Flood Plain Mapping Study
Dunsford Creek

Final Technical Report
April 2019

KAWARTHA
CONSERVATION
Discover · Protect · Restore

KAWARTHA LAKES

Greck
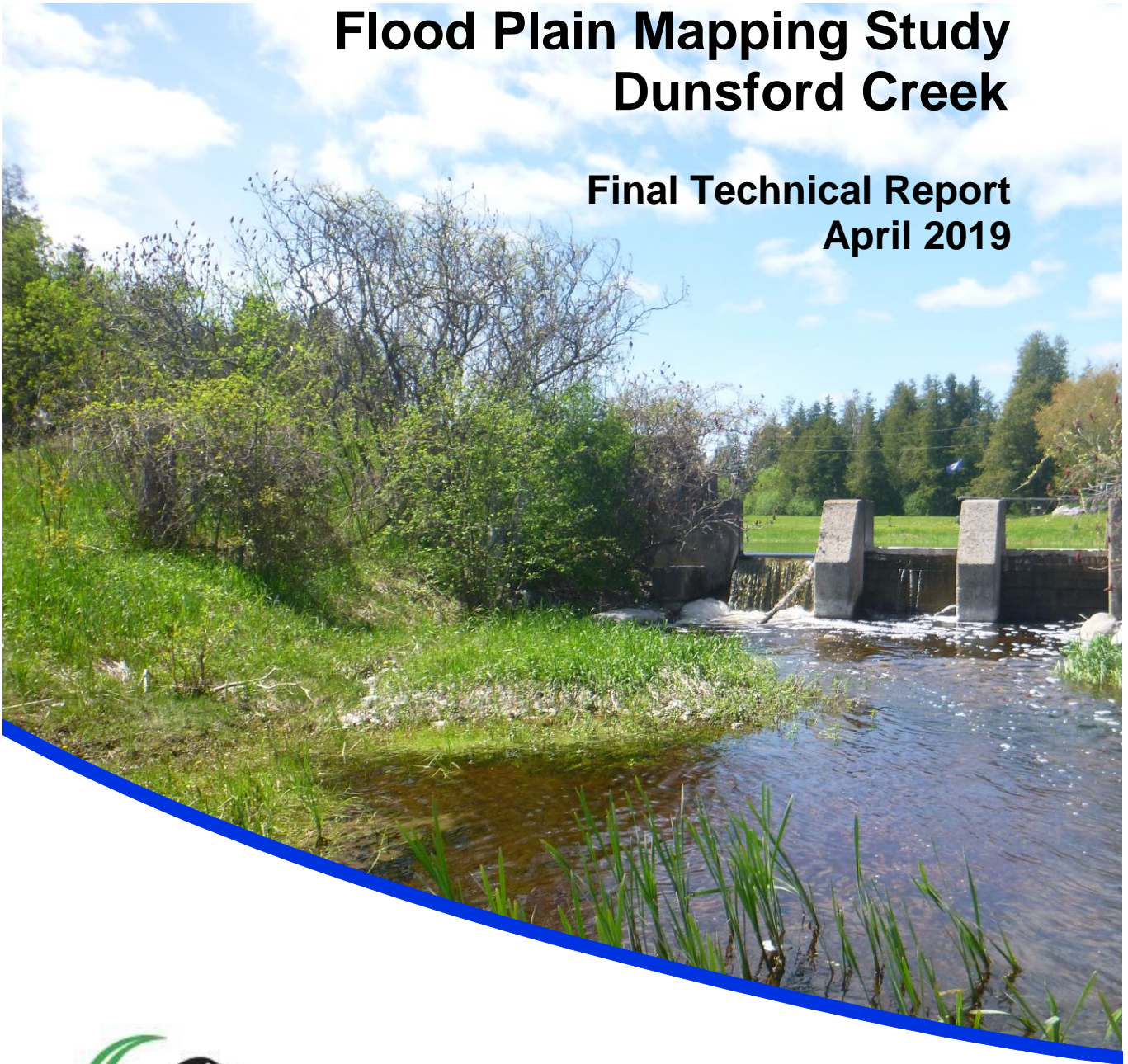
This page intentionally left blank

Executive Summary

The primary goals of this study are to create hydrologic and hydraulic models of the watershed and produce flood plain mapping for Dunsford 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 Dunsford 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 Ganaraska Conservation, the City of Kawartha Lakes, and Kawartha Conservation.

Topics discussed in this study include:

- Collection of LiDAR and Orthophoto data
- Proposed land use
- Delineation of hydrology subcatchments
- Creation of a Visual OTTHYMO hydrology model
- Calculation of subcatchment hydrology model parameters
- Derivation of flow peaks at key nodes along the watercourse
- Creation of a HEC-RAS hydraulic model
- Creation of flood plain maps

Key findings of this study include:

- Peak flows from the Timmins Regional storm event exceed peak flows of the 100 year storm, therefore the Timmins Reginal storm may be used to define the Regulatory flood hazards on Dunsford Creek.
- The flood hazard at the lower reach of Dunsford Creek are defined by the 100 year flood elevations in Emily Creek/ Sturgeon Lake.
- The regulatory floodplain is generally contained within the natural valley lands of Dunsford Creek with the exception of the following locations where flood waters spill due to the natural topography of the land and/or limited the hydraulic capacity of culverts and roadway designs:
    1. Highway 36 (West) crossing
    2. Glen Cedar Road
    3. Highway 36 (East) crossing

Key recommendations of this study:

This study recommends the final floodplain mapping be endorsed and maintained by the Kawartha Conservation Board of Directors and be used to regulate land uses and manage flood hazards within the Dunsford Creek watershed.

This page intentionally left blank

Table of Contents

Contents

List of Tables

List of Figures

1. Introduction

1.1. Objective

The objective of this study is to generate regulatory floodplain mapping for the Dunsford Creek watercourse to protect the public from flooding hazards. This is the fifth flood plain study in a multi-year flood plain 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 representatives from each of the City of Kawartha Lakes, Kawartha Conservation, and Ganaraska Conservation) 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. Five 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

- Floodplain mapping

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 Limited and was found to be satisfactory.

1.3. Watercourse Context and Description

Dunsford Creek (the watercourse) originates in a series of wetland areas south and west of the village of Dunsford. The creek flows in a northeasterly direction through forests and agricultural

fields. Dunsford Creek changes direction just north of the intersection of Kawartha Lakes Road 36 and Cedar Glen Road to flow southeast through Nestle In, RV Resort Village into Emily Creek.

The majority of the watershed consists of forests, wetlands, and farm fields. The hamlet of Dunsford (through which the creek flows) has fairly large residential lots with low levels of imperviousness. Similarly, the RV campsites are not heavily urbanized. The watershed has a size of 3075 hectares (30.75 km^2) and the length of the watercourse is about 4.8 km. The average slope is 0.7%. Please refer to **Figure 1.1** for a plan-view of the overall watershed.

Figure 1.1: Study Area

1.4. Background Information

No previous studies have been carried out for this creek.

1.5. Modeling Approach

Flooding was assessed by deriving peak flows using the SCS standard unsteady flow methods using Visual OTTHYMO Suite 5.0 (VH Suite 5) and conducting standard step steady flow methods using HEC-RAS version 5.0.1.

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 (T_p), and time of concentration (T_c).

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.*

Unless specified otherwise, default parameters/values were used within VH Suite 5 and HEC-RAS.

Neither the hydrologic or hydraulic models were calibrated, as no flow monitoring or flood elevation data is available.

2. Rainfall

When applying flood standards, the Flooding Hazard Limit (or the "Regulatory Floodline") is the greater of the Regional storm, the 100-year, or a documented maximum observed flood event including ice jams. In some instances, it is not unusual to have the 100 year storm exceed the peak flow of the regional storm event, therefore in this study the 100 year and regional storm peak flows were compared.

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 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. Further details regarding the assessment of the two gauging stations is provided in Appendix B.

