HYDROLOGY & HYDRAULICS REPORT

Lower Trent Region Conservation Authority

And

The Township of Stirling-Rawdon

FHIMP ON22-003

For the

Rawdon Creek Floodplain Mapping Update

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1 Introduction

Lower Trent Region Conservation Authority (LTC) has partnered with the Township of Stirling-Rawdon (S-R) along with provincial and federal partners to lead the Rawdon Creek Floodplain Mapping Update.

With acquired funding through the federal Flood Hazard Identification Mapping Program (FHIMP) and S-R, LTC has undertaken a leadership role in the production of updated mapping for Rawdon Creek. The objective is to provide a floodplain mapping update that will allow LTC and S-R staff to make informed planning and regulation decisions. Jewell Engineering Inc. (Jewell) is pleased to support this initiative through the technical analysis and reporting described herein.

The driving forces for this project include climate change, improved modelling techniques and software programs, improved data acquisition tools, land use changes, and updated infrastructure that can dramatically influence flood behaviour and floodplain extents.

The need for accurate, detailed floodplain mapping that factors in climate change forecasting has become increasingly evident as flood damages become the largest cost to the Canadian economy out of any other natural hazard. Updated floodplain maps are needed to protect human life, property, and infrastructure from the damaging effects of flooding that are occurring with increased frequency.

The funds deployed by the federal and local governments to complete this updated floodplain mapping provide a dual benefit; it protects the local community from potential flood hazards and reduces the dependence on provincial and federal funds associated with the Disaster Financial Assistance Arrangements (DFAA) administered by Public Safety Canada.

Rawdon Creek was previously mapped in 1985 and the data is no longer current. The study area is defined as a one-zone floodplain and has been updated accordingly.

The Rawdon Creek watershed flows into S-R from the north, and eventually outlets into the Trent River. The urban area within S-R is subject to the maximum peak flows produced by the watershed and as a result, the flood hazard has been updated for the study area presented in Section 3.

2 Background

Previous studies (see list below) were commissioned with the intent to reduce the flood risk at certain locations along Rawdon Creek.

- *1975 Report on Mayhew Creek Flood Plain Mapping in Trenton, Ontario and Rawdon Creek Flood Plain Mapping and Channelization in Stirling, Ontario* prepared by Kilborn Engineering Ltd.
- 1985 *Flood Damage Reduction Study* prepared by Kilborn Engineering Ltd.

A brief summary of key findings from each of the above reports is provided below.

1975 Report on Mayhew Creek Floodplain Mapping in Trenton, Ontario and Rawdon Creek Flood Plain Mapping and Channelization in Stirling, Ontario; Kilborn Engineering Ltd.

Pertaining to Rawdon Creek, this report was prepared to outline the procedure undertaken to prepare floodplain mapping of Rawdon Creek from Highway 33 to just upstream of the Village of Stirling, and to investigate flood and erosion problems along Rawdon Creek in the studied area. The authors concluded that the Timmins and 100-yr storms resulted in peak flows of 10,037 cfs (284.2 m^3/s) and 3,083 cfs (87.3 m^3/s), respectively. In an effort to address flooding and erosion issues along Rawdon Creek, Kilborn recommended embankment protection options and additional alternatives that could be done to accommodate for the peak flows.

1985 Flood Damage Reduction study; Kilborn Engineering Ltd.

Kilborn Limited was consulted for the *Flood Damage Reduction Study* to develop a Flood Plain Management Plan for the Village of Stirling in order to minimize flooding and its resulting damages. As part of the study, Kilborn provided a floodplain map with the regulatory and 100-year flood plain. The authors updated the Timmins and 100-yr storm peak flows to be 10,311 cfs (292 m^3/s) and 3,409 cfs (96.5 m^3/s), respectively. A floodplain management plan, known as Scheme 2C, was recommended, and included the following flood damage reduction measures: the rehabilitation of the Stirling Dam, construction of an earth dyke along James and Mill Street, construction of an earth dyke on the east bank downstream of Highway No. 14, raising the elevation of Highway No. 14, removal of the concrete weir, and an active flood proofing program.

3 Study Area

The study area for the Rawdon Creek floodplain mapping update was outlined by LTC at the beginning of the project. The study area focuses on the communities adjacent to Rawdon Creek primarily located within the urban area at the Village of Stirling. An excerpt of the historic flood limits from LTC is provided in Figure 3-1, where the red line represents the existing flood line.

The Rawdon Creek watershed extends from near Moira Lake at the north to the Trent River to the south (see Appendix B-1). Due to the topography of the watershed, the flow drains in a southward direction towards the Village of Stirling. Stirling lies at the confluence of the two main branches, where there is a defined channel that discharges to the Trent River.

Existing and future build-out conditions were considered. Guidance for future development was obtained from Schedule A (South) of the Hastings County Official Plan that outlines land use designations for the S-R Urban Area. Schedule A is included in Appendix A.

Figure 3-1: Historic LTC Rawdon Creek Floodplain Mapping Limit

4 Hydrology

The hydrology assessment was prepared for several nodes of interest throughout the Rawdon Creek watershed. See Appendix B for the location and a description of each node. Various methodologies were applied and compared to determine representative peak flows at each node. Each methodology was carefully considered prior to the selection of the peak flows for use in the hydraulic model, including potential flow impacts due to spring melt conditions.

The subject watershed is within Zone 3 of Flood Hazard Criteria Zones for Ontario Conservation Authorities. Therefore, the flood standard is the 100-yr or Timmins event; whichever produces the greater peak flow.

The detailed hydrologic analysis for the purpose of quantifying the peak flow rates is described below.

4.1 Data Sources

Data collection is an integral component of the hydrologic assessment. A description of each primary data source applied in the analysis is provided below.

4.1.1 LiDAR, Catchment Areas & Terrain

The Rawdon Creek watershed has a total area of 174km^2 and traverses the Town of S-R before it eventually outlets into the Trent River. Catchment boundaries are identified in Appendix B.

Jewell discretized the watershed into several sub-catchments based on confluence points and nodes of interest. A particular hydrologic node of interest is Node B; this node corresponds to the stream flow gauge location for Rawdon Creek.

Catchment areas were delineated using topographic information from the following sources.

1) LiDAR provided by LTC flown for Quinte-Hastings specifically for use in the floodplain mapping updates was reviewed in combination with ESRI server data information to assist in delineation of the sub-catchment boundaries. The sub-catchment configurations are similar to those delineated in the 1985 Flood Damage Reduction Study, however, Jewell completed a detailed review of the contour information and updated the sub-catchment boundaries accordingly.

2) Jewell completed a topographic survey and inspections of each crossing/hydraulic structure along Rawdon Creek within the study area.

4.1.2 Soils and Land Cover

A soils map is provided in Appendix C. Soil type information was obtained from the Soil Survey Complex database produced by the *Ontario Ministry of Agriculture, Food and Rural Affairs* in cooperation with the *Ontario Ministry of Natural Resources and Forestry*. Soil composition was obtained from the *Soil Survey of Hastings County*.

The Rawdon Creek watershed is largely comprised of loam, particularly Dummer and Otonabee Loam at the north end of the watershed, and Bondhead Sandy Loam at the south end of the watershed (Canada

Department of Agriculture, 1962). The soils in the watershed are mainly classified as Hydrologic Soils Group (HSG) B. The HSG classification for soils is used to identify drainage characteristics for various soil types. An excerpt from Chapter 8 of the *1997 MTO Drainage Management Manual* that describes drainage characteristics for each HSG is provided below. The Rawdon Creek watershed has 71% HSG B coverage as shown in Appendix B and Table 4-1.

Table 4-1: Rawdon Creek HSG Summary

HSG Soils Group	Area (km^2)	Land coverage (%)
A	8.2	5
B	124.3	71
C	26.9	15
D	14.8	9

The hydrologic soil group is used to classify soils into groups of various runoff potential.

The Soil Conservation Service (SCS) classifies bare thoroughly wet soils into four hydrologic soil groups (A, B, C and D). SCS descriptions of the four groups, modified slightly to suit Ontario conditions, are as follows: (Design Chart 1.09)

A: High infiltration and transmission rates when thoroughly wet, eg. deep, well drained to excessively-drained sands and gravels. These soils have a low runoff potential.

B: Moderate infiltration and transmission rates when thoroughly wet, such as moderately deep to deep open textured loam.

C: Slow infiltration and transmission rates when thoroughly wet, eg. fine to moderately fine-textured soils such as silty clay loam.

D: Very slow infiltration and transmission rates when thoroughly wet, eg. clay loams with a high swelling potential. These soils have the highest runoff potential.

In Ontario, soils have been found to lie between the main groups given above, and have therefore been interpolated as AB, BC, CD as appropriate, such as Guelph loam, which is classified as BC.

Figure 4-1: Excerpt from 1997 MTO DMM Describing Hydrologic Soils Group Classifications

The soils data is used to develop curve numbers (CNs) that are a key modelling parameter used in the Soil Conservation Service (now known as the *National Resources Conservation Service*) methodology for estimating the proportion of precipitation that will runoff the lands and the portion that will infiltrate. CNs are a function of soil type, land cover, slope, and land use. The higher the CN – the greater the proportion of precipitation that is expected to runoff the lands. CNs are representative of the pervious portion of the watershed. Jewell followed the guidance in MTO Design Chart 1.09 to determine curve numbers for the discretized catchments.

Land cover information was obtained from the Ontario Land Cover Compilation (OLCC), a database owned by *Land Information Ontario,* provided by the *Ontario Ministry of Natural Resources and*

Forestry. A review of land coverage for the Rawdon Creek watershed shows that the land use is predominantly cultivated land, woods, and water. A summary of land coverage percentage is provided in Table 4-2.

Table 4-2: Rawdon Creek Land Cover Summary

Land Cover	Area (km^2)	Land Coverage (%)
Woods	41.7	24
Cultivated	85.9	49
Urban	5.1	3
Water	40.7	23
Bedrock	0.7	0

The Rawdon Creek watershed has been divided into eight sub-catchments based on confluence points and nodes of interest (see Appendix B).

Karst topography was also reviewed since it is expected to influence the stream flow gauge readings and subsequently the peak flow simulations. Karstic areas are discussed further in Section 4.3.2. *The Karst Study for Southern Ontario* authored by The Ontario Geological Survey was used to identify the karst limits. A map of expected karst regions as they relate to the Rawdon Creek watershed is included in Appendix C.

