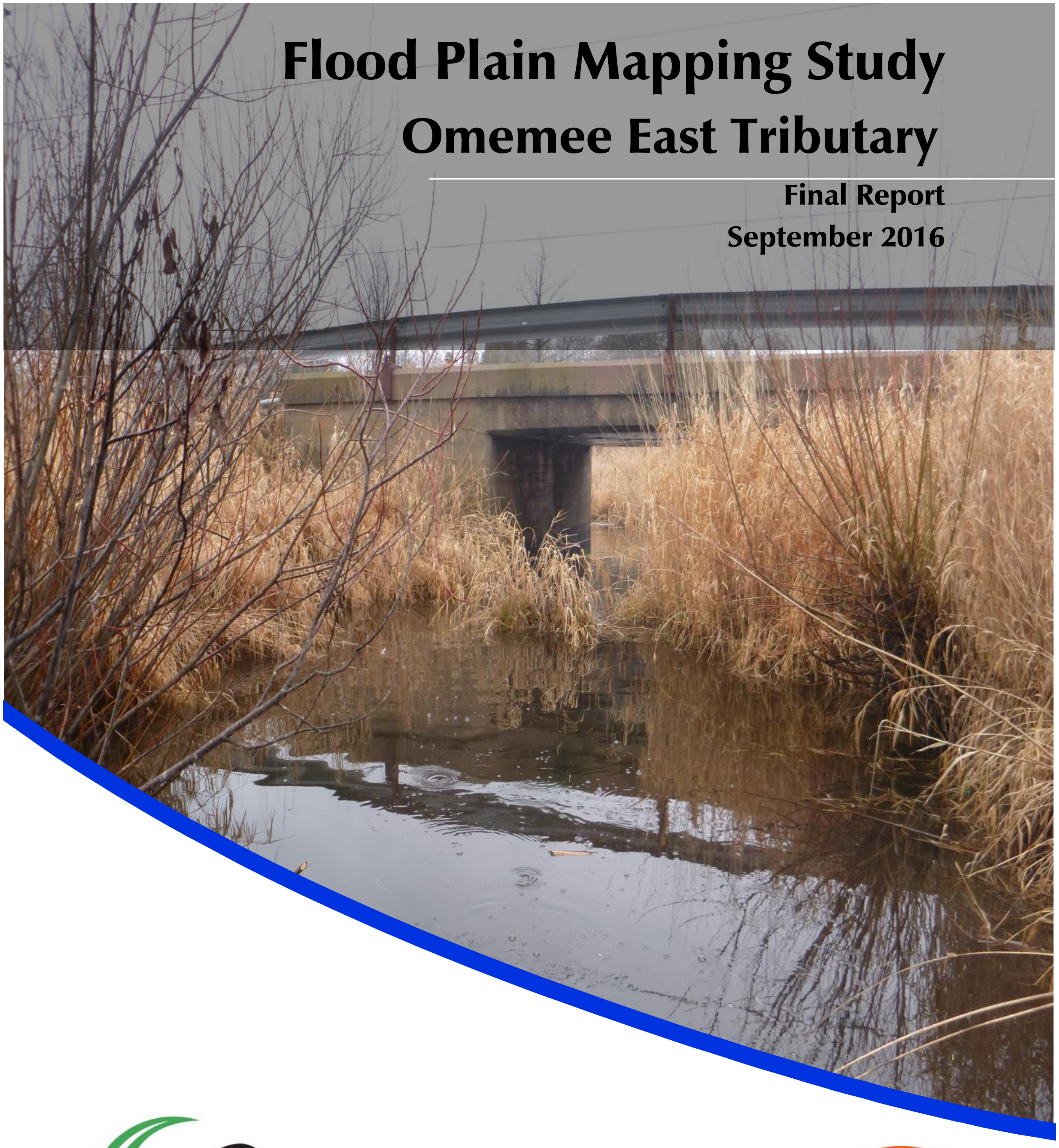
Flood Plain Mapping Study
Omemee East Tributary

Final Report
September 2016



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November 28, 2016

Mr. Mark Majchrowski
Director, Integrated Watershed Management and Science
Kawartha Conservation
277 Kenrei (Park) Road
Lindsay, Ontario
K9V 4R1

Dear Mr. Majchrowski:

Re: Flood Plain Mapping Study Omemee

Please find attached the final report, maps, and technical appendices for the above-mentioned flood plain mapping study, as part of the multi-year City and Authority flood plain mapping project.

This report and accompanying maps supersede all previous flood plain maps and studies for the watershed.

Yours truly,

Christie Peacock, P. Eng.
Water Resources Engineer

Executive Summary

The primary goals of this study are to create hydrologic and hydraulic models of the watershed and produce flood plain mapping for Omemee 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 Omemee 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:

- Previous studies in the area
- 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 the Queen Street road crossing
- Creation of a HEC-RAS hydraulic model
- Creation of flood plain maps

Key findings of this study include:

- Peak flows at key nodes are based on the 6-hour SCS storm
- The majority of the flood plain is determined not by creek flooding, but by the backwater associated with the dam on the Pigeon River
- The Timmins storm is the Regulatory event for the watercourse
- Flood plain maps were created based on the higher flood elevation of either the riverine flooding or the flooding caused by the backwater of the Pigeon River dam

Key recommendations of this study:

- The maps created from the results of the HEC-RAS model for Omemee Creek should be endorsed by the Kawartha Conservation Board

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Table of Contents

List of Tables

List of Figures

Flood maps are at the end of the report

1. Introduction

1.1. Objective

The objective of this study is to generate updated floodplain mapping for the Omemee watercourse to protect the public from flooding hazards. This is the fourth 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) processes to be applied to all projects in the multi-year initiative. The Q/A methodology for each component ensures that the project design meets industry standards, and that the work outline and planned deliverables are valid. The three goals of the Q/C component are: that the product is consistent with standards and generally accepted approaches; that the study results meet the Technical Committee's requirements, and that the products and results are scientifically defensible. Each methodology was peer-reviewed for Q/A and Q/C by an external firm or agency. Four separate components of the project were established for Q/A and Q/C:

- 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 performed accuracy checks on the LiDAR-derived project Base DEM and orthoimagery using the Terms of Reference found in **Appendix S**. For accuracy check results, refer to the "*Digital elevation Model and Orthoimagery Data Accuracy Assessment Report – Flood Plain Mapping Study – Omemee"* in **Appendix Q**.

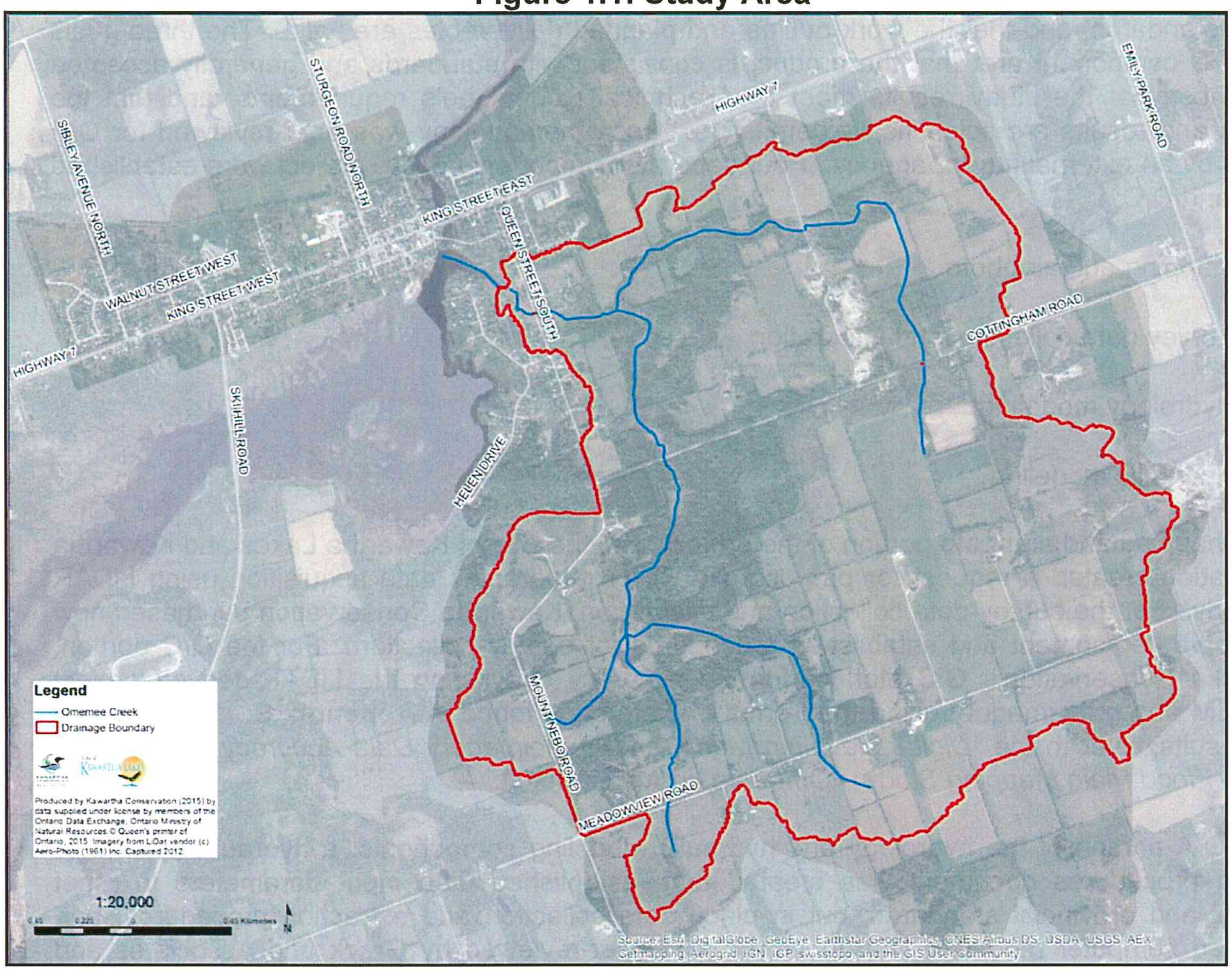
For the Q/A portion of the hydrology and hydraulic modeling components, a hydraulic/hydrologic modeling procedures document was created that: established data input parameters to meet municipal and provincial standards; put in place data collection and extraction procedures; and short-listed computer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

1.3. Watercourse Context and Description

The Omemee watercourse has two branches. The east branch originates in a wooded area just upstream of Cottingham Road. The branch flows north through mainly wooded land interspersed with some open agricultural fields before flowing westward to the junction with the west branch near Queen Street South. The west branch has its origins south of Meadowview Road. The branch flows northward exclusively through forested areas to the junction with the east branch near Queen Street South.