The IDF data used is presented in **Table 2.1 and Table 2.2.**

Table 2.1 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 2.2 Rainfall Depths from Lindsay AES Station (24 years of data)

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

2.2. Design Storms

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

Total Volume

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

Storm Duration

Watershed drainage areas and the conveyance of flood flows respond differently to different rainfall durations. As such, a variety of rainfall durations (6, 12, and 24 hours) for 2-100 year return periods were tested. For the 100-year event a 4-hour duration was also examined. 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 watersheds 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 Chicago storm distribution is one of the commonly used distributions for designing and analyzing surface runoff in urban areas. The distribution of rainfall is generally in the center of the storm and the peak of storm is quite intense.

The worst case storm (the duration and distribution producing the greatest discharges at key nodes) was selected as the critical event for the watershed. This provides the most conservative protection for the community of Dunsford. Detailed rainfall information is shown in **Appendix B**.

2.3. Regional Storm

The Timmins storm with a total rainfall of 193 mm is the Regional storm event for this part of Ontario. The full storm is defined by Chart 1.04 of the *MTO Drainage Manual*. The Ontario Ministry of Natural Resources (and Forestry) technical manuals provide a rainfall reduction table for

the Timmins storm. Given the size of the Dunsford Creek subcatchment a 97% areal reduction factor was used. Antecedent moisture condition II (AMC II), was applied.

2.4. Snowmelt and Snowmelt/Rainfall Events

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

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 their flood plain mapping studies. This approach was peer-reviewed by Greck and Associates Limited, and their findings concluded the methodology is valid. All parameters and modeling approaches described within this report follow the recommendations presented in **Appendix A** unless otherwise noted. For this study Kawartha Conservation extracted hydrologic parameters from a combination of LiDAR and pixel-autocorrelated elevation data, Arc Hydro watershed boundaries, Official Plan, Secondary Plan, zoning data, and field surveys.

## 3.2. Digital Elevation Model (DEM)

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 majority of the Dunsford Creek watershed, two points per square metre LiDAR data was acquired. However, this area of interest had to be extended for hydrology purposes, necessitating the use of existing pixel-autocorrelation elevation data holdings derived from South Central Ontario Orthophotography Project 2013 (SCOOP 2013) acquisition deliverables. ArcGIS version 10.5.1 and LP360 computer software programs were used to produce bare earth Base DEM using best available raster and point cloud data which preserved the LiDAR acquisition data where possible and effectively added supplementary pixel-autocorrelated data where needed. The Base DEM was produced at a 0.5 m cell resolution within the LiDAR acquisition area, and 2 m cell resolution in the supplementary pixel-autocorrelation data areas.

3.3. Subcatchment Discretization

In order to discretize subcatchments, watershed flow paths were generated using ArcHydro version 10.2 software. Surveyed bridge and/or culvert data was enforced into the Base DEM to create a hydrologically-conditioned DEM (referred to as a Hydro DEM) at a 0.5 m cell resolution. This allows flow connections under road barriers to a downstream channel or subcatchment; flow barriers and other impediments were 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 (please refer to the **Appendix D** for the nodes of the subcatchments).

Figure 3.1 illustrates the subcatchments areas.
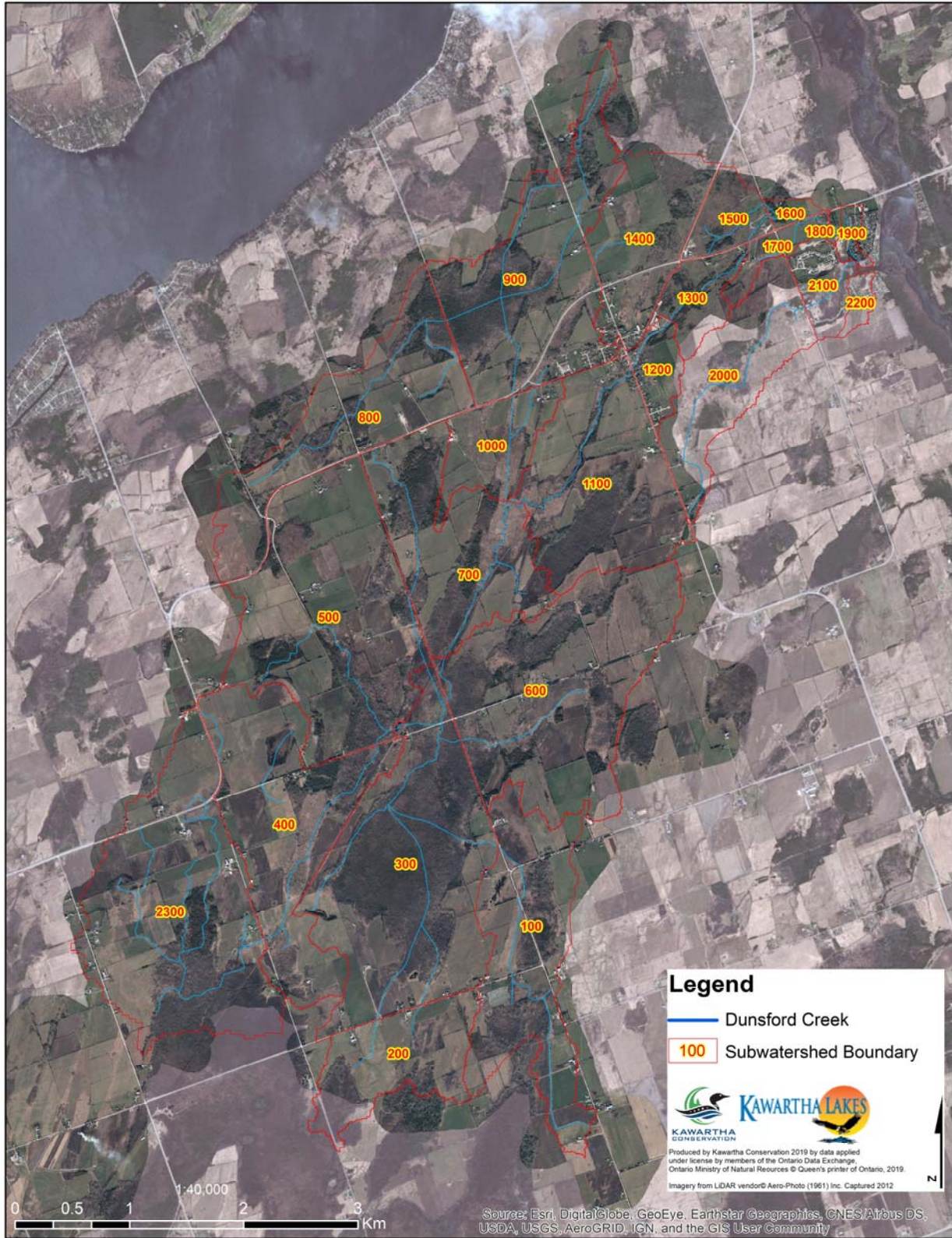
Figure 3.1: Subcatchment Boundaries

3.4. Land Use

Portions of four different former townships within the City of Kawartha Lakes (Verulam, Ops, Fenelon and Emily) are contained within the watershed. In order to extract the land use information for the study area, the Official Plan and Zoning Schedules of all four former townships were obtained.

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 and Secondary/Zoning Plan Schedules 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 G**.

3.5. Time of Concentration

Time of Concentration (T_c) is a key variable for calculating peak flow in rural subcatchments. This is the time it takes for the flow wave to travel from the hydraulically farthest point of a subcatchment to where it joins the creek. 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 of Concentration requires a measurement of the catchment area length and slope.

The boundaries of each subcatchment were initially derived using ArcHydro 10.4. Some adjustments were made by the engineer using best judgment. Overland flow routes within each subcatchment were created using the Longest Flowpath tool in ArcHydro; the longest overland and channel flow paths for each subcatchment were generated in order to calculate the Time of Concentration as requirement for the Hydrology model. A review of the automated flowpath routes resulted in adjustments where appropriate, which were carried out manually with GIS software (**Appendix D** contains a series of figures showing each subcatchment and their respective lengths and **Appendix C** contains tables calculating flow lengths).