4.1.3 Meteorologic Inputs

Environment Canda (EC) intensity-duration frequency (IDF) curves for data collected at the Trenton Airport station is the best available data record (see Appendix E). Jewell reviewed the station data from Kingston, Belleville, and Trenton. The Trenton station yields the longest record of data and is in closest proximity to Rawdon Creek.

Additionally, Environment and Climate Change Canada (ECCC) provided precipitation and stream flow gauge data for the Rawdon Creek Station at Highway 62 in Huntingdon. The discharge values are part of the Water Survey of Canada's primary products and considered a reliable data source. The precipitation data however is provided as-is; meaning the sensor selection, calibration, and placement are not standardized.

Jewell reviewed the data set provided by ECCC. The data provides flows at a 30-minute time steps. The rate of change between time steps was typically less than 0.02cms, but infrequently was 0.05cms. This demonstrates that the record is not likely to be sensitive to artificially high peak flows induced by a short time step. As the modelled hydrograph was 'smooth', this indicates that the likelihood of underestimating a peak due to a sharp watershed response is very low.

The precipitation records were provided in 1-hour increments. Again, the flow record indicates a slow response to the hydrologic input and suggests the 1-hour time increments would be acceptable.

The precipitation data sources were limited. The ECCC precipitation data for the gauge at Rawdon Creek was used only for understanding the distribution of the rainfall. Precipitation depths were not used. Since the precipitation gauge data is noted to be used with caution, the *distribution* of the rainfall gauge was the primary interest from the received rain gauge data set. The distribution was applied to the total depth observed at the more reliable Environment Canada Trenton Airport station.

On September 8th and 9th of 2004 there was a large rainfall event of 24hr duration, which was a large tropical depression from what was Hurricane Frances. This event produced extreme rainfall volumes locally. The *2009 Draft Mayhew Creek Master Drainage Plan* published the hyetograph of the storm rainfall at the Trenton Airport station. This hyetograph is reproduced in Figure 4-2 and has a cumulative rainfall depth of 111.8mm (as reported in the online climate data for Trenton Airport 6158875). Incidentally, the published IDF curves for this station lists the total precipitation recorded for the event as 123.7mm.

LTC provided precipitation and flow gauge data for another large rainfall event that recently occurred in September of 2021. The Rawdon Creek rain gauge was not working during this time period, however LTC provided the precipitation data for the next closest station with a functioning gauge at Squires Creek. This event was selected for the validation event since it produced a significant rainfall volume of 103.2mm (based on an average of the depths at the Trenton Airport EC Station and the LTC manual gauge), and occurred outside of the snow melt season. The precipitation inputs for the event are shown in Figure 4-3.

An important consideration in the precipitation data is the potential impact to rainfall depths due to climate change. LTC, in partnership with FHIMP representatives, identified the recommended approach to quantify increased rainfall depths due to climate change. The methodology, rainfall depths, and peak flow results associated with the climate change scenario are discussed further in Section 4.5.

Jewell also participated in discussions with ECCC staff regarding precipitation statistics and the approach to assess and calculate outliers. As part of these discussions, Jewell acquired and reviewed the ECCC precipitation statistics tool. This review confirmed Jewell's in-house spreadsheet is consistent with the ECCC methodology. Jewell's in-house precipitation tool was used to include the 200- and 500-yr events since these return period events are not included in the standard Environment Canada IDF curves. The spreadsheet calculates the precipitation frequency curve using a Gumbel distribution.

Jewell included a test for outliers in the precipitation records. The 2004 Frances event produced extreme rainfall volumes between Cobourg and Kingston, including the Trenton and Stirling areas. The precipitation totals reported for the event at nearby stations are included below.

Table 4-3: September 8/9 2004, Hurricane Frances Precipitation Summaries

Station		M.S.C			EC IDF Curves	
		Sept 8	Sept 9	Total	12-Hr	24-Hr
Kingston	6104175	57.2	64.6	121.8	NA	NA
Belleville	6150689	81.4	35.5	116.9	114.4	124.5
Trenton	6158875	4.6	107.2	111.8	109.6	123.7
Cobourg	6151689	66.4	27.4	93.8	NA	NA
Cobourg	6151684	N/A			81.8	82.2

For context, the nearest station at Trenton Airport has a 100-yr statistical rainfall depth for 12-hr and 24-hr durations of 96.5mm and 108.1mm respectively. The Hurricane Frances event produced rainfall volumes in excess of a 100-yr statistical storm for a similar duration.

A rainfall depth with a standard deviation of 2.5 would be within the 95% confidence interval; the 2004 rainfall depth was found to be 5.7 times the standard deviation from the mean, corresponding to a theoretical 312-yr return period. All of the measured rainfall data has been included in this analysis. However, we note that the outlier resulting from the 2004 data may be omitted for statistical correctness at the discretion of the project partners. The large rainfall event in 2004 that skews the data set (see Table 4-5) could be considered a historic event, and it may be reasonable to have it categorized alongside the Timmins storm. This suggests that the estimate of 100-yr return period peak flows will be conservative.

The Timmins event has a rainfall depth of 193.0mm and follows the distribution identified in *Design Chart 1.04* of the *1997 MTO Drainage Manual*. Given the size of the Rawdon Creek watershed, an area reduction factor of 76% is applied to the precipitation data. This reduction factor was selected from Table D-4 of the *MNR 2002 Technical Guide* using an equivalent circular area of 391.6 km^2. The equivalent circular area was derived from an equivalent circular diameter of 22.3 km, when measured at the point of interest at Node E (see Appendix B-2). Since the Timmins event is more severe than the 2004 rainfall, it would continue to govern in an assessment of historical storms.

Table 4-4: Timmins Event with Areal Reduction

Equivalent Circular Diameter.		**22.3**	**km**
Equivalent Circular Area		**391.6**	**km2**
% of Timmins Storm Required		**76%**	
Hour	**Depth (mm) No Reduction**	**Depth (mm) With Reduction**	**Depth (mm) Climate Change**
1	15	11.4	14.3
2	20	15.2	19.0
3	10	7.6	9.5
4	3	2.28	2.9
5	5	3.8	4.8
6	20	15.2	19.0
7	43	32.68	40.9
8	20	15.2	19.0
9	23	17.48	21.9
10	13	9.88	12.4
11	13	9.88	12.4
12	8	6.08	7.6
TOTAL	**193.0**	**146.7**	**183.4**

Table 4-5: Unadjusted vs. Adjusted Trenton Airport 24-Hr Rainfall Depths

Storm Event	Rainfall Volume (mm)		% Reduction
	Unadjusted	2004 Outlier Removed	
5-yr	65.9	62.0	6.3%
50-yr	98.6	86.7	13.7%
100-yr	108.1	94.3	14.7%
200-yr	117.6	101.8	15.6%
500-yr	130.1	111.6	16.5%
*Timmins	193.0	-	-

**Timmins Storm from MTO Design Chart 1.04*

The recommended return period storms for floodplain mapping are derived using SCS and AES distributions with varying durations. As suggested by the MNR 2002 *Flooding Hazard Limit Technical Guide*, Jewell tested various storm durations for the rainfall-runoff simulation to determine the storm duration that best reflected the watershed characteristics and was most similar to the watershed time of concentration. In this assessment, Jewell compared the peak outflows from the HEC-HMS hydrologic model (see Section 4.6) for the 6, 12, and 24-hr duration events with both distributions. Any event less than 6 hours was not tested, as the Rawdon Creek watershed is rather large, and shorter duration events would not produce significant enough rainfall volumes to govern as the regulatory storm event nor would they reflect an appropriate time of concentration for the watershed.

The results are summarized in Table 4-6. Since the 24-hr duration with an SCS distribution produces the largest peak runoff rate and would most closely reflect the time of concentration of the watershed, this criterion was selected for the rainfall-runoff model discussed further in Section 4.3. The AES distribution was not selected since it produced lesser flows than the SCS Type II distribution. The 24-hr SCS type II distribution was carried forward to the HEC-HMS modelling effort.

Table 4-6: Comparison of 100-Yr SCS Distributions with Varying Storm Durations (No Adjustment to Rainfall Depths)

Storm Duration (hr)	SCS Type II (m^3/s)
6	27.5
12	36.1
24	42.1

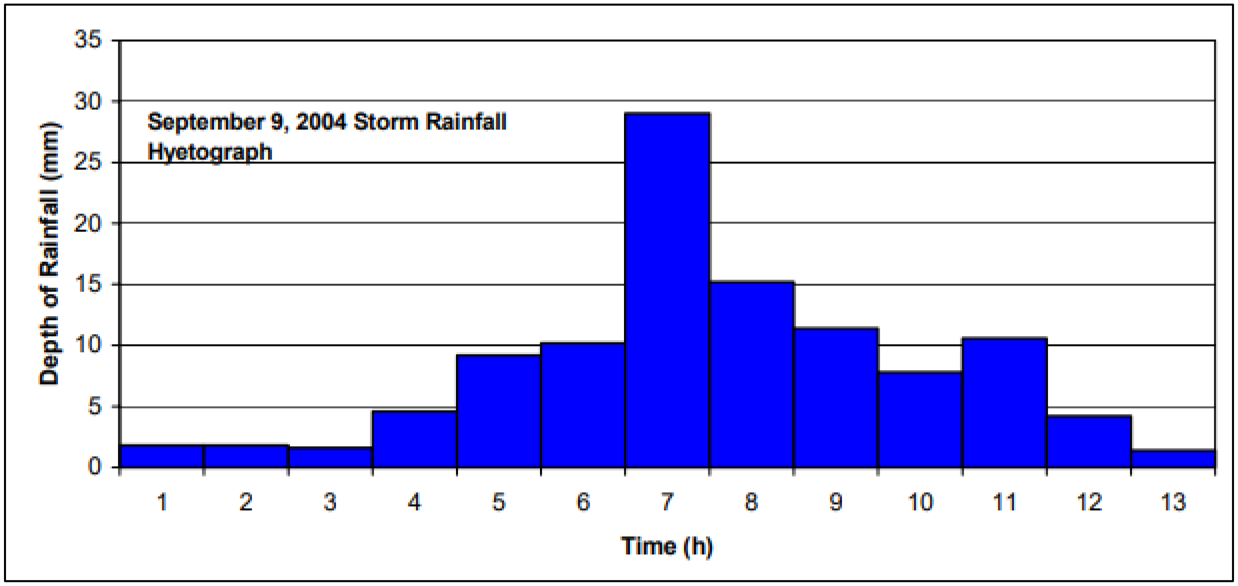


Figure 4-2: Excerpt from 2008 Potter Creek MDP Illustrating the September 2004 Hurricane Frances Rainfall Hyetograph

