Flood Plain Mapping Study

McLarens Creek

Technical Report

April 2021

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Executive Summary

The primary goals of this study is to generate flood plain maps for the community of Cambray utilizing hydrologic and hydraulic software to computationally assess flows of the McLaren Creek under a variety of storm event conditions including the Regional (Timmins) Storm. The mapping will allow the City of Kawartha Lakes and Kawartha Conservation staff to make informed decisions about future land use and identify flood hazard reduction opportunities.

The McLaren Flood Plain Mapping Study was subject to a comprehensive peer review for core components: data collection, data processing, hydrologic modeling, hydraulic modeling, and map generation. The process was supported throughout by a Technical Committee consisting of technical/managerial staff from the City of Kawartha Lakes, Kawartha Conservation, and the Ganaraska Region Conservation Authority.

Key Findings

- Peak flows from the Timmins Regional storm event exceed peak flows of the 100-yr storm, therefore are used to define the Regulatory flood hazards for McLaren Creek.
- The private dam structure/on-line pond upstream of Cambray Road creates a significant backwater condition under Regional Storm conditions that has the potential to cause flows to spill to the east toward the centre of Cambray. The model was indicating flood elevations that are within centimetres of the grade elevations along portions of the east side of the pond upstream of the dam.
- Both Cambray Road and Elm Tree Road overtop under Regional Storm conditions while 100-year flows are conveyed through each respective structure.
- A minor spill is expected along the east side of Elm Tree Road, opposite Kings Lane as indicated on the mapping.
- A total of 16 structures were found to be located within the flood plain within the study area**.**

Key Recommendations

This study recommends that the proposed flood plain mapping be endorsed and accepted by the Kawartha Conservation Board of Directors and be used to regulate land uses and manage flood hazards within the Community of Cambray.

- Further study is recommended to refine the extent of expected flooding within the village of Cambray utilizing a 2-Dimensional hydraulic model as resources allow.
- This work should be accompanied by additional topographical field survey to confirm and refine the elevations and locations at which such a spill would occur along the east side of the pond and dam upstream of Cambray Road.

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1.0 Introduction

1.1 Study Objective

The objective of this study is to generate flood plain maps for McLaren Creek through the community of Cambray located approximately 10 km northwest of Lindsay. Models were created using hydrologic and hydraulic modelling software and were developed based on provincial guidelines. Data sources used in the model development incorporated soils information, future land-use conditions, aerial photography, survey data for the culvert and bridge structures and utilized the latest LiDAR ground survey to create digital elevation models. The flood plain mapping that has been produced will allow the City of Kawartha Lakes and Kawartha Conservation staff to make informed decisions about future land use and identify flood hazard reduction opportunities within the community of Cambray.

1.2 Study Process

At the commencement of the project, the Technical Committee (consisting of representative from the City of Kawartha Lakes, Kawartha Conservation, and the Ganaraska Region Conservation Authority) created quality assurance (QA) and quality control (QC) processes to be applied to all projects in the multi-year initiative. The QA methodology for each component ensures that the project design meets industry standards and that the work outline and planned deliverables are valid. The three goals of the QC component are that the product is consistent with standards and generally accepted approaches; that the study results meet the Technical Committee's requirements, and that the products and results are scientifically defensible. Each methodology was peer-reviewed for QA and QC by an external firm or agency. Four separate components of the project were established for QA and QC.

- Elevation data and Orthoimagery
- Survey data collection and integration
- Hydrology modeling
- Hydraulic modeling

For the Q/A portion of the hydrology and hydraulic modeling components, a hydraulic/hydrologic modeling procedures document was created that established data input parameters to meeting municipal and provincial standards; put in place data collection and extraction procedures; and short-listed computer models. The document was peer-reviewed by Greck and Associates Limited and was found to be satisfactory.

Details of the study are separated into four primary sections.

- Hydrology
- Hydraulics
- Report
- Flood plain Mapping

1.3 Study Area

The contributing drainage area of McLaren Creek under this flood plain study is 33 square kilometers (km^2), as illustrated in **Figure 1.0**. The drainage area contains a large wetland of approximately 7 km^2 just north of Cambray, which is surrounding Goose Lake. This water feature plays an important role in reducing flood peaks due to its significant capacity to store water. In addition, a small dam structure north of Cambray Road maintains a small amount of storage that is used for private agricultural uses but does not provide any flood storage function. The land use for the McLaren Creek watershed predominantly consists of agricultural and forested lands. The community of Cambray consists largely of low density development within the watershed located just below the dam.

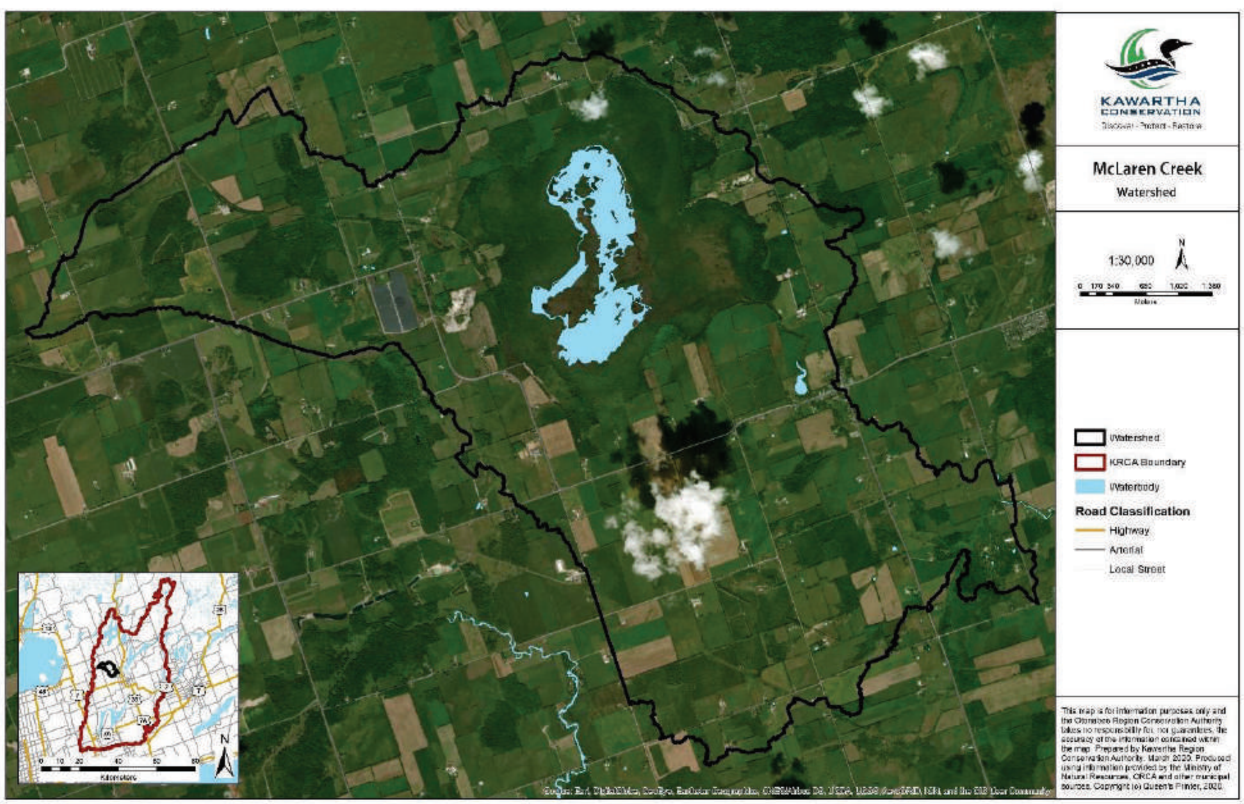


Figure 1.0 McLaren Watershed-Study Area

1.4 Background Studies

Table 1.0 provides a chronology of the previous related reports and/or mapping that have been created for the study area. As noted in the Table, previous flood plain mapping studies for the area have not been produced and the only mapping that exists is related to estimates of natural hazard areas for watercourses and wetland features regulated under the Conservation Authorities Act. See **Appendix C** for scanned copies previous natural hazard mapping.

Table 1.0 Previous Reports on Mariposa Flood Plain

Report/Study	Description	Author
Fill Line Mapping Project	Establishment of "Fill, Construction, and Alteration to Waterways" regulations under Section 28 of the Conservation Authorities Act	KRCA (1989)
Generic Regulation Update	Updated mapping of hazard areas in support of "Ontario Regulation 182/06: Regulation of Development, Interference with Wetlands and Alterations to Shorelines and Watercourses"	KRCA (2006)

1.5 Modeling Approach

Flood plain estimates were assessed by deriving peak flows using the Soil Conservation Service (SCS) standard unsteady flow methods using Visual OTTHYMO Suite 5.0 (VO5) and conducting standard step steady flow methods using HEC-RAS version 5.0.7. A hydrological model was set up using the single-event approach to estimate flood flows at key location along McLaren Creek. The provincial guidelines (MNR, 1986, 2002) discuss this method as the return period design storm method. For this study specific design rainfall events utilizing prescribed return periods are applied to generate respective peak flows within the hydrological system. As such, runoff hydrographs have been generated for the 2-year to 100-year and Regional (Timmins) storm events. The source rainfall data used for this analysis (for 2-year through 100-year) is from Environment Canada's rain gauge that was historically located at the Lindsay Filtration Plant. The observed rainfall data for the Timmins Storm is defined in Table D-4 of the document "MNRF River and Stream Systems: Flooding Hazard Limit" (2002).

In order to determine the suitable 100 year design storm within the study area a comparison of following peak flows was carried out for following design storms distributions:

1. Atmospheric Environment Service (AES) – 100yr, 6hrs, 12hrs, 24hrs
2. Chicago (CHI) – 100yr, 6hrs, 12hrs, 24 hrs
3. Soil Conservation Service (SCS) – 100yr, 6hrs, 12hrs, 24 hrs

The highest peak flows were returned applying the AES-100yr 6hr storm event and was therefore applied for further flood assessment.

Official Plans of the City of Kawartha Lakes were consulted to derive land use designations within the study area. This information (residential area, industrial, commercial, wetlands, rural, etc.) is available in digital form and is therefore utilized within Geographic Information Systems (GIS) to extract the data for each sub catchment delineated to obtain the parameters described in the Hydrology Modeling Parameters Selection document (refer to **Appendix A**). Values such as imperviousness, SCS Curve Numbers (CN), time to peak (Tp) and time of concentration (Tc) are then calculated. The methodology is discussed further in Section 3.3.

To delineate sub-catchment areas a digital terrain model (DTM) was used to apply the ArcHydro Tool within the GIS environment. The methodology is described further in Section 3.2 'Digital Terrain Model and Orthoimagery'. The GIS catchment delineation was refined and adjusted with input from the professional engineer.

Among the available runoff-generating modules in the Otthymo model software, only the NASHYD command is considered for calculating runoff from rural catchments. The NASHYD command is generally used for rural areas with imperviousness levels less than 20%, which is the predominant condition for the McLaren study area. The hydrological model input requirements for this command require the selection of a Curve Number (CN) value, which is a function of land use and hydrologic soil groups (HSG). The CN calculation is based on the following approach:

An area-weighted CN value is calculated utilizing land use and hydrologic soil group data for each sub-catchment delineated. A CN is derived for each land use and is then weighted to find the aggregate CN for the entire sub catchment. A conversion of the resultant CN is carried out from CN to modified CN (CN*) using the procedure outlined within the VO5 Reference Manual.

Time to Peak

The time to peak (Tp) is defined as the time between the onset of the rainfall event and the corresponding peak flow within the catchment. Time to peak is calculated based on time of concentration (Tc). The Tc of a catchment is defined as the time it takes for the resulting runoff to move from the furthest catchment boundary to its outlet.

Tc is calculated first followed by calculating the Tp based on the equation:

$Tp=(N-1)/N*Tc$ or $Tp=0.67Tc$.

Various methods are described in the literature and the VO5 reference manual. There are number of different methods used to calculate *Tc* and/or *Tp*, including the following:

- Upland's Method
- Bransby- Williams Method
- Airport Method
- Watt and Chow Method; and
- HYMO Method

For this study, either the Airport method or the Bransby-Williams method was employed. The Airport method was used when C (runoff coefficient) < 0.4 and the Bransby-Williams Method was used when C (runoff coefficient) > 0.4 as outlined in formulas 8.15 and 8.16 from the 1997 MTO Drainage Manual.

Sensitivity analysis was carried out as part of study to determine the impact of changing model input parameters on the flows generated through model output. This approach was peer-reviewed by Greck and Associates Limited in August 2013 and was found to be acceptable, as documented in the separate report titled *Peer Review Services for Terms of Reference of Hydrologic and Hydraulic Assessments, Final Report.* Where not specified, default parameters/values were used within Visual OTTHYMO and HEC-RAS models and modified where appropriate. This approach results in realistic peak flows and associated floodlines along the watercourse in the study area. No stream gauging or flow monitoring data is available within the study area to calibrate the hydrologic model.

2.0 Rainfall

2.1 Rainfall Data

Rainfall Intensity–Duration–Frequency (IDF) curves were used to extract relevant local rainfall characteristics. IDF curves describe the relationship between rainfall intensity, rainfall duration and return period. Rainfall volumes were taken from Lindsay's Atmospheric Environment Services (AES) gauge which was removed from service in 1989. In the initial flood plain study, carried out for Ops #1/Jennings Creek, an investigation was carried out to determine the relevancy of using data from this inactive rain gauge. The Peterborough AES rain gauge has a longer period of record and has captured higher rainfall volumes than what was captured by the Lindsay rain gauge. It is unknown whether this increase is attributable to Peterborough's longer period of data capture (36 years, from 1971 to 2006 vs. Lindsay's 24 years, from 1965-1989) or to the effects of climate change. After completing some sensitivity analyses on the rainfall data it was decided that the Lindsay AES gauge data was appropriate for use in the Ops #1/Jennings Creek study. It was further decided that for all subsequent flood plain studies, the Lindsay IDF data would be used to provide continuity from study to study and to ensure consistency in the sizing of infrastructure. Further details regarding the assessment of the two gauging stations is provided in **Appendix B**.

