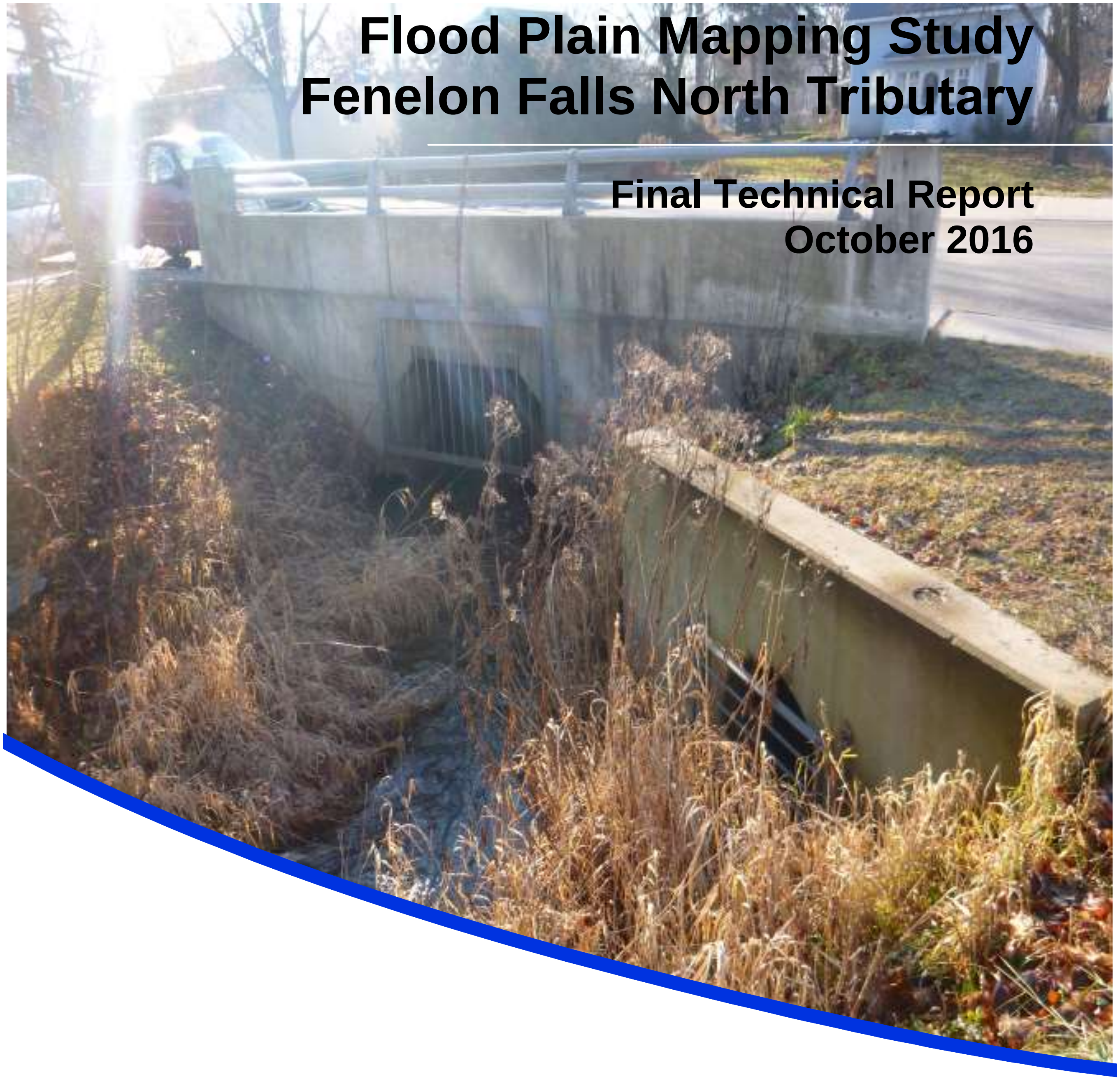
Flood Plain Mapping Study
Fenelon Falls North Tributary

Final Technical Report
October 2016



KAWARTHA CONSERVATION

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Executive Summary

The primary goals of this study are to create hydrologic and hydraulic models of the watershed and produce flood plain mapping for Fenelon Falls North Creek. 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 Fenelon Falls North Flood Plain Mapping Study has been 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 Ganaraska Conservation, the City of Kawartha Lakes, and Kawartha Conservation.

Topics discussed in this study include:

- Previous work completed
- Collection of LiDAR and Orthophoto data
- Proposed land use
- Delineation of hydrology subcatchments
- Creation of a Visual Ott-HYMO hydrology model
- Calculation of subcatchment hydrology model parameters
- Derivation of flow peaks at key nodes along the watercourse
- Survey of existing road crossing structures
- Creation of a HEC-RAS hydraulic model
- Open-channel flow analysis of the spills
- Creation of flood plain maps

Key findings of this study include:

- The flood flows are lower than what were previously in the 1996 Flood Damage Reduction Study. This is due to:
    - Although the overall catchment size is the same as what was derived in 1996, the outer boundary is different. This is due to the greater quality of mapping data available to the study team.
    - This study created thirteen subcatchments, as compared to only four in the 1996 study. This allowed the study team to refine subcatchment hydrology values.
    - Due to the greater quality of elevation data provided by the LiDAR data, more realistic overland flow routes and lengths were captured by the study team, resulting in longer times to peak for each subcatchment.
    - Channel routing in this study is based on elevation data derived from LiDAR, and provide more realistic channel slopes, lengths, and channel shapes. Flow attenuation in this model has a greater impact than on the 1996 model.
- Peak flows at key nodes are based on the 6-hour AES storm
- Flood elevations are lower than in the 1996 study. This is due to both the reduced flow rates and greater refinement of cross-section elevations derived from LiDAR data.
- Updated flood plain maps have been produced based on the output of the HEC RAS model for the Timmins storm.

Key recommendations of this study include:

- The spill areas identified in the 1996 study are upheld in this study for the following reasons:
    - The spill designation meets Ministry of Natural Resources and Forestry (MNRF) policy
    - It avoids basing flood elevations on an unknown flow rate at the Francis Street culvert, depleted by the western spill along John Street
    - Special Policy Areas (SPAs), are historical policies based on flood elevations. In recent times, the Province has been reluctant to approve new SPAs
- Should more detailed analysis of the spill areas be undertaken in the future, a two-dimensional (2D) hydraulic model is required to provide reliable and scientifically-defensible flood elevations.
- The maps created from the results of the HEC RAS model for the Fenelon Falls North watercourse should be endorsed by the Kawartha Conservation Board.

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1. Introduction

1.1. Objective

The objective of this study is to generate updated floodplain mapping for the Fenelon Falls North watercourse to protect the public from flooding hazards. This is the third flood plain study in a multi-year flood line mapping update project undertaken by Kawartha Conservation and the City of Kawartha Lakes. 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.

1.2. Study Process

At the project beginning, the Technical Committee (consisting of one representative from each of the City of Kawartha Lakes, Kawartha Conservation, and Ganaraska Conservation) created quality assurance (Q/A) and quality control (Q/C) standards to be applied to all projects in the multi-year initiative. The Q/A methodology for each component ensures a two-fold benefit: 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 meets 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:

- Mapping and air photo
- Survey data collection and integration
- Hydrology modeling
- Hydraulic modeling

For the mapping and air photo portion of the project Q/A, the City of Kawartha Lakes and Kawartha Conservation created a request for proposal (RFP) for geographic data acquisition using LiDAR technology. For the survey data collection and integration, Kawartha Conservation purchased new digital survey equipment and established procedures for survey collection. For the Q/C portion, Ganaraska Conservation's GIS staff peer-reviewed the project **D**igital **E**levation **M**odels (DEMs) and confirmed the data is in compliance with the Province of Ontario's 2009 "*Imagery and Elevation Acquisition Guidelines*" (herein referred to as the *2009 Ontario Guidelines*).

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 meet 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 and was found to be satisfactory.

1.3. Watercourse Context and Description

Rural drainage from land east of Cedar Tree Road drains to an un-named watercourse, which in turn flows into small wetland pockets. From these wetlands the creek flows southwest and forms the upstream channel of what eventually becomes the Fenelon Falls North creek. Once it reaches the urban residential area at Albert Street, the creek flows southwest, crossing under Princes Street West before doubling back southeast under John St. From this point onward, the flow is south, passing through culverts under Queen, Louisa, and Bond in somewhat natural channel form. South of Bond Street, the creek is contained within a narrow gabion basket-lined channel between a series of commercial and municipal buildings. At Francis Street West, the creek flows east within an enclosed rectangular culvert before outletting in the Trent Severn Waterway downstream of the locks south of the municipal parking lot. Please refer to **Figure 1.1**.

The upper portion of the watershed north of the Fenelon Falls is rural farmland and wetlands. Within Fenelon Falls, the watershed is mainly residential; at its outlet it drains a highly impervious commercial area. The watershed has a size of 646 hectares (6.46 km^2). The Fenelon Falls North watercourse main channel is about 6.1 km long, with an average slope of 0.7 %.

Figure 1.1: Study Area

1.4. Background Information

The Fenelon Falls North watercourse has flooded in the past, causing structural damage to existing buildings. Flooding issues appear to be the result of undersized culverts and channel, and blocked culvert inlets. Because of the historical flooding problems, several studies have been carried out in the past to attempt to understand and reduce future flooding. Relevant excerpts from key reports are included in **Appendix A**.

The engineering firm Environmental Water Resources Group (EWRG) was retained by the Kawartha Region Conservation Authority to carry out the *Village of Fenelon Falls Flood Damage Reduction Study* in December 1996. The firm calculated runoff for the Fenelon Falls North watershed, created flood plain maps, estimated flood damages, and brought forward recommendations to reduce flooding problems. The firm used 1977 air photos and 1”=200’ scale topographical maps with 5ft contours within Fenelon Falls town limits, and 1992 air photos and 1984 1:10,000 scale topo maps with 5m contours for the rural lands outside town limits to delineate subcatchment areas. The computer model Ott-HYMO was used to simulate design storm runoffs using rainfall data from the Atmospheric Environment Services’ (AES) rain gauge at the Lindsay sewage plant. Calibration was not possible since no rain or flow monitoring had been carried out. The firm verified their hydrology model by comparing peak flow rates using regional frequency analyses. The consulting company ran steady state modeling using HEC-RAS to assess the capacity of the watercourse and determine flood plain extents. Three separate models were created to simulate the unusual hydraulic conditions of the watercourse: the first model simulated flooding between the Trent-Severn waterway and the downstream end of the Francis St rectangular buried culvert; the second model simulated flooding between the upstream and downstream ends of the buried culvert; the third model simulated the watercourse upstream of the buried culvert to the study limits. The study determined five (5) spill areas on the flood maps. Relevant excerpts are found in **Appendix A.1**.

In 2005 the engineering consulting firm Totten Sims Hubicki (TSH) carried out the *Stormwater Drainage Study 2004 Update – Phase 1 Fenelon Falls and Immediate Area* for the City of Kawartha Lakes. This was an update to their previous 1978 study (no copy of which is available for review by the current study team) of the village’s stormwater minor system (i.e. storm sewer system). The main outputs from the 2005 study are a Rational Method analysis of the minor system, a structural assessment of the Francis Street rectangular box culvert, and capacity analyses of the Francis Street and Bond Street culverts. Relevant excerpts are found in **Appendix A.2**.

1.5. Modeling Approach

Flooding was assessed using standard steady flow methods derived using Visual Ott-HYMO Suite 3.0 (VH Suite 3) and HEC-RAS version 4.10.

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 B**), and to calculate values such as imperviousness, SCS Curve Numbers (CN), time to peak (T_p), and time of concentration (T_c).

Urban subcatchments have been delineated reviewing engineering reports and field inspection for the Fenelon Falls North watercourse, where applicable.

Runoff hydrographs have been generated for the 2-, 5-, 10-, 25-, 50-, and 100-year events as well as the Regional (Timmins) storms. The source rainfall data utilized for this analysis is from Environment Canada's rain gauge that was historically located at the Lindsay Filtration Plant.

Sensitivity analyses have been carried out to determine the impact of changing model parameters on the calculated flows. No flow monitoring data is available to calibrate the hydrologic model. 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 VH Suite 3 and HEC-RAS.

Taking such an approach results in realistic peak flows and associated flood lines along the Fenelon Falls North watercourse. Comparisons of all results to previous studies will be undertaken to evaluate the change in floodplain elevations and extents.

2. Rainfall

2.1. Rainfall Data

Rainfall Intensity–Duration–Frequency (IDF) curves define the rainfall input for modeling and provide estimates of the extreme rainfall intensity for different return periods. Rainfall volumes are taken from Lindsay's Atmospheric Environment Services (AES) gauge which was removed from service in 1989. In the initial flood plain study 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 time span, 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.

As outlined in the June 2014 *Flood Plain Mapping Study, Ops #1 Drain/Jennings Creek* report, several rainfall sensitivity analyses were carried out to see the effect on peak flows and associated flood elevations in the Ops #1 drainage basin. The initial analysis adjusted the total Lindsay rainfall volumes +/-10%. The second analyses used the Peterborough AES gauge data. Increasing the Lindsay 100-year rainfall volumes by 10% caused an insignificant increase in flood elevation in the Lindsay commercial district; decreasing the rainfall volume by 10% did not cause an appreciable difference in flood elevation. When the 100-year Peterborough AES gauge data was input to the models, no difference in flood elevations was noted in the Lindsay commercial district. The Lindsay AES gauge data was therefore used for all analyses in the Ops#1/Jennings Creek flood plain study. It was decided that for all subsequent flood plain studies, the Lindsay IDF data would be used for two key reasons: to provide continuity from study to study, and because City of Kawartha Lakes infrastructure has been designed using this gauge data. Details of the Peterborough-Lindsay rain comparison are found in **Appendix C**.

The Ontario Ministry of Natural Resources (MNR) technical manuals provide a rainfall reduction table for the Timmins storm. Given the size of the Fenelon North subcatchment no areal reduction factors are used.

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

$I = a/(t+b)^c$, where

I in mm/hr

T in minutes

The City of Kawartha Lakes engineering design standards state the relevant IDF parameters for the gauge are:

Table 2.1: IDF Parameters in the City of Kawartha Lakes' Engineering Standards

Return Period (yr)	A	B	C
2	628.107	5.273	0.78
5	820.229	6.011	.768
10	915.845	6.006	.757
25	1041.821	6.023	.748
50	1139.702	6.023	.743
100	1230.783	6.023	.738

Through the course of the 2013 *Ops #1 Drain/Jennings Creek Flood Plain Mapping Study* it was discovered that when the a, b, and c parameters listed above were input into the hydrology models, the corresponding total rainfall volumes generated for a 12-hour storm overestimated the measured AES volumes by as much as 25%. As a result, Kawartha Conservation staff re-calculated the a, b, and c parameters (listed below in **Table 2.2**). These values calculate rainfall depths within 1% of the measured volumes shown in **Table 2.3**. These are the values used for the base hydrology scenarios.

