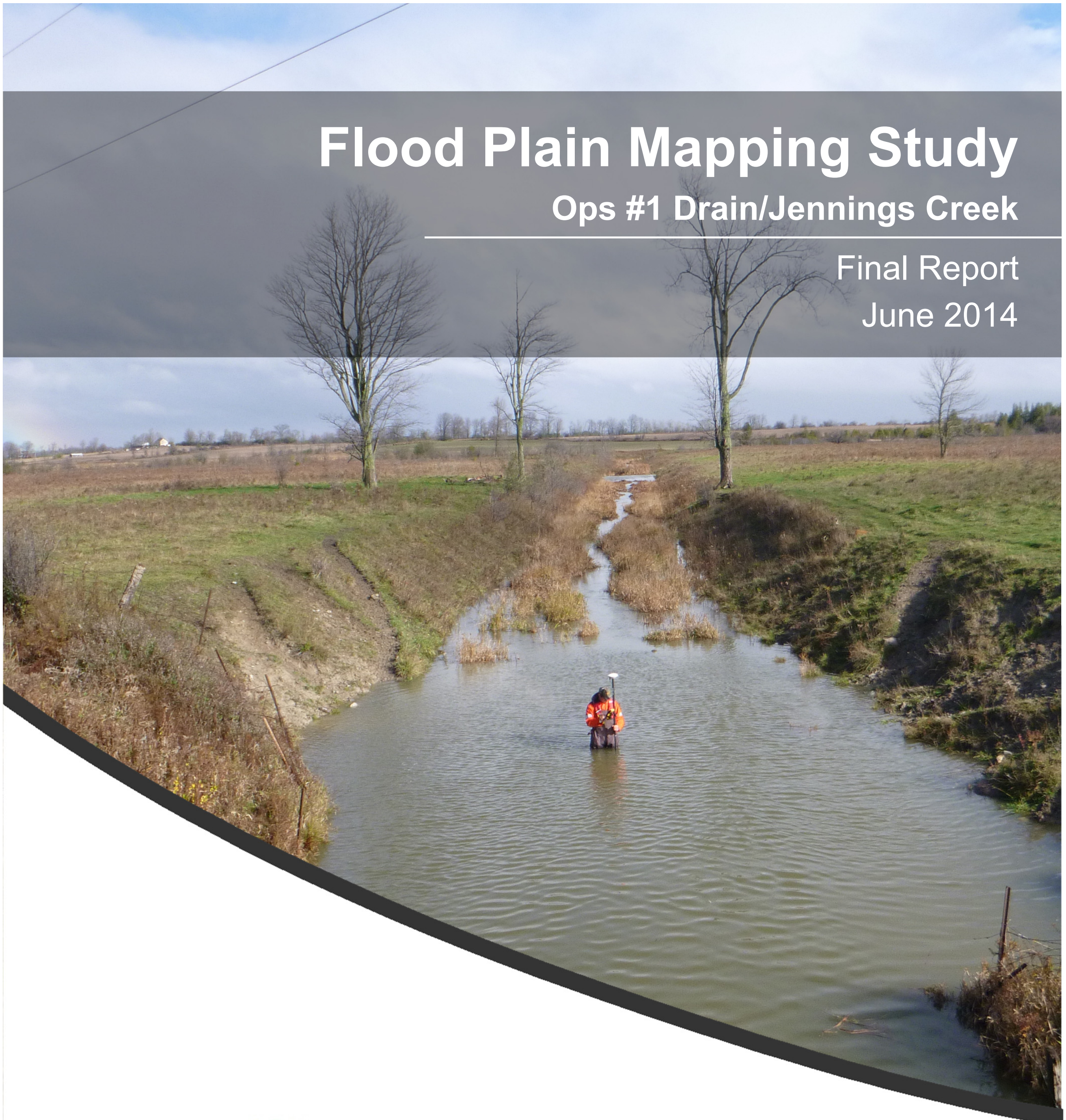
Flood Plain Mapping Study

Ops #1 Drain/Jennings Creek

Final Report

June 2014



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About Kawartha Conservation

A plentiful supply of clean water is a key component of our natural infrastructure. Our surface and groundwater resources supply our drinking water, maintain property values, sustain an agricultural industry and support tourism.

Kawartha Conservation is the local environmental agency through which we can protect our water and other natural resources. Our mandate is to ensure the conservation, restoration and responsible management of water, land and natural habitats through programs and services that balance human, environmental and economic needs.

We are a non-profit environmental organization, established in 1979 under the Ontario *Conservation Authorities Act* (1946). We are governed by the six municipalities that overlap the natural boundaries of our watershed and voted to form the Kawartha Region Conservation Authority. These municipalities include the City of Kawartha Lakes, Township of Scugog (Region of Durham), Township of Brock (Region of Durham), the Municipality of Clarington (Region of Durham), Cavan Monaghan, and the Municipality of Trent Lakes.

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Executive Summary

Since late 2001, a number of floodplain studies have been undertaken by the City of Kawartha Lakes for Ops # 1 Drain/Jennings Creek. However, each of the studies encountered mapping and technical issues and none of them have been signed off by a professional engineer. In 2010, a study commissioned by the City, prepared by Greck and Associates “Ops 1 Drain Flood Hazard Management Guidelines” recommended that a comprehensive study be completed to address the previous modeling issues. It further recommended a number of flood hazard reduction measures and suggested applying a two zone flood management policy in different portions of the watershed.

The primary goal of this study is to create hydrologic and hydraulic models of the watershed, refine and build upon existing recommendations, and produce flood plain mapping for the Ops #1 Drain and Jennings Creek. The mapping will allow both the City of Kawartha Lakes (CKL) and Kawartha Conservation staff to make informed decisions about future land use and identify flood hazard reduction opportunities.

The second goal of this study is to explore previous results and recommendations and confirm or establish revised recommendations using the comprehensive baseline produced.

The Ops #1 Drain/Jennings Creek Flood Plain Mapping Study has been subject to a comprehensive peer review process for core components of the study including data collection and processing standards, hydrologic and hydraulic modeling methods and parameters and study results. The process was supported throughout by a Technical Committee comprised of technical/managerial staff from Ganaraska Conservation, the City of Kawartha Lakes, and the Ministry of Transportation (as necessary) in addition to Kawartha Conservation staff.

Topics discussed in this study include:

- Previous work completed
- Use of static and dynamic hydraulic modeling for the study area
- Future developments and the flood plain
- Capital infrastructure and its impacts on flood elevations
- Applicability of the One-Zone and Two-zone policy concepts to the study area
- Future recommendations

Key findings of this study include:

- The historical rainfall data that had been collected at the Lindsay filtration plant by the now-defunct AES gauge were determined to be the most suitable data for use. Local comparisons and a sensitivity analysis were performed to determine if the truncated filtration plant data was still valid. Results showed no significant difference so the Lindsay data was used with no aerial reduction.
- Use of the future land use condition peak flows for input flows to the static hydraulic HEC-RAS model.
- Use of the future land use conditions catchment hydrographs from PCSWMM model as input to the dynamic hydraulic HEC-RAS model.
- Unsteady flow using a dynamic wave is the preferred hydrology modelling approach
- Regulatory flood plain mapping will be based on the output of the static hydraulic HEC-RAS model for future land use conditions.
- For both the existing and future land use conditions, the upstream urban portion of the watershed experiences higher peak flows from the 100-year Chicago storm. Downstream of Highway 7 however, the Timmins storm provides higher peaks.
- No aerial reductions were used

The following is a list of technical recommendations prepared from the work completed by this study:

- Dynamic hydraulic model should not be used for the Regulatory Flood Plain Mapping and it is recommended that the static water surface elevation for future land use conditions be used for flood plain mapping. Reasons to support static hydraulic modelling include:
    - o Provincial guidance supports static modeling only;
    - o There is no guarantee that a roadway would remain in place during a flood event. The road and/or culvert could wash out and the downstream flows would not be attenuated; and
    - o Any future culvert and/or road improvement would increase the downstream flood plain.
- The Lindsay Commercial Area should not be included in the Regulatory flood plain for the following reasons:
    - o Static model flood volumes are overestimate actual runoff volumes

- o Most of the area does not fall within the Ministry of Natural Resources' (MNR's) recommended 125 Hectare upstream drainage area cut-off limit for flood plain mapping
  - o The intensely-developed commercial and industrial land use is in sharp contrast to the rest of the Ops #1 catchment area land use.
  - o Roadside ditches form a large portion of the drainage system within this area. The flooding problems associated with this area are therefore urban flooding, not riverine flooding.
  - o The 100-year Chicago storm causes the most flooding; this is sharp contrast to the rest of the Ops #1 drainage basin which floods more severely with the Timmins storm.

- The Lindsay Commercial are should still be regulated to address the existing flooding hazard based on the results of the dynamic hydraulic HEC-RAS model output.
- Capital infrastructure improvements can help alleviate the flood hazard throughout the watershed. A comprehensive plan is recommended to coordinate these improvements.

The following policy recommendations are also made based on this study:

- Within the Regulatory flood plain of the Ops#1 Drain and Jennings Creek it is recommended that the one-zone policy concept be applied. There is preliminary analysis in this study to indicate that areas of the flood plain may be suitable for application of the two-zone concept however further analysis is required to demonstrate how flood fringe development would proceed and that there is no significant upstream or downstream effect and no new hazards are created.
- It is further recommended that Kawartha Conservation and the City of Kawartha Lakes, in coordination with stakeholders, create a two-zone policy in order to allow proponents to demonstrate the effects of their developments using the baseline modeling prepared in this study.
- For the Lindsay commercial area, future development in this area must recognize flood hazards caused by restrictive capital infrastructure. Future development should be controlled using a development policy based on the results of the Timmins dynamic model and appropriate for this urban drainage flooding hazard.

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1 Introduction

1.1 Objective

The Ops#1 Drain/Jennings Creek flood plain study is being conducted to assist the City of Kawartha Lakes in generating accurate and defensible hydraulic and hydrological models. This is the first flood plain study in a multi-year flood line mapping update project undertaken by Kawartha Conservation and the City of Kawartha Lakes. The objective of the overall study is to generate updated flood plain mapping for the Ops #1 Drain/Jennings Creek. The mapping will allow both the City of Kawartha Lakes (CKL) and Kawartha Conservation staff to make informed decisions about future land use and identify flood hazard reduction opportunities.

The Ops #1 Drain/Jennings Creek drains an area of significant growth within the Community of Lindsay. The results of the hydrology modeling work will provide design storm flows for the 2- through 100-year return periods as well as the Timmins Storm, to be used as input to a hydraulic model which will establish Regulatory flood lines within Community of Lindsay. The study area is shown in **Figure 1.1**.

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 (QA) and quality control (QC) standards to be applied to all projects in the multi-year initiative. The QA methodology for each component ensures a two-fold benefit: that the project design meets industry standards, and that he work outline and planned deliverables are valid. The three goals of the QC component are: that the product is consistent with standards and generally accepted approaches; that the study results meets Technical Committee's requirements; and that the products and results are scientifically defensible. Each methodology was peer-reviewed for QA and QC by an external firm or agency. Four separate components of the project were established for QA and QC:

- mapping and air photo
- survey data collection and integration
- hydrology modeling
- hydraulic modeling

For the mapping and air photo portion of the project QA, 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. The GIS staff from Ganaraska Conservation peer-reviewed the RFP and survey purchase/procedure and confirmed they met industry standards. For the QC portion, Ganaraska Conservation's GIS staff peer-reviewed the LIDAR data and confirmed the data meets the Province of Ontario's 2009 "Imagery and Elevation Acquisition Guidelines". The survey data was also peer-reviewed by GIS staff from Ganaraska Conservation and it was confirmed it meets standards.

For the QA 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. Both the hydrology model and report and the hydraulic model and report were peer-reviewed for QC purposes by the water resources engineer for Quinte Conservation. The models and report were found to be satisfactory.

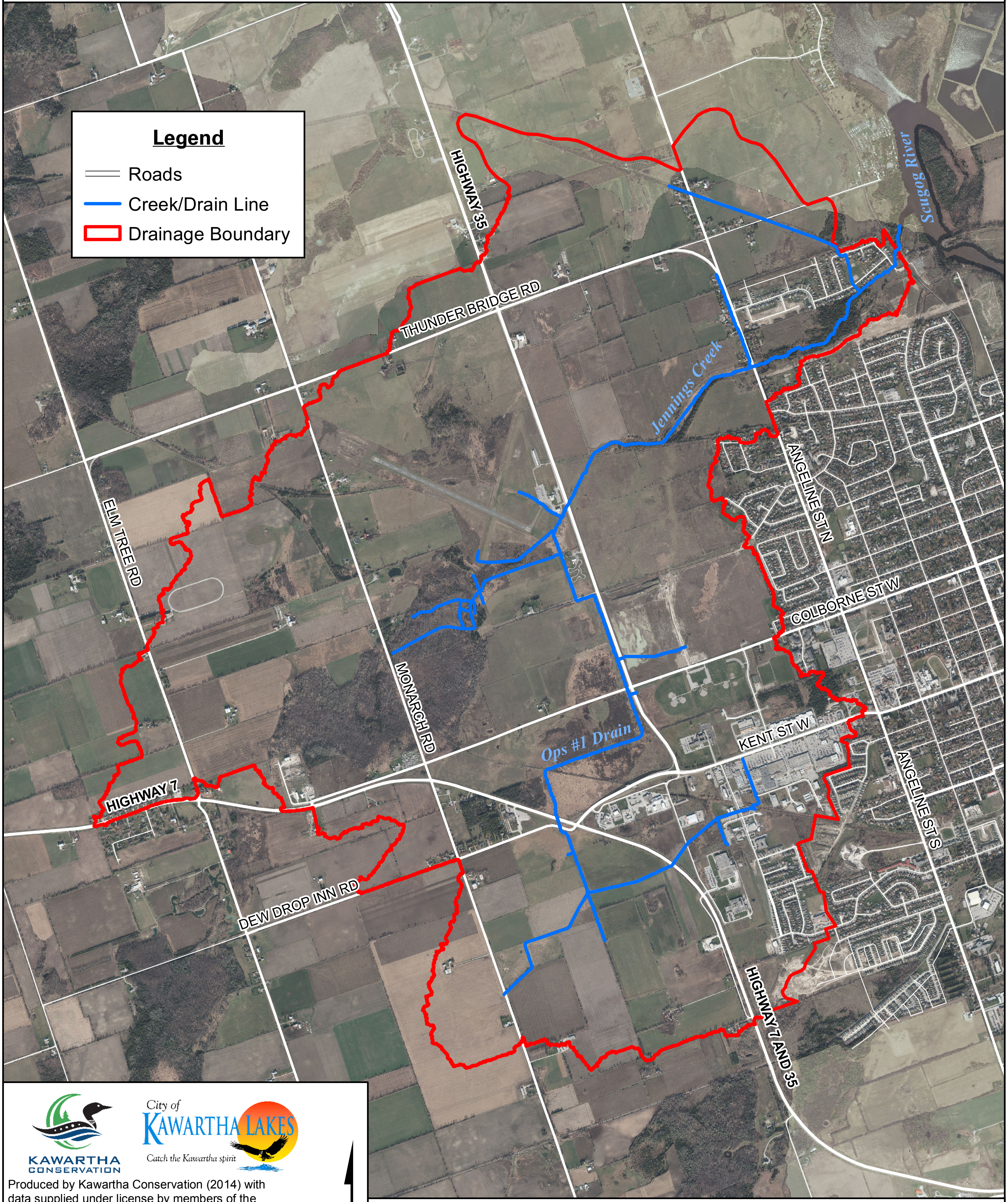
1.3 Watercourse Context and Description

Urban drainage from Lindsay Square and other lands adjacent to Kent Street drain to a series of roadside ditches. From Commerce Road the roadside ditches drain to a man-made channel, becoming the Ops #1 Drain just east of McLaughlin Road. Crossing McLaughlin Road, it picks up drainage from both commercial properties and residential development. From here the watercourse continues west to Highway 35 in an east-west channel within agricultural lands. The drain turns north, running parallel to Hwy 35 before it re-crosses Highway 35 moving in an easterly direction. It becomes Jennings Creek just upstream of Angeline Street and flows within part of the older built area of the community of Lindsay. The Ops #1 Drain/Jennings Creek discharges to the Scugog River and eventually to Sturgeon Lake. Please refer to **Figure 1.1**.

As a result of the amalgamation of Ops Township and the Town of Lindsay, the City of Kawartha Lakes has municipal jurisdiction over the entire drain and its watershed. Originally the drain was constructed to improve the drainage of agricultural lands by serving as the discharge point for agricultural tile drainage systems and local surface drainage systems. In addition, it removes excess urban storm water collected by roadside ditches, residential lots, industrial lands, commercial lands and any other properties along its path.

The majority of the watershed west of the Community of Lindsay is rural farmland, wetlands, and individual rural homes/lots. The watershed has a size of 1675 hectares. The Ops #1 Drain/Jennings Creek main channel is about 7.6 km long. Ops #1 Drain is extremely flat, with an average slope of 0.1%. Jennings Creek is steeper, with a 1% slope.

Figure 1.1 : Ops #1 Drain and Jennings Creek Study Area



Produced by Kawartha Conservation (2014) with data supplied under license by members of the Ontario Data Exchange, Ontario Ministry of Natural Resources.

1.4 Background Information

The Ops #1 Drain and Jennings Creek are a vital component of the local infrastructure and are facing pressure from continued growth and future urban expansion. Both watercourses are subject to flooding due to urbanization and its associated land use changes. Existing flooding concerns may be amplified by future growth and need to be addressed in order to manage flood water flow within the drainage area. In particular flooding has been experienced at Commerce Road close to the south Entrance of the Lindsay Square Shopping Mall, Highway 7/35, McLaughlin Road, and west of Highway 35 adjacent to the airport and at Jennings Creek in the vicinity of the Victoria Recreation Transportation Corridor. Flooding issues appear to result from increased runoff due to change of land use from agricultural to mixed residential and commercial uses. Because of increased development pressures in the upstream area, numerous studies have been carried out in the past to attempt to understand and reduce future flooding. Relevant excerpts from key reports are included in **Appendix A**.

The engineering firm Aquafor Beech was retained by the City of Kawartha Lakes to carry out the *Ops# 1 Drain Functional Storm Water Management Study*, a draft copy of which was written in November 2001. The firm developed flood plain maps for the Ops #1 Drain catchment upstream of Hwy 35. The consulting company utilized a steady state modeling approach in HEC-RAS to assess the capacity of the drain. Since input of hydrographs into HEC-RAS was not an option at the time of the study, Aquafor Beech recommended an unsteady model approach be undertaken to refine flows to obtain more accurate peak flow rates and water surface elevations. This recommendation was based on the concern that the storage characteristics of this very flat watershed were not being appropriately considered. Relevant excerpts are found in **Appendix A.1**.

Following this work, the engineering consulting firm AECOM, formerly Totten Sims Hubicki, extended the Aquafor Beech study downstream to include Jennings Creek. Their draft report, *Ops #1 Drain Flood plain Mapping Update*, was produced in April 2010. This firm was retained by the City of Kawartha Lakes to identify and investigate stormwater management opportunities in both the upstream developed areas and downstream underdeveloped areas. Additionally they were to assess existing flooding issues and flooding impacts of anticipated future development. They replaced the VO_2/HEC-RAS simulation with an unsteady model employing the EXTRAN module of SWMM 4.4 to assess the storage characteristics of the flood plain. While their results differed from the steady flow assessment carried out by Aquafor Beech, the changes and their impacts on flood plain extents were not significant. This was attributed to the fact that the underlying mapping products used for both studies were not refined enough to translate modeled elevations and cross-section survey data used in the analysis into an accurate representation of the flood plain. Relevant excerpts are found in **Appendix A.2**.

An additional study by Greck and Associates, *Ops 1 Drain Flood Hazard Management Guidelines*, was finalized in July 2011. The study reviewed AECOM's flood lines to develop guidelines to address flood hazards. Six separate management areas were recommended, and specific flood hazard reduction measures suggested for each. In addition, the study re-examined the dynamics of flooding using an unsteady flood flow hydraulic analysis (XP

STORM program), which is similar to the EXTRAN module. This analysis confirmed that flood elevations derived by usual standard methods (such as steady-state flow analysis) may not sufficiently capture the complex hydrology and hydraulics found within the Ops #1 Drain/Jennings Creek. The study also concluded that the mapping information was not detailed enough to portray the routing characteristics of the flood plain. Relevant excerpts are found in **Appendix A.3**, but listed below are the main recommendations that pertain to this current study:

1. *Prepare suitable regulatory flood line mapping stamped by a professional engineer for the entire Ops#1 Drain and Jennings Creek watershed. This will require significant refinements to the latest hydrologic and hydraulic models developed for the entire watershed… A number of improvements are particularly required in the routing of flood flows and the hydraulic modeling. The hydraulic model should be georeferenced for integration with other municipal planning documents and for day to day operations and review of development applications and approvals. The completion of this work is important for the planning and implementation of policy development and related flood hazard management works in the watershed.*
3. *The Kawartha Conservation together with the City of Kawartha Lakes should develop specific policy related to the use of a two zone policy in designated areas. To develop this policy a two zone study is required in management Areas 4, 5, and 6. Specifically this study should include but not be limited to defining the floodway and flood fringe areas, roadways with ingress and egress constraints, allowable areas of encroachment in the flood fringe, and requirements for flood proofing. This work could be completed as part of Recommendation 1 or completed subsequent to the completion of Recommendation 1. Due to the implications to current and future land uses and possible impacts to private and public lands this should be completed following the spirit of Conservation Authority's Class Environment Assessment process.*

Another study used for reference information is the March 2004 Totten Sims Hubicki Associates' *City of Kawartha Lakes Community of Lindsay Storm Sewer Servicing Study.* The firm was hired to analyze the minor storm system within Lindsay town limits to determine areas of surplus capacity and to determine capital project upgrades. The chief item used from this project is minor drainage area mapping. Relevant excerpts are found in **Appendix A.4**.

Greck and Associates produced another report in May 2012, titled *Jennings Creek Flood Plain Mapping Study* for the Woods of Jennings Creek subdivision, located mainly on the south side of Jennings Creek between Angeline St North and William St North. This study used two new topographical surveys to create the cross-sections in a new geometry file; the AECOM hydrology flows were used as input. The report concluded that parts of the proposed subdivision would encroach into the 5-year to Regulatory flood plain, and suggested various flood relief options. Relevant excerpts can be found in **Appendix A.5**.

1.5 **Modeling Approach**

As noted above, in past studies, the dynamics of flooding have been assessed using standard steady flow hydrologic methods as input to steady-state HEC-RAS models. However, this approach does not account for attenuation and backwater effects from undersized culverts and relatively large shallow flood storage areas within the watershed.

Thus the modeling of unsteady flow using a dynamic wave (instead of kinematic) is the preferred method for Ops #1 Drain/Jennings Creek. This approach will ensure that the discharge will vary in space and attenuate as it moves downstream to account for the time and volume dynamics of complex flood plain storage, to reflect the system-wide storage resulting from vast shallow overbank flows. Both steady-state and dynamic hydraulic analyses will be carried out for this study.

The hydrologic modelling was carried out using PCSWMM. This software implements a dynamic wave routine similar to EXTRAN and is therefore suitable to route flow through the Ops #1 Drain/Jennings Creek.

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

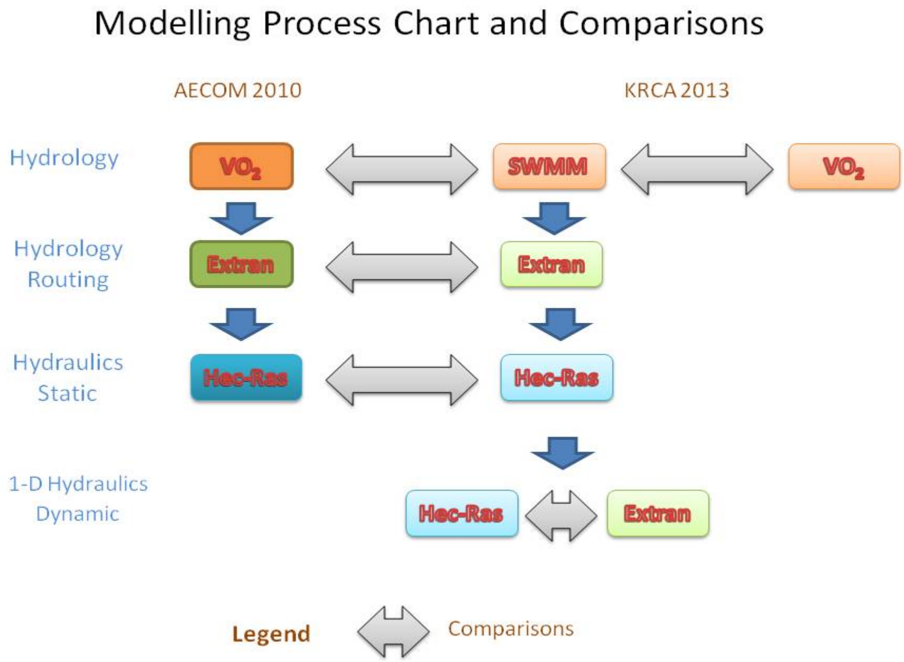
Individual catchments have been refined reviewing stormwater management and engineering reports and drawings for the Ops #1 Drain and Jennings Creek where applicable.

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

Sensitivity analyses have been carried out to determine the impact of changing model parameters on the calculated flows.

It is expected that taking such an approach would result in the establishment of more realistic peak flows and associated flood lines along the Ops #1 Drain/Jennings Creek. In addition to the work noted above, comparisons of all results were undertaken to evaluate the change in floodplain elevations and extents. The chart below shows how results have been compared.

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.*



2 Rainfall

2.1 Rainfall Data

Rainfall Intensity–Duration–Frequency (IDF) values and curves are used to define the amount of rainfall that will be input into a model. IDF values provide estimates of the extreme rainfall intensity for any given duration corresponding to different return periods. Rainfall volumes are taken from Lindsay's Atmospheric Environment Services (AES) gauge which was removed from service in 1989. Other rainfall stations, such as Peterborough (AES) and Ontario Ministry of Transportation (MTO) were considered, however earlier reports utilized the Lindsay station precipitation data. It was decided to carry on with the use of the Lindsay station as the values for the Lindsay station are similar to other local station's values. Additionally, use of the Lindsay Filtration Plant values provides for continuity as much of the infrastructure in the community has been designed using this curve. Finally, it was felt that this gauge proved the most representative data for the study area.

The Ontario Ministry of Natural Resources (MNR) technical manuals provide a rainfall reduction table for the Timmins storm. For drainage areas larger than 25 km^2, an aerial reduction is applied to the Timmins point rainfall based on 24 hr isohyets as shown in Table D-5 of the MNR manual. Given the size of the catchment no areal reduction factors were used.

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

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

I in mm/hr

t in minutes

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

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

Return Period (yr)	A	B	C
2	628.11	5.273	0.780
5	820.23	6.011	0.768
10	915.85	6.006	0.757
25	1041.80	6.023	0.748
50	1139.70	6.023	0.743
100	1230.80	6.023	0.738

Through the course of this study it was discovered that when the a, b, and c parameters listed above were input into the hydrology models, the corresponding total rainfall volumes generated for a 12-hour storm overestimated the measured AES volumes by as much as 25%. As a result, Kawartha Conservation staff re-created the a, b, and c parameters which are listed below in **Table 2.2**; these values provided rainfall depths within 1% of measured volumes. These are the values used for the base hydrology scenarios. Further discussion of the differences can be found in **Appendix C**.

Table 2.2: IDF Parameters calculated by Kawartha Conservation

Return Period (yr)	A	B	C
2	808.3	7.413	0.835
5	1248.1	9.760	0.857
10	1486.8	10.440	0.859
25	1917.8	11.842	0.873
50	2142.0	12.182	0.872
100	2465.5	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

As outlined in **Appendix S**, the peer review of the September draft hydrology report included a rainfall volume analysis of the Lindsay and Peterborough AES gauges. The review concluded that the Peterborough gauge captured an increase in rainfall volumes that was not reflected in the Lindsay data. It is unclear 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 a result, several rainfall sensitivity analyses were carried out to see the effect on peak flows and associated flood elevations: total Lindsay gauge rainfall volumes adjusted by $^{+}$/-10%, and using the Peterborough AES gauge data.

Increasing the Lindsay 100-year rainfall volumes by 10% caused a 0.12m 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 PCSWMM model, no difference in flood elevations was noted in the Lindsay Commercial district. The Lindsay AES gauge data is therefore used for all analyses. More details can be seen in **Appendix C**.

2.2 Design Storms

Three different elements are reviewed regarding rainfall to generate return period events: the total volume of rain, the storm duration, and the rainfall distribution. Rainfall distribution is the specific apportionment of rain over time, or the shape of the storm being considered. The relative importance of these factors varies with the characteristics of a catchment. It is accepted practice to test different design storms to determine the most conservative response of a hydrologic system. It is the intent of this study to use the most conservative of commonly used approaches to ensure the most appropriate protection for the community of Lindsay.

In order to determine conservative catchment response generated by different rainfall storm events, a variety of rainfall durations (6- and 12-hours) for 2-100 year return periods were tested. Additionally, in order to determine the critical design storm creating the highest peak discharges, different sets of rainfall distribution were tested. The following discusses the rainfall distributions evaluated in this study.

The Soil Conservation Service Type II (SCS) distribution is a rainfall distribution curve which represents high-intensity rainfall rates generally associated to a 24-hr rainfall. For more than a century, the Natural Resources Conservation Service (US) has continued working on the development of empirical formulas to improve the Soil Conservation (SCS) method for predicting storm runoff from design storm events. The SCS method (1973) presents the 24-hr Type I, IA, II, and IIA rainfall time distributions for runoff predictions. The Type II curve is applied to much of the United States, Puerto Rico, and the Virgin Islands. Generally, other distributions are recommended for coastal area of the country. The Type II distribution is generally tested in hydrology studies undertaken in southern Ontario. The bulk of the rainfall occurs in the second half of the storm.

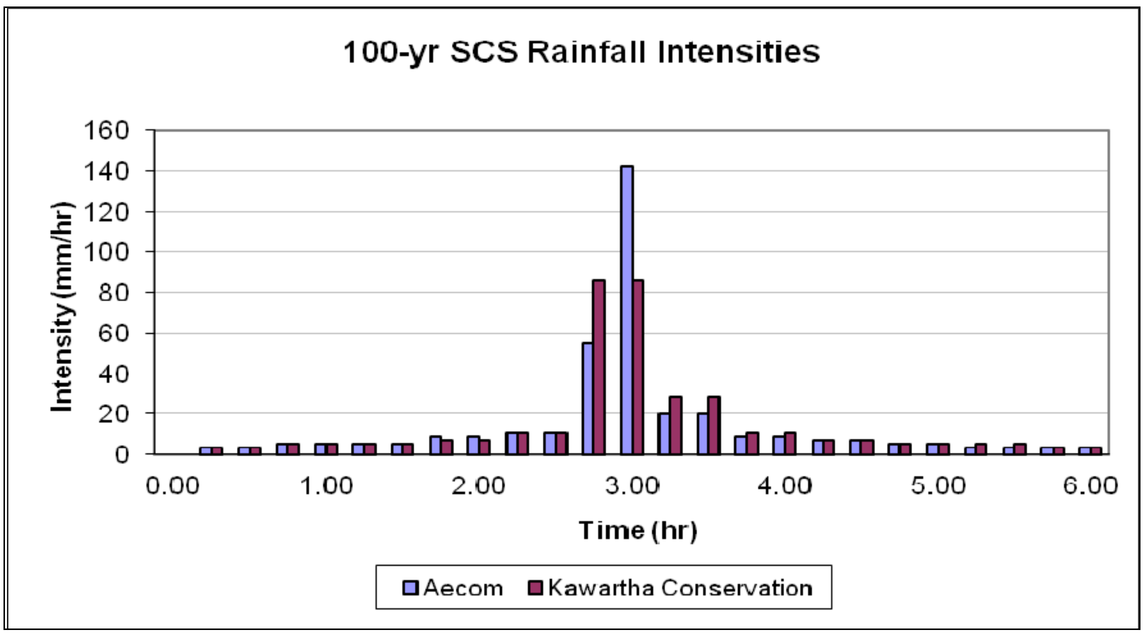
Environment Canada has developed a design storm for southern Ontario. When compared to the SCS distribution, the majority of the rainfall in the Atmospheric Environment Service

(A.E.S.) 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 the design and analysis of storm sewer systems within urban areas. The distribution of rainfall is generally in the centre of the storm and the peak of storm is quite intense. Some investigators consider that this distribution yields unrealistically "peaky" hyetographs, especially when a small time step is used.

The 2010 AECOM report concluded that the runoff from a 6-hour SCS storm is the most critical for the Ops #1 Drain/Jennings Creek catchment. Kawartha Conservation staff analyzed a variety of storm events (i.e. 6- and 12-hour Chicago, SCS, and AES storms) for 2-100 year return periods, using the design storm hyetographs as determined by both the MTO and MNR.

Figure 2.1: Comparing AECOM and Kawartha Conservation SCS Rainfall Intensities



The worst case storm (the duration and distribution producing the highest discharges at key nodes) is selected as the critical event for the watershed. Detailed rainfall information is shown in **Appendix C**.