The Time to Peak (T_p) is defined by VH SUITE model via the equation: $T_p = (^2/_3) * T_c$

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

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 C**.

3.6. Soils

The Dunsford Creek watershed originates in the drumlinized till plain in Victoria county (Chapman and Putnam 1984, The Physiography of Southern Ontario). This physiographic region provides the primary source for the basic soil types located within the watershed. The overlaying soils are presented in **Figure 3.2.** Soils within area are predominately classified as B with some D and C. Soils classified as B have moderate infiltration characteristics which would result in less runoff than C or D classified soils.

3.7. CN Values

The Soil Conservation Service (SCS) curve number (CN) is used to determine runoff. Users must choose which **a**ntecedent **m**oisture **c**ondition (AMC I, II, or III) is relevant for the model; AMC I 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 were queried to extract land use, drainage area, and hydrologic soils group data. A weighted CN (AMC II) value was calculated, as shown in **Appendix C**.

The VH SUITE program requires that the CN value be transformed to CN* (AMC II). These calculations are included in **Appendix C**. **Figure 3.2** provides soils information while **Figure 3.3** shows the future land use of the watershed based on Secondary Plan data. Spreadsheets with the calculations are provided in **Appendix C**.
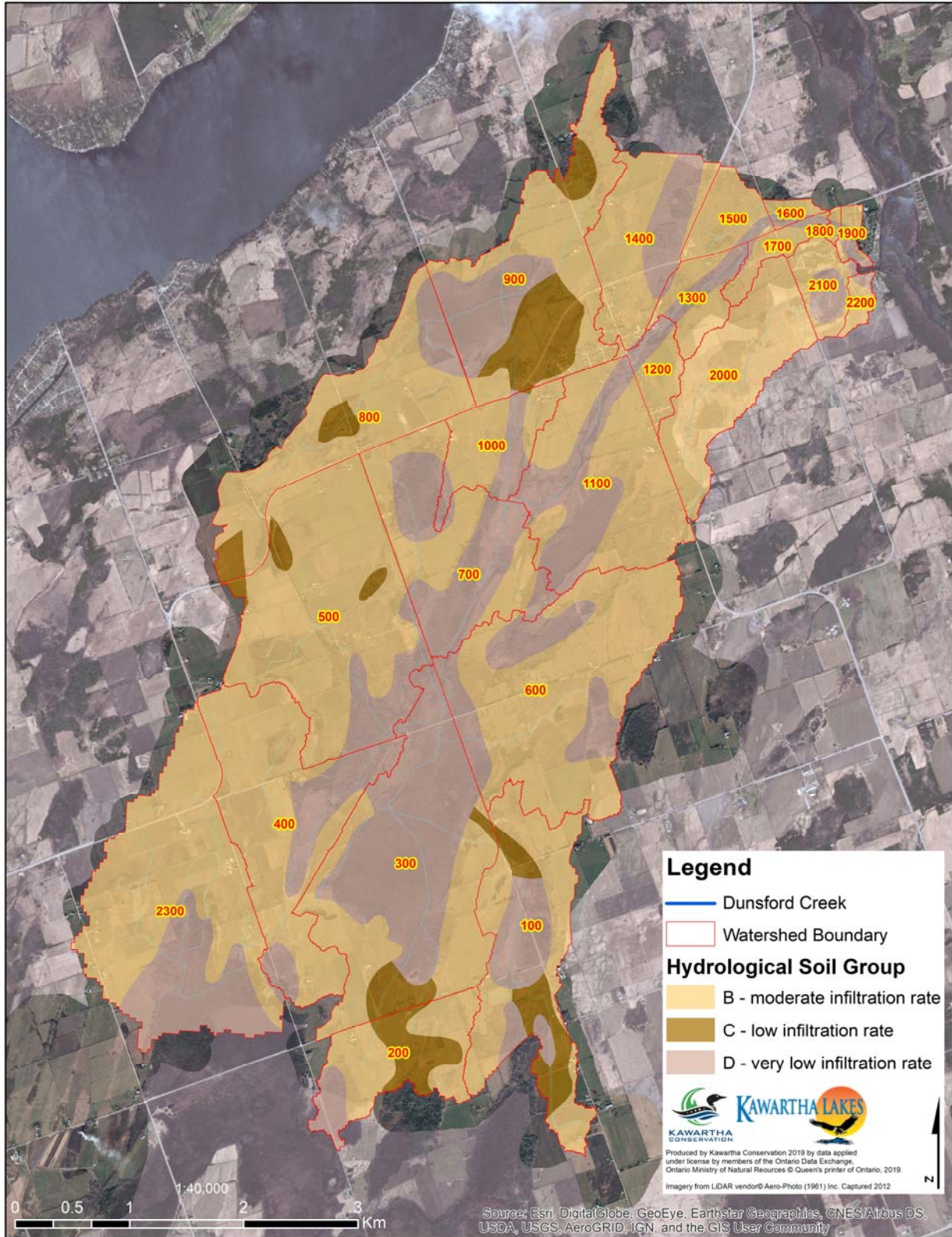
Figure 3.2: Soils
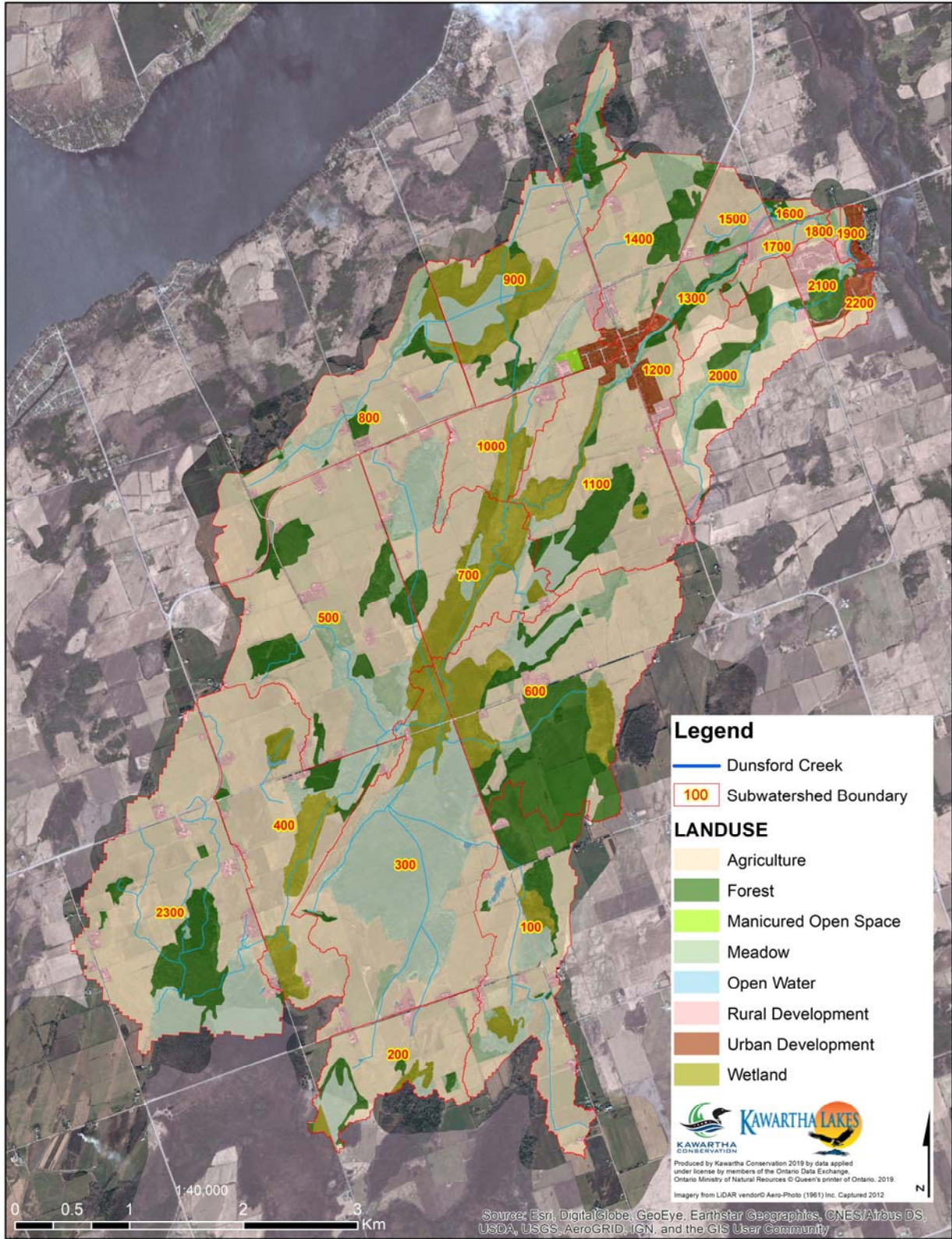
Figure 3.3: Future Land Use

3.8. Impervious Land Use and Runoff Coefficients

The detailed land use denoted in the OP 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 A**.