Table 2.2: IDF Parameters calculated by Kawartha Conservation

Return Period (yr)	A	B	C
2	808.299	7.413	0.835
5	1248.097	9.76	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 2.3: 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

Table 2.4, **Table 2.5**, and **Table 2.6** compare the 6-, 12-, and 24-hour volumes using the City's and KRCA's a, b, and c parameters. Details of the a, b, and c parameter recalculations are found in **Appendix C**.

Table 2.4: Comparing 6-hour Rainfall Volumes (City vs. KRCA IDF equations)

Return Period Storm	Rainfall Volumes (mm)				
	Measured	CKL a, b, c	% Diff	KRCA a, b, c	% Diff
2	36.6	37.8	103%	35.0	96%
5	50.8	52.9	104%	47.1	93%
10	60.2	63.0	105%	55.6	92%
25	72.1	75.6	105%	65.6	91%
50	80.9	85.2	105%	73.7	91%
100	89.7	94.7	106%	81.1	90%

Table 2.5: Comparing 12-hour Rainfall Volumes (City vs. KRCA IDF equations)

Return Period Storm	Rainfall Volumes (mm)				
	Measured	CKL a, b, c	% Diff	KRCA a, b, c	% Diff
2	39.8	44.3	111%	39.6	99%
5	53.2	62.5	117%	52.6	99%
10	62.2	75.0	121%	62.1	100%
25	73.4	90.6	123%	72.7	99%
50	81.8	102.4	125%	81.7	100%
100	90.1	114.3	127%	89.6	99%

Table 2.6: Comparing 24-hour Rainfall Volumes (City vs. KRCA IDF equations)

Return Period Storm	Rainfall Volumes (mm)				
	Measured	CKL a, b, c	% Diff	KRCA a, b, c	% Diff
2	43.6	51.7	119%	44.5	102%
5	56.4	73.6	131%	58.5	104%
10	64.8	89.1	137%	68.9	106%
25	75.4	108.2	143%	79.9	106%
50	83.3	122.7	147%	89.9	108%
100	91.2	137.5	151%	98.2	108%

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
A variety of rainfall durations (6, 12, and 24 hours) for 2-100 year return periods are tested.

Storm Distribution
Rainfall distribution is the specific apportionment of rain over time, or the shape of the storm. The relative importance of these factors varies with the characteristics of a subcatchment. It is standard practice to test different design storms to determine the most conservative flows.

For more than 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 Chicago storm distribution is one of the commonly used distributions for designing and analyzing storm sewer systems in urban areas. The distribution of rainfall is generally in the centre of the storm and the peak of storm is quite intense.

The worst case storm (the duration and distribution producing the highest discharges at key nodes) is selected as the critical event for the watershed. This will provide the most appropriate protection for the community of Fenelon Falls. Detailed rainfall information is shown in **Appendix C**.

2.3. Regional Storm

The Timmins storm with a total rainfall of 193mm is the Regional storm event for this part of Ontario. The full storm is defined by Chart 1.04 of the *MTO Drainage Manual*. Antecedent moisture content (AMC) condition II, referred to as AMC (II), was applied. An aerial reduction factor was not applied to the Regional model.

2.4. Snowmelt and Snowmelt/Rainfall Events

These types of analyses were not carried out for this report.

2.5. Climate Change

Climate change considerations were not included within the terms of reference for this study.

3. Hydrology Model Input Parameters

3.1. Overview

In 2012, the City of Kawartha Lakes and Kawartha Conservation produced a standardized methodology for undertaking flood plain mapping studies within their jusrisdictions. This approach was peer-reviewed by Greck and Associates Limited, and their findings conclude the methodology is valid. All parameters and modeling approaches described within this report follow the recommendations presented in **Appendix B** unless otherwise noted. For this study Kawartha Conservation extracted hydrologic parameters from LiDAR elevation data, Arc Hydro watershed boundaries, Official Plan, Secondary plan, zoning data, and field surveys.

3.2. Digital Elevation Model (DEM)

A LiDAR and orthoimagery full-suite remote sensing data were acquired by the City of Kawartha Lakes in 2012. The acquisition included orthoimagery, LiDAR-derived point cloud data, elevation raster tiles, and other geospatial/non-geospatial datasets produced by the vendor. At the time of the acquisition, the *2009 Ontario Guidelines* was the technical document that set geospatial data acquisition specifications in Ontario and defined geospatial data accuracy targets based on levels or risk.

For the Fenelon Falls North watercourse watershed, two points per square meter LiDAR data was acquired. ArcGIS version 10.1 computer software programs translated were utilized to produce bare earth and hydrologically-conditioned DEMs at 0.5m cell resolution.

Using the accuracy testing and reporting components of the 2014 American Society of Photogrammetry and Remote Sensing (ASPRS) standards for quantifying, testing, and reporting accuracy of geospatial data ("*ASPRS Positional Accuracy Standards for Digital Geospatial Data (2014)",* a Q/C of the vendor-provided DEM was undertaken to determine the positional accuracy of the digital geospatial data. The DEM was found to be in compliance with *2009 Ontario Guidelines*. The accuracy of the base DEM was determined as follows:

> *This dataset was tested to meet ASPRS Positional Accuracy Standards for Digital Geospatial Data (2014) for a 15cm RMSEz Vertical Accuracy Class. Actual NVA accuracy was found to be RMSEz = 0.17m, equating to +/- 0.33m at 95% confidence level. Actual VVA accuracy was found to be +/- 0.42m at the 95th percentile.*

3.3. Orthoimagery

The *2009 Ontario Guidelines* also states the minimum horizontal geospatial data accuracy to be used for the risk. The 2014 ASPRS standards was used to carry out full Q/C testing of the horizontal accuracy of the orthoimagery. The orthoimagery data accuracy was determined as follows:

This dataset was tested to meet ASPRS Positional Accuracy Standards for Digital Geospatial Data (2014) for a 30.0cm RMSEx/RMSEy Horizontal Accuracy Class. Actual NVA accuracy was found to be RMSEx = 0.29m and RMSEy = 0.23m which equates to Positional Horizontal Accuracy = +/- 0.63m at the 95% confidence level.

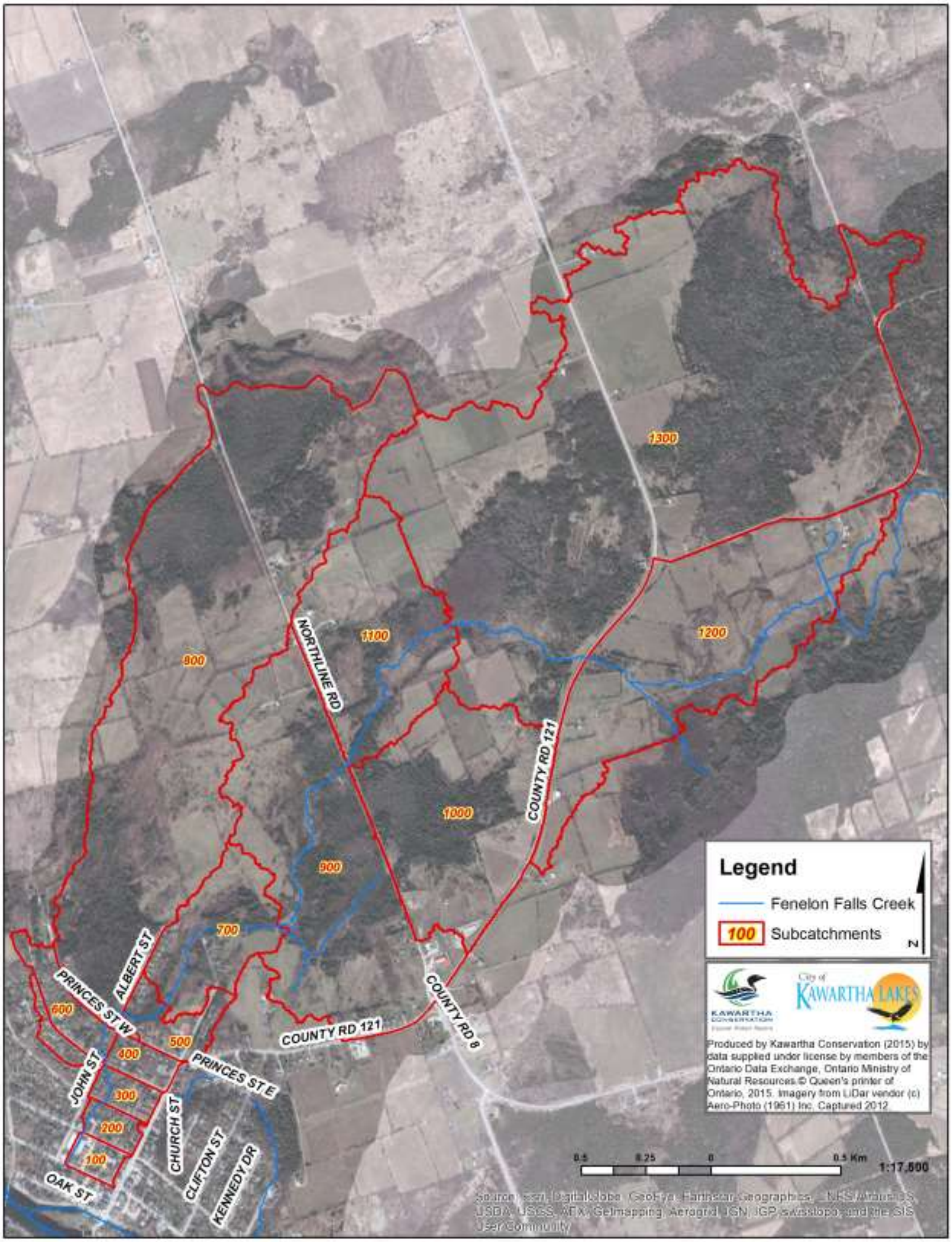
3.4. Subcatchment Discretization

In order to discretize subcatchments, watershed flow paths were generated using ArcHydro version 10.1 beta software. Surveyed bridge and/or culvert data was merged into the vendor-derived DEM to create a hydrologically-conditioned DEM. This allows flow connections under road barriers to a downstream channel or subcatchment; flow barriers and other impediments are therefore removed from GIS calculations.

Critical nodes within the watershed were selected by the engineer as the basis to delineate the initial subcatchments in ArcHydro. ArcHydro is suitable for the delineation of rural subcatchments.

For urban subcatchments the ArcHydro tool cannot account for sub-surface pipe networks nor can it determine overland flow pathways where the topography forms a concave shape. To overcome this gap, field visits were carried out to verify urban subcatchment boundaries. Manual adjustments of the urban subcatchments were carried out under the direction of the engineer and approval of the technical committee. **Figure 3.1** illustrates the creek subcatchments.

Figure 3.1: Subcatchment Boundaries



3.5. Land Use

The July 2012 Schedule ‘A’ Land Use Plan for the Village of Fenelon Falls, and the March 17, 2011 Schedule ‘A-5’ for the Verulam and Fenelon Townships within the City of Kawartha Lakes’ Official Plan (OP) are the base data referenced for land use patterns. The July 2012 Schedule ‘A’ zoning map for the Village of Fenelon Falls is also used for reference.

Land values in the hydrology model do not reflect current land use; instead, the model assumes that all developable areas indicated in the Official Plan are fully built out. The rationale for this decision is that the City has approved in principle the proposed land use and therefore the flood lines should reflect the most conservative flood scenario. Copies of the schedules’ maps are found in **Appendix T**.

3.6. Rural Subcatchment Properties

The longest flow paths of each rural subcatchment were derived using ArcHydro. In this process, the downstream node is selected by the user, and ArcHydro calculates the longest overland and channel flow paths. **Appendix D** contains a series of figures showing each subcatchment and their respective lengths.

3.7. Calculation of Slope

For rural subcatchments, spreadsheets were created that calculate channel and subcatchment slopes, based on overland and channel flow data. Details can be found in **Appendix E**.

3.8. CN Values

The Soil Conservation Service (SCS) curve number (CN) is used to determine runoff. Users must choose which antecedent moisture condition (AMC I, II, or III) is relevant for the model; AMC II represents a dry soil condition, and AMC III represents saturated soil. 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. A weighted CN (AMC II) value was calculated, as shown in **Appendix E**.

The VH SUITE 3 program requires that the CN value be transformed to CN* (AMC II). These calculations are included in **Appendix E**. **Figure 3.2** provides soils information while **Figure 3.3** shows the future land use of the watershed, based on Official Plan (OP) and/or Secondary Plan data. Spreadsheets with the calculations are provided in **Appendix E**.