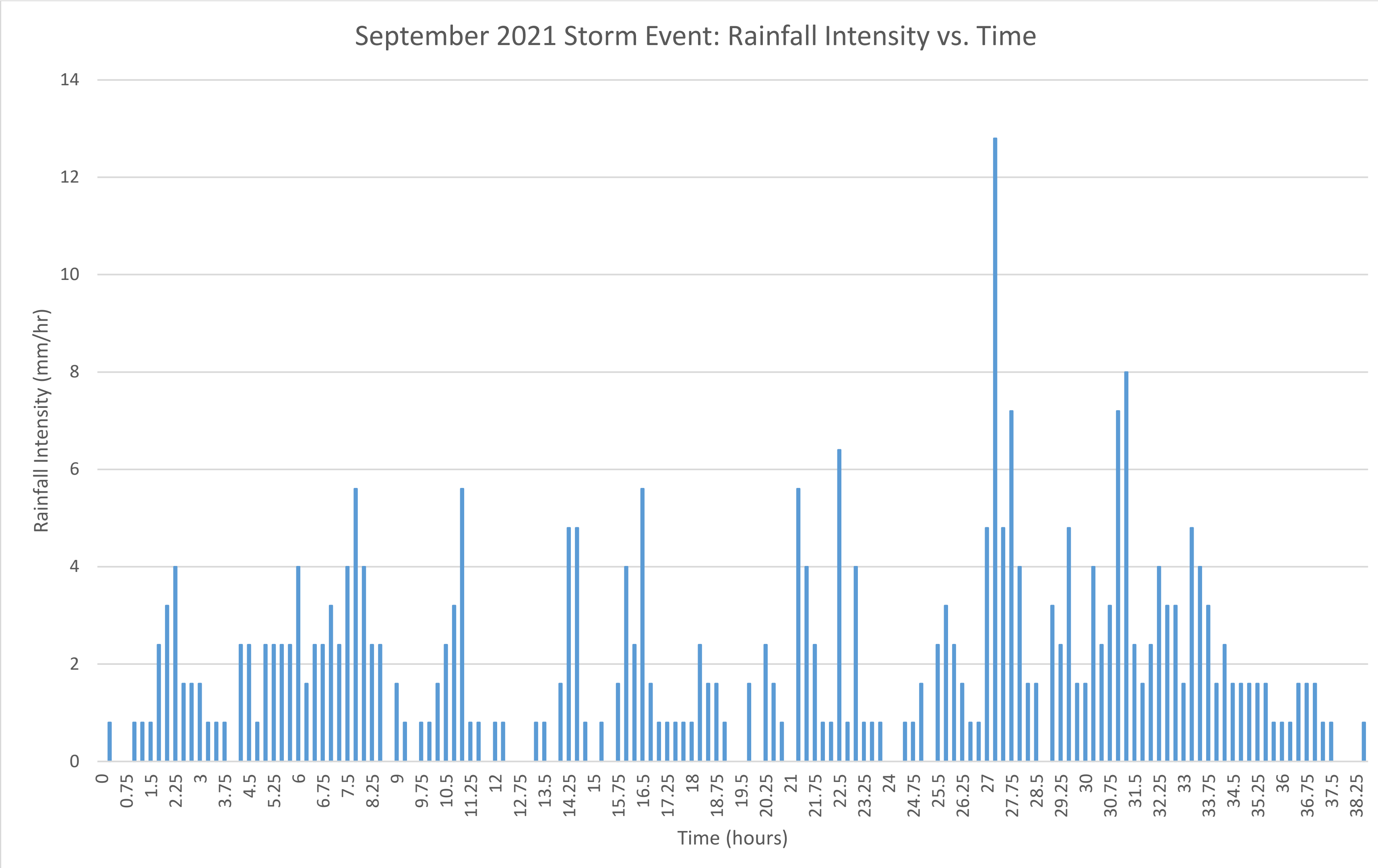


Figure 4-3: Measured Rainfall Input for September 2021 Validation Event

4.1.4 Water Survey of Canada Stream Flows

Water Survey of Canada (WSOC) operates a stream gauge (station 02HK008) named 'Rawdon Creek Near West Huntingdon'. The gauge is located just south of the intersection of Highway 62 and County Road 8, with a receiving drainage area that includes the upper half of the Rawdon Creek watershed.

The flow data of interest is the *Annual Maximum Instantaneous Peak Discharge.* The record length for the gauge is from 1983 to 2022, with 31 years of annual instantaneous maximum peaks. The stream flow gauge location is shown in Figure 4-4. Discharge data was provided from ECCC and LTC in 5-minute intervals for use in the calibration and validation model runs.

Table 4-7: Rawdon Creek Stream Flow Gauge Information

Name	Station ID	Length of Record	Drainage Area (km^2)
Rawdon Creek near West Huntingdon	02HK008	1983 - 2022	93.0

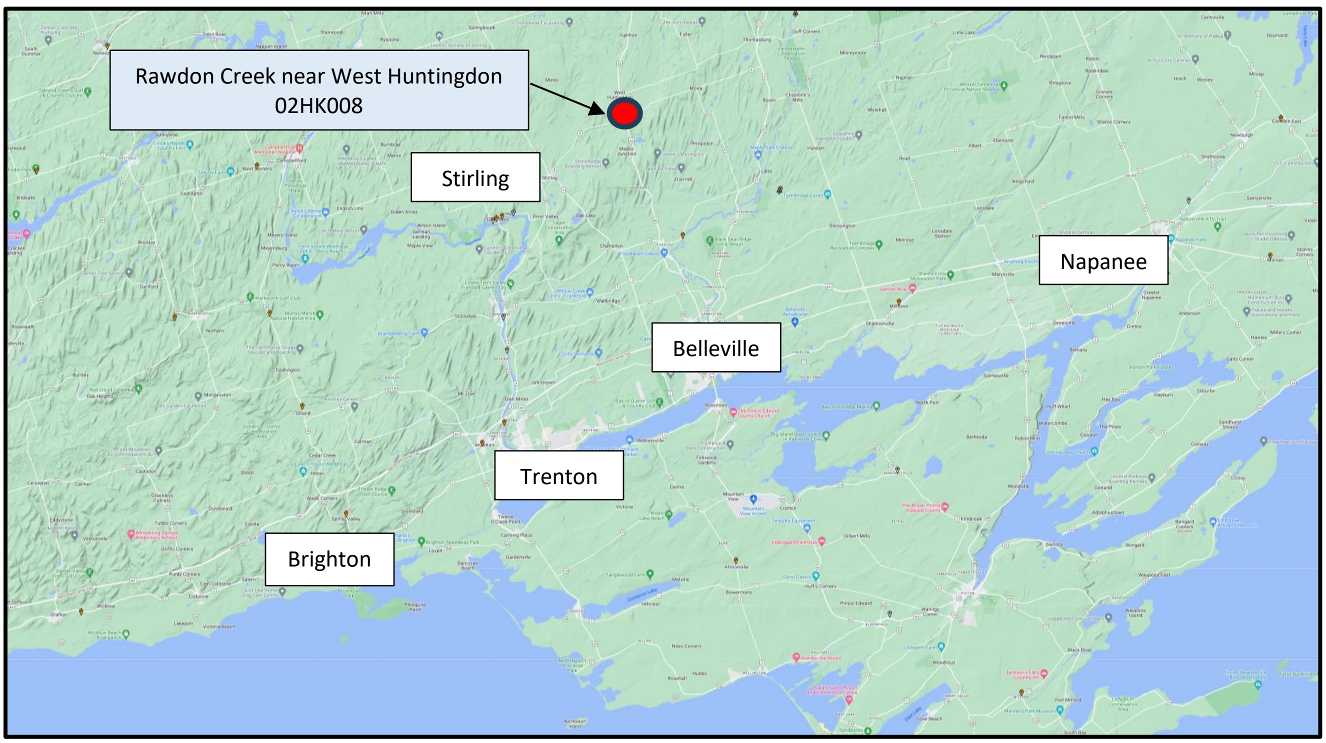


Figure 4-4: Rawdon Creek Water Survey of Canada Stream Flow Gauge Location

4.2 Flood Frequency Analysis

The Consolidated Frequency Analysis (CFA) V3.1 was used to complete the general frequency analysis with the 3-parameter lognormal distribution. The detailed results are reported in Appendix G for frequencies from the 2-yr up to the 500-yr return period.

From an assessment of the stream flow records, it is evident that the majority of the annual instantaneous peaks occur in the spring. For the 31-yr data record of annual instantaneous peaks at the WSOC Rawdon Creek flow gauge, only three (3) years had instantaneous peaks outside the months of January to April. This suggests a 90% probability that a severe flood event would be the result of a snow-melt event, or a combination of a snow-melt and precipitation event. The stream flow gauge records provide the best indication of the anticipated flow rates during a snow-melt and/or combined snow-melt plus precipitation event.

The CFA results for 2- through 500-yr return periods are summarized in Table 4-8. The results in Table 4-8 represent the expected peak flows at the Rawdon Creek flow gauge location. For return period flows that include the entire Rawdon Creek watershed, a transposition of flows can be investigated (see Figure 4-5). The transposed return period flows for the full watershed (point of interest E) are summarized in Table 4-9.