Detailed rainfall information is provided in **Appendix B**. Rainfall intensity is calculated by the formula:

$I = a/(t+b)^c$, where
I in mm/hr
t in minutes

The IDF data used is presented in **Table 2.0 and Table 2.1**

Table 2.0 IDF Parameters calculated by Kawartha Conservation

Return Period (yr)	a	b	c
2	808.299	7.413	0.835
5	1248.097	9.760	0.857
10	1486.792	10.44	0.859
25	1917.848	11.842	0.873
50	2142.007	12.182	0.872
100	2465.522	12.897	0.879

Table 2.1 Rainfall Depths from Lindsay AES Station (24 years of data)

Return Period (yr)	6-hour (mm)	12-hour (mm)	24-hour (mm)
2	36.6	39.8	43.6
5	50.8	53.2	56.4
10	60.2	62.2	64.8
25	72.1	73.4	75.4
50	80.9	81.8	83.3
100	89.7	90.1	91.2

2.2 Design Storms

Design storms are characterized by three elements: total volume, storm duration, and rainfall distribution.

Total Volume

Section 2.1 discussed the volumes collected by the Lindsay AES gauge that are used in this study.

Storm Duration

Watershed drainage areas and the conveyance of flood flows respond differently to different rainfall durations. As such, a variety of rainfall durations (6, 12, and 24 hours) for 2-100 year return periods were tested. For the 100-year event, 6, 12 and 24 hour durations were tested. Short duration design storms typically have greater rainfall intensities and lower total rainfall volumes compared to longer duration storms.

Storm Distribution

How the rainfall is distributed over time for a given duration can also influence rates of surface runoff. Various distributions of rainfall have been derived from historical data and are typically tested to examine the watershed's response. It is standard practice to test different design storms to determine the most conservative flows. The most common distributions examined in southern Ontario include the SCS Type II, Chicago and AES.

For over a century, the American Natural Resources Conservation Service has continually refined empirical formulas for the Soil Conservation Service (SCS) method of predicting storms. Their SCS Type II distribution represents a high-intensity storm based on a 24-hour rainfall, and can be used in hydrology studies in Southern Ontario. The bulk of the rainfall occurs in the second half of the storm.

Environment Canada's AES has developed a design storm for southern Ontario. When compared to the SCS distribution, the majority of the rainfall in the AES storm occurs at the beginning of the storm. The Southern Ontario 30% curve is used in this study.

The worst case storm (the duration and distribution producing the highest discharges at key nodes) is selected as the critical event for the watershed. **Tables 2.2** to **2.4** show the worst case storm (100 yr-AES-6hr) producing the highest flows at key location of the watershed.

Table 2.2 6, 12, 24hr 100-yr-Chicago

Key Nodes	100yr CHI 6hr	100yr CHI 12hr	100yr CHI 24hr
30000	13.21	14.11	15.02
50000	13.49	14.39	15.29
60000	13.07	14.15	15.03
90000	19.98	21.79	23.09
110000	23.26	24.89	26.30

Table 2.3 6, 12, 24hr 100-yr-AES

Key Nodes	100yr-AES-6hr	100yr-AES-12hr	100yr-AES-24hr
30000	16.67	14.19	10.15
50000	16.92	14.54	10.47
60000	16.08	14.29	11.02
90000	24.25	22.07	17.45
110000	29.40	26.95	21.92

Table 2.4 6, 12, 24hr 100-yr-SCS

Key Nodes	100yr-SCS-6hr	100yr-SCS-12hr	100yr-SCS-24hr
30000	16.28	13.82	11.68
50000	16.56	14.15	11.98
60000	15.84	14.13	12.05
90000	24.03	21.93	18.76
110000	28.25	25.45	22.33

The Timmins storm was a historical storm event that occurred in September 1961 and is designated as the provincial Regional Storm event within the subject area. The observed Timmins storm event resulted in a total rainfall of 193 mm and is defined in Table D-4 of the document "MNRF River and Stream Systems: Flooding Hazard Limit" (2002).

The Regional (Timmins) Storm and the range of design storms were analyzed as part of this study. The Regulatory Storm Event is defined as the storm event that produces the greatest level of flooding between the Regional (Timmins storm) and the 100-year event. In all areas throughout the watershed, the Timmins Storm produces the Regulatory Flood flows for the watershed.

Rainfall events can have significant variation throughout a watershed as the entire watershed does not receive the same rainfall at a constant rate. Typically, an areal reduction factor, based on the watershed size, is applied to rainfall intensity for the Regional Storm to estimate the variation of rainfall intensities throughout the watershed. The following reduction factors have been applied as per Section 1.2 and Table D-5 of the MNRF Technical Guide – River and Stream Systems. Flooding Hazard Limits:

Table 2.5 Areal Reduction Factors Applied to the Rainfall for the Study as Per MNRF Table D-5

Location	Approx. Diameter (m)	Equivalent Circular Area (km2)	Reduction Factor (per table D-5 of MNRF Guidelines)
Top of System (Node 30000)	7.10	39.59	97
Node 50000	7.93	49.44	97
Cambray Road	8.21	52.94	94
Kings Lane	8.81	60.96	94
Elm Tree Road	9.32	68.22	94

3.0 Hydrology Model Input Parameters

3.1 Overview

In 2012, the City of Kawartha Lakes and Kawartha Conservation produced a standardized methodology for undertaking their flood plain mapping studies. This approach was peer-reviewed by Greck and Associates Limited, and their findings concluded that the methodology is valid. All hydrology modelling parameters are presented in **Appendix A** unless otherwise noted. For this study, Kawartha Conservation extracted hydrologic parameters from a combination of LiDAR and pixel-auto correlated elevation data, Arc Hydro watershed boundaries, Official Plan, and field surveys.

The purpose of the hydrological modeling is to determine the peak flows that occur at key points along McLaren Creek such that these flows can be used in the hydraulic model. The determination of peak flows requires rainfall information and a variety of parameters (such as land use, soils information, etc.) to characterize the response of the ground surface to varying rainfall intensities. As mentioned above, in most of the areas throughout the watershed, the Timmins Storm with the appropriate areal reduction factor would generate Regulatory Flood flows for the watershed.

Visual Otthymo v.5.1 was selected as the hydrologic model for this project. **Figure 3.0** contains a detailed schematic of the hydrology model.

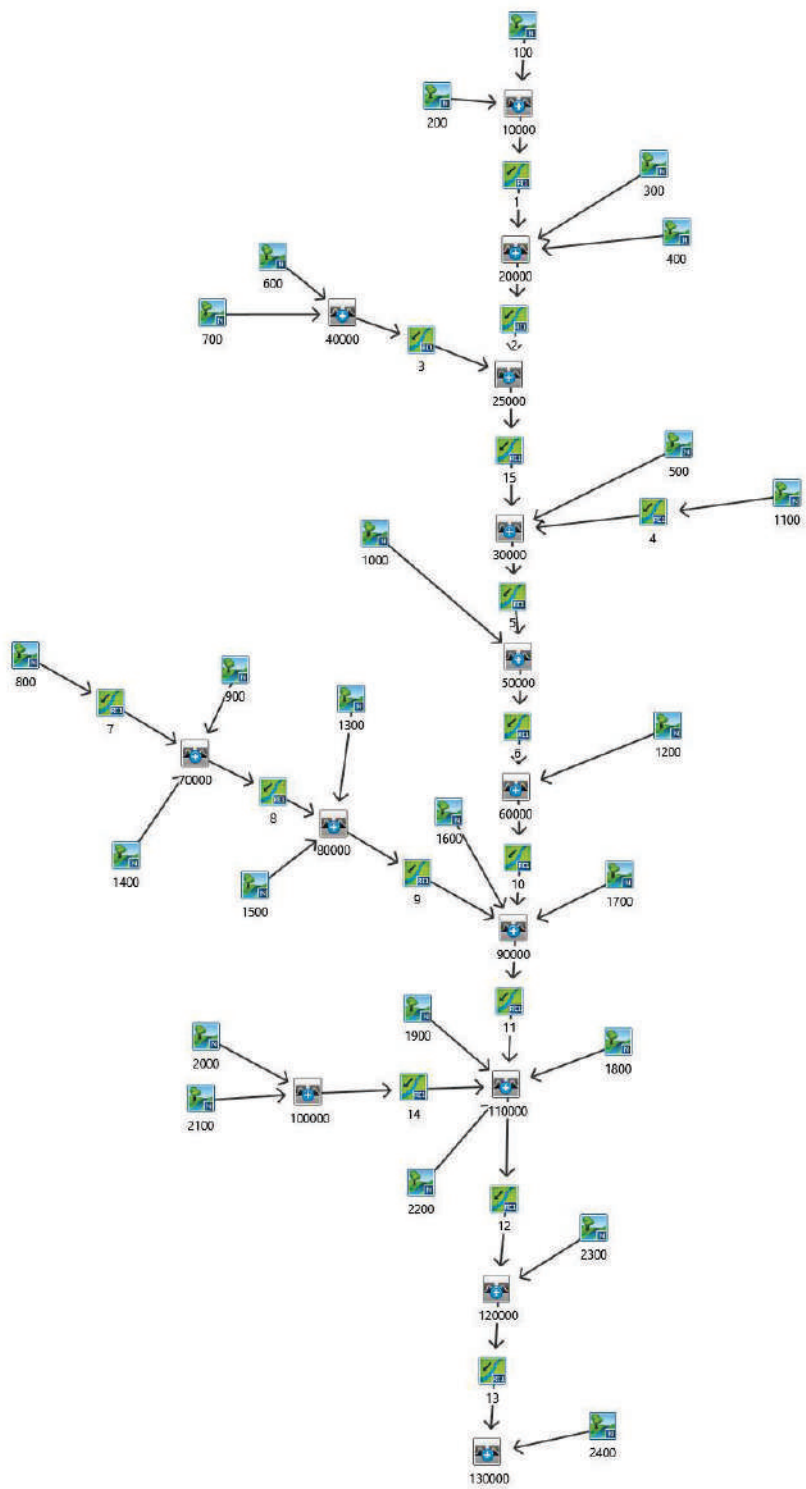


Figure 3.0 Detailed Schematic of McLaren VO5 Hydrologic Model

3.2 Digital Terrain Model (DTM) and Orthoimagery

The fundamental dataset which underlies all stages of any floodplain mapping project is digital topographic base data with full coverage of the study area. Topographic data for the McLaren Creek was obtained from the City of Kawartha Lakes and Land Information Ontario. This topographic data was received in the form of two digital terrain models (DTM) with one produced using Light Detection and Ranging (LIDAR) data acquired in Fall 2012, and one produced from existing pixel-autocorrelation elevation data holdings derived from South Central Ontario Orthophotography Project 2013 (SCOOP2013) acquisition deliverables. The SCOOP2013-derived DTM was used purely for hydrology purposes whereas the LiDAR-derived DTM was used to support hydraulic modeling. A DTM is a 3D topographic representation of a bare earth surface; all vegetation and buildings are removed by way of post-processing of the LiDAR data Example of the digital topographic data are found in **Figures 3.1** through to **3.9**.

