

Flood Plain Mapping Study Mariposa Brook Final Technical Report March 2020



Executive Summary

The primary goals of this study are to create hydrologic and hydraulic models of the Mariposa Brook watershed and produce flood plain mapping for the community of Little Britain. 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 Mariposa 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 of this study include:

- Peak flows from the Timmins Regional storm event exceed peak flows of the 100-yr storm, therefore the Regional storm is used to define the Regulatory flood hazards for Mariposa Brook.
- The Regional storm peak flows in the west branch through the community of Little Britain are comparable to the previous study done in 1990 while the peak flows in the main branch downstream of Country Road 4 decreased by approximately 17%.
- The flood plain limits through the community of Little Britain are more accurate than the previous study given the use of LiDAR and the creation of the digital terrain model resulting in 17 buildings being removed from the flood plain and 32 buildings being added.

Key recommendations of this study:

This study recommends that the revised 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 Little Britain.

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1.0 Introduction

1.1 Study Objective

The objective of this study is to update flood plain mapping along the Mariposa Brook through the community of Little Britain located 15km southwest of Lindsay, using new hydrologic and hydraulic models based on the latest ground survey, future land-use conditions, topographic maps, aerial photography and provincial guidelines. The updated flood plain 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 within the community of Little Britain.

1.2 Study Process

At the commencement of the project, the Technical Committee (consisting of representatives from the City of Kawartha Lakes, Kawartha Conservation, and the Ganaraska Region Conservation Authority) created quality assurance (Q/A) and quality control (Q/C) processes to be applied to all projects in the multi-year initiative. The Q/A 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 Q/C 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 Q/A and Q/C by an external firm or agency. Four separate components of the project were established for Q/A and Q/C.

- 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 acceptable.

Details of the study are separated into four primary sections.

- Hydrology
- Hydraulics
- Report
- Flood plain Mapping

The hydrology section examines the modeling methods, setup and results of the study. The purpose of the hydrological modeling is to determine the peak flows that occur at key points along the Mariposa Brook.

1.3 Study Area

The contributing drainage area of Mariposa Brook under this flood plain study is 182.21 square kilometers (km²), as illustrated in **Figure 1.0**. The drainage area is divided into two large tributaries, the north tributary (119.83 km²) and the west tributary (62.32 km²). The land use for the Mariposa Brook watershed predominantly consists of agricultural and forested lands. There are two low density developments located within the watershed; the community of Oakwood in the north tributary catchment and the community of Little Britain along the west tributary.

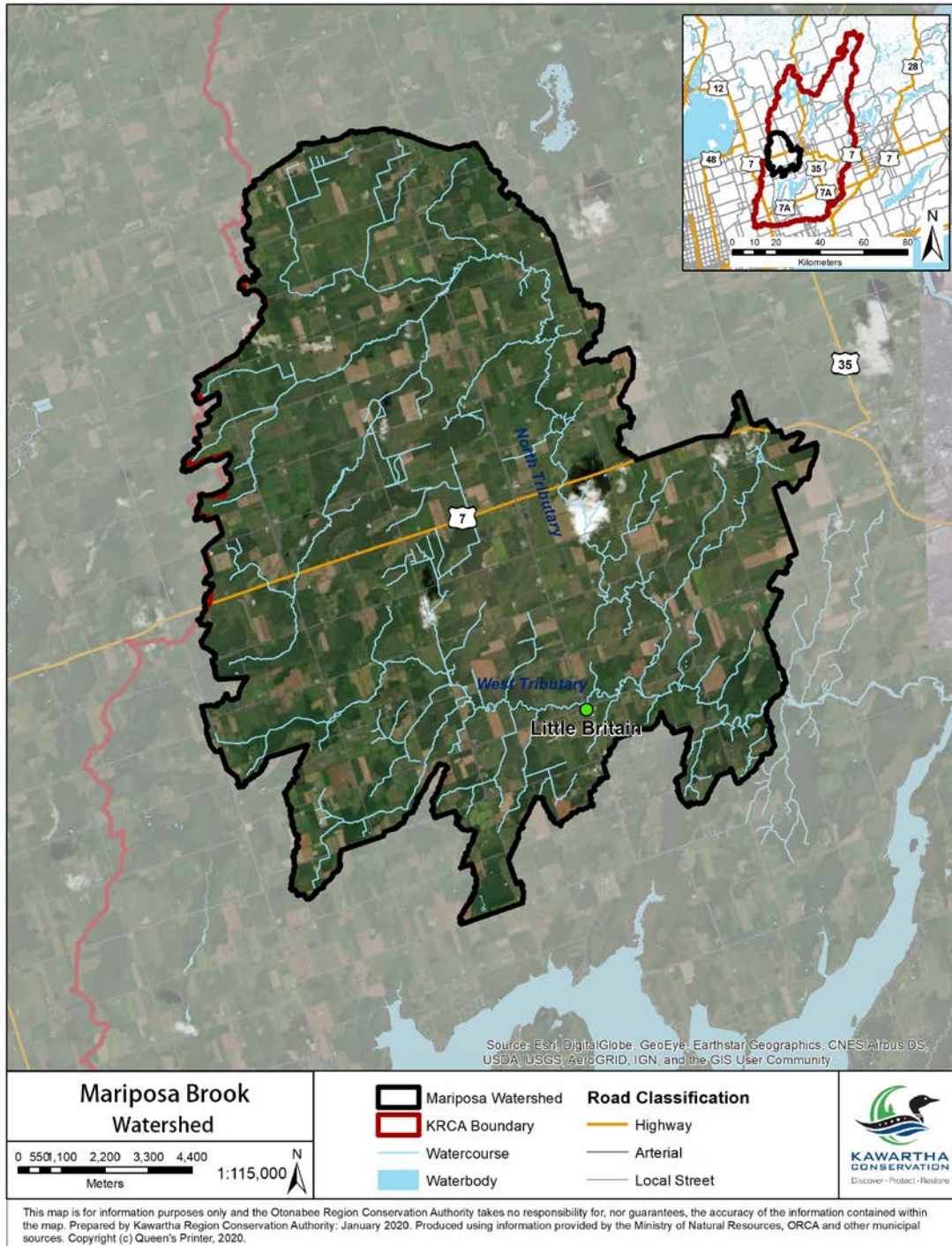


Figure 1.0: Mariposa Watershed - Study Area

1.4 Background Studies

Table 1.0 provides a chronology of the previous flood related reports, models and mapping that have been created for the study area. **Table 1.0** does not include minor studies that may have been completed as part of site-specific development proposals. See **Appendix C** for scanned copies and maps of available previous studies.

Table 1.0: Previous Reports on Mariposa Flood Prone Areas

Report/Study	Description	Author
Preliminary Identification of Flood Prone Areas	Provided a preliminary identification of flood prone areas; Little Britain was identified as the highest priority flood prone area	MacLaren Plansearch Inc. (1981)
Fill Line Mapping Project	Establishment of “Fill, Construction, and Alteration to Waterways” regulations under Section 28 of the Conservation Authorities Act	KRCA (1989)
Mariposa Brook Flood Plain Study	Flood plain delineated for the study area	KRCA (1990)

1.5 Modeling Approach

Flooding was assessed using standard steady flow computer simulation modelling methods derived using the latest version of Visual Otthymo v.5.1 and HEC-RAS v. 5.07.

Geographic data (such as subcatchment area, land use, topography and soil types) was extracted from GIS for each subcatchment to obtain the parameters described in the Hydrology Modeling Parameters Selection document (refer to **Appendix A**), and to calculate values such as imperviousness, SCS Curve Numbers (CN), time to peak (Tp) and time of concentration (Tc).

Runoff hydrographs have been generated for the 100-year and Regional (Timmins) storm events. The source rainfall data used for this analysis is from Environment Canada’s rain gauge that was historically located at the Lindsay Filtration Plant.

Sensitivity analysis is carried out in the report to determine the impact of changing model parameters on the calculated flows. 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 flow monitoring data is available 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

$$I = a/(t+b)^c, \text{ where}$$

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 yrs 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.

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. Therefore, the 100-year AES 6-hour storm was utilized to simulate the 100-year storm event.

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

Key Nodes	100yr CHI 6hr	100yr CHI 12hr	100yr CHI 24hr
7	43.621	47.792	49.637
13	65.781	71.087	72.611
14	103.299	114.362	118.044
15	77.898	90.847	94.559
16	57.282	70.799	78.167

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

Key Nodes	100yr-AES-6hr	100yr-AES-12hr	100yr-AES-24hr
7	53.656	51.39	47.906
13	83.07	73.526	54.672
14	128.89	121.732	102.469
15	96.531	94.334	88.033
16	72.923	72.355	71.391

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

Key Nodes	100yr-SCS-6hr	100yr-SCS-12hr	100yr-SCS-24hr
7	53.28	49.634	45.886
13	81.775	70.389	60.126
14	128.118	116.725	100.587
15	96.335	92.831	83.928
16	72.839	71.794	69.894

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 Timmins storm event resulted in a total rainfall of 193 mm. The storm is defined in Table D-4 of the “MNRF River and Stream Systems: Flooding Hazard Limit” (2002).

The Regional (Timmins) Storm and the range of 100-year 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 most areas throughout the watershed, the Timmins Storm will produce the Regulatory Flood for the watershed.

The flood plain study reach is located primarily within the subwatershed of the west tributary; however, the downstream reach goes beyond the confluence of west and north tributaries by approximately one kilometer. Therefore, to get an accurate estimate of flows for the downstream reach, the entire watershed (north and west tributaries) is modeled in the VO5 model.