Table 4-8: Summary of Maximum Peak Flows at 02HK008 from CFA General Frequency Analysis

Return Period	Peak Flow (m^3/s)
2-yr	7.1
5-yr	9.8
10-yr	11.9
20-yr	14.1
50-yr	17.1
100-yr	19.6
200-yr	22.2
500-yr	26.0

Transposition and interpolation of data from a stream gauge can be done based on the Modified Index Flood method as follows:

$Q2 = Q1\ [A2 / A1]^{0.75}$
Where:
Q1 = Known peak discharge
Q2 = Unknown peak discharge
A1 = Known basin area
A2 = Unknown basin area

Figure 4-5: Excerpt from MTO Online Drainage Manual (Ministry of Transportation Ontario, 1997)

Table 4-9: Summary of Maximum Peak Flows at Node E using Transposition of Station 02HK008

Return Period	Peak Flow (m^3/s)
5-yr	16.2
50-yr	25.2
100-yr	27.7
200-yr	30.4
500-yr	34.0

The transposition of flows technique assumes the watershed conditions to the flow station would be representative of the ungauged portion. But this assumption would not hold true here. The Rawdon Creek flow gauge provides representative flow data for the *upper* half of the watershed. It receives the upstream drainage areas represented by Sub-Catchments 401, 402, and 301. In a review of the karst topography map in Appendix C, one can see that the catchments to the Rawdon Creek flow gauge are almost entirely within areas of potential karst and therefore may not be representative of the downstream watershed hydrologic response. In a comparison of the flow gauge readings to other local gauges, and standard hydrologic inputs as applied in the 1975/1985 Rawdon Creek floodplain studies, the Rawdon Creek flow gauge yields peak flows far less than one would expect given its relatively large receiving area of 93 km^2.

LTC, the federal partners, and Jewell participated in a meeting on August 25, 2023 to discuss the lower-than-expected flow records. LTC's engineering representative has extensive experience with the behaviour of local creeks within the Lower Trent region, including Rawdon Creek; LTC noted that Rawdon Creek is prone to karstic landscapes and that karst topography has been observed in historical site visits within the vicinity of the Rawdon Creek.

In the context of the karstic landscapes the observations of the muted shape, peak, and volume of the measured runoff events becomes more reasonable. In addition, it has already been noted that there is substantial lake and wetland coverage upstream of the stream flow gauge. Lakes and wetlands have the capacity to store a large volume of runoff, contributing to the low peak flows at the stream gauge.

This interpretation of the stream flow gauge readings is further discussed in the calibration and validation subsections (see Section 4.3.5 and 4.3.6). Ultimately, the known and potential karst landscape areas cover majority of the catchment areas contributing to the flow gauge. However, for the sub-catchment areas *downstream* of the flow gauge but *upstream* of the Village of Stirling, the karst coverage becomes less prominent although still significant.

The transposition of flows technique employs accurate data up to the hydrologic node that coincides with the flow gauge location. However, the transposition method is based on the assumption that catchment characteristics are uniform over the area subject to the transformation. Since this is not a valid assumption at Rawdon Creek, the transposition approach would *underestimate* the peak flow values at the outlet.

Given the smaller percentage of lands with potential karst downstream of the flow gauge, the prudent approach is to apply a hydrologic model rather than the transposition of flows equation for the purpose of determining the flows at the Village of Stirling.

The calibration and validation of the hydrologic model was completed at the location of the flow gauge. Downstream of the flow gauge, calibrated losses are then *only* applied to areas with a karst designation per the GIS mapping illustrated in Appendix C. Further discussion on the rainfall-runoff modeling is provided in the following subsection.

4.3 Rainfall-Runoff Modeling

The SCS Curve Number (CN) method is commonly applied in hydrology models for precipitation-driven runoff modeling applications. It relies on the soils and land use data to establish the loss method with calculation of a CN. The modeling approach is supported by Visual OTTHYMO (VO) and HEC-HMS.

All modelling programs are simplifications of reality and limited in their capabilities. While VO and HMS are both well-established and recommended software programs, they are limited by input parameters and the uncertainty associated in the data sets and calculations used to produce these inputs. The modelling programs are acceptable for simulating peak flows to be used in the hydraulic model. The most recent software publications have been used for this project.

Both models were employed to simulate the watershed response, but the HEC-HMS model provided more satisfactory results and was selected for the modelling exercise. In discussions with the ECCC technical team, it was agreed that HEC-HMS is the more suitable program for the subject watershed.

Notable input parameters for the HEC-HMS model include:

- Precipitation – intensity, duration and frequency as well as distribution.
- Catchment area.
- Soil conditions – these determine how much and how quickly water will be removed from runoff through infiltration. This may be expressed as a curve number, or by a runoff coefficient or using an infiltration model such as Horton's Infiltration.
- Slope – peak flows increase with slope.
- Initial abstraction – depth of precipitation input that is subtracted from the model and does not contribute to runoff.
- Manning's n – frictional coefficient that affects the time to peak.
- Basin lag or time to peak.

Baseflow was not included as an input parameter for the model, as baseflows observed from the provided data were very low (less than 0.5cms). Design flows used by Jewell were generally conservative, as best seen in Section 4.6, where there was substantial fitting to mute the modelling response, and the resulting peaks are still believed to be conservative. The inclusion of additional base flow would be a 'rounding error' in the mapping.

4.3.1 Hydrologic Input Summary

A hydrology input summary is provided below for existing land cover and full development conditions based on the zoning identified in the Official Plan. This summarizes the area, curve number or imperviousness, and time of concentration applied for each sub-catchment. Watershed length is calculated as the longest flow path. The input summary is shown for AMC II conditions as this setting was applied for the peak flows in the hydraulic modeling. For the calibration events, the only difference is that AMC I was applied to avoid underestimating the peak flows used in the floodplain modeling. For AMC I, the curve numbers were converted using MTO Design Chart 1.10.

Table 4-10: Summary of Hydrology Inputs for Rawdon Creek Catchments for AMC II

Catchment ID	Area (km2)	Initial Abstraction	CN	Impervious (%)	Time of Concentration (hr)	Storage Coefficient (hr)
100	4.96	5.00	75.5	12.9	8.00	23.98
101	20.30	5.00	73.2	1.0	9.25	27.65
201	15.95	5.00	68.3	1.0	8.72	26.15
301	20.08	5.00	62.8	1.0	7.70	23.07
401	52.73	5.00	51.3	1.0	20.91	62.72
402	16.69	5.00	54.1	1.0	13.16	39.44
501	23.25	5.00	58.6	1.0	15.14	45.44
502	20.60	5.00	56.5	1.0	13.29	39.41

Note that the initial abstraction was set to a value of 5mm for all catchments. Conversations with ECCC led to the conclusion that an adjustment to sub-catchment curve numbers, as opposed to initial abstraction, was more appropriate for accounting for the karst and wetlands in the calibration of the model (discussed further later).

4.3.2 Loss Method

Jewell selected the SCS curve number (CN) loss method since it accounts for both land cover and hydrologic soils group information. It was also selected because of the reputable sources available for this information. CNs were selected based on guidance from the CVC SWM guidelines in addition to MTO Design Charts. A look-up table was used to connect each land cover sub-area to its corresponding soil type. Attribute tables in ArcGIS were utilized to develop the detailed weighted curve number applied to each sub-catchment.

AMC II, per Chapter 8 of the MTO Drainage Manual, was applied for antecedent moisture conditions (AMC). This represents ‘average’ soil conditions. Saturated soil conditions (AMC III) were not selected because this condition, combined with the statistical return period rainfall events, would produce a peak flow beyond the selected return period frequency. Saturated conditions were also not selected because the General Frequency Analysis already accounts for spring melt conditions since the instantaneous annual peaks in the flow gauge data sets consistently occur during the spring snow-melt season.

The CNs were adjusted to account for karst areas. A map of the karstic areas within the sub-watersheds is shown in Appendix C. For non-karstic areas, the CNs were applied based on standard values for AMC II conditions. For karst areas, a factor of 0.85 was applied to the standard CN values (see Table 4-11). The karst factor was established based on the calibration and validation process. A weighted CN was applied to each sub-catchment based on its percentage of expected karst topography. For example, Sub-catchment 502 is entirely susceptible to karst landscapes, and subsequently the standard CN for the watershed is adjusted by a factor of 0.85. On the other hand, Sub-catchment 100 has 0% anticipated karstic landscape, and subsequently has no factor applied.

Table 4-11: Calibrated Curve Number Adjustments for Karstic Areas

[1]Catchment	Area (km^2)	Potential Karst Coverage	CN		Weighted CN
			[2]Karst	AMC II	
100	3.7	0%	64.2	75.5	75.5
101	20.3	23%	64.4	75.8	73.2
201	15.9	0%	58.1	68.3	68.3
301	20.1	51%	57.8	67.9	62.8
401	52.9	99%	51.2	60.2	51.3
402	16.2	81%	52.4	61.6	54.1
501	23.2	90%	57.6	67.7	58.6
502	20.6	100%	56.5	66.5	56.5

[3]Karst Factor:	0.85

[1]Sub-catchments 301, 401, & 402 draining to WSOC flow gauge used in calibration of curve number due to potential karst topography.

[2]Karst CN is equal to AMC II multiplied by the karst factor.

[3]Karst factor derived from calibration & validation of model results to WSOC flow gauge recordings.

4.3.3 Lag Time / Time of Concentration

Jewell applied the SCS Lag Time method to determine time of concentration and lag time values. This method was selected since it is derived from a study of watersheds that have drainage areas up to 24 km^2 with an upper limit of approximately 50 km^2. The sub-catchments within Rawdon Creek are less than 24 km^2 and within their recommended limits. The exception is Sub-Catchment 501 that has a drainage area of 53.0 km^2, which is near the upper limit of the acceptable range. However, Sub-Catchment 501 is within the portion of the watershed that contributes to the stream flow gauge, meaning it has been adjusted to account for the measured lag times in the calibration and validation process. The SCS lag time method was also selected because it accounts for land cover and soil types by incorporating the CN value to estimate a retardance factor. The SCS lag time method is described in the *Hydrology National Engineering Handbook* published by the United States Department of Agriculture and the Natural Resources Conservation Service.

In part of the calibration process, the measured lag time was used to adjust the time of concentration calculated from the SCS lag time method. It was found that the measured lag time was significantly longer than the lag time obtained from the SCS method. Therefore, lag times were adjusted to replicate that timing of the hydrographs from past storm events (see comparison of observed vs. modeled events in Figures 4-7 and 4-10). With the application of the Clark Unit hydrograph, a storage coefficient was also carefully selected. Different storage coefficients were applied in the calibration method to identify a storage coefficient that is representative of the shape of observed hydrographs from past events.

The time of concentration was tested with and without adjustments for karst-specific areas to determine whether adjustments should be made separately for karst regions. The testing showed that adjustments *only* to areas with karst landscapes would create a dual peak in the hydrograph outputs since it would create a large separation between karst region lag times and non-karst region lag times. In reality, the observed hydrographs are smooth and show no signs of dual peak action. Therefore, the lag time adjustments in the calibrated model were applied to each sub-catchment rather than karst-specific regions only.