The majority of the watershed is rolling rural farmland and woodlots. Within the Village of Omemee downstream of the junction of the two branches, the watershed is residential within the Secondary Plan. The watershed discharges to the pond upstream of the Pigeon River Dam. The watershed has a size of 750 hectares (7.5 km^2). The east branch is 2.3km long, and is the steeper of the two with an average slope of 2.7%. The west branch is 1.8km long at an average slope of 0.9%. The main channel from the junction to Pigeon River Dam Pond is 660m long at a slope of 0.6%. Please refer to **Figure 1.1**.

Figure 1.1: Study Area



1.4. Background Information

Omemee has experienced flooding in the past. The village is in a low-lying area adjacent to the Pigeon River. A dam on the Pigeon River controls flow and is located just upstream of Mary Street.

The engineering firm Cummings Cockburn Limited (CCL) was retained by the **M**inistry of **N**atural **R**esources (MNR) to carry out a dam safety review of the Omemee dam, whose findings were summarized in the April 2000 *Dam Safety Assessment* report. The key hydrologic and hydraulic items confirmed in the report are:

- The dam is not used for flood control but is used to regulate the headpond level
- The surface area of the reservoir is 200Ha
- The Pigeon River watershed tributary to the dam is $256km^2$, or 25,600Ha
- The dam spillway can pass a flood up to the 100-year flood

The MNR's 2002 document, *Omemee Dam, Operation Plan and Maintenance Manual – Volume 1 of 4 – Dam Operations Manual* stated the normal operating water level of the dam is 248.2m.

A 1992 **O**ntario **M**unicipal **B**oard (OMB) ruling established the Pigeon River dam headpond Regulatory flood elevation as 250.4m.

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

Sensitivity analyses 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.*

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

2. Rainfall

2.1. Rainfall Data

Rainfall Intensity–Duration–Frequency (IDF) curves provide estimates of the extreme rainfall intensity for different return periods. 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.

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 B**.

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

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

I in mm/hr

T in minutes

The 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 *Flood Plain Mapping Study, Ops #1 Drain/Jennings Creek* 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	9.760	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 2.3: 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 B**.

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 were tested. For the 100-year event, 4-hour durations were tested.

Storm Distribution
Rainfall distribution over time determines 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 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 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) was selected as the critical event for the watershed. This provides the most appropriate protection for the community of Omemee. Detailed rainfall information is shown in **Appendix B**.

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*. The Ontario Ministry of Natural Resources (MNR) technical manuals provide a rainfall reduction table for the Timmins storm. Given the size of the Omemee Creek subcatchment no areal reduction factors were used. Antecedent moisture content (AMC) condition II, referred to as AMC (II), was applied.

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 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 LiDAR 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 Omemee watercourse watershed, two points per square meter raw LiDAR data was acquired. ArcGIS version 10.1 computer software programs were to be used to produce bare earth Base DEM using best available raster and point cloud data from the project LiDAR/ortho acquisition. The Base DEM was produced at a 0.5m cell resolution.

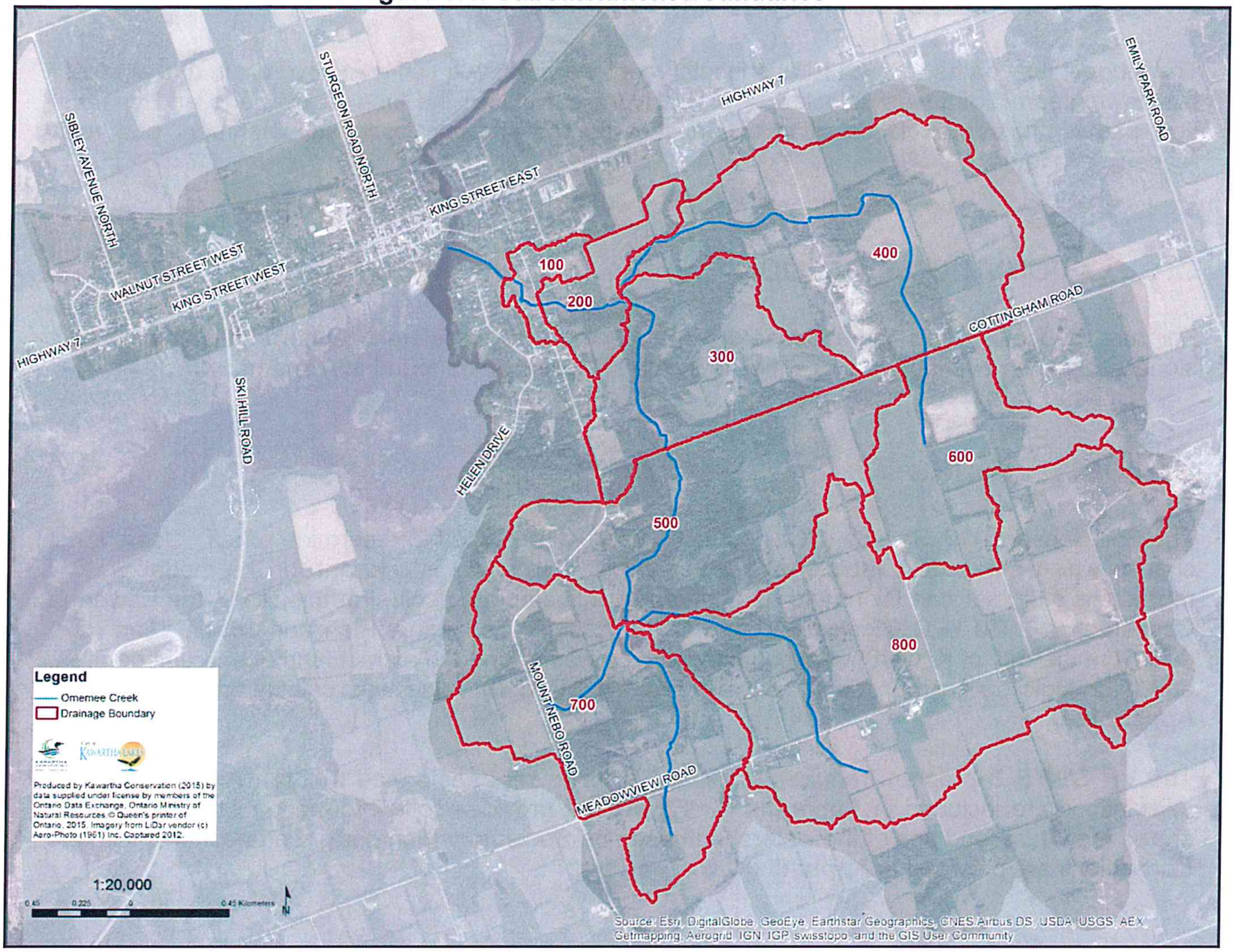
3.3. Subcatchment Discretization

In order to discretize subcatchments, watershed flow paths were generated using ArcHydro version 10.1 beta software. Surveyed bridge and/or culvert data was enforced into the Base DEM to create a hydrologically-conditioned DEM (referred to as a Hydro DEM) at a 0.5m 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.

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



3.4. Land Use

The draft April 2013 Schedule 'F-4' Land Use map version from the Secondary Plan Project, Omemee Settlement Area is the base data referenced for land use patterns. The January 2008 Schedule 'A' zoning map from the Village of Omemee Zoning By-Law 1993-15 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 Secondary 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 H**.

3.5. Rural Subcatchment Properties

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

3.6. 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 C**.

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 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 C**.