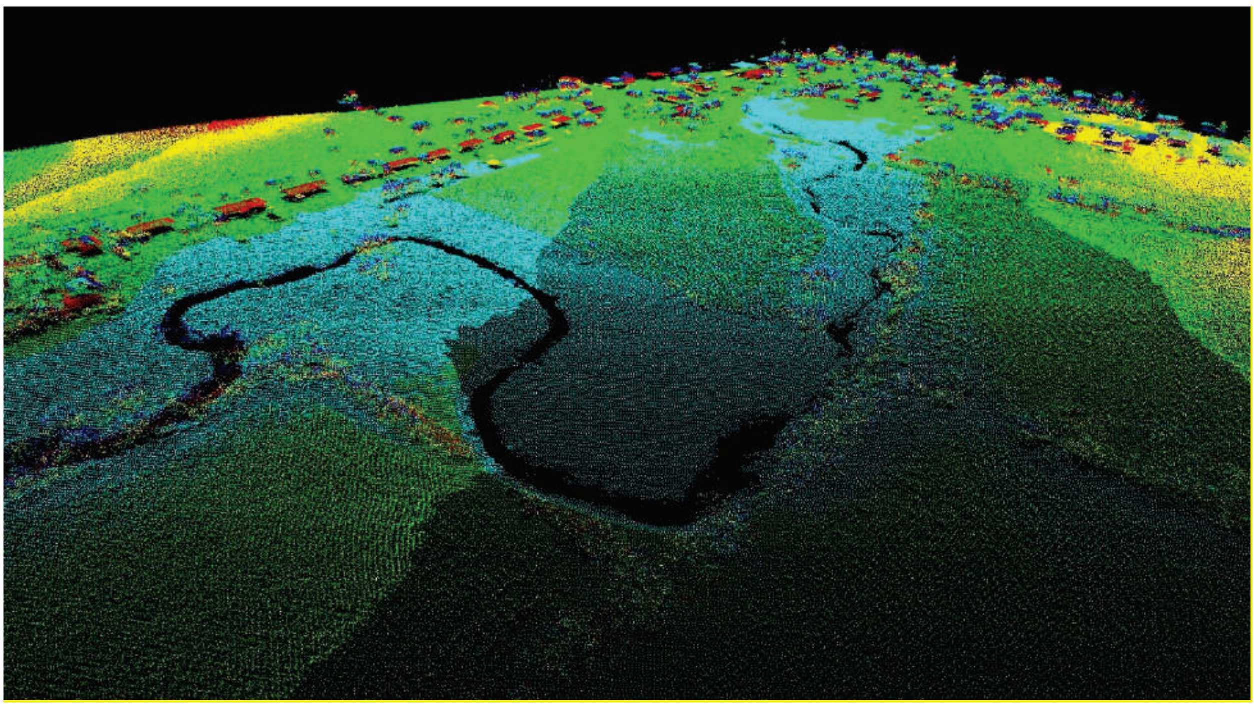


Figure 3.1 Classified LiDAR Point Cloud

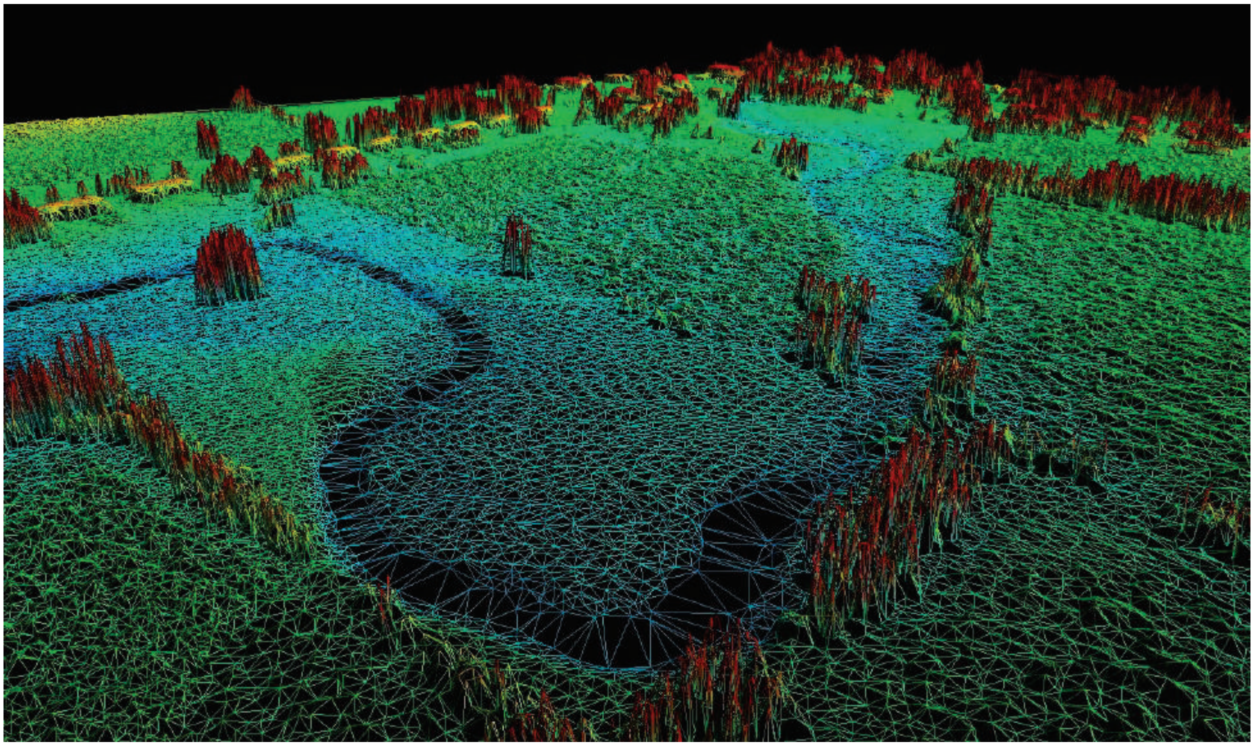


Figure 3.2 Triangular Irregular Network (TIN) Produced From LiDAR Point Cloud

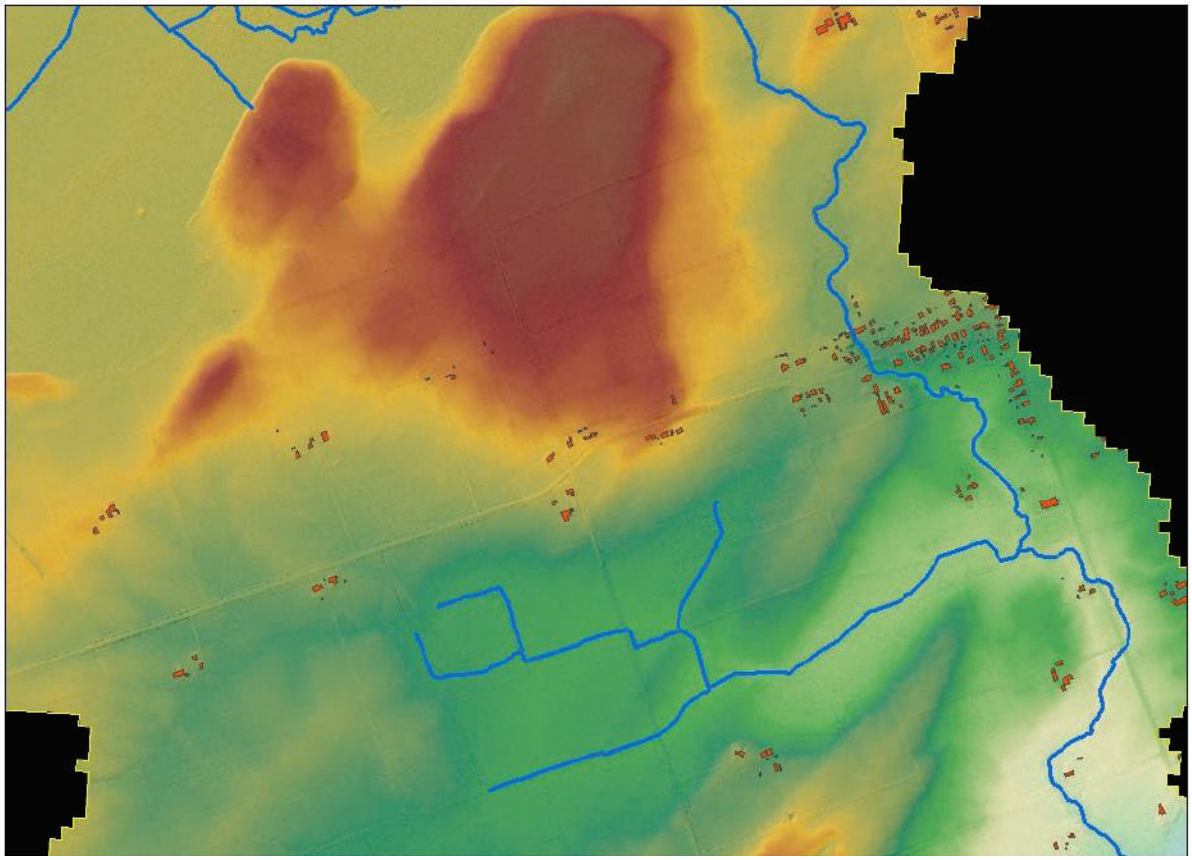


Figure 3.3 Digital Terrain Model (DTM) Produced From LiDAR Point Cloud with Building Footprints (Orange) and Watercourses (Blue) Overlain.

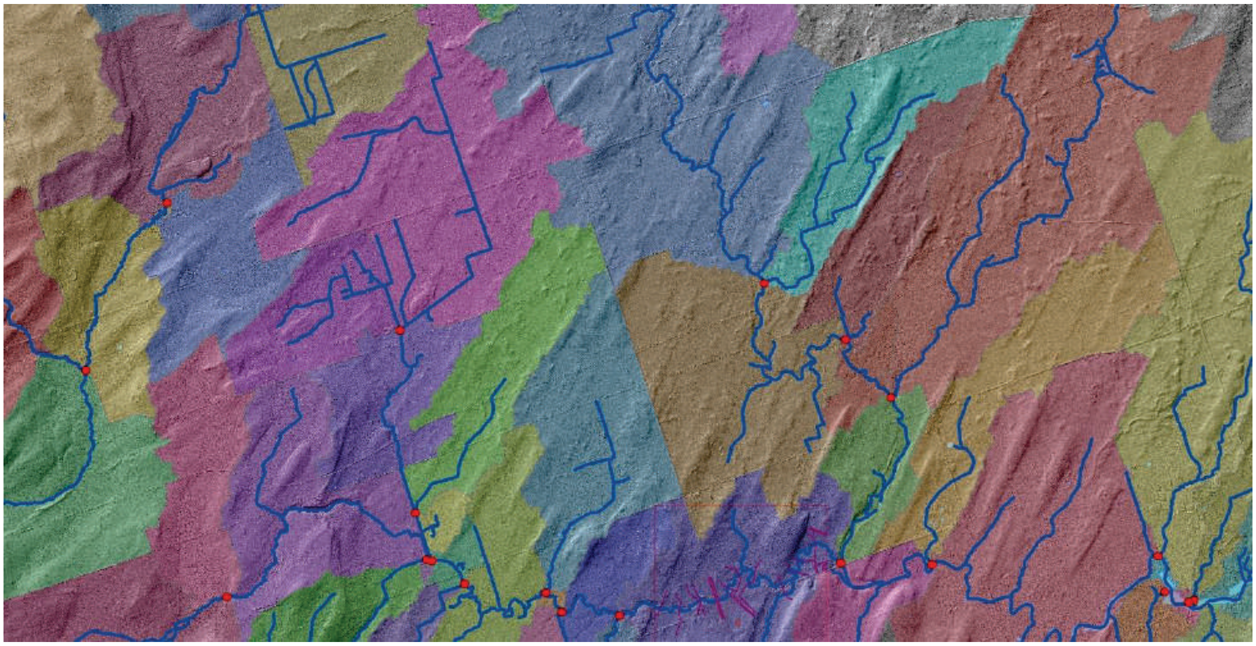


Figure 3.4 Hydrology Subcatchments Overlain On Hydrologically-Conditioned DTM Used to Delineate Into Polygon Layer

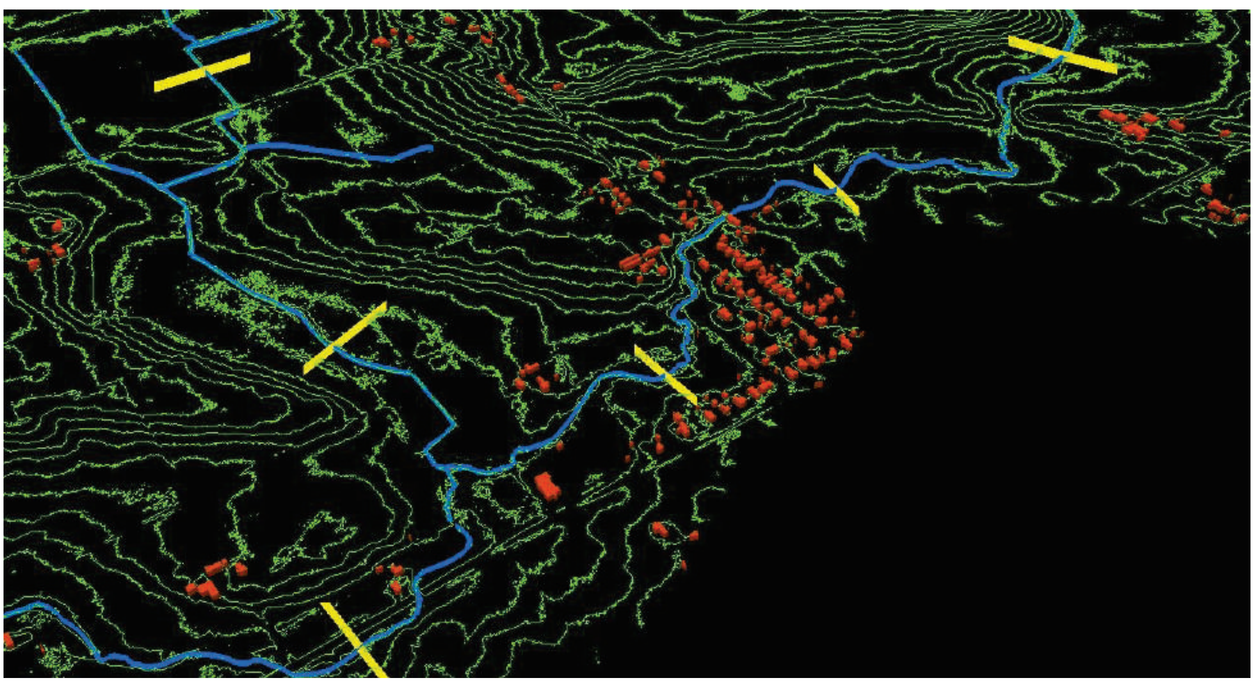


Figure 3.5 DTM-Derived 3D Contours With DTM-Derived Hydraulic Cross-Sections (Yellow), 3D Building Footprints (Orange), and Watercourses (Blue).