4.3.4 Channel Routing

Channel routing was completed using the Muskingum-Cunge method. This method is applicable for reaches with relatively small slopes and allows the user to input a cross-section to represent the ground surface data for the channel and overbank areas. Cross-sections were obtained from the terrain data and then simplified into eight-point cross-sections that are representative of their respective reach length (see Appendix K). The Muskingum-Cunge method was also selected since it incorporates Manning's n values to represent expected roughness for the channel and overbank areas. The applied Manning's n values are based on the design charts in the *MTO Drainage Manual*. Table 4-12 summarizes the reach name, length, slope, and Manning's values.

Table 4-12: Muskingum-Cunge Channel Routing Dimensions

Route	Length (m)	Slope (%)	Manning's n			Index Celerity
			Left	Middle	Right	
Reach 3	7310	0.09	0.05	0.05	0.05	0.4
Reach 4	10277	0.09	0.05	0.05	0.05	0.4
Reach 5	9375.2	0.07	0.05	0.05	0.05	0.4

4.3.5 Calibration

The in-depth calibration and validation assessment for Rawdon Creek was triggered by large discrepancies between the historical 1985 peak flow estimate and the observed stream flow gauge results. The flow gauge produces far lower runoff rates relative to the 1985 peak flow estimates. The 1985 hydrologists would not have had a sufficient stream flow record available at the time of their analysis to verify their peak flow calculations. In 2024, a sufficient record of data has been accumulated at the Rawdon Creek flow gauge for use in a statistical assessment of the peak runoff rates. The Rawdon Creek flow observations suggest that "off-the-shelf" design charts and equations to calculate losses and lag times would not adequately account for unique characteristics attributed to this watershed; emphasis was placed on the need for calibration and validation of the Rawdon Creek hydrologic model.

The calibration event for Rawdon Creek is driven by the Frances event that primarily occurred in September of 2004 as described previously in Section 4.1.3.

The measured precipitation data was supplied to the HMS model (see previous Figure 4-2). The objective is to obtain an outflow hydrograph from the HMS model that produces similar values to the observed flow gauge hydrograph. In doing so, the CN is adjusted to replicate the volume and amplitude of the response, and then the time of concentration parameter was adjusted to fit the observed time of peak. Since each parameter had an impact on the time of peak and amplitude, the process was iterated until a reasonable response was determined. A further refinement of CN was made for areas with Karst regions by inclusion of a factor for the CN. The factor was adjusted until a good approximation of the peak flow resulted.

The flow gauge hydrograph is shown below in Figure 4-6. The key hydrograph metrics are peak flow, time to peak, and runoff volume.

The observed flow hydrograph has a shallower falling limb relative to the rising limb due to the local karst topography. This shape is similar to what occurs in a flow attenuation scenario, such as a reservoir with a controlled outflow. In this case, it is not a surface reservoir that is believed to be causing this hydrograph shape. Rather, it is expected to be the result of the karst topography. In karst landscapes, water can seemingly disappear from the surface runoff and reappear as baseflow downstream. The shape of the observed flow hydrograph suggests this to be the case and is further supported by an assessment of the observed flow hydrograph and the *measured* runoff coefficient.

Recall that the precipitation depth is 111.4mm. The watershed area contributing to the flow gauge is 93 km^2. The total rainfall volume is 768 ha-m. The *runoff* volume at the stream flow gauge is 289 ha-m. In other words, the measured runoff yield (as a ratio of runoff to rainfall volume) is 0.38. This is high relative to published values from provincial guidelines, particularly considering that the watershed contributing to the flow gauge is comprised of predominantly rural lands with 30 percent lake and wetland coverage (Ministry of Transportation Ontario, 2023). It is also high relative to the runoff coefficient outputs from the calibrated HMS model. The weighted runoff coefficient output for the three sub-catchments contributing to the flow gauge in the calibrated HMS model is 0.18.

This raises the question: *why would a stream flow gauge with notoriously low peak flows produce measured runoff coefficient nearly double the expected value?* The conclusion is summarized below.

In the studied event, the greater than expected runoff volume and lower hydrograph peak indicates there is more runoff volume being produced at the flow gauge than can reasonably be expected from a rainfall-runoff model when accounting for the full length of the falling limb. This supports the understanding that precipitation that is withdrawn through infiltration is rapidly returning to the surface and contributing to the creek flow. High infiltration soils can also return flows to the creek system as base flow, which can extend out the shape of the hydrograph recession. It is likely, however, that high infiltration soils would contribute more to the hydrograph recession than the peak. The Karst, on the other hand, would contribute to both the peak and the recession curve.

Rainfall-runoff models for hydrology applications will remove runoff volume from the system based on the selected loss method and not return it, whereas in reality precipitation can be *temporarily* removed, subsequently detained in the subsurface, and ultimately contribute to baseflow later in the system. This helps to understand the shape of the hydrograph, its runoff volume, and its peak.

A karst factor was applied to the CNs to calibrate the HMS simulations. The karst factor was described previously in Section 4.3.2. This adjustment factor was determined specifically for Rawdon Creek based on the calibration and validation tests described herein.

Antecedent moisture condition (AMC) I was selected for the model calibration given that the large past rainfall storms occurred during relatively dry periods of the year with no significant precipitation leading up to the events. An AMC II adjustment was completed to ensure the flows used in the regulatory mapping is reflective of average moisture conditions.

The HMS peak flow compares well with the measured peak as shown in Figure 4-6. The validation component is described in the following subsection.

In summary, calibration of the model was completed through a 3-step process.

- ***Step 1:*** Calibrate HEC-HMS model to the Frances event by adjusting CN, storage coefficient and time of concentration.
- ***Step 2:*** Calibrate model to the GFA 1% AEP flow using a global adjustment to curve number.
- ***Step 3:*** Calibrate to remaining return period events with dynamic AMC adjustment.

4.3.5.1 Step 1: Calibrate to Hurricane Frances

Firstly, the HEC-HMS model was calibrated to the 2004 Hurricane Frances storm event. The Hurricane Frances event occurred on September 9th 2004 during a dry period and the base flows were small. Given the dry conditions, antecedent moisture condition (AMC) I was selected for the model calibration.

Calibration parameters included the CN, storage coefficient and time of concentration. Parameters were adjusted until the shape of the modelled hydrograph achieved a reasonable fit relative to the recorded data. Figure 4-6 shows the hydrograph of calibrated model and observed flows.

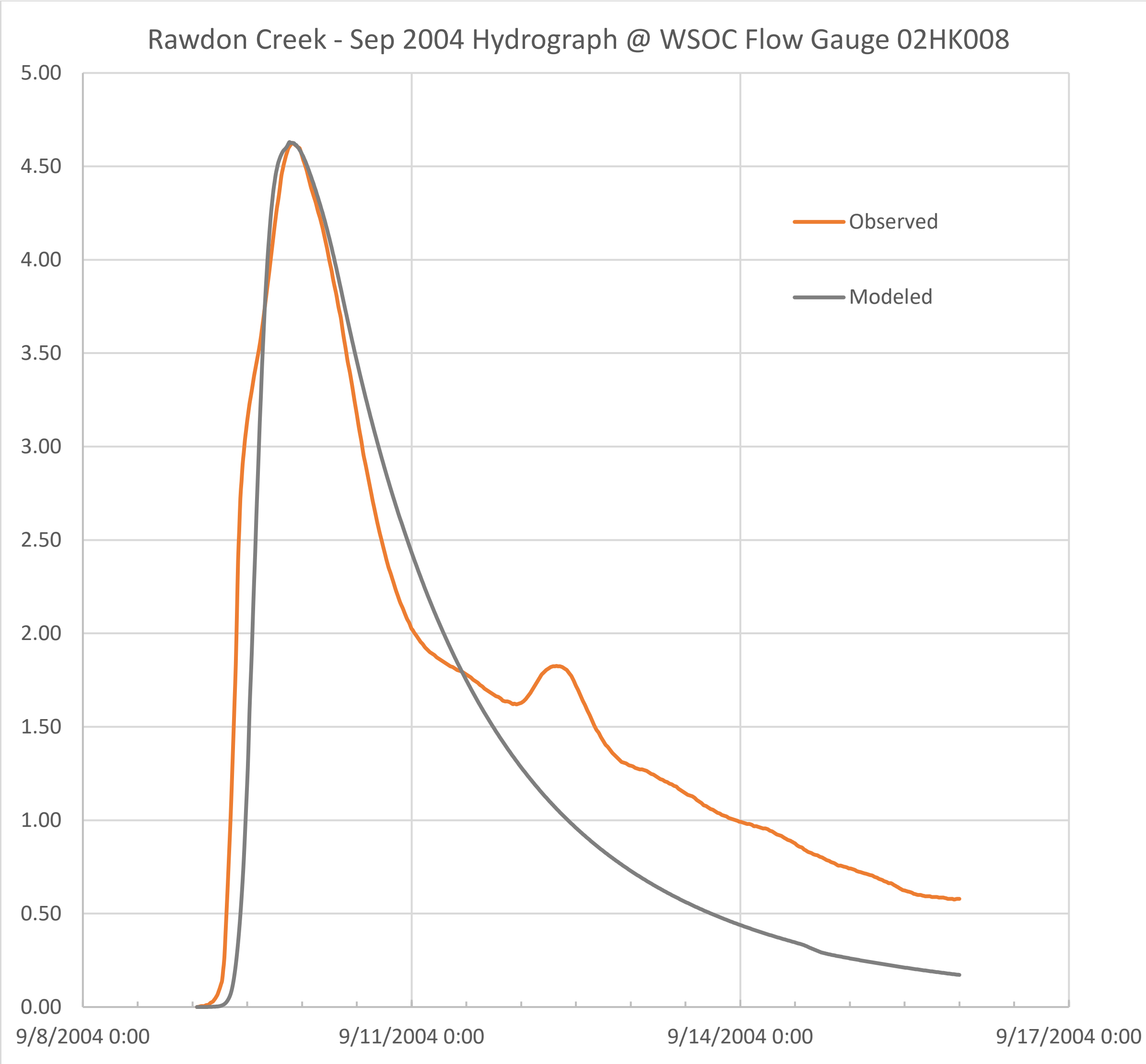


Figure 4-6: Hurricane Frances Calibration to Rawdon Creek at WSOC Flow Gauge 02HK008

4.3.5.2 Step 2: Adjust the HMS model to 100-Yr GFA

The general frequency analysis results are plotted on a semi-log scale in Figure 4-7 as the blue line. Since the gauge results are dominated by spring melt event and the Timmins is a late summer rain only storm, the general frequency analysis was re-run using the rain only events. This produced the green line. By extension, a perfectly fitted hydrologic model would project larger events following the trajectory of the green line for all return period events and for the Timmins event.