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 area-specific 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. These reduction factors should be applied on an equivalent circular area. The circular area for the watershed at the confluence is 182.21 km², therefore, a reduction factor of 84% is applied at the confluence. Whereas, for the west branch with an area of 61.32 km², a reduction factor of 94% is applied to the Timmins storm.

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 parameters and modeling 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 the Mariposa Brook 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 area-specific 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.

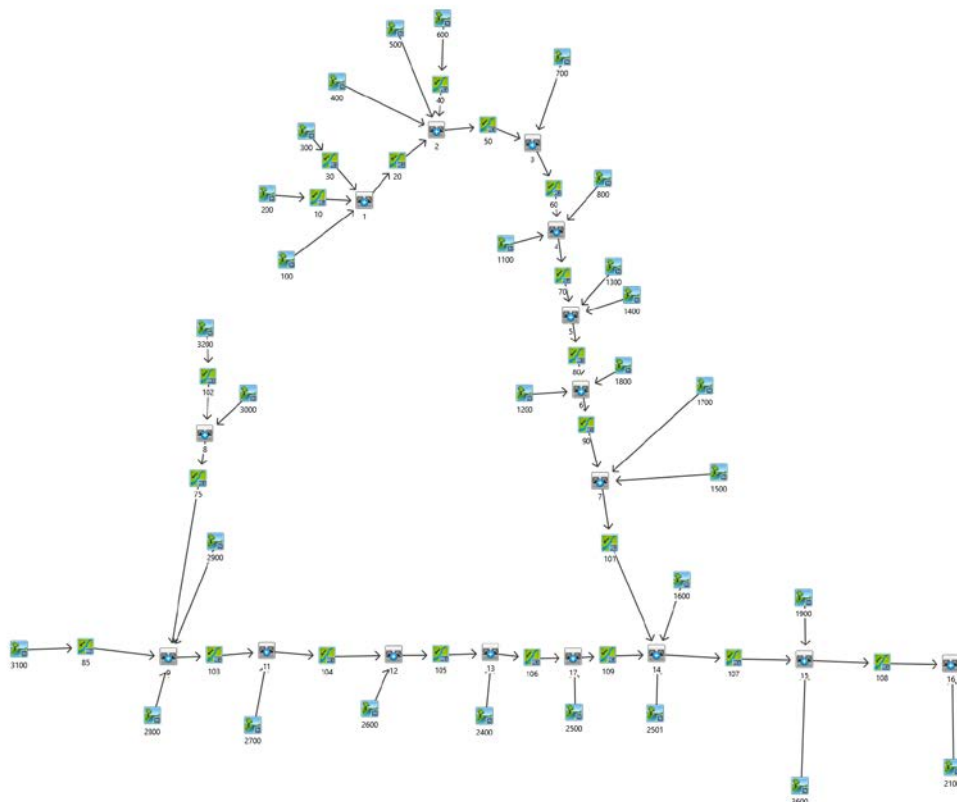


Figure 3: Detailed Schematic of the Mariposa Brook V05 Hydrologic Model

3.2 Digital Terrain Model (DTM) and Orthoimagery

The fundamental dataset which underlies all stages of any flood plain mapping project is digital topographic base data with full coverage of the study area. Topographic data for the Mariposa Brook was obtained from the City of Kawartha Lakes. This topographic data was received in the form of a digital terrain model (DTM) which was produced using Light Detection and Ranging (LiDAR) data acquired in Fall 2012. 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. Examples of the digital topographic data are found in **Figures 3.1** through to **3.9**.