Figure3.2: Soils

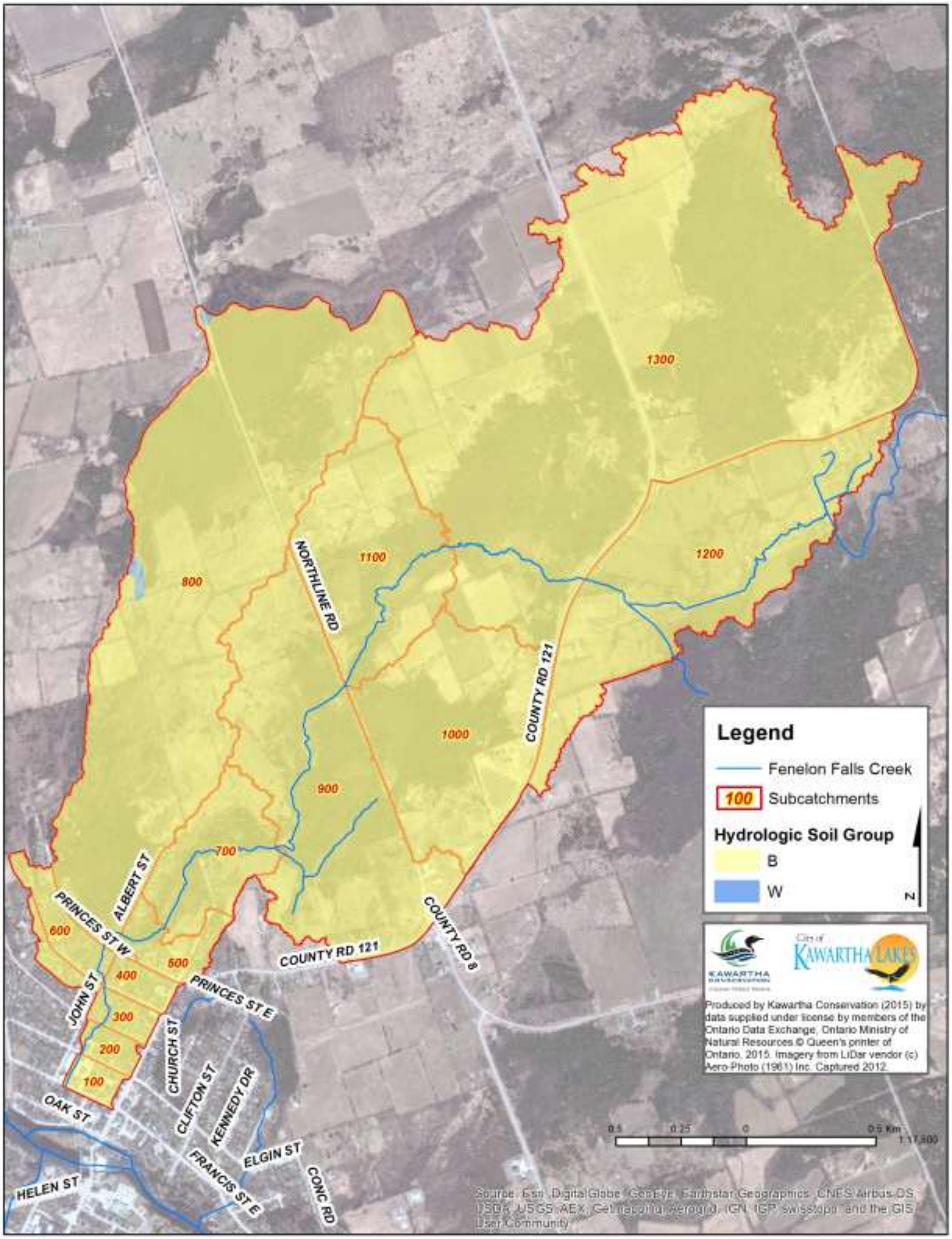
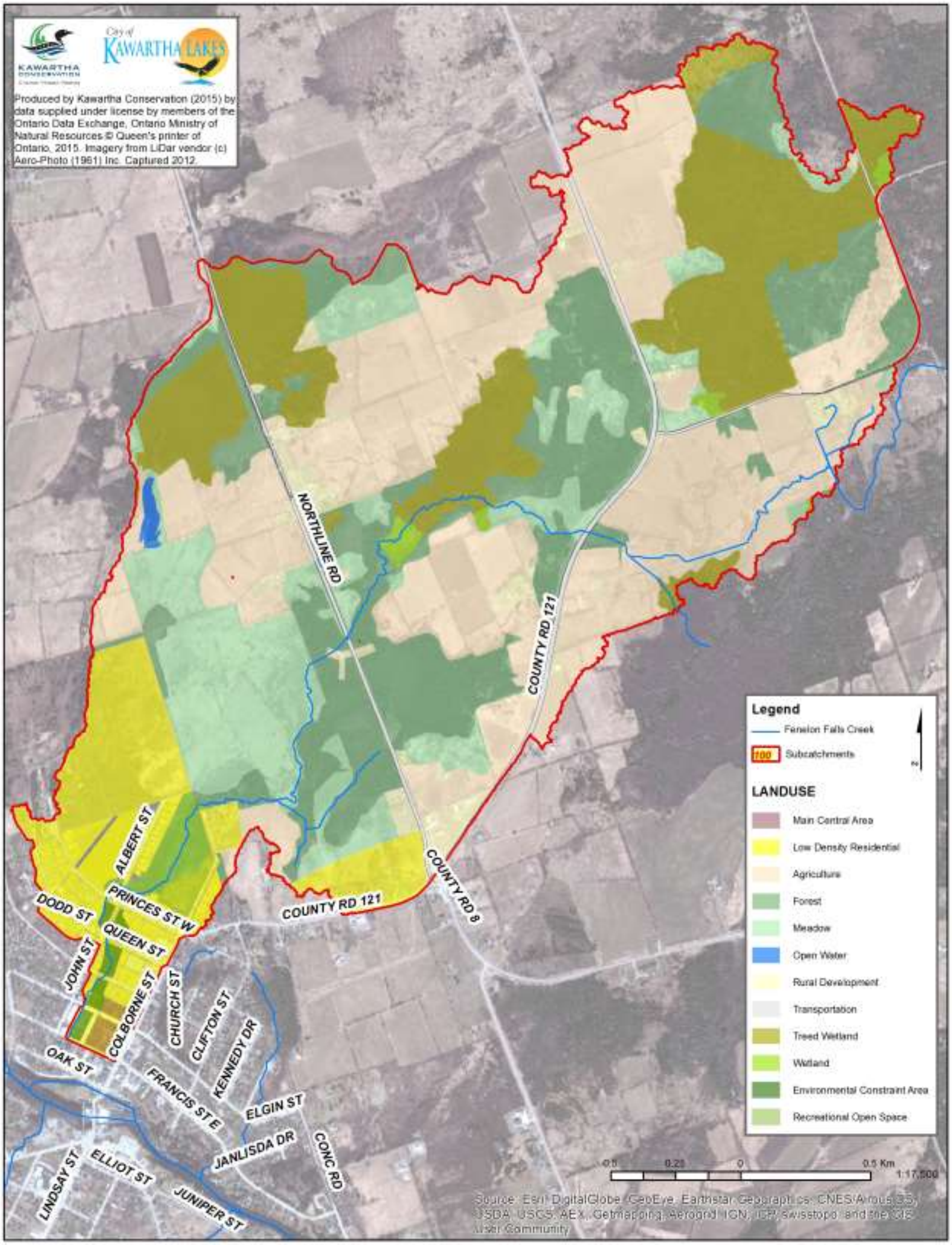


Figure 3.3: Land Use



3.9. Impervious Land Use & Runoff Coefficients

The detailed land use denoted in the OP, Secondary plan, and zoning data determine the weighted total impervious area (T_{imp}), directly-connected impervious area (X_{imp}), and runoff coefficient (C) for each subcatchment using the tables from the Hydrologic Parameters List in **Appendix B**.

Subcatchments with a T_{imp} value greater than 20% were modeled with the StandHYD command; otherwise the NashHYD command was used. Spreadsheets with the calculations are provided in **Appendix E**.

3.10. Time of Concentration

Time of concentration (T_c) is a key variable for calculating peak flow. This is the time it takes for the flow wave to travel from the hydraulically farthest point of a subcatchment to the subcatchment's downstream node.

Time of concentration was calculated using the Airport method for subcatchments with a C value less than 0.4; the Bransby-Williams method was chosen if the C value exceeded 0.4.

The Time to Peak (T_p) is defined by VH SUITE 3 model via the equation: $T_p = (2/3) * T_c$

Time to peak is used in the NashHYD command only. For urban subcatchments, neither the T_c or T_p are used. Spreadsheets with the T_c and T_p calculations are found in **Appendix E**, using the flow lengths shown in the subcatchment figures found in **Appendix D**.

3.11. Channel Routing

Channel routing in VH SUITE 3 accounts for the time lag of flows being routed in the main channel. HEC-RAS cross sections are input to the Route Channel command within VH SUITE 3. One representative cross-section from HEC-RAS was selected for each channel reach. Reach channel and overbank Manning's n values were averaged, as were the channel and overbank slopes.

3.12. 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 flood upon which 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.

4. Hydrologic Model

4.1. Schematic

The information gathered in the preceding sections was used to build a VH SUITE 3 model of the watershed, as shown schematically in **Appendix F.**

4.2. Calibration

Since no rain or flow gauge data is available for this watershed, no calibration can be performed.

4.3. Sensitivity Analyses

The model was tested for sensitivity for the following input parameters: Manning's n, CN values, initial abstraction, model time step, removal of channel routing, channel flow lengths, and straight-line overland flow lengths. Detailed information can be found in **Appendix H**.

MANNING's n

The manning's n for all channel cross-sections were modified $^{+}/_{-}$ 20%. Flows at key nodes were investigated to see the impact of the changes. When a 20% increase was applied to the channel manning n values (thus simulating a channel with a rougher surface), the model calculated an average 4% decrease in peak flows for the Timmins event, and an average 8% decrease for the 100-year event. Similarly, when the manning's n values were decreased by 20%, the model calculated higher peak flows at key nodes, by an average of 6% for the Timmins storm, and an average of 10% for the 100-year event. The n value is not a sensitive input parameter.

CN*

Flows at key nodes were investigated to see the impact of changing the CN* value. When CN* increased 20%, the model calculated an average 22% increase in peak flows for the Timmins event, and an average 41% increase for the 100-year event. Similarly, when CN* decreased 20%, the model calculated lower peak flows at key nodes: by an average of 23% for the Timmins storm, and by an average of 30% for the 100-year event. 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.

CN* is determined by land use and soil type. Soil type information is extracted from the digitized Victoria County soils map that had been originally produced as a joint venture by the federal department of agriculture and the Ontario Agricultural College. Land use is derived from the City of Kawartha Lakes' Official and Secondary Plans and zoning maps as well as the 2010 ELC mapping. This base data is valid, and therefore any calculated value (such as CN*) based on this data truly represents the land.

Since CN* is derived directly from measured parameters whose values are valid, there is confidence that the calculated CN* is correct.

Initial abstraction (I_a)
The initial abstraction was changed +/- 50%. Decreasing I_a by 50% increases the peak flows by an average of 1% for the Timmins storm, and by 4% for the 100-year storm. Increasing I_a by 50% increases the peak flows by an average of 1% for the Timmins storm, and by 4% for the 100-year storm. Therefore changing the initial abstraction does not result in significantly different flows.

Model Time Step (DT)
The model time step was changed +/- 50%. No difference was noted in peak flows at key nodes. Therefore changing the time step does not result in significantly different flows.

Channel routing removed
The model was modified as if there were no channel routing. For the Timmins storm, peak flows increased by an average of 20% at key nodes; for the 100-year storm, peak flows increased by an average of 60%. This is caused by the lack of attenuation in the channels. The inclusion of channel routing is therefore a significant item. Since the channel length, slope, and cross-section information is derived from a highly-detailed DEM, there is confidence that the data is correct.

Channel Flow Length
The channel flow lengths were modified +/- 20%. Increasing the length by 20% decreases the peak flows by an average of 5% for the Timmins storm, and by an average of 10% for the 100-year storm. Decreasing the length by 20% increases the peak flows by an average of 4% for the Timmins storm, and by an average of 10% for the 100-year storm. Changing the channel length does not result in significantly different flows.

Overland Flow Length
The peer reviewer suggested carrying out a sensitivity analysis for the overland flow length by assuming a straight line between the furthest upstream point and the lowest discharge point in each rural subcatchment. This would have an immediate impact on the Time to Peak (T_p) input parameter for rural subcatchments 700 through to 1300. The average decrease in times to peak ranged 8% to 44%. The increase in peak flows at key nodes averaged 12% for the Timmins storm, and 18% for the 100-year event. The overland flow length therefore has a significant impact of calculated flows. Since it is derived from a highly-detailed DEM, there is confidence that the data is correct.

4.4. Model Input Data

Since the upper portion of the watershed is rural, its subcatchments (700 to 1300), are modeled using the NashHyd command. The lower subcatchments within the village are urban, and are modeled using the StandHYD command. The model input data are highlighted in **Table 4.1** below. More details can be found in **Appendix E**.

Table 4.1: Ott-Hymo Model Input Parameters

Catchment	Area (Ha)	C	T_p (hr)	CN* (II)	CN* (III)	Ia (mm)	X_{imp}	T_{imp}
100	2.8	0.90	*N/A*	67	83	1.5	0.70	0.79
200	3.0	0.60	*N/A*	85	94	1.5	0.43	0.51
300	3.4	0.48	*N/A*	78	90	1.5	0.24	0.36
400	6.0	0.45	*N/A*	80	91	1.5	0.24	0.35
500	6.9	0.36	*N/A*	73	87	1.5	0.14	0.20
600	6.5	0.45	*N/A*	74	88	1.5	0.24	0.35
700	19.6	0.31	0.5	63	80	5.0	*N/A*	*N/A*
800	139.2	0.33	2.0	60	78	5.0	*N/A*	*N/A*
900	70.7	0.34	0.9	64	81	5.0	*N/A*	*N/A*
1000	49.7	0.32	1.0	67	83	5.0	*N/A*	*N/A*
1100	40.4	0.26	0.8	64	81	5.0	*N/A*	*N/A*
1200	67.3	0.34	1.2	73	87	5.0	*N/A*	*N/A*
1300	230.3	0.23	2.6	57	75	5.0	*N/A*	*N/A*

5. Hydrology Model Output

5.1. Comparing model inputs: 1996 EWRG vs. 2015 Kawartha Conservation

The Fenelon Falls North watercourse was modeled in 1996 by Environmental Water Resources Group (EWRG). As discussed in the previous section, Kawartha Conservation re-created the hydrologic breakdown using the most recent LiDAR and GIS data. Differences between the 1996 and 2015 data were discovered with respect to drainage areas, land use, and ground elevation.

Tributary Area

Area differences are highlighted in **Table 5.1** and in **Figure 5.1.** EWRG created five (5) subcatchments, Kawartha Conservation uses thirteen (13). When comparing cumulative areas at key nodes, the EWRG and Kawartha Conservation discretizations are close (within 85 % total tributary area values); the exception is at the upstream reaches, where the Kawartha Conservation discretization has smaller areas, in the range of only 23% - 69% of EWRG's areas.

Table 5.1: Comparing Tributary Areas at Key Nodes

Key Node Location	Tributary Area (Ha)		*% Diff*
	EWRG	**Kawartha Conservation**	
County Rd 121	290	67	*23%*
Northline Road	490	338	*69%*
Albert St W	590	485	*82%*
Princes St W	730	634	*87%*
John St	--	630	--
Queen St	--	637	--
Louisa St	--	640	--
Bond St W	--	643	--
Francis St W	759	646	*85%*

The reason for the differences in the upper subcatchments is that the Kawartha Conservation staff has access to LiDAR data which is a more accurate representation of the topography. Three subcatchments in the upstream portion of the watershed are significantly different that what was delineated by EWRG, as highlighted in **Figure 5.2**, and **Figure 5.3**, and explained below:

- The EWRG subcatchment FF4 (140 Ha) is essentially the same size as Kawartha Conservation's subcatchment 800 (139 Ha). EWRG's boundaries extend further west than the Kawartha Conservation boundary but does not extend as far east as the Kawartha Conservation boundary. Field work by Kawartha Conservation staff has determined that the area east of Northline Rd is hydrologically connected to the land west of Northline Rd.