After the shape and amplitude of the hydrograph was calibrated to the Frances event, the second step was to calibrate to the 100-yr return period event. This was completed by applying a global factor adjustment to the CNs until the 100-yr peak was satisfactorily reproduced by the model. This adjustment took into account the change from AMC I to AMCII conditions such that the adjusted model represented the AMC II conditions.

Once fitted, the other return period events were simulated and are plotted as the yellow line. It is evident that the calibrated HEC-HMS model would be expected to underestimate the Timmins event.

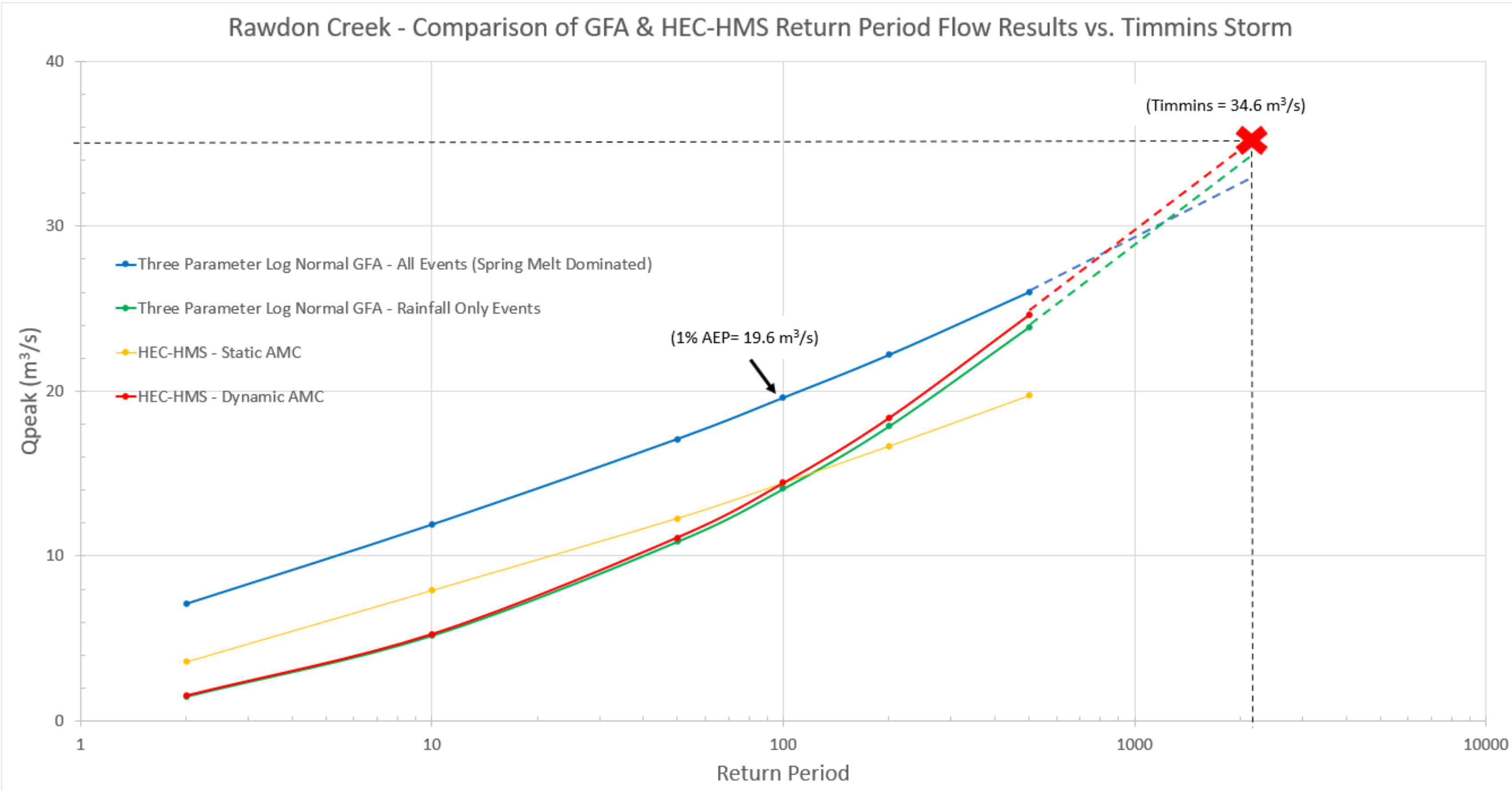
4.3.5.3 Step 3: Calibration to the Full Range of Return Period Events

In order to correct the model for the full range of return period events, the HEC-HMS model is fitted to the GFA curve using a dynamic AMC adjustment. Antecedent moisture conditions, accounted in the curve numbers, which were matched at the 100-yr return period frequency, were adjusted higher for the more frequent return period events and lower for the less frequent events. CN value adjustments were tested in the HEC-HMS model by iteration until a good agreement was found with the GFA results. The process was repeated for each of the return period events.

CN adjustments were factored from the calibrated value such that the 100-yr factor is 1, the 2-yr factor is 0.55 and the 500-yr factor is 1.17. The Timmins factor of 1.29 is found by extrapolating the relationship to the Timmins precipitation depth (see Figure 4-9).

The dynamically corrected model is presented at the red line that follows closely, but slightly above the green GFA curve. The Timmins flood event is found to be 34.6cms at the location of the stream gauge. By this method, a slightly higher prediction of Timmins flood is made as compared to a prediction that would have resulted from a hydrologic model simply calibrated to the GFA results (blue line). The dynamic adjustments are indicated in Figure 4-8.

It is important to note that the Timmins peak flow was derived from calibration to rainfall only events. For the other return period storms, the spring melt flows increase the flows relative to rainfall only events as shown by the blue line. In the hydraulic model simulations, the peak flows that correspond to points along the blue line were applied for the return period events. This was done by applying a factor >1 to the inflow hydrographs obtained from the calibrated AMC II rainfall model by an appropriate until the peak flows matched the points shown along the blue line in Figure 4-7.



**Flow results in above chart measured at Rawdon Creek Stream Flow Gauge Location (02HK008)*

Figure 4-7: Comparison of GFA & HEC-HMS Return Period Flows vs. Timmins Storm

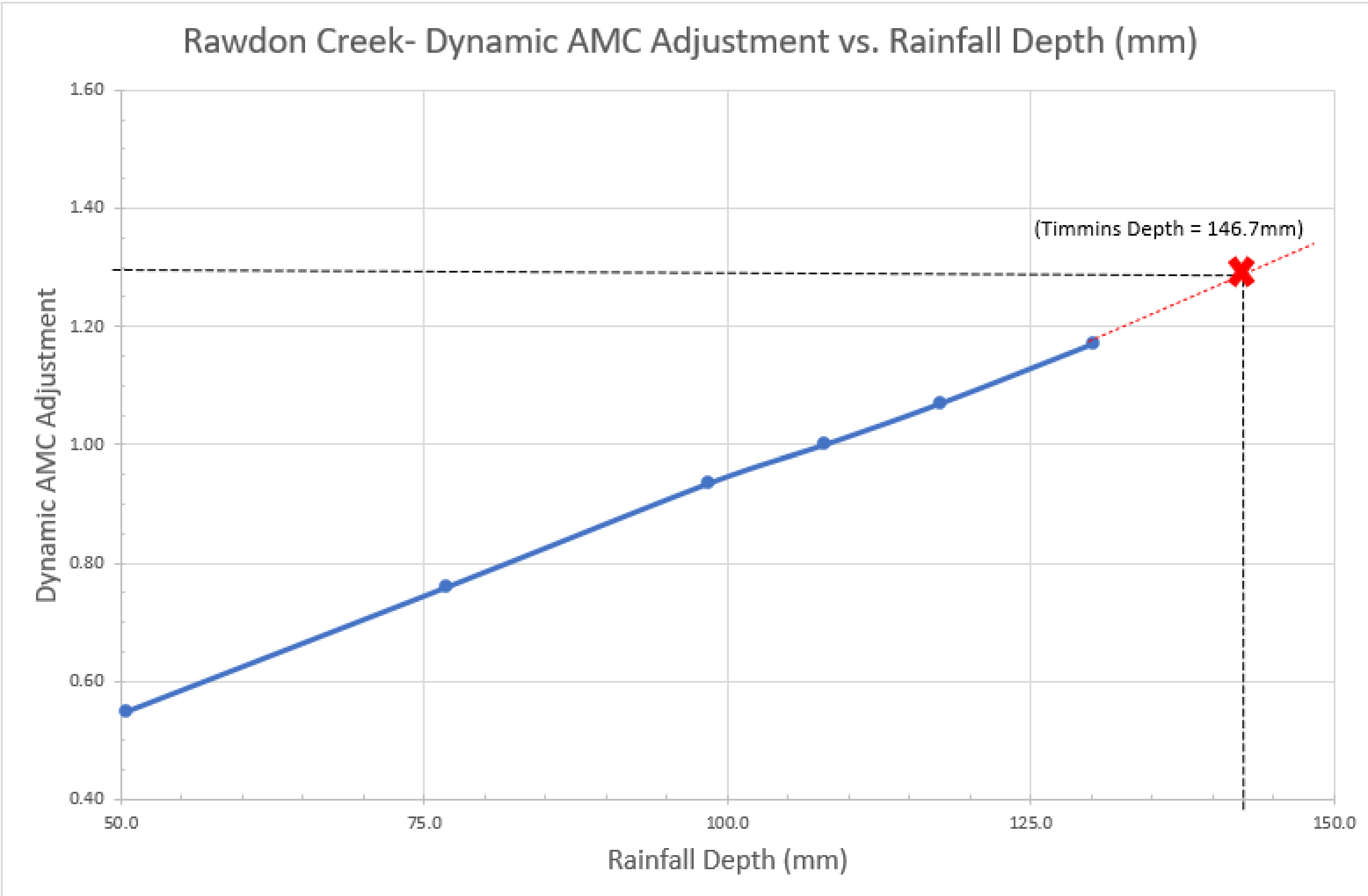


Figure 4-8: Dynamic AMC Adjustment with Varying Rainfall Depths

4.3.6 Validation

LTC provided precipitation and flow gauge data for a large rainfall event that recently occurred in September of 2021. Recall the precipitation data for this event was illustrated in Figure 4-3. Shortly after this storm event, LTC's Water Resources Manager issued a memo (**see Appendix H**) identifying how the precipitation affected the local streams. Two of nine measured creeks reached their 2-yr bankfull flow. Five creeks reached more than half of their 2-yr bankfull flow, and two did not reach half of their 2-yr flow. The two that did not reach their bankfull flow were Rawdon Creek and Hoards/Squires Creek; both of which are noted to have karstic landscapes.