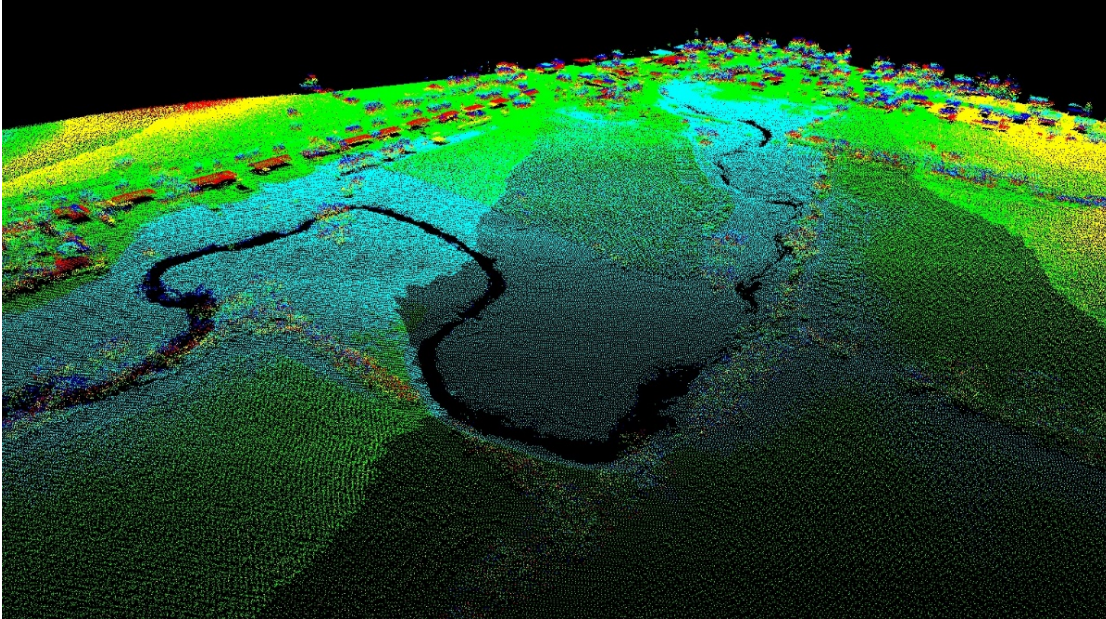


Figure 3.1: Classified LiDAR Point Cloud

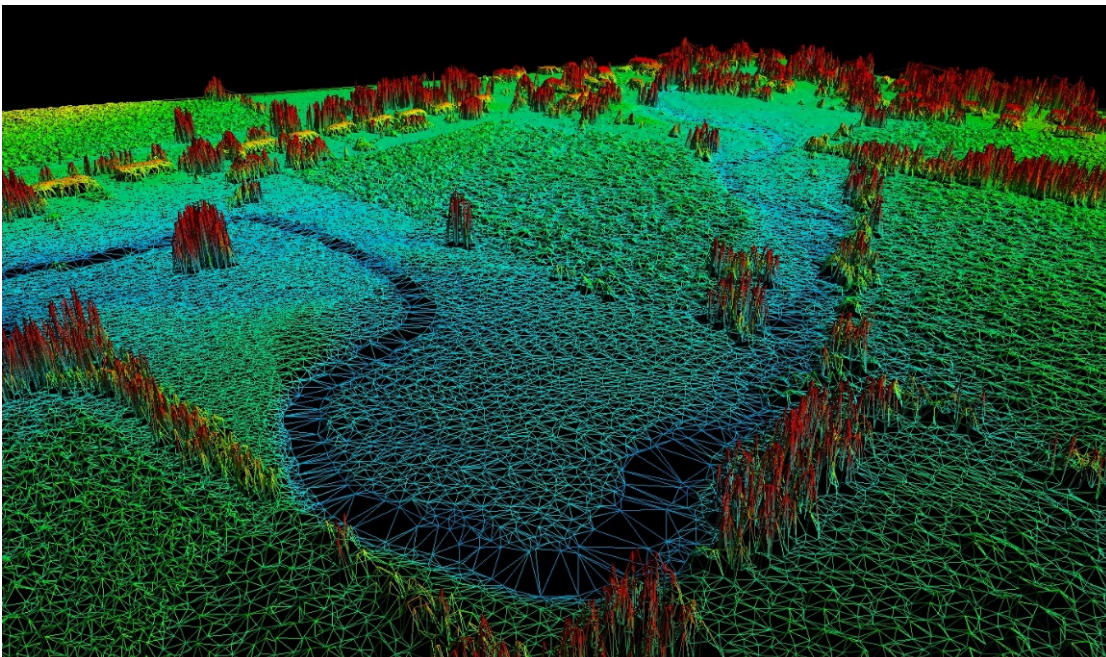


Figure 3.2: Triangular Irregular Network (TIN) Produced from LiDAR Point Cloud

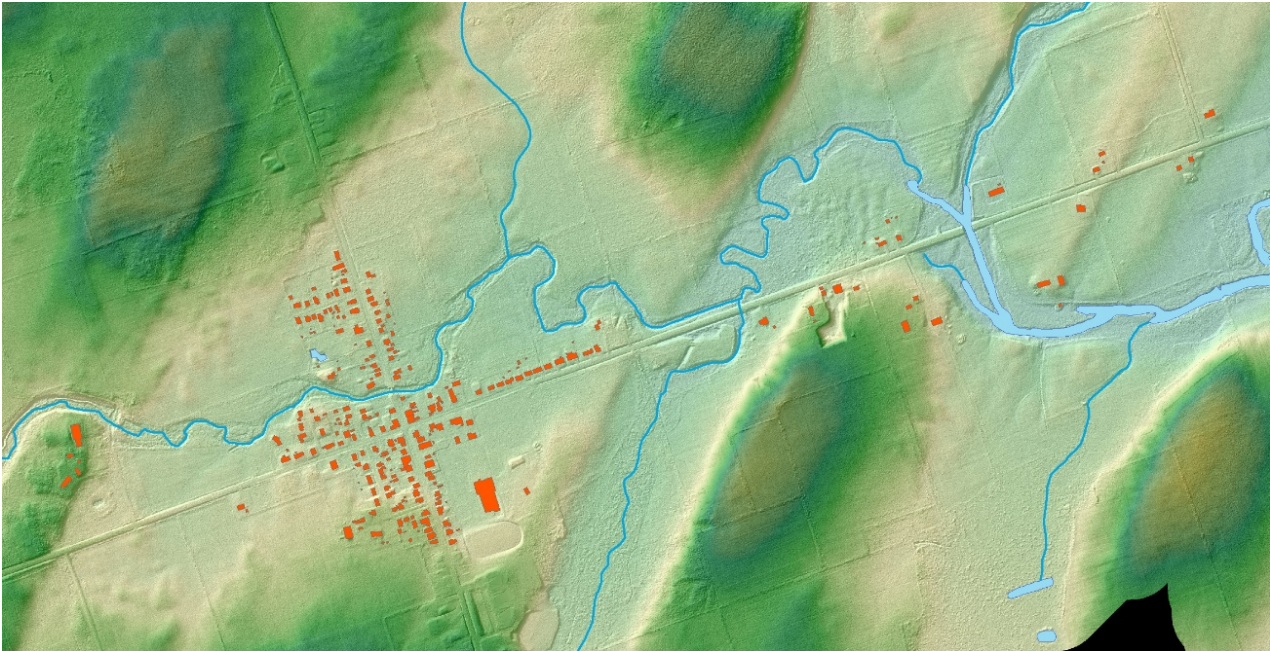


Figure 3.3: Digital Terrain Model (DTM) Produced from LiDAR Point Cloud with Building Footprints (Orange) and Watercourses (Blue) Overlain

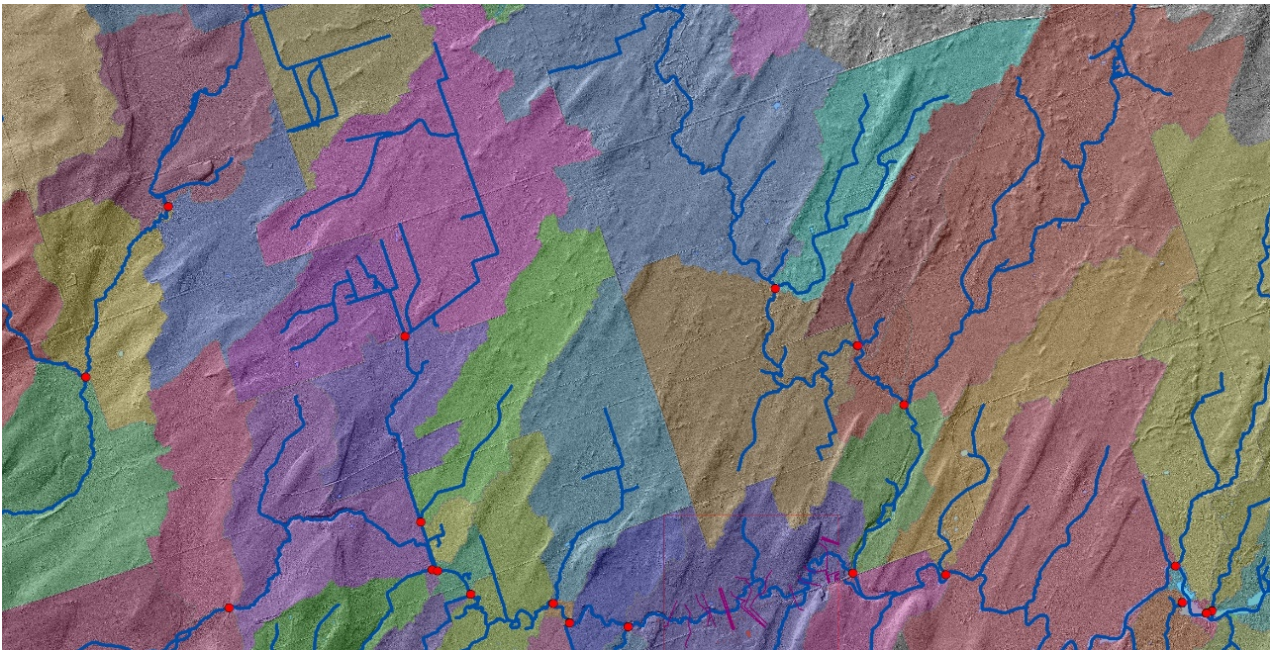


Figure 3.4: Hydrology Subcatchments Overlain on Hydrologically-Conditioned DTM Used to Delineate Into Polygon Layer

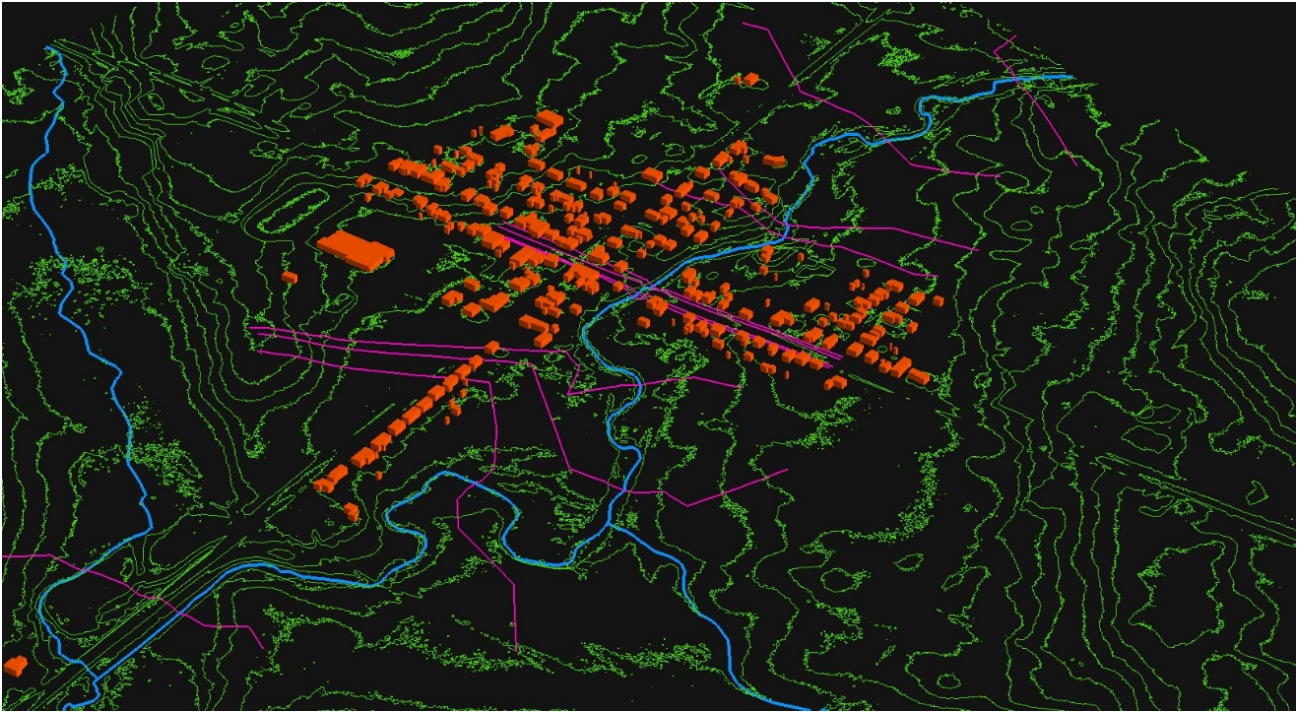


Figure 3.5: DTM-Derived 3D Contours with DTM-Derived Hydraulic Cross-Sections (Pink), Buildings (Orange) and Watercourse (Blue)

With the aid of GIS software, the 2012 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 subcatchment boundaries and labels are indicated in **Figure 3.12**.

The 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 Mariposa Brook 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.

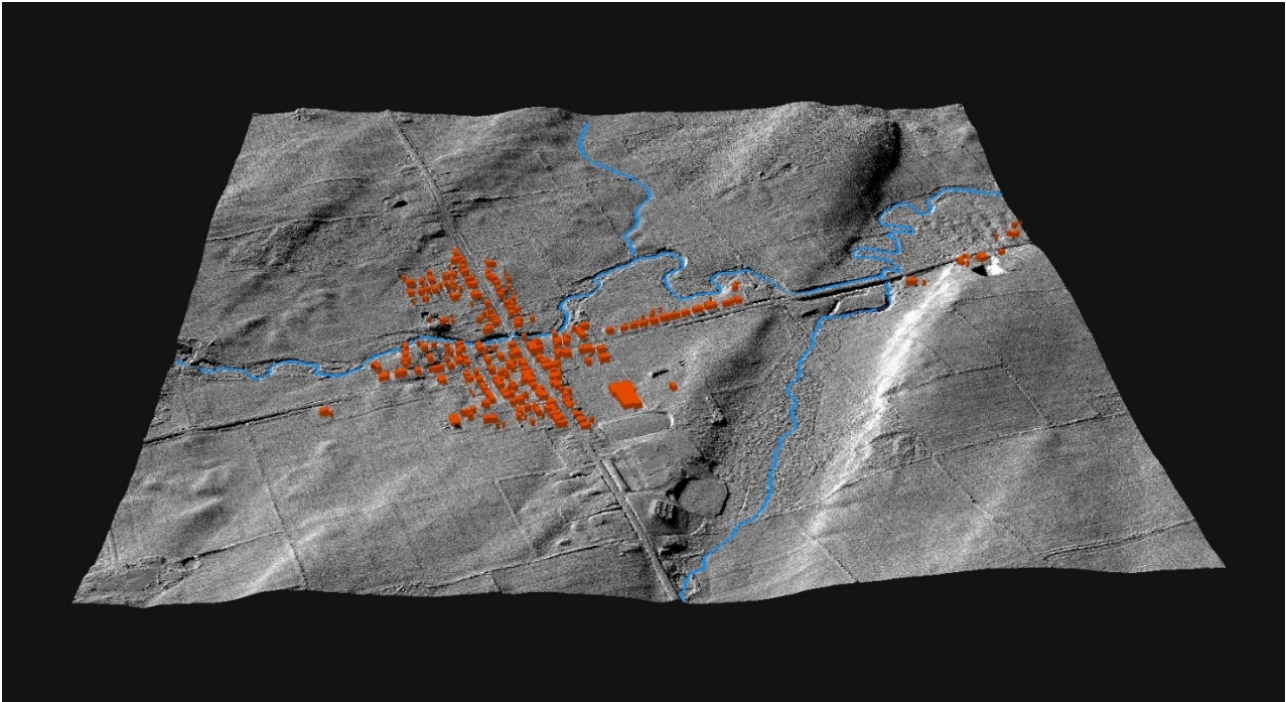


Figure 3.6: Oblique Rendering of DTM Hillshade with Buildings (Orange) and Watercourse (Blue)