The VH SUITE 3 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

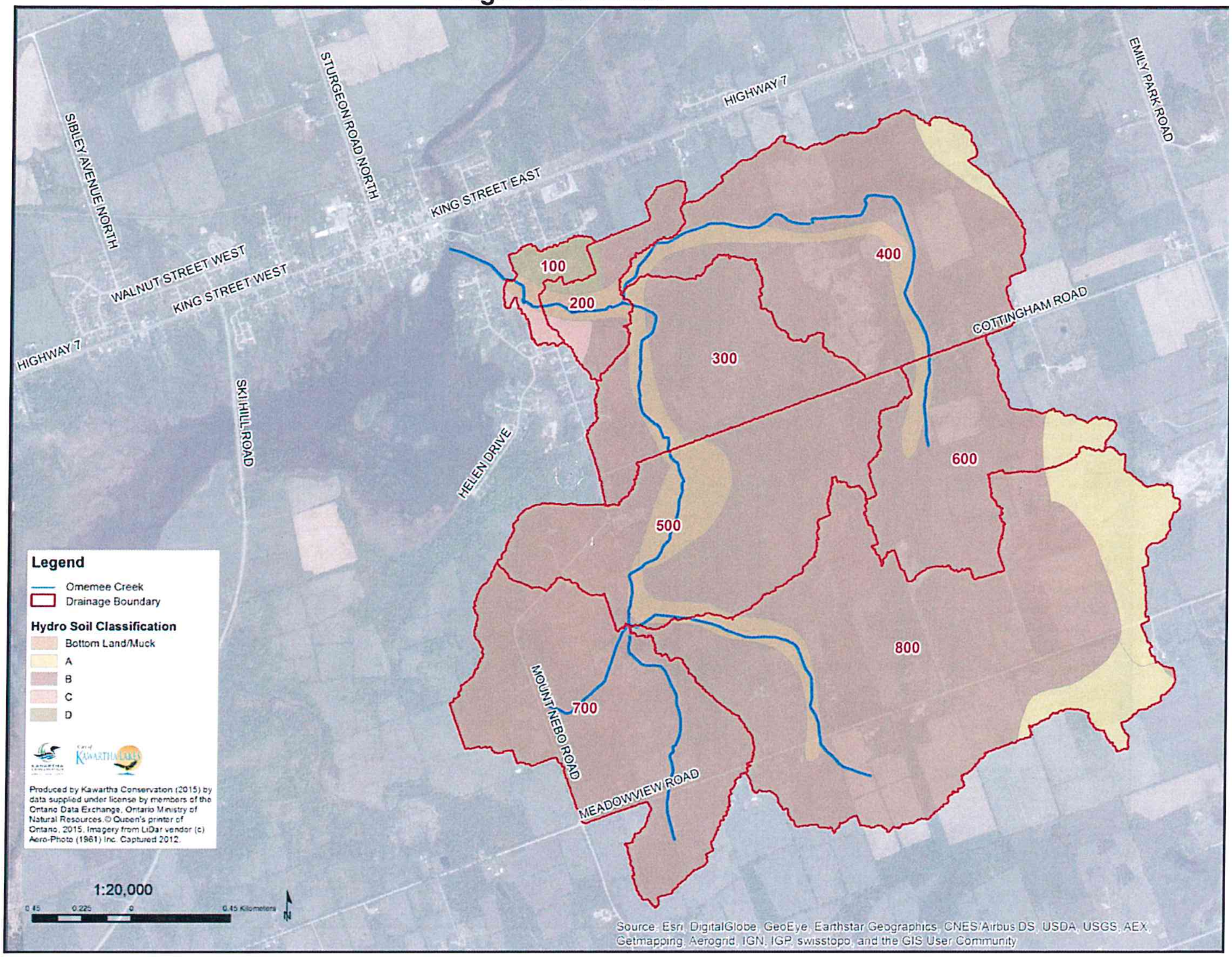
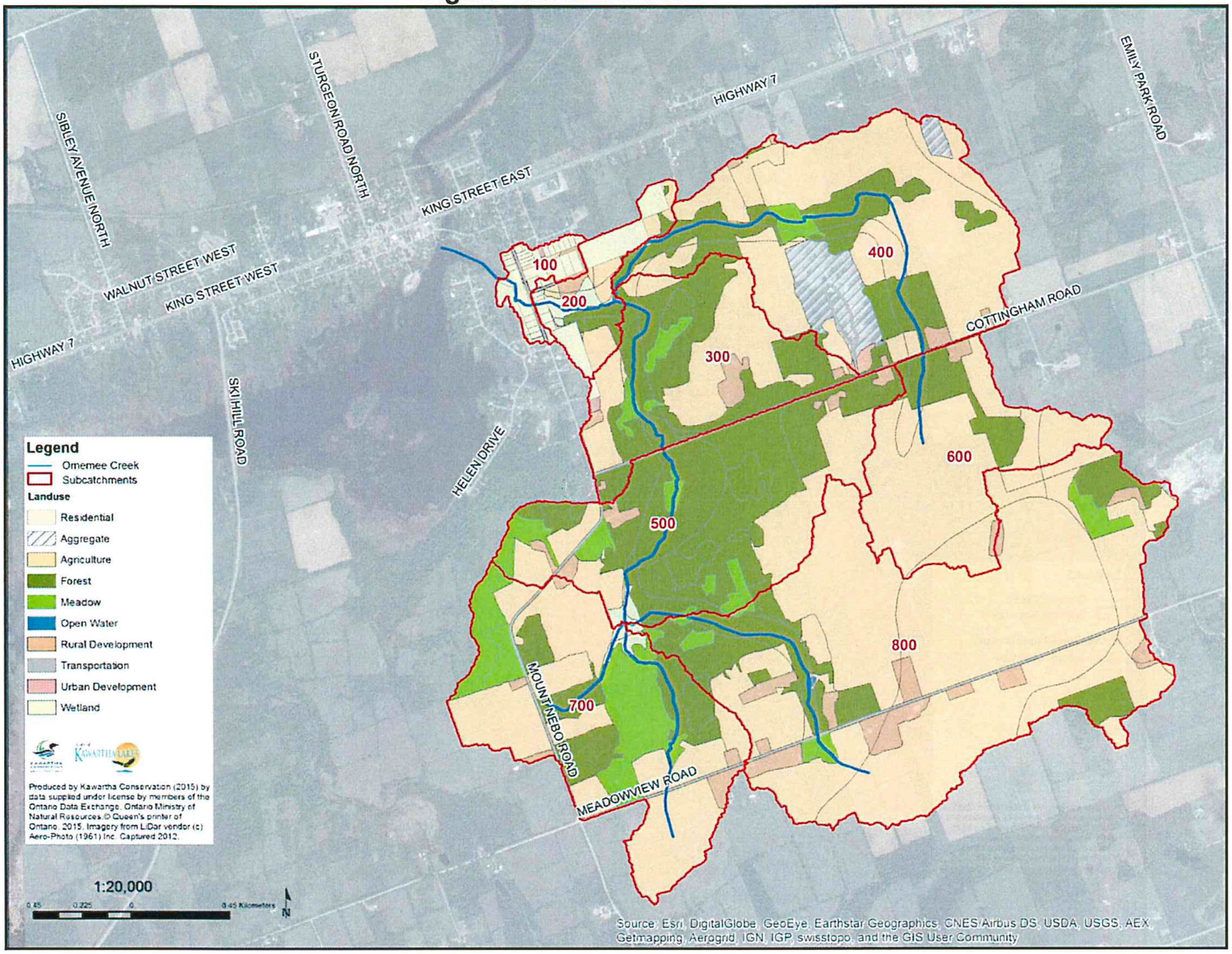


Figure 3.3: Land Use



3.8. Impervious Land Use & Runoff Coefficients

The detailed land use denoted in the 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 A**.

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 C**.

3.9. 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 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. 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**.

3.10. 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 was used for each channel reach. Reach channel and overbank Manning's n values were averaged, as were the channel and overbank slopes.

3.11. Stormwater Management (SWM) Ponds

There are no SWM facilities in the study area.

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 E.**

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. The 100-year and Timmins storm model was modified as outlined below. Detailed information can be found in **Appendix G**.

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 2% decrease in peak flows for the Timmins event, and an average 4% 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 1% for the Timmins storm, and an average of 4% for the 100-year event. The n value is therefore 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 26% increase in peak flows for the Timmins event, and an average 47% increase for the 100-year event. Similarly, when CN* decreased 20%, the model calculated lower peak flows at key nodes: by an average of 25% for the Timmins storm, and by an average of 33% 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 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 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% has no impact for the Timmins storm, and decreases the peak flows by an average of 2% 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 7% at key nodes; for the 100-year storm, peak flows increased by an average of 16%. This is caused by the lack of attenuation in the channels. The inclusion of channel routing is therefore a significant item for the 100-year event. Since the channel length, slope, and cross-section information is derived from a highly-detailed Base DEM, there is confidence that the data is correct.

Channel Flow Length
The channel lengths were modified +/- 20%. Increasing the length by 20% decreases the peak flows by an average of 3% for the Timmins storm, and by an average of 6% for the 100-year storm. Decreasing the length by 20% increases the peak flows by an average of 2% for the Timmins storm, and by an average of 6% for the 100-year storm. Changing the channel length does not result in significantly different flows.

4.4. Model Input Data

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

Table 4.1: Ott-Hymo Model Input Parameters

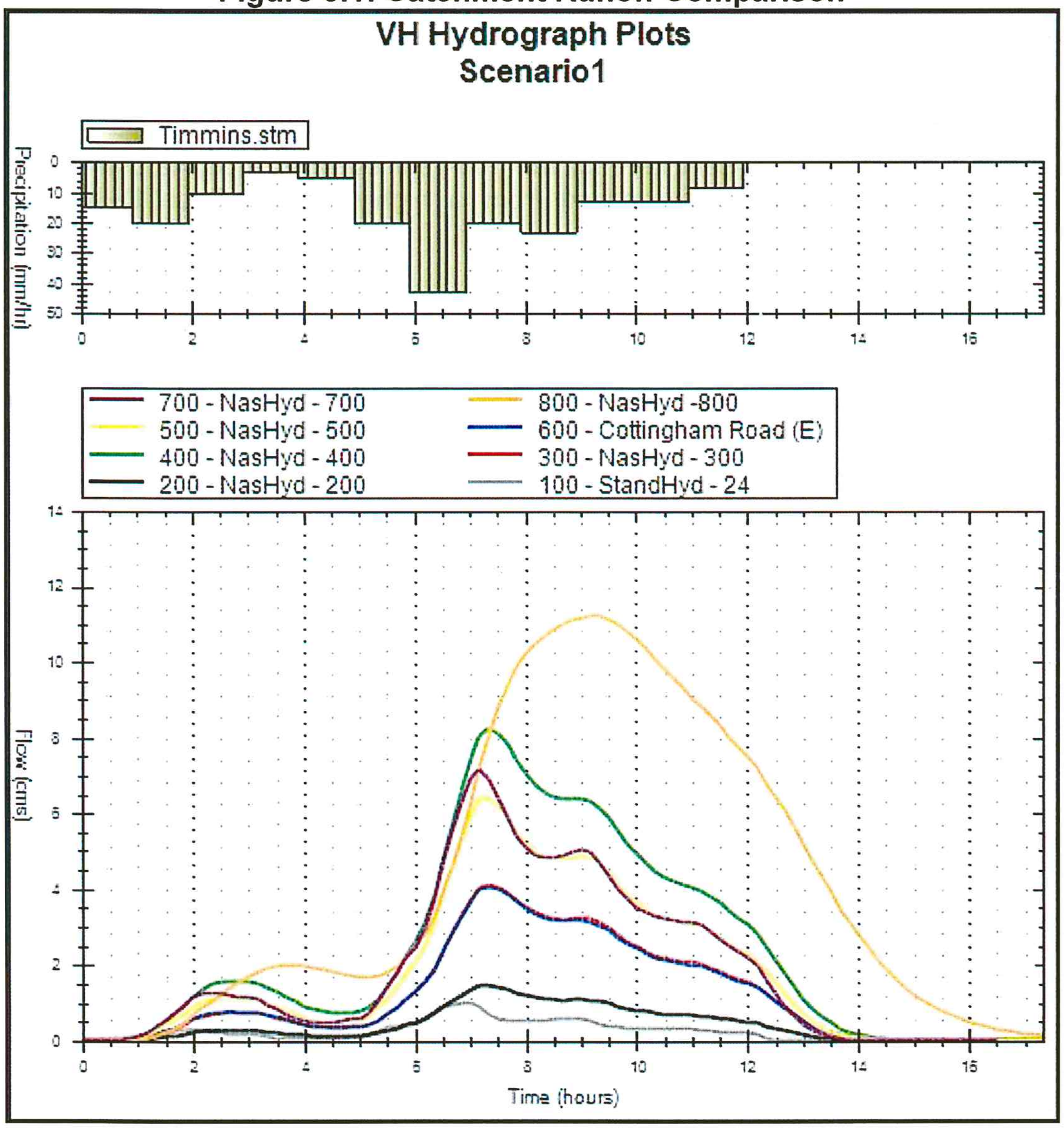
Catchment	Area (Ha)	C	T_p (hr)	CN* (II)	Ia (mm)	X_{imp}	T_{imp}
100	9.8	0.45	N/A	80	1.5	0.14	0.22
200	21.6	0.40	0.65	70	5.0	N/A	N/A
300	69.8	0.31	0.69	64	5.0	N/A	N/A
400	130.1	0.34	0.71	68	5.0	N/A	N/A
500	107.1	0.30	0.63	63	5.0	N/A	N/A
600	65.4	0.33	0.70	68	5.0	N/A	N/A
700	109.0	0.32	0.52	64	5.0	N/A	N/A
800	234.4	0.32	1.49	67	5.0	N/A	N/A