With the aid of GIS software, the SCOOP2013-derived DTM was used to produce geospatial data required for hydrologic and hydraulic modeling. For hydrologic modeling, this 3D data was post-processed in order to delineate subcatchment drainage areas, runoff lengths and slopes for runoff rate calculations, among other input geospatial data. The LiDAR-derived DTM was used to define the overbank portions of cross sections for input into the hydraulic model as well as the base dataset upon which the resultant flood lines are delineated. Coordinates used throughout this study are expressed using NAD83 (CSRS) horizontal datum and CGVD28 vertical datum. All future development proposals within the regulated area of McLaren Creek will need to be presented on the same coordinate system and datum to ensure a direct comparison, including referencing a control monument of appropriate accuracy. The subcatchment boundaries and labels are indicated in **Figure 3.12**

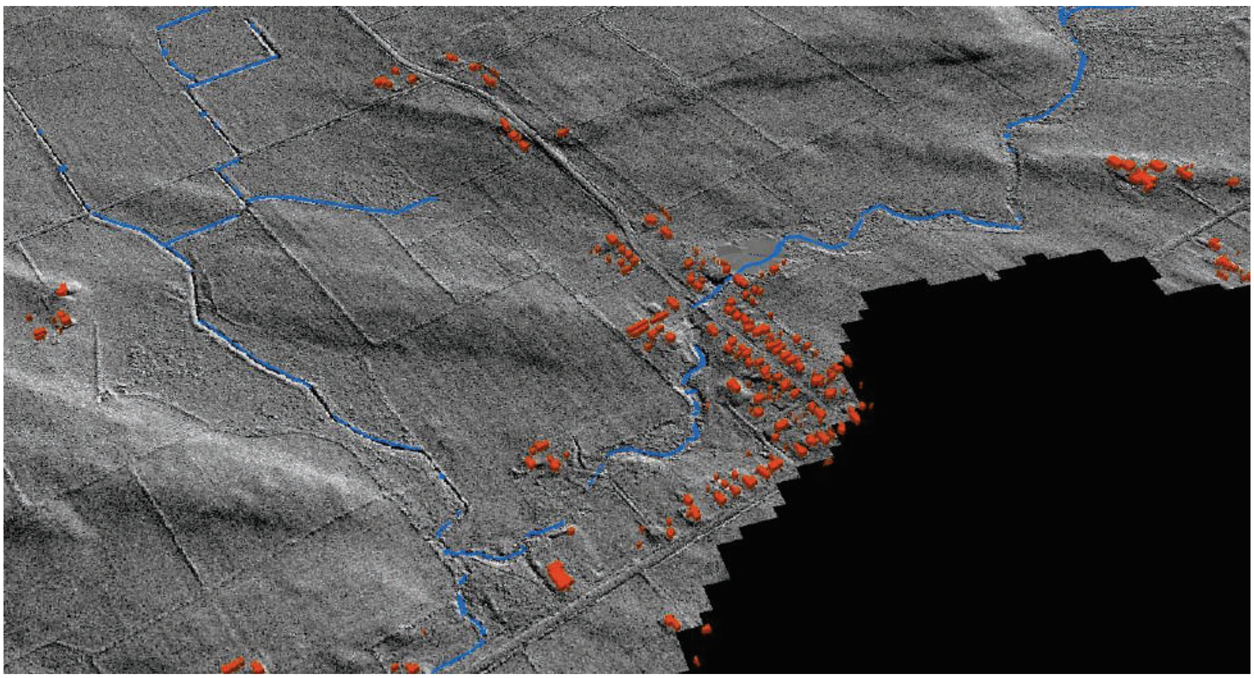


Figure 3.6 Oblique Rendering of DTM Hillshade With Buildings (Orange) and Watercourse (Blue)

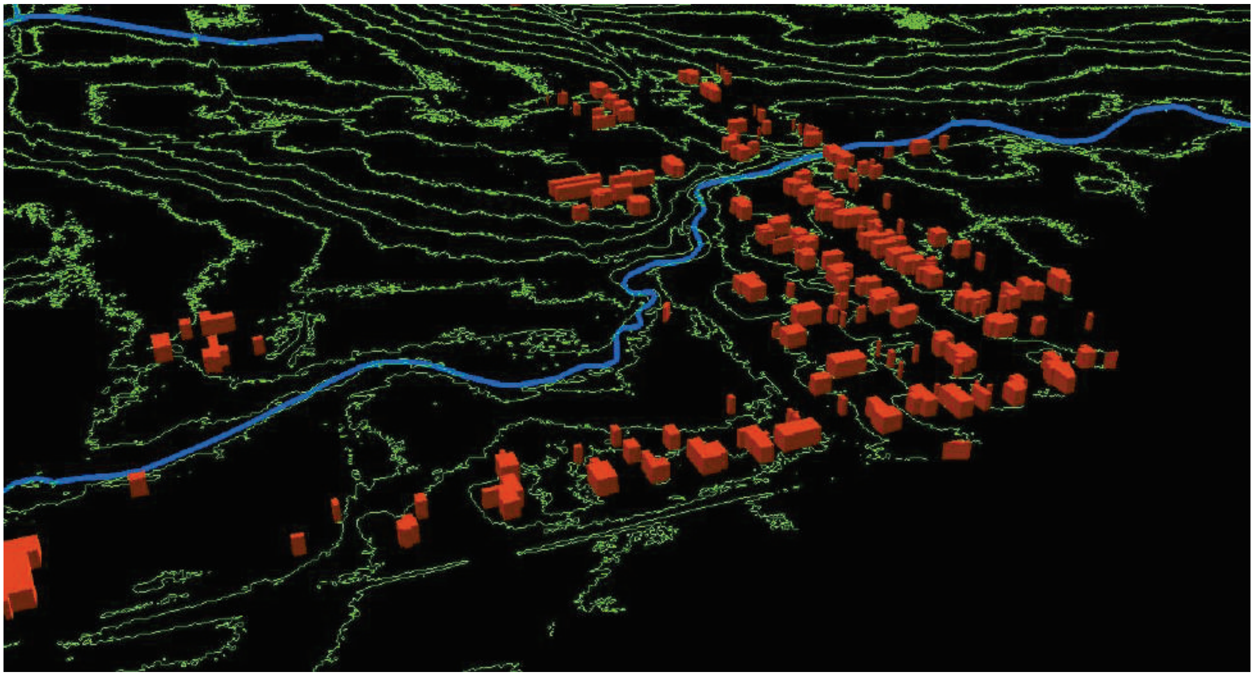


Figure 3.7 Oblique Rendering of DTM-Derived Contours With 3D Buildings (Orange) and Watercourses (Blue)

Figure 3.8 Oblique Rendering of 3D Orthoimagery With Buildings (Orange) and Watercourses (Blue)

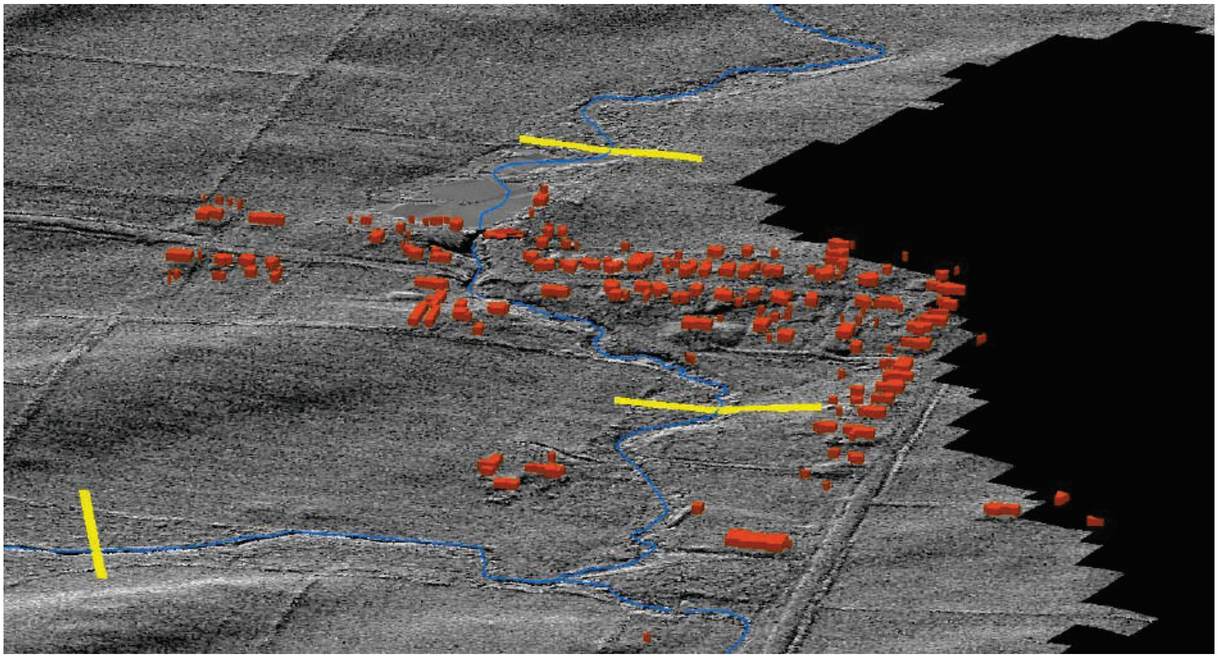


Figure 3.9 Oblique Rendering of DTM Hillshade With DTM-Derived 3D Hydraulic Cross-Sections (Pink)

Orthoimagery acquired through the Provincial Imagery Strategy and obtained through Land Information Ontario – namely, the South Central Ontario Orthophotography Project 2013 (SCOOP2013) – was used as best available full-coverage aerial imagery for the project area.

The DTM and Orthoimagery used in the project underwent a rigorous independent accuracy assessment. For further information and results, see ***Appendix K****: Digital Terrain Model and Orthoimagery Data Accuracy Assessment Report*.

3.3 Land Use

For this study, the Kawartha Conservation 2010 ELC (Ecological Land Classification), Secondary Plan and Official Plan (OP) data from the City of Kawartha Lakes was queried to extract land use data. Schedules 'B to G' for the Fenelon Township from 06 June 2014 Land Use map version as delivered by City of Kawartha Lakes was also used to confirm the land use. A copy of the OP and zoning bylaw schedules are provided in **Appendix H**.

Land uses in the hydrology model do not reflect current land use within the subcatchment boundaries; instead, the model assumes that all developable areas indicated in the Official Plan/Secondary Plan are fully built out. The rationale for this decision is that the municipality has approved in principle the proposed land use and therefore the catchment hydrology and corresponding flood lines should reflect the most conservative flood scenario. The land uses for the study area are indicated in **Figure 3.10**

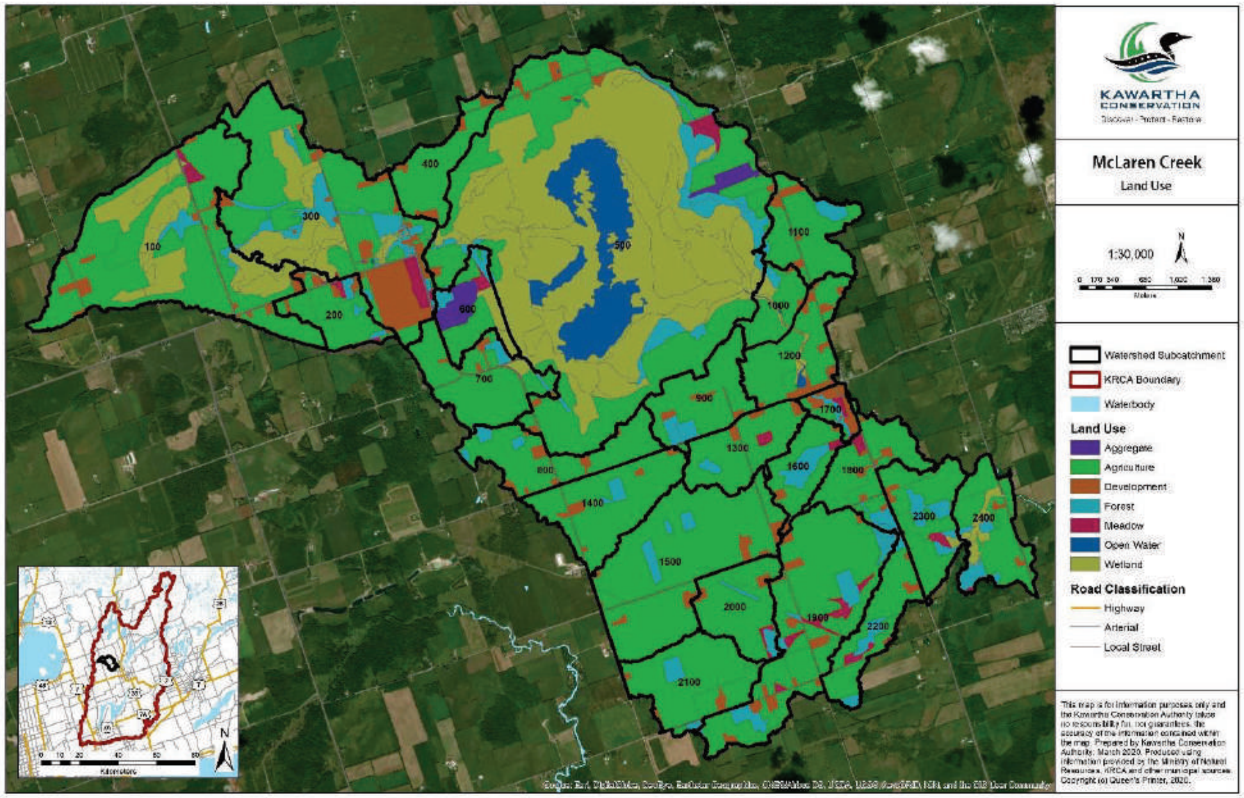


Figure 3.10 McLaren Creek Land Use

3.3.1 Soils

Soils are classified by the Natural Resource Conservation Service into four Hydrologic Soil Groups (HSG) based on soils runoff potential. The four Hydrologic Soils Groups are A, B, C and D with Group A soils being well drained and generally having the smallest runoff potential and Group D soils being poorly drained and have the greatest runoff potential.