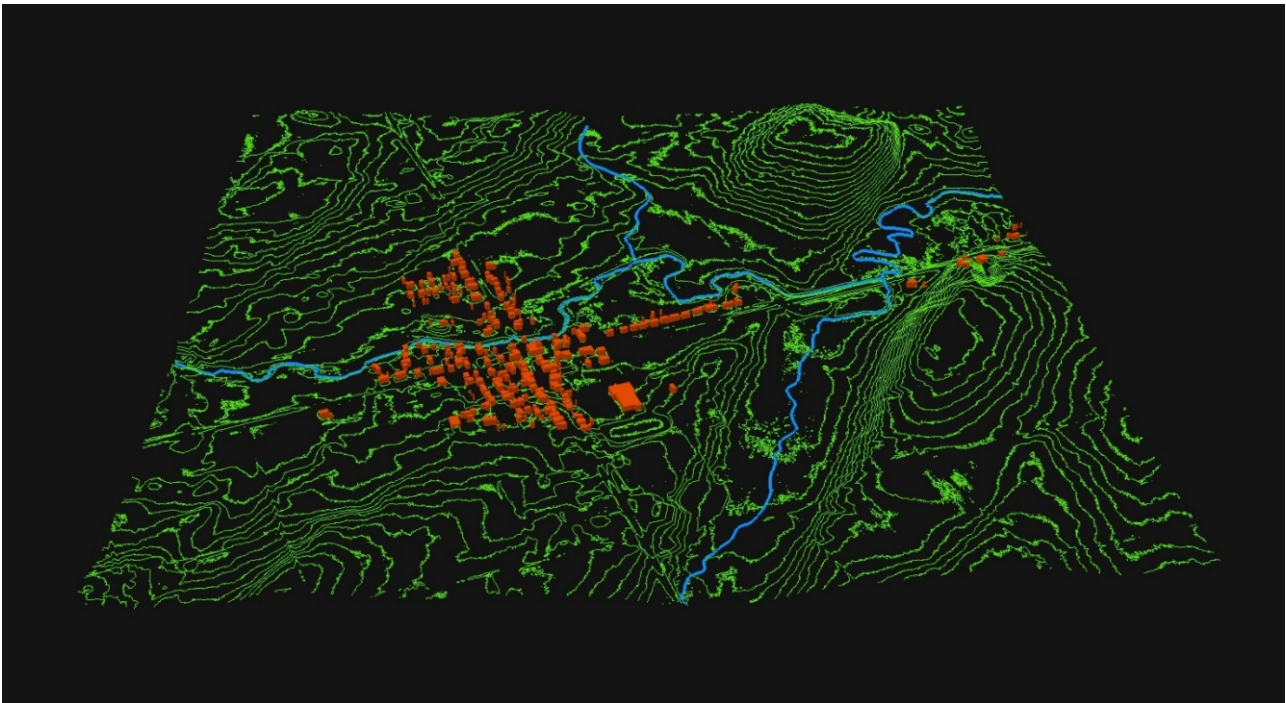


Figure 3.7: Oblique Rendering of DTM-Derived Contours with 3D Buildings (Orange) and Watercourses (Blue)

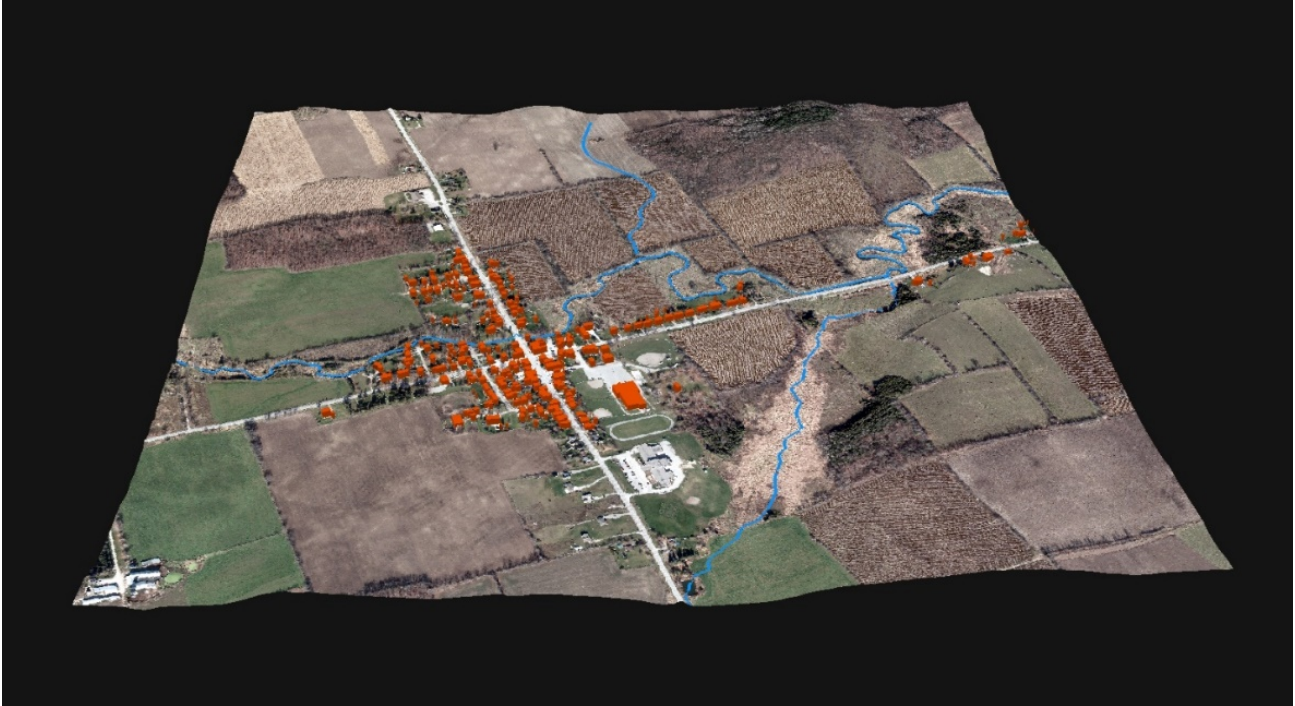


Figure 3.8: Oblique Rendering of 3D Orthoimagery with Buildings (Orange) and Watercourse (Blue)

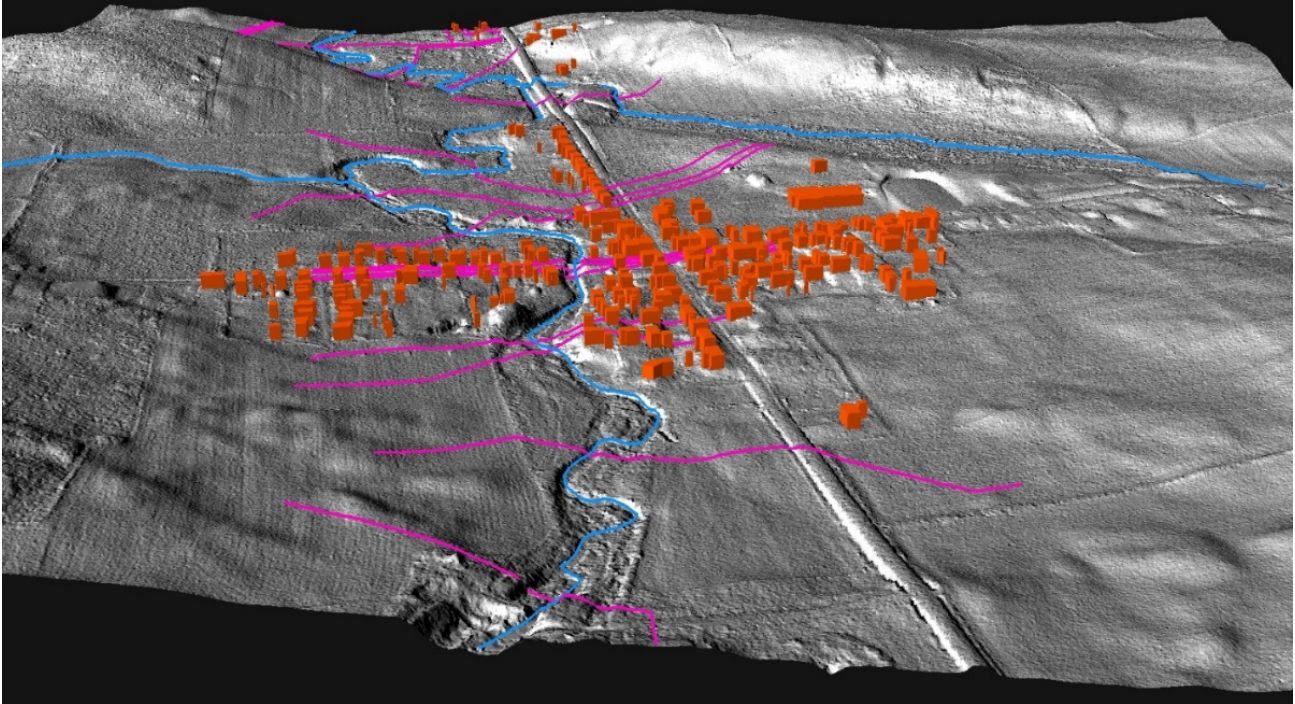


Figure 3.9: Oblique Rendering of DTM Hillshade with DTM-Derived 3D Hydraulic Cross-Sections (Pink)

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 D**: Digital Terrain Model and Orthoimagery Data Accuracy Assessment Report.