Figure 5.1: EWRG vs. Kawartha Conservation Subcatchments

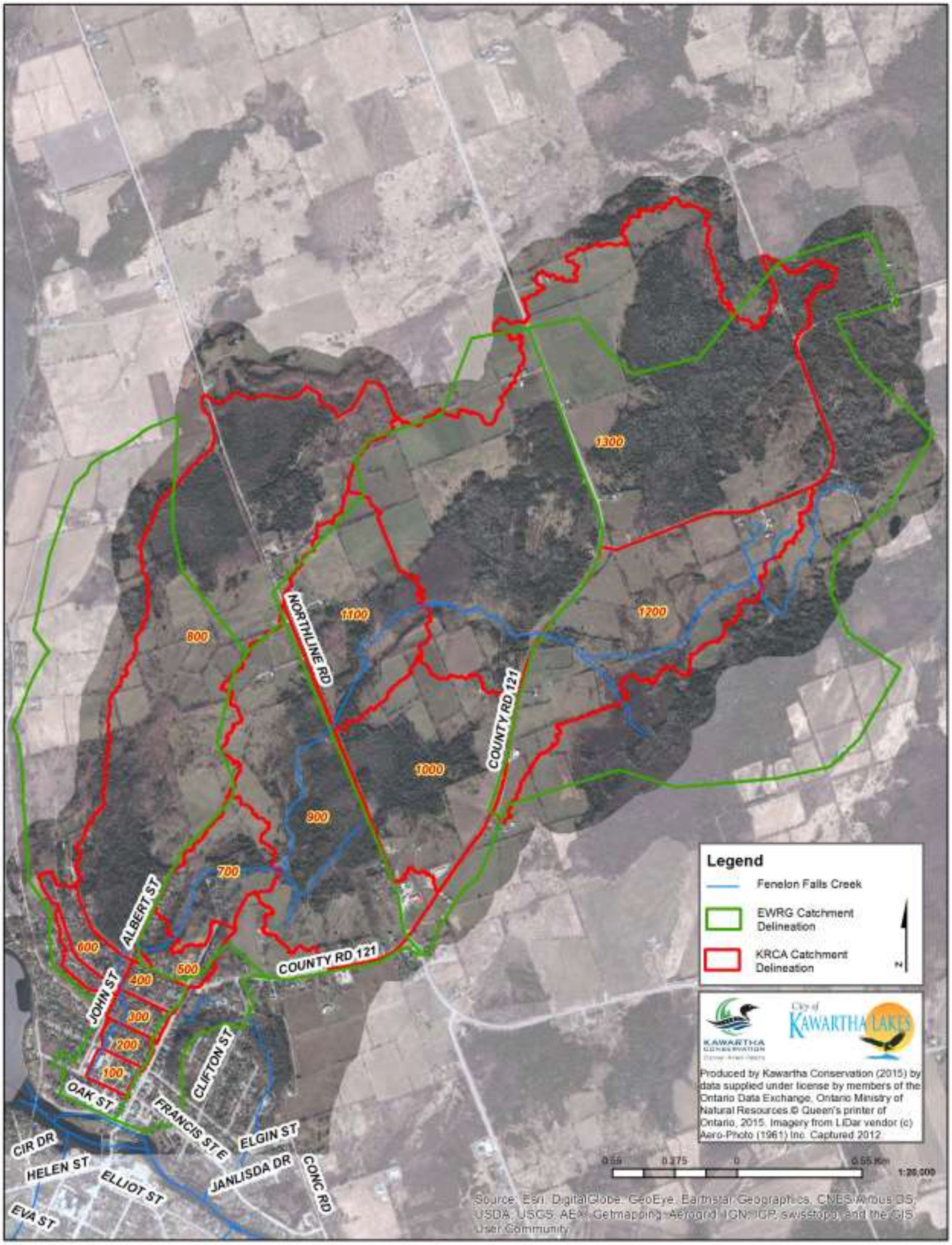


Figure 5.2: EWRG vs. KRCA Subcatchment Differences (west)

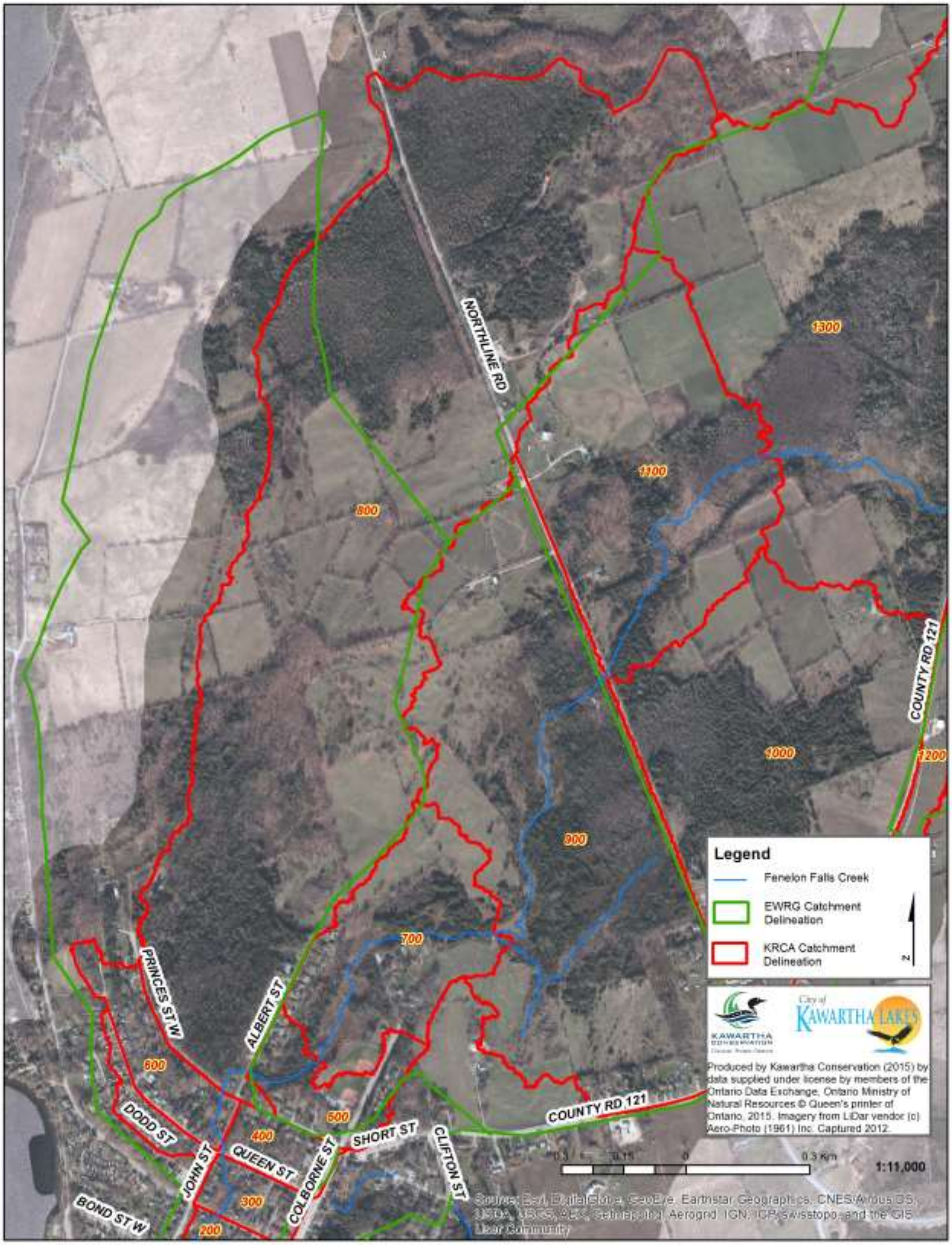
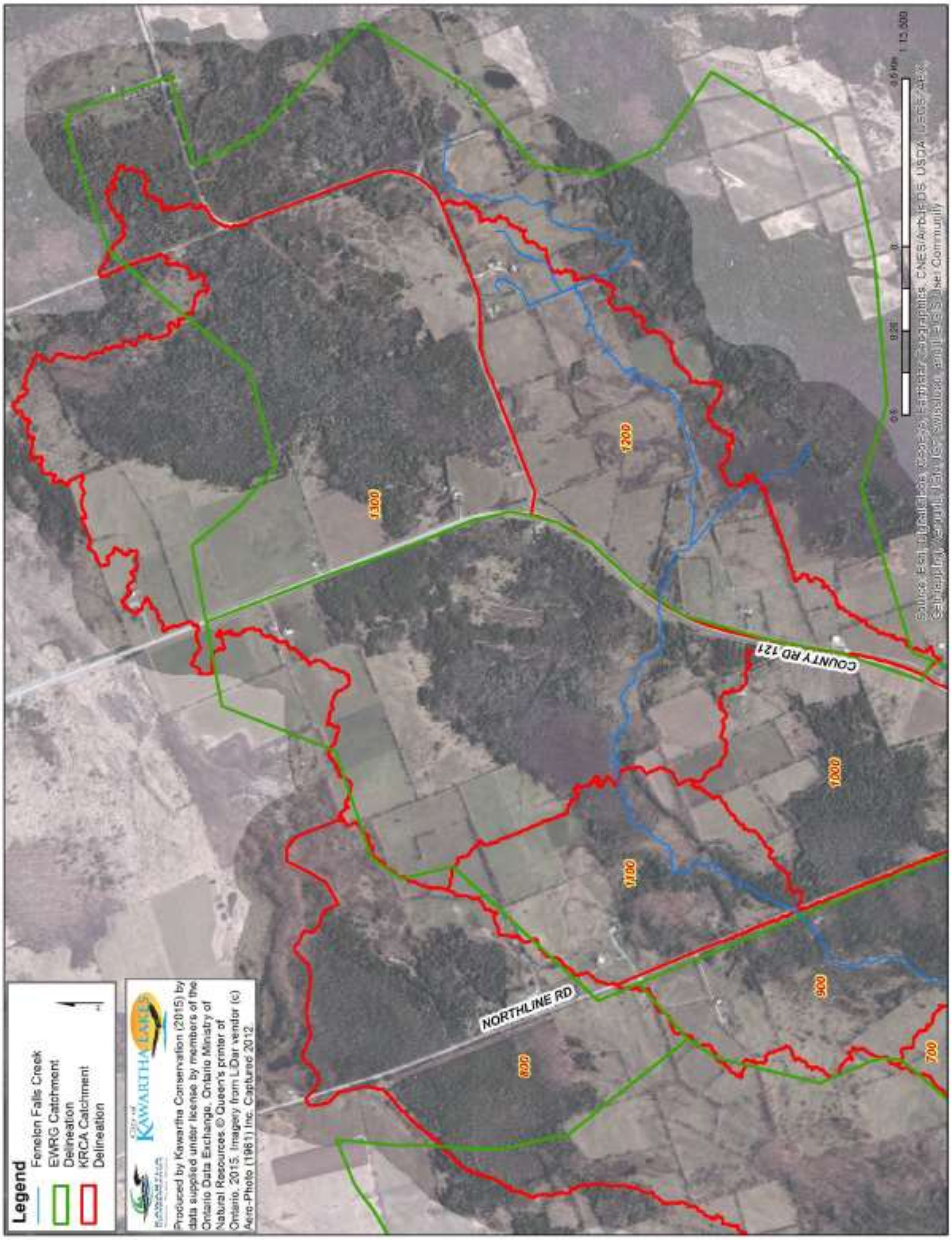


Figure 5.3: EWRG vs. KRCA Subcatchment Differences (east)



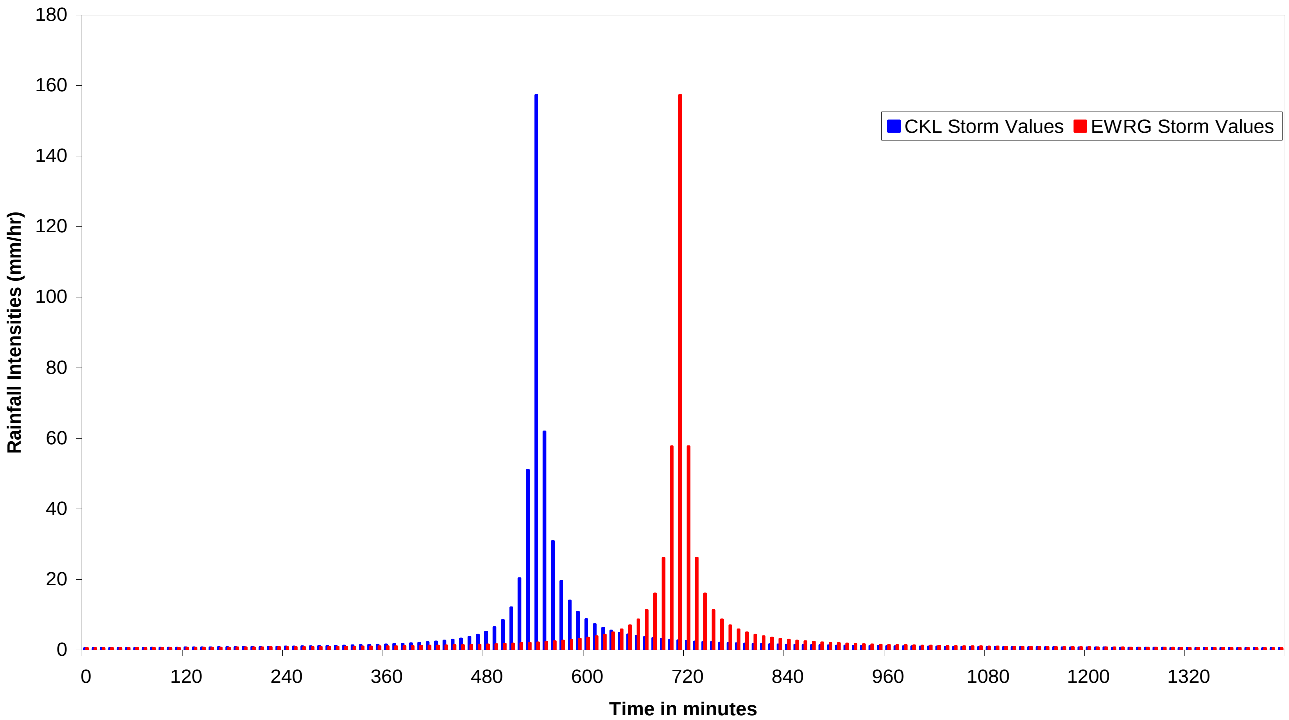
- The eastern limit of EWRG's subcatchment FF2 is County Rd 121, but Kawartha Conservation staff has determined that the land east of County Rd 121 is hydrologically connected via three separate culverts under the road. As a result, Kawartha Conservation's subcatchment 1100 is larger than EWRG's subcatchment FF2.

- For the land east of County Rd 121, Kawartha Conservation staff has set the northern limit further north than EWRG's, but the east and south boundaries do not extend as far as EWRG's. These changes are due to the refinement and quality of elevation data provided by LiDAR.