2.3 Regional Storm

The Timmins storm with a total rainfall of 193mm was applied to the Ops #1 Drain/Jennings Creek as the Regional storm event. The full storm is defined by Chart 1.04 of the *MTO Drainage Manual*. Antecedent moisture content (AMC) condition II, referred to as AMC (II), was applied. For a conservative estimate, and to be consistent with previous studies, saturated ground conditions reflected by AMC (III) were also applied. An aerial reduction factor was not applied to the Regional model as previously discussed in section 2.1.

2.4 Snowmelt and Snowmelt/Rainfall Events

These analyses were not carried out for this report because there is no recorded data that has captured the runoff from a specified combination of snowmelt and precipitation.

3 Hydrologic Parameters

3.1 Overview

In 2012, the City of Kawartha Lakes and Kawartha Conservation agreed to produce a standardized methodology for completion of a number of flood plain mapping studies within its watersheds. This approach was peer-reviewed by Greck and Associates Limited, and their findings conclude the methodology is valid. All parameters and modeling approaches described within this report follow the recommendations presented in **Appendix B** unless otherwise noted.

As previously mentioned, for the Ops #1 Drain/Jennings Creek watershed, the modeling of unsteady flow using a dynamic wave (instead of kinematic) is recommended. This approach will ensure that the discharge will vary in space and attenuate as it moves downstream to account for the time and volume dynamics of complex flood plain storage.

Hydrological modeling was carried out generating runoff hydrographs produced by PCSWMM. This software implements a dynamic wave flow and is therefore suitable to route flow through the Ops #1 Drain/Jennings Creek. The PCSWMM program is based on the US EPA SWMM5 engine, and is capable of modeling either kinematic or dynamic wave routing. In previous versions of SWMM, dynamic wave routing was carried out by a subroutine known as EXTRAN. Although the EXTRAN subroutine is no longer used in SWMM5, for comparison purposes in this study, dynamic wave routing results will be compared to previous EXTRAN results.

For this study Kawartha Conservation updated base information for the watershed including newly acquired LiDAR elevation data, orthoimagery, updated Arc Hydro watershed boundaries, and field surveys. Kawartha Conservation staff modified the AECOM model using the improved input data and found significant differences in model results.

3.2 Digital Elevation Model

In order to generate a highly accurate Digital Elevation Model (DEM) for the study area, two points per square meter LiDAR data was acquired. ArcGIS version 10.1 computer software programs translated the collected data points as a Triangulated Irregular Network (TIN) in order to isolate ground elevation points from the full dataset. This resulting data was converted to a 0.5 m raster digital elevation model (DEM), which in turn provides elevation information for the model. LiDAR data was also used in conjunction with Real Time Kinematic (RTK) Global Positioning System (GPS) survey data of culvert locations and invert elevations to create a drainage network.

The validity of the DEM was analyzed in June 2013 as detailed in the report titled, *Peer Review of Remote Sensing and GIS Data Support for Flood Line Mapping – Ops No. 1 Drain/Jennings Creek*. It was shown to be in compliance with the 2009 Ontario Imagery and Elevation Acquisition Guidelines.

3.3 Catchment Discretization

In order to discretize catchments, ArcHydro version 10.1 beta software was utilized to generate flow path ways within the watershed using the DEM as input data. The resultant watercourse layer was employed to enforce water routing through roads and other impediments which can act as obstacles to channel flow (i.e. culverts and bridges).

Critical nodes within the watershed were the basis to delineate the initial catchments in ArcHydro. ArcHydro is suitable for the delineation of catchments within rural areas; however in urban areas where a stormwater collection system exists, the ArcHydro tool has deficiencies for including sub-surface pipe networks. ArcHydro also has drawbacks for determining overland flow pathways in urban areas where the topography forms a concave shape. To overcome this gap, field visits were carried out to verify and modify catchment boundaries as required. Existing plan/profile drawings and stormwater management reports for the Ops #1 Drain/Jennings Creek were reviewed in order to manually adjust urban catchments upon direction of the engineer. **Figure 3.1** illustrates the catchments.

Total imperviousness (T_{imp}) was determined by digitizing all impervious surfaces within the watershed, including parking lots, roofs and driveways. Percentage imperviousness was then calculated for each catchment.

Figure 3.1: Catchment Boundaries

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City of KAWARTHA LAKES
Catch the Kawartha spirit

Produced by Kawartha Conservation (2013) with data supplied under license by members of the Ontario Data Exchange, Ontario Ministry of Natural Resources.

3.4 Geometric Properties

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

3.5 Calculation of Slope

The slope calculation requires information of the flow paths for overland flow, channel flow within the catchment, and main channel flow from where the catchment flow path intersects the main channel to the downstream channel node. Spreadsheets calculating channel and catchment slopes, and individual catchment time of concentration (T_c) and time to peak (T_p) calculations are found in **Appendix D.**

3.6 CN Values

The Soil Conservation Service (SCS) curve number (CN) is used to determine runoff. Users must choose which antecedent moisture condition (AMC I, II, or III) is relevant for the model; AMC II represents a dry soil condition, and AMC III represents saturated soil.

For this study, existing rural land use (based on CN value categories) was digitized from the projects orthophotography, land use zoning from the City of Kawartha Lakes and other GIS data were also queried to extract land use, drainage area, and hydrologic soils group data. A weighted CN (AMC II) value was calculated, using the values found in **Appendix E**.

The VO_2 program requires that the CN value be transformed to CN* (AMC II). For the Regional storm, the CN (AMC II) was converted to CN* (AMC III). These calculations are included in **Appendix E**. **Figure 3.2** provides soils information while **Figure 3.3** shows the existing land use of the watershed. Spreadsheets with the calculations are provided in **Appendix E**.

3.7 Catchment 13

A recent flooding problem brought to the attention of City of Kawartha Lakes engineering staff highlighted a drainage area tributary to the Ops #1 Drain/Jennings catchment that was previously unknown. Catchment 13 drains to a man-made channel that runs southeast from Angeline St North and ultimately outlets to Jennings Creek. This area lies beyond the LiDAR acquisition area. As a result, Kawartha staff analyzed 2m contours from the 2002 GTA elevation acquisition project in conjunction with 2008/2009 orthoimagery to manually delineate drainage boundaries. These boundaries were reviewed by the engineer and subsequently digitized and incorporated to complete catchment boundaries for the Ops #1 Drain/Jennings Creek watershed.

Figure 3.2: Soils

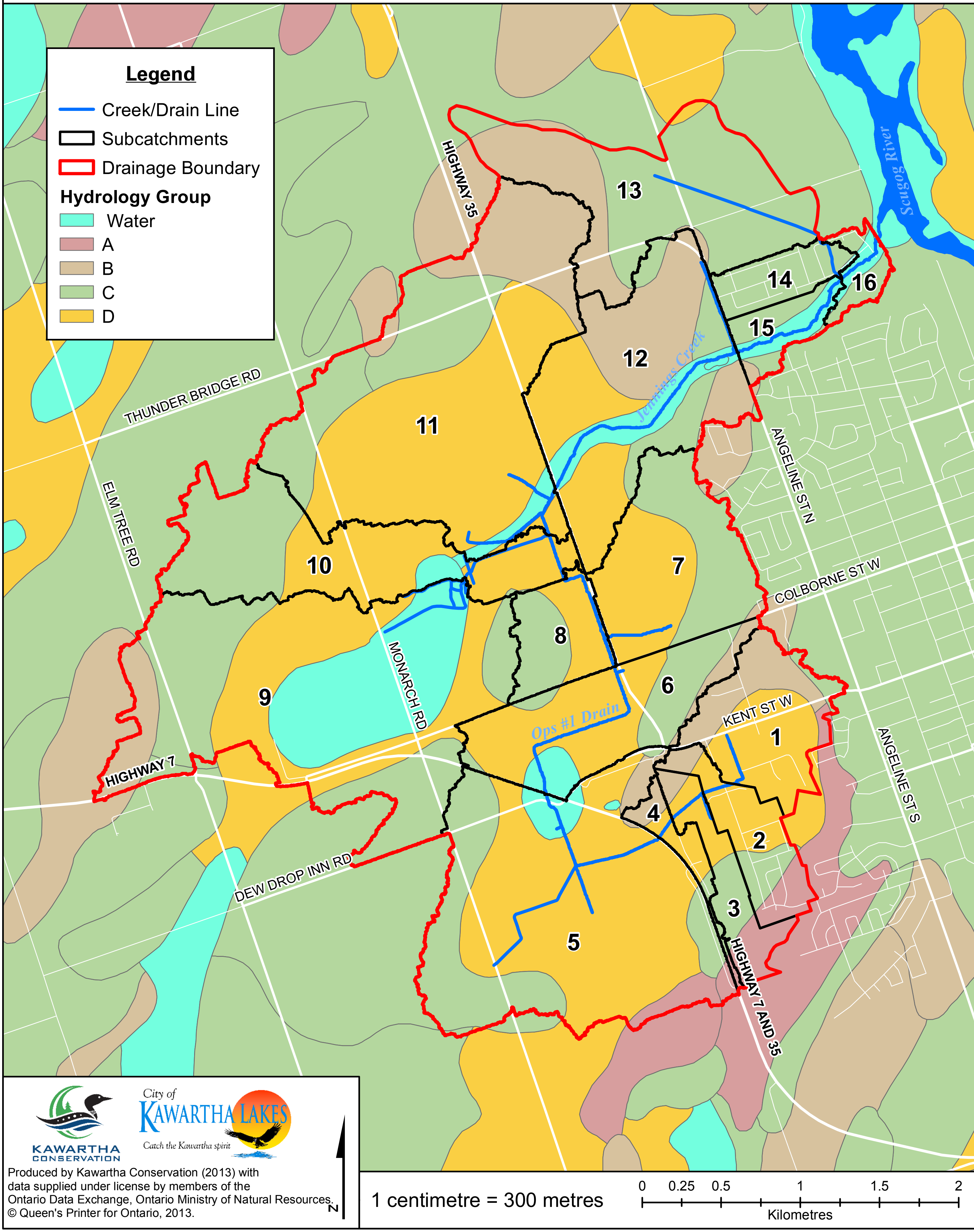

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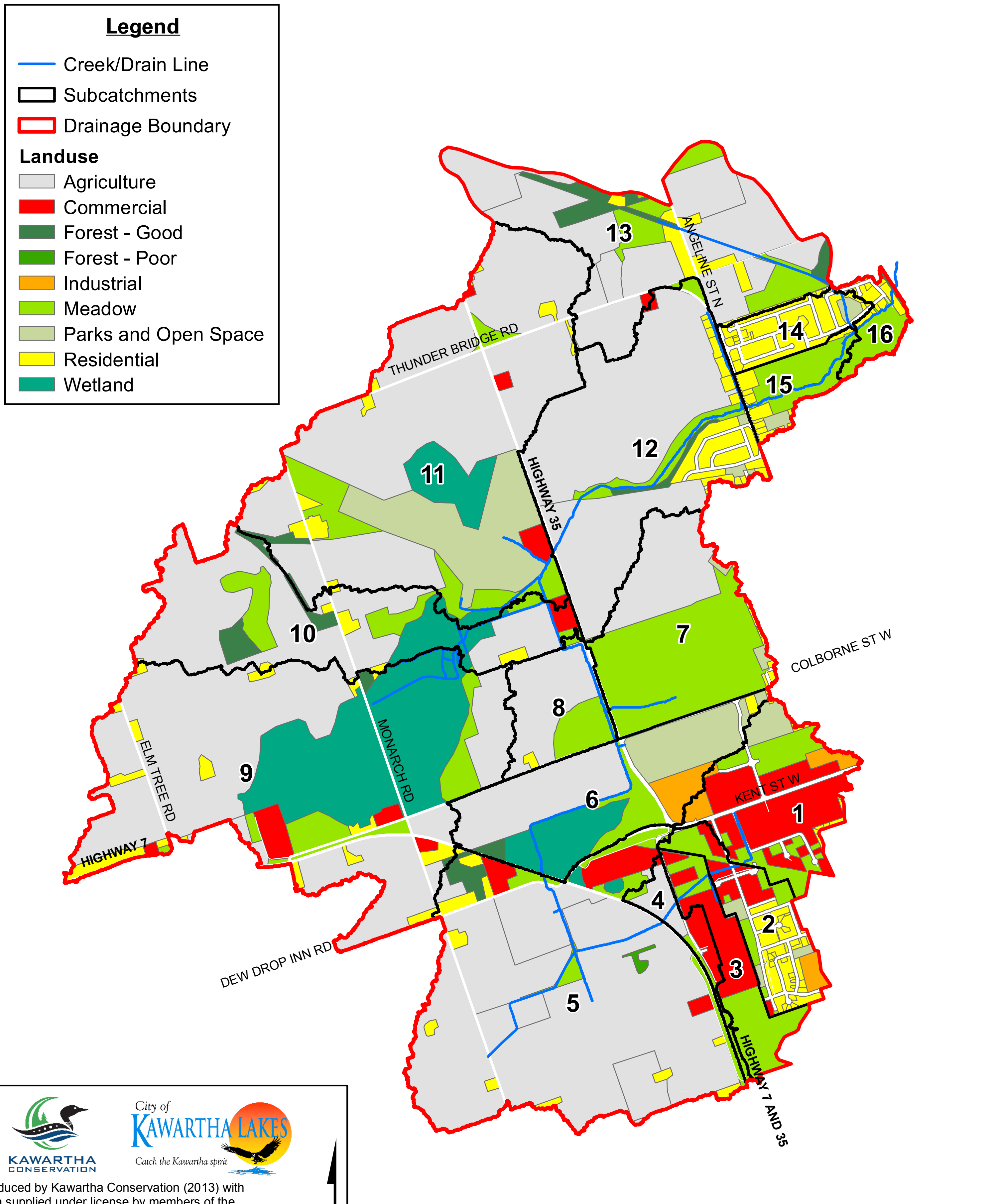
Figure 3.3: Existing Land Use



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Land use information bot produced by KRCA and provided by The City of Kawartha Lakes

3.8 Stormwater Management (SWM) Facilities

No approved or proposed SWM facilities are included in the hydrological model, due to several reasons. The Ministry of Environment (MOE), Kawartha Conservation, and the City of Kawartha Lakes require SDWEM facilities for quality and quantity control for storm events up to the 100-year return period. However, flood plain mapping is generally based on a Regional event which is beyond the design range of a SWM facility. Secondly, the worst-case scenario is assumed, wherein all structures fail. Thirdly, for private sites having stormwater controls, the City of Kawartha Lakes and Kawartha Conservation have no ability to enforce regular maintenance or inspection of the facilities and therefore there is no assurance they will continue functioning as designed.

3.9 Catchment Widths

The catchment width parameter in PCSWMM is typically estimated by first measuring a representative overland flow path length, then dividing the area by this length (W=A/L), where A is the catchment area and L is the length. Calculating the width of a catchment is not a straight-forward procedure since the overland flow path length cannot simply be linked to the pipe length within the catchment. Often catchments are irregular in shape where pipes are not on the side of the catchment.

Two separate calculations were used to derived catchment width; one for urban areas, another for rural catchments. For urban areas, it is fairly easy to measure the actual overland flow route to pipes and/or major overland flow route. A skew factor was applied which is given as:

$S_k = (A_i - A_j)/A$ DiGiano et al. (1977)

Where S_k = Skew factor

A_i= area to one side of pipes

A_j=area to other side of pipes

A= total area

The width is calculated using a weighted equation for the measured overland flow length (L),

Where $W = (2 - S_k) * L$

For rural areas, a different approach was used as suggested by the developers of PCSWMM, Computational Hydraulics International (CHI). Representative overland flow paths to the catchment outlet were digitized in GIS, using contours as a guide. PCSWMM analyzed the lengths to derive an average flow length.

For more information see **Appendix F**.

4 Hydrologic Model

4.1 Channel Routing

As mentioned in Section 1, the dynamics of flooding within the Ops #1 Drain/Jennings Creek were previously assessed using standard steady flow methods in Otthymo and steady-state HEC-RAS evaluations by other consulting companies. However, this approach does not account for attenuation and backwater effects from undersized culverts and relatively large shallow flood storage areas. Therefore, AECOM replaced the Otthymo/HEC-RAS simulation with dynamic modeling employing the EXTRAN module of SWMM 4.4, which was originally proposed by Aquafor Beech, 2001. It was expected that taking such an approach would result in the establishment of more realistic peak flows and associated flood lines along the Ops #1 Drain/Jennings Creek.

An additional study (Greck and Associates, 2011) re-examined the dynamics of flooding in the existing Kent Street developed commercial area using an unsteady flood flow hydraulic analysis (XP STORM program), which is similar to the EXTRAN module. This analysis confirmed that flood elevations derived by usual standard methods (such as steady-state flow analysis) will not sufficiently capture the complex hydrology and hydraulics found within the Ops #1 Drain/Jennings Creek.

Thus, the modeling of unsteady flow using dynamic wave (instead of kinematic) is the preferred method in this case. This approach will ensure that the discharge will vary in space and attenuate as it moves downstream to account for the time and volume dynamics of complex floodplain storage.

The hydrological modeling is carried out generating runoff hydrographs produced by PCSWMM using a dynamic wave flow routing routine.

4.2 Cross Sections

The cross-section geometric data used in hydraulic modeling was extracted from the DEM using HEC-GeoRAS. The use of HEC-GeoRAS ensures spatial referencing of geometry data when imported into HEC-RAS. Cross-sections were cut in the LiDAR-derived DEM. Since LiDAR does not return laser points for any ground below the water surface it is necessary to supplement these areas with surveyed data to create accurate river geometry. Bathymetric survey points were taken in-channel up to the top of bank throughout the project area. The surveyed data was fused into the cross-sections generated by HEC-GeoRAS. Data sources generated by different entities were placed into the same projection and datum for consistency in processing.

All cross-sections are oriented looking downstream. The initial cross-section is at the mouth of Jenning's Creek where it joins the Scugog River; cross-section nomenclature reflects the distance in meters relative to the initial cross-section. Distances were determined using GIS measurement tools.

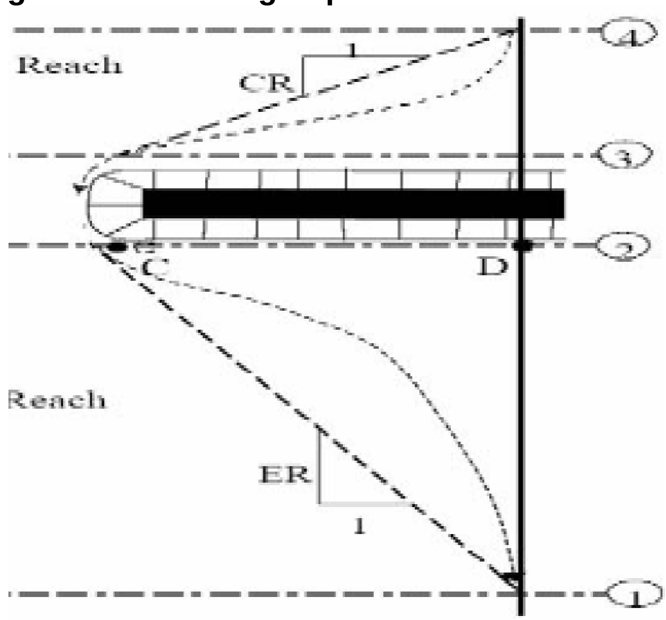
In order to represent channel and overbank attenuation along the drain, simulations were carried out with culverts removed to eliminate ponding of water by structures. Affected channels upstream and downstream of the culverts were merged into one representative channel. This has an additional benefit of eliminating short sections of channel (typically less than 10m); PCSWMM is unstable with such short sections. A spreadsheet comparing HEC-RAS and PCSWMM channel lengths can be found in **Appendix G**.

Stream crossings have been identified and positioned by reviewing the most recent aerial orthophotography in conjunction with field reconnaissance and information utilized by previous reports. Full photographic records of all stream crossings are found in **Appendix H**.

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 G.** The main channel n values range from **.**035 to .04 and the overbank n values range from .016 to 0.1 and were chosen based on air photo and survey notes/photos.

Where buildings are located within or between the cross-sections, ground elevations were artificially increased by a minimum of 5m to replicate obstruction 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; the actual building width is reduced at a 1:1 ratio from each end of the building face. For instance, if a 30m building is 5m downstream of a cross-section, the representative building width 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 30m building is 8m upstream of a cross-section, the representative building width in the cross-section is 26m wide. A representation of the expansion/contraction effects of a building location is shown in **Figure 4.1** below.

Figure 4.1: Building Expansion/Contraction Effects



4.3 Calibration

Since no flow data exists for this drainage basin, no calibration is possible.

4.4 Schematic

The information gathered in the preceding sections was used to build a PCSWMM model of the watershed, as shown schematically in **Appendix I.** The model was run with no culverts in place since the main purpose is to assess channel and flood plain attenuation and avoid structure ponding.

4.5 Sensitivity Analyses

The model was tested for sensitivity for the following input parameters: catchment widths, Manning's n, initial abstraction, and CN values. **Appendix J** has detailed information.

4.6 Catchment Width

Catchment widths were modified by +/- 50%. Reducing catchment widths by 50% lowers peak flows by an average of 32% within the entire watershed. The largest reduction is by 38% for catchment 10, and the smallest reduction is by only 20% for catchment 1. Similarly, increasing catchment widths to 150% of their original value increases peak flows by an average of 24% within the entire watershed. The largest increase is by 31% for catchments 9-11, and the smallest increase is by only 10% for catchment 1.

Such large changes in peak flows demonstrate that the catchment width is a sensitive input parameter. Since the width is not a measureable parameter and is somewhat subject to intuition, the methods recommended by the model authors were used as described in section 3.9.

4.7 Manning's n Values

The manning's n value for all channel cross-sections was modified by +/- 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 rougher surface) the model calculated a 10% decrease in peak flows. Similarly, when the manning's n values were decreased by 20% the model calculated slightly higher peak flows at key nodes, a 7% increase. The n value is therefore not a sensitive input parameter.

4.8 CN*

CN* was changed +/- 20%. Decreasing CN* by 20% lowers flow peaks by an average of 44% within the entire watershed. The largest reduction is by 59% for catchment 15, and the smallest reduction is by 19% for catchment 2. Similarly, increasing CN* to 150% of their original value increases flow peaks by an average of 82% within the entire watershed. The largest increase is by 128% for catchment 8, and the smallest increase is by 19% for catchment 1.

The large increase in peak flows indicates it is imperative to get an accurate CN* value. Since CN* is a value that is derived directly from measured parameters (land use and soil type), there is confidence that the calculated CN* is correct.

4.9 Initial Abstraction (Ia)

The initial abstraction was changed +/- 50%. Decreasing I_a by 50% increases flow peaks by an average of 9% within the entire watershed. The largest increase is by 17% for catchment 6, and the smallest increase is by 2% for catchment 1. Similarly, increasing I_a to 150% of the original value decreases flow peaks by an average of 9% within the entire watershed. The largest decrease is by 14% for catchment 6, and the smallest decrease is by 3% for catchment 1.

Changing the initial abstraction does not result in significantly different flows.

5 Hydrology Model Results

5.1 Comparing Kawartha Conservation model output to AECOM 2010 Model

EXISTING LAND USE

Ops Drain #1 and Jennings Creek were modeled in 2010 by AECOM. As discussed in the previous section, Kawartha Conservation re-created the hydrologic breakdown using the most recent LiDAR and GIS data. Significant differences between the 2010 and 2013 data were discovered with respect to drainage areas, land use, and ground elevation.

Area and land use differences are highlighted in **Table 5.1** and in **Figure 5.1.** AECOM created 32 catchments, Kawartha Conservation uses 16. When comparing cumulative areas to key nodes, it is seen that the AECOM and Kawartha Conservation discretization are close (within 5% total tributary area values); the exception is at the upstream reaches, where the Kawartha Conservation discretization has differences of up to 126%.

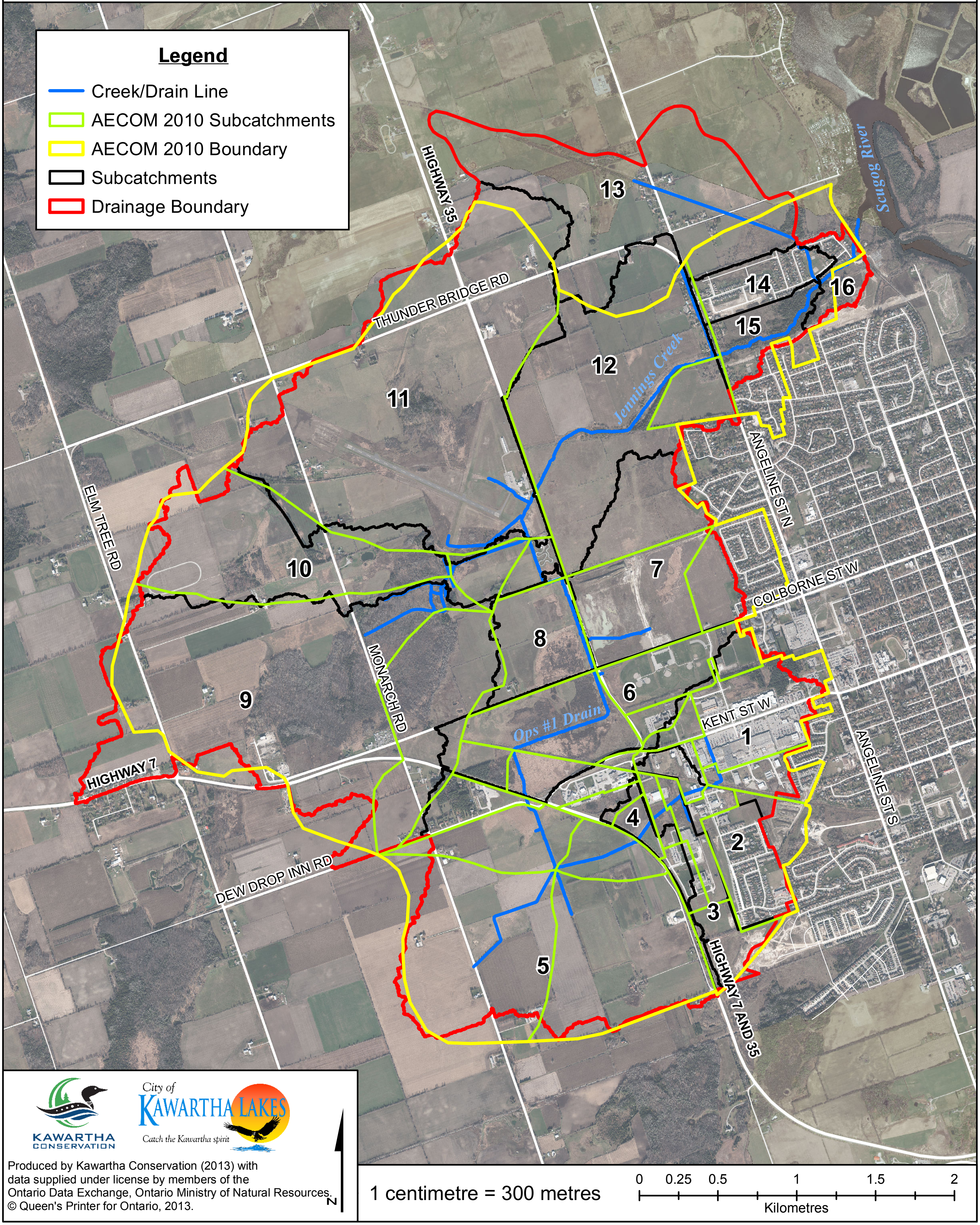
Table 5.1: Comparing Tributary Areas at Key Nodes

Approximate Map Location*	Node Location	Tributary Area (Ha)		*% Diff*
		AECOM	Kawartha Conservation	
3	Node 1 (Kent St ditch)	53.4	67.2	*126%*
5	Node 3 (Moose Lodge culvert)	129.5	106.9	*83%*
6	Node 4 (Greenfield Rd)	134.9	139.6	*103%*
10	Node 5 (Hwy 7 S. of Kent)	164.6	154.2	*94%*
11	Node 7 (Hwy 7 W. of Kent)	397.1	404.2	*102%*
13	Node 8 (Colborne St W)	526.1	497.7	*95%*
17	Node 11 (Hwy 35 @ Airport)	1,340.0	1,318.5	*98%*
18	Node 12 (Angeline St)	1,500.1	1,476.7	*98%*
19	Node 13 (William St N)	1,590.0	1,661.3	*104%*
20	Outlet at Scugog River	1,590.0	1,675.0	*105%*

**Note: Refer to Figure 5.5 for approximate node location*

As discussed in Section 3.7, recent flooding problems highlighted a new drainage area tributary to the Ops #1 Drain/Jennings Creek catchment that was not included in the AECOM model. Catchment 13 drains to a man-made channel that runs southeast from Angeline St North, terminating behind the houses on Mohawk Drive. A ditch inlet directs low-flow water to a pipe located under Springdale Garden Park that ultimately outlets to Jennings Creek. Field investigation has discovered there is no apparent overland flow route for flow exceeding the capacity of the minor system. An analysis of the capture rate and pipe/channel capacity was beyond the scope of this floodplain study. It was assumed all of catchment area 13 drains to Jennings Creek for the purposes of this study.

Figure 5.1: AECOM vs. Kawartha Conservation Catchments



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In the AECOM model, a residential area bounded by Angeline St to the east, Oak Street to the north, and Walker Street to the south was included in the study watershed. Although the minor system from this neighbourhood drains to the vacant lands to the west, the major flow drains east to the Scugog River. Since flood plain mapping studies are based on flooding caused by large events where the minor system input is insignificant, this residential area was not included in this flood plain study.

For the existing land use model scenario, AECOM used standard municipal averages to estimate the percent impervious land use whereas this study used digitized actual impervious surfaces; these differences are highlighted in **Table 5.2** for the upstream catchments. This study's hydrology model therefore has less impervious area. For the upstream portion of the watershed, the differences in flows calculated using standard impervious land values and actual values is significant.

Table 5.2: Comparing Tributary Areas at Key Upstream Nodes

Key Node Location*	*AECOM*			Kawartha Conservation		
	Area (Ha)	*% Imp*	*Total Imp (Ha)*	Area (Ha)	% Imp	Total Imp (Ha)
Node 1 (Kent St ditch)	*53.4*	*72%*	*38.4*	67.2	46%	*30.9*
Node 3 (Moose Lodge culvert)	*129.5*	*58%*	*74.7*	106.9	43%	*45.5*
Node 4 (Greenfield Rd)	*134.9*	*56%*	*75.5*	139.6	36%	*49.8*

**Note: Refer to Table 5.1 and Figure 5.5 for approximate node location*

As previously stated, this study used provincially-recommended storm hyetographs for all analyses. The hydrology model was modified to use the AECOM SCS storm hyetograph in order to more directly compare the results from the two hydrology models. As can be seen in **Table 5.3** below, there is a significant increase in peak flows for catchments 1 and 3.

Table 5.3: Comparing 100-year Runoff for Upstream Catchments Using AECOM SCS Storm

Catchment ID*	100yr 6hr SCS Runoff (m^3/s)	
	AECOM	Kawartha Conservation
1	7.7	11.9
2	15.5	3.7
3	3.0	1.6

**Note: Refer to Figure3.1 for location of catchments*

The Kawartha model was modified again by using not only the AECOM SCS hyetograph but also AECOM's impervious land use values in order to compare the results of the two hydrology models. The results for catchment 1 shown in **Table 5.4** below indicate the Kawartha Conservation and AECOM model predict similar results; the peak flows are 0.36

m^3/s/Ha and 0.33 m^3/s/Ha, respectively. For catchment 2, the peak flows are 0.22 m^3/s/Ha and 0.21 m^3/s/Ha, respectively.

Table 5.4: Comparing 100-year Runoff for Upstream Catchments Using the AECOM SCS Storm and Impervious Land Values

Catchment ID*	100yr 6hr SCS Runoff (m^3/s)	
	AECOM	Kawartha Conservation
1	17.7	24.4
2	15.5	8.5
3	3.0	4.3

**Note: Refer to Figure3.1 for location of catchments*

5.2 Catchment Connectivity

The land west of the Lindsay Airport and north of Highway 7 is exceedingly flat, with scattered pockets of wetland. In the AECOM report, the VO_2 hydrograph from catchments 20, 21, 22, and 24 was used as input to the Extran model. It appears that catchment 23 was not accounted for. The Kawartha Conservation model includes catchment 23.

There is a pocket of commercial development west of Greenfield Drive immediately south of the Hwy 35 overpass. There are culverts through the embankment under the overpass. In the AECOM hydrology model, this commercial area was included in catchment 15 and connected to the Ops Drain at a point west of the overpass. However, field reconnaissance by Kawartha Conservation staff discovered this area drains under Hwy 7 to the west; it is within Kawartha Conservation catchment 5 and connects to the Ops Drain immediately upstream of Dew Drop Inn Road.