3.3 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. It has 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. It generally has the highest runoff potential.

The Mariposa Brook Subwatershed predominantly consists of drumlinized till plain and clay plain (Map P.2715 of the *Physiography of Southern Ontario*, Ontario Geological Survey). 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 3.10**. Soil types B, C, D are equally distributed throughout the subwatershed, whereas the northern portion has some pockets of type A with low runoff and high infiltration.

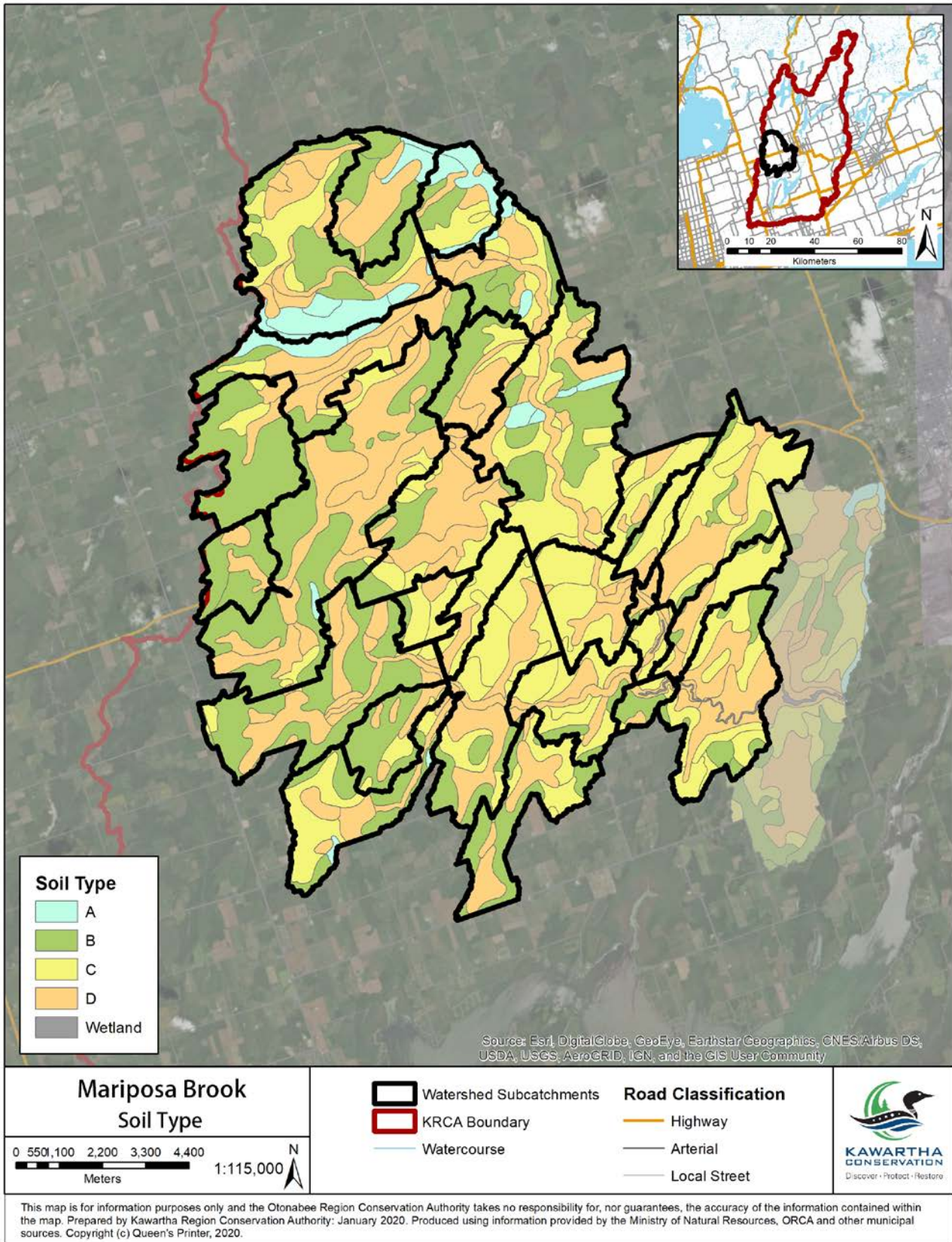


Figure 3.10: Mariposa Brook Soil Type

3.4 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, and soil type was queried to extract land use, drainage area, and hydrologic soils group data. The March 2012 Schedule 'A' for the TOWNSHIP OF MARIPOSA BY-LAW 94-07 Land Use map version as delivered by City of Kawartha Lakes was used to discretize the land use.

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 majority of the subwatershed consists of farm fields, forested landscapes and wetlands. The communities of Little Britain (through which west tributary of Mariposa Brook flows) and Oakwood have fairly large residential lots with low levels of imperviousness. The land uses for the study area are indicated in **Figure 3.11**.

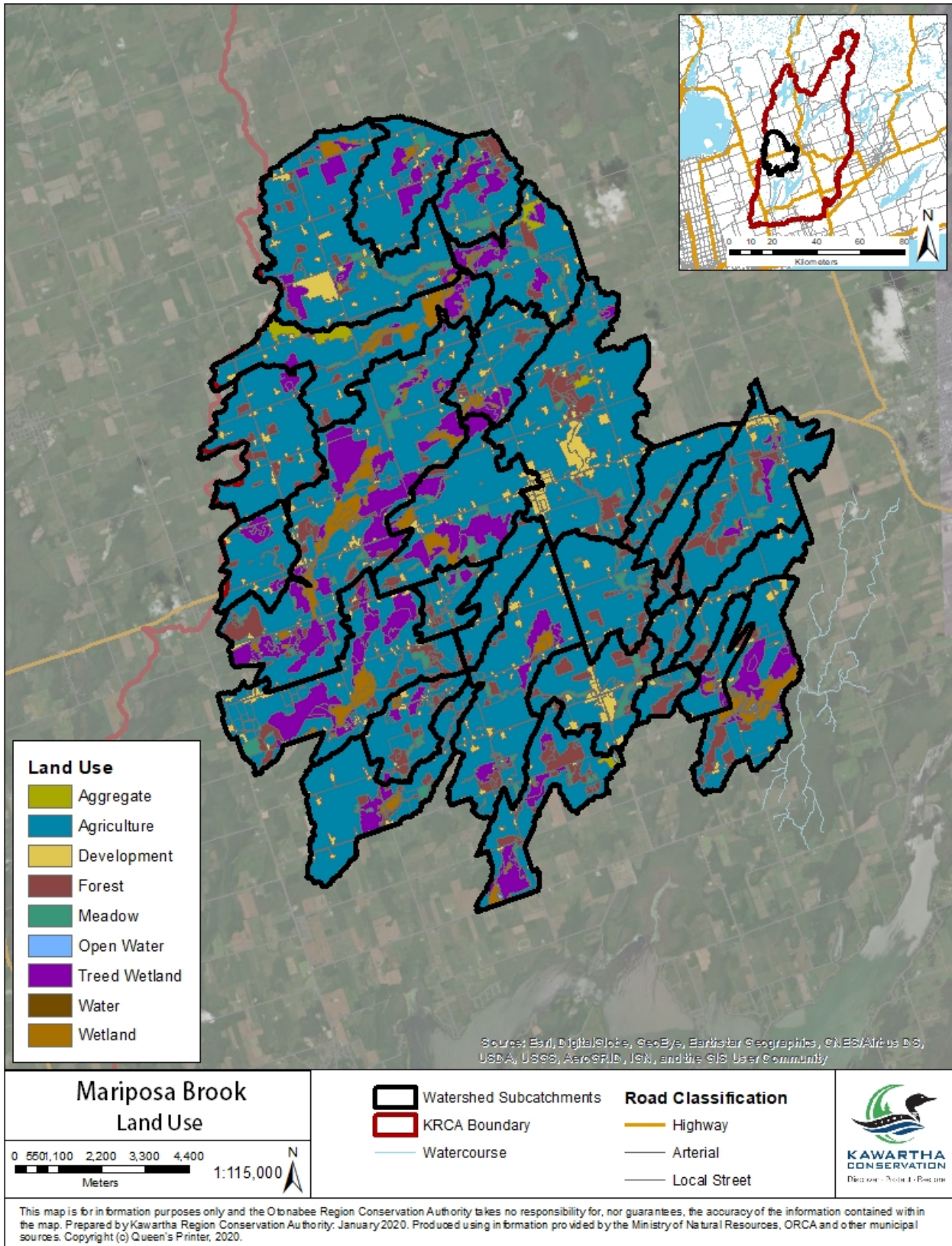


Figure 3.10: Mariposa Brook Land Use

3.5 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 (ToC). A review of the automated flowpath routes resulted in adjustments where appropriate, which were carried out manually with GIS software under the direction of the engineer for the various catchments.