100-year Storm input

The EWRG model used only the Chicago storm for modeling the 2-100 year return periods, whereas the KRCA model analyzed AES, SCS, as well as Chicago storms. Although the a, b, c values used by EWRG are slightly different than what was used by KRCA modeling team, the calculated intensities were the same for the 100-year storm. The main difference is that EWRG used a ratio 0.5 for the time to peak rainfall intensity to total storm duration (r) whereas the KRCA modeling team used a value of 0.38 in accordance with MTO guidelines. **Figure 5.4** below compares the two Chicago storm hyetographs.

Figure 5.4: Comparing EWRG and KRCA 100-year Chicago Storm Hyetographs



Time to Peak (T_p)

As discussed in section 3.10, the time to peak (T_p) is a key factor in determining subcatchment peak flow. The lower the T_p, the higher is the peak flow. As seen in **Table 5.2** below, EWRG had significantly different T_p values than what was calculated by the KRCA modeling team. The chief reason for this is the more detailed LiDAR data to which KRCA had access. Overland flow paths and their elevation points are more refined than what was available to EWRG.

The hydrology peer reviewer provided a solid explanation for this difference. At the time of the EWRG study, an overland flow distance was measured as a straight line between the furthest upstream point in a subcatchment and the lowest discharge point on the paper contour maps. For this current study, ArcHydro was used to route flow lengths based on the underlying DEM. Flow paths meander and do not follow a straight line. The overland flow lengths are therefore longer in the current study, which results in longer times to peak in the rural subcatchments.

Table 5.2: Comparing EWRG vs KRCA Time to Peak

Subcatchment ID	T_p (hour)		
	EWRG	KRCA	% difference
FF1	1.25	1.20	105%
FF2	0.77	4.40	18%
FF3	0.66	1.40	47%
FF4	1.07	2.00	54%
FF5	0.28	N/A	N/A

CN Values

VH Suite 3 recommends that CN numbers be converted to CN* to better represent southern Ontario values. In the original EWRG study, it does not appear that the CN values were modified. **Table 5.3** below highlights the differences in CN values used in the models.

Table 5.3: Comparing EWRG CN vs KRCA CN* values

Subcatchment ID	CN Values	
	EWRG (CN)	KRCA (CN*)
1300	64	57
1200	64	73
1100	64	64
1000	64	67
900	64	64
800	64	60
700	64	63
600	71	74
500	71	73
400	71	80
300	71	78
200	71	85
100	71	67

Channel Routing

Subcatchment flows are attenuated as they are routed along a channel. The longer and flatter the channel, the more the flow is attenuated. As seen in **Table 5.4** below, the differences in channel lengths and slopes between the EWRG and KRCA model are highlighted. The chief reason for this is the more detailed LiDAR data to which KRCA had access. Similarly to the explanation provided for the longer subcatchment overland flow lengths, the channel lengths are longer in this study since GIS measurements include fine meandering and are not a straight line. The channels and their elevation points are more refined than what was available to EWRG.

Table 5.4: Comparing EWRG vs KRCA Channels

Channel Routing	Channel Length (m)		Channel Slope (%)	
	EWRG	KRCA	EWRG	KRCA
Within FF2 (Cty Rd 121 to Northline Road)	1270	1538	0.5	0.5
Within FF3 (Northline Rd to Albert St)	1700	2602	0.5	0.5
Within FF5 (Albert St to Francis St)	710	698.5	0.1	1.0

5.2. Comparing Hydrology model output: 1996 EWRG vs. 2015 Kawartha Conservation

For the 2-100 year storm events, EWRG modeled only the 24-hour Chicago storm. Kawartha Conservation staff modeled the 6-, 12-, and 24-hour AES, SCS, and Chicago events. In this study, the 6-hour AES storm produced the highest peak flow at key nodes. Flow comparisons for the 2-100 year events are highlighted in **Table 5.5** below. **Table 5.6** lists the flows at key nodes for the Timmins storm. Summary output is in included in **Appendix G**.

Table 5.5: 100-year Flows at Key Nodes

Node	Flow from critical storm in m^3/s						
		6-hour AES					
	EWRG 24hr Chicago	2yr	5yr	10yr	25yr	50yr	100yr
County Rd 121	4.30	0.48	0.90	1.22	1.66	2.01	2.37
Northline Road	8.50	0.78	1.57	2.18	3.01	3.68	4.36
Albert St W	9.70	0.81	1.73	2.44	3.42	4.21	4.99
Princes St W	12.00	1.23	2.54	3.56	4.97	6.15	7.30
John St	12.00	1.23	2.55	3.57	4.98	6.16	7.31
Queen St	12.00	1.24	2.55	3.59	4.98	6.17	7.32
Louisa St	12.00	1.24	2.56	3.59	4.99	6.17	7.32
Bond St W	12.00	1.25	2.56	3.59	4.99	6.17	7.32
Francis St W	12.00	1.25	2.56	3.59	4.99	6.17	7.32

Table 5.6: Timmins Flows at Key Nodes

Node	Timmins Flows in m^3/s EWRG	Timmins Flows in m^3/s Kawartha Conservation
County Rd 121	12.00	3.76
Northline Road	23.00	10.77
Albert St W	26.00	14.42
Princes St W	31.00	19.26
John St	31.00	19.46
Queen St	31.00	19.66
Louisa St	31.00	19.77
Bond St W	31.00	19.87
Francis St W	31.00	19.96

When comparing subcatchment runoff peaks for the 100-year events, the urban subcatchments responded with the highest peak runoff for the 24-hour Chicago storm while the rural subcatchments peak runoff rates were determined from either the 6-hour AES or SCS storm. However what is of interest to flood plain mapping is the routed flow peak at key nodes and the 6-hour AES storm consistently provided the most conservative flow peak. On average, the KRCA model calculates peak flows that are 58% of the EWRG model's for the 100-year events, and 57% for the Timmins storm.

As previously explained in section 5.1, the T_p values used in the KRCA model are for the most part substantially higher than what was used in the EWRG model. The T_p in the KRCA model ranges from 2 to 6 times longer than what was used in the EWRG model and would account for a large difference in peak flows. In order to determine the impact of T_p on the models, the VH Suite model was modified by altering the T_p values by the percentage values in **Table 5.7** and **Table 5.8** below. In order to obtain a more valid comparison, EWRG's 24-hour Chicago storm hyetograph was used for runoff comparisons. As seen in **Table 5.7** and **Table 5.8**, the modified KRCA model calculates flows close to what was derived by EWRG in the previous flood plain mapping study.

Since the LiDAR data used in this study is more detailed than what was available at the time of the EWRG study, it is felt that the KRCA model more truly reflects actual hydrological conditions in the watershed.

Table 5.7: 100-year Flows at Key Nodes with reduced T_p

Node	Tp Factor	Flow (m^3/s) EWRG 24hr Chi	Flow (m^3/s) KRCA 6hr AES	%	Area EWRG	Area KRCA	%
County Rd 121	105%	4.30	2.65	62%	290	67.3	23%
Northline Road	18%	8.50	6.16	72%	490	338.0	69%
Albert St W	47%	9.70	6.82	70%	590	484.9	82%
Princes St W	54%	12.00	10.71	89%	730	624.1	85%
John St	N/A	12.00	10.80	90%	N/A	630.6	85%
Queen St	N/A	12.00	10.87	91%	N/A	636.6	85%
Louisa St	N/A	12.00	10.92	91%	N/A	640.0	85%
Bond St W	N/A	12.00	10.98	92%	N/A	643.0	85%
Francis St W	N/A	12.00	11.00	92%	759	645.8	85%

Table 5.8: Timmins Flows at Key Nodes with reduced T_p

Node	Tp Factor	Flow (m^3/s)		%	Area		%
		EWRG	KRCA		EWRG	KRCA	
County Rd 121	105%	12.00	3.71	31%	290	67.3	23%
Northline Road	18%	23.00	12.80	56%	490	338.0	69%
Albert St W	47%	26.00	16.88	65%	590	484.9	82%
Princes St W	54%	31.00	22.68	73%	730	624.1	85%
John St	N/A	31.00	22.95	74%	N/A	630.6	85%
Queen St	N/A	31.00	23.21	75%	N/A	636.6	85%
Louisa St	N/A	31.00	23.34	75%	N/A	640.0	85%
Bond St W	N/A	31.00	23.48	76%	N/A	643.0	85%
Francis St W	N/A	31.00	23.59	76%	759	645.8	85%

5.3. Analyzing Storm Durations

As noted in section 5.2, the 6-hour storm provided the highest peak flow for the 100-year event. The peer reviewer pointed out that the flow peaks became increased as the storm durations decreased. One of the peer recommendations was to run shorter duration storms to find the upper bound of the peak flows, starting with a four-hour duration.

AES gauges do not tally the 4-hour storm volumes. Using the revised a, b, c parameters, a 4-hour Chicago storm was run to calculate the rainfall volume: 76.1mm. SCS and AES storm mass files were input into VH Suite for this 4-hour volume.

As is seen in Table 5.9 below, the 4-hour peak flows are less than the 6-hour peak flows at key nodes. Because of this, 6-hour flow peaks will be used for the 2-100 year events as the critical event.

Table 5.9: Comparing 4-hour and 6-hour Flow Peaks at Key Nodes

Node	100-year Storm Q_p (m^3/s)					
	4-hr SCS	4-hr Chi	4-hr AES	6-hr SCS	6-hr Chi	6-hr AES
County Rd 121	2.08	2.02	1.66	2.48	2.18	2.37
Northline Road	3.36	3.31	2.51	4.22	3.45	4.36
Albert St	3.65	3.64	2.71	4.77	3.80	4.99
Princes St W	5.39	5.28	4.07	6.83	5.41	7.30
John St	5.39	5.29	4.07	6.87	5.45	7.31
Queen St	5.39	5.31	4.08	6.92	5.48	7.32
Louisa St	5.39	5.31	4.08	6.95	5.50	7.32
Bond St W	5.39	5.32	4.09	6.97	5.52	7.32
Francis St W	5.39	5.33	4.09	6.99	5.54	7.32

6. Flow Input to the Hydraulic Model

For the HEC RAS model
The results of the new VH SUITE 3 hydrological model for the Fenelon Falls North watercourse are reasonable and the best estimate of flow and therefore should be input to a hydraulic model to establish new Regulatory floodlines for the watershed. **Table 6.1** shows the representative peak flows to be input to the HEC-RAS model; the 2-100 year flows are derived from the 6-hour AES storm.

Table 6.1: Input Flows to HEC-RAS

Node	Flows in m^3/s						
	2yr	**5yr**	**10yr**	**25yr**	**50yr**	**100-year**	**Timmins**
County Rd 121	0.48	0.90	1.22	1.66	2.01	2.37	3.76
Northline Road	0.78	1.57	2.18	3.01	3.68	4.36	10.77
Albert St W	0.81	1.73	2.44	3.42	4.21	4.99	14.42
Princes St W	1.23	2.54	3.56	4.97	6.15	7.30	19.26
John St	1.23	2.55	3.57	4.98	6.16	7.31	19.46
Queen St	1.24	2.55	3.59	4.98	6.17	7.32	19.66
Louisa St	1.24	2.56	3.59	4.99	6.17	7.32	19.77
Bond St W	1.25	2.56	3.59	4.99	6.17	7.32	19.87
Francis St W	1.25	2.56	3.59	4.99	6.17	7.32	19.96

7. Hydraulic Model Input Parameters

7.1. Cross Sections

The cross-section geometric data was extracted from the LiDAR DEM using HEC-GeoRAS. This ensures geo-referencing of the geometry data when imported into HEC-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. The surveyed data was fused into the cross-sections generated by HEC-GeoRAS. Data sources generated by different entities were placed into the same projection and datum for consistency in processing. Stream crossings have been identified and positioned through the use of the LiDAR orthoimagery, field reconnaissance, and information in previous reports. Full photographic records of all stream cross sections are found in **Appendix I**.

As per HEC-RAS requirements, all cross-sections are oriented looking downstream. The initial cross-section is at the upstream end of the Francis Street culvert; cross-section nomenclature reflects the distance in meters relative to the initial cross-section.

Left overbank, main channel, and right overbank downstream lengths were measured from the GIS. As per HEC-RAS recommendations, the overbank distances are measured from each overbank centroid.

7.2. Culvert and Road Crossings

Cross-sections are cut at culvert crossings and other restricting structures to accurately represent channel flow. All culvert crossings are represented by two upstream and two downstream bounding cross sections. Representative deck elevations were extracted from the DEM. All culverts were field surveyed to ensure accuracy. Invert elevations, height/width dimensions, length, and channel bottom were surveyed with total station or GPS. Table 7.1 provides key details; other relevant data and photographs are found in **Appendix J**.

Table 7.1: HEC-RAS Structure Data

Street	River Sta.	Material	Bottom	Shape	Invert Elevation (m)		Length (m)	Size (mm)	
					U/S	D/S		Span	Rise
Francis St	0	Concrete	Closed	Rect.	258.37	252.243	N/A	1.83	1.22
Bond St	137	Concrete	Open	Rect.	258.37	258.18	17.1	2.43	1.01
Louisa St	285	Concrete	Open	Arch	260.37	260.05	12.77	2.08	1.59
Queen St	447	Concrete	Open	Arch	262.29	262.23	9.73	2.41	1.34
John St	478	CSP	Closed	Circular	262.29	262.22	21.43	1.83	1.83
Princes St	618	CSP	Closed	Arch	263.36	263.18	13.48	1.83	1.12
Albert St	723	CSP	Closed	Circular	264.34	264.26	13.32	1.83	1.83
Neal Farm	1613	CSP	Closed	Circular	268.59	268.55	4.87	0.9	0.9
Neal Farm	1613	CSP	Closed	Circular	269.11	269.17	6.14	0.6	0.6

7.3. Expansion/Contraction Coefficients

The model uses the HEC-RAS recommendations of 0.1 and 0.3 for contraction and expansion coefficients at all normal cross sections. At culvert crossings, the values were increased to 0.6 and 0.8, respectively. No bridges were coded in the model.

7.4. Manning's n Values

Manning's n values for channel, left and right overbanks were based on recommended values in Table 3-1 of the *HEC-RAS River Analysis System Technical Manual*, included in **Appendix K.** The main channel n values range from .025 to .050 and the overbank n values range from .025 to .200. These values were chosen based on air photo and survey notes/photos. The main channel and overbank lengths were determined by performing measurements in GIS.