Group A is sand, loamy sand or sandy loam types of soils. It has low runoff potential and high infiltration rates even when thoroughly wetted.

Group B is silt loam or loam. It has moderate infiltration rate when thoroughly wetted and consists chiefly or moderately deep to deep, moderately well to well drained soils with moderately fine to moderately coarse textures.

Group C soils are sandy clay loam. They have low infiltration rates when thoroughly wetted and consist chiefly of soils with a layer that impedes downward movement of water and soils with moderately fine structure.

Group D soils are clay loam, silty clay loam, sandy clay, silty clay, or clay. This HSG generally has the highest runoff potential.

The McLaren Creek watershed predominantly consists of drumlinized till plain and clay plain (Map P.2715 of the *Physiography of Southern Ontario*, Ontario Geological Survey), which are categorized as type C in the hydrological soil group classification. This physiography provides the primary source for the basic HSG types located in the subwatershed. Soil classifications for the study area are indicated in **Figure 3.11**. Soil types B, C, D are distributed throughout the subwatershed, whereas the northern portion has some pockets of type A with low runoff and high infiltration compared to the southern portion of the watershed, which is predominantly comprised of HSG types C and D.

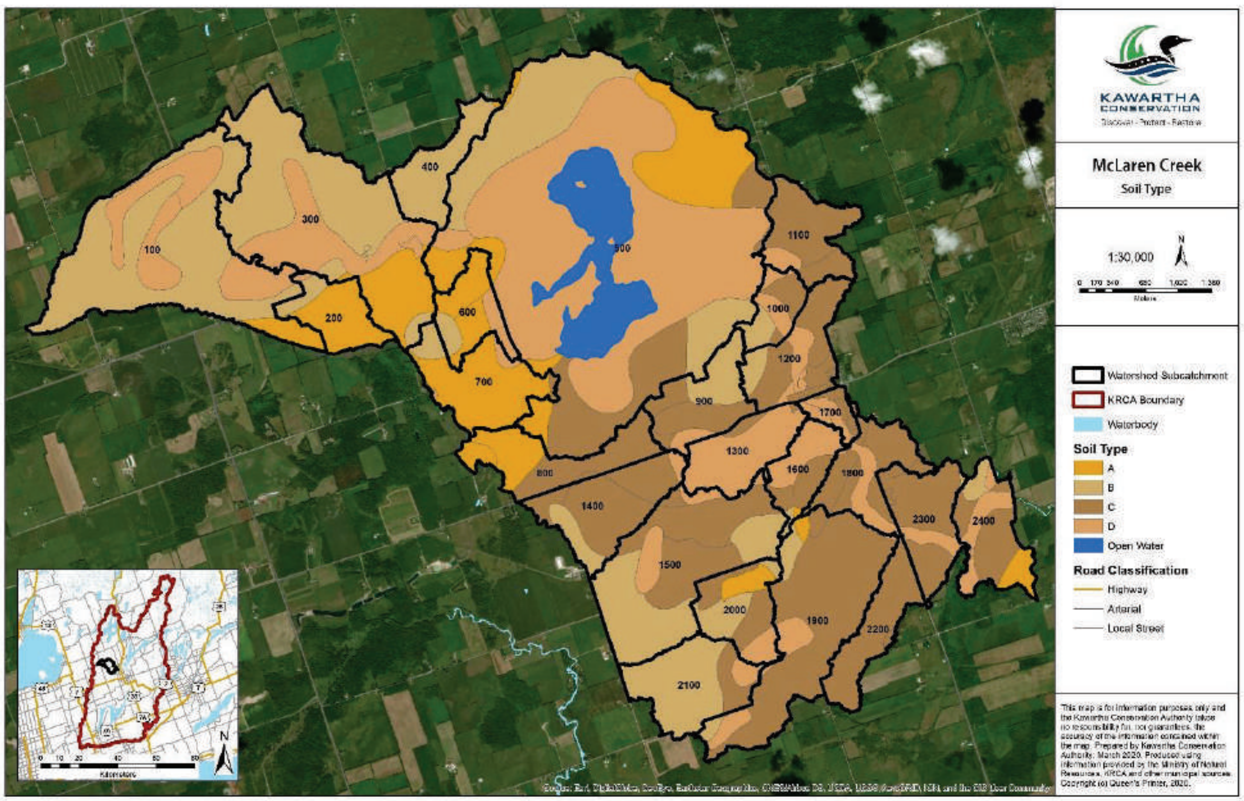


Figure 3.11 McLaren Soil Type

3.3.2 Rural Subcatchment Properties

To calculate runoff in the rural catchments, where the SCS CN method was used, the longest flow path is required. The flow paths were derived using the GIS program ArcHydro. In this process, the downstream node location for each catchment is selected by the engineer using professional judgement, and ArcHydro is used to calculate the longest overland and channel flow paths in order to calculate the Time of Concentration (*Tc*). A review of the automated flow path routes resulted in adjustments where appropriate, which were carried out manually with GIS software under the direction of the engineer for the various catchments. The various subcatchments in the subwatershed are indicated in **Figure 3.12**.

Figure 3.12 Mariposa Brook Subcatchments

3.3.3 Calculation of Slope

Ground slope is required to determine runoff rates in rural and urban catchments. The digital elevation model (DEM 2012) was used to calculate the average ground slope of the longest flow path over the subcatchment. The results are presented in **Appendix D.**

3.3.4 Curve Number (CN) Values

The Soil Conservation Service (SCS) curve number (CN) is used to determine runoff for rural catchments. CN values are based on a combination of land use, underlying soils and antecedent moisture conditions (AMC). A higher CN represent soils with less infiltration potential, while lower CN values represent soils with greater infiltration potential. (AMC). The average antecedent moisture conditions (AMC II) are used for modelling the 2 to 100 year return period design and Timmins storm events.

In Visual OTTHYMO, the rainfall losses in the rural areas are computed by means of the modified curve number procedure (depicted as CN*). The critical storms for rural conditions are long-duration storms such as the Southern Ontario Regional Storm with a peak intensity of 52.83 mm/hr. The modified SCS method (CN*) is used in such conditions, which was first proposed by Paul Wisner & Associates in 1982.

In the SCS runoff equation, it is assumed that the initial abstraction (IA) is set to IA=0.2S, where S is the potential maximum retention value (soil storage). However, it has been found that this IA assumption is an overestimation for conditions in Ontario and consequently the rainfall-runoff responses are underestimated. Based on research by P. Wisner & Associates (1982) who monitored rural and urban catchments in Canada, the modified SCS method is applied in Ontario as it correlates well with observed flows. Rather than having a varying IA parameter, as in the SCS method, the IA is fixed and the CN is altered (hence the name 'modified CN').

For this study, we have used the modified curve number (depicted as CN*) tool within the VO5 program (Section 1.2: Modified Curve Number, CN*, Reference Guide, Visual OTTHYMO, Version 5.1).

3.3.5 Urban Subcatchment Properties

The detailed land uses denoted in the OP (**Appendix H**) were used to determine the weighted total impervious area (Timp), directly-connected impervious area (Ximp) and runoff coefficient (C) for each subcatchment using the tables from the Hydrologic Parameters List in **Appendix A**.

Subcatchments with a Timp value greater than 20% are typically modeled with the StandHyd command. This command is used to determine runoff from urban catchments and makes use of the total impervious (Timp) and directed connected impervious (Ximp) values. However, due to the rural nature of the watershed only the NashHyd command was applied. Spreadsheets with the parameter summaries and calculations are provided in **Appendix A.**

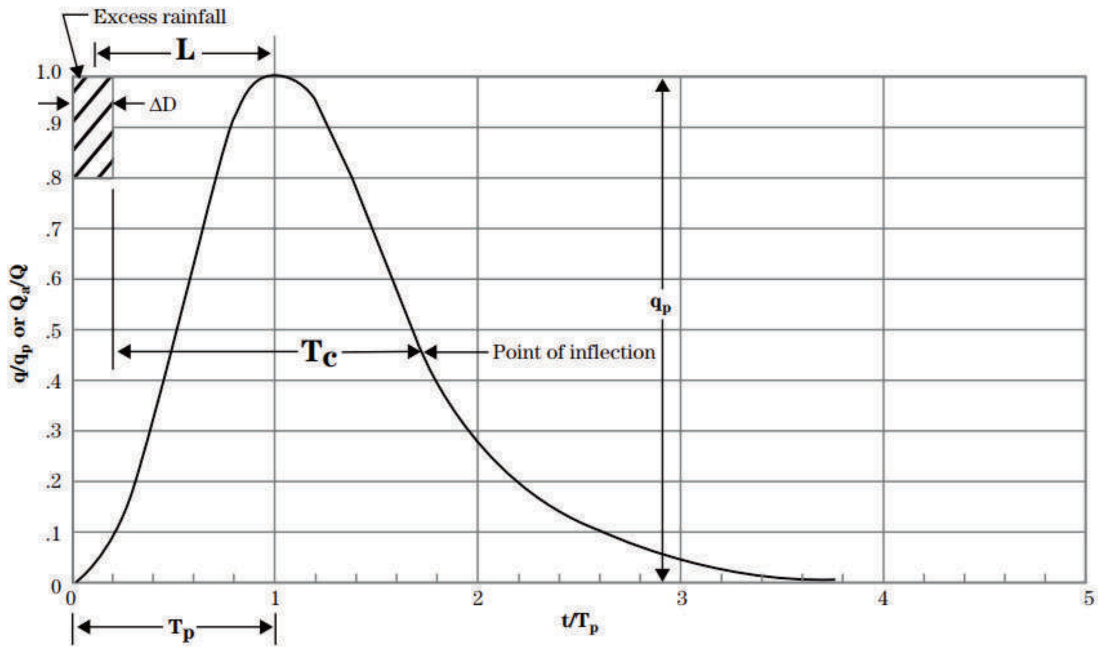
3.3.6 Time of Concentration

Time of concentration (*Tc*) is a key variable for calculating peak flow in the rural catchments. Time of concentration of a watershed is defined as the time required for water to move from the most remote part of the subcatchment to its outlet. This relationship is depicted in the graph in Figure 3.13.

As per industry standards in Southern Ontario, time of concentration was calculated using the Airport method for subcatchments with a C value less than 0.4 and the Bransby-Williams method was chosen if the C value exceeded 0.4.

The Time to Peak (*Tp*) is defined by the equation: $Tp = (2/3) * Tc$

Time to peak is used in the NashHYD command only. Spreadsheets with the *Tc* and *Tp* calculations are found in **Appendix D**, using the flow lengths shown in the subcatchment figures found in **Appendix E**.



where:

L = Lag, h

T_c = time of concentration, h

T_p = time to peak, h

ΔD = duration of excess rainfall, h

t/T_p = dimensionless ratio of any time to time to peak

q = discharge rate at time t, ft^3/s

q_p = peak discharge rate at time T_p, ft^3/s

Q_a = runoff volume up to t, in

Q = total runoff volume, in

Figure 3.13 Example of the relation of time of concentration and lag time to the dimensionless unit hydrograph (Figure 15–3, from: chapter 15, National Engineering Handbook, Part 630, May 2010)

3.3.7 Channel Flow Routing

The storage in the channel has a major impact on hydrographs by reducing or lagging the peaks and redistributing the hydrograph volume. Factors impacting the shape of the hydrograph are channel slope, roughness and shape as well as available storage between two points along the channel. Thus, channel routing in this study was used to appropriately simulate the flood wave travel times and attenuation of peak discharge propagate downstream.

Channel routing in VO5 accounts for the time lag due to the storage of flows as they are conveyed within the main channel and associated flood plain. Channel flow routing was performed by the ROUTE CHANNEL command. Input data required include channel length and slope, representative cross sections and Manning's n values. The watercourse length was measured in ArcGIS. Channel slope was calculated from upstream and downstream watercourse centreline elevations extracted from the DEM. Although these are not true ground elevations because LiDAR cannot penetrate water, they can still provide the relative elevation difference needed to calculate slope. One or two representative cross sections per channel reach were cut from the DEM with the in-channel elevation data replaced with survey data where available.

3.4 GIS Application

An easy to use GIS Application has been developed to illustrate and analyze the following layers:

- Land use
- Soil classification
- Hydrology
- DTM
- Routing paths
- Elevation at start and endpoints for routing parameters
- Multiple map images
- Multiple tools to measure distance and area

The link to the McLaren Creek Application [App] is:

http://camaps.maps.arcgis.com/apps/webappviewer/index.html?id=614e6f215d794b609ea292b94ccdb4a3

Access to the app can be made available upon request.

3.5 Other Considerations

Stormwater Management (SWM) Facilities

SWM facilities are designed to control runoff to 100-year levels, whereas the Regulatory event upon which flood plain mapping is based is a greater storm (such as the Timmins storm). Secondly, flood plain mapping is based upon a worst-case scenario where infrastructure such as SWM facilities may fail. Thirdly, since maintenance of private ponds and/or SWM facilities are not the responsibility of the municipality, there is no assurance they will continue to function as originally designed. Upon review of the areal imagery no ponds or SWM facilities were present in the watershed.

Online Pond North of Cambray Road

A small online pond exists to the north of Cambray Road which provides a small amount of storage that is used for private agricultural uses. This pond was not included in the hydrologic model as it has very limited active storage and thus does not provide any flood storage function.