3.6 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 over the subcatchment. The results are presented in **Appendix D**.

3.7 Curve Number (CN) Values

The Soil Conservation Service (SCS) curve number (CN) is used to determine runoff for rural catchments. Area-weighted CN values were calculated based on Antecedent Moisture Conditions (AMC) conditions and the land use and soil hydrologic soil group (HSG). AMC (II) conditions were applied for the Timmins storm.

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.

For this study, we have used the modified curve number method (depicted as CN*). The equations used to convert CN values to CN* can be found in **Appendix A** with a conversion summary table. IA was modified based on the relationships in **Table 3.0** (excerpt from VO5 v 5.1 technical manual).

Initial abstraction (IA) in mm was calculated as: $IA = 0.2S$

Table 3.0: CN & IA Relationship Guidelines (VO5 v.5.1 Technical Manual)

CN	IA
CN ≤ 70	IA = 0.075(S)
70 < CN ≤ 80	IA = 0.1(S)
80 < CN ≤ 90	IA = 0.15(S)
CN > 90	IA = 0.2(S)

Given the objective of this study to update the Regional and 100-year flows for Mariposa Brook, it is worth noting that the default initial abstraction values (5 mm) were applied to the model as per the study Terms of Reference and the above CN and IA relationships were not used. As such the design flows produced by the model for the 2 through 50-year storms will underestimate design flows. Should these

flows be required for use in future detailed studies, the IA parameters for the model should be revised to reflect the IA values in the above table.

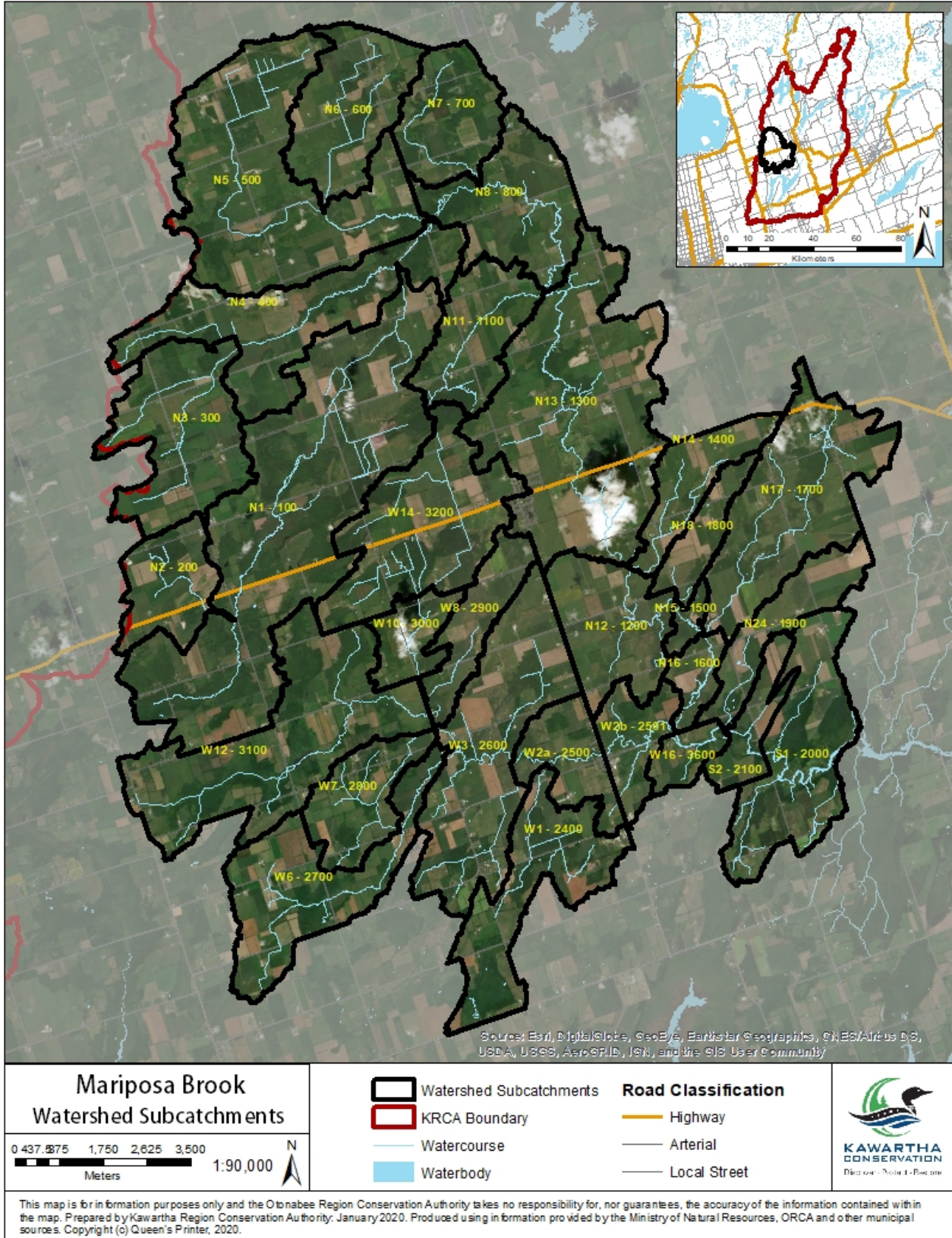


Figure 3.12: Mariposa Brook Subcatchments

3.8 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% were modeled with the StandHYD command; otherwise the NashHYD command was used. Spreadsheets with the parameter summaries and calculations are provided in **Appendix A**.

StandHYD is used to determine runoff from urban catchments and makes use of the Timp and Ximp values.

3.9 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.

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: $T_p = (2/3) * T_c$

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

3.10 Channel Flow Routing

Channel flow routing deals with unsteady flows. Unsteady flows are flows that change relative to time. Hydrographs are an excellent example of unsteady flows. The storage in the channel has a major impact on hydrographs by reducing 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.

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. Generally, channel lengths less than 100 metres in length were not included in the model routing as they do not have significant impacts to overall model results.

3.11 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
- Multiple map images
- Multiple tools to measure distance and area

The link to the Mariposa Flood Mapping Application [App] is:

<http://camaps.maps.arcgis.com/apps/webappviewer/index.html?id=6914df55ee0c42e1a7997647f9870f0e>

3.12 Other Considerations

Stormwater Management (SWM) Ponds

No SWM facilities are included in the hydrological analyses for several reasons. 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 SWM facilities are not the responsibility of the City, there is no assurance they will continue to function as originally designed.

Wetlands

There are several wetlands and waterbodies throughout the watershed. Runoff from wetlands was modeled as a regular rural subcatchment, using overland flow lengths to determine time to peak. Lakes and wetlands provide attenuation of peak flows due to their relatively flat, longitudinal profiles and therefore tend to act as reservoirs. This attenuation has been considered by applying a Curve Number of 50 for all lakes and wetlands.

3.13 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 included within **Appendix F. Table 3.1** shows the result summary of drainage area and Timmins flows compared to previous study at the key nodes.

Table 3.1: Flow Comparison to Previous Studies at Key Nodes of the Watershed

Location	KRCA-1990 Drainage Area (km ²)	KRCA-2020 Drainage Area (km ²)	KRCA-1990 Flows (cms)	KRCA-2020 Flows (cms)
West Branch	73.9	62.32	205.59	201.87
Mariposa d/s of County Road 4	197.2	182.21	373.41	310.43

The Regional storm peak flows in the west branch through the community of Little Britain are comparable to the previous study done in 1990. The resulting flows in the west branch through the community of Little Britain are comparable to the previous study done in 1990 (less than 2%) while the peak flows in the main branch downstream of Country Road 4 decreased by approximately 17%. This is largely due to the smaller and more accurate drainage area being used in the current study. The level of discretization of the drainage areas has improved over the last 30 years with the introduction of state of the art GIS and geospatial technologies, producing better results for the hydrological models.

It is worth indicating that the peak flow in the model decreases downstream of County Road 4 due to the extensive channel routing and attenuation provided by the downstream wetland and a very wide flood plain with a very low slope.

Design storm (2 year through 100-year) flows were also calculated as part of the study. All flows are summarized in **Table 3.2**.

Table 3.1: Hydrology Output

Location	River Station	Node	2yr-AES-6hr	5yr-AES-6hr	10yr-AES-6hr	25yr-AES-6hr	50yr-AES-6hr	100yr-AES-6hr	Timmins94	Timmins84
Start of North Trib	1063	7	9.27	18.43	25.66	36.01	44.51	53.88	178.65	147.3
Start of West Trib	7959	12	14.38	30.18	43.34	61.98	77.32	91.89	186.86	159.43
West of Eldon Rd	7415	13	11.54	24.93	36.33	52.84	66.54	81.54	195.23	165.96
Eldon Rd	5967	17	11.05	24.13	35.12	51.11	64.84	79.45	201.87	171.36
Little Britain Rd	2939.5 1	14	18.61	40.66	59.08	86.19	108.16	131.88	372.46	310.43
Outlet	-176	16	8.97	19.46	29.07	43.12	56.53	72.83	267.47	220.68

3.14 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 (Timmins-84 event). 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 (complete results of sensitivity analysis are at **Appendix G**):

Curve Number (CN*)

Flows- cubic meter per second (cms) 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 33% at all key flow nodes during the Timmins storm event. Decreasing CN* by 20% resulted in an average decrease in

peak flow of 29% at all key flow nodes during the Timmins 84 storm event (**Table 3.2**). Because there is a significant difference in peak flow values as a result of modifying the CN* value, it is imperative to get an accurate CN* value.