Subcatchments with a T_{imp} value greater than 20% were modeled with the StandHYD command; otherwise the NashHYD command was used. Generally, the total area of impervious surfaces within the Dunsford watershed is small, and therefore all subcatchments were modelled as a NashHYD>

3.9. Channel Routing

Channel routing in VH SUITE accounts for the storage of flows as they are conveyed within the main channel and floodplain. Accounting for the conveyance of the unsteady flows along the channel results is referred to as channel flow routing. Channel flow routing results in the attenuation (lowering) and a latter (lag) in peak flows.

Cross-sections are input to the Route Channel command within VH SUITE.   One representative cross-section (Figure 3.4) was used for each channel reach.   Reach channel and overbank Manning's n values were averaged, as were the channel and overbank slopes.

3.10. Stormwater Management (SWM) Ponds

There are no SWM facilities in the study area.
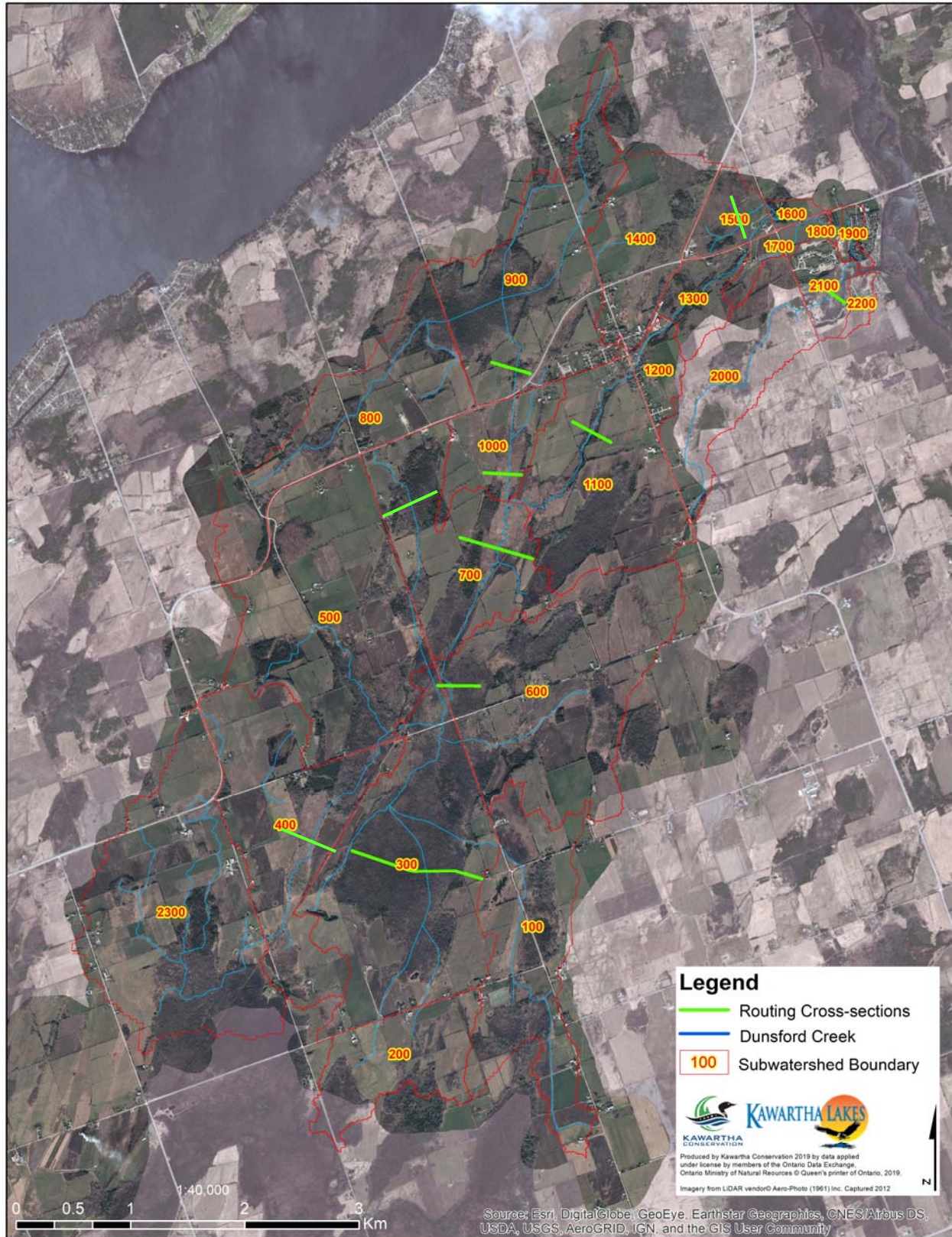
Figure 3.4: Routing Cross-sections

4. Hydrologic Model

4.1. Schematic

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

4.2. Calibration

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

4.3. Model Input Data

The input parameters were calculated as described in section 3, and are summarized in **Table 4.1** below.

Table 4.1: Visual OTTHYMO Model Input Parameters

Catchment	Area (Ha)	C	T_p (hr)	CN* (II)	Ia (mm)	X_{imp}	T_{imp}
100	197.6	0.35	2.63	67	5.0	0.01	0.02
200	107.2	0.33	0.87	68	5.0	0.01	0.02
300	315.5	0.35	2.03	54	5.0	0.01	0.01
400	228.1	0.35	1.65	67	5.0	0.01	0.01
500	302.3	0.35	1.31	67	5.0	0.01	0.01
600	299.6	0.32	1.85	63	5.0	0.01	0.01
700	181.2	0.36	1.89	60	5.0	0.01	0.01
800	183.2	0.33	1.53	68	5.0	0.01	0.02
900	236.7	0.33	1.97	64	5.0	0.01	0.02
1000	69.4	0.36	1.06	67	5.0	0.01	0.02
1100	203.1	0.37	1.74	70	5.0	0.01	0.02
1200	30.8	0.4	0.85	78	5.0	0.07	0.12
1300	42.9	0.42	0.53	80	5.0	0.02	0.03
1400	117.7	0.37	0.97	74	5.0	0.03	0.03
1500	37.3	0.4	0.72	74	5.0	0.08	0.11
1600	8.9	0.37	0.78	74	5.0	0.02	0.02
1700	21.8	0.35	0.82	74	5.0	0.02	0.03
1800	3.82	0.42	0.10	78	5.0	0.04	0.07
1900	10.25	0.53	0.33	80	5.0	0.11	0.18
2000	125	0.31	1.50	67	5.0	0.01	0.02
2100	34.3	0.42	0.36	80	5.0	0.11	0.17
2200	9.7	0.41	0.21	78	5.0	0.09	0.13
2300	308.8	0.33	1.85	64	5.0	0.01	0.02

4.4. Sensitivity Analyses – Hydrologic Analysis

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 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. This was done to assess flood elevation sensitivity relative to the accuracy of peak flow estimates in the hydraulic modelling. Results of the sensitivity analysis are provided in **Appendix F**. The following parameters were tested for sensitivity:

- Curve Number (CN*) (+/- 20%);
- Initial Abstraction (+/- 50%);
- Model Time Step (+/- 50%);
- Removal of Channel Routing;
- Channel Routing Length (+/- 20%);
- Subcatchment Travel Length (+/- 20%); and
- Model Time Step (DT (+/- 50%).