The calibrated parameters were applied to the validation event. An AMC I condition was applied given that the minimum instantaneous discharge for 2021 occurred a few weeks prior on the 4th of September and the was no meaningful precipitation on the days immediately prior to the event, suggesting dry moisture conditions.

A comparison of peak flow results between the flow gauge data and model outputs in the validation storm shows that the reality vs. modeled outputs compare well with one another (see below). This confirms that the calibration parameters are suitable for use in the rainfall-runoff model.

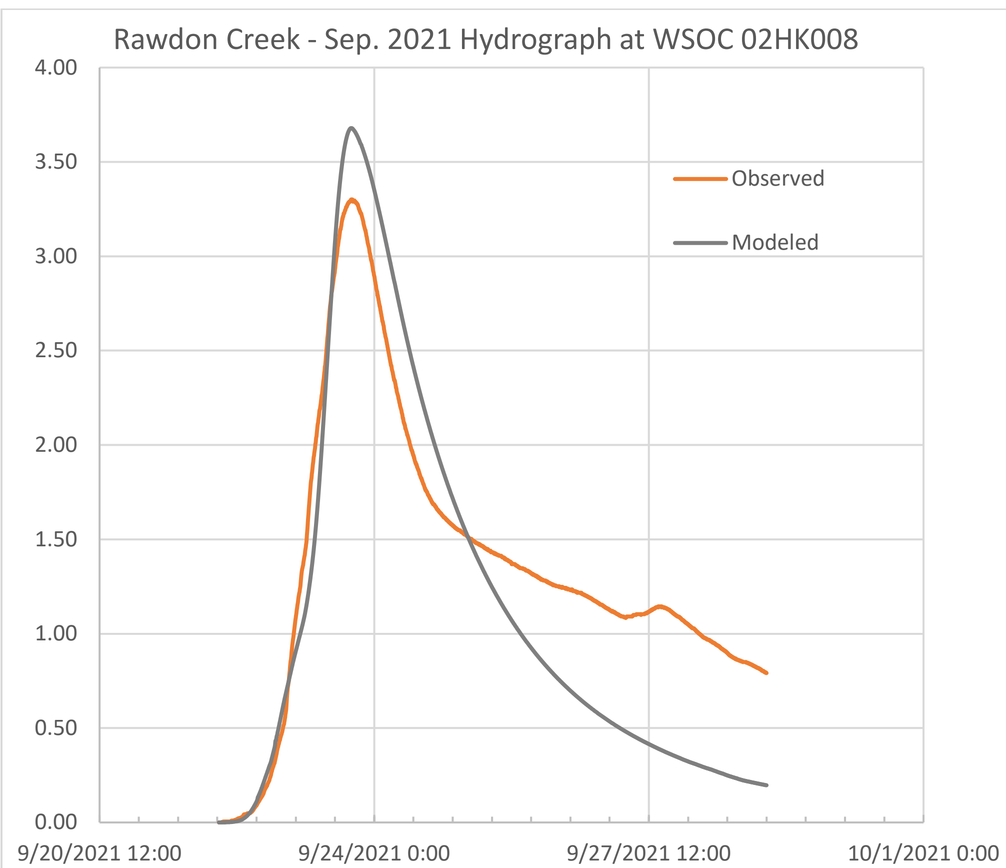


Figure 4-9: Sept 22, 2021 Validation Event – Observed vs Modelled showing Good Agreement

Table 4-13: Comparison of Observed vs. Modeled Results for Calibration and Validation Events

Parameter		Sep. 2004		Sep. 2001	
		Observed	**Modeled**	**Observed**	**Modeled**
1	Q_{peak} (m^3/s)	4.63	4.63	3.30	3.68
2	Time to Peak (hr)	21.0	20.3	40.6	0.0
3	Volume (ha-m)	48.9	47.4	29.0	30.4
4	Soil Conditions	Dry	AMC I	Dry	AMC I

1. *Q_{peak} represents instantaneous peak flow.*
2. *Time to peak measured from start of precipitation to time of hydrograph peak.*
3. *Volume represents runoff volume until inflection point on receding limb of observed hydrograph.*

4.4 Index Flood Analysis

The Index Flood Analysis was employed following the methodology established by the Ontario Ministry of Natural Resources to estimate design flows and assess the hydrology of the contributing drainage area.

The Index Flood method relates the annual peak instantaneous flow determined for 247 stream gauges across Ontario to drainage area (Ministry of Natural Resources and Forestry, 2020). Twelve regions across the province were identified as having similar characteristics and a regression curve was developed for each region. See Figure 4-8. Note that the Rawdon Creek watershed is located near the boundaries of Regions 1 and 9.

The Index Flood method is a useful tool to estimate return period flows for many local creek systems. However, since the Index Flood method borrows from stream gauges that are not located within karst landscapes, it overestimates peak runoff rates for the Rawdon Creek watershed.

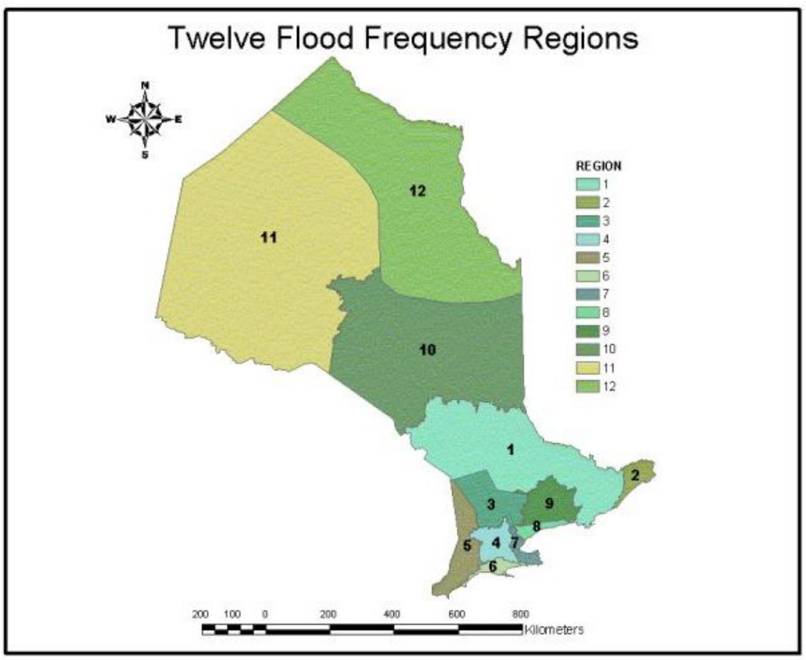


Figure 4-10: Index Flood Regions (Ministry of Natural Resources and Forestry, 2020)

The 2-yr flows are resolved directly from the equation using the constant and exponent from Table 4-14: Table of Constant (C) and Exponent (n) for use in the Modified Index Flood Equation. Other return period flows may be derived from the 2-yr flow by multiplying with the factors provided in Table 4-15. The region is based on the location of the catchment and selects the appropriate constants; Rawdon Creek is near the boundary of Regions 1 and 9. Therefore, both methods are included in the Rawdon Creek presentation of peak flows in Section 4.6 (see Table 4-20).

Equation 1: Index Flood Method

$$Q_2 = CA^n$$

Where:

Q_2 = 2-year return period (3 parameter Log Normal) flood
A = Drainage Area (km^2)
C = constant
n = exponent (slope of the line)

Table 4-14: Table of Constant (C) and Exponent (n) for use in the Modified Index Flood Equation

Region	Constant (C)	Exponent n
1(a)	0.22 (A < 60 km^2)	1.000
1 (b)	0.73 (A > 60 km^2)	0.707
2	0.51	0.896
3	0.20	0.957
4	0.71	0.842
5	0.45	0.775
6	0.41	0.806
7	1.13	0.696
8	0.73	0.785
9	0.40	0.810
10	0.28	0.849
11	0.38	0.706
12	0.59	0.765

Table 4-15: Ratio of Various Flood Frequencies to Q_2

Region	$Q_{1.25}/Q_2$	Q_2/Q_2	Q_5/Q_2	Q_{10}/Q_2	Q_{20}/Q_2	Q_{50}/Q_2	Q_{100}/Q_2	Q_{200}/Q_2	Q_{500}/Q_2
1	0.95	1.00	1.24	1.43	1.62	1.86	2.04	2.23	2.48
2	0.94	1.00	1.29	1.52	1.74	2.04	2.25	2.45	2.72
3	0.93	1.00	1.33	1.62	1.89	2.25	2.54	2.82	3.19
4	0.93	1.00	1.32	1.57	1.80	2.13	2.37	2.60	2.92
5	0.94	1.00	1.27	1.50	1.74	2.06	2.34	2.62	2.96
6	0.91	1.00	1.43	1.78	2.13	2.60	2.96	3.33	3.84
7	0.94	1.00	1.27	1.47	1.66	1.90	2.07	2.24	2.47
8	0.92	1.00	1.43	1.85	2.30	2.96	3.46	4.00	4.77
9	0.94	1.00	1.27	1.50	1.72	2.02	2.26	2.49	2.80
10	0.95	1.00	1.20	1.35	1.48	1.64	1.77	1.90	2.07
11	0.93	1.00	1.33	1.62	1.90	2.32	2.67	3.05	3.55

Region	$Q_{1.25}/Q_2$	Q_2/Q_2	Q_5/Q_2	Q_{10}/Q_2	Q_{20}/Q_2	Q_{50}/Q_2	Q_{100}/Q_2	Q_{200}/Q_2	Q_{500}/Q_2
12	0.94	1.00	1.22	1.38	1.52	1.68	1.80	1.90	2.05

Table 4-16: Limitation of Application of Index Flood Method Based on Drainage Area

Region	Minimum (km^2)	Maximum (km^2)
1	0.11	9270
2	76.1	3816
3	86.0	3960
4	2.5	5910
5	14.2	4300
6	5.2	697
7	63.5	293
8	4.9	800
9	24.3	1520
10	18.6	11900
11	0.7	24200
12	4250	94300

4.5 Climate Change

The technical requirements to address climate change were provided from the project partners in a technical memorandum titled *Incorporating Climate Change in Floodplain Mapping under the Flood Hazard Identification and Mapping Program.*

Rawdon Creek is located within Zone 3 of the *Flood Hazard Criteria Zones of Ontario and Conservation Authorities*. The Timmins event produces a significantly larger peak flow than the 100-yr storm. Therefore, the Timmins storm is the regulatory event.