5. Hydrology Model Output

Flow Results

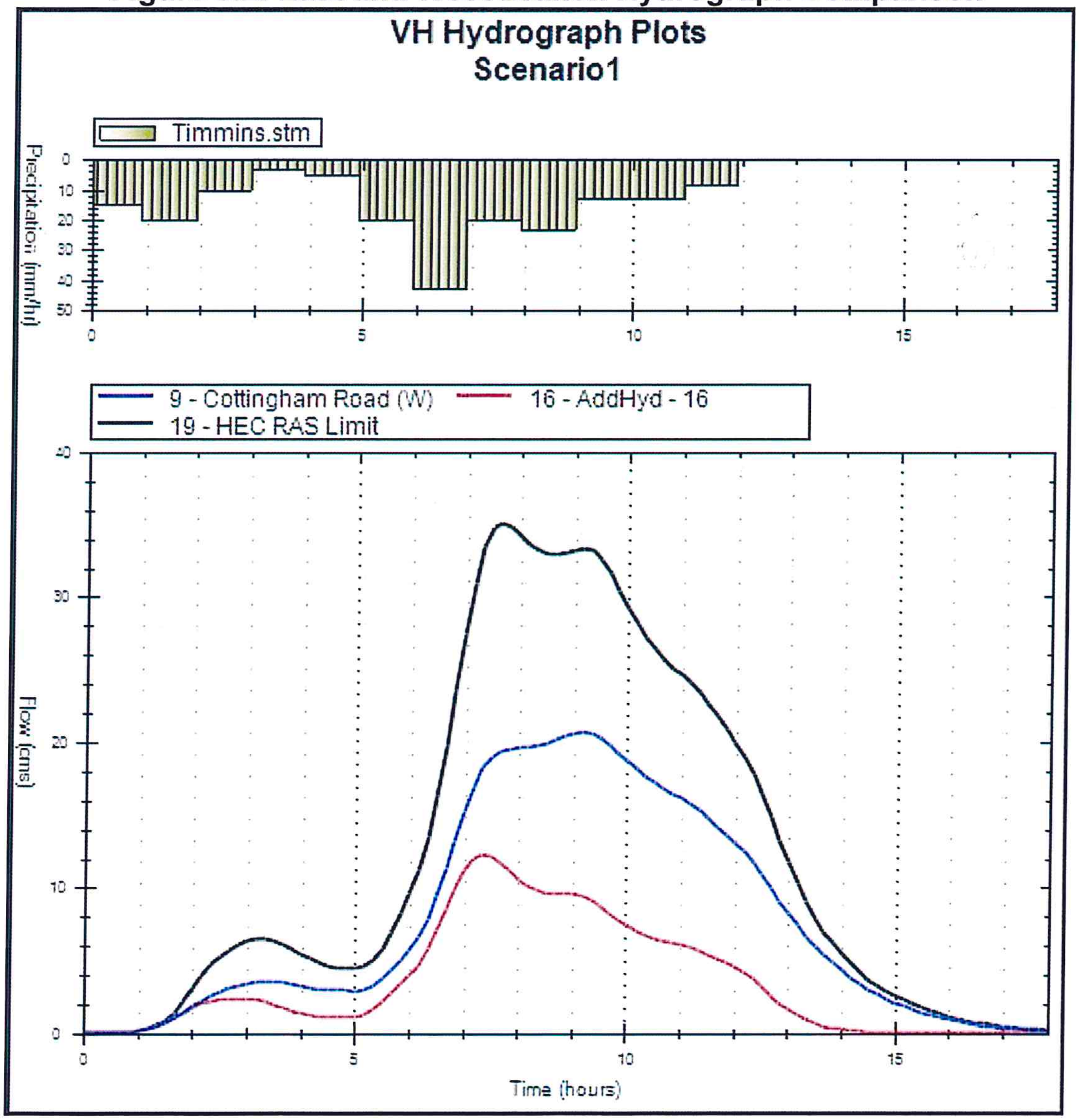
As can be seen in **Figure 5.1** below, catchments 100-700 have similar response patterns, with the flow peak occurring around 7 hours after the beginning of the Timmins event. Catchment 800, being so much larger, takes slightly longer to come to peak.

Figure 5.1: Catchment Runoff Comparison



As discussed in the section 1.3 and shown in **Figure 1.1** there are two branches to the creek; the junction is just east of Queen Street. The response time for each branch is similar, with peak flows taking place between 7 and 9 hours from the start of the Timmins storm. This is shown in **Figure 5.2**, with the east branch in red, and the west branch in blue. The resulting hydrograph at the junction, in black, shows how the flow in the main channel downstream of the junction is significantly higher than the peaks in each branch.

Figure 5.2: East and West Branch Hydrograph Comparison



Additional Storm Analyses

For the draft report, it was stated that the 6-hour SCS storm provided the highest peak flow for the 100-year event. The peer reviewer commented that the flow peaks increase as the storm durations decrease. The request was made 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 4-hour volumes. Using the revised a, b, and c parameters a 4-hour Chicago storm was run, and calculated a rainfall volume of 76.1mm. SCS and AES storm files were input into VH Suite for a 4-hour event. As seen in **Table 5.1** below, the 4-hour peak flows are less than the 6-hour peak flows at key nodes. Because of the, 6-hour flow peaks will be used for the 2-100 year events as the critical event.

Table 5.1: Comparing 4-hour and 6-hour Peak Flows

Node	100-year Peak Storm Flows in m^3/s			
	4hr Chicago	4hr AES	4hr SCS	6hr SCS
Cottingham Road (E)	2.51	2.43	2.58	3.03
Cottingham Road (W)	9.21	10.02	9.51	11.51
Creek Junction	18.20	18.86	18.84	22.45
Queen St	18.82	19.55	19.44	23.41
Pigeon River Dam Pond	18.98	19.69	19.67	23.56

Flow Output

Table 5.2 shows the representative peak flows to be input to the HEC-RAS model; the 2-100 year flows are derived from the 6-hour SCS storm. Details can be found in **Appendix F**.

Table 5.2: Input Flows to HEC-RAS

Node	Peak Storm Flows in m^3/s						
	2yr	5yr	10yr	25yr	50yr	100-year	Timmins
Cottingham Road (E)	0.54	1.05	1.46	2.03	2.49	3.03	4.17
Cottingham Road (W)	1.99	3.95	5.50	7.72	9.54	11.51	20.75
Creek Junction	3.84	7.63	10.65	14.96	18.58	22.45	35.17
Queen St	3.93	7.82	11.01	15.49	19.18	23.41	36.23
Pigeon River Dam Pond	3.96	7.94	11.09	15.64	19.42	23.56	36.78

6. Hydraulic Model Input Parameters

6.1. Cross Sections

Cross-section geometric data was extracted using HEC-GeoRAS 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 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 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 I**.

As per HEC-RAS requirements, all cross-sections are oriented looking downstream. The initial cross-section is at the outlet of the creek at the Pigeon River Dam pond; 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.

6.2. Culvert and Road Crossings

The Queen Street culvert is the only structure within the study area. Four cross-sections were cut at this culvert crossing to accurately represent channel flow: two upstream and two downstream bounding cross sections. Representative deck elevations were extracted from the Base DEM. The culvert was 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 and other relevant data and photographs are found in **Appendix J**.

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
Queen St	564	Concrete	Closed	Rect.	248.10	247.96	5.99	4.26	1.94

6.3. 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 value is 0.07 and the overbank n values range from .025 to .100. 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.

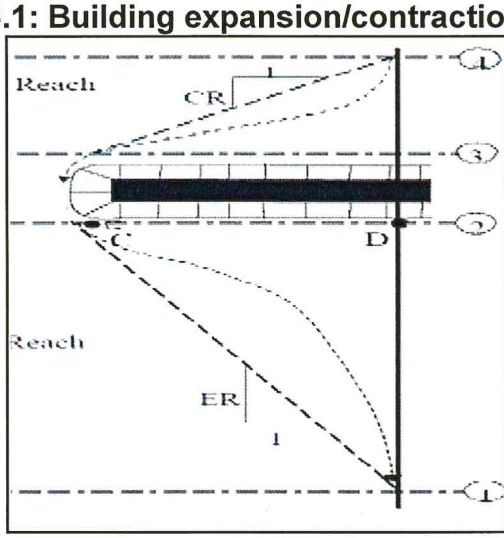
Table 6.2: Manning's n values in HEC-RAS model

River Station	Left Overbank		Channel	Right Overbank	
	1st	2nd		1st	2nd
971.145	0.080		0.070	0.080	0.025
861.008	0.080		0.070	0.080	
742.888	0.080	0.050	0.070	0.050	0.025
647.890	0.025	0.050	0.070	0.050	0.025
587.094	0.025	0.050	0.070	0.050	0.025
568.777	0.025	0.050	0.070	0.050	0.025
564 Queen St	Culvert				
562.450	0.050	0.050	0.070	0.050	0.025
550.758	0.025	0.050	0.070	0.050	0.025
425.004	0.025		0.070	0.050	
201.669	0.025		0.070	0.025	
99.412	0.100		0.070	0.025	

6.4. 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 6.1** below. Detailed calculations are found in **Appendix L**.