In this study catchment 5 closely matches the boundaries of AECOM's catchments 10, 11, 12, 13, and 14. AECOM had identified several ephemeral channels with separate connection points to the Ops Drain. Furthermore, the distance between the Hwy 7 and Dew Drop Inn Rd culverts is only 25m. A decision was made to connect this drainage area to the Drain at a point immediately upstream of the Hwy 7 culvert.

5.3 Channel Routing

As previously explained, channel cross-sections are taken from the LiDAR-based DEM. This can be seen in **Figure 5.2**, **Figure 5.3**, and **Figure 5.4** showing depressions that cause significant flow attenuation. Due to the unavailability of the detailed LiDAR based DEM, AECOM models were not able represent these depressions. Their model output did not decrease flow peaks in a downstream direction. **Table 5.5** highlights the differences in calculated flow peaks within this area.

Table 5.5: Comparing 100-year Runoff at Key Nodes Using the AECOM SCS Storm

Key Node*		100Yr 6hr SCS Runoff (m^3/s)	
		AECOM	Kawartha Conservation
5	D/S of McLaughlin Rd culvert	11.8	8.2
7	U/S of Hwy 7 culvert	11.8	10.5
8	West of Hwy 7 at 90-degre bend	12.8	11.1
9	North of 90-degree bend	25.2	11.5
10	*D/S of Dew Drop Inn Rd*	*25.6*	*7.9*
11	D/S of Hwy 7	26.0	9.7

** U/S = upstream, D/S = downstream

*Note: Refer to Figure 5.5 for approximate node location

Figure 5.2: Depressed Area South of Dew Drop Inn Road

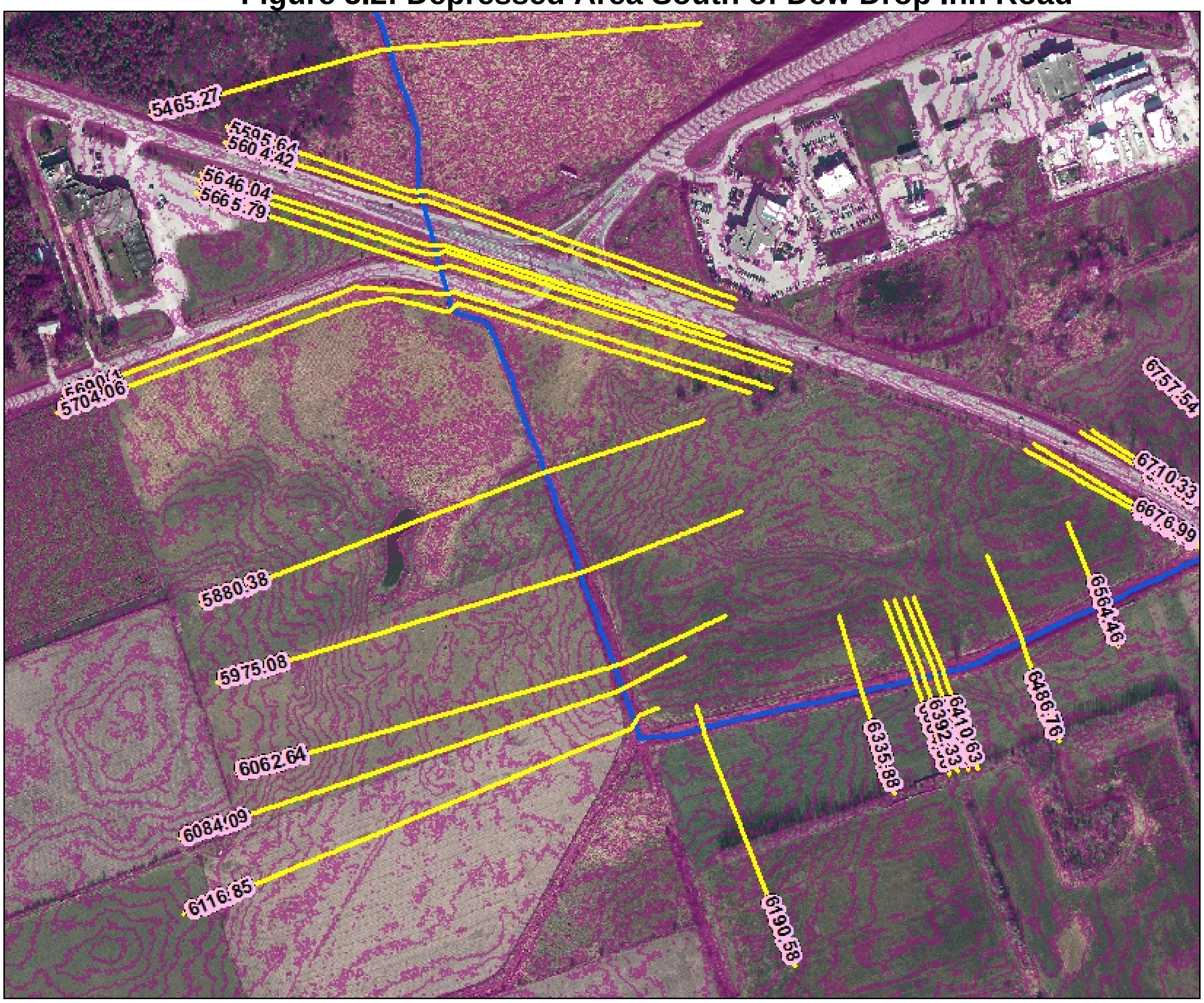
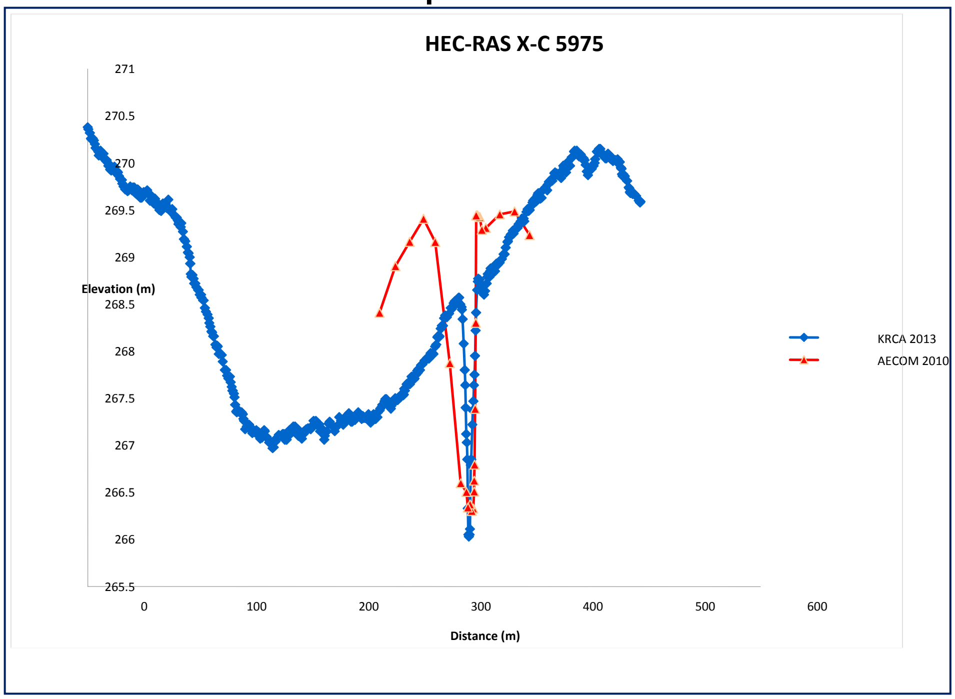


Figure 5.3: Comparing AECOM and Kawartha Conservation Cross-sections in Depressed Area



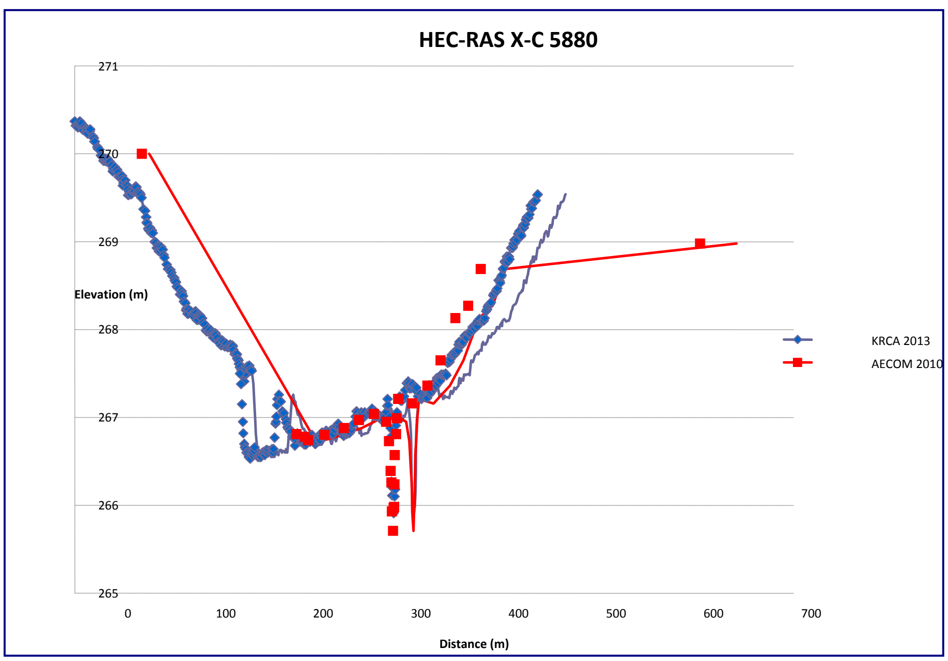
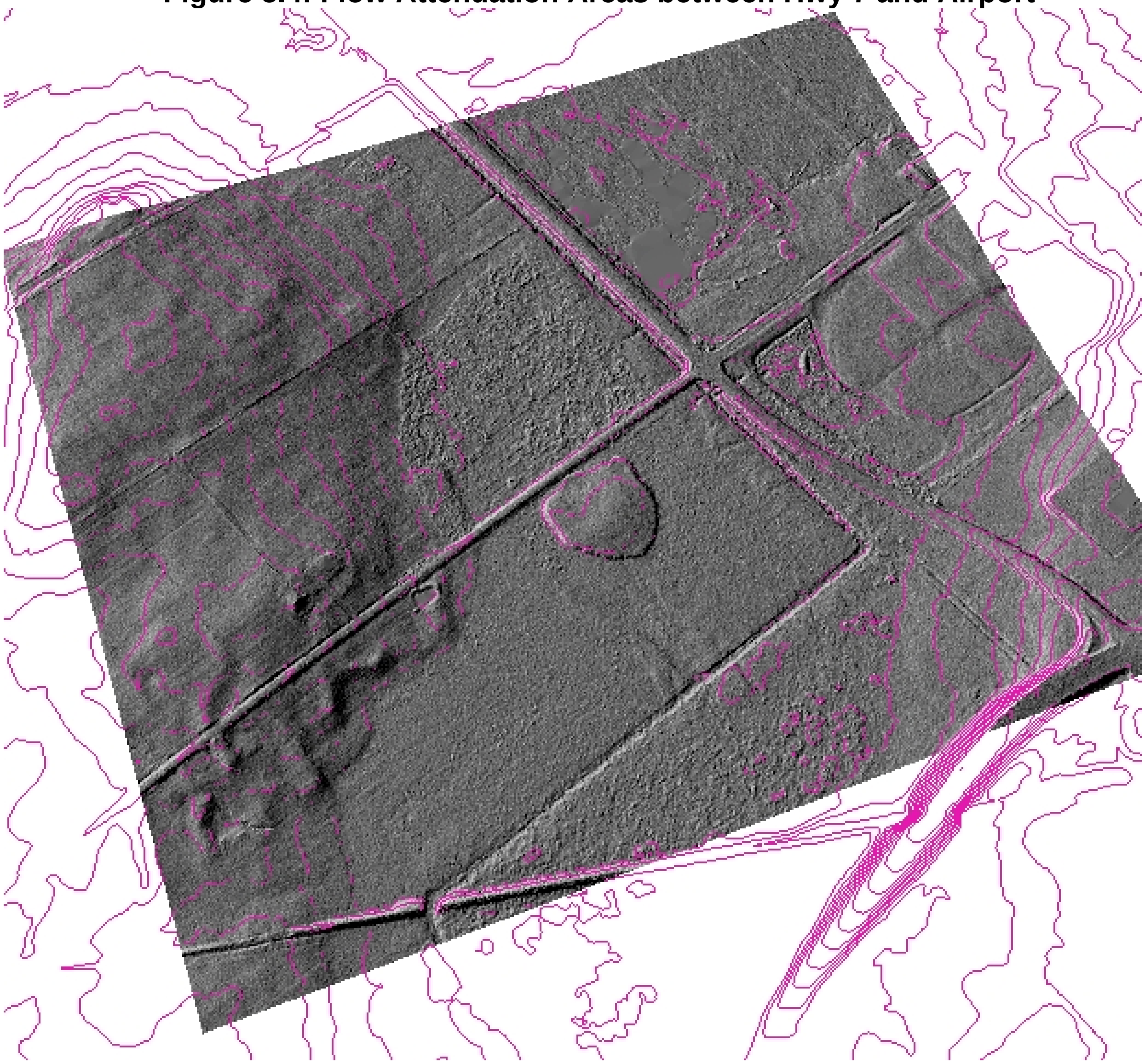


Figure 5.4: Flow Attenuation Areas between Hwy 7 and Airport



For existing land use conditions, it was found that the 12-hour Chicago storm is the significant storm for the watershed for the 100-year event. Flow comparisons are highlighted in **Table 5.6** below. **Table 5.7** lists the flows at key nodes for the Timmins storm. Detailed output is in included in **Appendix K**.

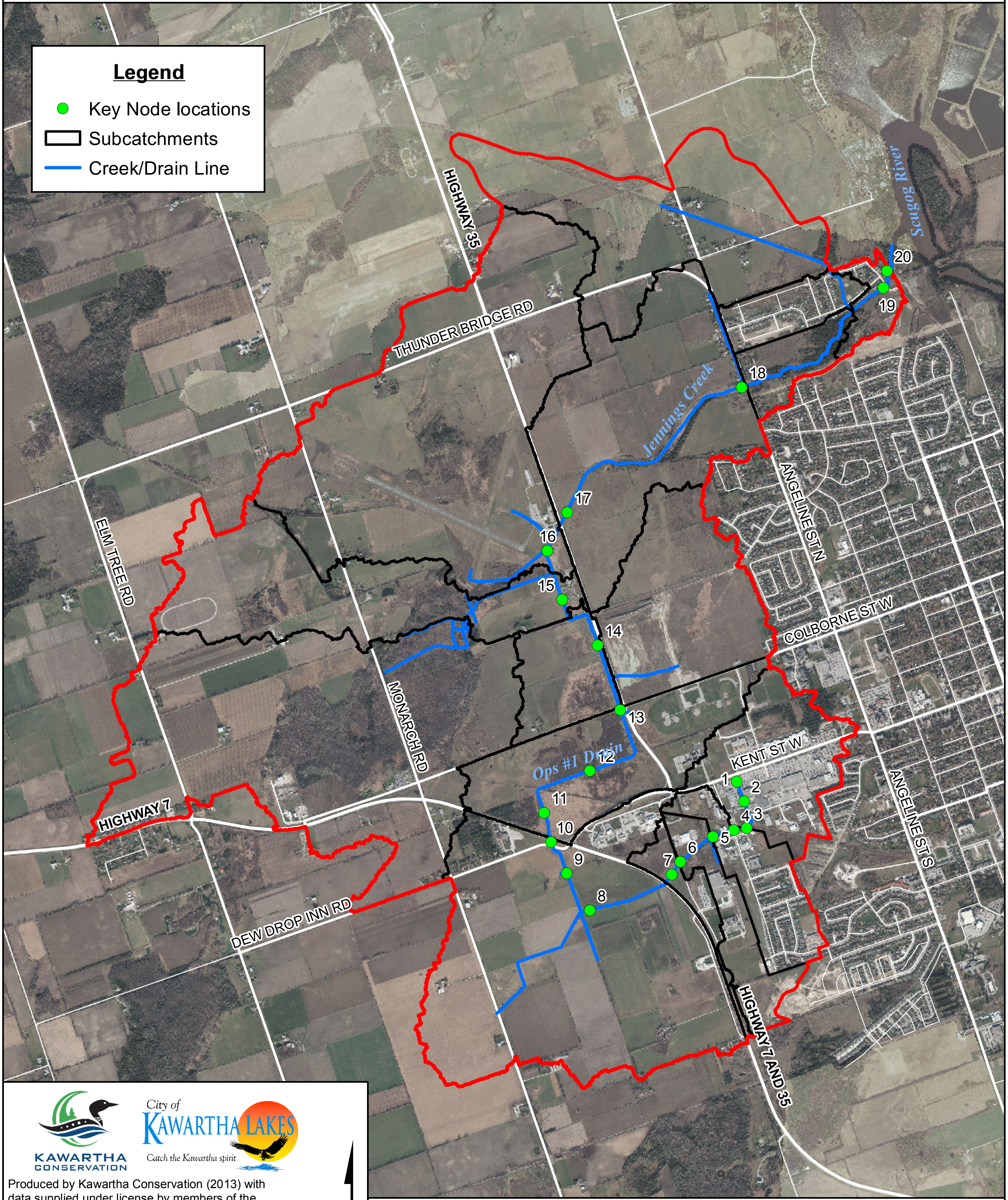
Table 5.6: 100-year Flows at Key Nodes for Existing Land Use Conditions

Node*		Flow from critical storm in m^3/s	
		AECOM 6hr SCS	Kawartha Conservation 12hr Chicago
1	Start of system	N/A	19.74
2	Commerce Rd culvert	N/A	16.26
3	Former train track culvert	N/A	15.41
4	McLaughlin Rd culvert	N/A	14.11
5	Ballfield Culvert	11.8	18.24
6	Greenfield Rd culvert	N/A	16.27
7	Hwy 7 culvert	12.8	19.26
8	W of Hwy 7 at 90-degree bend in drain	13.7	17.14
9	North of 90-degree bend	25.2	13.12
10	Between Dew Drop Inn Rd & Hwy 7	25.6	24.78
11	D/S of Hwy 7	26	23.77
12	North of Hwy 7 past 90-degree bend	24	19.17
13	Colborne St culvert	20.6	12.97
14	D/S of Colborne St	19.4	11.58
15	D/S of private driveway	21.9	12.92
16	U/S of airport	31	27.24
17	D/S of Hwy 35 (near airport)	47.4	27.19
18	Angeline St culvert	55.2	33.93
19	D/S of Angeline St	60.4	38.85
20	William St culvert	N/A	39.23

** U/S = upstream, D/S = downstream

**Note: Refer to Figure 5.5 for map of Key Nodes*

Figure 5.5: Key Node Locations



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N

1 centimetre = 300 metres

0 0.25 0.5 1 1.5 2
Kilometres

Table 5.7: Timmins storm Flows at Key Nodes for Existing Land Use Conditions

Node*		Flow from Timmins Storm in m^3/s		
		AECOM (2010)	Kawartha Conservation	
		CN (III)	CN* (III)	CN * (II)
1	Start of system	N/A	8.22	8.03
2	Commerce Rd culvert	N/A	8.12	7.92
3	Former train track culvert	N/A	8.08	7.87
4	McLaughlin Rd culvert	N/A	8.10	7.89
5	Ball field culvert	9.9	12.43	12.00
6	Greenfield Rd culvert	N/A	12.49	12.04
7	Hwy 7 culvert	12.0	17.29	16.19
8	W of Hwy 7 at 90-degree bend in drain	14.1	17.24	16.14
9	North of 90-degree bend	28.6	17.11	15.85
10	Between Dew Drop Inn Rd & Hwy 7	30.1	39.13	35.52
11	D/S of Hwy 7	30.9	37.24	33.94
12	North of Hwy 7 past 90-degree bend	30.8	29.09	27.10
13	Colborne St culvert	28.7	27.85	27.58
14	D/S of Colborne St	30.7	22.86	22.83
15	D/S of private driveway	35.6	24.07	24.01
16	U/S of airport	58.6	61.08	51.25
17	D/S of Hwy 35 (near airport)	74.7	60.56	50.96
18	Angeline St culvert	105.0	69.33	58.28
19	D/S of Angeline St	95.9	80.16	67.94
20	William St culvert	N/A	80.94	68.67

** U/S = upstream, D/S = downstream

**Note: Refer to Figure 5.5 for map of Key Nodes*

For existing land use conditions, the Kawartha Conservation model calculates much lower flows than those calculated by AECOM.

5.4 Comparing Kawartha Conservation PCSWMM and VO2 Model Output

As recommended in the August 2013 modeling approach peer review by Greck and Associates Limited, a second hydrology model was set up in VO_2. No channels were included in the models since the purpose was solely to compare catchment runoff. The hydrographs from VO_2 were input directly into the appropriate junctions in the PCSWMM model for channel routing. The attenuated flows were extracted from the PCSWMM model at key nodes for comparison.

As can be seen in **Table 5.8**, the runoff calculated by VO_2 differs from what is calculated in PCSWMM. Although the 100-year VO_2 peak flows are on average 87% of the PCSWMM peaks for the watershed, the difference in individual catchments ranges from 43% to 182%. A similar finding is made for the Timmins storm, where VO_2 peak flows are on average 80% of the PCSWMM peaks for the watershed, the difference in individual catchments ranges from 63% to 102%.

Table 5.8: Comparing Catchment Runoff Peaks

Catchment ID*	100yr Chicago Storm Qp (m3/s)		Timmins CN(III) Qp (m3/s)	
	PCSWMM	VO2	PCSWMM	VO2
1	20.67	19.06	8.24	7.89
2	5.97	10.84	4.55	4.66
3	2.55	1.33	3.51	2.51
4	2.33	1.17	1.74	1.43
5	15.20	13.19	25.82	19.54
6	6.31	4.34	10.47	7.70
7	4.16	2.73	9.20	5.83
8	3.41	1.76	4.28	2.83
9	4.72	5.69	20.31	16.07
10	3.98	3.71	10.57	7.74
11	8.57	8.22	24.25	17.68
12	10.44	7.78	16.81	11.90
13	3.27	3.33	11.27	8.27
14	4.76	7.47	2.87	2.86
15	4.68	2.51	2.72	2.43
16	3.74	1.61	1.68	1.51

**Note: Refer to Figure3.1 for location of catchments*

As can be seen in **Table 5.9 below**, flows at key nodes differ slightly when comparing the 100-year VO_2 runoff hydrographs routed in PCSWMM versus the PCSWMM runoff hydrographs routed in the model. On average the VO_2 routed flow peaks are 97% of the PCSWMM routed hydrographs: the maximum peak flow node difference is 119%, and the minimum peak flow difference is 86%.

The results are quite similar when comparing the Timmins VO_2 runoff hydrographs routed in PCSWMM versus the PCSWMM runoff hydrographs routed in the model. On average the VO_2 routed flow peaks are 94% of the PCSWMM routed hydrographs: the maximum peak flow node difference is 105%, and the minimum peak flow difference is 80%.

Table 5.10: Comparing Node Flow Peaks

Node*		100yr Chicago Storm Qp (m3/s)		Timmins CN(III) Qp (m3/s)	
		PCSWMM	VO2	PCSWMM	VO2
1	Start of system	19.24	17.33	8.22	7.86
2	Commerce Rd culvert	16.26	14.34	8.12	7.74
3	Former train track culvert	15.41	13.50	8.08	7.69
4	McLaughlin Rd culvert	14.11	13.12	8.10	7.67
5	Ball field culvert	18.24	19.92	12.43	12.07
6	Greenfield Rd culvert	16.27	18.21	14.49	11.82
7	Hwy 7 culvert	19.26	19.12	17.29	15.41
8	W of Hwy 7 @ 90-degree bend in drain	17.14	19.01	17.24	15.39
9	North of 90-degree bend	13.12	11.48	17.11	13.67
10	Between Dew Drop Inn Rd & Hwy 7	24.78	21.36	39.13	32.34
11	D/S of Hwy 7	23.77	20.42	37.24	31.36
12	North of Hwy 7 past 90-degree bend	19.17	16.71	29.09	26.93
13	Colborne St culvert	12.97	15.38	27.85	29.12
14	D/S of Colborne St	11.58	11.58	22.86	22.79
15	D/S of private driveway	12.92	13.28	24.07	24.08
16	U/S of airport	27.24	27.73	61.08	60.33
17	D/S of Hwy 35 (near airport)	27.19	27.72	60.56	58.62
18	Angeline St culvert	33.93	31.87	69.33	67.06
19	D/S of Angeline St	38.85	35.44	80.16	76.74
20	William St culvert	39.23	35.58	80.94	77.25

** U/S = upstream, D/S = downstream

**Note: Refer to Figure 5.5 for map of Key Nodes*

Since both models calculate a similar runoff coefficient and runoff depth, the main reason for the difference in flow peaks seems to be a difference in timing. VO_2 response time is directly linked to the Time of Concentration (T_c), which is based on the longest flow path of a catchment. As described in section 3.9, in PCSWMM, it is the average flow length that is used to calculate catchment width. Further information can be found in **Appendix K**.

6 Hydrology Model Results for Future Land Use

6.1 Comparing Kawartha Conservation model output to AECOM 2010 Model

At the outset of the study period, the City of Kawartha Lakes provided the Kawartha Conservation with copies of the OP, zoning and secondary plans within the study area. Digital GIS layers were provided as well.

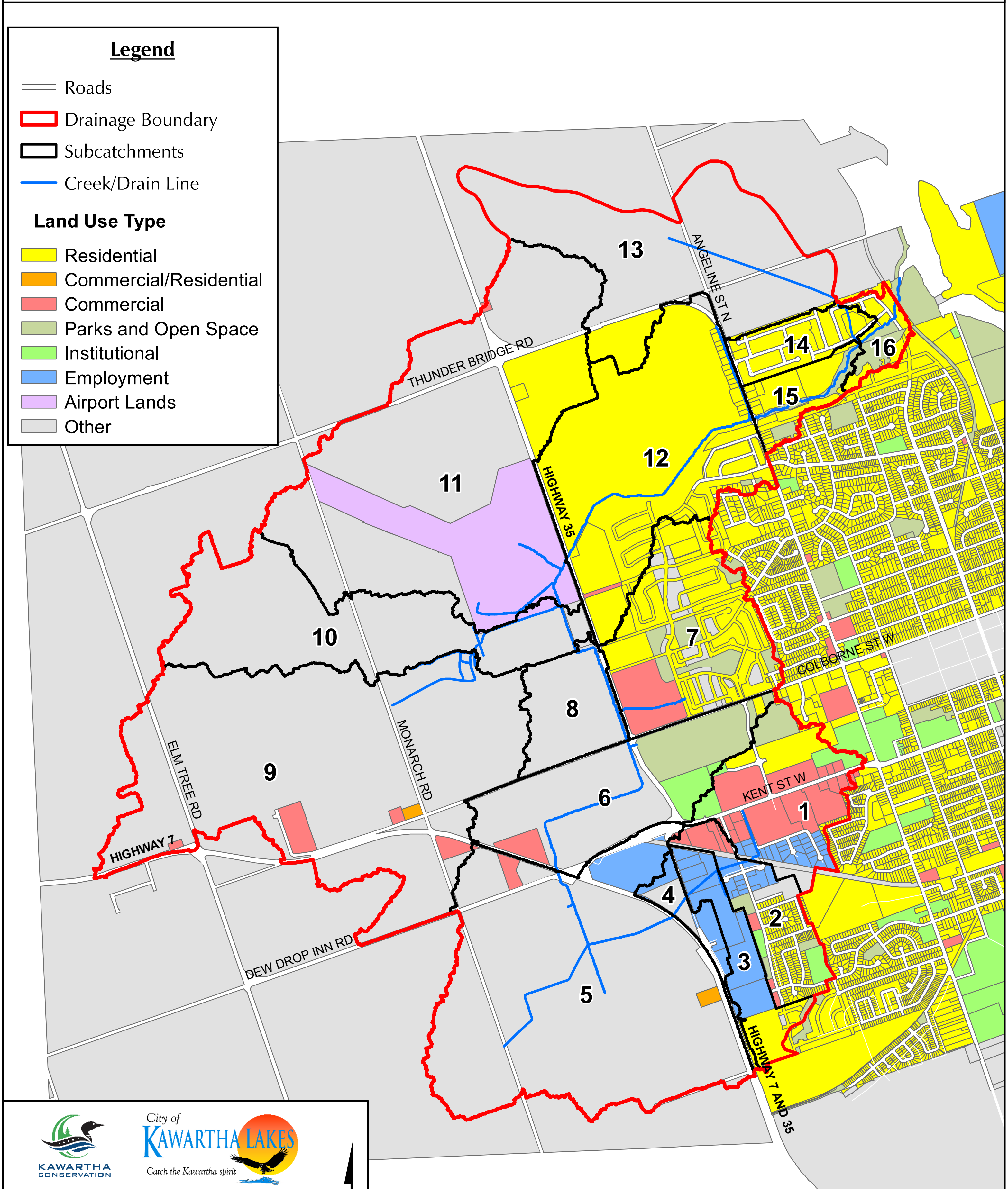
In the creation of the future land use conditions model, it was assumed that catchment boundaries would not change for the future land use conditions, and that all developable areas as indicated in the OP and secondary plans would be fully built out. The imperviousness of each catchment was calculated using the parameters outline in **Appendix B**.

The Greck peer review recommended incorporating specific development plans and their stormwater management requirements into the hydrology model. At the outset of the study period, no such plans and/or requirements have been approved by the City; this step was therefore not carried out for this study.

Figure 6.1 shows the future land use conditions. Using the methods described in section 3 the area, channel length, overland flow lengths, and slopes for rural catchments were derived from ArcHydro. CN values were calculated as well. Further details can be found in **Appendix L**.

Since the existing land use hydrology models indicated that the 12-hour Chicago storm is the critical storm for the watershed, the future hydrology models were set up only for the Timmins and the 12-hour Chicago storms. Both VO_2 and PCSWMM models were set up to replicate future land use conditions. The hydrographs from each model were routed in PCSWMM to calculate the flow attenuation. Peak attenuated flows at key nodes were compared to determine which values to input into the static hydraulic model. Detailed flow comparisons are located in **Appendix M**.

Figure 6.1: Future Land Use as Characterized by Official Plan



KAWARTHA CONSERVATION

City of KAWARTHA LAKES
Catch the Kawartha spirit

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1 centimetre = 300 metres

As can be seen in **Table 6.1** below, the 100-year future flows are higher at the upstream end of the watershed than what had been calculated in the AECOM report. At the downstream end, however, the 100-year flows are slightly lower.

When comparing the Timmins storm VO_2 runoff hydrographs routed in PCSWMM versus the PCSWMM runoff hydrographs routed in the model, on average the VO_2 routed flow peaks are 99% of the PCSWMM routed hydrographs: the maximum peak flow node difference is 125%, and the minimum peak flow difference is 81%.

Table 6.1: Comparing 100-year Future Peak Flows at Key Nodes

Node*		100-year Flow in m3/s		
		AECOM Future 6hr SCS Storm	Kawartha Conservation PCSWMM 12 hr Chicago	
			PCSWMM	VO2
1	Start of system	N/A	23.01	22.19
2	Commerce Rd culvert	N/A	19.15	15.82
3	Former train track culvert	N/A	17.78	14.47
4	McLaughlin Rd culvert	N/A	16.20	14.16
5	Ball field culvert	11.8	22.24	22.52
6	Greenfield Rd culvert	N/A	19.34	20.94
7	Hwy 7 culvert	12.8	26.22	27.55
8	W of Hwy 7 @ 90-degree bend in drain	13.7	25.02	26.28
9	North of 90-degree bend	25.2	20.42	19.51
10	Between Dew Drop Inn Rd & Hwy 7	25.6	33.48	31.27
11	D/S of Hwy 7	26.0	32.03	29.30
12	North of Hwy 7 past 90-degree bend	24.0	24.37	21.82
13	Colborne St culvert	20.6	16.71	20.83
14	D/S of Colborne St	19.4	15.70	17.20
15	D/S of private driveway	21.9	17.43	16.57
16	U/S of airport	31.0	43.20	41.54
17	D/S of Hwy 35 (near airport)	47.4	41.24	40.55
18	Angeline St culvert	55.2	54.49	60.16
19	D/S of Angeline St	60.4	61.92	64.31
20	William St culvert	N/A	62.60	65.18

** U/S = upstream, D/S = downstream

**Note: Refer to Figure 5.5 for map of Key Nodes*

As can be seen in **Table 6.2**, the Timmins future flows are generally higher at the upstream end of the watershed than what had been calculated in the AECOM report. At the downstream end, however, the flows are substantially lower.

When comparing the 100-year VO_2 runoff hydrographs routed in PCSWMM versus the PCSWMM runoff hydrographs routed in the model, on average the VO_2 routed flow peaks are 120% of the PCSWMM routed hydrographs: the maximum peak flow node difference is 213%, and the minimum peak flow difference is 95%.