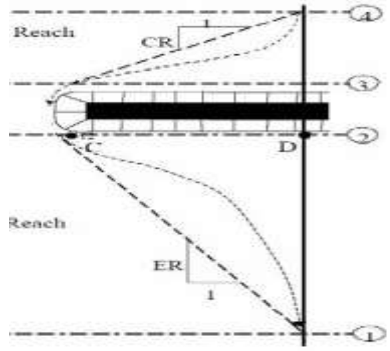
7.5. Ineffective Flow Elevations

Ineffective flow areas were introduced at all culvert crossings. The upstream bounding cross-section ineffective flow elevations are set to the top deck elevations at locations immediately to the left and right of the culvert opening. For the downstream bounding cross-section, the ineffective flow elevations are set at a point midway between the deck and the culvert obvert elevation.

7.6. Building Obstructions

Where buildings are located within or between the cross-sections, the cross-section was modified by introducing obstructions to flow. The effect of a building can be felt upstream and downstream of a cross-section. A 1:1 contraction effect was used for a cross-section upstream of a building; whereby the actual building width is reduced at a 1:1 ratio from each end of the building face. For instance, if a cross-section is 5m upstream of a 30m-wide building, the obstruction representing the building in the cross-section is 20m wide. A 4:1 expansion effect was used for a cross-section downstream of a building. For instance, if a cross-section is 8m downstream of a 30m-wide building, the obstruction representing the building in the cross-section is 26m wide. A representation of the expansion/contraction effects of a building location is shown in **Figure 7.1** below. Detailed calculations are found in **Appendix L**.

Figure 7.1: Building expansion/contraction effects



7.7. Boundary Conditions

Mixed flow analyses (including both sub- and supercritical flow regimes) were run for all scenarios. The upstream boundary condition is set to critical depth.

The creek flows into a buried culvert at Francis St and outlets at the river to the southeast, as shown in **Figure 7.3**. No detailed culvert survey was carried out; its location is approximated from a figure in the EWRG report. City of Kawartha Lakes engineering staff confirms that no engineering drawings exist for the culvert. The only source of reliable information is the 2005 TSH report which shows that the culvert consists of different pipe sizes, shapes, and materials cobbled together into a continuous conduit, as seen in **Table 7.1** below. The limiting section is a 1.83m x 1.22m rectangular concrete pipe. An alternative HEC-RAS model, called *FFNculvert*, was created containing only the Francis St culvert and several cross-sections above and below the culvert ends. The culvert is represented as a 315m long, 1.83m x 1.22m rectangular concrete pipe in the *FFNculvert* model.

Table 7.2: Equivalent Pipe Area

Location	Stationing		Dimensions		Shape	Area (m^2)
	Upstream	Downstream	Width(m)	Height(m)		
Francis St	0	151.7	1.83	1.22	rectangular	2.23
	151.7	182.4	1.80	1.50	rectangular	2.70
	182.4	216.8	6.20	1.80	odd shape	> 2.5
	258	314.9	1.80	1.80	round CSP	2.54

The deck elevation for Francis St was cut from the DEM to represent weir flow over the roadway. Various flows were input to get a rating curve for the culvert: 27 different flow peaks were input to capture the culvert's hydraulic capabilities. **Figure 7.2** shows the rating curve as calculated by HEC-RAS. This rating curve is the downstream boundary condition for the Fenelon Falls North creek main model.

Figure 7.2: Francis St Culvert Rating Curve

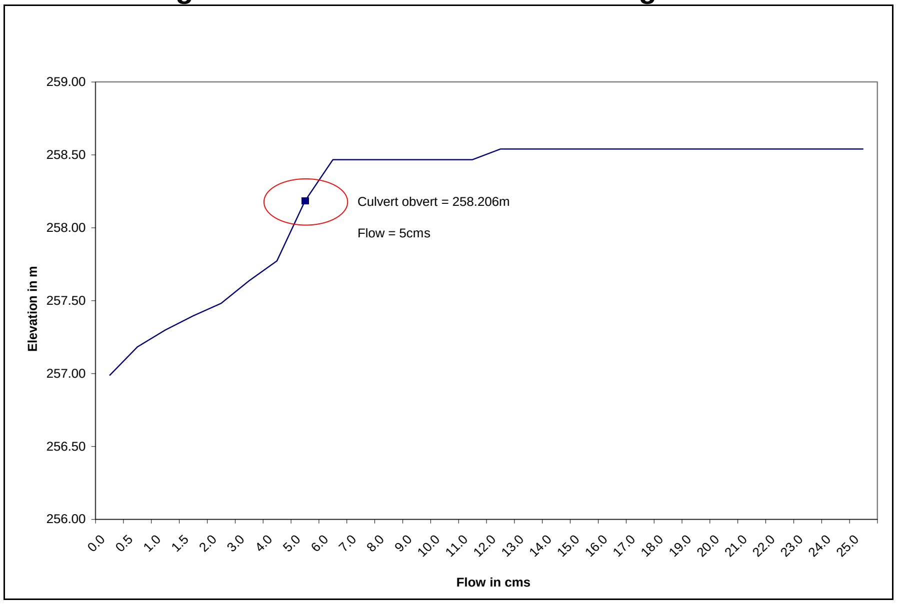


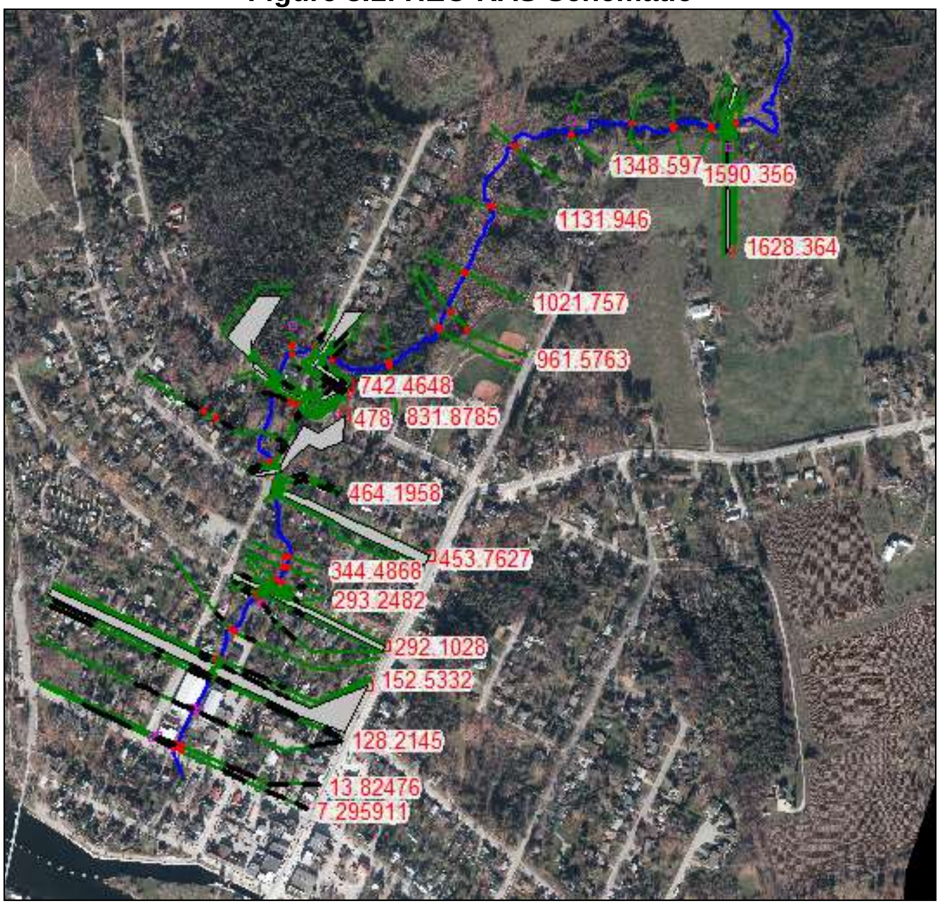
Figure 7.3: Francis St Culvert Location

8. Hydraulic Model

8.1. Schematic

The information gathered in the preceding section was used to build a HEC-RAS model of the watercourse. The geometry of the model is shown schematically in **Figure 8.1**.

Figure 8.1: HEC-RAS Schematic



8.2. Sensitivity Analyses

The HEC RAS model was tested for sensitivity to the Manning's n and starting water surface elevation. **Appendix O** has detailed information on these analyses.

Increasing Manning's n by 20%

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 0.1.
By increasing the Manning's numbers by 20%, the flow is being subjected to a watershed with higher friction forces acting upon it. It was found that overall there is little impact to the calculated water surface elevations. Only 42% of the cross sections experienced a rise in water surface elevations. The average change was only 2cm; the maximum increase in elevation was 9cm.

Decreasing Manning's n by 20%
By decreasing the Manning's numbers by 20%, the flow is being subjected to a watershed with lower friction forces acting upon it. It was found that overall there is little impact to the calculated water surface elevations. Only 38% of the cross-section experienced a drop in water surface elevations. The average change was 2cm.

Downstream Boundary Condition
As previously explained, the creek discharges into a buried culvert under Francis St. There are two main downstream boundary conditions that are possible: the Francis St culvert is operating within its rating curve limits, or the culvert is completely blocked. The base model uses the rating curve of the Francis St culvert. As an alternative, the downstream boundary was changed to a starting water surface elevation equal to the top of Francis St roadway (at 258.5m, this simulates the culvert being blocked).

Calculated water surface elevations were unchanged for the majority of the model: only the two most downstream cross-sections were impacted. For those bottom two cross-sections, flood elevations are lower. The flood elevation for the initial cross-section is only 4cm lower. For the second cross-section, the model indicated a hydraulic jump was occurring. Because of this, the results derived from the rating curve are felt to be more representative for the area.

8.3. Fenelon Falls Arena

Prior to the study's initiation, the Fenelon Falls arena at the southeast corner of John and Bond Streets had been condemned. During the winter of 2016, the arena was demolished. Although the land is currently vacant, it is most likely that the property will be re-developed.

The Technical Committee directed the modeling team to include the demolished building in the HEC RAS model, and calculate the resulting flood plain extents and elevations as if the arena were still in place.

8.4. Albert/Princes/John Crossings

The creek at this intersection poses some flood challenges. At the Albert St crossing, the creek flows west. Just downstream of Albert St. the channel bends 90° south. South of Princes St., the creek bends another 90° to flow toward the John St culvert. The floodplain rotates through a gentle 135° bend. The topography in this area slopes predominantly from northeast to southwest from the east side of Albert St to the south side of Princes St; south of Princes St the topography slopes southeast. **Figure 8.2** below shows the slopes. Cross sections were cut perpendicular to both the channel and overbanks in order to properly model the flood flows. **Figure 8.3** shows the typical cross-sections (cut perpendicular to the creek channel) and the cross-sections used in the model cut perpendicular to the channel and overbank flow direction.

Figure 8.2: Land Configuration at Albert/Princes/John Intersection

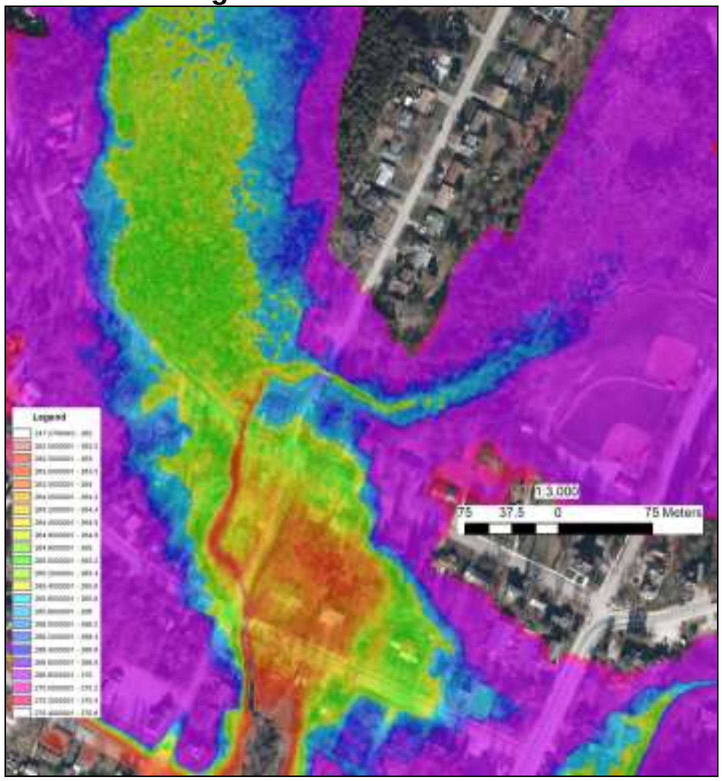
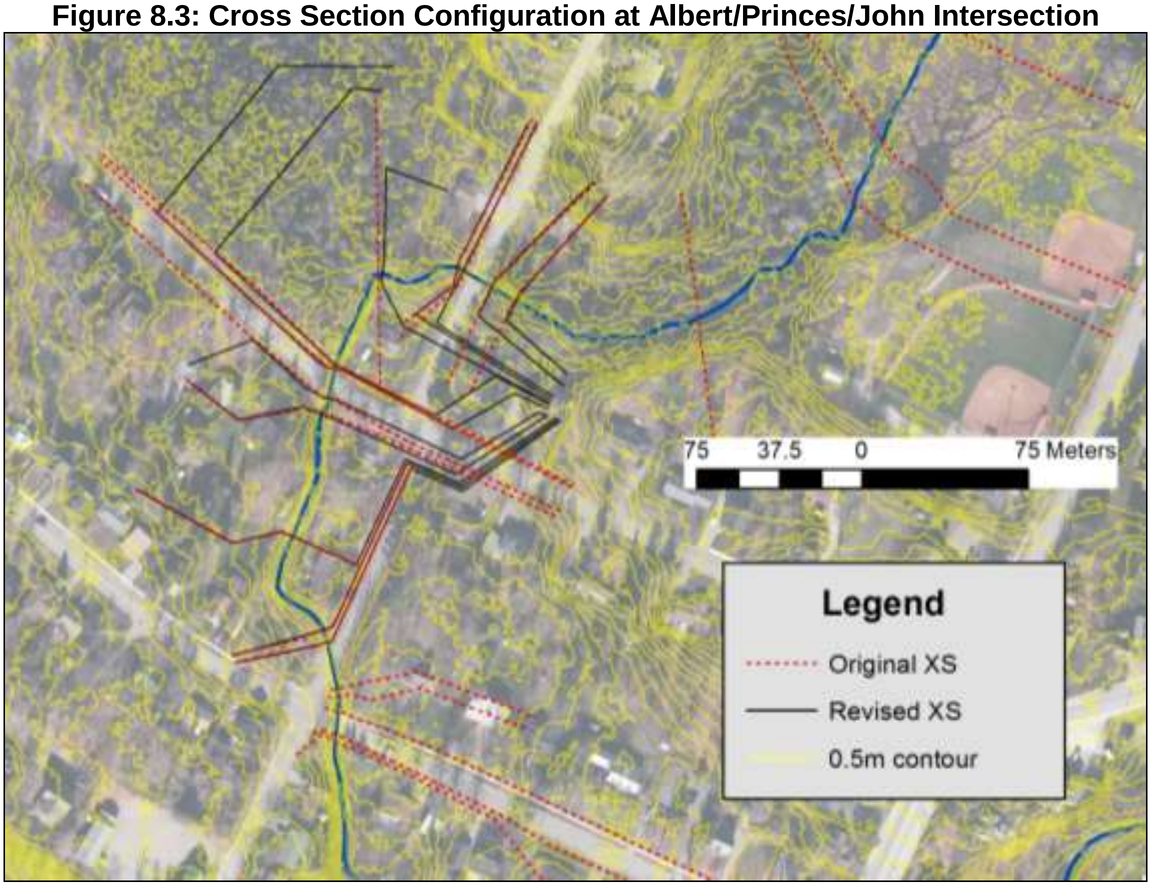


Figure 8.3: Cross Section Configuration at Albert/Princes/John Intersection



9. Hydraulic Model results

9.1. Comparing Model Data Input (Kawartha vs. EWRG)

The Fenelon Falls North watercourse was originally modeled in 1996 by EWRG using paper-based maps. As discussed in previously, the City of Kawartha Lakes recent acquired a LiDAR-based DEM. Due to this greatly improved data set, significant differences exist between the 1996 and 2016 models.