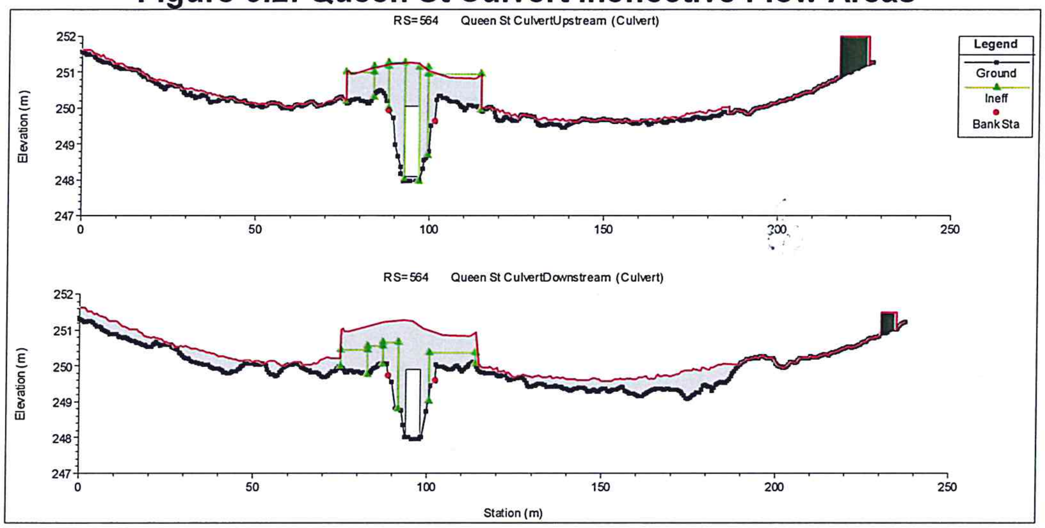
Figure 6.1: Building expansion/contraction effects



6.5. Ineffective Flow Elevations

Multiple ineffective flow areas were introduced at the Queen Street culvert crossing to capture the varying guard rail elevations, as shown in **Figure 6.2** below. For the upstream bounding cross-section, ineffective flow elevations are equal to the rail elevations. For the downstream bounding cross-section, the ineffective flow elevations are set at a point midway between the guard rail and the culvert obvert elevations.

Figure 6.2: Queen St Culvert Ineffective Flow Areas



6.6. Boundary Conditions

For the subcritical flow analysis, the downstream boundary condition is the normal headpond operating level of 248.2m, controlled by the MNR-operated Pigeon River dam.

6.7. Expansion/Contraction Coefficients

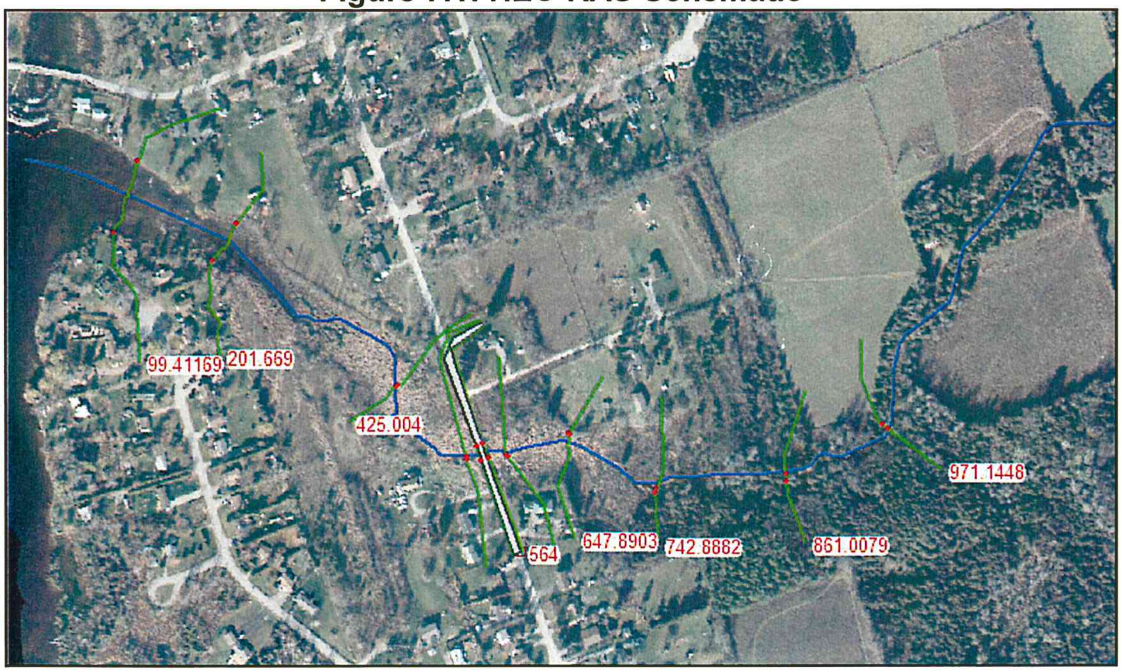
The model uses the HEC-RAS recommendations of 0.1 and 0.3 for contraction and expansion coefficients at all normal cross sections. At the Queen Street culvert, the values were increased to 0.6 and 0.8, respectively.

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 HEC RAS model was tested for sensitivity to the Manning's n and starting water surface elevation. **Appendix N** has the 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 the average increase in water surface elevation for the 11 cross-sections was 5cm, and the highest increase was 8cm.

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.

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 the average decrease in water surface elevation for the 11 cross-sections was 5cm, and the highest decrease was 12cm.

Downstream Boundary Condition
The creek flows into the pond upstream of the Pigeon River dam. The normal water operating level of 248.2m was used for the base model; sensitivity analyses were carried out by varying the starting water surface elevation.

The initial sensitivity test was carried out to determine the effect of lowering the headpond elevation by 0.2-0.7m (to 248.0m, 247.8m, and 247.5m). For most cross-sections in the creek the flood elevation was unchanged. Only the first three cross-sections' flood elevation differed. The first cross-section, 99.41169, had its elevation drop either to the revised starting water surface elevation or to the critical depth of 247.86m. The second cross-section, 201.669, had its elevation drop minimally (1cm – 5cm). The third cross-section, 201.669, had its elevation drop 1cm. The impact of lowering the starting water surface elevation is limited in scope since most of the cross-sections remain unchanged, and the program defaulted to the critical water surface elevation of 247.76m at the most downstream end.

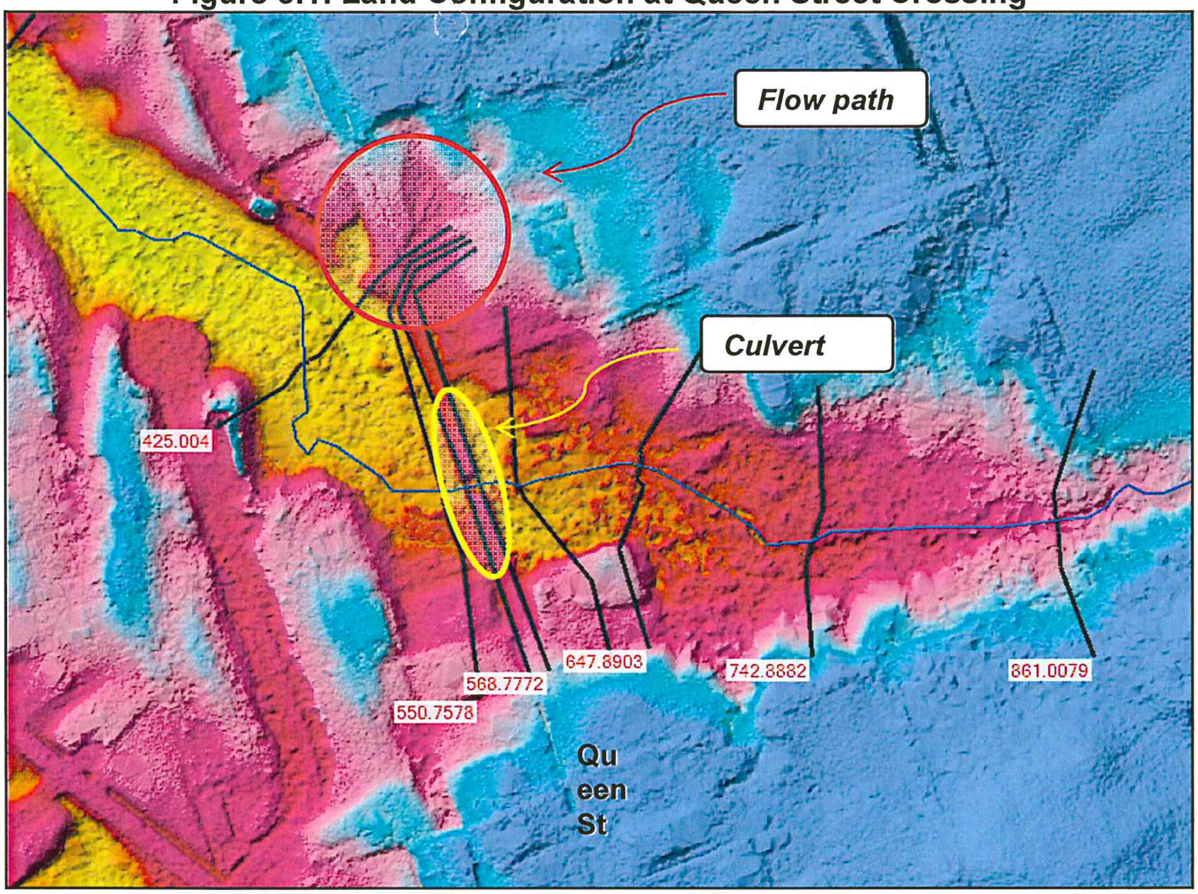
The second test of the sensitivity of the starting water surface elevation was carried out by increasing the elevation 0.2m-0.6m (to 248.4m, 248.6m, and 248.8m). For most cross-sections in the creek the flood elevation was unchanged. Only the first three cross-sections' flood elevation differed. The first cross-section, 99.41169, had its elevation rise equal to the revised starting water surface elevation. The second cross-section, 201.669, had its elevation rise range from 2cm to 23cm. The third cross-section, 425.004, had a 1cm-3cm drop in flood elevation. The impact of increasing the starting water surface elevation is significant for the initial two cross-sections, but not for the remaining nine cross-sections. However within this portion of the watercourse, the Regulatory flood line is set by the flood caused by the Pigeon River dam and not Omemee Creek riverine flooding.

