVILLE STREET	85.55
MILLENNIUM TRAIL	84.37
MAPLE STREET	83.22
LAKE BREEZE	82.60
NILES STREET	81.53
MAIN STREET	79.99

JEWELL ENGINEERING

Quinte CONSERVATION

LEGEND:

- LANE CREEK CENTRELINE
- 100-YR REGULATORY FLOOD HAZARD LIMIT
- NODE
- 0+093 / 78.8 — STATION # / 100-YR WSEL (m)
- × 82.5 — ROAD ELEVATION

DATUM: CGVD28

LICENSED PROFESSIONAL ENGINEER E.J.W. FLEDDERUS 100502141 Apr. 19, 2022 PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER B. C. KEENE Apr 19/22 PROVINCE OF ONTARIO

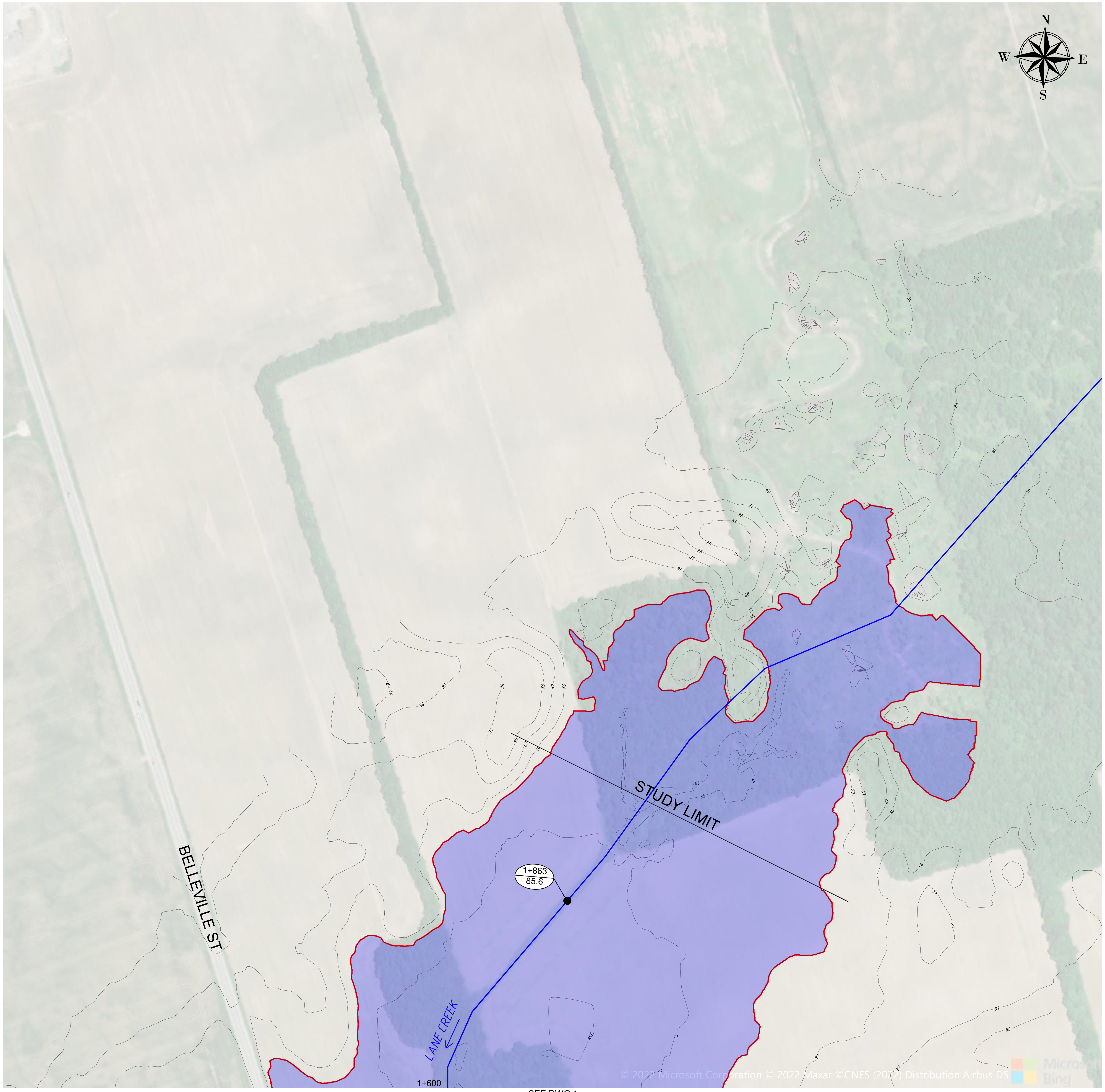
QUINTE CONSERVATION
LANE CREEK

WELLINGTON, ONTARIO

LANE CREEK FLOODPLAIN MAPPING
COMMUNITY OF WELLINGTON

DRAWN BY: MB	PROJECT NO: 210-4981-2
DESIGNED BY: NG & EF	DATE: April 2022
CHECKED BY: EF	CONTRACT NO:
APPROVED BY: EF & BK	
SCALE: HORIZONTAL - 1:2,000; CONTOUR INTERVAL - 1.0m	
	DRAWING NO: 1