Base DEM

The model established by Kawartha Conservation is geo-referenced from the recent LiDAR acquisition, whereas the EWRG model is not georeferenced. As previously mentioned in section 1.4, EWRG used 1"=200' scale topographical maps with 5' contours within Town limits, and 1:10,000 scale topographical maps with 5m contours for rural lands outside Town limits. EWRG did not supply digital CAD or GIS files for the flood maps; only paper maps were included in their final report.

Cross-sections

There are some locations where elevation differences are noted between the EWRG and KRCA models, due to the lack of a DEM available to EWRG. **Figure 9.1** and **Figure 9.2** are examples of the types of differences seen for many of the cross sections.

Figure 9.1: Comparing Cross-section 675

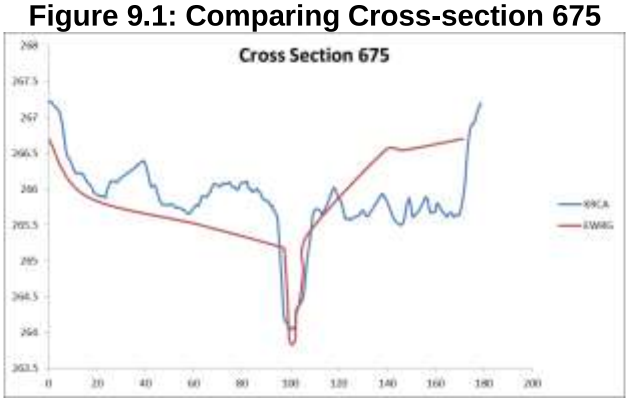
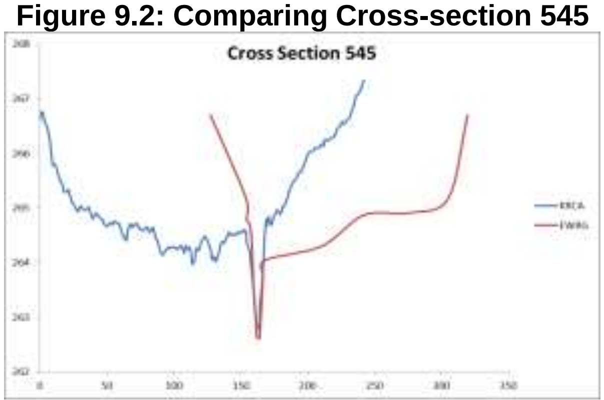


Figure 9.2: Comparing Cross-section 545



Flow Input

The input flows in the Kawartha Conservation HEC-RAS model are different than what was used in the EWRG model. These differences are fully discussed in section 5.2. Peak flows are lower than what was used by EWRG, as shown in **Table 9.1** and **Table 9.2**. More details can be found in **Appendix G**.

Table 9.1: Comparing 100-year Flood Flows

Node	Flow from critical storm in m^3/s	
	EWRG 24hr Chicago	KRCA 6hr AES
County Rd 121	4.30	2.37
Northline Road	8.50	4.36
Albert St W	9.70	4.99
Princes St W	12.00	7.30
John St	12.00	7.31
Queen St	12.00	7.32
Louisa St	12.00	7.32
Bond St W	12.00	7.32
Francis St W	12.00	7.32

Table 9.2: Comparing Regulatory Flood Flows

Node	Timmins Flows in m^3/s	
	EWRG	KRCA
County Rd 121	12.00	3.76
Northline Road	23.00	10.77
Albert St W	26.00	14.42
Princes St W	31.00	19.26
John St	31.00	19.46
Queen St	31.00	19.66
Louisa St	31.00	19.77
Bond St W	31.00	19.87
Francis St W	31.00	19.96

Manning's n

HEC-RAS requires unique values assigned to the overbanks and channel. In over 80% of the cross-sections, the KRCA's main channel n values are the same as EWRG's values. It is in the overbank areas where n values differ significantly; all of the overbanks have values that are different. The EWRG report stated only two values were used to represent overbank roughness: 0.080 for woods and 0.050 for meadows. In reality the EWRG model used 0.055 for meadows, however. The KRCA model used a wider range of n values to represent overbank n, as can be seen in **Table 9.3** below. More information can be found in **Appendix K**.

Table 9.3 Manning n Values

River Station		Left Overbank			Channel	Right Overbank	
		1st	2nd	3rd		1st	2nd
1628.364		0.035	0.100		0.035	0.060	
1615.809		0.025			0.035	0.050	0.100
1613 Neal Farm Roadway							
1607.321		0.025	0.050		0.035	0.100	0.060
1590.356		0.025	0.050		0.035	0.100	0.050
1529.641		0.025			0.035	0.100	
1455.284		0.025	0.050		0.035	0.050	
1348.597		0.025	0.050		0.035	0.100	
1245.553		0.025	0.016	0.050	0.035	0.050	
1131.946		0.100	0.050		0.035	0.050	0.100
1021.757		0.050	0.040		0.035	0.040	0.100
961.5763		0.025			0.035	0.100	
931.2015		0.025			0.035	0.100	
831.8785		0.025	0.100		0.035	0.100	
742.4648		0.100			0.030	0.100	
730.1611		0.025			0.030	0.025	
723 Albert St							
715.7344		0.025			0.030	0.025	
711.8738		0.025			0.030	0.025	
672.5879		0.025			0.050	0.100	
626.7126		0.025			0.030	0.025	
625.0958		0.250			0.030	0.250	
618 Princes St							
610.8494		0.025			0.030	0.025	
603.895		0.060			0.030	0.060	
545.5701		0.060			0.030	0.060	
492.6015		0.025			0.030	0.025	
486.0407		0.025			0.030	0.025	
478 John St							
464.1958		0.025			0.030	0.025	
460.7104		0.025			0.030	0.025	
453.7627		0.025			0.030	0.025	
447 Queen St							
443.1591		0.025			0.030	0.025	
440.1843		0.040			0.030	0.040	
344.4868		0.060			0.030	0.060	
328.7629		0.060			0.030	0.060	
307.6374		0.025	0.060		0.030	0.060	
293.2482		0.025	0.060		0.030	0.060	
292.1028		0.025			0.030	0.025	
285 Louisa St							
277.0925		0.200	0.025		0.035	0.025	0.200
267.3787		0.200	0.040		0.035	0.040	0.200
203.5257		0.200	0.040		0.035	0.040	0.200

River Station		Left Overbank			Channel	Right Overbank	
		1st	2nd	3rd		1st	2nd
152.5332		0.200	0.025		0.035	0.025	0.200
146.1264		0.200	0.025		0.035	0.025	0.200
137 Bond St							
128.2145		0.025			0.025	0.025	
121.9908		0.025			0.025	0.025	
70.5519		0.025			0.025	0.025	
13.82476		0.025			0.025	0.025	
7.295911		0.025			0.025	0.025	

Boundary Conditions

As mentioned in section 1.4, EWRG created three separate models to simulate the unusual hydraulic conditions of the watercourse: one model (called the downstream model) simulated flooding between Sturgeon Lake and the downstream end of the Francis St culvert; the second model (called the May St model) simulated flooding between the upstream and downstream ends of the Francis St culvert; the third model (called the upstream model) simulated the watercourse upstream of the buried culvert to the study limits. Since the upstream cross-section of the May St model was also the initial downstream cross-section in the upstream model, the starting water surface elevation of the upstream model was the water surface elevation of the upstream cross-section of the May St model.

Only one model was created in this current study, from the upstream face of the Francis St culvert to the upper limits of the study area. As discussed in section 7.7, the Francis St culvert rating curve was used as the downstream boundary condition.

9.2. Comparing Hydraulic Model Output (Kawartha vs. EWRG)

Table 9.4 below showcases the differences between the EWRG and Kawartha Conservation flood elevations as calculated by HEC-RAS. More detailed information can be found in **Appendix N**.

Although EWRG did not provide digital floodlines, their paper-based flood lines were digitized onto the Fenelon Falls North model space by GIS staff for a visual comparison. **Figure 9.3** compares the two floodlines.

Figure 9.4 shows the profile of the creek and its Regulatory floodline.

In general, the Kawartha Conservation model calculated lower flood elevation values than the EWRG model.

Table 9.4: Comparing Regulatory Flood Elevations

Location	Flood Elevation (m)	
	EWRG	Kawartha Conservation
Neal Farm Roadway	*N/A*	270.19
At north end of Colborne St	268.33	268.06
At northern ball diamond	267.51	267.07
Albert St	267.25	266.36
Princes St	265.21	264.84
Midway between John and Princes	265.20	264.45
John St	*N/A*	264.42
Queen St	265.16	264.24
Midway between Louisa and Queen	263.40	261.93
Louisa St	*N/A*	262.25
Midway between Bond and Louisa	260.48	260.50
Bond St	*N/A*	259.72
At south face of rink	258.88	259.59
Francis St	258.88	258.55

Figure 9.3: Floodplain comparison

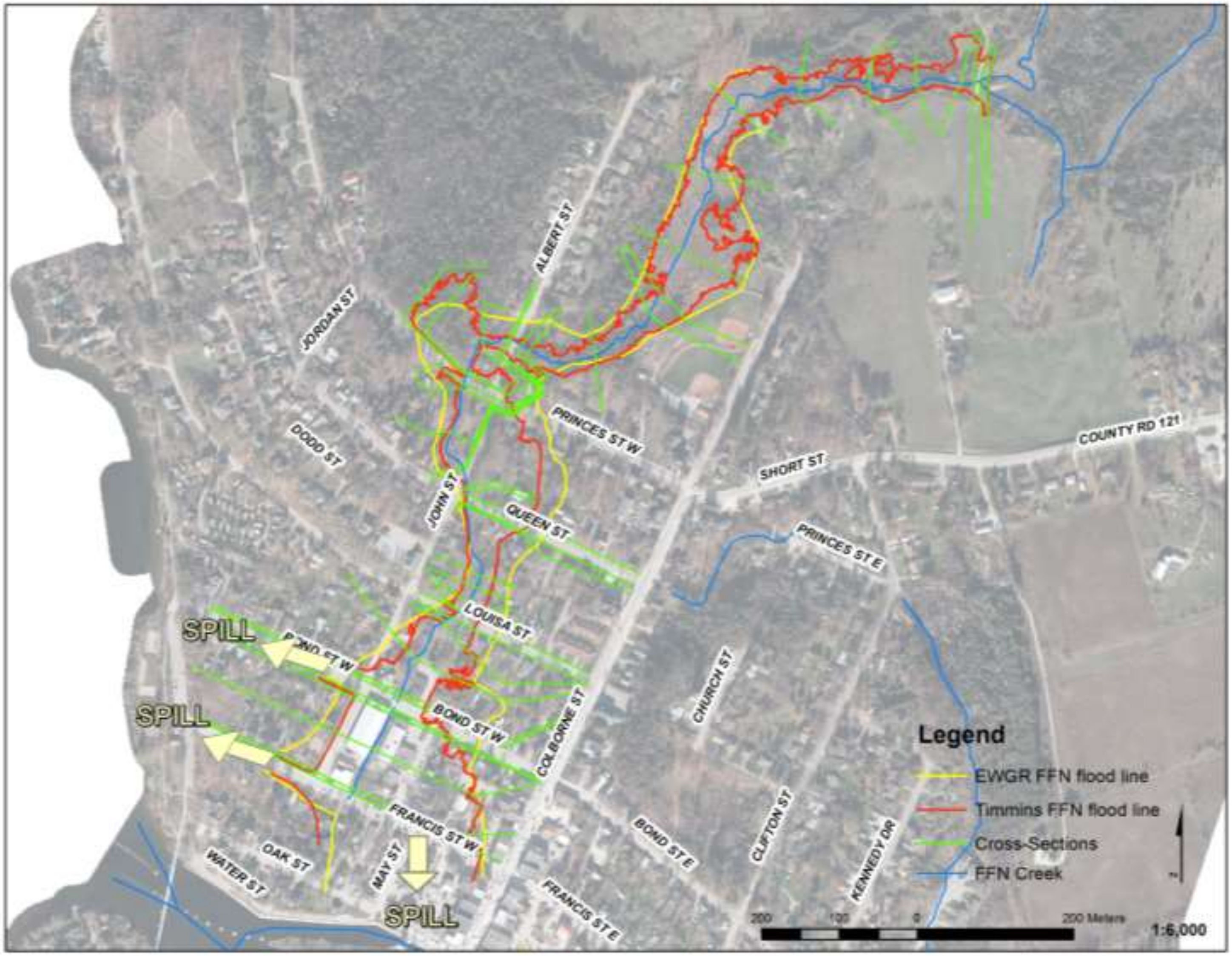
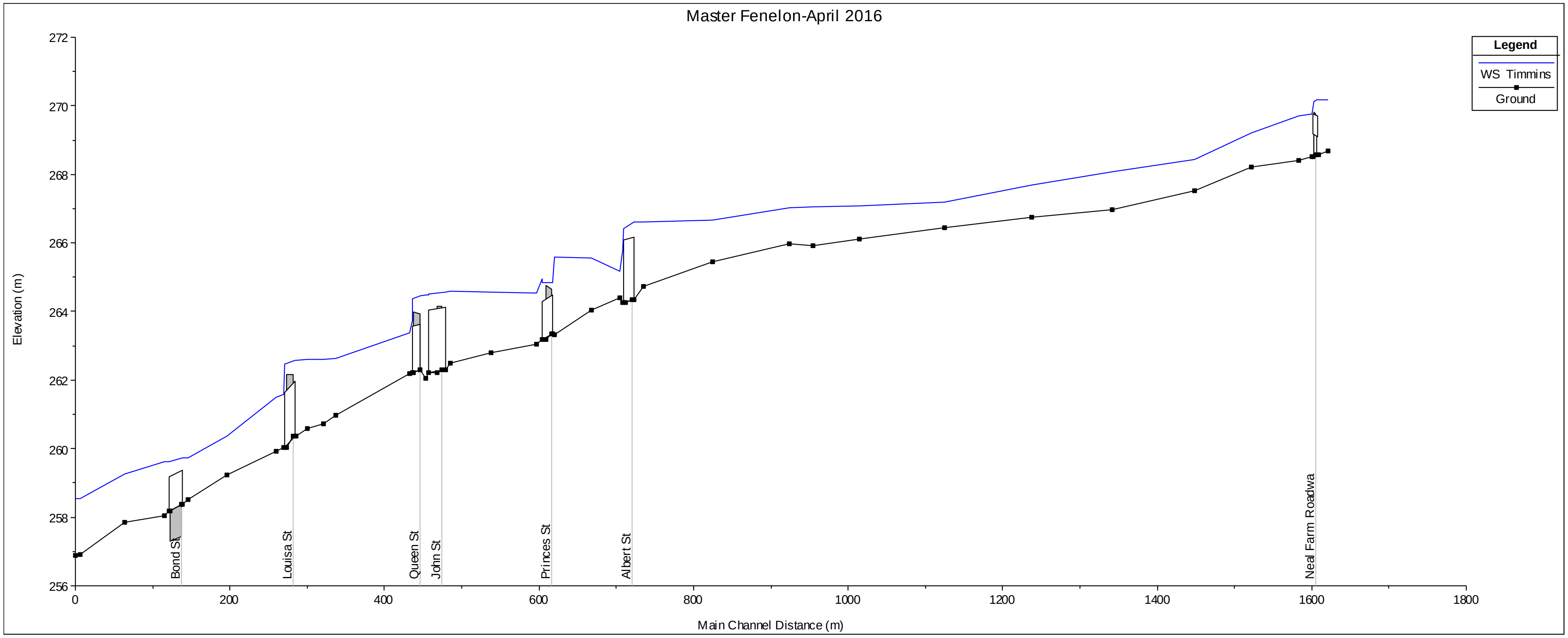