8. Hydraulic Model results

8.1. Flow over Queen Street

The topography of Queen Street near the culvert appears that it could pose some flood water conveyance difficulty. A low point in the land is north of the culvert; as floodwaters rise on the upstream side of the culvert, water would flow north before continuing west to the Pigeon River Dam Pond. The topography is shown schematically in the coloured hillshading in **Figure 8.1** below. Dark blue represents a higher elevation, and yellow is the lower channel elevation. For this reason, the cross-sections were oriented to in a hockey-stick shape to capture the flow over the roadway north of the culvert.

Figure 8.1: Land Configuration at Queen Street Crossing



8.2. Creek Flood Results

The Regulatory flood elevations in the creek are listed in **Table 8.1** below, as well as the 2- through 100-year events.

Table 8.1: HEC-RAS Flood Elevations for Omemee Creek

Node	HEC-RAS Creek Flood Elevations (m)						
	Timmins	100yr	50yr	25yr	10yr	5yr	2yr
971.1448	252.81	252.58	252.54	252.39	255.52	252.24	252.05
861.0079	251.65	251.52	251.35	251.44	254.37	251.11	251.00
742.8882	250.50	250.33	250.35	250.19	253.29	250.15	250.00
647.8903	250.18	250.05	250.28	249.99	252.26	249.57	249.49
587.0937	250.15	250.03	250.27	249.98	251.29	249.49	249.22
568.7772	250.02	249.84	249.25	249.53	248.34	249.30	249.10
564 Queen St Culvert							
562.4503	249.58	249.41	249.37	249.32	249.25	249.19	249.07
550.7578	249.53	249.39	249.34	249.29	249.22	249.15	249.05
425.004	249.29	249.12	249.06	248.99	248.88	248.76	248.53
201.669	248.65	248.49	248.43	248.37	248.30	248.26	248.22
99.41169	248.20	248.20	248.20	248.20	248.20	248.20	248.20

Figure 8.2 shows the Regulatory flood extents for the creek based on a starting water surface elevation of 248.2m, the normal headpond operating level.

Figure 8.3 shows the profile of the creek and its riverine Regulatory flood elevation.

Figure 8.2: Regulatory Flood extents for the Creek

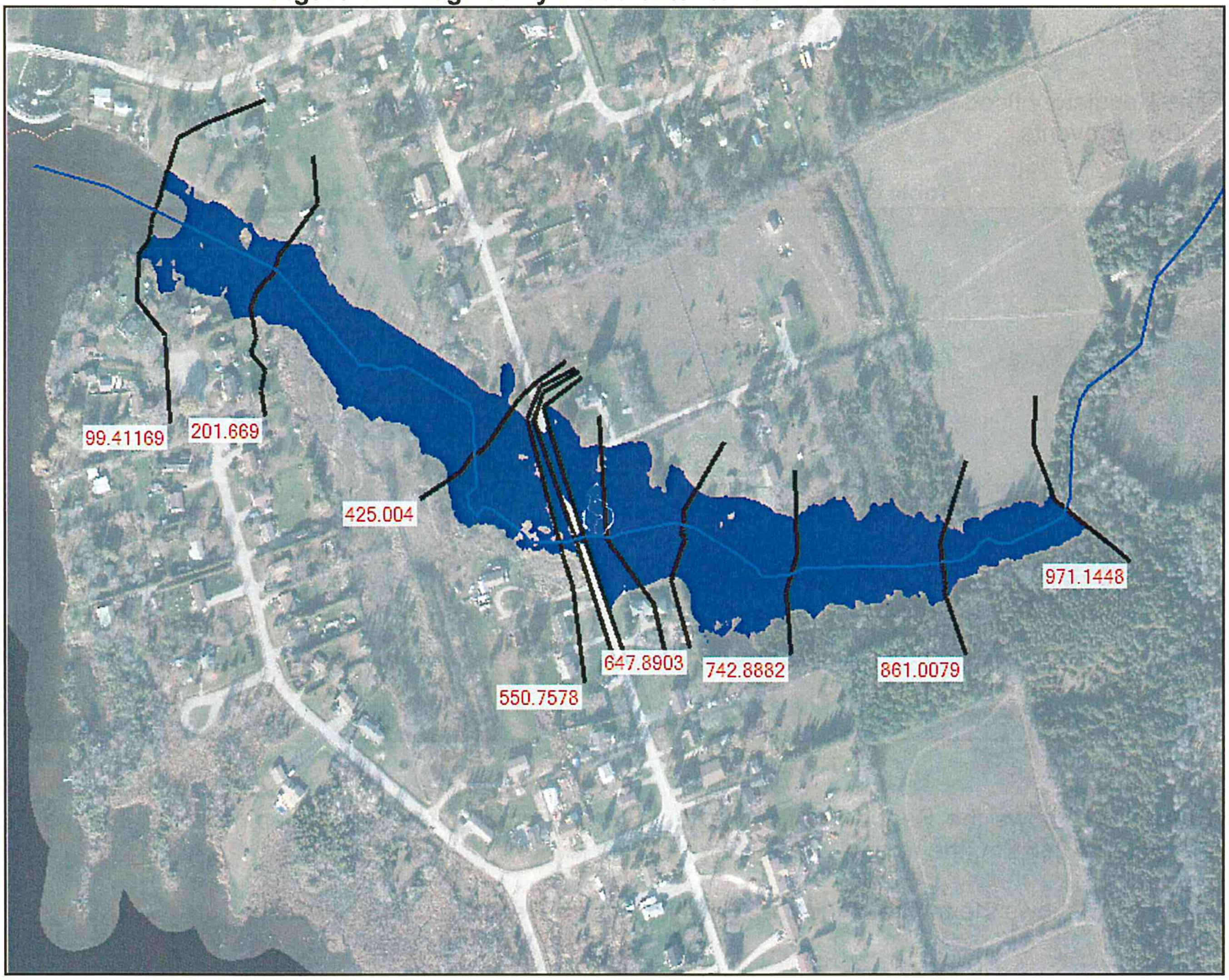
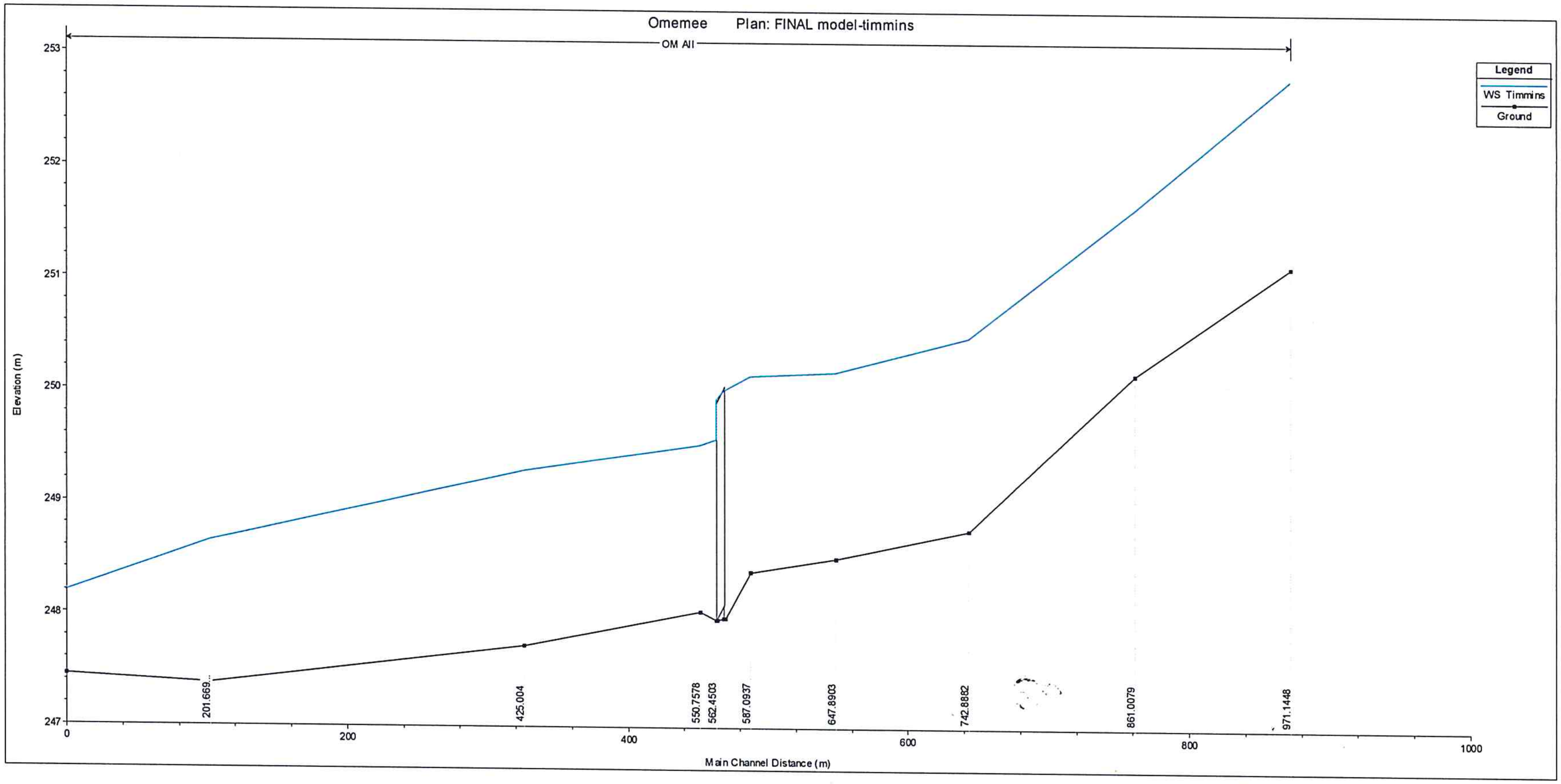