Wetlands

There are several wetlands and waterbodies throughout the watershed. Runoff from wetlands was generally modelled as a regular rural subcatchment, using overland flow lengths to determine time to peak. The natural attenuation provided by the wetland features has been considered by applying a Curve Number of 50 and an Initial Abstraction of 5mm.

The wetland surrounding Goose Lake was modelled using two differing scenarios to assess the function and performance of this feature. One scenario utilized a rating curve within the ROUTE RESERVOIR command to simulate the storage-discharge relationship of the wetland and the second scenario used the ROUTE CHANNEL command. The comparison of these approaches are discussed further in Section 3.7.

3.6 Hydrologic Model Schematic and Results

The information gathered in the preceding sections was used to setup a Visual Otthymo model of the watershed. The detailed output is at **Appendix F** and all flows for the design storms through to the Regional Storm at key nodes within the study area are summarized in **Table 3.1**.

Table 3.0 Hydrology Output

Location	River Station	Node	2yr-AES6	5yr-AES6	10yr-AES6hr	25yr-AES6hr	50yr-AES6hr	100yr-AES6hr	Timmins
Outlet of Wetland	4761	30000	3.05	5.88	8.13	11.29	13.91	16.67	43.61
Berm Pinch Point	4399	50000	3.04	5.91	8.18	11.44	14.15	16.92	44.52
South of Community Park	2511	90000	4.58	8.69	11.93	16.52	20.30	24.25	61.21
Elm Tree Road/End of Study Area	1054	110000	6.01	11.25	15.20	20.64	24.93	29.4	72.64

3.7 Sensitivity Analysis – Hydrology

The hydrologic model was tested for sensitivity for the input parameters in the list below. Input parameters were modified by varying degrees as outlined below for the Regional Storm event only. The increase/decrease in peak flows from the base scenario at a number of key nodes was noted to establish a level of confidence in peak flow estimations. The following parameters were tested for sensitivity at key nodes and a complete set of the results of the sensitivity analysis are included within **Appendix G**.

Curve Number (CN*)

Flows in cubic meter per second (m^3/s) at key nodes were investigated to see the impact of changing the CN* value. Increasing CN* by 20% resulted in an average increase in peak flow of **26%** at key flow nodes during the Timmins storm event. Decreasing CN* by 20% resulted in an average decrease in peak flow of 25% at all key flow nodes during the Timmins event (**Table 3.2**). Because there is not a significant difference in peak flow values as a result of modifying the CN* value, the model was not considered to be sensitive to this parameter.

Table 3.1 Sensitivity Analysis +/-20 percent CN

Key Nodes	Name	Base (m^3/s)	CN*+20% (m^3/s)	+ (%age)	CN*-20% (m^3/s)	-(%age)
30000	Outlet of Wetland	43.61	55.38	27%	32.78	-25%
50000	Berm Pinch Point	44.52	56.48	27%	33.53	-25%
60000	Cambray Road	42.25	53.95	28%	31.66	-25%
90000	South of Community Park	61.21	76.63	25%	46.21	-24%
110000	Elm Tree Road	72.64	89.56	23%	55.48	-24%
			Avg. change (%)	**26%**		**25%**

CN* is determined by land use and soil type. For this study, the Kawartha Conservation 2010 ELC (Ecological Land Classification), Secondary Plan and Official Plan (OP) data from the City of Kawartha Lakes, and soil type was queried to extract land use, drainage area, and hydrologic soils group data. Schedules 'B to G' for the Fenelon Township from 06 June 2014 Land Use map version as delivered by City of Kawartha Lakes, was used to discretize the land use.

This base data is a reasonable and accurate representation of the drainage catchments, and therefore any calculated value (such as CN*) based on this data can be considered reliable.

Initial abstraction (IA)

Initial abstraction is a parameter that accounts for losses such as infiltration, evaporation, surface depression storage etc. prior to the occurrence of any runoff. This value is typically very small in-comparison to the volume of rainfall for a larger storm event and has a larger effect on smaller storm events. Therefore, it is expected that initial abstraction would have little to no effect on a substantial event such as the Timmins Regional storm.

Increasing Initial Abstraction by 50% resulted in an average decrease in peak flow of 2% at all key flow nodes during the Timmins storm event. Decreasing initial abstraction by 50% resulted in an average increase in peak flow of approximately 2% at all key flow nodes during the Timmins Regional storm event (**Table 3.3**). Therefore, changing the initial abstraction does not result in significantly different flows for the Regional storm.

Table 3.2 Sensitivity Analysis +/- 50% Initial Abstraction (IA)

Key Nodes	Base	IA+50%	+/-(%age)	IA-50%	+/-(%age)
30000	43.61	42.88	-2%	44.33	2%
50000	44.52	43.79	-2%	45.27	2%
60000	42.25	41.51	-2%	43.00	2%
90000	61.21	60.26	-2%	62.14	2%
Avg. change (%)			-2%		2%

Channel Routing Lengths

Channel routing accounts for the storage of flow as it is conveyed along the watercourse and its flood plain and results in the attenuation of flows through a watercourse. The overall watershed involves a variety of connecting watercourses between subcatchment nodes, and therefore adjustment of the lengths of these channel elements could be expected to have an impact on peak flows.

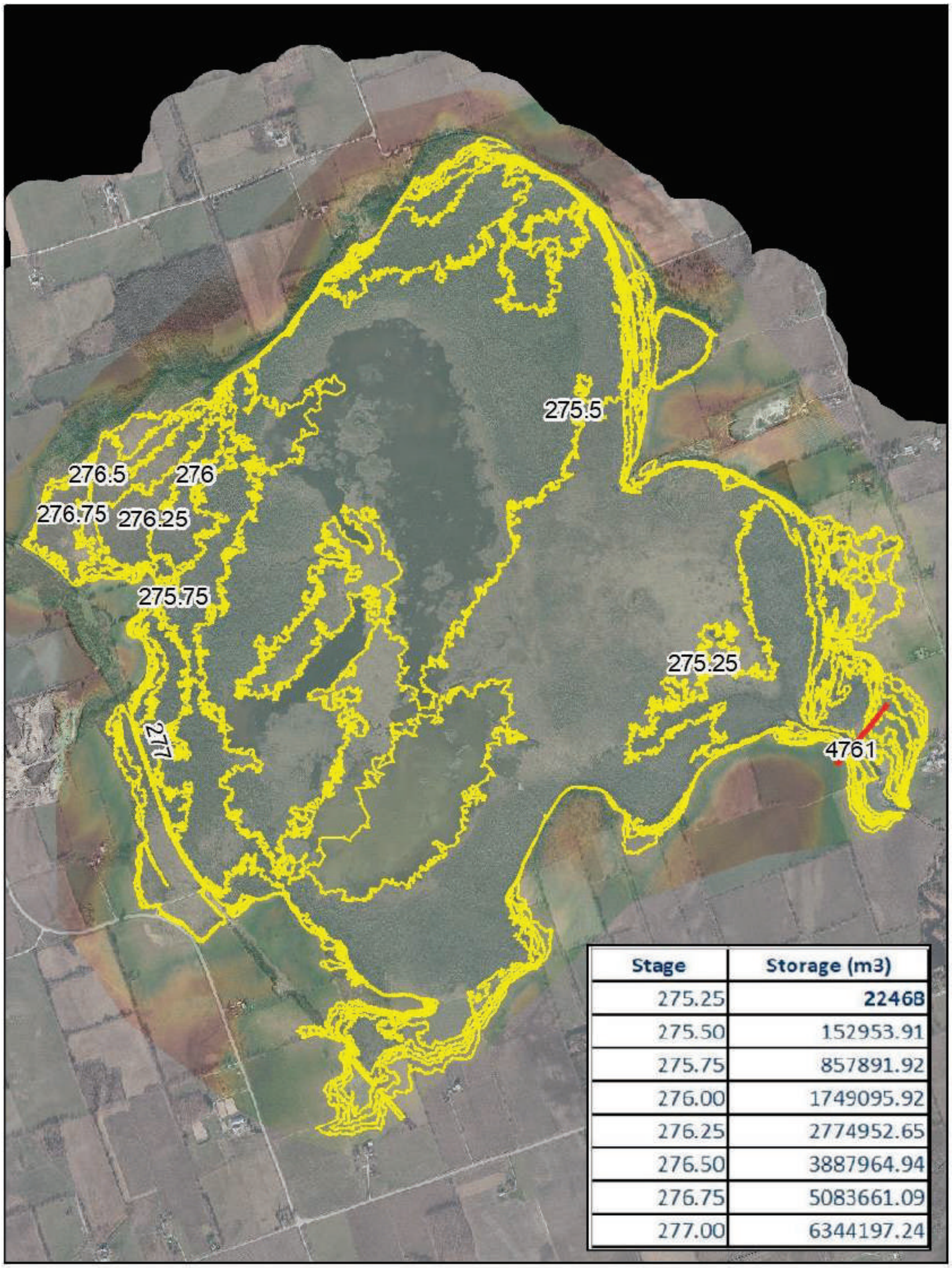
A scenario was created by adjusting the lengths of all channel routing within the model by 20%. By doing so, these adjustments resulted in an average change in peak flow of approximately minus 7% if the lengths were increased and 9% if the lengths were decreased at all key flow nodes during the Timmins storm event (**Table 3.4**). Therefore, channel routing length has a moderate effect on peak flows throughout the watershed.

Table 3.3 Sensitivity Analysis - Channel Routing Length Adjustments

Key Nodes	Base	FLWPLength+20%	+/-(%age)	FLWPLength-20%	+/-(%age)
30000	43.61	40.87	-6%	47.56	9%
50000	44.52	41.82	-6%	48.51	9%
60000	42.25	39.50	-7%	46.28	10%
90000	61.21	56.60	-8%	66.95	9%
Avg. change (%)			-7%		9%

Use of ROUTE RESERVOIR and ROUTE CHANNEL for Goose Lake Wetland

The wetland surrounding Goose Lake was modelled using two differing scenarios to assess the function and performance of this feature and the sensitivity to modelling approaches. One scenario utilized a rating curve within the ROUTE RESERVOIR command to simulate the storage-discharge relationship of the wetland to assess the impact on model results. The stage-storage relationship for the wetland was calculated using ArcGIS and the DEM for the study area and the stage-discharge relationship was taken from the HEC-RAS model at cross section 4761 (a controlling landform at the outlet of the wetland). See **Figures 3.14** and **3.15** below. The stage values within each curve were matched to produce the Storage-Discharge curve used in the model.



Stage	Storage (m3)
275.25	**22468**
275.50	152953.91
275.75	857891.92
276.00	1749095.92
276.25	2774952.65
276.50	3887964.94
276.75	5083661.09
277.00	6344197.24

Figure 3.14 Summary of Stage-Storage Relationship for Goose Lake Wetland

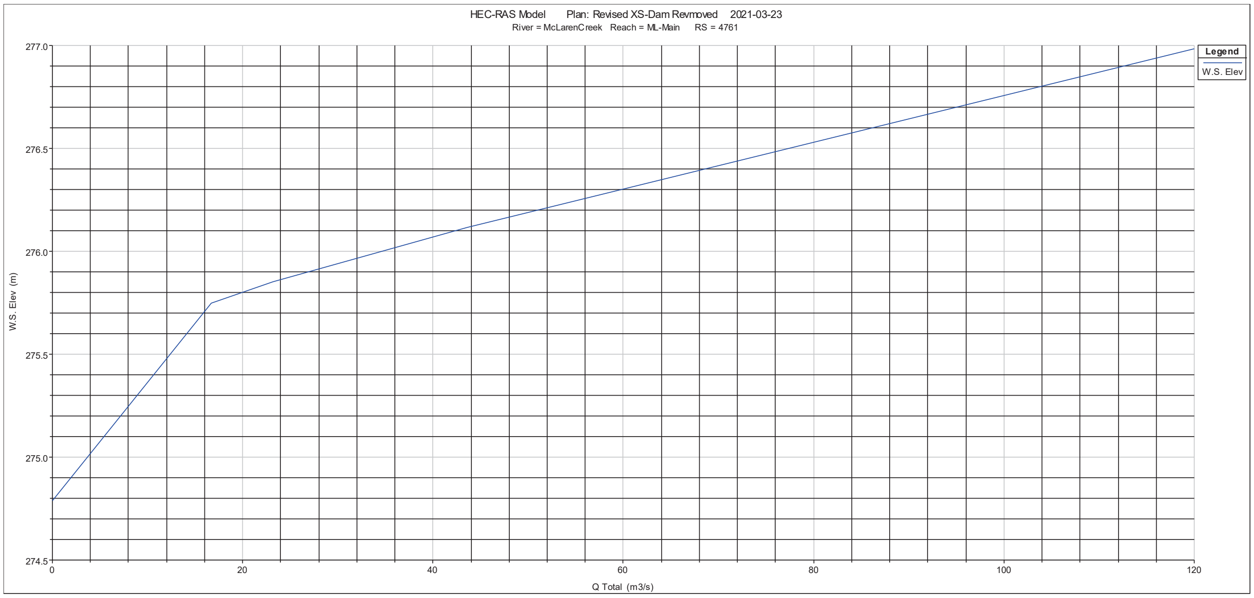


Figure 3.15 Summary of Stage-Discharge Relationship for Goose Lake Wetland (generated from HEC-RAS Cross Section 4761)

In the second scenario, the ROUTE CHANNEL command was used to simulate the routing effects of flows passing through the channel areas within the wetland. Both scenarios delivered reasonable peak flows which are summarized in **Table 3.5**.