CROSSING NAME	100-YR WSEL (m)
BELLEVILLE STREET	85.55
MILLENNIUM TRAIL	84.37
MAPLE STREET	83.22
LAKE BREEZE	82.60
NILES STREET	81.53
MAIN STREET	79.99



SEE DWG 1

JEWELL ENGINEERING

Quinte CONSERVATION

LEGEND:

LANE CREEK CENTRELINE

100-YR REGULATORY FLOOD HAZARD LIMIT

NODE

1+863 / 85.6 — STATION # / 100-YR WSEL (m)

DATUM: CGVD28

LICENSED PROFESSIONAL ENGINEER E.J.W. FLEDDERUS 100502141 Apr. 19, 2022 PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER B. C. KEENE Apr 19/22 PROVINCE OF ONTARIO

QUINTE CONSERVATION
LANE CREEK

WELLINGTON, ONTARIO

LANE CREEK FLOODPLAIN MAPPING
COMMUNITY OF WELLINGTON

DRAWN BY: MB	PROJECT NO: 210-4981-2
DESIGNED BY: NG & EF	DATE: April 2022
CHECKED BY: EF	CONTRACT NO:
APPROVED BY: EF & BK	
SCALE: HORIZONTAL - 1:2,000 CONTOUR INTERVAL - 1.0m	
	DRAWING NO: 2

Appendix G-2 – Floodplain Maps for Lesser Return Period Events

SEE DWG 2

JEWELL ENGINEERING

Quinte CONSERVATION

LEGEND:

2-YR FLOODLINE

DATUM: CGVD28

LICENSED PROFESSIONAL ENGINEER
E.J.W. FLEDDERUS
100502141
PROVINCE OF ONTARIO

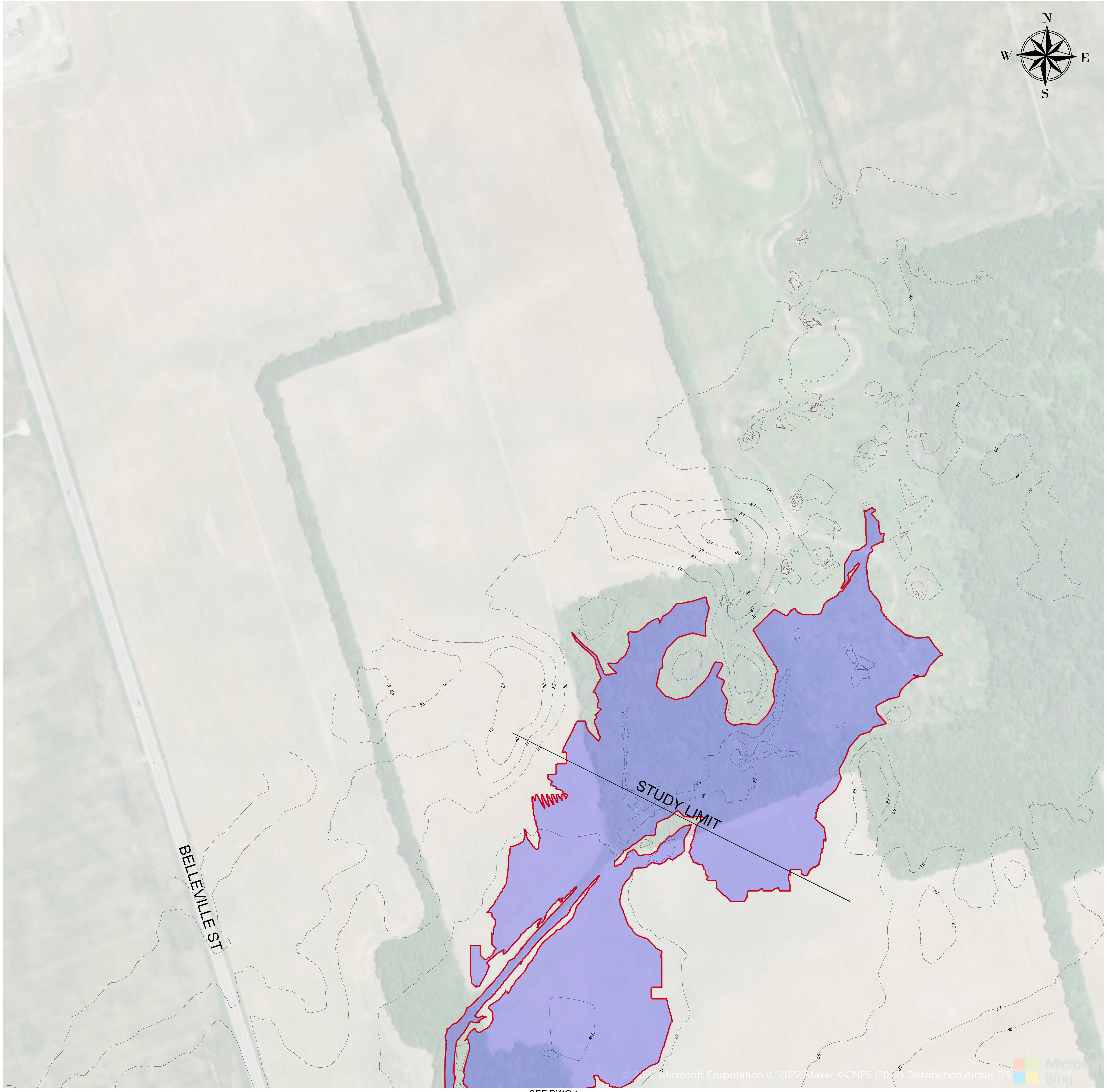
LICENSED PROFESSIONAL ENGINEER
B. C. KEENE
PROVINCE OF ONTARIO

QUINTE CONSERVATION
LANE CREEK

WELLINGTON, ONTARIO

2-YEAR
FLOODPLAIN DRAWING

DRAWN BY: MB	PROJECT NO: 210-4981-2
DESIGNED BY: NG & EF	DATE: April 2022
CHECKED BY: EF	CONTRACT NO:
APPROVED BY: EF & BK	
SCALE: HORIZONTAL - 1:2,000 CONTOUR INTERVAL - 1.0m	
	DRAWING NO: 1



SEE DWG 1

JEWELL ENGINEERING

Quinte CONSERVATION

LEGEND:

2-YR FLOODLINE

DATUM: CGVD28

LICENSED PROFESSIONAL ENGINEER E.J.W. FLEDDERUS 100502141 Apr. 19, 2022 PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER B. C. KEENE Apr 19/22 PROVINCE OF ONTARIO

QUINTE CONSERVATION
LANE CREEK

WELLINGTON, ONTARIO

2-YEAR
FLOODPLAIN DRAWING

DRAWN BY: MB	PROJECT NO: 210-4981-2
DESIGNED BY: NG & EF	DATE: April 2022
CHECKED BY: EF	CONTRACT NO:
APPROVED BY: EF & BK	
SCALE: HORIZONTAL - 1:2,000 CONTOUR INTERVAL - 1.0m	
	DRAWING NO: 2

JEWELL ENGINEERING

Quinte CONSERVATION

LEGEND:

5-YR FLOODLINE

DATUM: CGVD28

LICENSED PROFESSIONAL ENGINEER E.J.W. FLEDDERUS 100502141 Apr. 19, 2022 PROVINCE OF ONTARIO

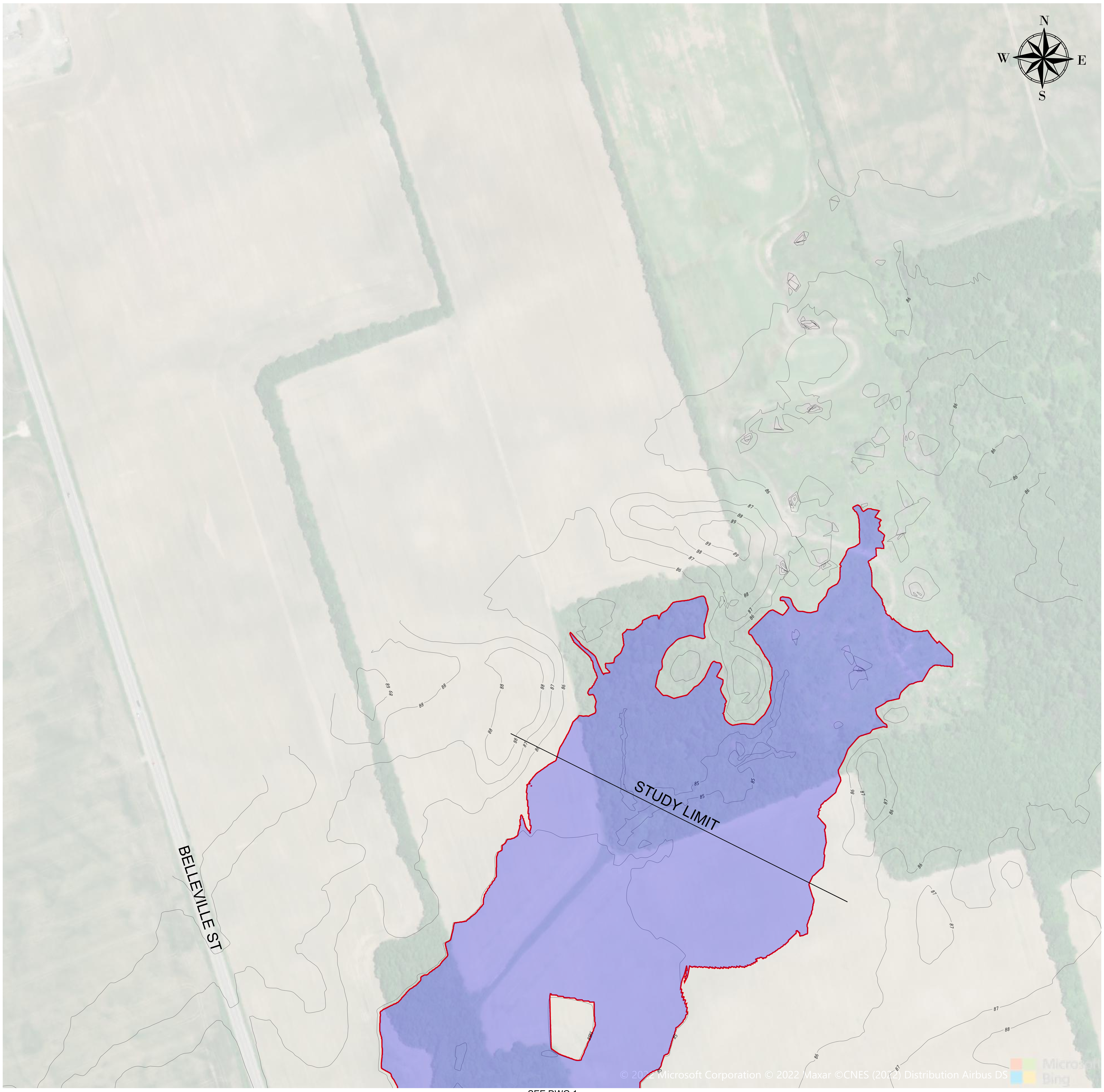
LICENSED PROFESSIONAL ENGINEER B. C. KEENE Apr 19/22 PROVINCE OF ONTARIO

QUINTE CONSERVATION
LANE CREEK

WELLINGTON, ONTARIO

5-YEAR
FLOODPLAIN DRAWING

DRAWN BY: MB	PROJECT NO: 210-4981-2
DESIGNED BY: NG & EF	DATE: April 2022
CHECKED BY: EF	CONTRACT NO:
APPROVED BY: EF & BK	
SCALE: HORIZONTAL - 1:2,000 CONTOUR INTERVAL - 1.0m	
	DRAWING NO: 1



SEE DWG 1

JEWELL ENGINEERING

Quinte CONSERVATION

LEGEND:

5-YR FLOODLINE

DATUM: CGVD28

LICENSED PROFESSIONAL ENGINEER
E.J.W. FLEDDERUS
100502141
Apr. 19, 2022
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
B. C. KEENE
Apr 19/22
PROVINCE OF ONTARIO

QUINTE CONSERVATION
LANE CREEK

WELLINGTON, ONTARIO

5-YEAR
FLOODPLAIN DRAWING

DRAWN BY: MB	PROJECT NO: 210-4981-2
DESIGNED BY: NG & EF	DATE: April 2022
CHECKED BY: EF	CONTRACT NO:
APPROVED BY: EF & BK	
SCALE: HORIZONTAL - 1:2,000 CONTOUR INTERVAL - 1.0m	
	DRAWING NO: 2

JEWELL ENGINEERING

Quinte CONSERVATION

LEGEND:

25-YR FLOODLINE

DATUM: CGVD28

LICENSED PROFESSIONAL ENGINEER
E.J.W. FLEDDERUS
100502141
Apr. 19, 2022
PROVINCE OF ONTARIO

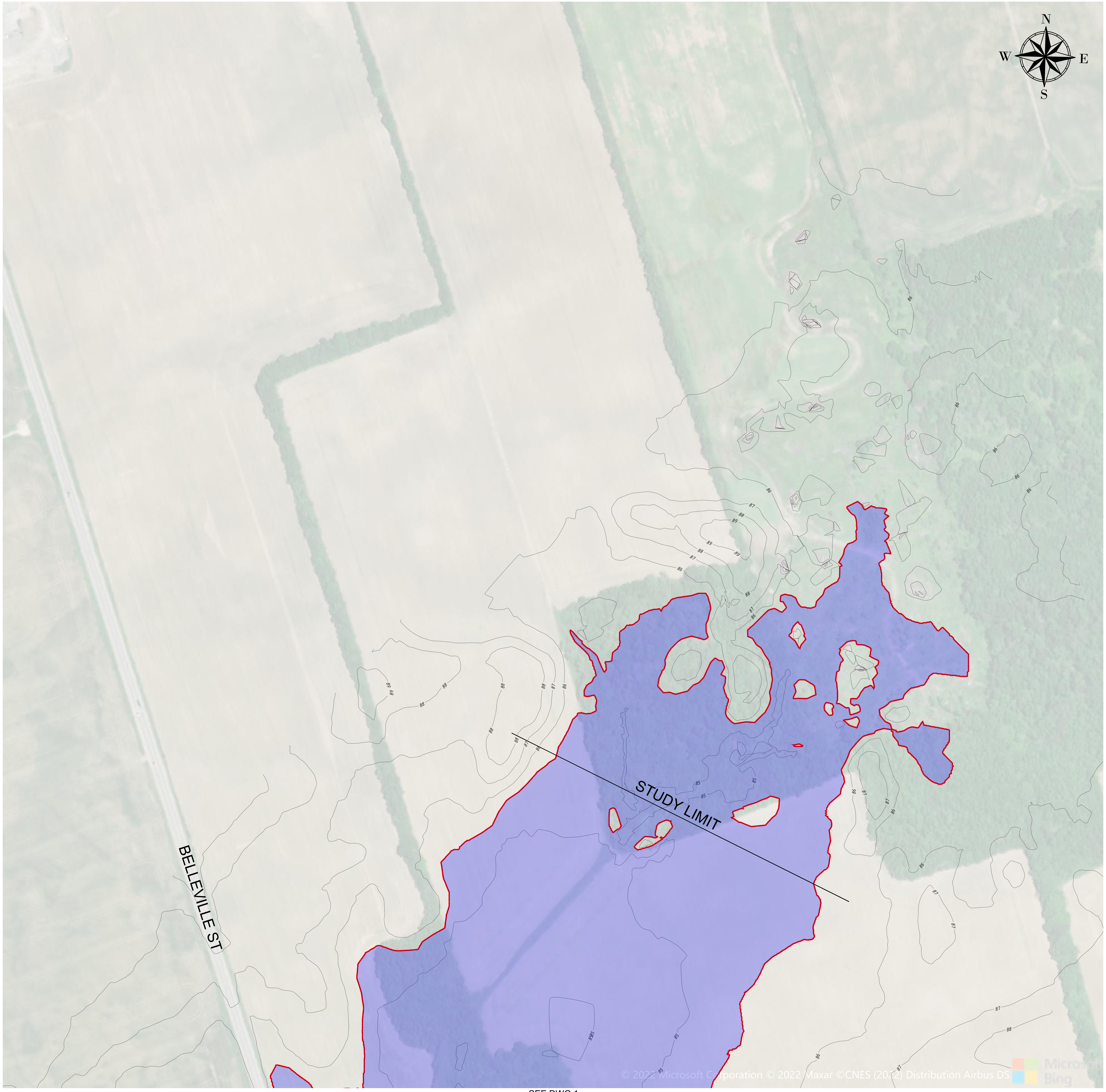
LICENSED PROFESSIONAL ENGINEER
B. C. KEENE
Apr 19/22
PROVINCE OF ONTARIO

QUINTE CONSERVATION
LANE CREEK

WELLINGTON, ONTARIO

25-YEAR
FLOODPLAIN DRAWING

DRAWN BY: MB	PROJECT NO: 210-4981-2
DESIGNED BY: NG & EF	DATE: April 2022
CHECKED BY: EF	CONTRACT NO:
APPROVED BY: EF & BK	
SCALE: HORIZONTAL - 1:2,000; CONTOUR INTERVAL - 1.0m	
	DRAWING NO: 1



SEE DWG 1

JEWELL ENGINEERING

Quinte CONSERVATION

LEGEND:

25-YR FLOODLINE

DATUM: CGVD28

LICENSED PROFESSIONAL ENGINEER
E.J.W. FLEDDERUS
100502141
Apr. 19, 2022
PROVINCE OF ONTARIO

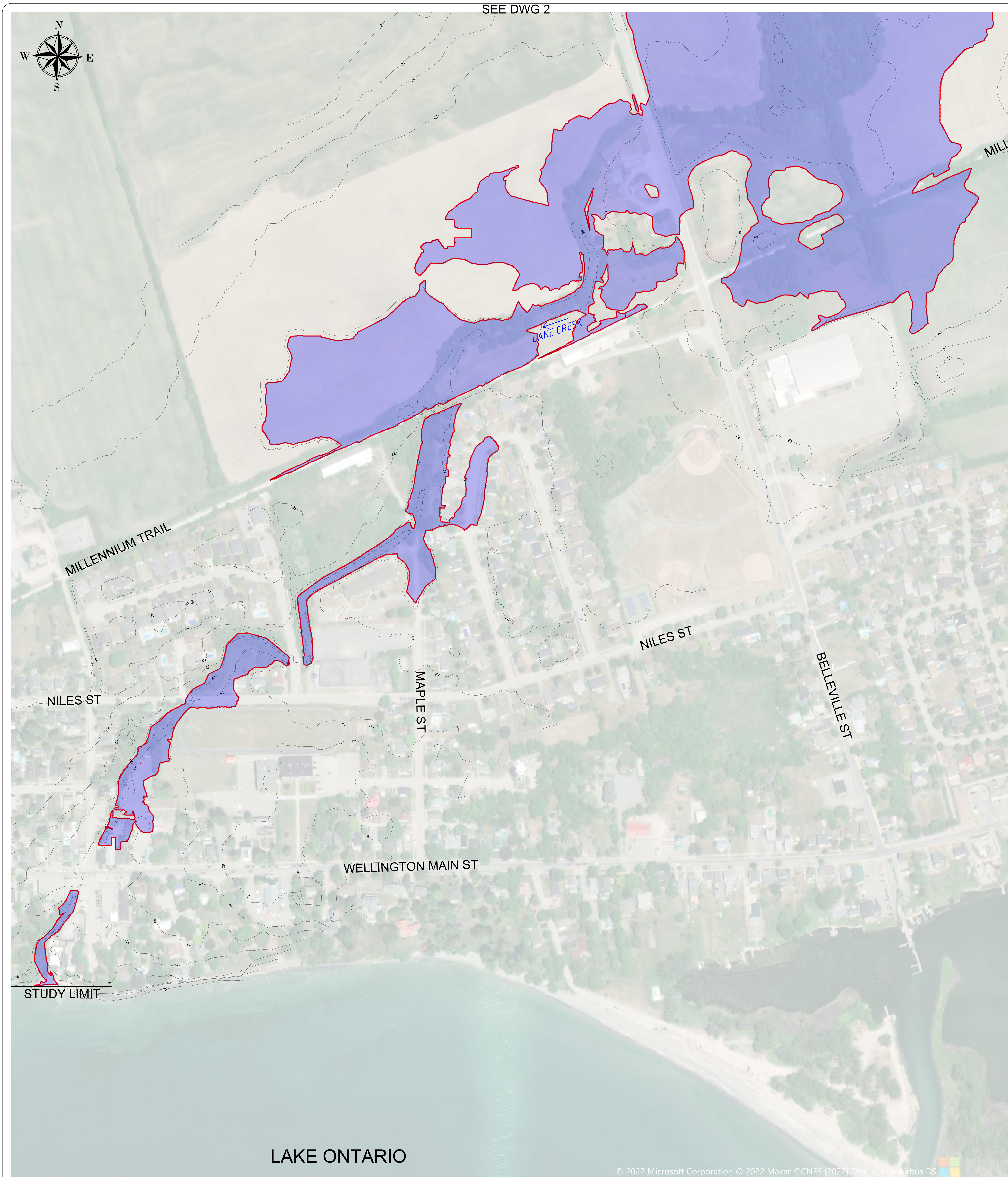
LICENSED PROFESSIONAL ENGINEER
B. C. KEENE
Apr 19/22
PROVINCE OF ONTARIO

QUINTE CONSERVATION
LANE CREEK

WELLINGTON, ONTARIO

25-YEAR
FLOODPLAIN DRAWING

DRAWN BY: MB	PROJECT NO: 210-4981-2
DESIGNED BY: NG & EF	DATE: April 2022
CHECKED BY: EF	CONTRACT NO:
APPROVED BY: EF & BK	
SCALE: HORIZONTAL - 1:2,000 CONTOUR INTERVAL - 1.0m	
	DRAWING NO: 2



JEWELL ENGINEERING

Quinte CONSERVATION

LEGEND:

50-YR FLOODLINE

DATUM: CGVD28

LICENSED PROFESSIONAL ENGINEER
E.J.W. FLEDDERUS
100502141
Apr. 19, 2022
PROVINCE OF ONTARIO

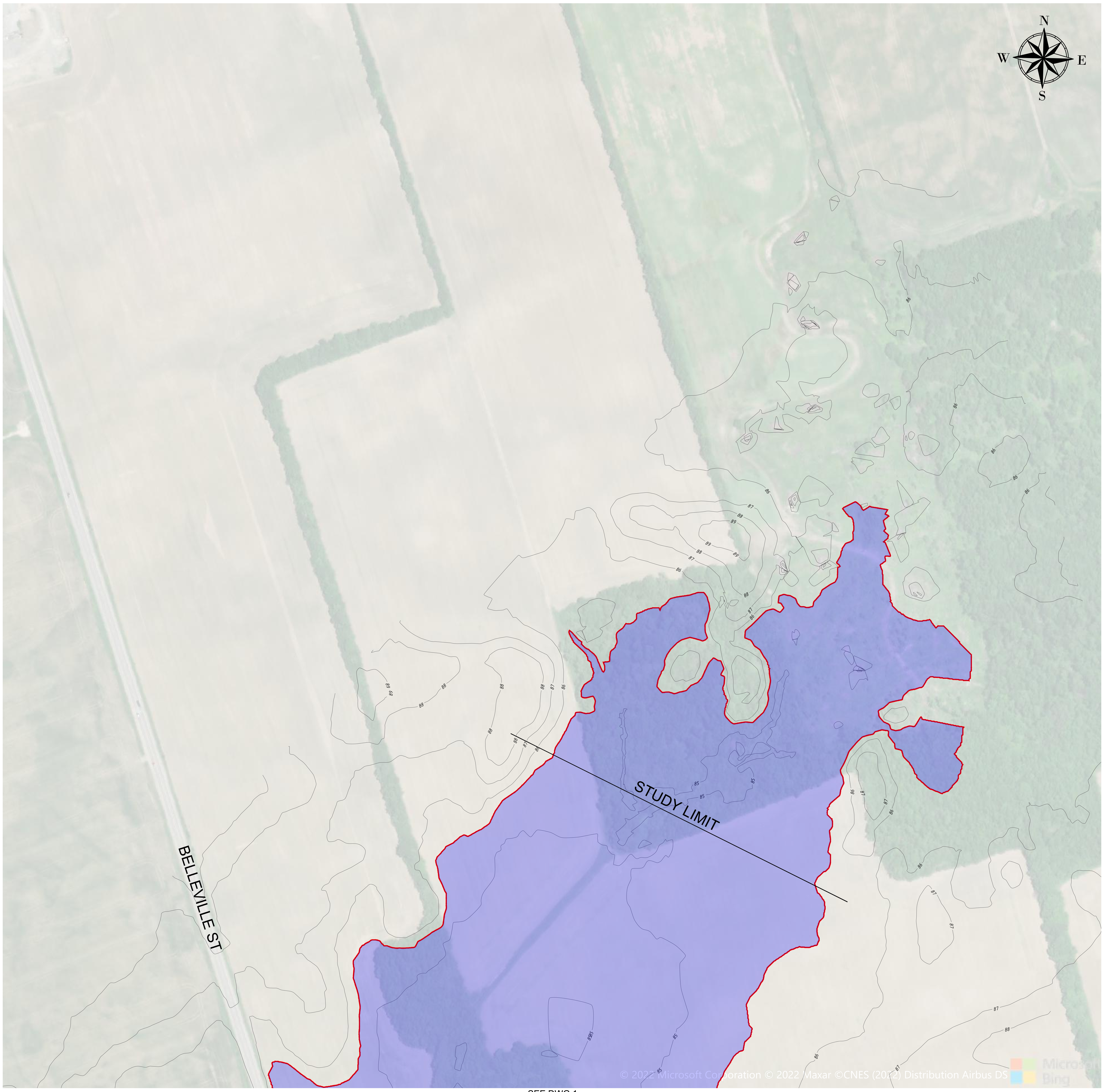
LICENSED PROFESSIONAL ENGINEER
B. C. KEENE
Apr 19/22
PROVINCE OF ONTARIO

QUINTE CONSERVATION
LANE CREEK

WELLINGTON, ONTARIO

50-YEAR
FLOODPLAIN DRAWING

DRAWN BY: MB	PROJECT NO: 210-4981-2
DESIGNED BY: NG & EF	DATE: April 2022
CHECKED BY: EF	CONTRACT NO:
APPROVED BY: EF & BK	
SCALE: HORIZONTAL - 1:2,000 CONTOUR INTERVAL - 1.0m	
	DRAWING NO: 1



SEE DWG 1

JEWELL ENGINEERING

Quinte CONSERVATION

LEGEND:

50-YR FLOODLINE

DATUM: CGVD28

LICENSED PROFESSIONAL ENGINEER
E.J.W. FLEDDERUS
100502141
Apr. 19, 2022
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
B. C. KEENE
Apr 19/22
PROVINCE OF ONTARIO