Figure 8.3 Regulatory Profile

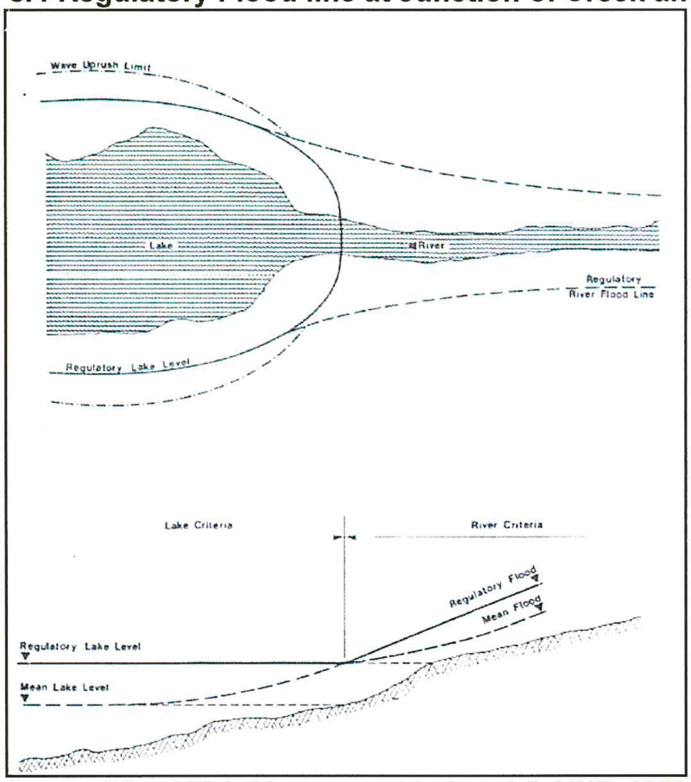


8.3. Flood Lines merged with Pigeon River Dam Pond Flooding

Omemee Creek flows into the pond upstream of the dam on the Pigeon River. Because the Pigeon River watershed is much larger than Omemee Creek's watershed, flooding on the Pigeon River is caused by a different storm event than what would cause flooding in Omemee Creek. According to the Ministry of Natural Resources and Forests (MNRF) 2002 document *Technical Guide, River and Stream Systems: Flooding Hazard Limit*, in a situation where a creek flows into a lake and where the high water levels are generated by two independent flood events, the Regulatory flood line should be based on the higher of:

I. The mean annual flood level in the creek and the flood hazard limit in the Pigeon River Dam Pond
II. The flood hazard limit in the creek and the mean monthly levels in the Pigeon River Dam Pond

Figure 8.4 Regulatory Flood line at Junction of Creek and Lake



Source: "Flood Plain Management in Ontario", MNR, 1988.

To establish the Regulatory flood line, two separate flood lines were merged: the 250.4m dam flood line and the Timmins creek flooding elevations. As can be seen in **Figure 8.5**, **Figure 8.6**, **Figure 8.7**, **Figure 8.8**, and **Figure 8.9** the headpond flooding elevation of 250.4m established by the 1992 OMB decision determines the flood elevation for the bulk of the study area, up to cross-section 742. The flood elevation of 250.4m is the flood elevation within the Omemee Secondary Plan area.

Final flood maps are at the end of the report.

Figure 8.5 Pigeon River Dam Pond Flood Extents

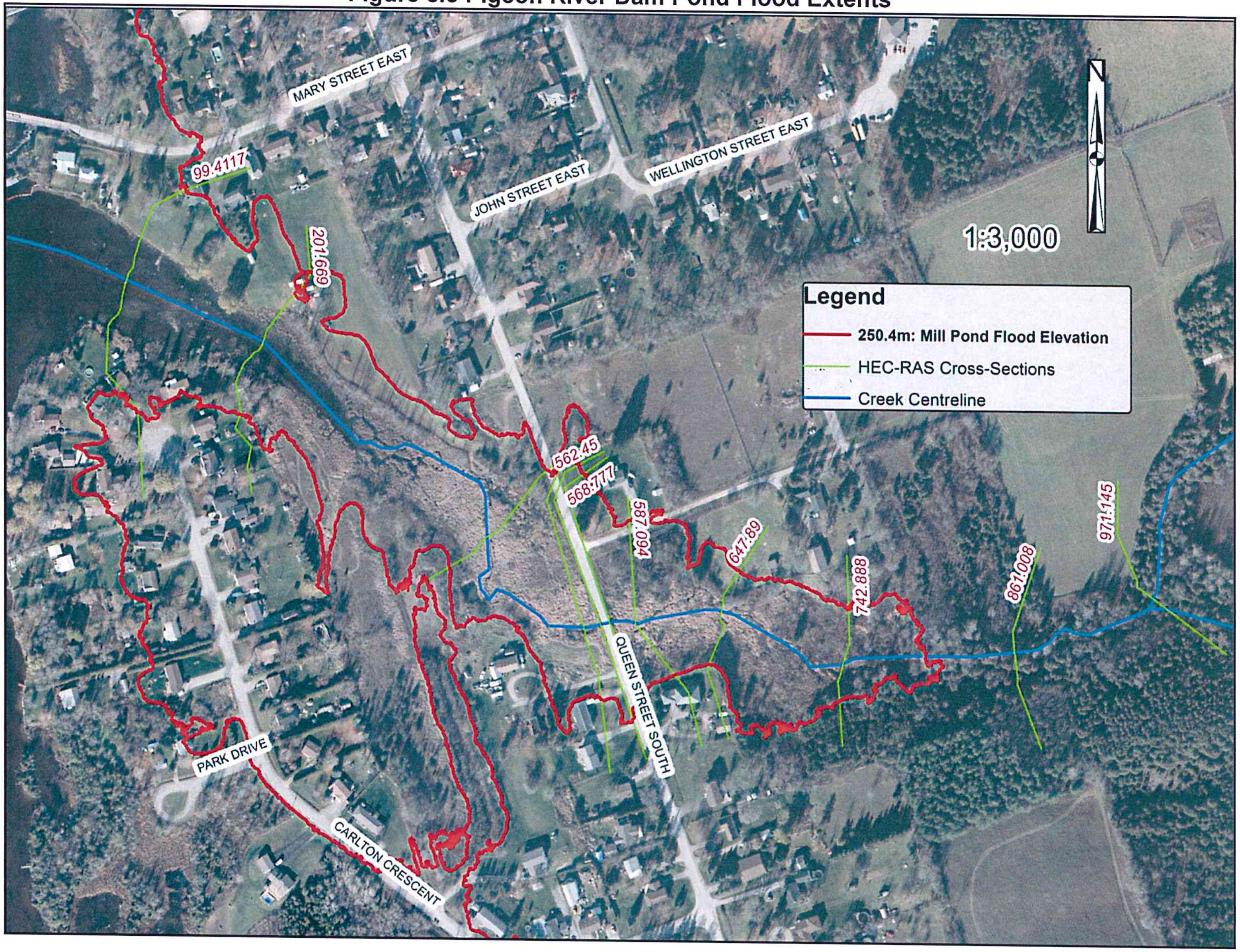


Figure 8.6 Creek Flood Extents

Figure 8.7 Merged Flooding Extents

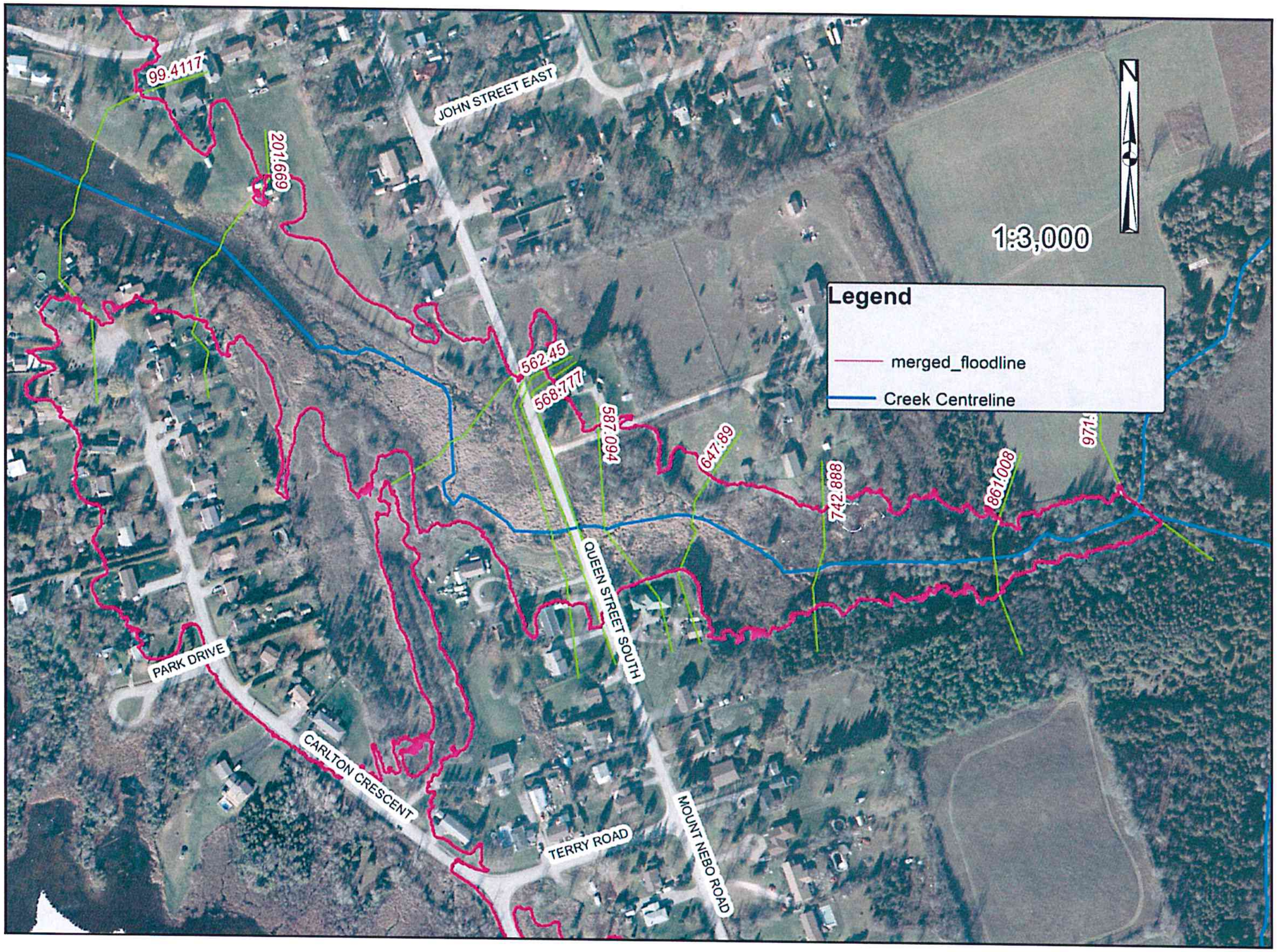


Figure 8.8 Comparing Pigeon River Dam Pond, Creek, and Regulatory Flooding Extents