Table 3.2: Sensitivity Analysis +/-20 percent CN

Key Nodes	Original	CN*+20%	+/-(%age)	CN*-20%	+/-(%age)
7	147.30	201.48	37	100.69	-32
13	165.96	211.35	27	122.96	-26
14	310.43	411.16	32	226.43	-27
15	269.35	363.22	35	189.98	-29
		Avg. change (%)	33		-29

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. The March 2012 Schedule 'A' for the TOWNSHIP OF MARIPOSA BY-LAW 94-07 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 around 2% at all key flow nodes during the Timmins Regional storm event (**Table 3.4**). Therefore, changing the initial abstraction does not result in significantly different flows for the Regional storm.

Table 3.3: Sensitivity Analysis +/- 50% Initial Abstraction (IA)

Key Nodes	Original	IA+50%	+/-(%age)	IA-50%	+/-(%age)
7	147.30	143.43	-3	151.146	3
13	165.96	163.46	-2	168.351	1
14	310.43	303.48	-2	317.078	2
15	269.35	263.41	-2	275.742	2
		Avg. change (%)	-2		2

Channel Routing Removed

Channel routing accounts for the storage of flow as it is conveyed along the watercourse and its flood plain. This results in the attenuation of flows through a watercourse. The overall watershed involves a variety of intricate watercourses connecting subcatchments together, and therefore it is expected that removing any channel routing would result in a substantial increase in peak flows.

Removal of channel routing assumes that peak flows from catchments occurs at one point, and therefore does not consider the effect of storage and travel time as flow travels between flow nodes.

A scenario was created by removing all channel routing within the model. Removing all channel routing resulted in an average increase in peak flow of 82% at all key flow nodes during the Timmins storm event (**Table 3.5**). Therefore, channel routing has a substantial effect on peak flows throughout the watershed. Eliminating all channel routing would not be considered valid, as the watershed is very long with a number of watercourses between each catchment.

Table 3.4: Sensitivity Analysis Channel Routing Removed

Key Nodes	Original	No Routing	+/-(%age)
7	147.30	332.081	125
13	165.96	191.851	16
14	310.43	545.406	76
15	269.35	563.665	109
		Avg. change (%)	82

4.0 Hydraulics

4.1 Hydraulic Model Input Parameters

The water surface elevations that are used to determine the limits of flooding within the Mariposa Brook 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 allows for 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 version of HEC-RAS, version 5.0.7, was used for the study.

The resultant water surface profiles are considered an accurate 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 split into three reaches as shown in **Figure 4.0**.

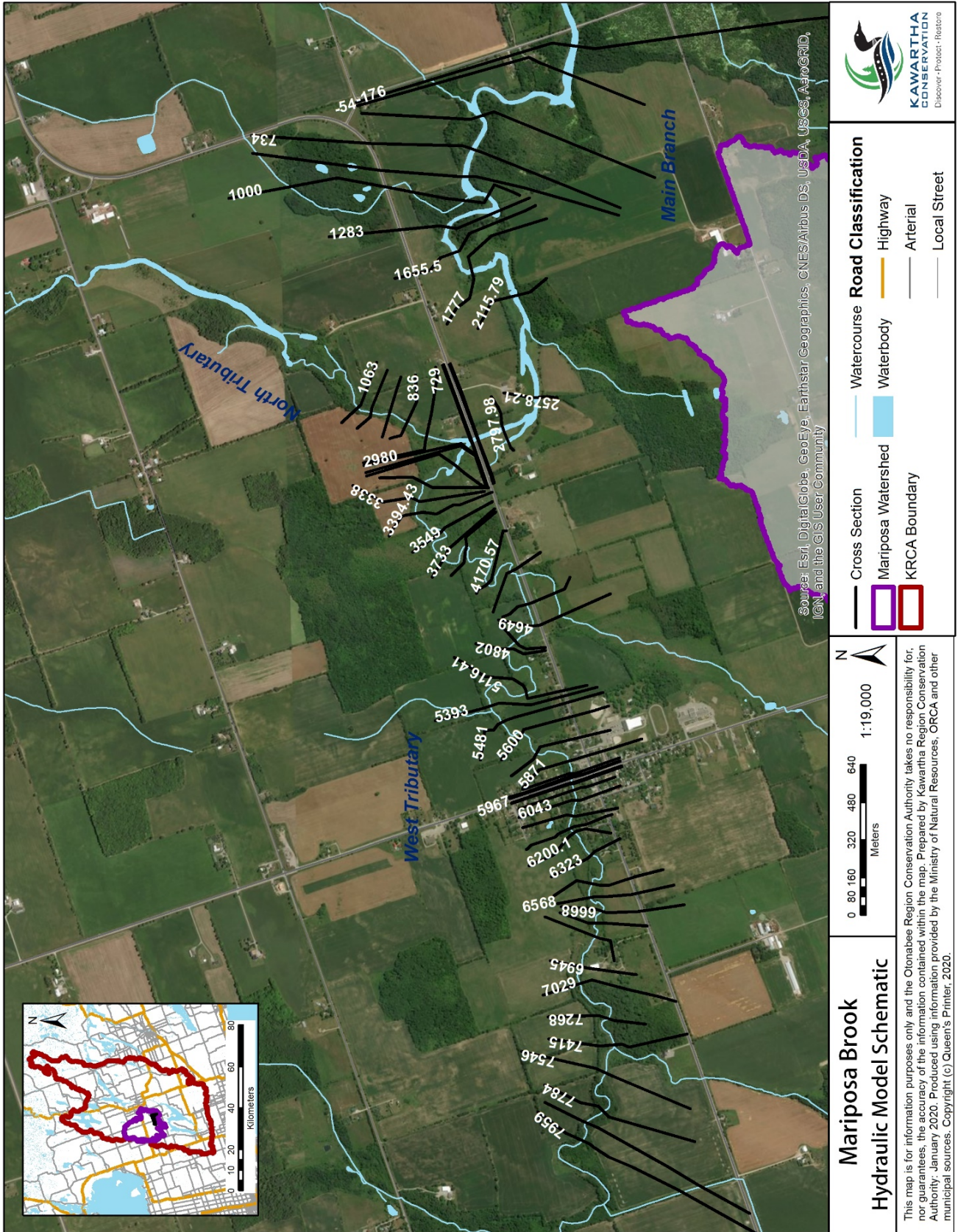


Figure 4.0: HEC-RAS Model Schematic

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 below.

Table 4.1: Flow Input Table

HEC-RAS/ID	Location	VO Flow Node	Peak Regional Flow (m ³ /s)	100-yr Flow (m ³ /s)
7959	US Eldon Rd	13	195.23	81.54
5967	DS Eldon Rd	17	201.87	79.45
1063	DS of Salem Rd	101	146.03	52.81
2939.51	US of County Rd #4	14	310.43	131.88
	DS of County Rd#4	15	269.35	99.22

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 surveyed bank-full 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 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.

4.4 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.

4.5 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.

4.6 Culvert and Road Crossing

Data for culvert and bridge crossings at roadways was obtained through a combination of a georeferenced topographic survey, the DEM and engineering drawings. 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 surveyed. Guard rails, parapet walls and fences were incorporated into the deck elevations. The two water crossing structures in the study reach of the Mariposa Brook Study Area are summarized in **Appendix I**.

Several private crossings within the study area were not surveyed due to access issues/concerns; however, they are relatively small and are not expected to significantly impact model results. 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.

4.7 Expansion/Contraction Coefficients

Contraction and expansion coefficients are specified by the user 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 4.0** 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 culvert and/or bridge crossings or abrupt transitions.

Table 4.2: Subcritical Flow Contraction and Expansion Coefficients

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

4.8 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 0.02 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 overbank areas, particularly for the reach through the community of Little Britain.

4.9 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 invert elevation. In instances where a sloping roadway deck occurs, the "Multiple Block" ineffective flow area option in HEC-RAS was utilized.

4.10 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.

4.11 Hydraulic Model Schematic

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

4.12 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 (Timmins-94). 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:

- Manning roughness coefficient (+/- 20%)
- Peak Regulatory Flow (+/- 30%)
- Downstream Boundary Condition (+/- 1.0 m)

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

Manning Roughness Coefficient

Flood elevations throughout the study reach were investigated to determine the impact of changing the Manning roughness coefficient. The Manning's number indicates the friction factor in a cross section. The higher the number, the rougher is 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 a watercourse with 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 32 cm.

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 24 cm.

Due to a minimal effect on the average, overall flood elevation throughout the study reach, it can be determined that the Manning roughness coefficients are generally found to be not sensitive on model results.

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 peak flows, it was found that the average increase in regional flood elevation throughout the cross sections was 10 cm, with the greatest of 51cm at cross section 6799.7.

By decreasing the peak flows, it was found that the average decrease in regional flood elevation throughout the cross sections was 12 cm, with the greatest decrease of 53 cm at cross section 7415.

While the flood elevations are somewhat sensitive to the peak flow rate, the variability of 30% in peak flow is considered significant. Therefore, with the model response to the variability of peak flow, the flood elevations are considered reasonable.

Downstream Boundary Condition

The Mariposa Brook flows into the wetland complex after it leaves the study area. Given that the wetland has a very flat slope (0.01%), normal depth boundary conditions were applied.