Table: 6.2 Comparing Timmins Future Peak Flows at Key Nodes

Node*		Flow from Timmins Storm in m3/s		
		AECOM (2010) CN (III)	Kawartha Conservation CN*(III)	
			PCSWMM	VO2
1	Start of system	N/A	8.26	7.92
2	Commerce Rd culvert		8.18	7.77
3	Former train track culvert		8.15	7.72
4	McLaughlin Rd culvert		8.20	7.79
5	Ball field culvert	9.6	12.71	12.21
6	Greenfield Rd culvert		12.90	15.88
7	Hwy 7 culvert	9.8	18.16	21.21
8	W of Hwy 7 @ 90-degree bend in drain	12.0	18.10	38.45
9	North of 90-degree bend	14.0	18.16	36.38
10	Between Dew Drop Inn Rd & Hwy 7	28.5	40.27	54.09
11	D/S of Hwy 7	30.0	38.27	50.32
12	North of Hwy 7 past 90-degree bend	25.7	28.98	31.70
13	Colborne St culvert	30.6	27.61	36.85
14	D/S of Colborne St	32.7	24.10	36.83
15	D/S of private driveway	34.3	24.02	24.39
16	U/S of airport	39.2	63.24	61.37
17	D/S of Hwy 35 (near airport)	62.7	63.16	61.01
18	Angeline St culvert	78.7	72.99	69.49
19	D/S of Angeline St	109.0	84.46	80.90
20	William St culvert	111.0	85.24	81.68

** U/S = upstream, D/S = downstream
**Note: Refer to Figure 5.5 for map of Key Nodes*

7 Recommendations for Flow Inputs to Hydraulic Model

7.1 For the Static HEC RAS Model

The results of the new PCSWMM hydrological model for Ops Drain #1/Jennings Creek are reasonable and the best estimate of flow and were therefore input to a hydraulic model to establish new Regulatory flood lines for the watershed. Since PCSWMM routed catchment runoff hydrographs downstream, the flow peaks account for channel and flood plain attenuation.

For both the existing and future land use conditions, the upstream urban portion of the watershed experiences higher peak flows from the 100-year Chicago storm. Downstream of Highway 7 however, the Timmins storm provides higher peaks. This is shown schematically in **Figure 7.1**. **Table 7.1** shows the representative peak flows for the existing land use while **Table 7.2** shows the representative peak flows for future land use.

Table 7.1: Input Flows to Static HEC-RAS Model for Existing Land Use

Node Location in the Drainage System*		Flow (m3/s)
1	Start of system	19.24
2	Commerce Rd culvert	16.26
3	Former train track culvert	15.41
4	McLaughlin Rd culvert	14.11
5	Ball field culvert	18.24
6	Greenfield Rd culvert	16.27
7	Hwy 7 culvert	19.26
8	W of Hwy 7 @ 90-degree bend in drain	17.24
9	North of 90-degree bend	17.11
10	Between Dew Drop Inn Rd & Hwy 7	39.13
11	D/S of Hwy 7	37.24
12	North of Hwy 7 past 90-degree bend	29.09
13	Colborne St culvert	27.85
14	D/S of Colborne St	22.86
15	D/S of private driveway	24.07
16	U/S of airport	61.08
17	D/S of Hwy 35 (near airport)	60.56
18	Angeline St culvert	69.33
19	D/S of Angeline St	80.16
20	William St culvert	80.94

** U/S = upstream, D/S = downstream

**Note: Refer to Figure 5.5 for map of Key Nodes*

Figure 7.1: Dominant Peak Flow Locations in Study Area

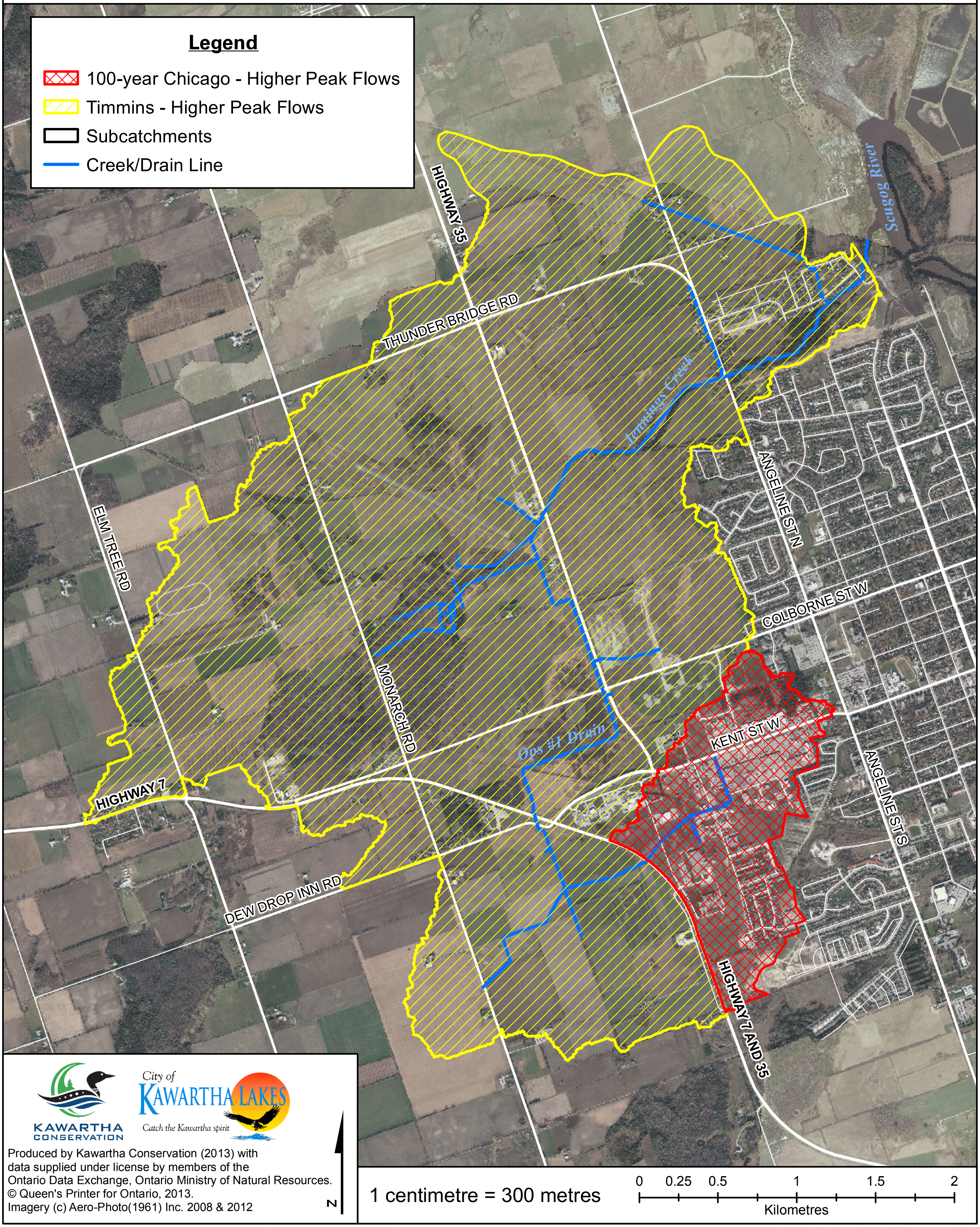

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Table 7.2: Input Flows to Static HEC-RAS for Future Land Use

Node Location in Drainage System*		Existing Flows (m3/s)	Future Flows (m3/s)
1	Start of system	19.24	23.01
2	Commerce Rd culvert	16.26	19.15
3	Former train track culvert	15.41	17.78
4	McLaughlin Rd culvert	14.11	16.20
5	Ball field culvert	18.24	22.24
6	Greenfield Rd culvert	16.27	19.34
7	Hwy 7 culvert	19.26	26.22
8	W of Hwy 7 @ 90-degree bend in drain	17.24	25.05
9	North of 90-degree bend	17.11	20.42
10	Between Dew Drop Inn Rd & Hwy 7	39.13	40.27
11	D/S of Hwy 7	37.24	38.27
12	North of Hwy 7 past 90-degree bend	29.09	28.98
13	Colborne St culvert	27.85	27.61
14	D/S of Colborne St	22.86	24.10
15	D/S of private driveway	24.07	24.02
16	U/S of airport	61.08	63.24
17	D/S of Hwy 35 (near airport)	60.56	63.16
18	Angeline St culvert	69.33	72.99
19	D/S of Angeline St	80.16	84.46
20	William St culvert	80.94	85.24

** U/S = upstream, D/S = downstream

Note: Refer to Figure 5.5 for map of Key Nodes

It is recommended that the values from **Table 7.1** and **Table 7.2** be used as input to the static HEC-RAS hydraulic model. The future flows will be used to create flood plain maps since their values are higher than existing flows.

7.2 For the Dynamic HEC RAS Model

The runoff hydrographs generated in PCSWMM for all 16 catchments were input directly to the appropriate cross-sections in the Dynamic HEC RAS model. For the future 100-year flood line calculations, the 12-hour Chicago storm hydrographs at 10-minute time steps were used. Future Timmins hydrographs at a 10-minute time step were also input to the dynamic HEC RAS model. Since HEC-RAS includes roadways and their culverts, the flow attenuation calculated by HEC-RAS differs from the PCSWMM attenuation since not only channel and flood plain attenuation were included, but also attenuation caused by roadways and their structures.

8 Hydraulic Model Parameters

8.1 Cross Sections

Cross-sections were created as previously explained in section 4.2. 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.

8.2 Culvert and Road Crossings

Cross-sections were cut at culvert crossings, bridges and other restricting structures to accurately represent channel flow. All culvert crossings are represented by two upstream and two downstream bounding cross sections. Representative deck elevations were extracted from the DEM.

All culverts were field surveyed to ensure accuracy. Invert elevations, height/width dimensions, length, and channel bottom were surveyed using either total station and/or RTK GPS survey equipment. All relevant data was noted and photographed, and can be found in **Appendix N**.

8.3 Expansion/Contraction Coefficients

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

8.4 Manning's n Values

As previously outlined in section 4.2, 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 G.** The main channel n values range from **.**035 to .04 and the overbank n values range from .016 to 0.1. It is noted that these values change in the interpolated sections created in the Jennings Creek portion of the dynamic model. HEC-RAS automatically calculates values for Manning's n when creating interpolated sections, based on values from bounding sections. In the dynamic model where interpolated sections were added, the main channel n values range from 0.035 to 0.1; the overbank n value range did not change.

8.5 Ineffective Flow Elevations

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

8.6 Boundary Conditions

For the static model, mixed flow analyses (including both sub- and supercritical flow regimes) were run for all scenarios. The downstream boundary condition is the starting water surface elevation. To be consistent with previous models (AECOM and Greck), the starting water surface elevation is the controlled Sturgeon Lake water level of 247.76m. It is noted that Sturgeon Lake is at a point greater than 1 kilometre downstream of the Jennings Creek outlet to the Scugog River. The upstream boundary condition is the normal depth based on the 0.1% slope of the channel.

For the dynamic model, the downstream boundary condition is normal depth based on the 1% slope of Jennings Creek. The upstream boundary condition is the runoff hydrograph from the Kent Street commercial catchment.

8.7 Flows

The input flows to the static HEC-RAS models are the PCSWMM model output listed in **Table 7.1** and **Table 7.2**.

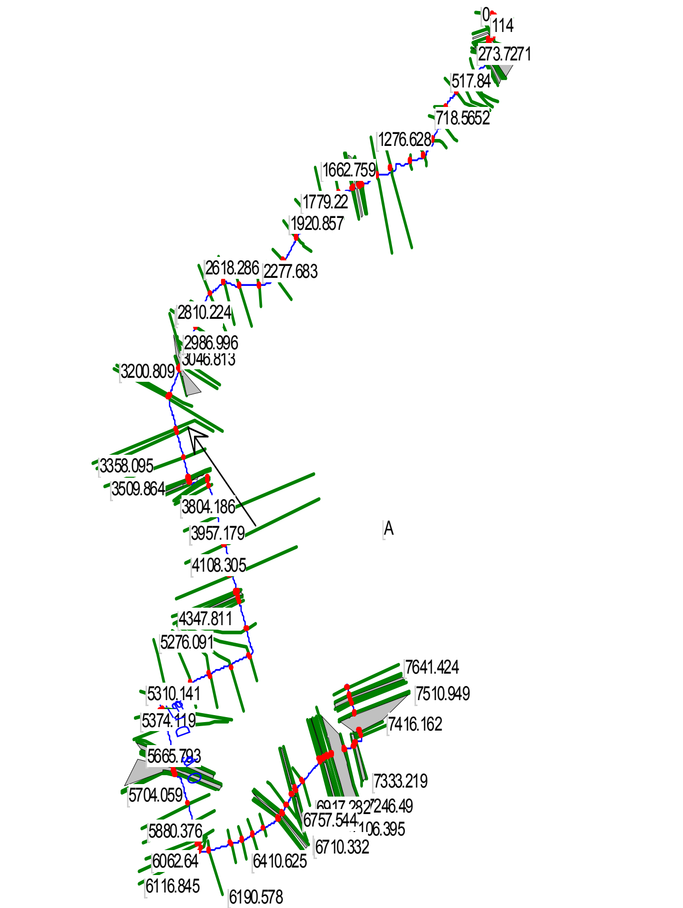
The PCSWMM catchment runoff hydrographs were input directly to the appropriate cross-sections in the dynamic HEC RAS model. For the future 100-year flood line calculations, the 12-hour Chicago storm hydrographs at 10-minute time steps were used. Future Timmins hydrographs at a 10-minute time step were used as input to the Dynamic HEC RAS model.

9 Hydraulic Model

9.1 Schematic

The information gathered in the preceding section was used to build a HEC-RAS model of the watercourses. The geometry of the model is shown schematically in **Figure 9.1**.

Figure 9.1: HEC-RAS Schematic



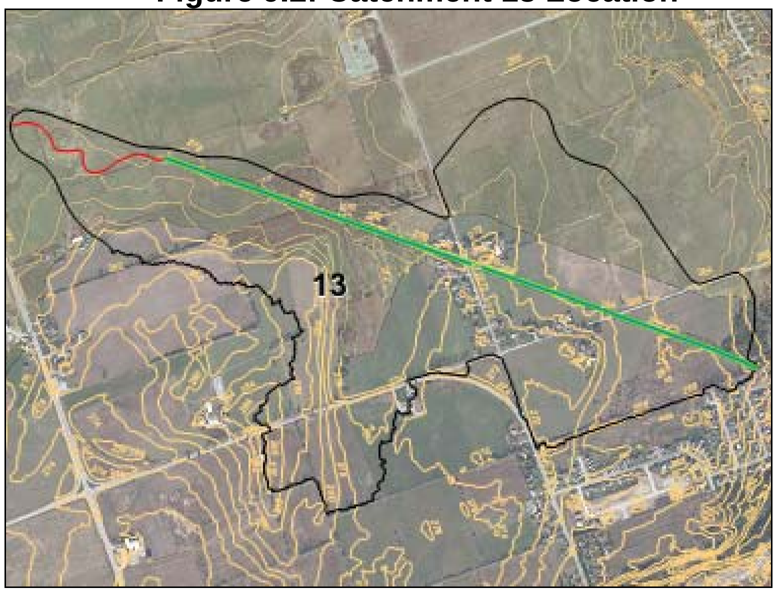
9.2 Sensitivity Analyses

The static HEC RAS model was tested for sensitivity to the Manning's n, starting water surface elevation, increase/decrease of recorded rainfall volumes, and reduction in tributary area. **Appendix O** has detailed information on these analyses.

REMOVAL OF CATCHMENT 13

As mentioned previously in section 3.7, the updated hydrologic model includes a new catchment (Catchment 13, shown in **Figure 9.2** below) that had not been included in the previous Aquafor Beech and AECOM studies. A ditch inlet catch basin captures flow into a pipe that runs through Springdale Garden Park and outlets to Jennings Creek immediately east of 36 Champlain Boulevard. For flow in excess of the pipe capacity, the exact connection of the major flow of Catchment 13 to the watercourse is unknown since LIDAR data did not fully cover area. Visual confirmation of a flow route was not possible due to dense vegetation. The connection point has been conservatively modeled as connecting to Jennings Creek in the vicinity east of 36 Champlain Boulevard. A sensitivity analysis was carried out to determine if removing Catchment 13 from the hydrology model would have an impact on peak flow rates in Jennings Creek.

Figure 9.2: Catchment 13 Location



The hydrology model was modified by removing Catchment 13, and the difference in peak flows in Jennings Creek was negligible (0.04% change in flow peaks). This is due to the fact that the hydrograph peak from Catchment 13 does not coincide with the flow peak in the main channel. The location of Catchment 13 inflow therefore has no impact on water surface elevations in Jennings Creek.

INCREASING MANNING'S N VALUE BY 20%

The Manning's number indicates the friction factor in a cross-section. The higher the number, the rougher is the surface against which water flows. For instance, a smooth concrete pipe has a Manning's n of 0.013 whereas a forest has a Manning's n value of 0.1.

By increasing the Manning's numbers by 20%, the flow is being subjected to a watershed with higher friction forces acting upon it. It was found that overall there is little impact to the calculated water surface elevations. Although 48% of the cross-sections experienced a rise in water surface elevations, most of those were less than 5cm. The largest changes in elevation were either in the agricultural field west of Highway 7, in the agricultural field east of the airport or in the deep valley portions of the Jennings Creek watercourse, where there is no impact to buildings.

DECREASING MANNING'S N VALUE BY 20%

By decreasing the Manning's numbers by 20%, the flow is being subjected to a watershed with lower friction forces acting upon it. It was found that overall there is little impact to the calculated water surface elevations. Although 34% of the cross-sections experienced a drop in water surface elevation, most of those were less than 5cm. Again, the largest changes in elevation were in the agricultural field west of Highway 7, in the agricultural field east of the airport or in the deep valley portions of the Jennings Creek watercourse, where there is little impact.

STARTING WATER SURFACE ELEVATION

The model was modified using different starting water surface elevations. As previously mentioned in Section 8.6, the base model uses the controlled Sturgeon Lake water level of 247.76m. An alternate water surface elevation of 248.4m was also used, which is the recorded 100-year Sturgeon Lake level.

It was discovered that the initial water surface elevation has little effect on the flood elevations. The program defaulted to critical depth for the initial water surface elevation. It appears that this is due to the fact that Sturgeon Lake is at too great a distance downstream of the watercourse outlet at the Scugog River. An attempt was made to find a more valid recorded water level for the Scugog River in the vicinity of the outlet. Municipal sewage and water treatment plants are located on the river. It was hoped that City staff would have a record of average water levels in the river that could be input to the model, but Kawartha Conservation was informed that no such recordings are kept by the City of Kawartha Lakes.

10% INCREASE IN RECORDED RAINFALL VOLUME

To account for the possibility of climate change, an analysis was carried out to see the effect of increasing the recorded volumes of the Lindsay AES rain gauge by 10%. The PCSWMM hydrology model was modified to account for this 10% volume increase, and the routed peak flows were extracted for input to static HEC-RAS model. The revised water surface elevations are higher in two locations: in the commercial area upstream of Hwy 7, and in the agricultural lands between Angeline St and Hwy 35. Only the commercial area is of interest, however, since the flood plain downstream of Hwy 35 is determined by the Timmins storm, not the 100-year event. In the commercial area, the Regulatory flood elevation increases by up to 0.12m due to the 10% increase in rain volumes.

10% DECREASE IN RECORDED RAINFALL VOLUME

To account for the possibility of climate change, an analysis was carried out to see the effect of decreasing the recorded volumes of the Lindsay AES rain gauge by 10%. The PCSWMM hydrology model was modified to account for this 10% volume decrease, and the routed peak flows were extracted for input to static HEC-RAS model. The only change in water surface elevation occurs in the agricultural lands between Angeline St and Hwy 35; the revised flood line is 0.49m lower. However, since the flood plain downstream of

Hwy 35 is determined by the Timmins storm and not the 100-year event, reducing the rain volume by 10% has no impact on Regulatory flood plain extents.

USING PETERBOROUGH RECORDED AES RAIN GAUGE VOLUME

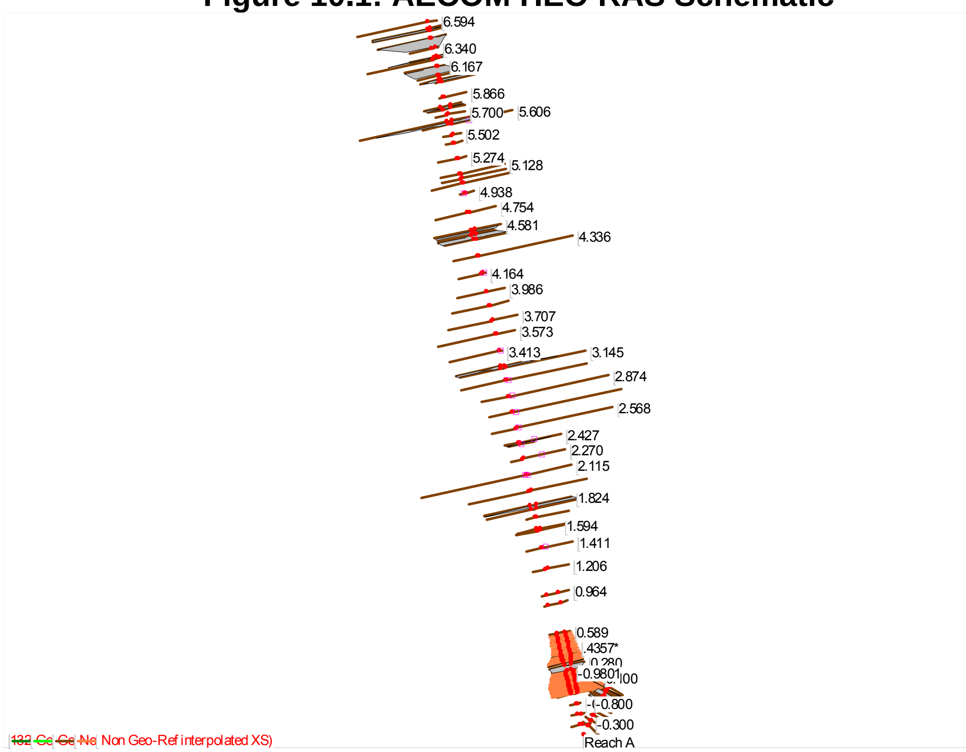
Because of the significant change in water surface elevation caused by a 10% increase in rainfall volume, an analysis was carried out using the Peterborough AES rain gauge data in the models to determine the impact. The Peterborough gauge has noted an increase in rainfall volumes in recent years. Because the Lindsay gauge has been decommissioned by the AES, the Lindsay data does not reflect recent increases. The PCSWMM hydrology model was modified and the routed peak flows were extracted for input to static HEC-RAS model. There was no change in the 100-year calculated flood elevation in the Lindsay commercial district. As a result, it has been determined that the using Lindsay rain gauge data is appropriate for the study.

10 HEC RAS Model Results

10.1 Comparing Model Data Input

Ops #1 Drain/Jennings Creek were modeled in 2010 by AECOM. As discussed in previous sections, the City of Kawartha Lakes acquired LiDAR elevation data for this project. Due to this greatly improved data set, significant differences exist between the 2010 and 2013 models.

Figure 10.1: AECOM HEC-RAS Schematic



BASE DEM

The model established by Kawartha Conservation is geo-referenced. There are no digital CAD or GIS files for the AECOM flood maps. As a result, it is difficult to make a map-based comparison of flood extents or elevations. General comparisons are possible by using descriptions such as road boundary delimiters.

CROSS-SECTIONS

The static hydraulic model includes 125 sections, whereas the AECOM model consisted of 84 original surveyed cross-sections and 50 interpolated cross-sections. Of the 50 interpolated sections:

- 9 consecutive sections were created over a 197m portion of Ops #1 Drain based on bounding sections 0.589 and 0.37
- 13 consecutive sections were created over a 60m portion of Jennings Creek based on bounding sections 0.37 and 0.305
- 19 consecutive sections were created over a 95m portion of Jennings Creek based on bounding sections 0.1 and 0

The static model consists of 50% more cross-sections than the AECOM model, all of which are taken from the DEM. This confirms the new model contains accurate, up-to-date, and reliable elevation data.

OBSTRUCTIONS

As previously mentioned in section 4.2, the models include building obstructions in the cross-sections. HEC-RAS does not include the area represented by the building obstructions when calculating the area and volume available for flow in each cross-section.

The AECOM model did not include building obstructions.

As an example of the difference in cross-section detail, Figure 10.2 below shows a cross-section through the Lindsay Mall north parking lot, as represented by the AECOM model; Figure 10.3 shows the representation of the parking lot in Kawartha Conservation's HEC-RAS model.

Figure 10.2: AECOM Model Cross-section

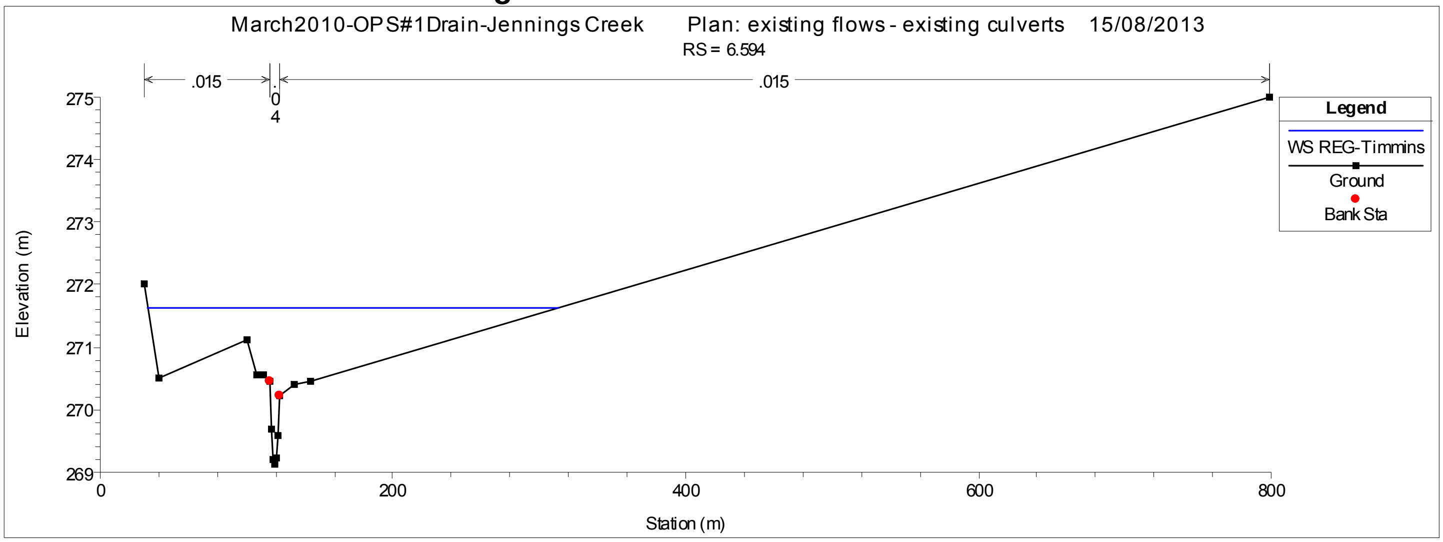
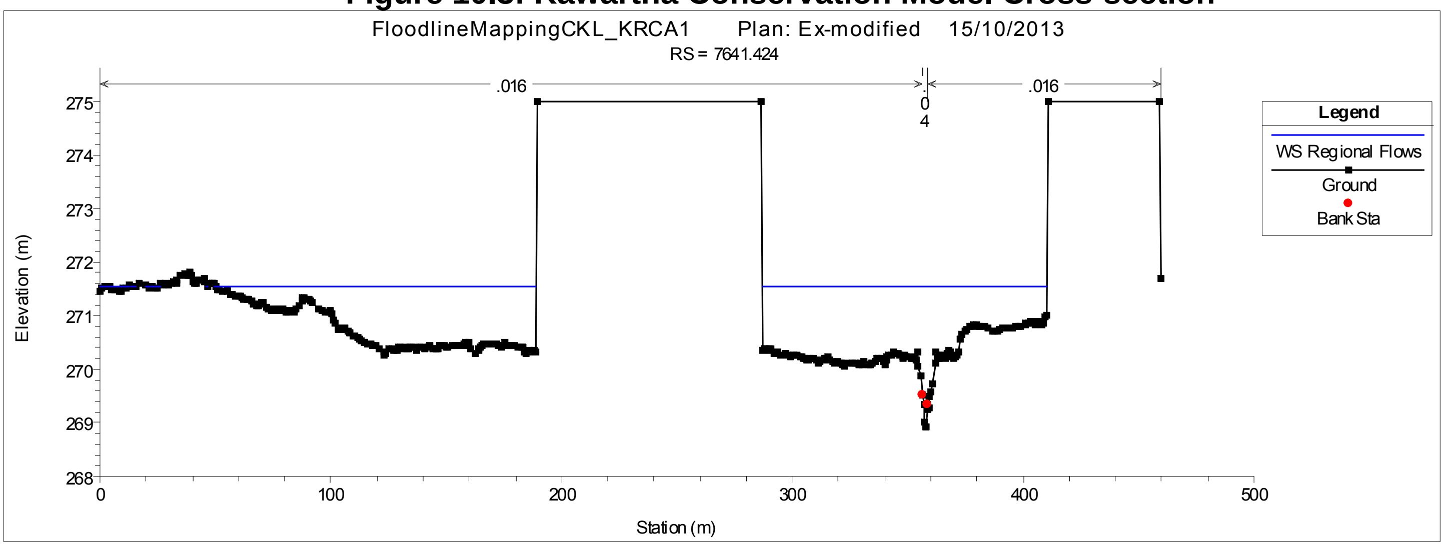


Figure 10.3: Kawartha Conservation Model Cross-section



FLOW INPUT

Both the Kawartha Conservation and AECOM hydrology models determined that the full build-out condition of the watershed results in higher flows than for the existing land use condition. As discussed previously in this report, due to the greater accuracy and breadth of data available in the new DEM data, the input flows in the Kawartha Conservation static HEC-RAS model are different than what was used in the AECOM model. In general, peak flows in the upstream portion of the watershed are higher than what was used by AECOM, while in the lower section of the watershed peak flows are lower, as shown in **Table 10.1** and **Table 10.2**. More details can be found in **Appendix P**.

Table 10.1: Regulatory Peak Flows for Static Model (Existing Land-use Conditions)

Node*		Flows in m^3/s	
		AECOM	Kawartha Conservation
1	Start of system	N/A	19.24
2	Commerce Rd culvert	N/A	16.26
3	Former train track culvert	N/A	15.41
4	McLaughlin Rd culvert	N/A	14.11
5	Ball field culvert	11.8	18.24
6	Greenfield Rd culvert	N/A	16.27
7	Hwy 7 culvert	12.8	19.26
8	W of Hwy 7 @ 90-degree bend in drain	13.7	17.24
9	North of 90-degree bend	25.2	17.11
10	Between Dew Drop Inn Rd & Hwy 7	30.1	39.13
11	D/S of Hwy 7	30.9	37.24
12	North of Hwy 7 past 90-degree bend	30.8	29.09
13	Colborne St culvert	28.7	27.85
14	D/S of Colborne St	30.7	22.86
15	D/S of private driveway	35.6	24.07
16	U/S of airport	58.6	61.08
17	D/S of Hwy 35 (near airport)	74.7	60.56
18	Angeline St culvert	105.0	69.33
19	D/S of Angeline St	95.9	80.16
20	William St culvert	N/A	80.94

** U/S = upstream, D/S = downstream

**Note: Refer to Figure 5.5 for map of Key Nodes*

Note: flows in the initial 9 rows represent the 100-year peak flows. Rows 10-20 represent the Timmins peak flows

Table 10.2: Comparing Regulatory Flood Flows (Future Build-out)

Node*		Flows in m^3/s AECOM	Flows in m^3/s Kawartha Conservation
1	Start of system	N/A	23.01
2	Commerce Rd culvert	N/A	19.15
3	Former train track culvert	N/A	17.78
4	McLaughlin Rd culvert	N/A	16.20
5	Ball field culvert	11.4	22.24
6	Greenfield Rd culvert	N/A	19.34
7	Hwy 7 culvert	12.3	26.22
8	W of Hwy 7 @ 90-degree bend in drain	13.4	25.05
9	North of 90-degree bend	24.9	20.42
10	Between Dew Drop Inn Rd & Hwy 7	30.0	40.27
11	D/S of Hwy 7	25.7	38.27
12	North of Hwy 7 past 90-degree bend	30.6	28.98
13	Colborne St culvert	32.7	27.61
14	D/S of Colborne St	34.3	24.10
15	D/S of private driveway	39.2	24.02
16	U/S of airport	62.7	63.24
17	D/S of Hwy 35 (near airport)	78.7	63.16
18	Angeline St culvert	109.0	72.99
19	D/S of Angeline St	111.0	84.46
20	William St culvert	N/A	85.24

** U/S = upstream, D/S = downstream

**Note: Refer to Figure 5.5 for map of Key Nodes*

The dynamic model uses flow hydrographs, not peak flows, to calculate flooding extents. Since the AECOM study did not include a dynamic model, a comparison of flow values is not possible.