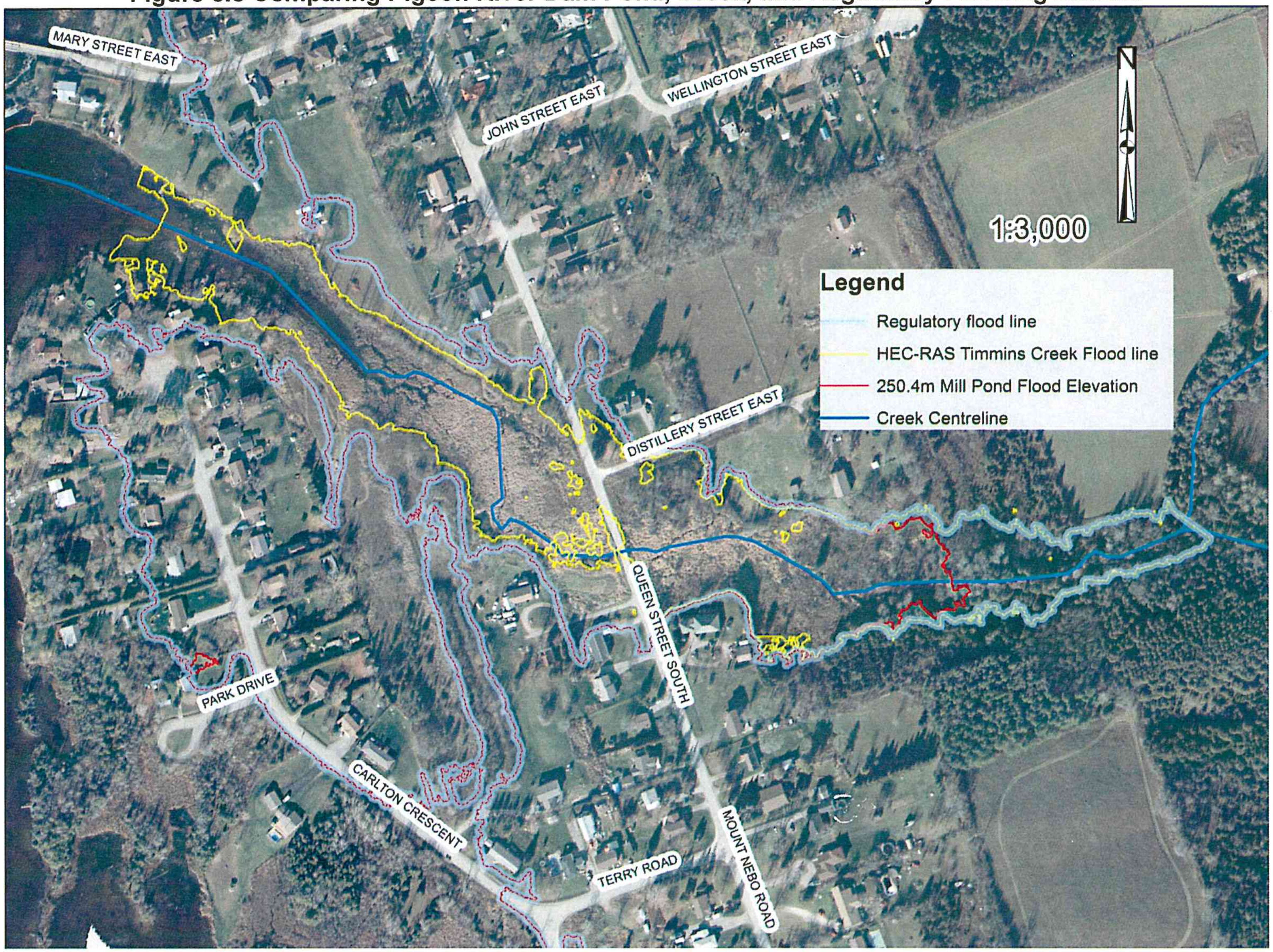
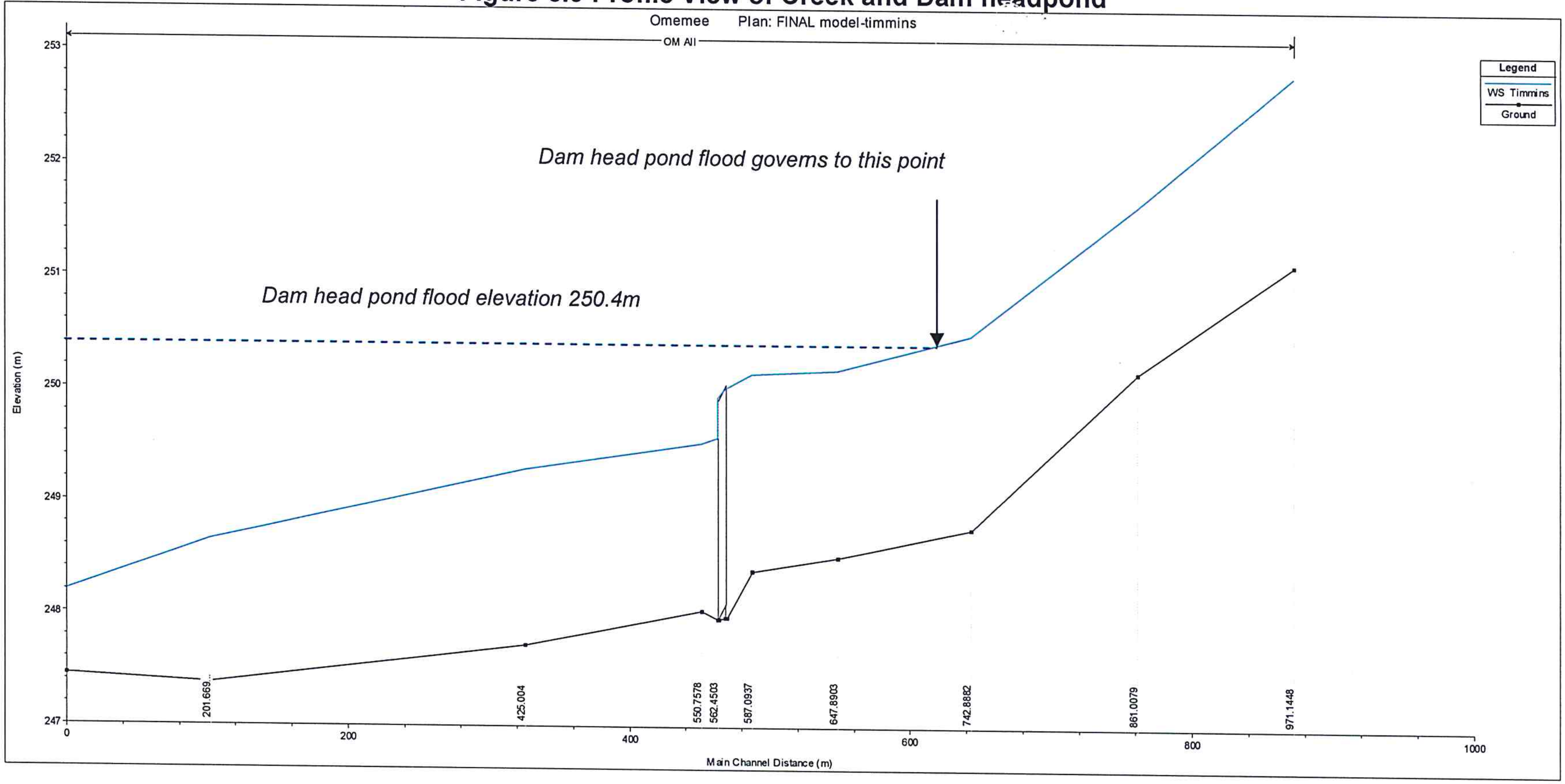


Figure 8.9 Profile View of Creek and Dam headpond



The Regulatory flood elevations for the study area are shown below in **Table 8.2** below.

Table 8.2: Flood Elevations for Omemee Creek

Node	Flood Elevations (m)							
	Regulatory	Timmins	100yr	50yr	25yr	10yr	5yr	2yr
971.1448	252.81	252.81	252.58	252.54	252.39	255.52	252.24	252.05
861.0079	251.65	251.65	251.52	251.35	251.44	254.37	251.11	251.00
742.8882	250.50	250.50	250.33	250.35	250.19	253.29	250.15	250.00
647.8903	250.40	250.18	250.05	250.28	249.99	252.26	249.57	249.49
587.0937	250.40	250.15	250.03	250.27	249.98	251.29	249.49	249.22
568.7772	250.40	250.02	249.84	249.25	249.53	248.34	249.30	249.10
564 Queen St Culvert								
562.4503	250.40	249.58	249.41	249.37	249.32	249.25	249.19	249.07
550.7578	250.40	249.53	249.39	249.34	249.29	249.22	249.15	249.05
425.004	250.40	249.29	249.12	249.06	248.99	248.88	248.76	248.53
201.669	250.40	248.65	248.49	248.43	248.37	248.30	248.26	248.22
99.41169	250.40	248.20	248.20	248.20	248.20	248.20	248.20	248.20

9. Conclusions and Recommendations

It is recommended that the results of the HEC-RAS model for Omemee Creek watercourse be used for generating the flood maps. The flood maps are found 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.

LICENSED PROFESSIONAL ENGINEER
C. J. PEACOCK
Nov 28, 2016
PROVINCE OF ONTARIO

10. 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: Hydrology Model Flow Summary
Appendix G: Hydrology Model Sensitivity Analyses
Appendix H: Official & Secondary Plan Maps
Appendix I: Cross-section Photo Inventory
Appendix J: Structure Photo Inventory Record
Appendix K: Manning's n Values
Appendix L: Cross-section Obstruction Calculations
Appendix M: HEC-RAS Output
Appendix N: HEC RAS Sensitivity Analyses
Appendix O: List of Model Files
Appendix P: Response to Peer Review Comments
Appendix Q: Digital Elevation Model and Orthoimagery Data Accuracy Assessment Report
Appendix R: Terms of Reference
Appendix S: Terms of Reference for Digital Elevation and Orthoimagery Data Quality Control