Due to the uncertainty of the starting water elevation, a sensitivity analysis was completed by varying the starting water level by approximately +/- 0.5 m above and below the water surface elevation estimated using the normal depth approach. This involved setting the water surface elevation to 251 m and 252 m. For most of the cross sections (except for the last few sections at the downstream portion of the reach), the Regional flood elevation remained unchanged.

When increasing the downstream boundary condition to 252 m, only the 5 downstream cross sections had a change in flood elevations, with a maximum of 32 cm through these sections and then it normalized to 0.

When decreasing the downstream boundary condition to 251 m the most downstream cross section's flood elevation decreased by a maximum of 62 cm with the remainder unchanged.

Due to the limited effects on flood elevations throughout the entire watershed, establishing the starting water elevation using the normal depth approach is considered acceptable for the model.

4.13 Hydraulic Model Results

The hydraulic model results show that floodlines for the current study are somewhat similar to the previous study limits however have expanded in some areas and are reduced in others. A comparison of the floodline limits along the study reach is depicted in **Figure 4.1**.

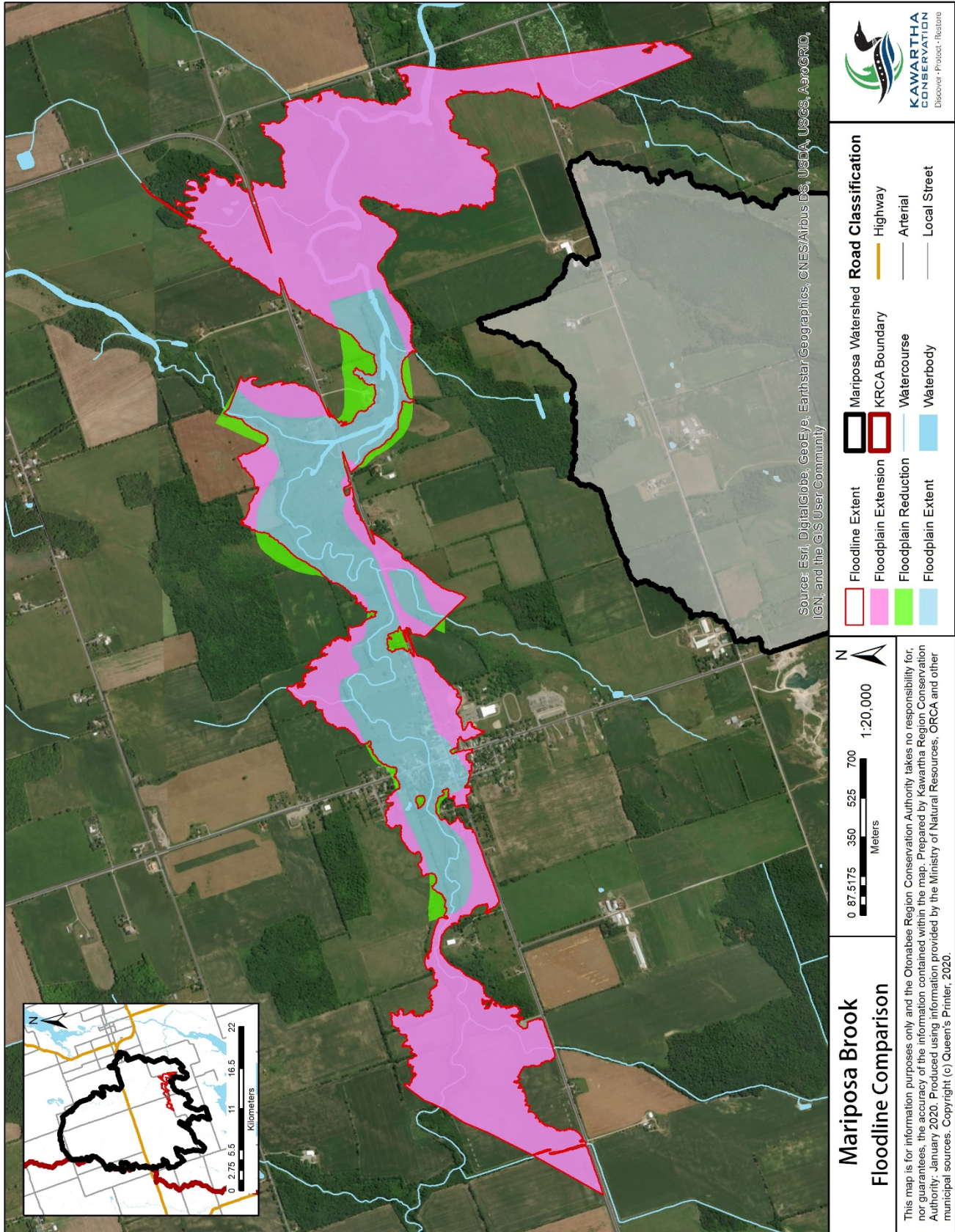


Figure 4.1: Mariposa Brook Floodline Comparison

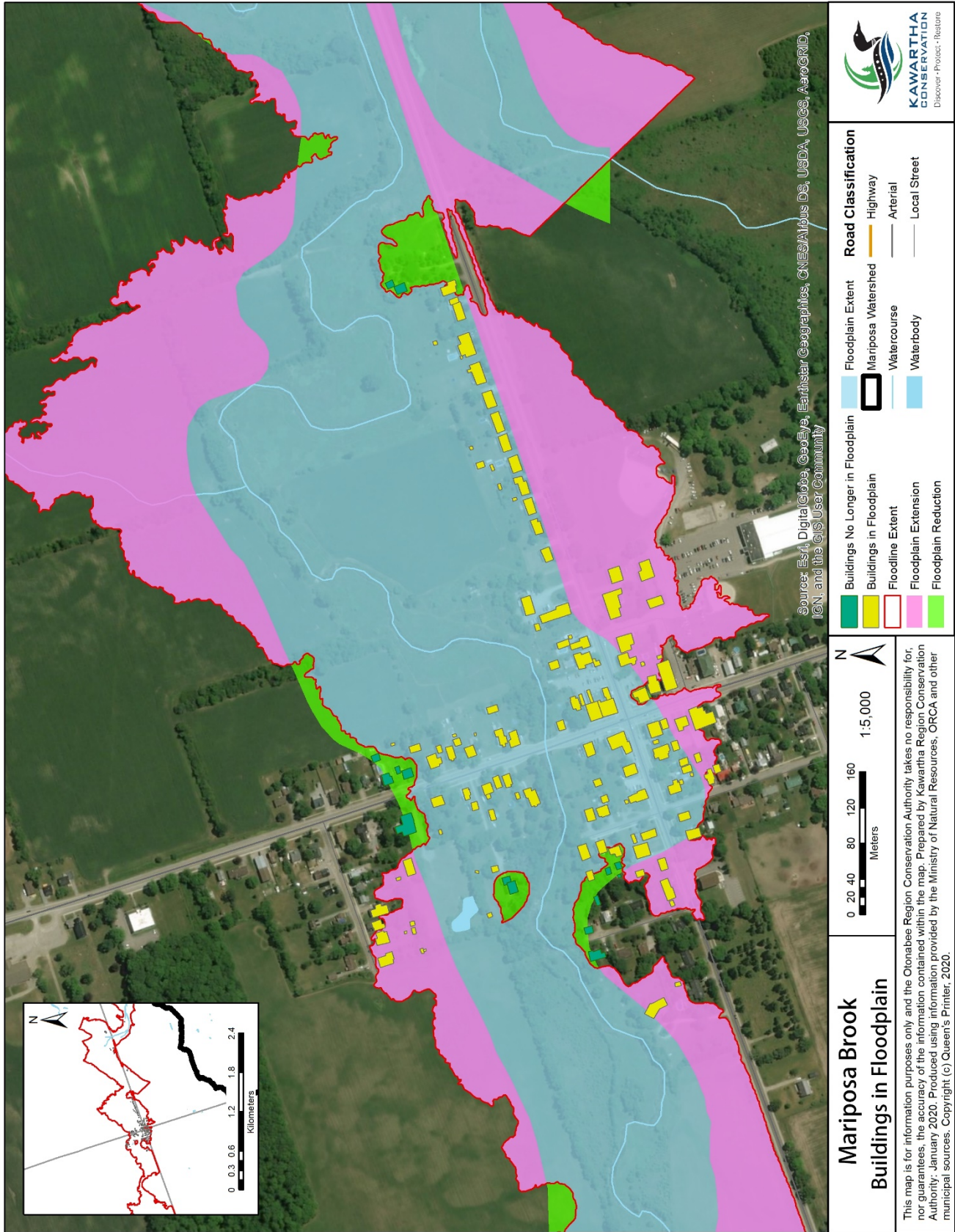


Figure 4.2: Comparison of Building Footprints Within Updated and Historic Flood Plain Limits

When compared to the previous flood plain study, the updated flood plain is somewhat wider but is more accurately representing the areas of inundation. Within the community of Little Britain, the change of flood plain limits has had an impact on the number of buildings in the flood plain. A GIS analysis was completed using the Building Footprints shapefile to calculate the number of buildings impacted by the updated floodline. A summary is provided below and can be seen in **Figure 4.2**:

Buildings in Previous Flood Plain Extent: 119

Buildings in Current Flood Plain Extent: 134

Buildings Added to the Flood Plain: 32

Buildings No Longer in the Flood Plain: 17

The updated flood plain limits through the community of Little Britain are more accurate than the previous study given the use of LiDAR and the creation of the digital terrain model and should be accepted as the new regulatory flood plain limits.

5.0 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