MANNING'S n VALUES

HEC-RAS requires unique values assigned to the overbanks and channel. In over 80% of the cross-sections, the new model's main channel n values are the same as AECOM's values. It is in the overbank areas where n values differ significantly; over 80% of the overbanks have values that are different. The new model has an average n value increase of 15%, ranging from a 70% decrease to a 190% increase. As mentioned previously in section 8.4, the interpolated sections created in the dynamic model modified the channel n values slightly. More details can be found in **Appendix G.**

10.2 Comparing Static HEC-RAS Model Output (Kawartha Conservation vs. AECOM)

The flood plain elevations calculated in the AECOM report were based solely on future land use conditions, assuming all land was fully built-out based on then-current Official Plan (OP) and Secondary Plan land use. The flood plain elevation comparison between AECOM's and this study's models are therefore only for future land use conditions. **Table 10.3** below showcases the differences in the AECOM and this study's flood elevations as calculated by HEC-RAS. More detailed information can be found in **Appendix Q**.

Table 10.3: Comparing Regulatory Flood Elevations (for Future Land use Conditions

Location	HEC-RAS Cross Section #	Flood Elevation (m)	
		AECOM	Kawartha Conservation
Lindsay Mall parking lot	7598	271.78	271.54
Commerce Rd	7506	271.78	271.54
Former train track	7328	271.78	271.53
McLaughlin Rd	7238	271.52	271.53
Ball field	7106	271.58	271.53
Greenfield Rd	6853	271.58	271.52
Hwy 7	6706	271.57	271.52
West of Hwy 7	6676	269.52	269.93
At 90-degree bend	6335	269.46	268.79
Dew Drop Inn Rd	5690	269.40	268.43
Hwy 7	5646	268.10	268.42
At 90-degree bend	5374	268.07	268.07
Colborne St	4380	267.95	268.03
Private laneway	3640	267.94	268.02
Hwy 35	3035	267.94	268.01
East of Hwy 35	3000	266.86	266.78
At 45-degree bend	2618	266.78	266.39
Angeline St	1448	266.69	265.99
East of Angeline St	1407	263.29	262.54
Elaine Drive	1021	257.64	257.02
William St	267	256.22	254.35
Former train track	220	256.21	254.33
Rail Trail	144	256.20	254.28
Scugog River	0	249.86	248.77

A mapped flood line comparison is not possible since digital flood lines are not available for the AECOM study. In general, the Kawartha Conservation model calculated lower flood elevation values than the AECOM model.

10.3 **Comparing Static HEC-RAS Model Output (Kawartha Conservation vs. Greck)**

The Greck model used the same future flows as the AECOM model. The Greck model modified the AECOM geometry file by including elevation data from two recent topographical surveys. The Manning's n values were also different than this study's model values. This study's overbank n values ranged from 0.035 to 0.1, whereas the Greck model overbank values were consistently 0.08. Both models used the same value for the channel (0.035). The difference in flood elevations is highlighted in **Table 10.4** below.

Table 10.4: Comparing Regulatory Flood Elevations

Location	HEC-RAS Cross Section #	Flood Elevation (m)	
		Kawartha Conservation	Greck
Angeline St	1448	265.99	266.33
east of Angeline St	1407	262.54	262.54
Elaine Drive	1021	257.02	258.38
William St North	267	254.35	254.84
Former Train Track	220	254.33	254.82
Rail Trail	144	254.28	254.81
Downstream of Rail Trail	114	249.82	250.49

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

10.4 **Comparing Existing and Future Flood Plain Extents for Static HEC-RAS Model**

The full build-out condition of the watershed results in higher flows than the existing land use condition. Therefore, it seems reasonable to assume that the full build-out will result in a deeper, wider flood plain. This study calculates two flood plains to confirm whether this assumption is valid: one flood plain was based on the existing land use flows, the second was based on the future land use flows. **Table 10.5** below lists the flow values used at key nodes**.**

Table 10.5: Comparing Existing and Future Land Use Regulatory Flows and Flood Elevations

Node		Flows in m3/s		Flood Elevation (m)	
		Existing	Future	Existing	Future
1	Start of system	19.24	23.01	271.56	271.54
2	Commerce Rd culvert	16.26	19.15	271.54	271.54
3	Former train track culvert	15.41	17.78	271.52	271.53
4	McLaughlin Rd culvert	14.11	16.20	271.52	271.53
5	Ball field culvert	18.24	22.24	271.51	271.53
6	Greenfield Rd culvert	16.27	19.34	271.50	271.52
7	Hwy 7 culvert	19.26	26.22	271.50	271.52
8	W of Hwy 7 @ 90-degree bend in drain	17.24	25.05	268.42	268.43
9	North of 90-degree bend	17.11	20.42	268.42	268.43
10	Between Dew Drop Inn Rd & Hwy 7	39.13	40.27	268.42	268.42
11	D/S of Hwy 7	37.24	38.27	268.05	268.07
12	North of Hwy 7 past 90-degree bend	29.09	28.98	268.01	268.03
13	Colborne St culvert	27.85	27.61	268.01	268.03
14	D/S of Colborne St	22.86	24.10	268.01	268.03
15	D/S of private driveway	24.07	24.02	268.00	268.02
16	U/S of airport	61.08	63.24	267.99	268.01
17	D/S of Hwy 35 (near airport)	60.56	63.16	266.73	266.78
18	Angeline St culvert	69.33	72.99	265.94	265.99
19	D/S of Angeline St	80.16	84.46	262.45	262.54
20	William St culvert	80.94	85.24	254.31	254.35

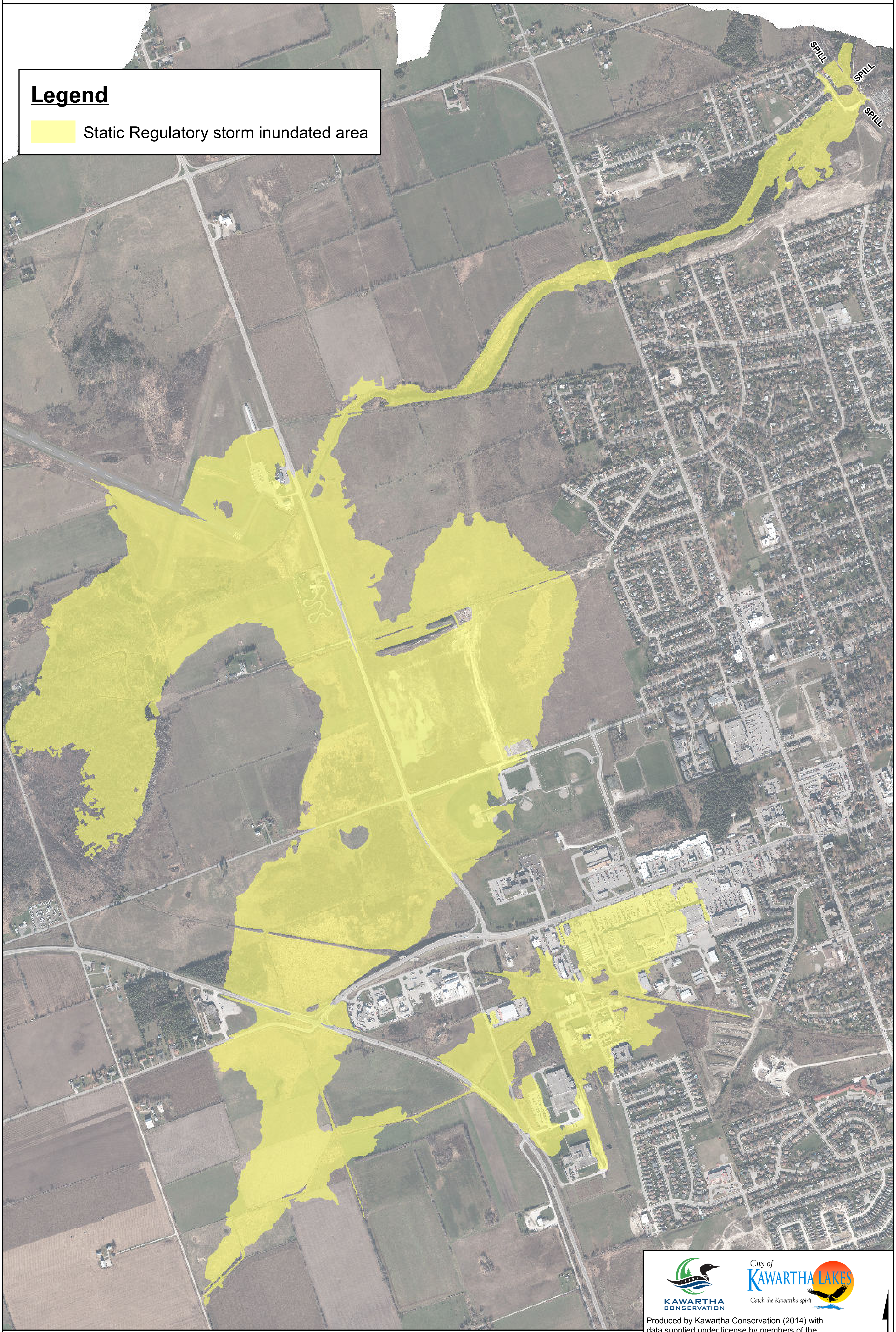
** U/S = upstream, D/S = downstream

Note: Refer to Figure 5.5 for map of Key Nodes

It was discovered that the calculated flood elevations did not differ significantly. The average increase in water surface elevation is 0.03m when using future land use flows. In one location in particular the water surface elevation fluctuates significantly; this is in the agricultural land immediately west of Hwy 7 at Pickseed. The increase is local, and dissipates within 350m of the highway.

Since the differences are too minute to be seen on a flood plain map, this report will produce only one map representing future land use flows. The extent of the flood plain is shown in **Figure 10.4**. The general profile is shown in **Figure 10.5**.

Figure 10.4: Flood Plain



1 centimetre = 125 metres

0 250 500 1,000
Metres

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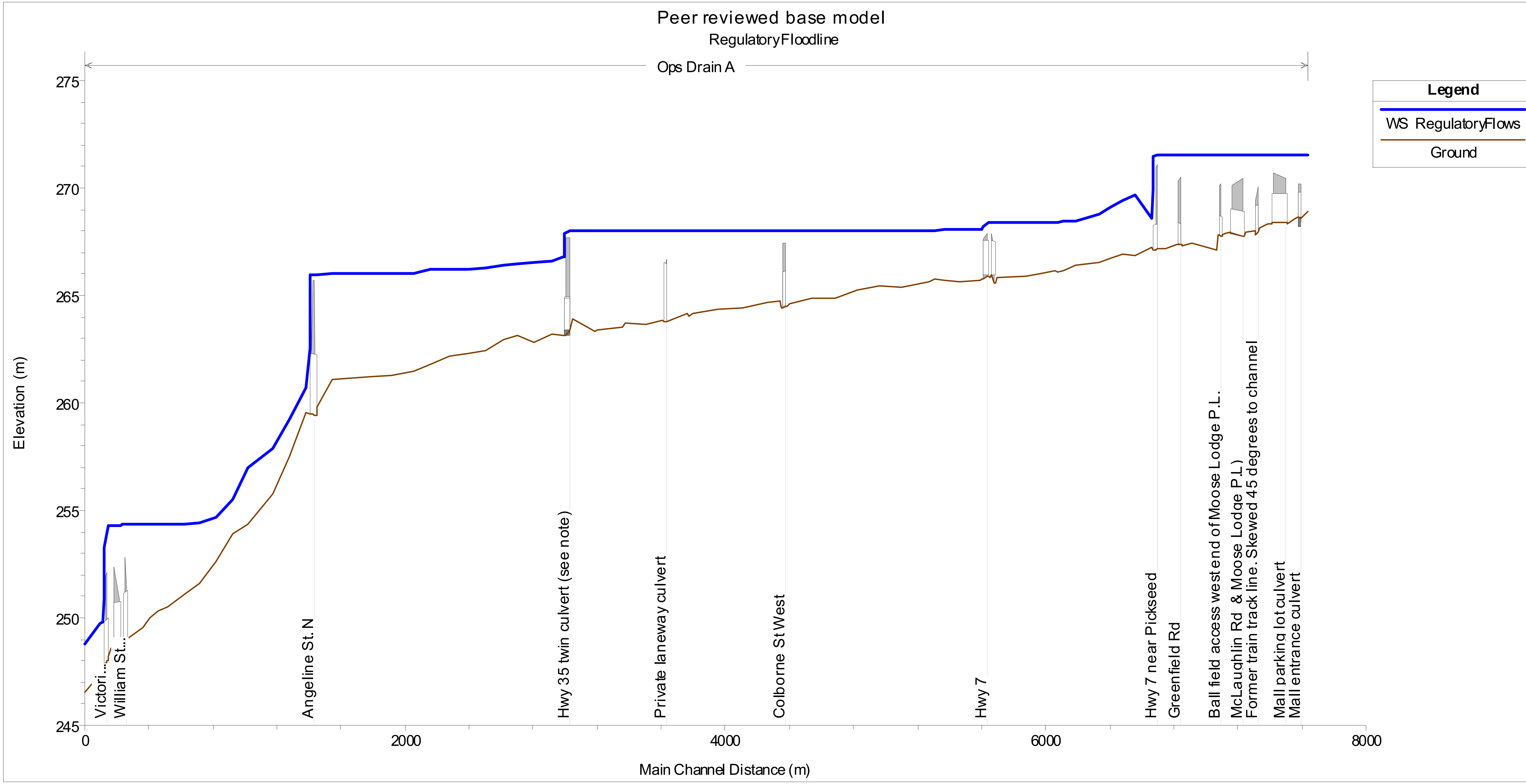
Figure 10.5: Regulatory Profile – Static



10.5 Spill

A spill occurs near the outlet of Jennings Creek near William St North. In the left overbank, water flows north in the depressed area between the Rail Trail and the houses on the east side of William St North. Detailed topographic information is not available to the north of this spill area, so this study was unable to determine precisely where it flows toward the Scugog River. On the right overbank, water spills in a south-easterly direction toward the Scugog River in two locations. Flows in the model are not reduced in the downstream cross-sections to be conservative.

10.6 Comparing Static and Dynamic Model Output for the 100-year Storm

The geometry file used in the dynamic model is the same geometry file from the static model; this ensures that the only difference between models is the flow (peak flows vs. hydrographs). As stated in Section 7.2, the future land use conditions flows were input to HEC-RAS for hydraulic analysis. For model stability issues, some hydrographs had to be input uniformly over a range of cross-sections instead of at a single cross-section.

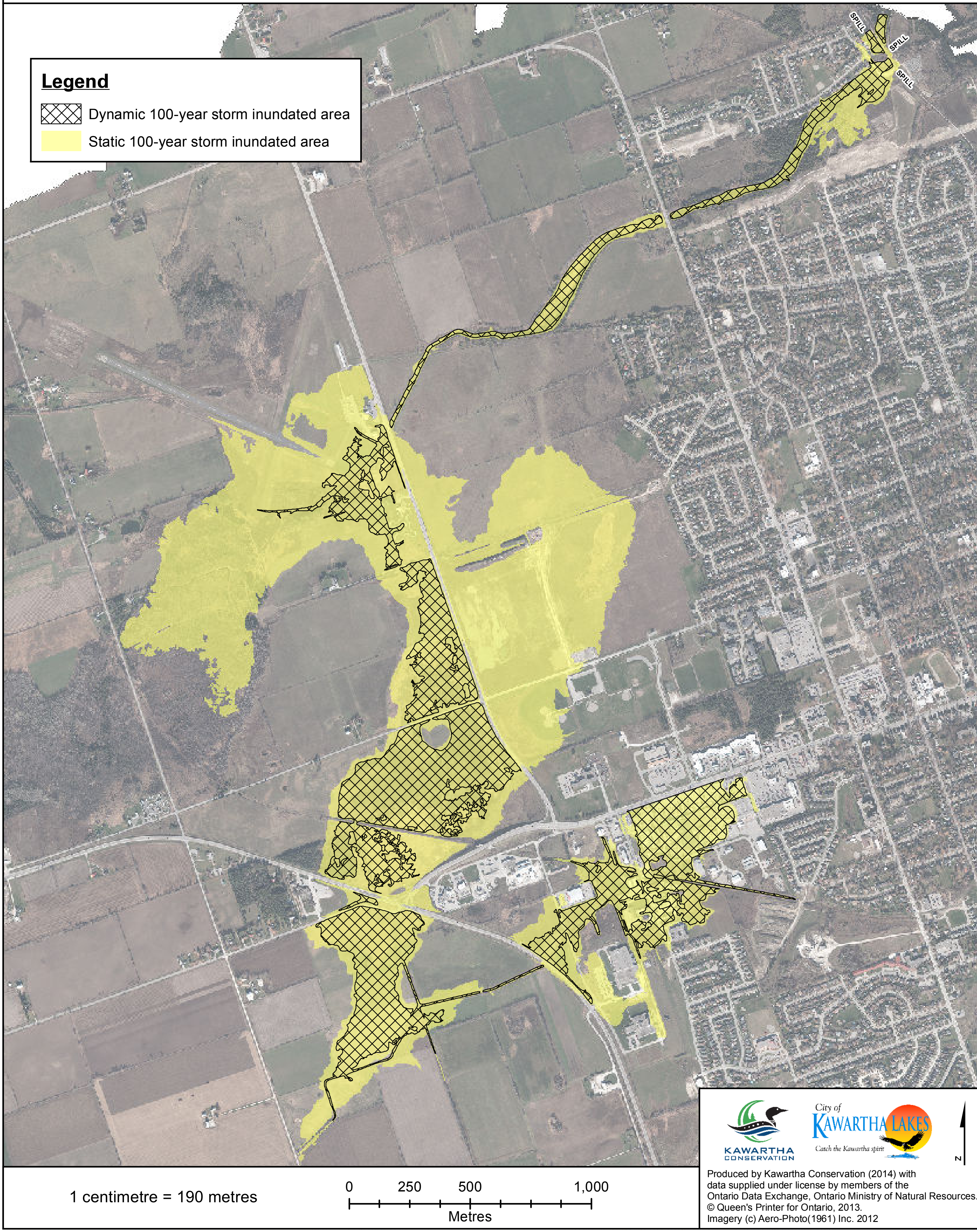
The water surface elevations calculated by the two models are compared in **Table 10.6** below. On average the flood elevation calculated by the dynamic model is 0.87m lower than the static model. There are two reasons why there is a drop in flood elevation. The flow value associated with the maximum water surface elevation in the dynamic model is not necessarily the peak flow of the hydrograph and this flow is much less than the flow peak in the static model. The second reason is due to the nature of dynamic modeling; roads and culverts restrict flows, therefore the maximum flow of the flood wave downstream of an obstruction will be lower than the flow peaks in the static model.

The flood plains delineated by the two models have the same general shape, as can be seen in **Figure 10.6**. More details can be found in **Appendix Q**.

Table 10.6: Comparing 100-year Flows and Flood Elevations for Static and Dynamic Models

Location	HEC-RAS Cross Section #	Flow Peak (m^3/s)		Flood Elevation (m)	
		Static	Dynamic	Static	Dynamic
Lindsay Mall parking lot	7598	23.01	6.05	271.54	271.39
Commerce Rd	7506	19.15	6.00	271.54	271.39
Former train track	7328	17.78	5.79	271.53	271.39
McLaughlin Rd	7238	16.20	5.27	271.53	271.03
Ball field	7106	22.24	5.02	271.53	270.90
Greenfield Rd	6853	19.34	6.25	271.52	270.89
Hwy 7	6706	26.22	5.47	271.52	270.39
Dew Drop Inn Rd	5690	20.42	8.85	268.41	267.92
Hwy 7	5646	33.48	8.85	268.37	267.52
Colborne St	4380	16.71	9.26	267.87	266.86
Private laneway	3640	15.70	4.70	267.87	266.67
Hwy 35	3035	43.20	25.98	267.87	266.54
Angeline St	1448	54.49	25.96	264.77	262.02
William St	267	62.60	23.93	254.30	253.19
Former train track	220	62.60	23.89	254.29	253.18
Rail Trail	144	62.60	24.82	254.27	253.08
Scugog River	0	62.60	24.80	248.58	248.25

Figure 10.6: Comparing 100-year Dynamic and Static Model Floodlines



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10.7 Comparing Static and Dynamic Model Output for the Timmins Storm

As with the 100-year model, the same geometry file was used in the dynamic Timmins model as in the static model; this ensures that the flows reflect the only difference in the models. The water surface elevations calculated by the two models are outlined in **Table 10.7** below. On average the flood elevation calculated by the dynamic model is 0.52m lower than the static model. The flood plains delineated by the two models have the same general shape, as can be seen in **Figure 10.7**. More details can be found in **Appendix Q**.

Table 10.7: Comparing Timmins Water Surface Elevations for Static and Dynamic Models

Location	HEC-RAS Cross Section #	Flow Peak (m^3/s)		Flood Elevation (m)	
		Static	Dynamic	Static	Dynamic
Lindsay Mall parking lot	7598	8.26	8.27	271.49	271.41
Commerce Road	7506	8.17	7.52	271.48	271.40
Former Train Track	7328	8.14	7.29	271.48	271.39
McLaughlin Rd	7238	8.20	2.78	271.47	271.06
Ball Field	7106	12.70	4.43	271.47	271.07
Greenfield Rd	6853	12.88	5.43	271.47	271.07
Hwy 7	6706	18.14	6.31	271.47	271.06
Dew Drop Inn Rd	5690	18.15	18.7	268.42	268.40
Hwy 7	5646	40.27	18.69	268.42	268.38
Colborne St	4380	27.61	12.05	268.03	267.67
Private Laneway	3640	24.10	12.01	268.02	267.40
Hwy 35	3035	63.24	32.18	268.01	267.37
Angeline St	1448	72.99	35.75	265.99	262.64
William St	267	84.46	48.81	254.35	254.42
Former Train Track	220	84.46	48.81	254.33	254.41
Rail Trail	144	85.24	35.31	254.28	254.25
Scugog River	0	85.24	49.64	248.77	248.54

10.8 Summary of HEC RAS Output

The flood elevations calculated in this study are generally lower than what had been calculated in previous reports. There is greater confidence in these calculated flood elevations since the source elevation data used in the hydrology and hydraulic models is far more detailed than what had been available for the previous reports.

A comparison of the static HEC-RAS flood elevations for existing versus future land use conditions shows that the future scenario yields elevations that are slightly higher (by 0.03m on average).

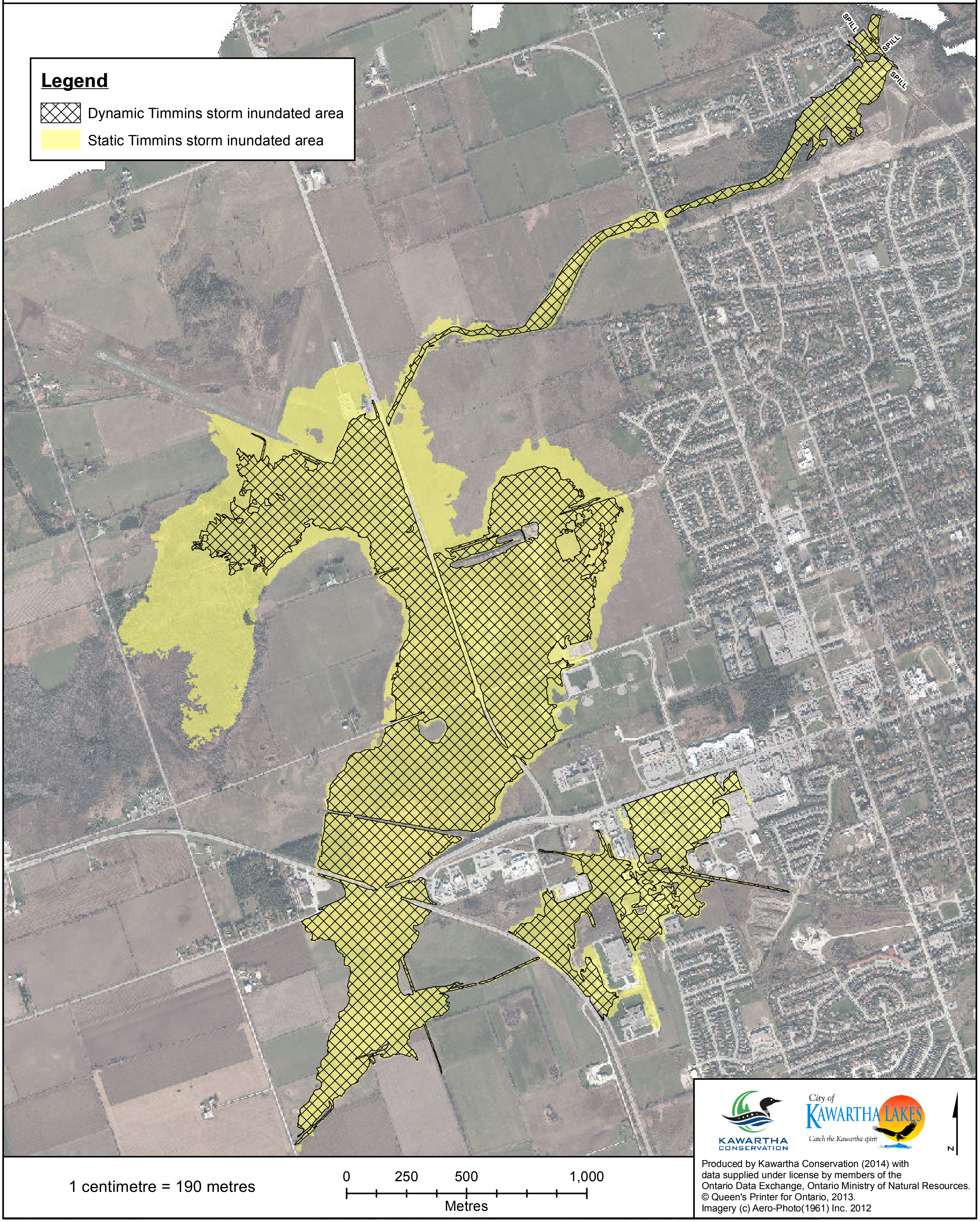
The hydrographs generated in the future land use condition hydrology model were input to HEC-RAS for the 100-year and Timmins dynamic models. A comparison of the 100-year static versus dynamic flood elevations shows that the water surface elevations for the dynamic model are lower than the static flood elevations (by an average of 0.87m). Similarly, the dynamic Timmins flood elevation is 0.52m lower than that calculated in the static Timmins model.

As stated in Section 10.6, roads and culverts restrict flow in dynamic modeling and attenuate the flood wave downstream of these obstructions. A static model does not attenuate the flood wave caused by obstructions. The MNR has stated that downstream of a culvert or bridge the natural flood line should be used for delineating the flood hazard, making no allowance for temporary upstream ponding. In other words, the flood plain map should represent both the flooding that occurs upstream of a culvert due to a roadway crossing, as well as the flooding that would result if the stream flow were not restricted by the roadway. MNR policy does not allow flood plains to be reduced due to road/culvert attenuation, for several reasons:

- Any future culvert and/or road improvement would increase the downstream flood plain.
- There is no guarantee that a roadway would remain in place during a flood event. The road and/or culvert could wash out and the downstream flows would not be attenuated

For these reasons, it is recommended that the static water surface elevation for future land use conditions be used for flood plain mapping. Reduced flood plain maps are located at the back of the report.

Figure 10.7: Comparing Timmins Dynamic and Static Model Floodlines



11 Impact of Future Developments

11.1 Future Development Lands Northeast of Colborne/Hwy 35 Intersection

To the east of Hwy 35, north of Colborne St, are lands slated for residential and commercial use. This study carried out an analysis to determine the impact on flood lines and flood storage if this land were removed from the flood plain. The HEC-RAS geometry was modified by assigning levees to appropriate stations along Hwy 35 to simulate the removal of the right overbank; flow was therefore restricted to west of the levees. The same cross-sections were altered to reflect in PCSWMM, which was re-run to get new flow peaks for the static HEC-RAS model. Since the dynamic HEC-RAS model uses the PCSWMM catchment runoff hydrographs for flow input; no PCSWMM modifications were necessary for the dynamic scenario. The resultant flows are compared in **Table 11.1** below.

Table 11.1: Comparing Static Model Peak Flows with Introduced Levees

Location	HEC-RAS Cross Section #	100-year Flows (m^3/s)		Timmins Flows (m^3/s)	
		No levee	Levee	No levee	Levee
West of Hwy 7	6676	26.22	26.74	18.14	18.14
at 90-degree bend	6190	25.02	26.74	18.09	18.14
Dew Drop Inn Rd	5690	20.42	20.58	18.15	18.15
Hwy 7	5646	33.48	33.67	40.27	40.69
at 90-degree bend	5374	32.03	32.19	38.27	38.79
Colborne St	4380	16.71	16.74	27.61	21.57
Private Laneway	3640	15.70	15.77	24.10	13.61
Hwy 35	3035	43.20	43.14	63.24	65.38
east of Hwy 35	3000	43.20	43.14	63.24	65.38
at 45-degree bend	2618	41.24	41.20	63.16	64.02
Angeline St	1448	54.49	54.44	72.99	73.45

It is intuitive to think that by removing large areas of land from the flood plain, the flood elevation would go up significantly in the narrower cross-section. However, for the static models, this is not the case. Upon closer investigation of the model output, for the cross-sections whose right overbanks lie within the affected land, the water is very slow-moving. By removing the right overbank area, water is shifted to the left overbank and velocity increases. The flow of water in the area therefore remains the same, with the result that there is no increase in flood elevation. The resulting changes in flood elevations for the static 100-year model are seen in **Table 11.2**. Key cross-sections where the flow area changes significantly are shaded for emphasis.

Table 11.2: Comparing 100-year Static Flood Elevations by Removing Land East of Hwy 35

Location	Flood Elevation (m)		Flow Area (m2)		Left Overbank Velocity (m/s)		Right Overbank Velocity (m/s)	
	No L	L	No L	L	No L	L	No L	L
at 90-degree bend	267.91	267.91	460.1	464.97	0.05	0.05	0.07	0.08
Colborne St (4546)	267.88	267.89	467.0	453.8	0.04	0.04	0.11	n/a
Colborne St (4263)	267.87	267.88	1088.2	426.1	0.01	0.03	0.02	n/a
Colborne St (4108)	267.87	267.88	1244.5	323.1	n/a	0.04	0.02	n/a
Colborne St (3957)	267.87	267.88	1056.6	221.8	0.01	0.07	0.01	n/a
Private laneway	267.87	267.87	126.0	126.3	0.10	0.10	0.11	n/a
Hwy 35	267.87	267.87	379.3	380.1	0.17	0.17	0.08	0.08
at 45-degree bend	265.53	265.53	22.7	22.7	1.53	1.53	1.36	1.36
Angeline St	264.77	264.76	164.3	163.9	n/a	n/a	n/a	n/a

It is beyond the scope of this study to suggest or investigate how these levees can be achieved in the field, and further engineering analysis is required to support this concept.

Resulting changes in flood elevations for the static and dynamic models are seen in **Table 11.3**. For the static models, there is a minimal increase in flood elevation (a maximum of 0.02m). Since the flood plain maps will be based on the output from the static model, it is feasible that development of this land could be entertained. This is discussed further in section 12. For the dynamic model however, there are significant increases in the flood elevations: there is an average 0.15m and 0.21m increase for the 100-year and Timmins dynamic models, respectively.