Curve Number (CN*)
Flows 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 27% at all key flow nodes during the Timmins storm event. Decreasing CN* by 20% resulted in an average decrease in peak flow of 24% at all key flow nodes during the Timmins storm 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 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' Secondary Plan and zoning maps as well as the 2010 Ecological Land Classification (ELC) mapping. Aerial orthophotography was reviewed to confirm land use throughout the watershed. This base data is valid, and therefore any calculated value (such as CN*) based on this data truly represents the land.

Initial abstraction (I$_a$)
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 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 less than 1% at all key flow nodes during the Timmins storm event. Therefore, changing the initial abstraction does not result in significantly different flows.

Subcatchment Travel Length (TL)
The travel length is used to determine the flow time of concentration for a subcatchment area. A small travel length increases peak flows, as smaller travel lengths also reduce the overland flow gradient which ca also increase peak flow. Flow lengths were delineated automatically using GIS

software, and was revised based aerial topography and engineering judgement (by straightening flow paths or realigning flow paths to channels, municipal drains or swales).

Increasing subcatchment travel length by 20% resulted in an average decrease in peak flow of 3% at all key flow nodes during the Timmins storm event. Decreasing subcatchment travel length by 20% resulted in an average increase in peak flow of 2% at all key flow nodes during the Timmins storm event. Changing the subcatchment travel length is considered to be significant for the study area.

Channel Routing Removed
Channel routing accounts for the storage of flow as it is conveyed along the watercourse and its floodplain. 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 75% at all key flow nodes during the Timmins storm event. 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.

Channel Routing Lengths
Channel routing lengths were varied by +/- 20% to determine the effects storage on peak flows. A smaller channel routing length would result in an increased slope and lower storage volume, therefore resulting in a reduced travel time and peak flow attenuation from node to node. Channel routing lengths were delineated automatically using GIS software, and was revised based aerial topography, known water courses and engineering judgement.

Increasing channel routing lengths by 20% resulted in an average decrease in peak flow of 13% at all key flow nodes during the Timmins storm event. Decreasing channel routing length by 20% resulted in an average increase in peak flow of 14% at all key flow nodes during the Timmins storm event. Therefore, changing the channel routing length results is somewhat significant.

Channel routing lengths can be considered relatively accurate, as watercourses can be visually confirmed via aerial orthophotography or official watercourses. Therefore, there is confidence that acceptable channel routing lengths were applied.

Model Time Step (DT)
The model time step of 10 minutes was modified by changing it by +/- 5 minutes at all subcatchments and channel routing. There was little to no affect on peak flows at all flow nodes during the Timmins Storm Event (less than 0.5%). Therefore, time step has no effect on the regulatory flows.

5. Hydrology Model Output

Flow Results

As can be seen in **Figure 5.1** below, the catchments display two distinct hydrological responses closely matching the rainfall pattern of the Timmins storm. The first peak occurs around 3 hours after the beginning of the storm event for all catchments. The second peak differs in response however – some catchments respond rapidly, others more slowly.
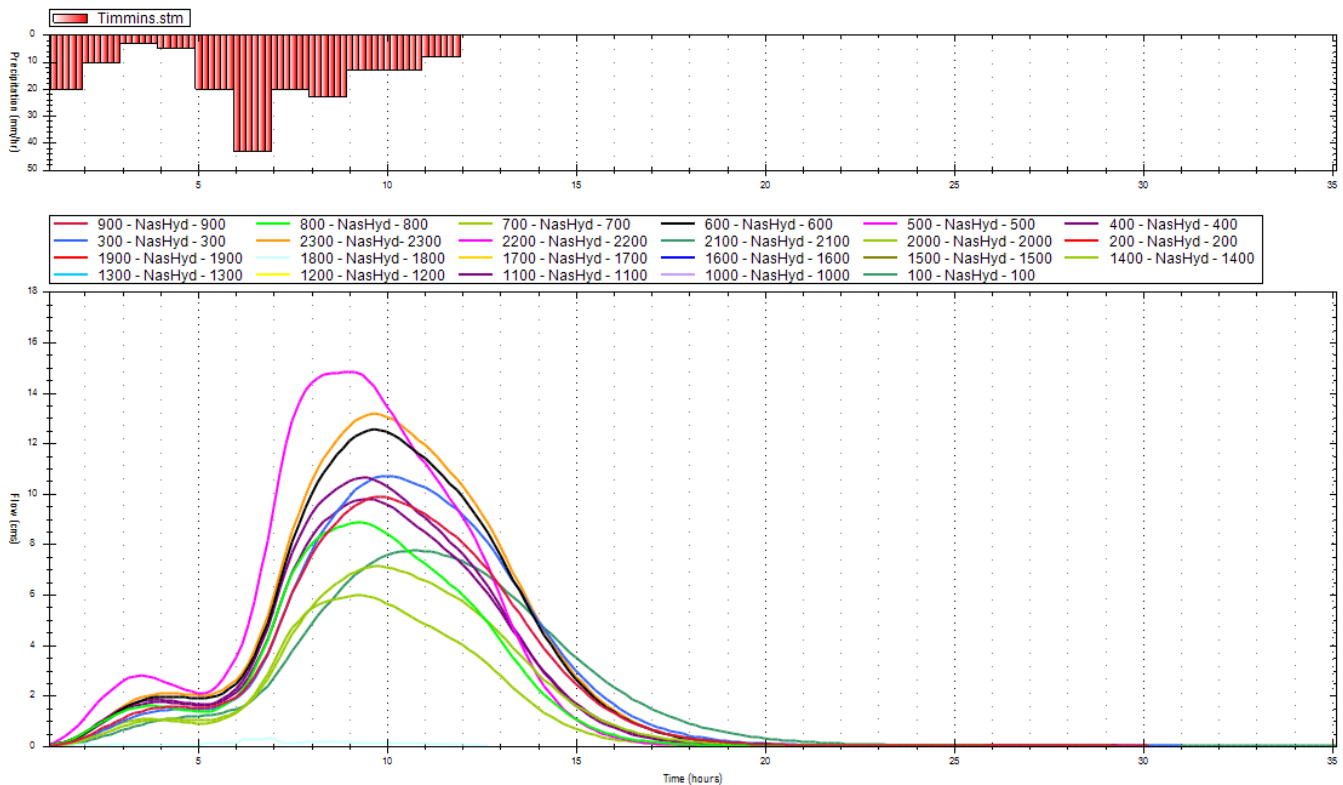
Figure 5.1: Catchment Runoff Comparison

Most of the subcatchments generate a sharp peak flow after the second intensive rainfall. This may be a result of combined hydrological responses such as a higher overland contribution, antecedent moisture conditions or low hydraulic conductivity of the soils, resulting in low storage capacity generating a faster runoff.

Flow Output

Table 5.1 shows the representative peak flows at key flow nodes of the various 100-year storm distributions in effort to determine the critical storm distribution of the watershed.

Table 5.1:Critical Storm Distribution Assessment – 100 Year Event

VO Flow Node	Peak 100 Year Flows (m³/s)								
	SCS Type II			Chicago				AES	
	6 Hour	12 Hour	24 Hour	4 Hour	6 Hour	12 Hour	24 Hour	1 Hour	12 Hour
90	19.77	16.51	14.10	13.96	15.22	16.49	17.87	7.85	17.98
101	19.84	16.77	14.33	13.97	15.40	16.70	18.07	7.84	18.15
121	21.23	17.90	15.35	15.02	16.40	17.78	19.19	9.73	19.65
110	21.30	17.96	15.40	15.07	16.45	17.83	19.23	9.99	19.72
123	21.48	18.11	15.53	15.21	16.58	17.96	19.37	10.61	19.93
130	21.48	18.13	15.55	15.21	16.58	17.97	19.39	10.61	19.93
150	21.33	18.16	15.57	15.14	16.56	17.98	19.35	9.97	19.92

Therefore, it can be established that the critical storm distribution is the SCS Type II, 6 hour event. All storm events for the 2 through 100 year event will be modelled as the SCS Type II, 6 hour distribution.