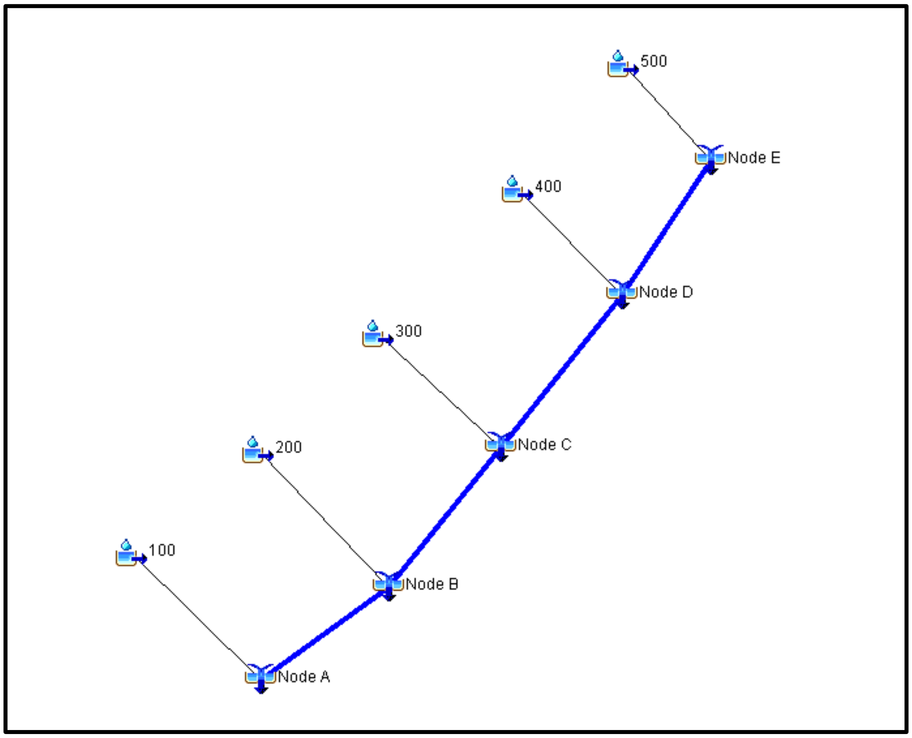
QUINTE CONSERVATION
LANE CREEK

WELLINGTON, ONTARIO

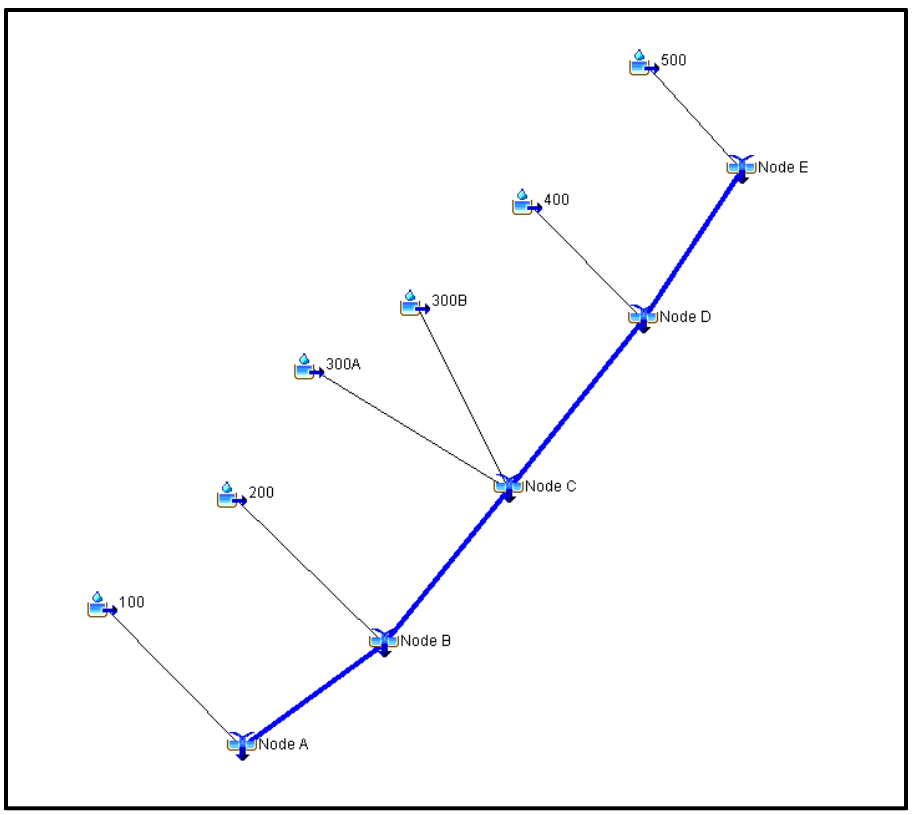
50-YEAR
FLOODPLAIN DRAWING

DRAWN BY: MB	PROJECT NO: 210-4981-2
DESIGNED BY: NG & EF	DATE: April 2022
CHECKED BY: EF	CONTRACT NO:
APPROVED BY: EF & BK	
SCALE: HORIZONTAL - 1:2,000 CONTOUR INTERVAL - 1.0m	
	DRAWING NO: 2

Appendix H – HEC-HMS Schematics and Model (see Attached)



Existing Conditions Model Schematic



Full Build-Out Conditions Model Schematic

Appendix I – HEC-RAS Model (see Attached)