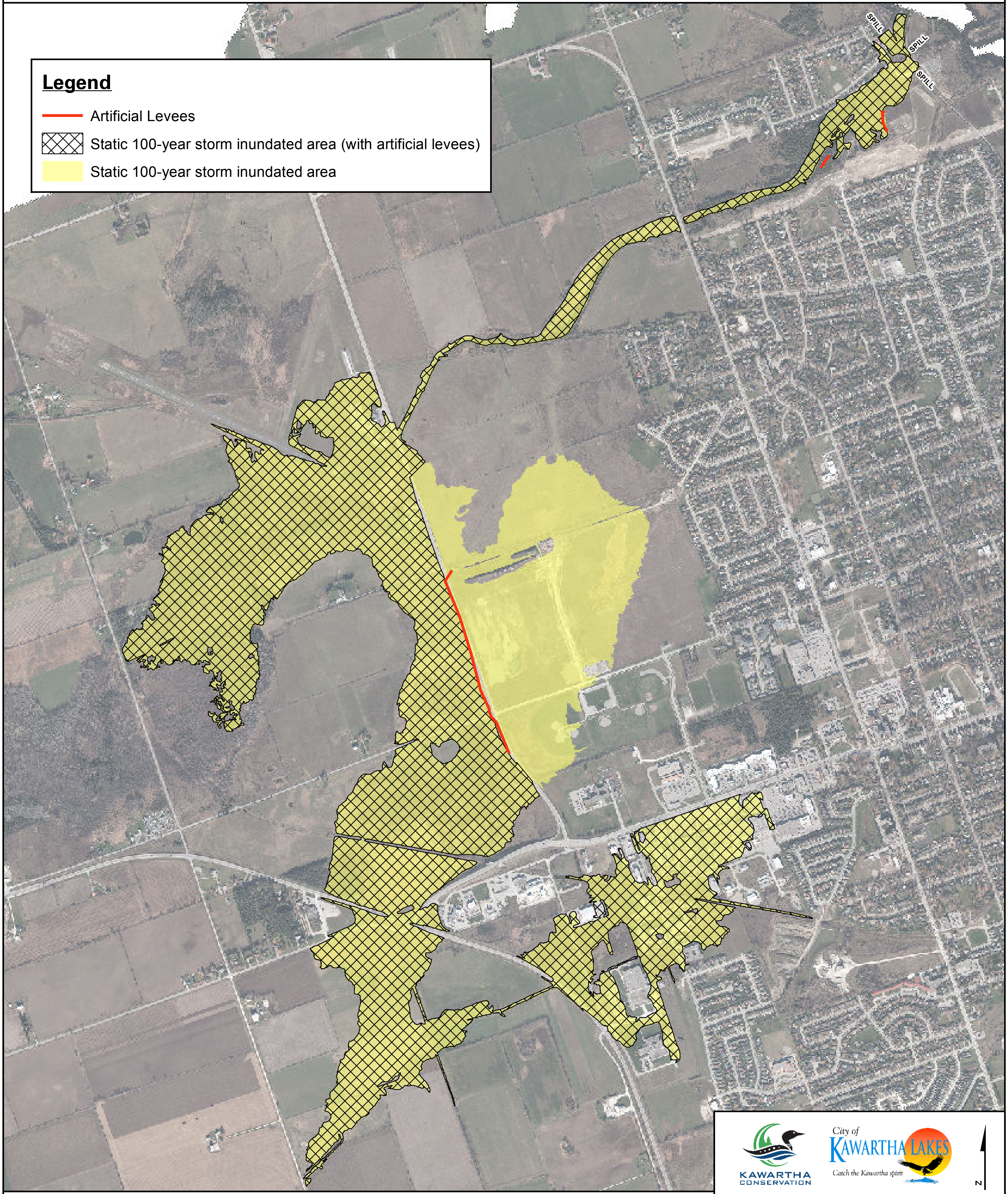
Table 11.3: Comparing Model Flood Elevations with Filling East of Hwy 35

Location	100-year Static Flood Elevation (m)		100-year Dynamic Flood Elevation (m)		Timmins Static Flood Elevation (m)		Timmins Dynamic Flood Elevation (m)	
	No	L	No	L	No	L	No	L
at 90-degree bend	267.91	267.91	267.02	267.10	268.07	268.08	267.68	267.75
Colborne St (4546)	267.88	267.89	266.88	267.03	268.03	268.05	267.67	267.74
Colborne St (4263)	267.87	267.88	266.71	266.89	268.03	268.04	267.41	267.73
Colborne St (4108)	267.87	267.88	266.71	266.89	268.03	268.04	267.41	267.72
Colborne St (3957)	267.87	267.88	266.71	266.88	268.03	268.04	267.41	267.72
Private laneway	267.87	267.87	266.67	266.87	268.02	268.04	267.40	267.72
Hwy 35	267.87	267.87	266.54	266.77	268.01	268.03	267.37	267.71
at 45-degree bend	265.53	265.53	264.87	264.95	266.39	266.40	265.37	265.46
Angeline St	264.77	264.76	262.02	262.04	265.99	265.99	262.64	262.70

Figure 11.1 highlights 100-year static flood plain changes whereas **Figure 11.2** shows the Timmins static flood plain changes. As mentioned previously, for the static models there is no significant change in water surface elevation. There is therefore no change in flood plain extents shown on **Figure 11.1** and **Figure 11.2.**

Figure 11.3 and **Figure 11.4** compare the 100-year and Timmins dynamic flood plain changes, respectively. These figures show that the flood plain is wider in the vicinity of the proposed development.

Figure 11.1: Change in 100-year Static Flood Plain due to Removal of Overbank East of Hwy 35



1 centimetre = 190 metres

0 250 500 1,000
Metres

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Figure 11.2: Change in Timmins Static Flood Plain due to Removal of Overbank East of Hwy 35

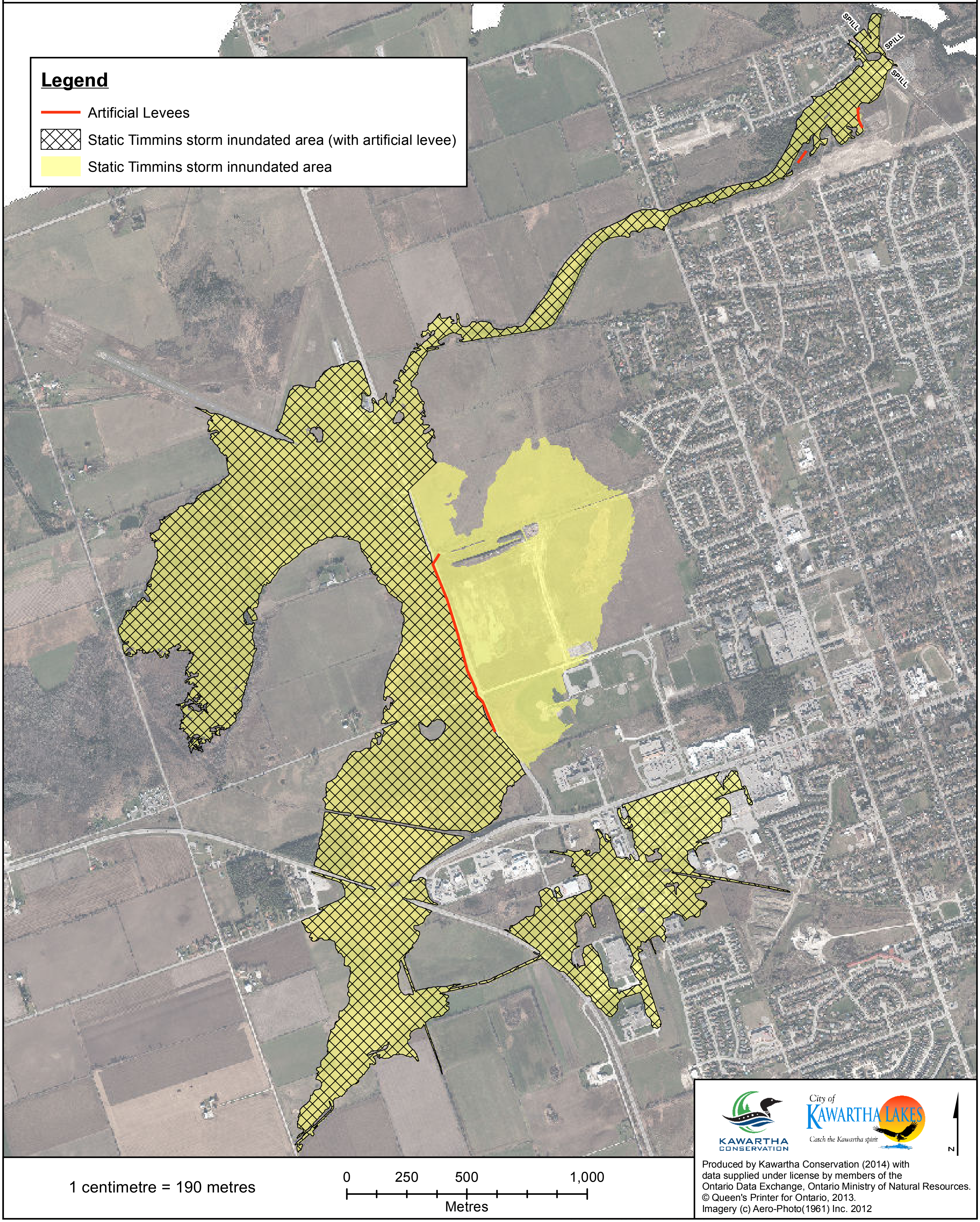

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Figure 11.3: Change in 100-year Dynamic Flood Plain due to Removing Overbank East of Hwy 35

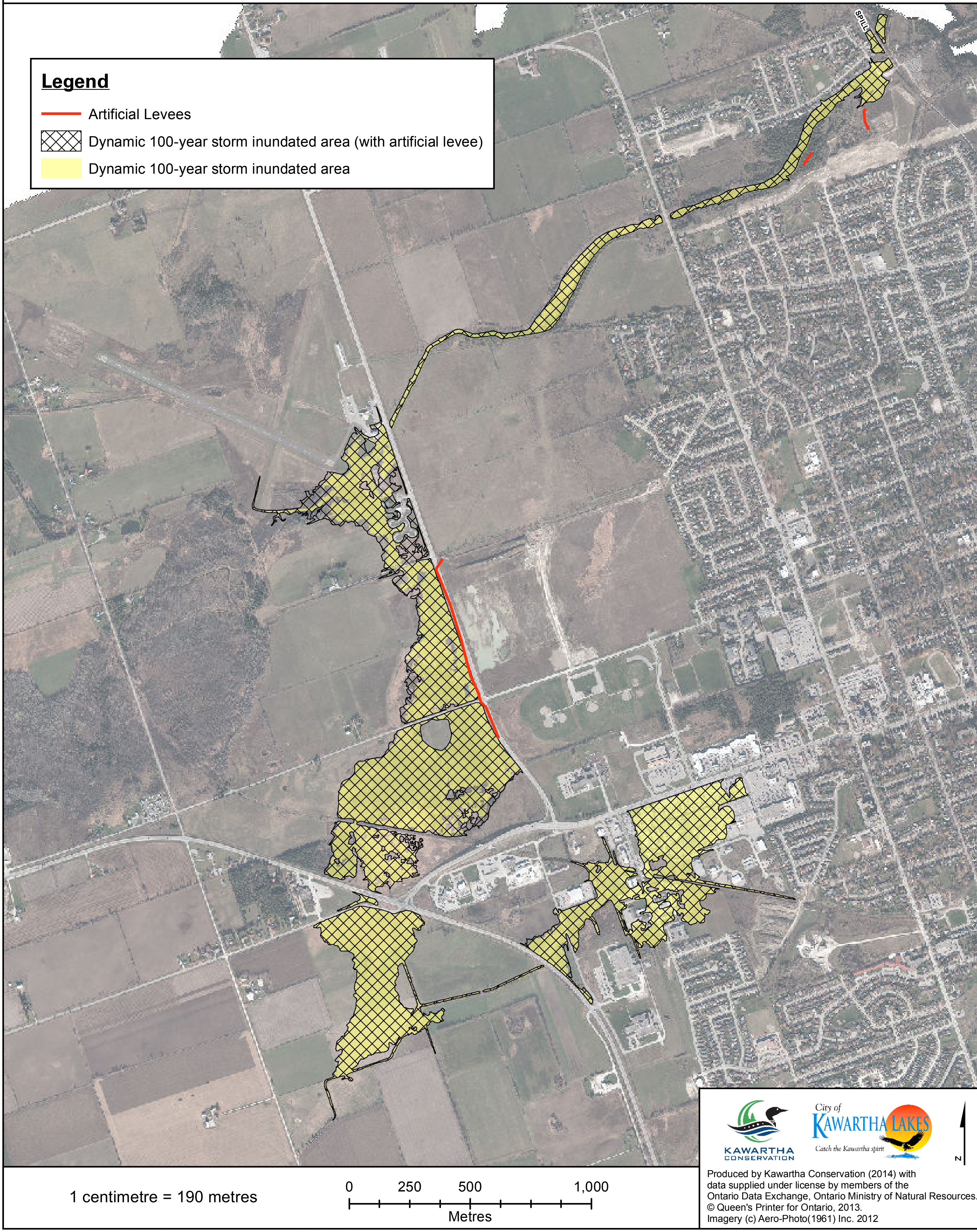

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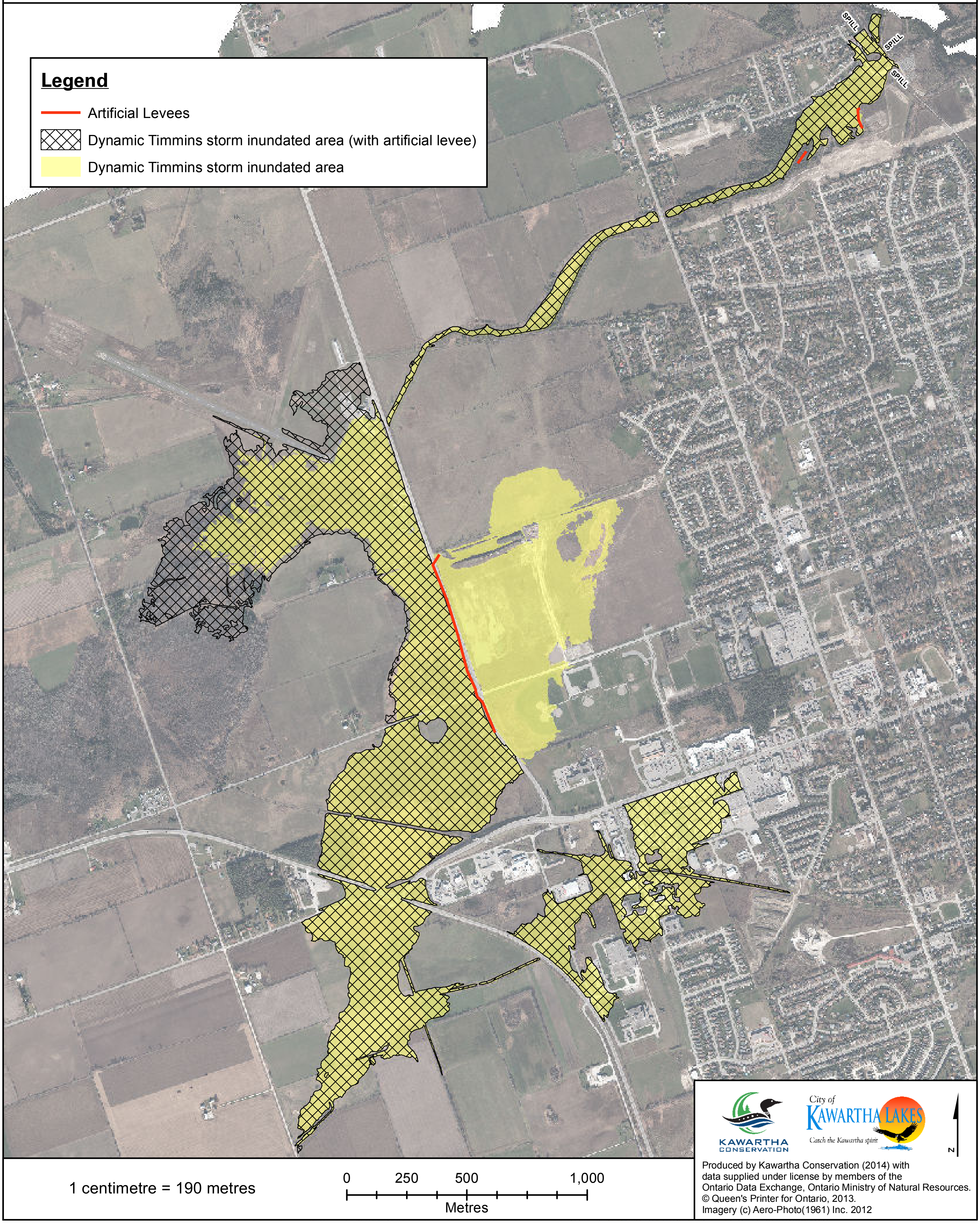
Figure 11.4: Change in Timmins Dynamic Flood Plain by Removing Overbank East of Hwy 35



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11.2 Subdivision at William St. North

Near the outlet at the Scugog River lies vacant land that is currently undergoing an Ontario Municipal Board (OMB) appeal for a potential subdivision. An analysis was carried out to determine the impact on flood lines and flood storage if some of the right overbank were removed from the flood plain.

Since grading plans were not available, the HEC-RAS geometry file was modified by assigning levees to appropriate stations to simulate the removal of the right overbank; flow was therefore restricted to the channel and overbank north of the levees. PCSWMM was re-run with the cross-section restrictions to get new flow peaks for the static HEC-RAS model. The dynamic HEC-RAS model uses the PCSWMM catchment runoff hydrographs for flow input; no PCSWMM modifications were necessary for the dynamic scenario.

The resulting change in flood elevations can be seen in **Table 11.4** below. At this location in the watershed, the Timmins storm is the critical event, thus the table reports only the Timmins flood elevations.

Figure 11.5 and **Figure 11.6** show the areas where flood plain has been removed and added due to these model variations.

Table 11.4: Comparing Timmins Flood Elevations with Some Removal of the Right Overbank

Location	HEC-RAS Cross Section #	Timmins Static Flood Elevation (m)		Timmins Dynamic Flood Elevation (m)	
		No levee	Levee	No levee	Levee
Angeline St	1448	265.99	265.99	262.64	262.70
East of Angeline St	1407	262.54	262.56	260.40	260.40
Elaine Drive	1021	257.02	257.02	256.31	256.34
William St	267	254.35	254.36	254.42	254.44
Former train track	220	254.33	254.34	254.41	254.42
Rail Trail	144	254.28	254.30	254.25	254.26
Scugog River	0	247.77	248.77	248.54	248.57

As can be seen in **Table 11.4**, **Figure 11.5**, and **Figure 11.6**, the simple preliminary analyses indicated there would be minimal impact to either the static or dynamic models' flood elevations caused by removing the right overbank. It is recommended if land development were to be considered in this area, more detailed engineering analyses should be carried out as part of the development review process to confirm there would be no impact to the flood plain (i.e. cut/fill analyses, modification to HEC-RAS model using proposed grading plans.

Figure 11.5: Change in Timmins Static Flood Plain by Removing Portions of Right Overbank

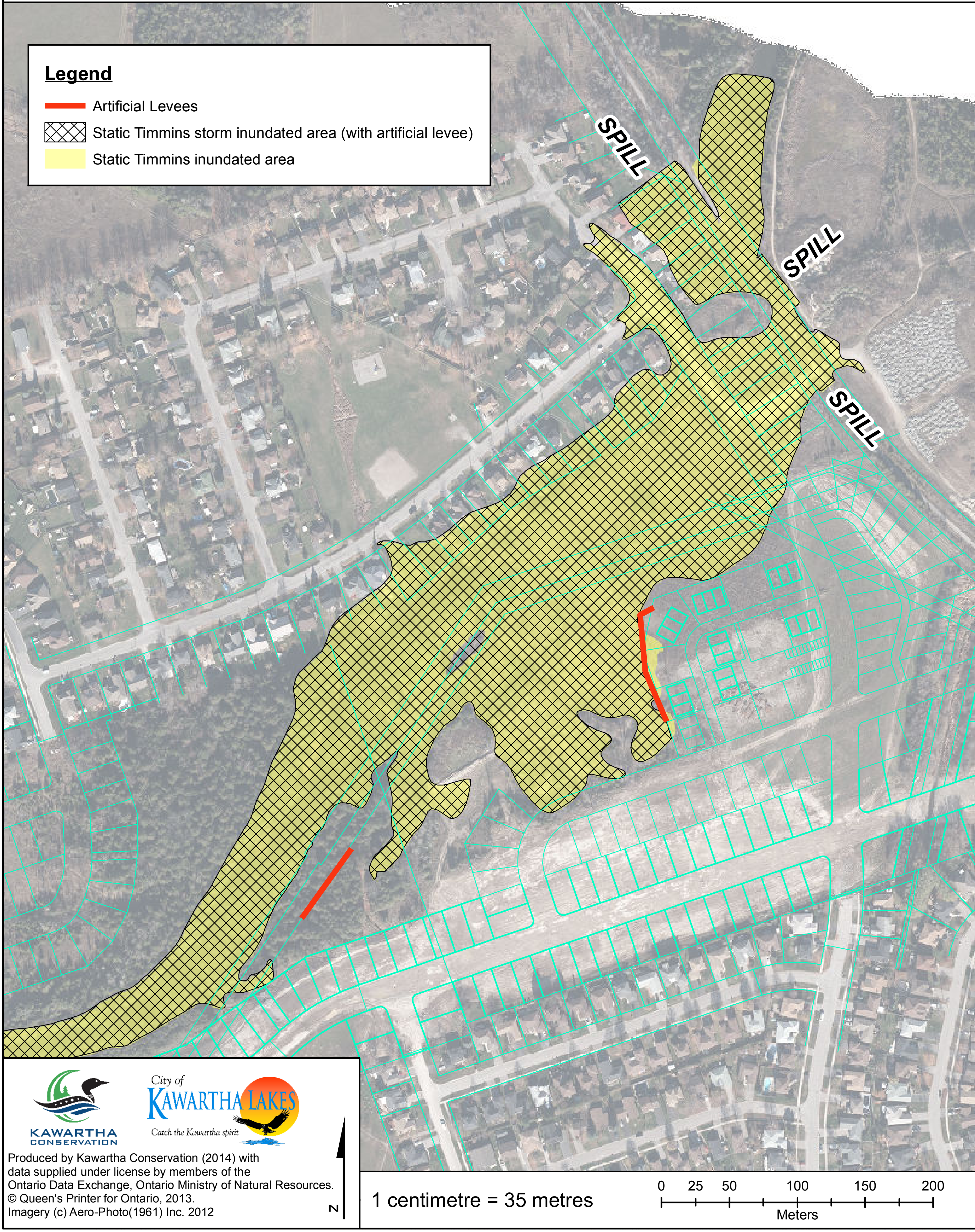

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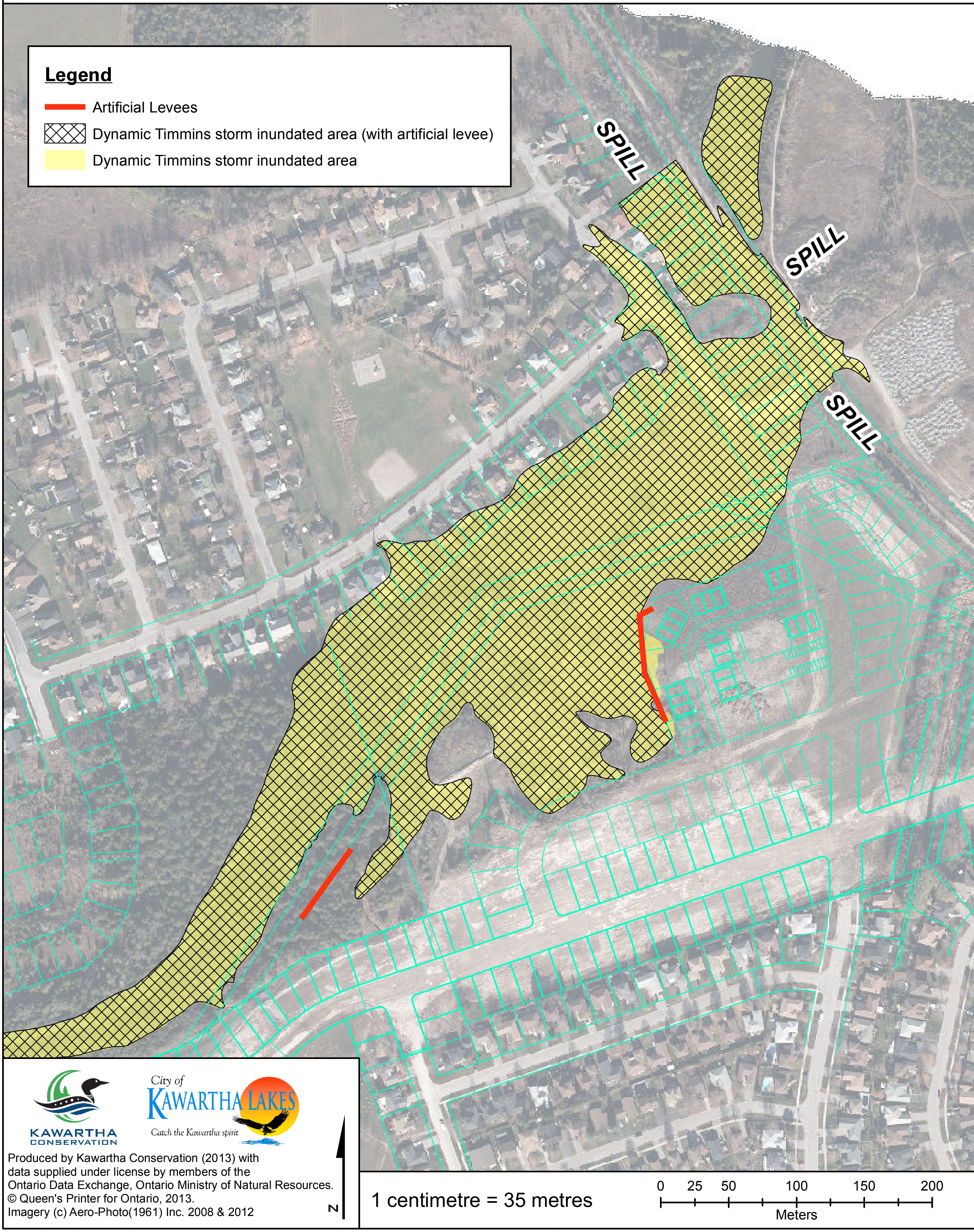
Figure 11.6: Change in Timmins Dynamic Flood Plain by Removing Portion of Right Overbank



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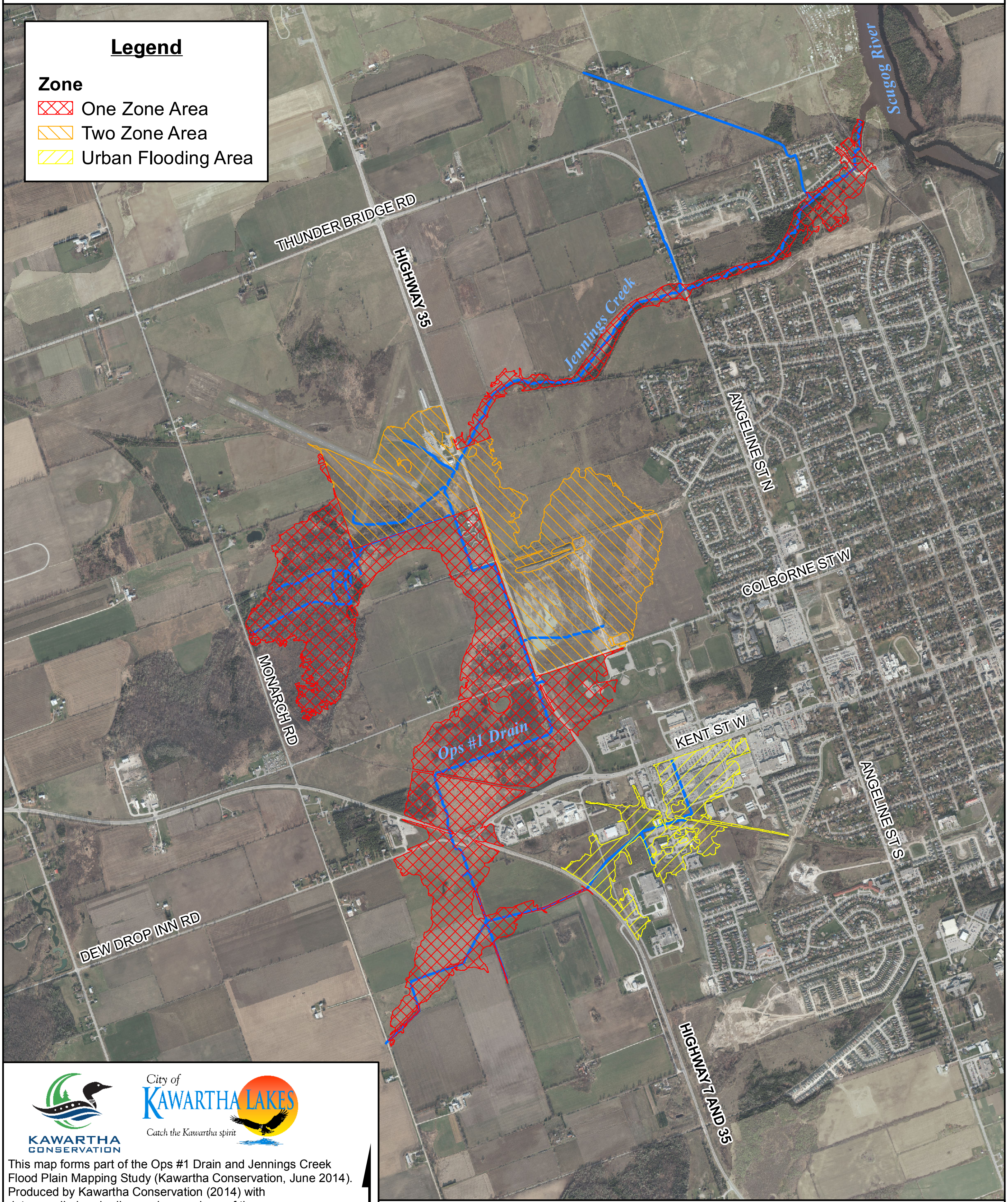
12 Recommendations for Flood Hazard Policy

As mentioned in Section 1 of this report, the July 2011 Greck study recommended two distinct methods of minimizing threats to public safety and property damage for proposed development within flood-prone areas: the one-zone and two-zone concepts. The one-zone concept states that development or site alteration would be prohibited within the floodplain (the entire flooding hazard limit). In the two-zone concept, the flood hazard limit is defined with an inner floodway (where development is prohibited or restricted) and outer flood fringe areas (between the floodway and the flood hazard limit) where some development may be permitted.

Based on the results of the HEC-RAS model for this study, for most of the Ops #1 Drain/Jenning's Creek flood plain it is recommended that the one-zone concept be used. The exception would be the land east of Hwy 35 between Colborne Street and the Ops # 1 Drain. Currently it is within the flood plain but as discussed in Section 11, preliminary analyses show that if the right overbank were removed there would be insignificant impact to flood elevations and velocities in the vicinity of the proposed development. For the area shown in **Figure 12.1** it is recommended the two-zone concept be implemented. Further, it is recommended that Kawartha Conservation and the City of Kawartha Lakes, in coordination with stakeholders, create a two-zone policy in order to allow the proposed development the possibility to proceed.

Figure 12.1: Proposed Flood Hazard Policy Area Map



This map forms part of the Ops #1 Drain and Jennings Creek Flood Plain Mapping Study (Kawartha Conservation, June 2014).
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13 Lindsay Commercial Area

The Lindsay Commercial Area is located upstream of the crossing of Ops #1 Drain and Highway 7/35 near the intersection of Greenfield Rd and Highway 7/35. Throughout the study and through further analysis it became more apparent that this area is different from the rest of the system in a number of ways. As a result the recommendations for management of this area are different as well. Analysis and findings for this area are contained below.

13.1 Quantifying Modeled Flood Volumes

In the July 2011 Greck study, it was calculated that the flood volumes reported in previous studies had exceeded the total runoff volumes from the worst-case scenario storm event. A similar analysis was carried out for this study. The static and dynamic flood line polygons were superimposed upon the DEM; buildings were highlighted and removed from the volume calculations. The floodwater volume was calculated between a reference surface (having an elevation of 0 m) and the flooded area above that reference surface.

Figure 13.1 schematically shows how GIS represents the flooded area for the 100-year static event in the Lindsay commercial area. **Table 13.1** compares the PCSWMM rainfall runoff volumes and the static and dynamic flood volumes The volume of flood water calculated in the static HEC-RAS model calculates is greater that the runoff volume: 100,000 m3 more (or 180% more). In contrast to this, the flood volumes calculated by the dynamic HEC-RAS model are only 82% of the runoff volume.

Figure 13.1: Water Budget Analysis in the Lindsay Commercial Area

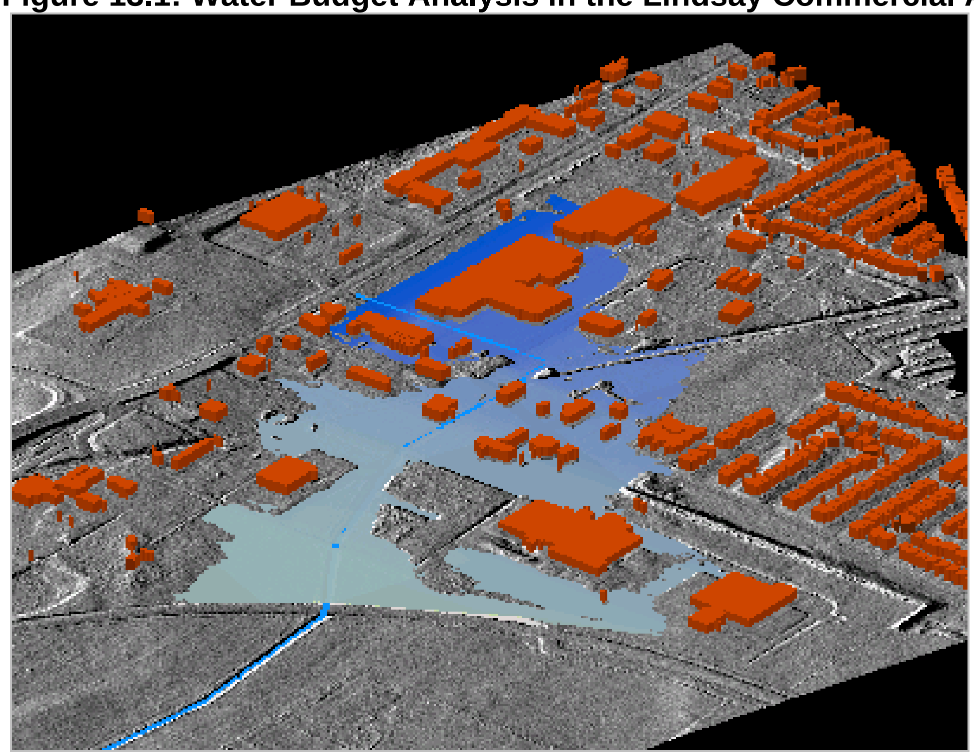


Table 13.1: Water Budget

*(**Note**: values are for Timmins event unless otherwise labelled)*

Catchments	Runoff from PCSWMM (m3)	Static Flood Volume (m3) (% of runoff)	Dynamic Volume (m3) (% of runoff)
1-4**	125,400	226,000 (180%)	102,450 (82%)
1-6	893,600	982,300 (109%)	726,000 (81%)

*** represents 100-year volumes*

Note: Refer to Figure3.1 for location of catchments

It can be seen that the static model greatly over-estimates the flood depths and extents in the commercial area when compared to the flood volumes calculated by the dynamic model.