Table 5.2 shows the representative peak flows to be input to the HEC-RAS model for the Regulatory storm event and the 6-hour SCS 100 year storm.

Table 5.2: Input Flows to HEC-RAS

HEC-RAS ID	Location	VO Flow Node	Peak Regional Flow (m³/s)	100 Year Flow (m³/s)
3532	US of Sturgeon	90	64.73	19.01
2886	DS of Community Centre	101	65.55	19.07
1815	DS of HWY 36 (W)	121	69.77	20.51
1498	DS of Cedar Glen Road	110	70.01	20.59
1186	DS of HWY 36 (E)	123	70.55	20.78
883	DS of Herons Landing	130	70.61	20.78
755	DS of Herons Landing 2	150	70.76	20.71

6. Hydraulic Model Input Parameters

6.1. Overview

The following section presents the setup and findings for the hydraulic analyses. The calculated flood elevations were used to prepare regulatory floodplain maps for the Dunsford Creek watershed. Steady flow hydraulic analyses were completed using GeoHECRAS™ (Civil GEO) software. The procedures used were based on the 2012 City of Kawartha Lakes and Kawartha Conservation standardized methodology for undertaking their flood plain mapping studies.

6.2. Cross-Sections

Cross-section geometric data was extracted using GeoHECRAS™ from the base DEM to ensure geo-referencing in HEC-RAS. Since bathymetric data acquisition was outside the scope of the project LiDAR acquisition, it was 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 replaced the DEM-derived elevations within the in-channel portion of the cross-sections generated by GeoHECRAS™. Data sources generated by different entities were placed into the same projection and datum for consistency in processing. Stream crossings were selected based on project orthoimagery, field reconnaissance, and information in previous reports. Full photographic records of all stream cross-sections are found in **Appendix H**.

As per HEC-RAS requirements, all cross-sections are oriented looking downstream. The initial cross-section is at the outlet of the creek as it enters Emily Creek. with minor exceptions, the cross-section nomenclature reflects the distance in metres relative to the initial cross-section.

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

6.3. Culvert and Road Crossings

Cross-sections are cut at culvert and bridge crossings to accurately represent channel flow. All road crossings are represented by two upstream and two downstream bounding cross-sections. Representative deck elevations were extracted from the base DEM. All culverts and bridges were field-surveyed to ensure accuracy. Invert elevations, height/width dimensions, length, and channel bottom were surveyed with either total station or GPS. **Table 6.1** provides key details. Other relevant data and photographs are found in **Appendix I**.

Table 6.1: HEC-RAS Structure Data

Street	River Sta.	Material	Bottom	Shape	Invert Elevation (m)		Length (m)	Size (mm)	
					U/S	D/S		Span	Rise
Sturgeon Rd	3503	Concrete	Open	Arch	269.25	269.13	15.74	4.9	1.2
Community Centre	2906	CSP		Circular	267.31	267.30	11.54	0.9	
Community Centre	2906	CSP		Circular	267.34	267.31	9.88	0.9	
Community Centre	2906	CSP		Circular	267.38	267.36	9.28	0.9	
County Rd 36 W	1834	Concrete	Open	Box	256.45	256.37	16.66	6.0	2.5
Cedar Glen Rd	1513	Concrete	Open	Arch	253.34	253.25	9.58	4.0	2.1
County Rd 36 E	1249	Concrete	Open	Box	250.34	250.13	16.28	6.85	1.95
Herons' Crossing N	891	bridge			247.44	247.44	4.65	5.2	1.97
Herons' Crossing S	763	bridge			247.64	247.01	3.04	13.43	1.28

A private dam is on the creek between Cedar Glen Road and Kawartha Lakes Road 36 (east) crossing, as seen in **Figure 6.1** below. The condition of the dam was not evaluated as part of this study so its structure integrity is unknown. There are stop logs controlling flow, but it is not known how actively the owner moves the stop logs. As a result, the structure was not included in the hydraulic analysis of the creek.

Figure 6.1: Private Dam

6.4. 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*. The main channel n values are 0.035 and 0.045, and the overbank n values range from 0.03 to 0.1. 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.

6.5. Ineffective Flow Elevations

Ineffective flow areas were introduced at each culvert crossings and selected cross sections to identifies areas which would not contribute to the conveyance of flood flow. Typically, the upstream bounding cross-section at culverts the ineffective elevations was set to the low elevation in the roadway. For the downstream bounding cross-section, the ineffective flow elevations were typically set at a point midway between the low roadway elevation and the culvert obvert elevations.

6.6. Boundary Conditions

For the subcritical flow analysis, the downstream boundary condition is the normal Sturgeon Lake operating level of 247.76 m, controlled by the Trent Severn Waterway. It is recognized that the lake is approximately 4.5 km downstream of the creek outlet, but since there is no recorded water level data on Emily Creek the only reliable water elevation data available is for Sturgeon Lake.

6.7. Expansion/Contraction Coefficients

The model uses the HEC-RAS recommendations of 0.1 and 0.3 for contraction and expansion coefficients, respectively at all normal cross-sections. The values were increased at culverts and bridges and culverts (typically to 0.3 and 0.5, contraction and expansion, respectively) to account for more significant changes in flow conveyance velocity.

6.8. 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 5 m upstream of a 30 m-wide building, the obstruction representing the building in the cross-section is 20 m wide. A 4:1 expansion effect was used for a cross-section downstream of a building. For instance, if a cross-section is 8 m downstream of a 30 m-wide building, the obstruction representing the building in the cross-section is 26 m wide.

7. Hydraulic Model

7.1. 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 7.1**.
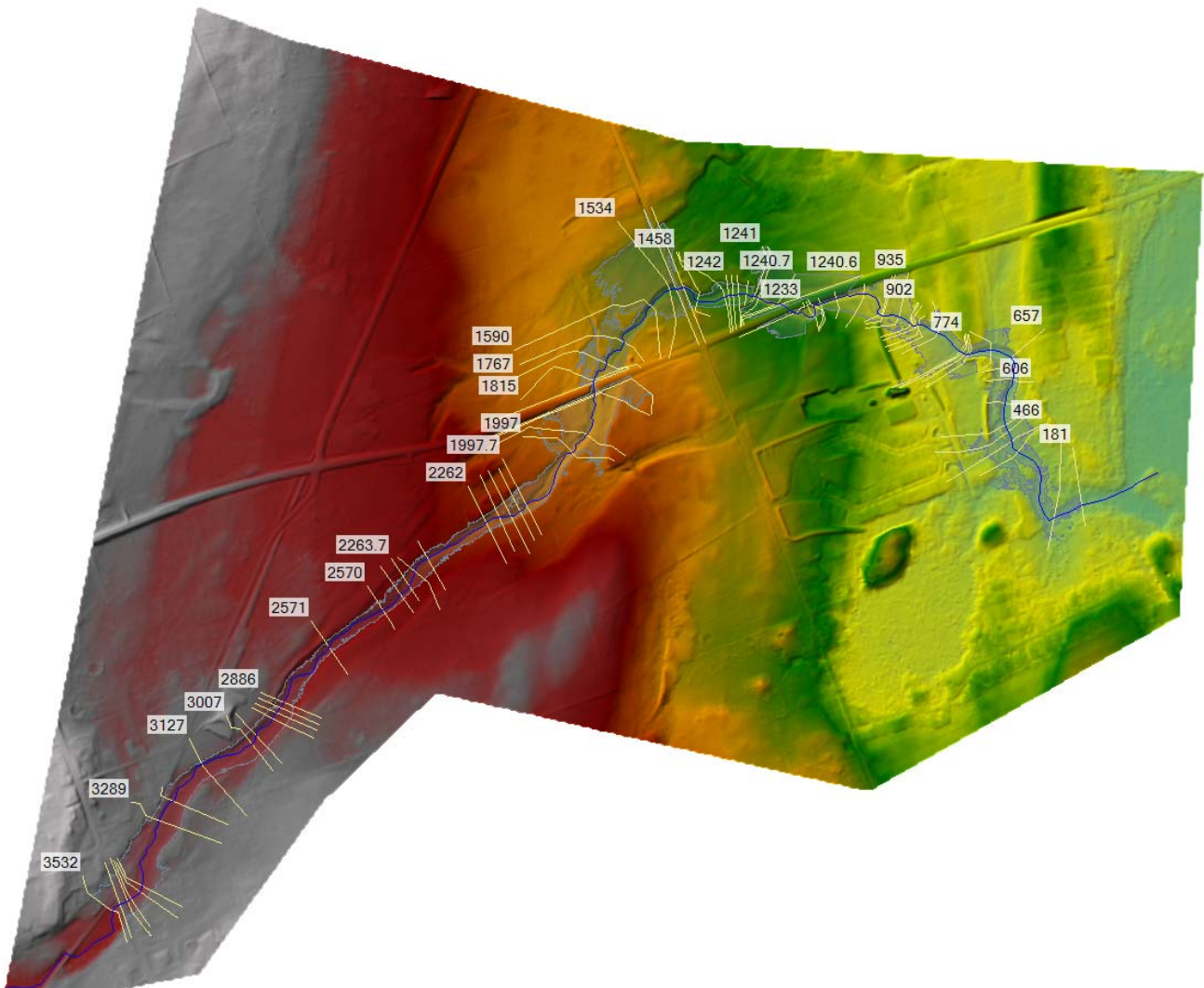
Figure 7.1: HEC-RAS Schematic