Figure 9.4 Regulatory Profile



9.3. Spills

The EWRG study identified five spills:

- Three at the Bond St culvert:
  - easterly on Bond St to Market St then south on Market to May St
  - westerly on Bond St toward Cameron Lake
  - westerly on Bond St, south on John St, west to Cameron Lake
- Two at the Francis St culvert
  - easterly to May St, south along May St to the Trent canal
  - westerly toward Cameron Lake

EWRG attempted to quantify the flow in the downtown core; flood elevations were calculated and mapped in the report figures. The spills toward Cameron Lake were not quantified.

In the 2016 KRCA model, two spills were identified:

- at the Bond St culvert.
- at the Francis St culvert.

Bond Street Spill

A plan was set up to estimate the spill of flood water at the Bond St culvert. The rate of spill is controlled by the centreline of John St. since the road acts as a lateral weir. Centreline data was extracted from the DEM as represented by the red line in **Figure 9.5** below.

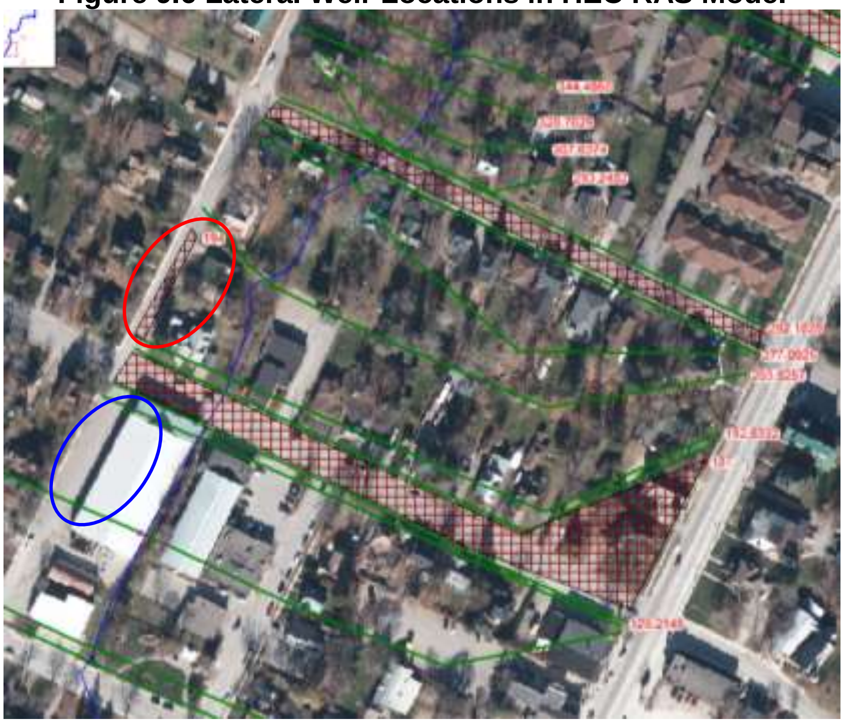
Figure 9.5 John St Spill (Weir Location)

Because HEC RAS cannot have a lateral weir located at the same river stationing as a culvert crossing, two alternate scenarios were created:

- Scenario #1 contains a single lateral weir on the right overbank between cross sections 203 and 146 (as highlighted by the red oval in **Figure 9.6**)
- Scenario #2 contains an additional lateral weir on the John St centreline south of Bond St (as highlighted by the blue oval in **Figure 9.6**). Both lateral weirs are active under this scenario

The model was set up to optimize flow out of the system based on the water surface elevation for both scenarios.

Figure 9.6 Lateral Weir Locations in HEC RAS Model

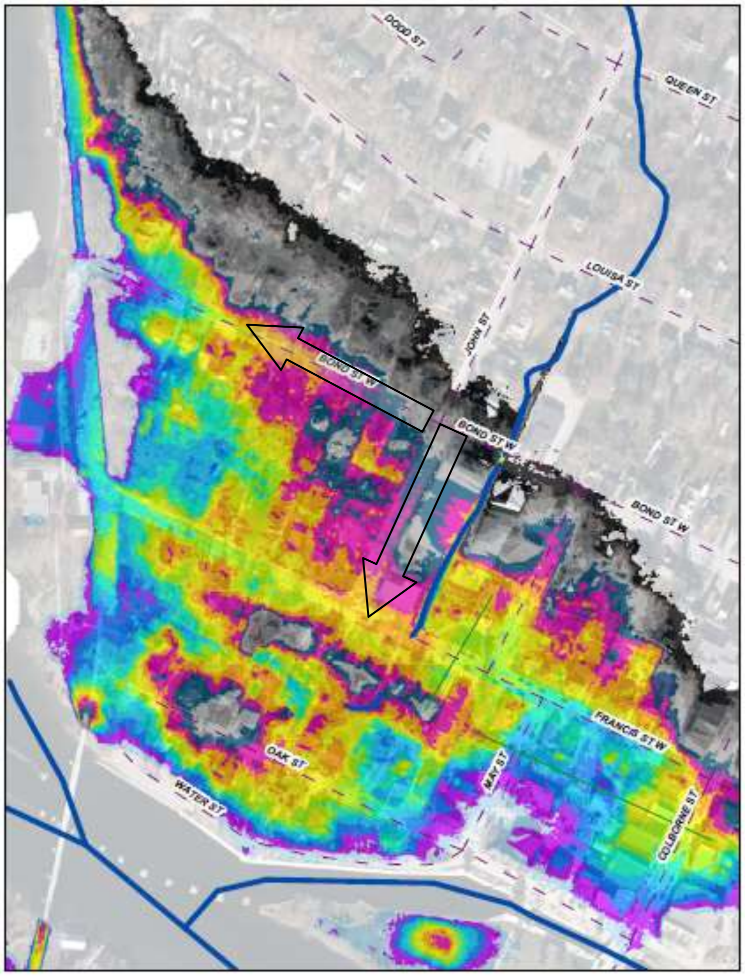


It was calculated that the north (red) lateral weir may divert up to 6.5m^3/s out of the channel toward Cameron Lake during the Regulatory event. The flow lost by the south (blue) lateral weir may be as high as 3.0m^3/s. The net result is that flow at the Frances Street culvert could be reduced to 13.3m^3/s (with one lateral weir) or 10.3m^3/s (with two lateral weirs). This compares to an unreduced Regulatory flow of 20m^3/s if no western spill were occurring.

However, HEC RAS is limited to calculating spill losses in one dimension only. In the two scenarios modeled, all flow is assumed to travel west only. But based on the coloured elevation-based map shown in **Figure 9.7**, water can spill either west on Bond Street, or south along John Street. It is likely that some water will leave the system, but the exact quantity is unknown. To completely understand the spill volumes and directions requires a two-dimensional model which is beyond the scope of this study.

The full flow will therefore be used for flood elevation calculations for all cross-sections downstream of Bond St, for a conservative approach.

Figure 9.7 John St Spill



Francis Street Spill

The Technical Committee directed the model team to calculate flood elevations in the commercial area south of Francis Street. Although there is a westward spill out of the system at the Bond Street culvert, the Committee directed the model team to assume all flow remains in the channel at the Francis Street culvert to give a conservative estimate of flood depths in the commercial core.

Assuming open channel flow using manning's equation, depths of flow in the downtown commercial area were calculated based on two flows: 20cms (assuming the Francis St culvert is blocked and all flow travels overland), and 15 cms (assuming the Francis St culvert is flowing at its rated capacity of 5 cms). Representative cross-sections were cut from the DEM for both Francis and May Streets, and average slopes were calculated along the overland flow paths. Calculated flood elevations are shown **Table 9.5**. Please refer to **Figure 9.8** for the location of the flow path and representative cross-sections. Further details are outlined in **Appendix P**.

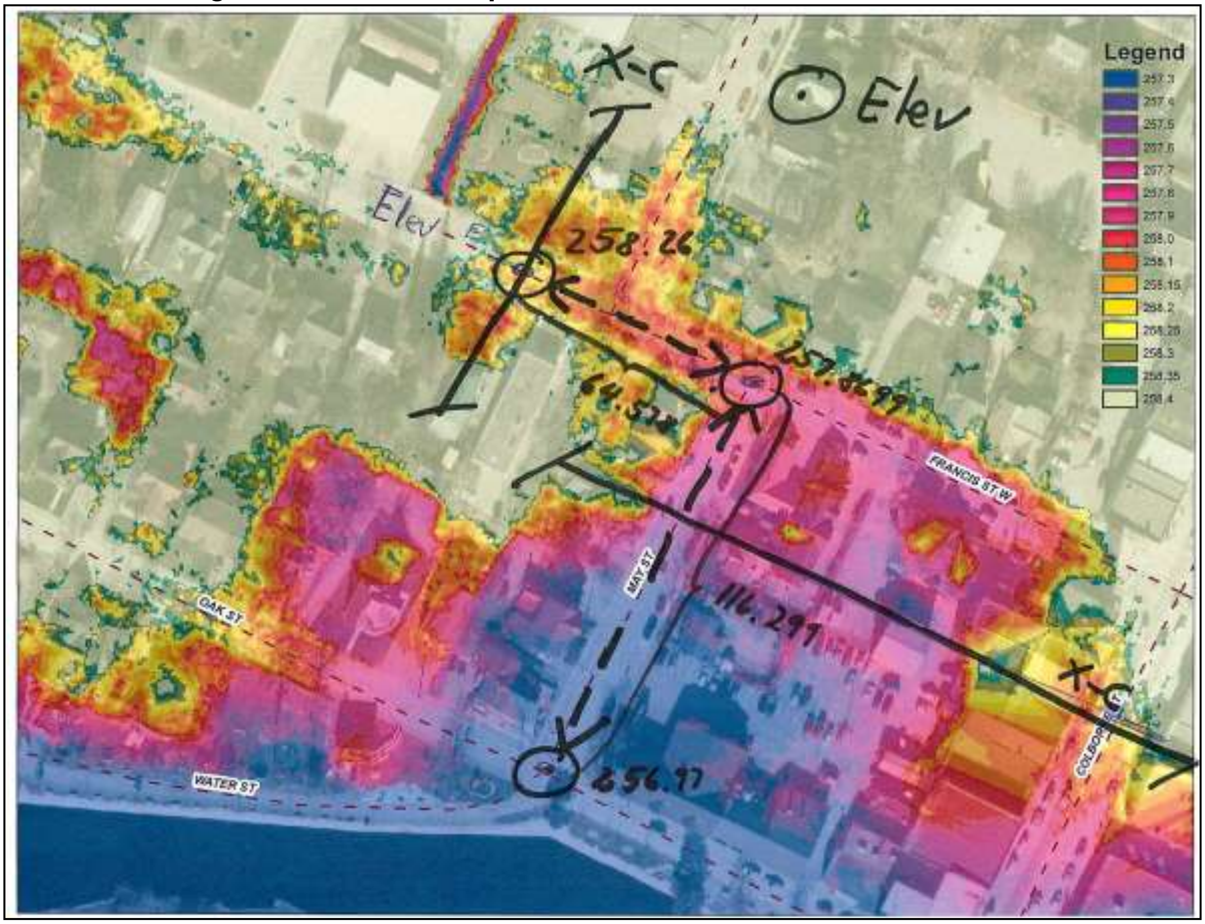
Table 9.5: Assumed Spill Flood Elevations in Commercial Area

Flow (cms)	Flood Elevation (m)	
	Francis St	May St
20	258.36	257.75
15	258.33	257.72

The analysis concluded that the spill designation should remain in place for the commercial area, for the following reasons:

1. The spill designation meets Ministry of Natural Resources and Forestry (MNRF) policy
2. It avoids basing flood elevations on an unknown flow rate at the Francis Street culvert, depleted by the western spill along John Street
3. Special Policy Areas (SPAs), are historical policies based on flood elevations. In recent times, the Province has been reluctant to approve new SPAs.

Figure 9.8: Possible Spill Flow Path in Commercial Area



10. Conclusions and Recommendations

It is recommended that the results of the HEC-RAS model for the Fenelon Falls North watercourse be used for generating the flood maps. Copies of the flood plain maps are appended at the back of this report. The results of the models are reasonable and could be used to establish new Regulatory floodlines for the watershed.

11. Appendices

(Bound in a separate document)

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Appendix O: HEC RAS Sensitivity Analyses
Appendix P: Commercial Area Spill Analysis
Appendix Q: List of Model Files
Appendix R: Peer Review Correspondence
Appendix S: Official & Secondary Plan Maps

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