Upon further review of this area, it is recommended that this area **not** be included in the Regulated flood plain based on the static hydraulic model. There are several reasons why this area should not be included in flood plain mapping:

- As seen above, it is unique in having the static model flood volumes so greatly overestimating actual runoff volumes
- It is located at the most upstream limit of the Ops #1 drainage basin. Most of the area does not fall within the Ministry of Natural Resources' (MNR's) recommended 125 Hectare upstream drainage area cut-off limit for flood plain mapping
- The intensely-developed commercial and industrial land use is in sharp contrast to the rest of the Ops #1 catchment area land use. As such it has a distinct historical land use pattern quite separate from the rest of the watershed.
- Roadside ditches form a large portion of the drainage system within this area. The flooding problems associated with this area are therefore urban flooding, not riverine flooding.
- The 100-year Chicago storm causes the most flooding; this is sharp contrast to the rest of the Ops #1 drainage basin which floods more severely with the Timmins storm.

As previously stated in Section 1.3, the July 2011 Greck report recommended certain areas of the watershed be developed using a two-zone designation including the Lindsay commercial area. Although this study is recommending that this area not be part of the Regulated flood plain mapping, future development in this area must recognize the flood hazard caused by restrictive stormwater infrastructure. Therefore it is recommended that Kawartha Conservation and the City of Kawartha Lakes establish a development policy that recognizes the nature of this flooding hazard.

13.2 Flood Hazard Criteria

The criteria used to carry out the analyses for this report is taken from Appendix D of the October 1988 *Flood Plain Mapping Policy Statement Implementation Guidelines*. Key data can be found in **Appendix R**.

Thematic maps of the area were created showing the above factors in determining safety during a flood. Floodwater elevation and velocity data for each cross-section were extracted from the HEC-RAS models; the overbanks and main channel portions of each cross-section are analyzed separately. When superimposed over the DEM, the range of flow depths can easily be seen.

The flood depths for the 100-year dynamic storm are shown in **Figure 13.3**; flood depths for the Timmins dynamic storm are shown in **Figure 13.4**. The dynamic Timmins model produces greater depth of flooding than the dynamic 100-year model. Because of this, it is recommended that the future development policy should be based on the flood elevations calculated for the Timmins storm.

For the dynamic Timmins model, **Figure 13.5** shows the velocities; **Figure 13.6** shows the product of depth and velocity. The calculated velocities in this area are very small (less than 0.1m/s). Therefore neither the velocity nor the velocity-depth products are significant criteria of the flood hazard or would significantly limit development. The critical factor is therefore the flood depth and the future development policy should acknowledge this criteria.

13.3 Addressing Restrictive Stormwater Infrastructure

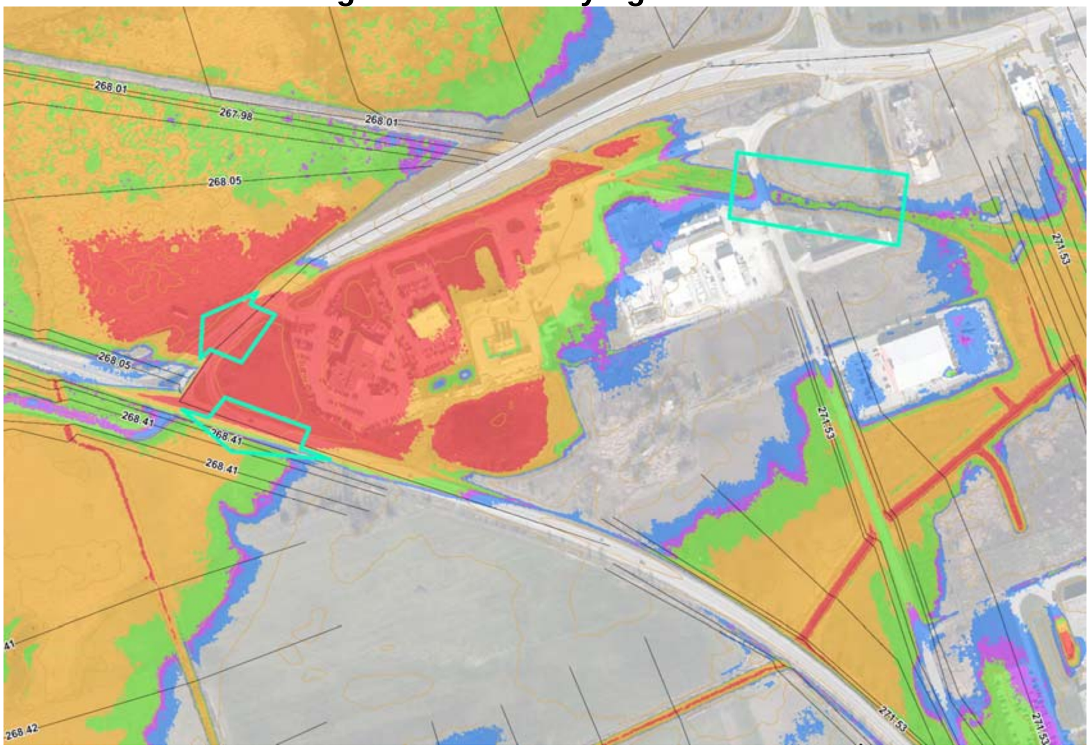
Based on the analysis for the Lindsay Commercial Area it is evident that the stormwater infrastructures (i.e. culverts) impede regional and 100-year storm flows significantly causing a backwater flooding effect. Preliminary analysis (discussed in Section 14) indicated that the culvert crossing at Highway 7/35 causes the highest extent of backwater flooding, however if this culvert was improved to convey the flow unrestricted the next upstream road crossing and culvert would then restrict flows and cause the same backwater effect at a lower elevation.

Therefore it is recommended that a comprehensive study such as a Master Drainage Plan (MDP) be completed for this area. At a minimum, the study should investigate preliminary roadway crossing improvements and scheduling, stormwater quantity control targets, and major overland flow routes. Infrastructure recommendations are further discussed in Section 14.

13.4 Low lying Area Connection

A small low-lying area along the former rail route allows water to flow to the intersection of Kent St and Highway 7, as shown in **Figure 13.2**. It is recommended that the City investigate placing berm material in the vicinity of the highlighted box to restrict water flow

Figure 13.2: Low-lying Area



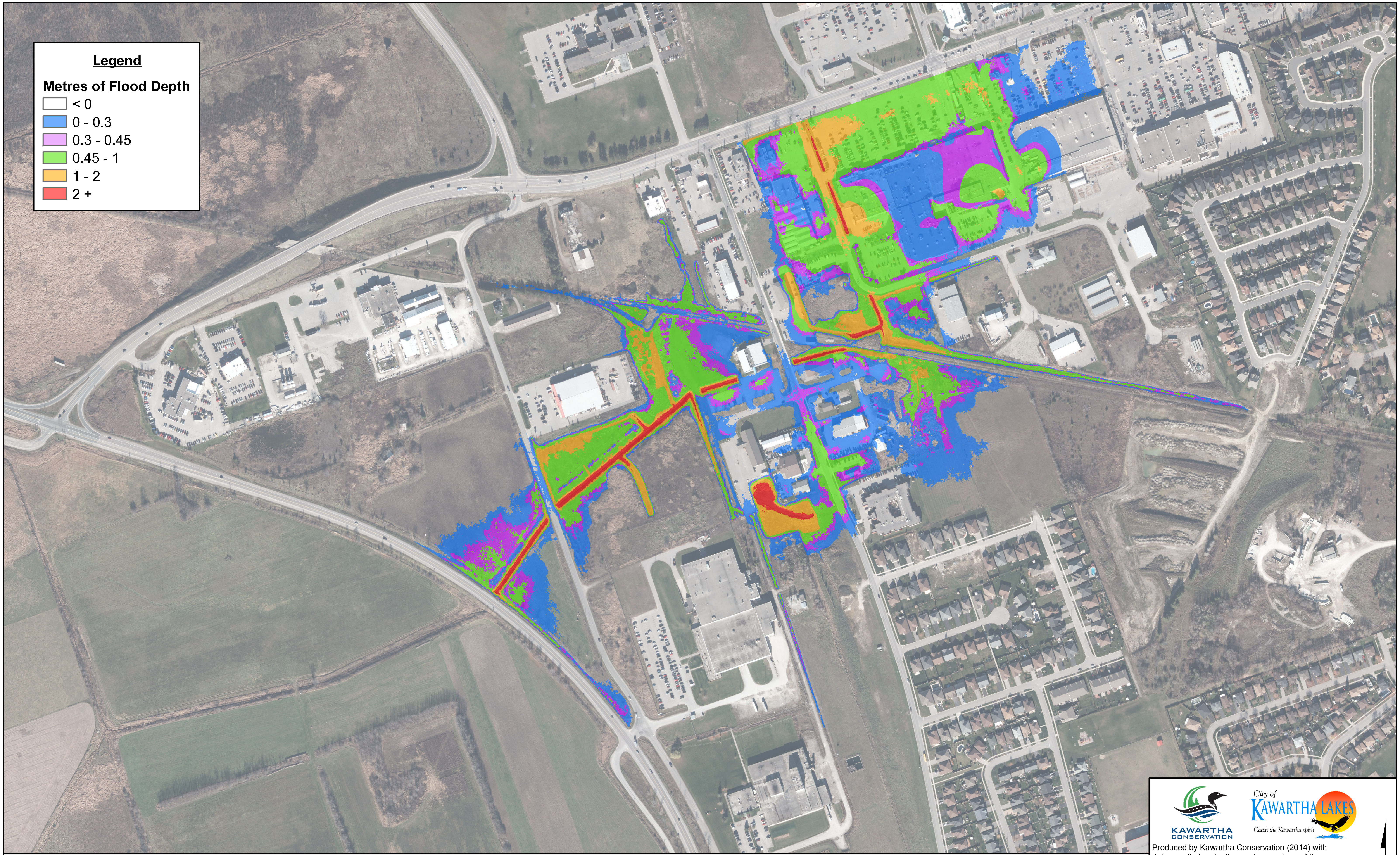


Figure 13.3: Flood Depths in Commercial Area - 100-year Dynamic Storm

1 centimetre = 45 metres

0 75 150 300
Metres

N

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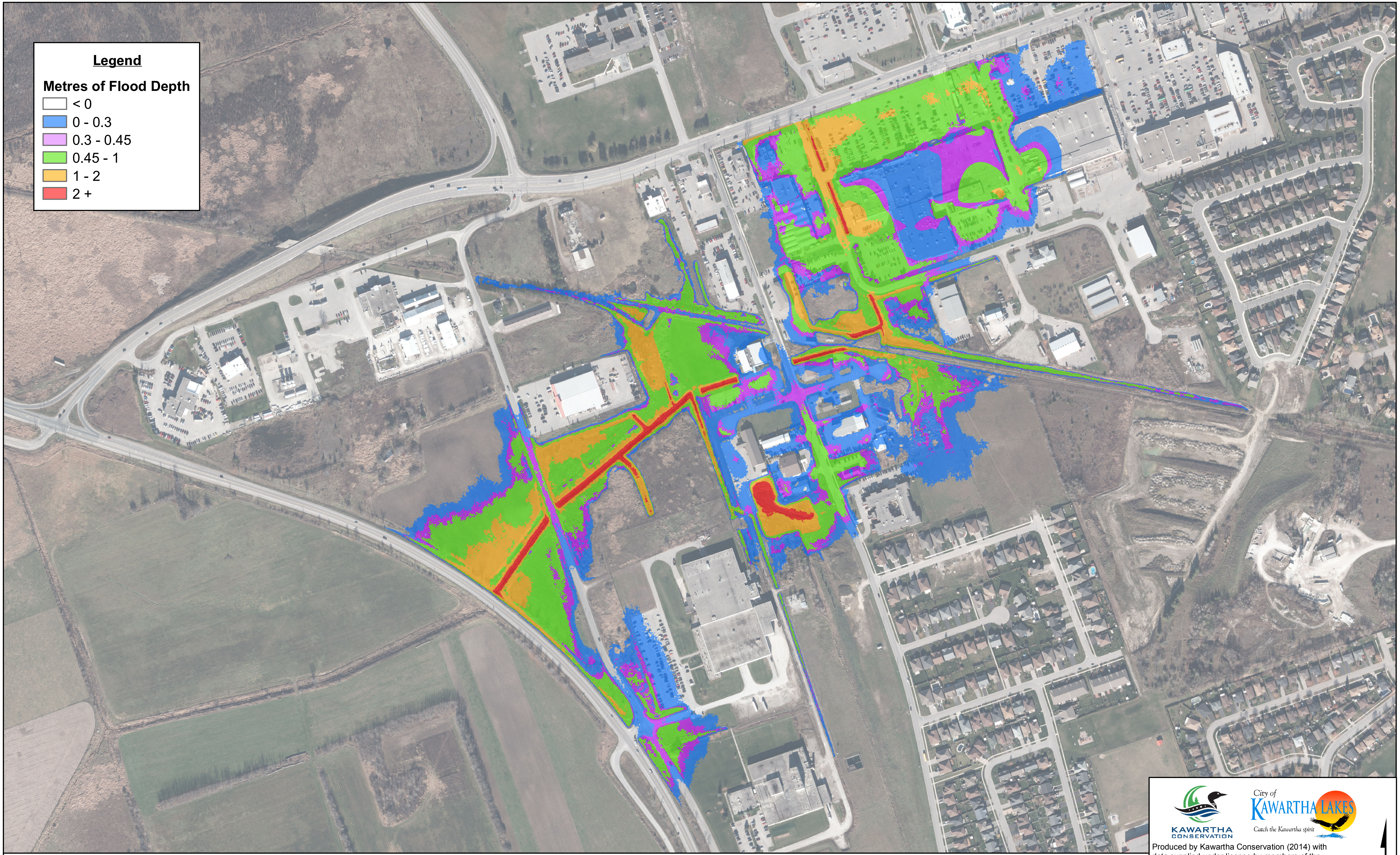

Figure 13.4: Flood Depths in Commercial Area - Timmins Dynamic Storm

1 centimetre = 45 metres

0 75 150 300
Metres

Produced by Kawartha Conservation (2014) with data supplied under license by members of the Ontario Data Exchange, Ontario Ministry of Natural Resources.

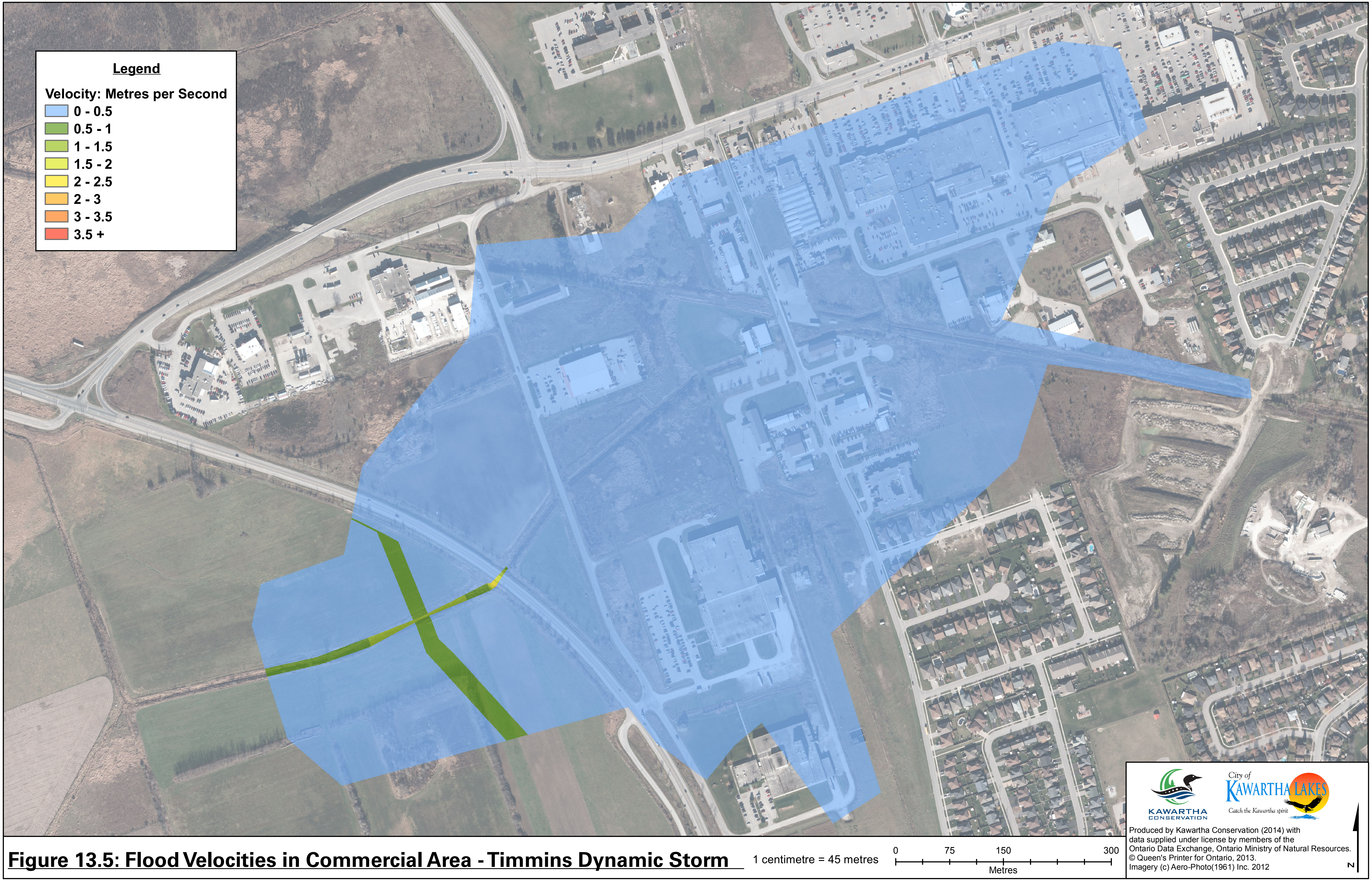

Figure 13.5: Flood Velocities in Commercial Area - Timmins Dynamic Storm

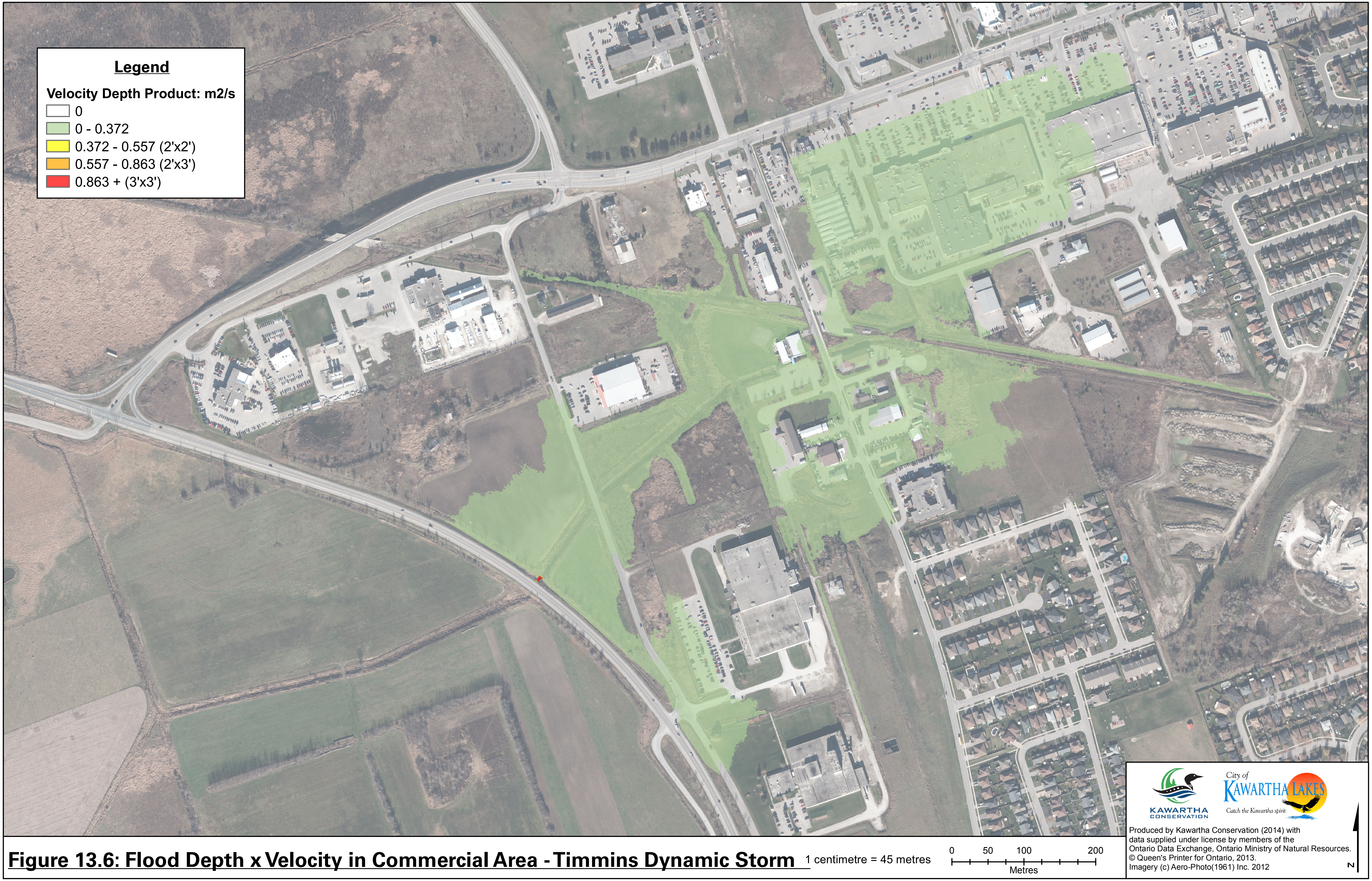

Figure 13.6: Flood Depth x Velocity in Commercial Area - Timmins Dynamic Storm

14 Infrastructure Improvements

As can be seen in **Figure 14.1**, there are key locations that cause sustained water flooding: William Street, Angeline Street, Hwy 35 at the airport, and Highway 7/35 near the intersection of Greenfield Rd and Highway 7/35. There is a strong possibility that flood elevations could be significantly lowered by enlarging culverts and/or lowering roadway centerlines at these points. Simple evaluations of culvert improvements were carried out to determine if flood elevations can be reduced through capital improvements.

Although it is beyond the scope of the flood plain study to outline specific infrastructure improvements to reduce flooding, several simple analyses were run to determine impacts on the flood elevations.

The existing culvert under the Rail Trail is a 2.5m x 2.0m metal box culvert. The HEC-RAS model was modified by increasing the culvert theoretically to a 7.5m x 6m concrete box culvert. The flood elevation upstream of the culvert decreased by almost 1.5m, from 254.45m to 252.86m).

A second modification was made to the HEC RAS model. The existing culvert under Highway 7/35 near the intersection of Greenfield Rd and Highway 7/35 is a 1.83m x 1.13m culvert. The HEC-RAS model was modified by increasing the culvert theoretically to a 3m x 3m concrete box culvert. The flood elevation upstream of the culvert decreased by 0.5m, from 271.52m to 271.02m).

Such simple analyses show that flood elevation reductions are possible through infrastructure improvements. However, near the outlet to the river and in the Lindsay commercial area, the high flood line is caused by multiple restrictions in series of culverts. Enlarging one culvert will reduce the flood line locally, but the next upstream culvert would become a pinch point. Future comprehensive engineering analyses are recommended to evaluate infrastructure improvements and determine the most cost-effective way to lower flood elevations. The analyses should include culvert upsizing, culvert removal, potential bridge installation, road centreline modifications.

Refer to Figure 14.1 and Figure 14.2 for locations of key locations that cause sustained flooding. **Figure 14.3** shows a comparison of the static water surfaces profiles in the vicinity of the Rail Trail and Highway 7.

Figure 14.1: Stormwater Infrastructure Pinch Point Location

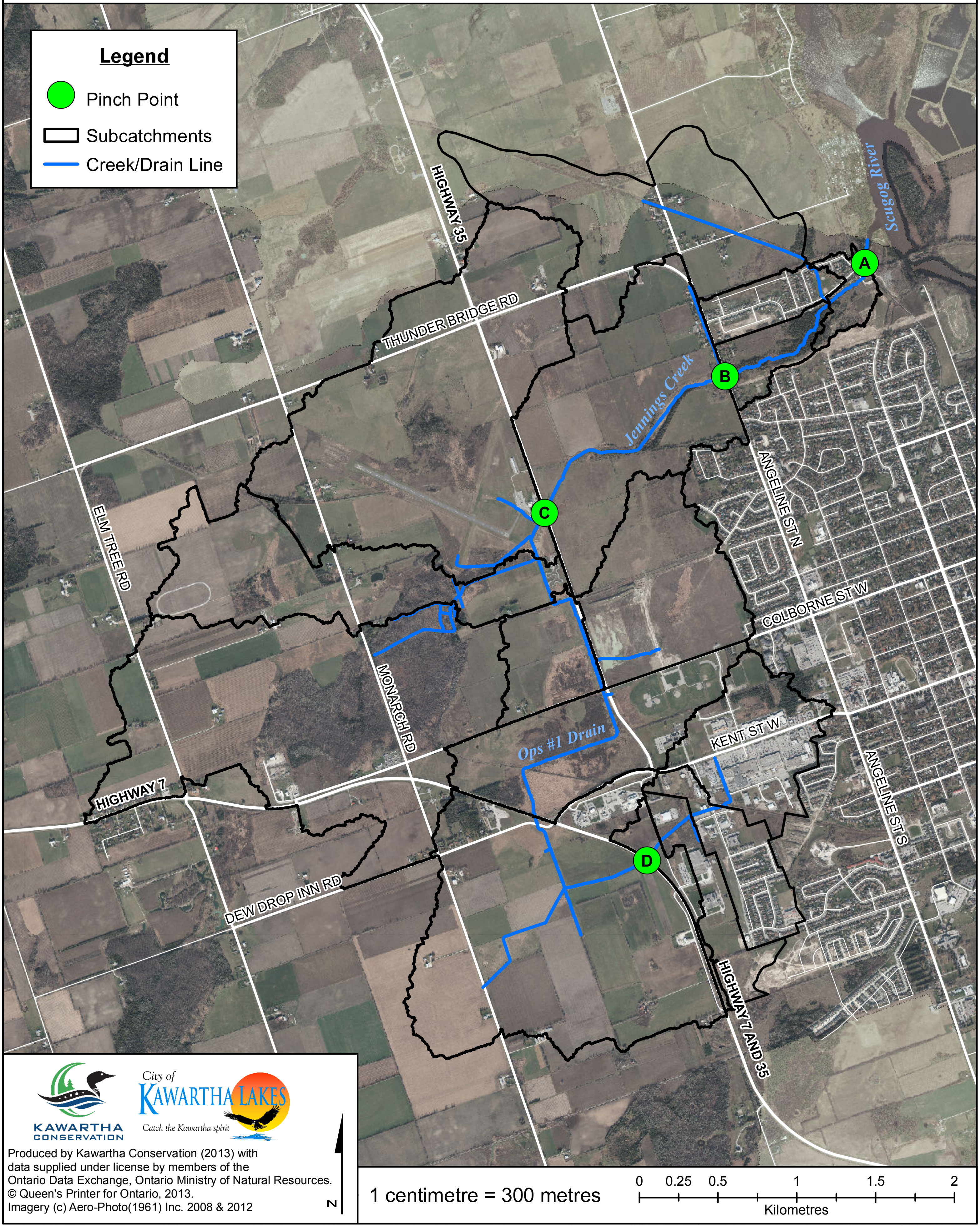

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Figure 14.1: Regulatory Profile – Static

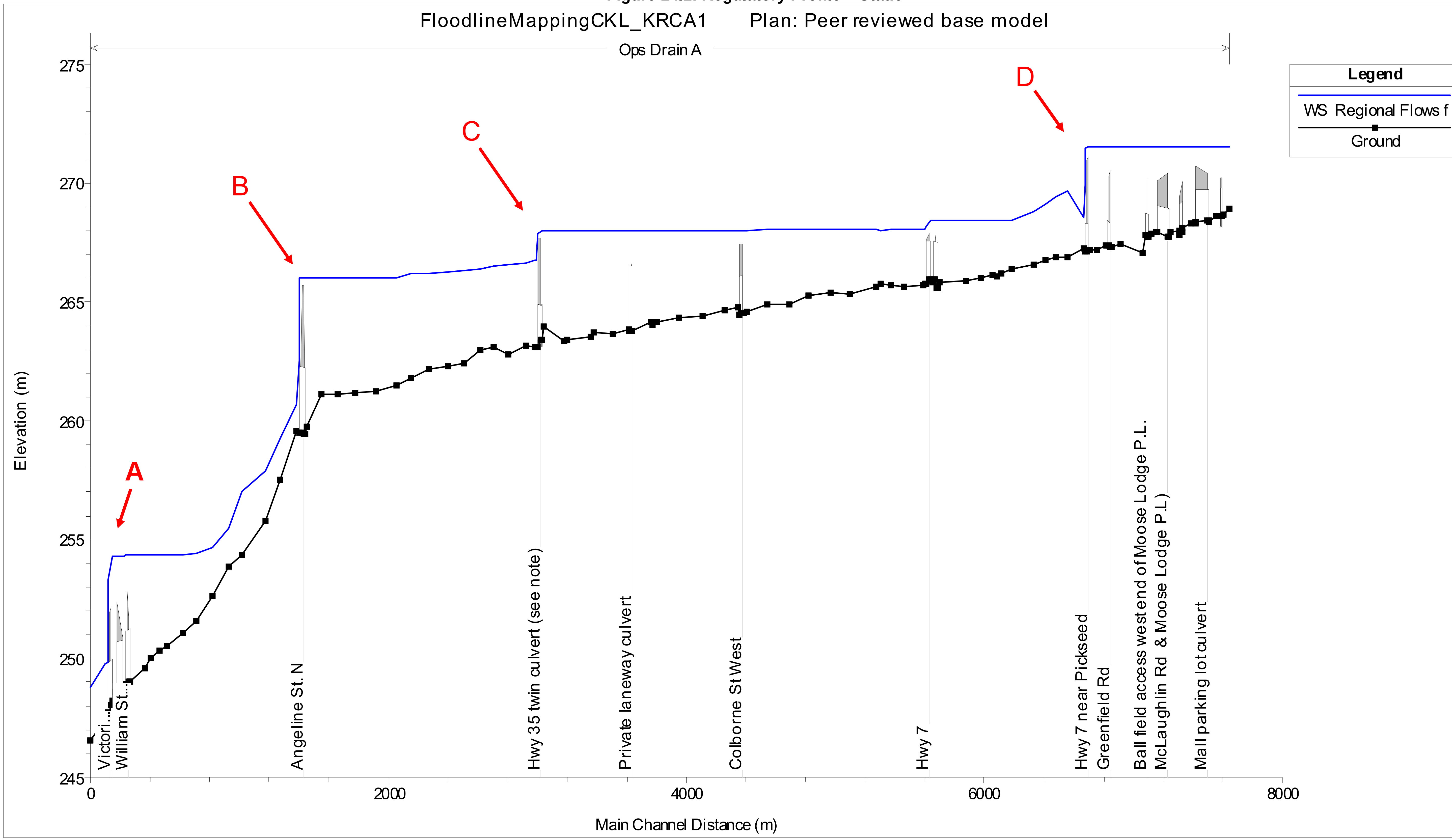
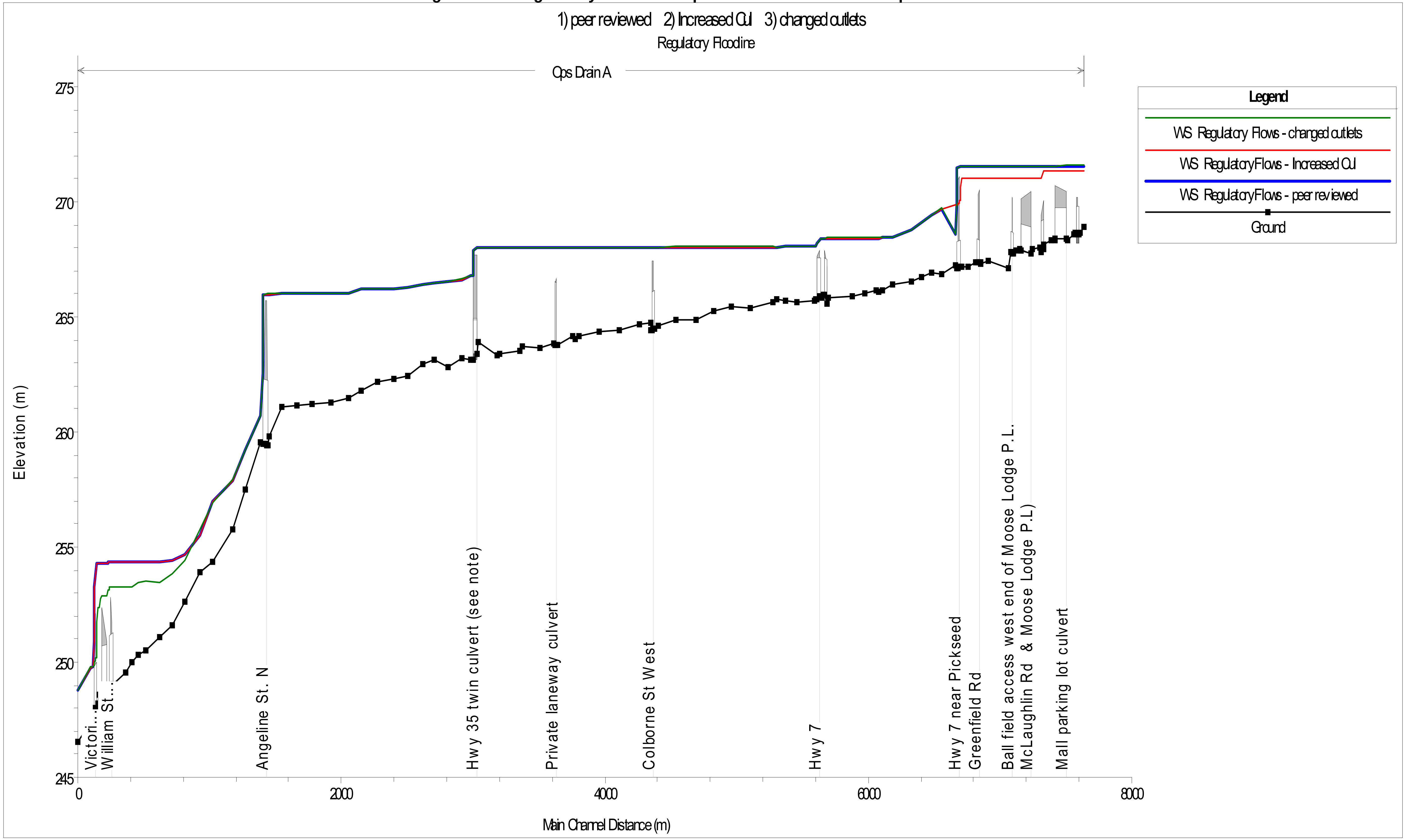


Figure 14.2: Regulatory Profile Comparisons for the Culvert Improvements



15 Conclusions and Recommendations

Throughout the watershed, the flood plain shape and extent of the dynamic model is quite similar to that of the static model. The routing in PCSWMM produces comparable attenuated flows to the dynamic HEC-RAS routing. However, since the PCSWMM model does not include the road crossings and culverts, the attenuation is due to the channel and overbanks and not the blockages caused by the roadways. In the dynamic HEC-RAS model flow is impacted by the road crossings and culverts and subsequently reduced downstream of each crossing.