7.2. Sensitivity Analyses

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 only (Timmins Event). The increase/decrease in flood elevation from the base scenario were noted to establish a level of confidence in flood elevation estimations. The following parameters were tested for 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 F.**

Manning roughness coefficient

Flood elevations throughout the project 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 0.1.

By increasing the Manning's n by 20%, the flow is being subject to a watercourse with greater friction forces acting upon it. It was found that the average increase in the regional water surface elevation throughout the 83 cross section was 5 cm, and the highest was 20 cm, at cross section 2570.

By decreasing the Manning's n by 20%, the flow is being subject to a watercourse with lower friction forces acting upon it. It was found that the average decrease in the regional water surface elevation throughout the 83 cross section was 5 cm, and the greatest was 31 cm, at cross section 1997.9.

Due to a minimal affect on the average, overall flood elevation throughout the study reach, it can be determined that the Manning roughness coefficients are acceptable.

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 flow varied by up to 27%, therefore peak flows within the hydraulic model were varied by +/- 30%.

By increasing the peak flows, it was found that the average increase in regional flood elevation throughout the 83 cross sections was 19 cm, with the highest greatest of 79 cm at cross section 1147. Cross section 1147 has a relatively narrow cross section, with limited floodplain access.

Therefore, increases in flood elevations are significant due to an entrenched channel. By decreasing the peak flows, it was found that the average decrease in regional flood elevation throughout the 83 cross sections was 24 cm, with the greatest decrease of 65 cm at cross section 1233.

Cross section 1233 is located immediately downstream of the Highway 36 bridge crossing. During the base scenario, Timmins flood elevation is immediately above an ineffective flow area (associated with the bridge). As flows are reduced, the flood elevation decreases below the ineffective flow area, causing flood elevations to decrease significantly.

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

Downstream Boundary Condition

The Dunsford Creek flows into the Emily Creek, a tributary to Sturgeon Lake. Due to the lack of known water levels immediately at the confluence of Dunsford Creek and Emily Creek, a downstream boundary condition was based on the Sturgeon Lake normal operating level of 247.76 m, as a flow gauge is located approximately 10.5 km north easterly in Bobcaygeon. Due to the uncertainty of the starting water elevation, a sensitivity analysis was completed by varying the starting water level by +/- 1.0m. For most of the cross sections, the regional flood elevation remained unchanged. When increasing the downstream boundary condition to 248.76m, only the 10 downstream cross sections had a change in flood elevations, with an average of 32 cm through these sections. The limit of this backwater effect ends at cross section 755.

The most downstream cross section (181) has a channel invert of 246.59 m. Therefore, when decreasing the downstream boundary condition to 246.76 m. The most downstream cross section's flood elevation decreased by 16 cm with the remainder unchanged.

Due to the limited effects on flood elevations throughout the entire watershed, the starting water elevation is considered acceptable for the study area.

8. Hydraulic Model results

8.1. Creek Flood Results

The resulting flood elevations for the 2 through 100 year, and Regional storm flood event for Dunsford Creek are listed in **Appendix J**. The regulatory flood elevation is defined as the greater of the 100 year or regional storm flood elevation. For the Dunsford Creek watershed the Regional storm defines the regulatory flood elevation. The Regulatory flood plain extents are illustrated in **Figure 8.1**.

There are several locations where flood waters cannot be contained within the natural valley lands of the Dunsford Creek or are redirected by the limited hydraulic capacity of culverts and configuration of roadways. This spilling of the flood water either finds its way back into Dunsford Creek or spills into Emily Creek. The spill locations include:

1. spills toward the east for the portion of the watershed between Kawartha Lakes Road 36 (east) and Cedar Glen Road.
2. Spill at the Highway 36 (W) crossing (structure 1816.5 within the HEC-RAS model). Spill overtops the roadway and is conveyed easterly along Highway 36. This spill would eventually be captured by road-side ditches where flows would be re-routed back towards the creek.
3. Spill at Highway 36 (E) crossing (structure 1237.34 within the HEC-RAS model). Spill overtops the roadway and is conveyed easterly along Highway 36. This spill would eventually be captured by road-side ditches where flows would be re-routed back towards the creek.

Further assessment of the spill areas was beyond the scope of the current project.