As can be seen in the results within **Table 3.5**, the 2 year – 100 year design events produce similar flows under both routing scenarios but the Regional Storm event results are quite different, with the ROUTE RESERVOIR flows being appreciably lower below the outlet of the wetland. This result is reasonable given the significant size of the wetland and the storage available for attenuation of flood flows. It should be noted that under the ROUTE RESERVOIR scenario, it is assumed that the full storage of the wetland is available for storage of flood water which may not be the case when such a storm occurs.

Table 3.4 Sensitivity Analysis Results – Routing Options for Goose Lake Wetland

	Location	Node	Area (ha)	River Station	Event (m^3/s)						
					2yr	5yr	10yr	25yr	50yr	100yr	Reg.
Flows with ROUTE CHANNEL within wetland	U/S of Goose Lake Wetland	20000	732.4	NA	2.00	3.97	5.58	7.85	9.73	11.76	26.20
	Node within Goose Lake	25000	880.8	NA	1.24	1.98	2.77	3.75	4.73	5.91	20.22
	D/S of Goose Lake Wetland	30000	1912.7	4761	3.05	5.88	8.13	11.29	13.91	16.70	43.61
	U/S of Cambray	50000	1950.2	4399	3.04	5.91	8.18	11.44	14.15	16.92	44.42
	Cambray Road	60000	2031.5	3450	2.88	5.59	7.75	10.83	13.40	16.08	44.44
	Confluence S of Community Centre	90000	2641.6	2511	4.58	8.69	11.93	16.52	20.30	24.25	61.21
	Elm Tree Road South Culvert	110000	3156	1054	6.01	11.25	15.20	20.64	24.93	29.40	72.64
Flows with ROUTE RESERVOIR within wetland	Goose Lake Proper	*20000*	*1912.7*	*NA*	*4.15*	*8.81*	*12.65*	*18.15*	*22.71*	*27.58*	*66.25*
	Route Reservoir for Goose Lake	RR2	1912.7	4761	3.92	8.32	10.08	11.76	13.72	14.81	23.22
	U/S of Cambray	50000	1950.2	4399	3.89	8.32	10.14	11.99	13.74	14.82	23.22
	Cambray Road	60000	2031.5	3450	3.68	7.85	10.00	11.87	13.58	14.77	23.07
	Confluence S of Community Centre	90000	2641.6	2511	5.33	10.81	14.13	17.35	20.03	22.56	43.19
	Elm Tree Road South Culvert	110000	3156	1054	6.15	11.68	15.85	21.41	25.63	29.96	61.80

Also, given that the range of active storage within the wetland is generally limited between the elevations of 275m – 276m, the performance of the outlet may be sensitive to the following seasonal and temporal variations that are difficult to control or predict. These could include, for example:

- starting water levels
- ice cover
- change in vegetation communities and thus expected conveyance (particularly near the outlet of the wetland)
- impacts from beaver activity/beaver dams, etc.

For these reasons, it was recommended that the more conservative flow estimate from the ROUTE CHANNEL scenario be used for the hydraulic model simulation to generate flood lines for the Regional Storm.

4.0 Hydraulic Model Overview and Input Parameters

4.1 Overview

The water surface elevations that are used to determine the limits of flooding within the McLaren Creek Study area were determined using the United States Army Corps of Engineers, Hydraulic Engineering Centre's River Analysis System, commonly referred to as HEC-RAS. HEC-RAS has the ability to perform one-dimensional and two-dimensional hydraulic calculations on a range of natural and constructed channels. To create a new model, water surface profiles were determined using the program's steady state analysis, which assumes gradually varied flow with a subcritical flow regime. The latest available 1D version of HEC-RAS, version 5.0.7, was used for the study.

The resultant water surface profiles are considered a reasonable representation of the worst case scenario flood elevations during a Regional event and are appropriate for the purpose of a flood plain mapping exercise.

4.2 Stream Network

The initial step in developing the HEC-RAS model involved determining the limits of the watercourse and identifying subsequent watercourse reaches as required. For this study, the watercourse is treated as a single reach and extends from south of the Goose Lake Wetland to downstream of the culvert at Kings Lane.

An overview of the study reach, including a schematic of the hydraulic model with cross section locations, is shown in **Figure 4.0**.

Figure 4.0 Hydraulic Model Schematic of Study Reach

4.3 Flow Input

Peak flows determined by the Visual Otthymo hydrologic model have been input directly into HEC-RAS at select locations along each reach as shown in **Table 4.0** below:

Table 4.0 VO5 Regional Storm and 100-year Peak Flows used in the Hydraulic Model

HEC-RAS/ID	Location	VO Flow Node	Peak Regional Flow (m^3/s)	100 -yr Flow (m^3/s)
4761	Outlet of Wetland	30000	43.61	16.70
4399	Berm Pinch Point	50000	44.42	16.92
3450	Cambray Road	60000	44.44	16.08
2511	South of Community Park	90000	61.21	24.25
1054	Elm St Rd South Culvert	110000	72.64	29.40

4.4 Cross Sections

The cross section geometric data used in the hydraulic model was extracted from the Digital Elevation Model (DEM) using GeoHEC-RAS. Since LiDAR does not return laser points for any ground below the water surface it is necessary to supplement these areas with surveyed data to create accurate river geometry. Bathymetric survey points were taken in-channel up to the top of bank throughout the project area. In areas where bathymetric surveys were not possible, channel dimensions have been estimated based on typical bankfull channel dimensions within the reach.

The DEM is a crucial component in the development of cross sections. The use of GeoHEC-RAS ensures spatial reference of geometry data when imported into HEC-RAS. Cross sections were cut in the LiDAR-derived DEM. The surveyed data was fused/conflated into the cross sections generated by GeoHEC-RAS. Cross sections are cut along the study reach with reduced spacing at culvert crossings, bridges and other restricting structures to accurately represent channel flow.

The location and orientation of the cross sections are chosen based on a combination of aerial photography and contour data, locations from past studies, site reconnaissance and general knowledge of the flood plain. Cross sections are generally located in areas that represent the average channel geometry within a reach, where there may be abrupt changes in geometry or slope and at the appropriate road crossing locations.

4.5 Reach Lengths

Reach lengths are distances between cross sections along the stream centerline or thalweg and within the left and right overbank area. Overbank reach lengths were measured along the anticipated path of the centre of mass of the overbank flow. Reach lengths were measured using

GIS tools within ArcGIS in addition to the creation of an overbank polyline to represent flood plain flow directions. Overbank flow distances were extracted from the polylines within GeoHEC-RAS.

4.6 Bank Stations

Bank stations generally represent the top of a stream bank at a location where, if flow exceeded the bank elevation, it would spread within the flood plain. Bank stations are used by HEC-RAS to subdivide the cross section and identify the location where the roughness coefficient changes for the overbank area. HEC-RAS subdivides each cross section to determine the conveyance capability of the channel and within the left and right overbank areas. When the user chooses to use multiple Manning's "n" values for a section (e.g. more than three), the section is subdivided based on the horizontal change in roughness.

Bank station locations within the model are based on collected survey data, aerial photography and elevation data along with available pictures of the channel.

4.7 Culvert and Road Crossings

Data for culvert and bridge crossings at roadways was obtained through a combination of a georeferenced topographic survey or the DEM. Cross sections and culvert data, including inverts, obverts, length, span and rise, were obtained via an RTK GNSS (Real Time Kinematic Global Navigation Satellite Systems) survey. During the survey, detailed field notes were taken, as were pictures at select locations. Detailed structure data sheets and structure photos for each crossing are contained in **Appendix I**. Roadway centreline elevations to be used for deck elevations were either cut from the DEM or collected through topographic survey. Guard rails, parapet walls and fences were incorporated into the deck elevations. There are a total of eight water crossing structures in the study reach of the McLaren Creek Study Area and they are summarized in **Appendix I**.

Several private crossings within the study area were not surveyed due to access issues/concerns however they were considered to be relatively small and are not expected to significantly impact flood limits for severe storm events such as the Regional or 100-year event. It is recommended however that consideration be given to surveying these structures in the future such that they can be added to the model for completeness and accuracy.

4.8 Expansion/Contraction Coefficients

Contraction and expansion coefficients were specified by the engineer at each cross section to define the energy losses between two cross sections of varying geometry. Where there is minimal change in the geometry or shape of two cross sections, the energy losses will be minimal. If the transition in geometry is abrupt, such as at a bridge or culvert, energy losses will be high.

Standard values for contraction and expansion coefficients, as specified in Table 3-3 of the "HEC-RAS River Analysis System Hydraulic Reference Manual" (2016) (HEC-RAS HRM), have been used throughout the current model. **Table 4.1** lists the contraction and expansion coefficients used within the model for subcritical flow. By default, all cross sections incorporate contraction/expansion coefficients of 0.1 and 0.3, except for bridge/culvert crossings or abrupt transitions.

Table 4.1 Subcritical Flow Contraction and Expansion Coefficients

	Contraction	Expansion
No Transition Loss Computed	0.0	0.0
Gradual Transitions	0.1	0.3
Typical Bridge/Culvert Sections	0.3	0.5
Abrupt Transitions	0.6	0.8

4.9 Manning's n Values

The value of Manning's "n" is highly variable and depends on a number of factors including surface roughness, vegetation, channel irregularities, channel alignment, scour and deposition, obstructions, size and shape of the channel, stage and discharge, seasonal changes, temperature and suspended material and bedload. The Manning's n values used in the HEC-RAS model were based on the recommendations in Table 3-1 of the HEC-RAS hydraulic reference manual (HRM).

The main channel Manning's n value is 0.035 and the overbank values ranged from 0.045 to 0.08. These values were determined for each cross section using a combination of a high resolution georeferenced aerial photograph, survey notes and photos. For cross sections with significant differences in Manning's values, additional coefficients were added to more accurately reflect the roughness values for the reach through the community of Cambray. Typical Manning's Coefficients used in the model are depicted in **Table 4.2.**

Table 4.2 Manning's Coefficients

Description	Manning's n Value
Channel	0.035
Lawn/Short Grass	0.045
Agricultural Fields	0.055
Heavily Treed/Dense Vegetation	0.080

4.10 Ineffective Flow Elevations and Levees

Ineffective flow areas are introduced at each culvert or bridge crossing in accordance with the recommendations contained in the HEC-RAS HRM. The ineffective flow area was generally used where flood water will occur but was considered not to contribute to the conveyance of flow. The upstream bounding cross section has the ineffective flow elevations equal to the top deck elevations, at locations immediately left and right of the culvert opening. At the downstream bounding cross section, the ineffective flow elevations were set at a point midway between the deck and the culvert obvert elevation.

4.11 Building Obstructions

The effect of a building within the flood plain can have a significant influence on the available conveyance and energy losses immediately upstream and for a distance downstream of the actual building. Where a building may influence a cross section upstream or downstream, the obstruction has been projected onto the affected section.

4.12 Hydraulic Model Schematic

The information gathered in the preceding sections was used to build a HEC-RAS model of the watercourse. As noted previously, the layout of the model is shown schematically in **Figure 4.0**.

4.13 Inline Structure/Dam

A small privately owned dam is located on McLaren Creek approximately 80 metres to the north of Cambray Road. This dam was modelled as an inline structure and the survey data collected on the site was used in the representation of the weir structure. It is noted that the Regional flows for the reach exceed the capacity of the dam causing flows to spill eastward into the adjacent field and southward through the lots fronting the north side of Cambray Road. As the model is one dimensional (1D) and the surrounding topography is relatively flat, it is difficult to accurately confirm the extent to which flows will spill (in two dimensions (2D)) to the east of the dam structure within the western portion of Cambray. The predicted Regional flood elevation upstream of the dam is 273.50 m which is similar to the elevations along the east side of the pond upstream of the dam structure. Flows would be expected to spill to the east and inundate the adjacent field to a similar elevation in addition to some of the properties east of North Street before spilling southward toward Cambray Road and eastward to Elm Tree Drive. The land east of the dam and below the field does gradually slope toward Cambray Road, which would allow a portion of the flood waters to return to McLaren Creek as storm flows recede however, shallow flooding could also be expected for the properties along Cambray Road between the bridge and North Street (outside of the mapped flood plain).

While the current approach provides a reasonable estimation of the flood plain through Cambray, consideration should be given to the use of a two dimensional (2D) hydraulic model to refine the extent of flooding within Cambray given the presence of the dam and the very flat topography of the areas east of the dam. This work should be accompanied by additional topographical field survey to confirm and refine the elevations at which such a spill would occur along the east side of the pond and dam.

4.14 Hydraulic Model Sensitivity Analysis

The hydraulic model was tested for sensitivity to input parameters in the list below. Input parameters were modified by varying degrees as outlined below for the Regional Storm event. The increase/decrease in flood elevation from the base scenario were noted to establish a level of confidence in flood elevation estimations. The following parameters were tested for impacts on model sensitivity:

- Peak Regulatory Flow (+/- 30%)
- Manning Roughness Coefficient (+/- 20%)
- Dam versus No Dam upstream of Cambray Road

Tabulated results of the hydraulic modelling sensitivity analyses are provided in **Appendix G.**

Peak Regulatory Flow

Flood elevations throughout the project reach were investigated to determine the impact of changing the Regional (Timmins Storm) peak flows. This was completed to account for uncertainty and assumptions as per the hydrologic modelling. From the hydrology sensitivity analysis, Regional peak flows were varied by +/- 30%.

By increasing the Regional Storm peak flows, it was found that the average increase in Regional flood elevation throughout the cross sections was 12 cm, with the greatest increase of 57cm at cross section 2065 (attributed to flow conditions transitioning from critical to subcritical downstream of Structure 1).

By decreasing the Regional Storm peak flows, it was found that the average decrease in Regional flood elevation throughout the cross sections was 13 cm, with the greatest decrease of 78 cm at cross section 3415 (which can be attributed to a localized impact associated with the flow conditions downstream of Structure 6/Cambray Road and thus not representative of the entire reach).