MNR policy does not allow flood plains to be reduced due to road/culvert attenuation, for several reasons:

- Any future culvert and/or road improvement would increase the downstream flood plain.
- There is no guarantee that a roadway would remain in place during a flood event. The road and/or culvert could wash out and the downstream flows would not be attenuated.

For these reasons, it is recommended that the results of the static hydraulic model for Ops #1 Drain and Jennings Creek be used for generating the Regulatory flood maps downstream of Highway 7/35. The results of the models are reasonable and can be used to establish Regulatory flood lines for the watershed. Reduced versions of these flood maps can be found at the end of this study.

Within the Regulatory flood plain of the Ops#1 Drain and Jennings Creek it is recommended that the one-zone policy concept be applied. There is preliminary analysis in this study to indicate that areas of the flood plain may be suitable for application of the two-zone concept however further analysis is required to demonstrate how flood fringe development would proceed and that there is no significant upstream or downstream effect and no new hazards are created. It is further recommended that Kawartha Conservation and the City of Kawartha Lakes, in coordination with stakeholders, create a two-zone policy in order to allow proponents to demonstrate the effects of their developments using the baseline modeling prepared in this study.

For the Lindsay commercial area, future development in this area must recognize flood hazards caused by restrictive stormwater infrastructure. Future development should be controlled using a development policy based on the results of the Timmins dynamic model and appropriate for this urban drainage flooding hazard. A comprehensive study such as a Master Drainage Plan should be completed for this area to address future stormwater infrastructure improvements aimed at reducing the flood hazard.

Outside the Lindsay commercial area, other stormwater infrastructure improvements can conceptually reduce the flooding hazard. Therefore it is recommended that these improvements be investigated as well.

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Flood Plain Map - Community of Lindsay

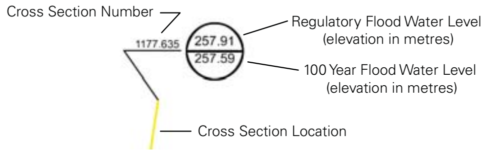
Ops #1 Drain and Jennings Creek

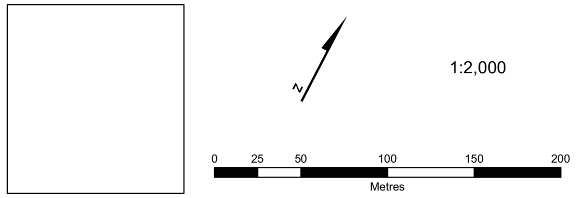
Map 1

Legend

- Main Channel
- Cross Sections
- Contours (0.5m)
- Contours (1.0m)
- Spill Locations
- Regulatory Floodline
- Parcel

Cross Section Numbering

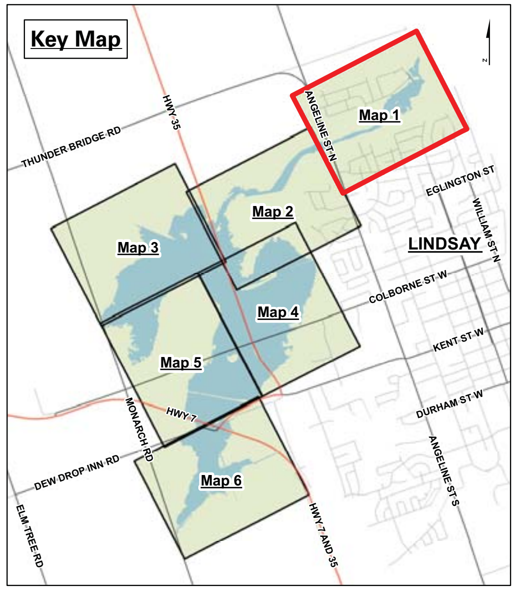


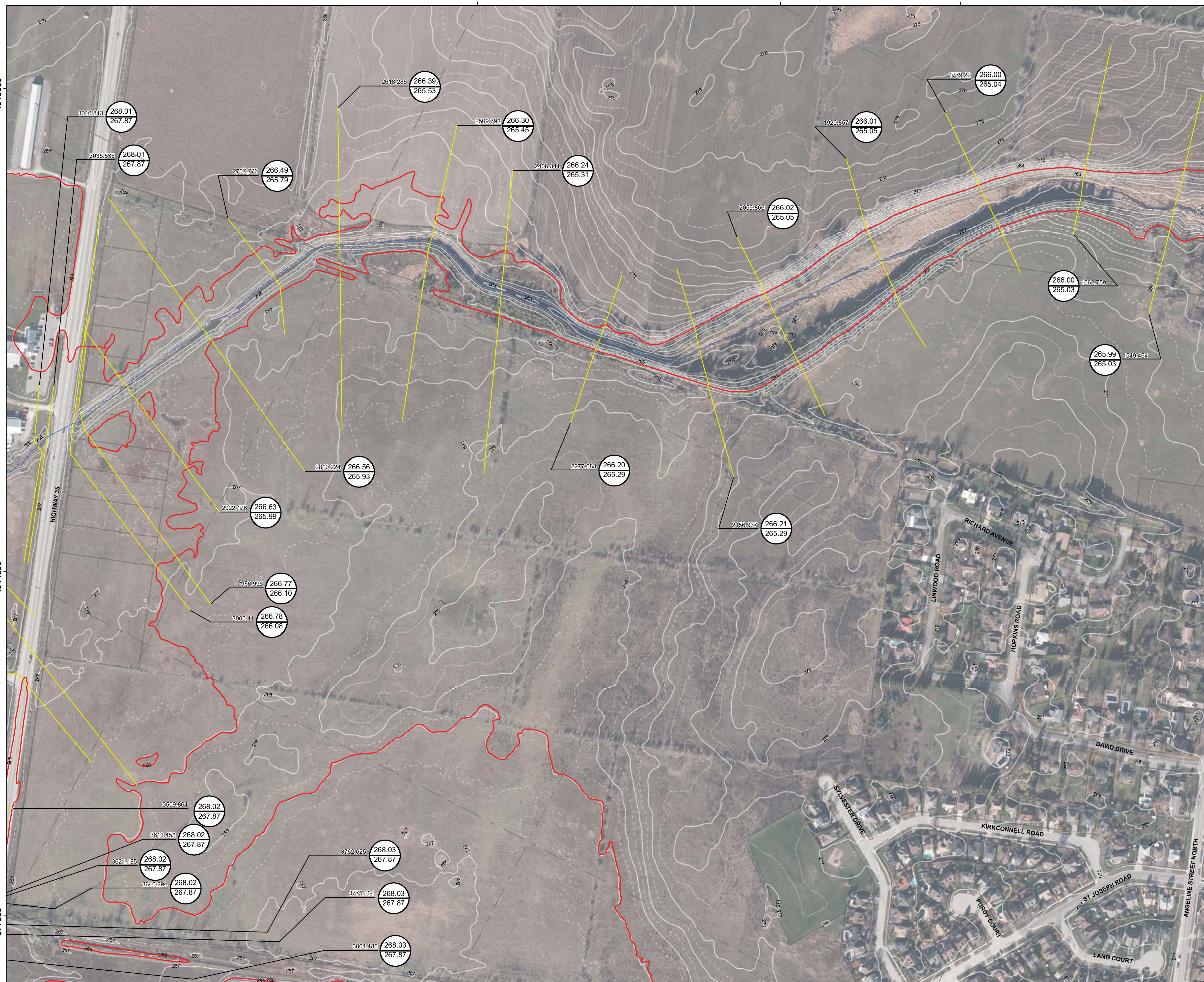


REVISIONS			
No.	Description	By	Date
1	Final Flood Plain Map	CP	June 2014

Notes:

1) LiDAR (Light Detection and Ranging) survey data collected November 4th, 5th, 6th, and 7th, 2013 by Aero-Photo (1961) Inc.
2) Contours produced by Kawartha Region Conservation Authority using LiDAR data.
3) Field Survey of structures by Kawartha Region Conservation Authority, using RTK GPS control points.
4) Orthophotography (16cm) collected November 8th, 2013 by Aero-Photo (1961) Inc.
5) The flood inudation areas were delineated using the information developed from the sources listed above in notes 1 through 4.
6) Produced using information provided by the Ministry of Natural Resources, Kawartha Region Conservation Authority, and City of Kawartha Lakes. Copyright (c) Queen's Printer, 2014
7) This map forms part of the Ops #1 Drain and Jennings Creek Flood Plain Mapping Study, June 2014





Flood Plain Map - Community of Lindsay

Ops #1 Drain and Jennings Creek

Map 2

Legend

- Main Channel
- Spill Locations
- Cross Sections
- Regulatory Floodline
- Contours (0.5m)
- Parcel
- Contours (1.0m)

Cross Section Numbering

Cross Section Number 1177.635
257.91 Regulatory Flood Water Level (elevation in metres)
257.59 100 Year Flood Water Level (elevation in metres)
Cross Section Location

N

1:2,000

0 25 50 100 150 200
Metres

REVISIONS			
No.	Description	By	Date
1	Final Flood Plain Map	CP	June 2014

Notes:

1) LiDAR (Light Detection and Ranging) survey data collected November 4th, 5th, 6th, and 7th, 2013 by Aero-Photo (1961) Inc.
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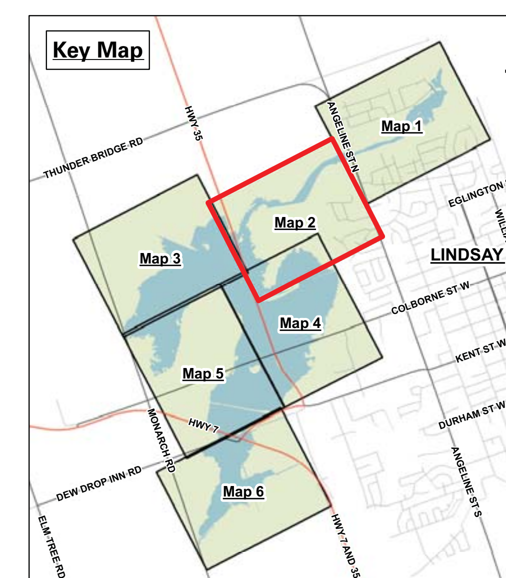
7) This map forms part of the Ops #1 Drain and Jennings Creek Flood Plain Mapping Study, June 2014



Flood Plain Map - Community of Lindsay

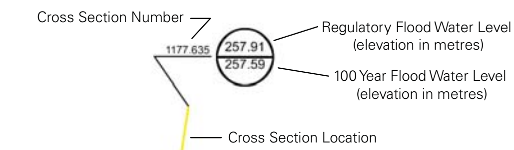
Ops #1 Drain and Jennings Creek

Map 3

Legend

- Main Channel
- Spill Locations
- Cross Sections
- Regulatory Floodline
- Contours (0.5m)
- Parcel
- Contours (1.0m)

Cross Section Numbering



1:2,000

0 25 50 100 150 200
Metres

REVISIONS			
No.	Description	By	Date
1	Final Flood Plain Map	CP	June 2014

Notes:

1) LiDAR (Light Detection and Ranging) survey data collected November 4th, 5th, 6th, and 7th, 2013 by Aero-Photo (1961) Inc.
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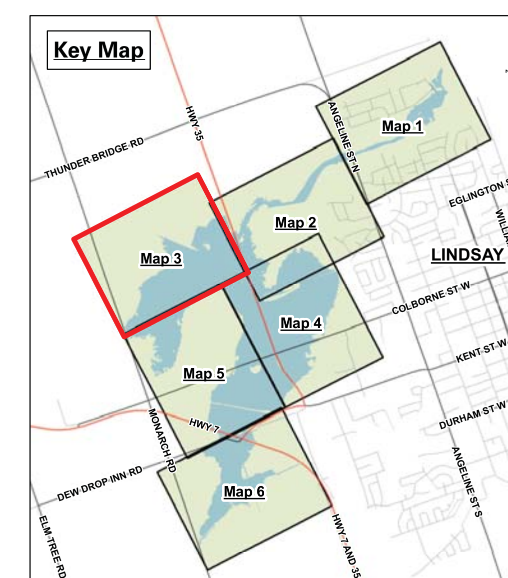
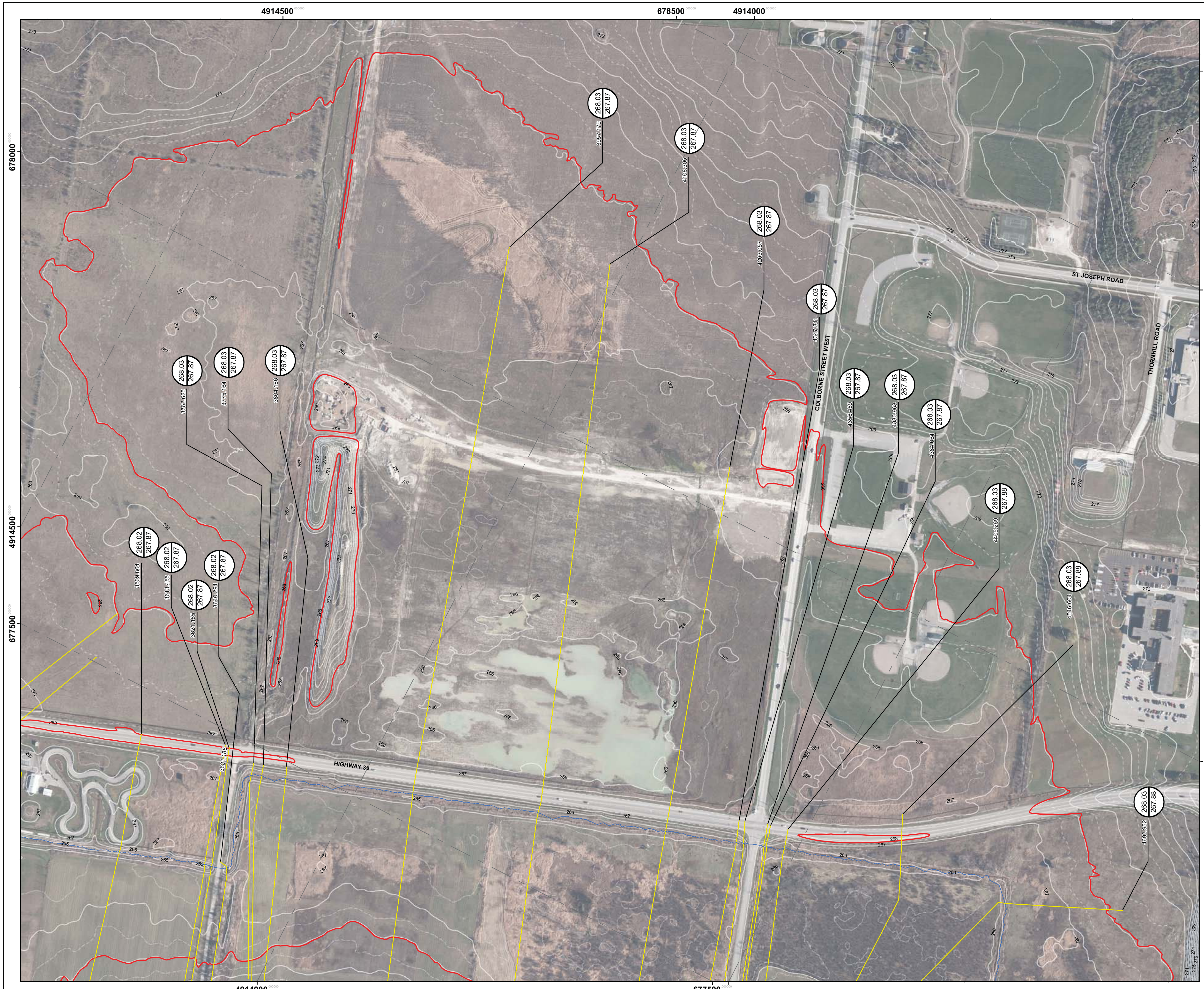
7) This map forms part of the Ops #1 Drain and Jennings Creek Flood Plain Mapping Study, June 2014





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Flood Plain Map - Community of Lindsay

Ops #1 Drain and Jennings Creek

Map 4

Legend

- Main Channel
- Cross Sections
- Contours (0.5m)
- Contours (1.0m)
- Spill Locations
- Regulatory Floodline
- Parcel

Cross Section Numbering

Cross Section Number
1177.635
257.91
257.59
Regulatory Flood Water Level (elevation in metres)
100 Year Flood Water Level (elevation in metres)
Cross Section Location

1:2,000

0 25 50 100 150 200
Metres

REVISIONS			
No.	Description	By	Date
1	Final Flood Plain Map	CP	June 2014

Notes:

1) LiDAR (Light Detection and Ranging) survey data collected November 4th, 5th, 6th, and 7th, 2013 by Aero-Photo (1961) Inc.
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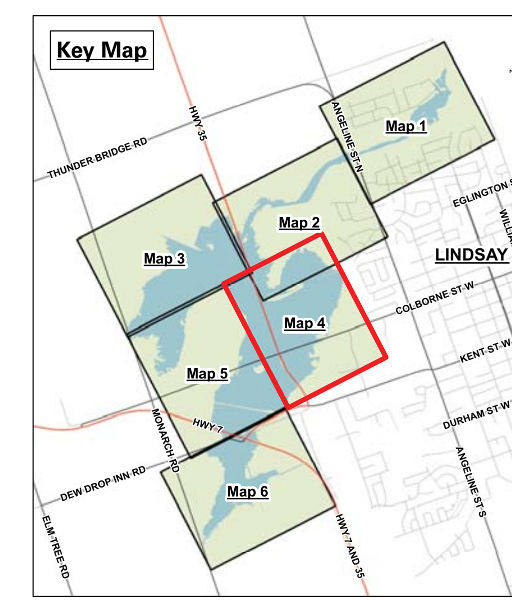
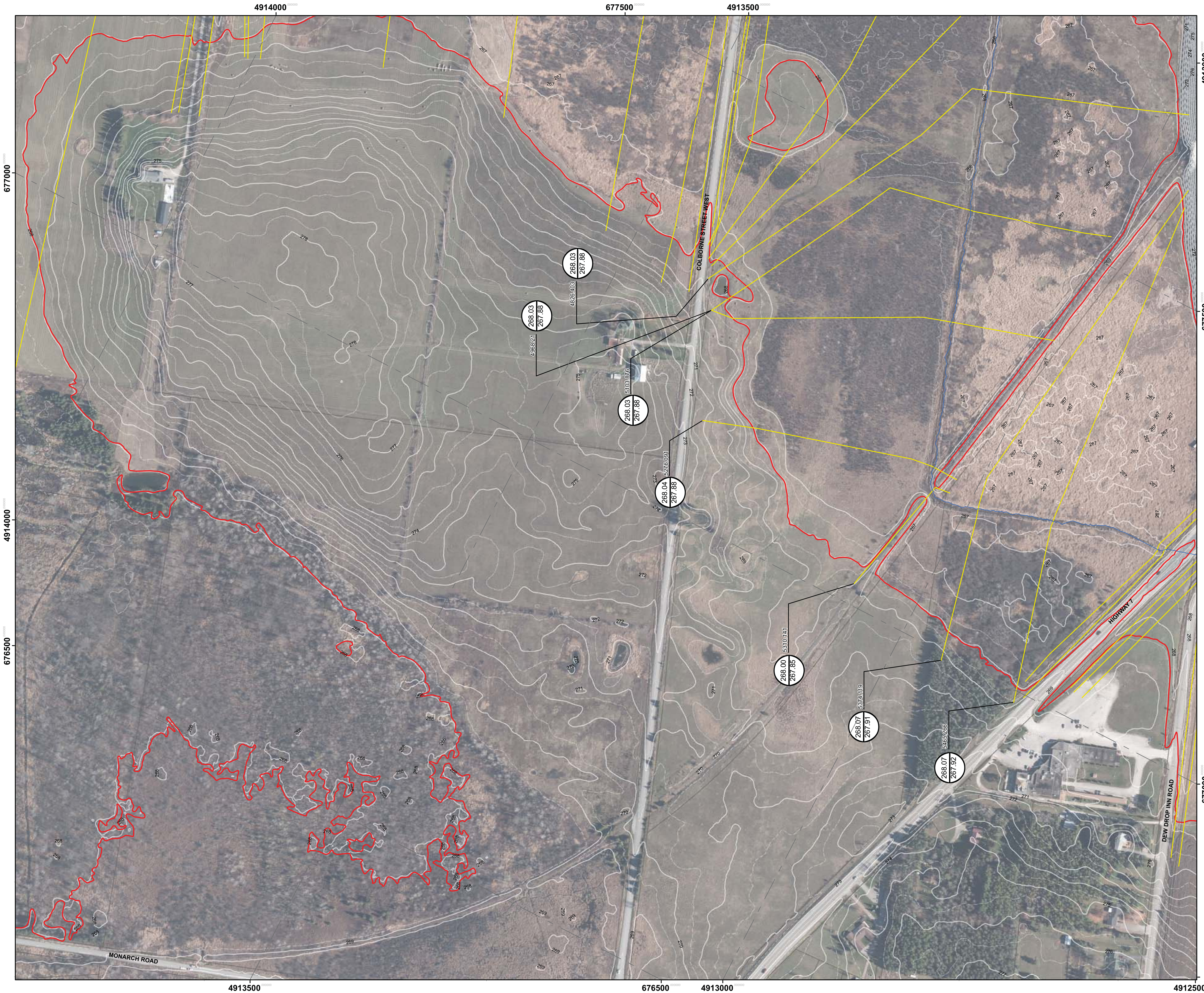
7) This map forms part of the Ops #1 Drain and Jennings Creek Flood Plain Mapping Study, June 2014





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Flood Plain Map - Community of Lindsay

Ops #1 Drain and Jennings Creek

Map 5

Legend

- Main Channel
- Cross Sections
- Contours (0.5m)
- Contours (1.0m)
- Spill Locations
- Regulatory Floodline
- Parcel

Cross Section Numbering

Cross Section Number: 1177.635

Regulatory Flood Water Level (elevation in metres): 257.91

100 Year Flood Water Level (elevation in metres): 257.59

Cross Section Location

N

1:2,000

0 25 50 100 150 200

Metres

REVISIONS			
No.	Description	By	Date
1	Final Flood Plain Map	CP	June 2014

Notes:

1) LiDAR (Light Detection and Ranging) survey data collected November 4th, 5th, 6th, and 7th, 2013 by Aero-Photo (1961) Inc.
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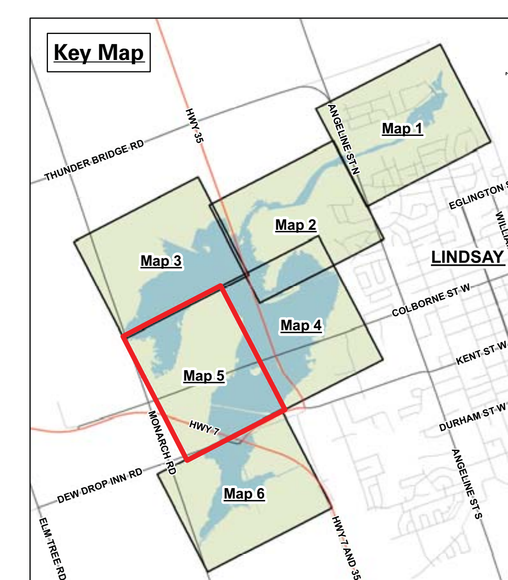
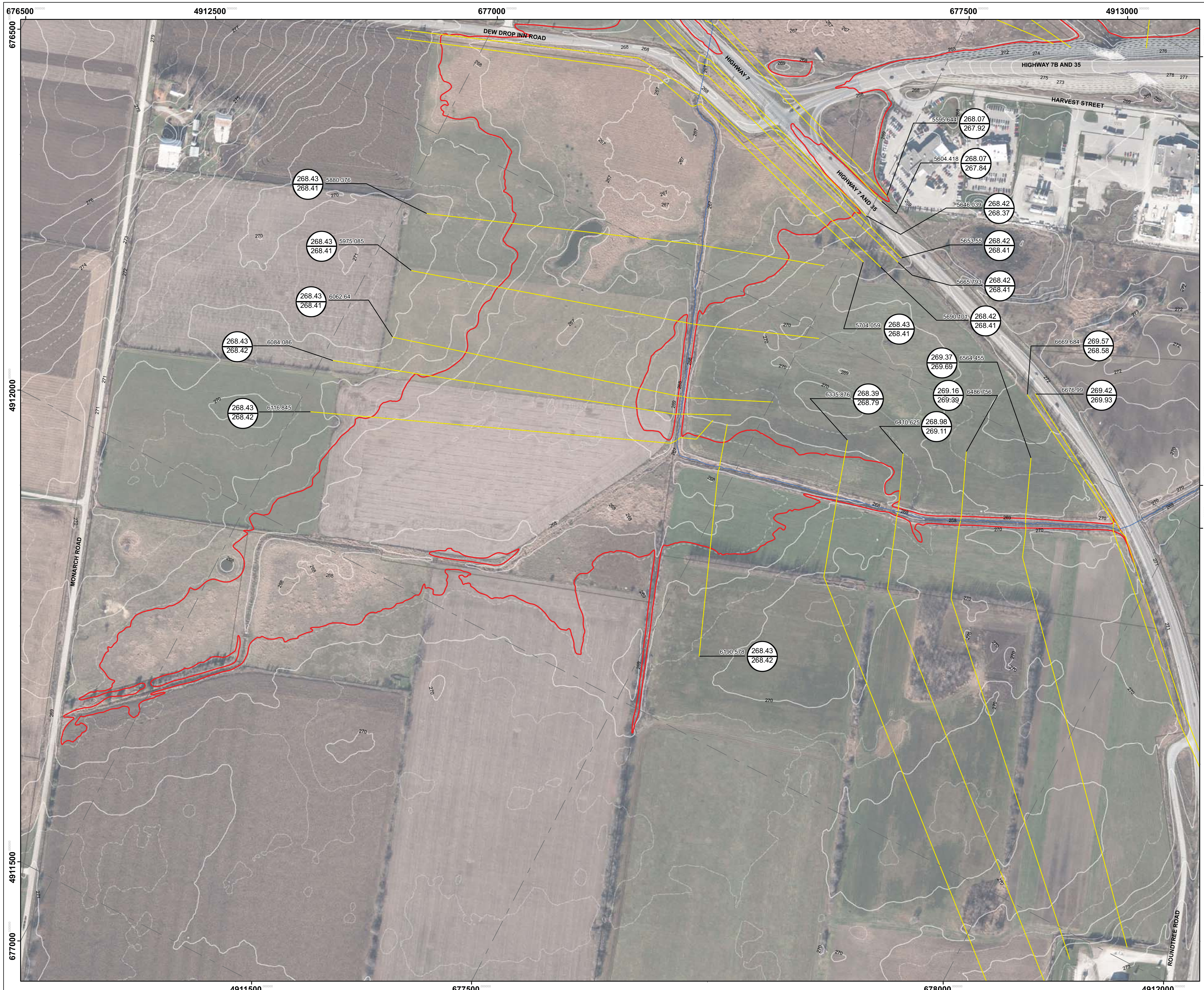
7) This map forms part of the Ops #1 Drain and Jennings Creek Flood Plain Mapping Study, June 2014

Key Map





Flood Plain Map - Community of Lindsay

Ops #1 Drain and Jennings Creek

Map 6

Legend

- Main Channel
- Cross Sections
- Contours (0.5m)
- Contours (1.0m)
- Spill Locations
- Regulatory Floodline
- Parcel

Cross Section Numbering

Cross Section Number — 1177.635

257.91 — Regulatory Flood Water Level (elevation in metres)

257.59 — 100 Year Flood Water Level (elevation in metres)

Cross Section Location

1:2,000

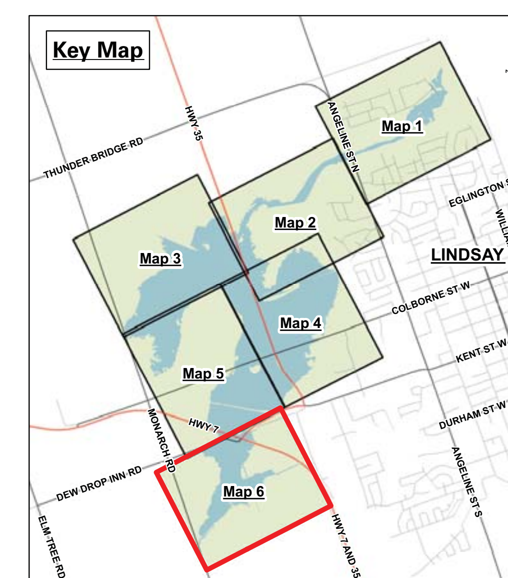
0 25 50 100 150 200

Metres

REVISIONS			
No.	Description	By	Date
1	Final Flood Plain Map	CP	June 2014

Notes:

1) LiDAR (Light Detection and Ranging) survey data collected November 4th, 5th, 6th, and 7th, 2013 by Aero-Photo (1961) Inc.
2) Contours produced by Kawartha Region Conservation Authority using LiDAR data.
3) Field Survey of structures by Kawartha Region Conservation Authority, using RTK GPS control points.
4) Orthophotography (16cm) collected November 8th, 2013 by Aero-Photo (1961) Inc.
5) The flood inudation areas were delineated using the information developed from the sources listed above in notes 1 through 4.
6) Produced using information provided by the Ministry of Natural Resources, Kawartha Region Conservation Authority, and City of Kawartha Lakes. Copyright (c) Queen's Printer, 2014
7) This map forms part of the Ops #1 Drain and Jennings Creek Flood Plain Mapping Study, June 2014



16 Appendices

Refer to Technical Appendices (separate document)

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