Figure 8.2 shows the profile of the creek and its riverine Regulatory flood elevation. The profile illustrates structures which have the capacity to pass the 100 year storm event and other which do not provide this capacity. The profile also illustrates where road crossing cause backwater effects onto upstream lands during the Regional storm event, for example at Highway 36 West. Not all culverts are required to pass the 100 year storm or regional storm event.
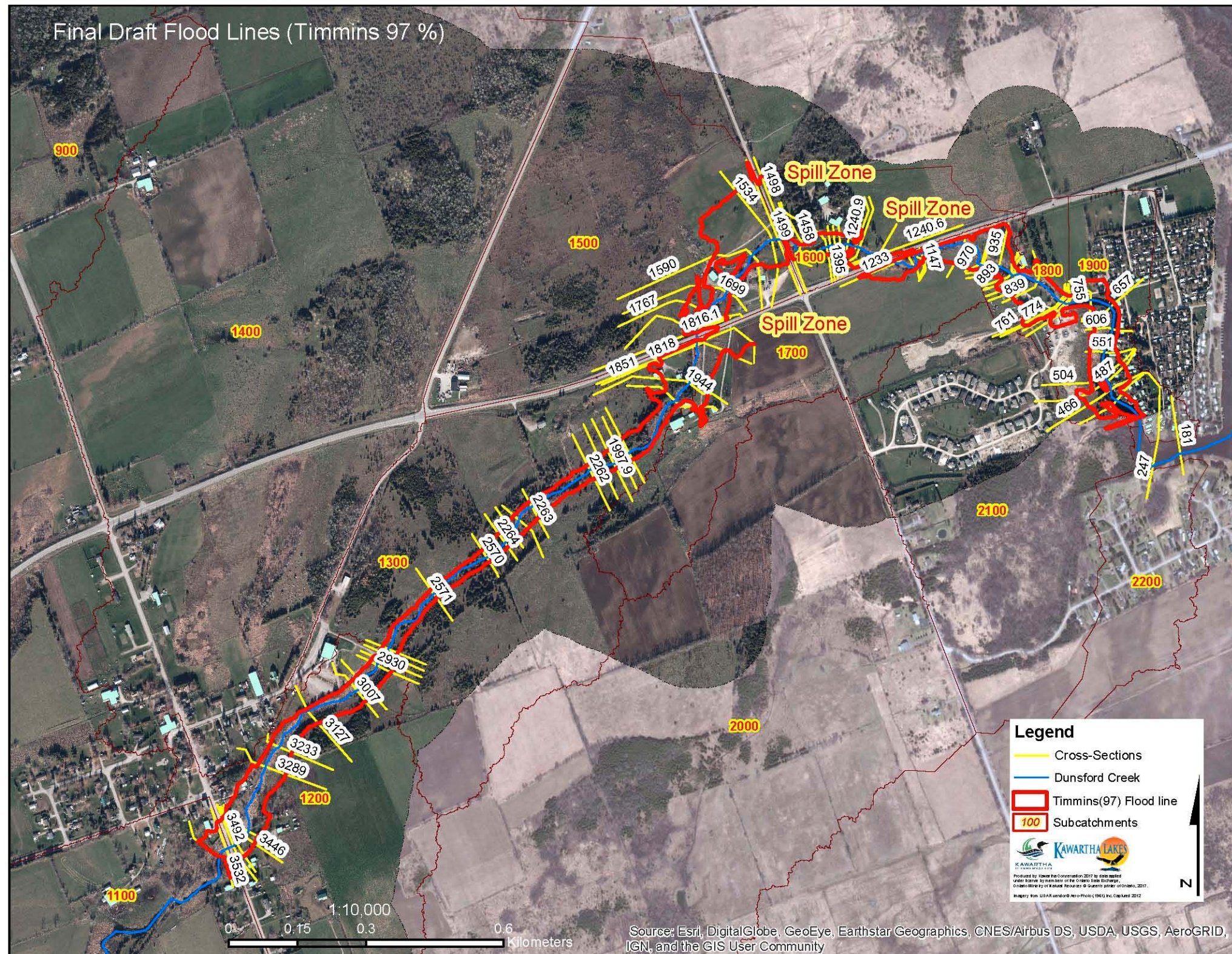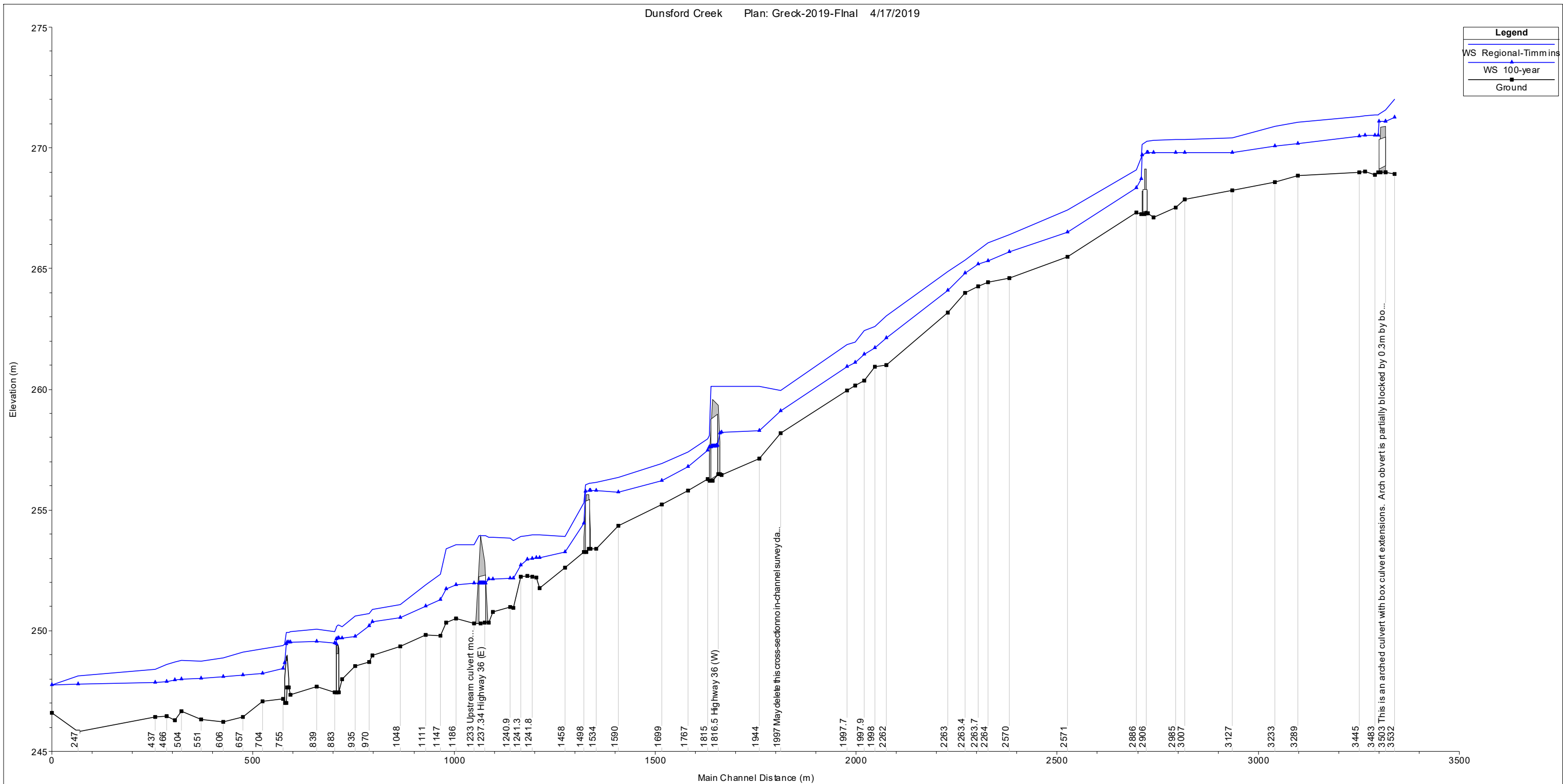
Figure 8.1: Regulatory Flood Plain Extents

Figure 8.2: Regulatory and 100 Year Profile

9. Conclusions and Recommendations

Dunsford Flood Plain Mapping Study is the first ever study for the Dunsford Creek watercourse to generate flood plain mapping. The procedures and methodologies for the hydrologic and hydraulic components are based on terms of reference approved by the Technical Committee (consisting of representatives from each of the City of Kawartha Lakes, Kawartha Conservation, and Ganaraska Conservation). Each methodology and procedure were peer-reviewed for quality assurance and control.

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. It is recommended that the water surface elevations and flood plain mapping be adopted for current and future planning purposes.

10. Limitations of Work

The maps associated with this study have been produced at a strategic, watershed level using an automated mapping process, and minor or local features may not have been included in their preparation. A Digital Terrain Model (DTM) is used to generate the maps. The DTM is a 'bare earth' model of the ground surface with manmade and natural landscape features such as vegetation, buildings, bridges and embankments digitally removed. Therefore, the maps should not be used to assess the flood risk associated with individual properties or point locations, or to replace a detailed local flood risk assessment.

The maps associated with this study were produced based on survey data captured prior to, and during the early part of the project. They do not account for changes in development, infrastructure or topography that occurred after the date of survey data capture.

The DTM is derived from aerial remote sensing data. The majority of this data is Light Detection and Ranging (LiDAR) data. In areas with no LiDAR data present, the best available DTM was used.

Detailed explanations of the methods of derivation, survey data used, etc. are provided in the relevant reports produced for the project under which the maps were prepared. Users of the maps should familiarize themselves fully with the contents of these reports in advance of the use of the maps.

11. Appendices

(Bound in a separate document)

Appendix A: **Modeling Parameters Selection**

Appendix B: **Rainfall Data**

Appendix C: **Subcatchment Data**

Appendix D: **Subcatchment Maps**

Appendix E: **VH Suite Output**

Appendix F: **Sensitivity Analysis**

Appendix G: **Official & Secondary Plan Maps**

Appendix H: **Cross-section Photo Inventory**

Appendix I: **Structure Photo Inventory Record**

Appendix J: **HEC RAS Output**