With the average change of 12-13 cm in water surface elevations for a variation in Regional Storm peak flow as significant as 30%, the sensitivity of the change on the model results can be considered moderately sensitive and thus, the flood elevations are considered reasonable.

Manning's Roughness Coefficient

Flood elevations throughout the study reach were investigated to determine the sensitivity of the model results to the impact of changing the Manning's roughness coefficient (Manning's n). The Manning's n indicates the friction factor in a cross section. The higher the number, the rougher the surface against which water flows. For instance, a smooth concrete pipe has a Manning's n of 0.013 whereas a forest has a Manning's n value of 0.08.

By increasing the Manning's n value by 20%, the flow is being subject to greater friction forces acting upon it. It was found that the increase in the Regional water surface elevation throughout the study area across all the cross sections reached a maximum of 51 cm at cross section 2065 (attributed to flow conditions transitioning from critical to subcritical downstream of Structure 1) with an average increase of 5 cm for the over all cross sections.

By decreasing the Manning's n value by 20%, the flow is being subject to a watercourse/flood plain with lower friction forces acting upon it. It was found that the greatest decrease in the Regional water surface elevation throughout the cross sections was 13 cm at cross section 1968 (attributed to flow conditions transitioning from critical to subcritical downstream of Structure 1) with an average decrease of 4 cm for the overall reach.

Due to a minimal impact on the average overall flood elevations (4-5 cm) throughout the study reach, it can be determined that the hydraulic model is not sensitive to changes to the Manning roughness coefficients.

Dam versus No Dam upstream of Cambray Road

The third and final test for the sensitivity of the hydraulic model involved the removal of the inline structure/dam upstream of Cambray Road. The code for the inline structure was removed from the base geometry file but the existing landform was left in in place. The model rerun to evaluate the impacts of the removal of the structure.

The results of the analysis indicated that there was a notable drop in water surface elevations under Regional Storm conditions with the removal of the structure. The highest reduction in flood elevations was 1.2 m at cross section 3533 immediately upstream of the dam which reduced to a drop of 1 cm at cross section 3632, approximately 100 m upstream of the dam.

There were several areas downstream of the dam that produced some variations in water surface elevations but these impacts could be attributed to changes in local flow conditions at several bridge structures and thus were not related to the removal of the dam.

It was determined through the analysis that the results produced by the model are sensitive to the removal of the inline structure/dam for the section of the model through the village of Cambray.

4.15 Hydraulic Model Results

The hydraulic model results have been exported into GIS to produce floodlines for McLaren Creek which are presented in **Figure 4.1**. The model output has been provided in **Appendix J**.

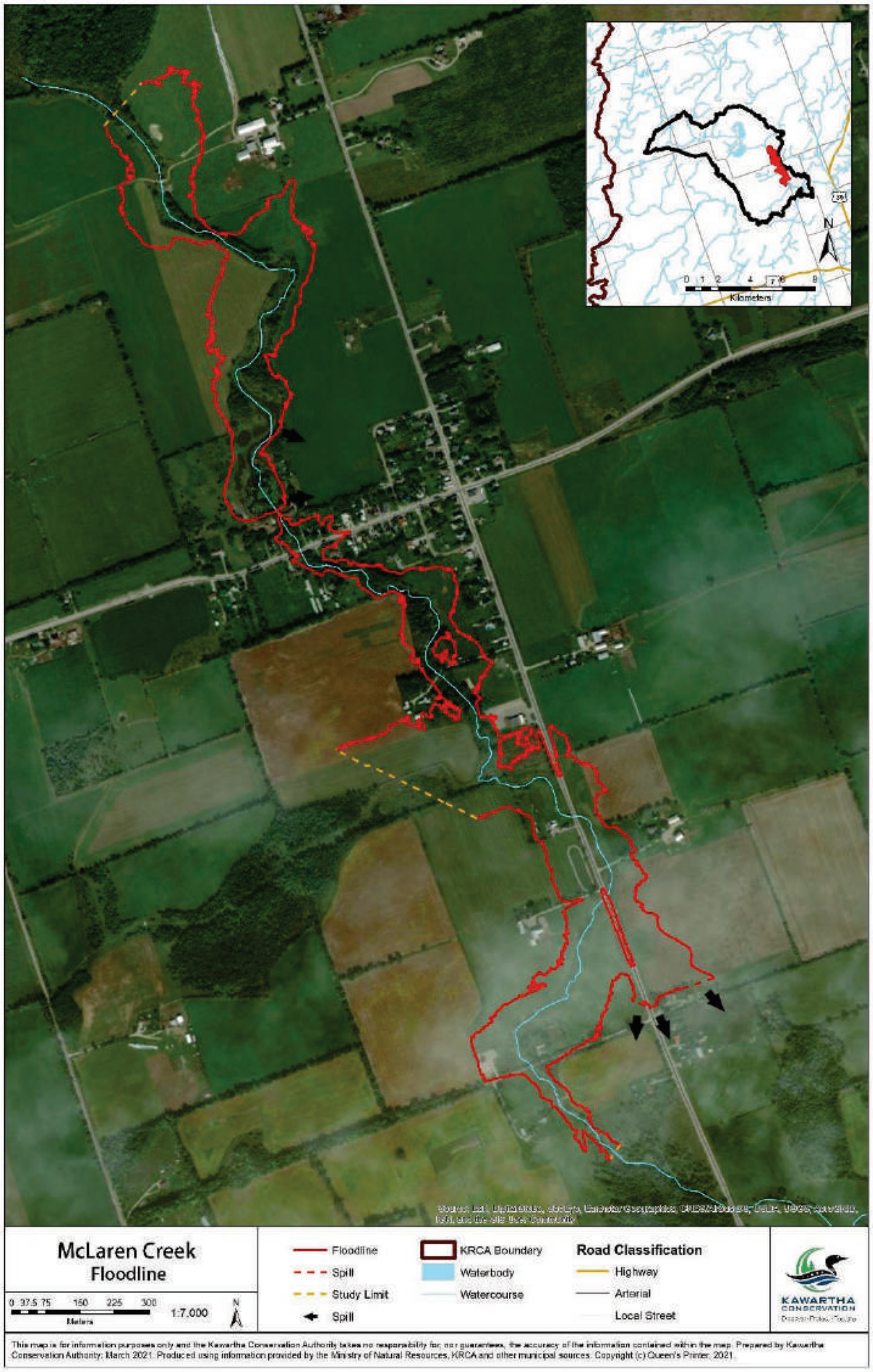


Figure 4.1 Floodline Limits for McLaren Creek

There are a number of interesting items to highlight from the model and associated mapping:

- Both Cambray Road and Elm Tree Road overtop under Regional Storm conditions while 100-year flows are conveyed through each respective structure.
- The private dam structure/on-line pond upstream of Cambray creates a significant backwater condition under Regional Storm conditions that has the potential to cause flows to spill to the east toward the centre of Cambray. The model was indicating flood elevations that are within centimetres of the grade elevations along portions of the east side of the pond upstream of the dam. Given that the current hydraulic model used in the study was 1-Dimensional, it is recommended that the extent of flooding for the village be refined through 2-Dimensional modeling in the future.
- It should be noted that a minor spill is also expected along the east side of Elm Tree Road, opposite Kings Lane as indicated on the mapping.
- A GIS analysis was completed using the Building Footprints shapefile to calculate the number of buildings impacted by the floodline. A total of 16 structures were found to be located within the flood plain within the study area as depicted within **Figure 4.2** below.

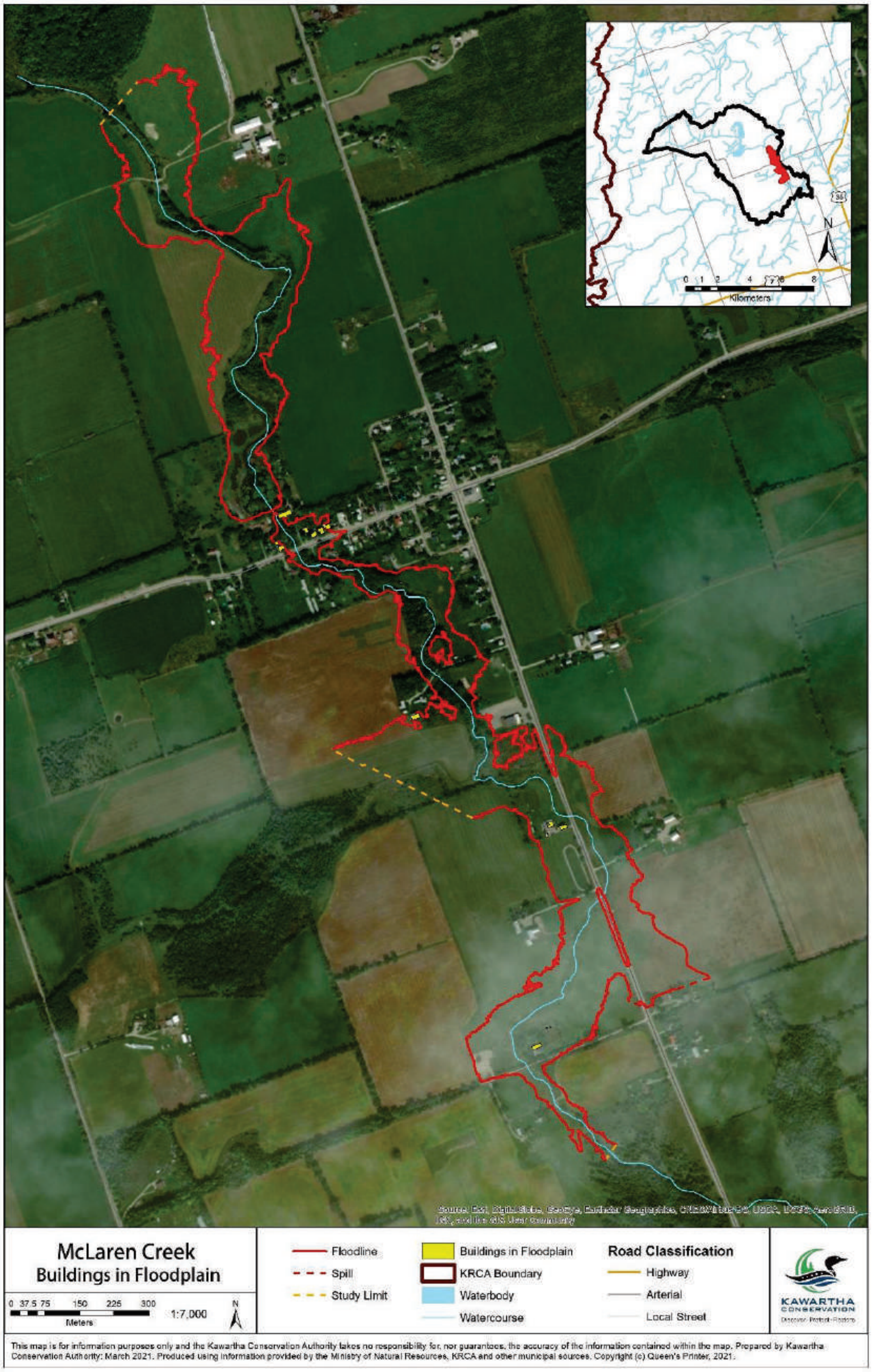


Figure 4.2 Building footprints within floodplain limits

5.0 Conclusions and Recommendations

The flood plain study report presents the following conclusions and recommendations:

- Peak flows from the Timmins Regional storm event exceed peak flows of the 100-yr storm, therefore are used to define the Regulatory flood hazards for McLaren Creek.
- The private dam structure/on-line pond upstream of Cambray creates a significant backwater condition under Regional Storm conditions that has the potential to cause flows to spill to the east toward the centre of Cambray. The model was indicating flood elevations that are within centimetres of the grade elevations along portions of the east side of the pond upstream of the dam.
- The private dam structure/on-line pond upstream of Cambray creates a significant backwater condition under Regional Storm conditions that has the potential to cause flows to spill to the east toward the centre of Cambray. The model was indicating flood elevations that are within centimetres of the grade elevations along portions of the east side of the pond upstream of the dam.
- Both Cambray Road and Elm Tree Road overtop under Regional Storm conditions while 100-year flows are conveyed through each respective structure.
- It should be noted that a minor spill is also expected along the east side of Elm Tree Road, opposite Kings Lane as indicated on the mapping.
- A GIS analysis was completed using the Building Footprints shapefile to calculate the number of buildings impacted by the floodline. A total of 16 structures were found to be located within the flood plain within the study area.
- Given that the current hydraulic model used in the study was 1-Dimensional, it is recommended that the extent of flooding for the village be refined through 2-Dimensional modeling in the future. This work should be accompanied by additional topographical field survey to confirm and refine the elevations and locations at which such a spill would occur along the east side of the pond and dam.

The estimated flood plain limits through the community of Cambray are considered reasonable given the use of LiDAR and the creation of the digital terrain model and should be accepted as the new regulatory floodplain limits.

6.0 Appendices

(Bound in a separate document)

Appendix A: Modeling Parameters Selection

Appendix B: Rainfall Data

Appendix C: Background Studies

Appendix D: Subcatchment Data

Appendix E: Subcatchment Maps

Appendix F: VH Suite Output

Appendix G: Sensitivity Analysis

Appendix H: Official & Secondary Plan Maps

Appendix I: Structure Photo Inventory Record

Appendix J: HEC RAS Output

Appendix K: Digital Terrain Model (DTM) Data Assessment Report