Per the memorandum, the hourly rainfall that corresponds to the regulatory storm was adjusted using the mean annual temperature change obtained from the federal climate data portal for Stirling, ON. Jewell followed the Ontario MNRFs recommendation of obtaining the value for the 50^{th} percentile of the mean annual temperature change based on the CMIP5, RCP 4.5 scenario.

The year 2071 was selected since this is the furthest projected date in the Excel download from the federal climate data portal. The mean annual temperature change for the year 2071 is an increase of 3.3 degrees Celsius. An excerpt from the technical memo defining the equation used to convert historic rainfall intensity and temperature change to the future estimated rainfall intensity is provided below.

Equation 2: Estimation of Future Rainfall Intensity (Environment and Climate Change Canada)

$$R_P = R_C \times 1.07^{\Delta T}$$

Where:

R_P = Future estimated rainfall intensity (mm/h)
R_C = Historic estimated rainfall intensity (mm/h)
T = Temperature (°C)

The increase in temperature results in a significant (25%) increase in precipitation volume (see Table 4-17). The Timmins storm would increase in precipitation volume from 147mm to 183mm. As later shown in Section 4.6, this increases the Timmins peak flow rate within the Rawdon Creek watershed by 30%; from 90.2 m^3/s to 117.3 m^3/s.

In Section 4.6 the 100-yr return period is calculated to be 50.4 m^3/s. With a Timmins storm that is 79% larger than the 100-yr peak flow, and then increasing that value by 30%, the resulting peak flow is considerably larger and will have a significant impact on the floodplain mapping in existing vs. climate change adjusted conditions.

Table 4-17: Future Estimated Rainfall Intensities for Timmins (Regulatory) Storm

Time		Historic Intensity	Historic Adjusted (R_c)	Percent of 12 hour	Future Estimated Intensity (R_p)	% Increase in Intensity
Hour	Minute	mm/hr	mm/hr	mm	mm/hr	
1	60	15	11.4	8	14.3	25.0%
2	120	20	15.2	10	19.0	25.0%
3	180	10	7.6	6	9.5	25.0%
4	240	3	2.3	1	2.9	25.0%
5	300	5	3.8	3	4.8	25.0%
6	360	20	15.2	10	19.0	25.0%
7	420	43	32.7	23	40.9	25.0%
8	480	20	15.2	10	19.0	25.0%
9	540	23	17.5	12	21.9	25.0%
10	600	13	9.9	6	12.4	25.0%
11	660	13	9.9	7	12.4	25.0%
12	720	8	6.1	4	7.6	25.0%
Total		**193**	**147**	**100**	**183**	**25.0%**

It should be noted that climate change impacts on peak flows are inherently difficult to quantify due to the reality of Earth's extremely complex atmospheric and hydrologic systems. The climate change adjustment applied above relies on the relationship between temperature increase and rainfall depth. Therefore, the adjustment addresses a climate change scenario for a precipitation-driven flood event.

Based on calculations and an assessment of the data, Jewell expects that climate change would have a more noticeable impact on precipitation-driven runoff events rather than a snow-melt driven runoff event.

The stream flow gauge data predominantly defines the expected return period flows that would occur during a freeze-thaw/snowmelt condition. Recall that 90% of the annual instantaneous peak flows for Rawdon Creek have occurred between the months of January and April. These snow-melt events produce high peak flows due to a large volume of stored water content that is released when warmer temperatures occur.

With warmer seasonal temperatures generally expected due to climate change, it is reasonable to expect less stored water content during the winter months, since the period of below-freezing temperatures would be shortened with higher average temperatures. With less stored water content, it is possible that instantaneous peaks produced in a spring melt condition may not increase even with increased rainfall depths for single event conditions. This supports the likelihood that climate change will have a greater impact on heavy precipitation-driven rainfall events rather than the freeze-thaw/snowmelt driven event. Therefore, Jewell followed the guidance and information from the federate climate data portal.

4.6 Presentation of Peak Flows

The peak flows simulated in HMS for each storm event at their respective node of interest are summarized below. Recall that the node locations are illustrated in the catchment drawings in Appendix B.

Table 4-18: HMS Modelled Peak Flows at Each Hydrologic Node of Interest (Existing Conditions)

Hydrologic Node	50%	10%	2%	1%	0.5%	0.2%	Timmins	Timmins + Climate Change
A	4.4	7.3	10.4	12.0	13.5	15.9	21.3	29.6
B	7.1	11.9	17.1	19.6	22.2	26.0	34.6	47.4
C	4.0	6.8	9.7	11.2	12.6	14.9	20.5	24.3
D	16.1	28.0	41.2	47.4	53.8	63.3	85.1	110.8
E	17.6	30.1	43.9	50.4	57.2	67.1	90.2	117.3

Future full build-out conditions for S-R per Schedule A of the Hastings County Official Plan were considered (see Appendix A). The Village of Stirling is located at the downstream end of the overall Rawdon Creek watershed. In a full build-out scenario, the increase in hardened surfaces within the urban boundary will increase the peak flows from local developments. However, the peak runoff from these development areas would have a shorter time to peak relative to their existing condition. The result is a separation between the early peak from the urban areas and the larger peak from the majority of the remainder of the Rawdon Creek watershed, creating a slight decrease in peak flow in the regulatory storm event. Since the bulk of the Rawdon Creek watershed produces a larger peak flow than the urban core of S-R, it governs in the maximum peak flow in the system. The existing conditions are used for the floodplain mapping update since this condition would yield wider flood extents.

The selected peak flows for the Rawdon Creek floodplain mapping update are summarized in Table 4-19. Since the Timmins storm yields a greater peak flow than the 100-yr event, the Timmins storm is selected as the regulatory peak flow. A climate adjustment is then applied to the regulatory storm to produce the climate-adjusted peak flow rate.

The peak flow rates in the table below will be applied in the hydraulic model to identify the flood hazard limits. Peak rates were selected after review of several hydrologic modeling techniques. The Timmins event was obtained using the SCS CN method since its peak flows for historic events can only be calculated using rainfall-runoff software programs.

Table 4-19: Summary of Peak Flows at Village of Stirling for the Rawdon Creek Floodplain Mapping Update

Return Period	Peak Flow (m^3/s)
50	43.9
100	50.4
200	57.2
500	67.1
*Timmins	90.2
Timmins + CC	117.3

**Denotes regulatory storm event.*

Table 4-20 provides a presentation of peak flows for each of the hydrologic modeling methods applied for Rawdon Creek. This includes the following methodologies described previously:

- General Frequency Analysis
- SCS Curve Number
- Index Flood Analysis
- Climate Change Adjustments

At first glance, the results in the table below are atypical in that the various modeling methods show little consistency with one another. Given the limitations of each of the flow estimation methodologies discussed previously and in recognition of the unique characteristics specific to Rawdon Creek, we found the HEC-HMS model, completed for this floodplain mapping update, provides the best estimate of peak flows and will yield the most accurate floodplain mapping extents throughout the Village of Stirling.

It is noted that the first published annual instantaneous peak flow for the Water Survey of Canada Rawdon Creek stream flow gauge occurred in 1983 and this information would not have been available to the previous hydrologists that authored the 1975 and 1985 studies. Further, their work did not account for the extensive Karst in the watershed. This explains why the 1985 report prepared by Kilborn produces noticeably larger peak flows for all return period events when compared to those from the

HMS model. Therefore, the current floodplain mapping update provides excellent opportunity to utilize current information for more accurate peak flow estimations.

In summation, the peak flows developed from the HEC-HMS model were selected for use in the development of the floodplain mapping. The model was supplemented by the statistical flows at the stream gauge location.

Table 4-20: Summary of Peak Flows from Alternative Methods for Rawdon Creek at the Outlet (m^3/s)

Return Period	1985 Kilborn	GFA**	Index Flood		HEC-HMS***
			Region 1	Region 9	
50	87.6	25.2	52.2	52.9	43.9
100	95.6	27.7	57.2	59.1	50.4
200	-	30.4	62.7	65.2	57.2
500	-	34.0	69.7	73.4	67.1
***Timmins**	**292.0**	-	-	-	**90.2**
Timmins + Climate Change					117.3

** Denotes regulatory storm event.*
*** Transformed to outlet.*
**** Supplemented with Statistical Flows at Stream Gauge*

5 Hydraulics

The hydraulic analysis was prepared using HEC-RAS version 6.4.1. The hydrology results from the HEC-HMS model were applied in the HEC-RAS model to delineate the flood hazard limits for the Rawdon Creek floodplain mapping update. This section describes the bathymetry, cross-sections, storage impacts, bridge/culvert crossings, flood prone areas, model sensitivities, and a comparison of historical mapping to the current draft flood hazard limits.

The Rawdon Creek floodplain is characterized by a well-defined channel, a series of bridges, the James Street Dam, and urban overbank areas within the Village of Stirling.

Upstream of the James Street Dam, the overbanks are a mixture of predominantly medium to dense brush with some agricultural lands. Medium to dense brush also dominates the overbank areas for the portion of creek immediately downstream of the Stirling downtown core between Henry Street and Frankford Stirling Rd.

5.1 Bathymetry, Cross-Sections, and Geometry for 2-Dimensional Modeling

The LiDAR data described in Section 4.1 was supplemented by site-specific survey data from Jewell survey crew using GPS and a total station. The GPS was the main equipment used for the bathymetric survey. The GPS survey results were converted to CGVD 2013 datum and imported into the terrain layer as an overlay to the LiDAR data. The projection settings in the model are NAD 1983 UTM Zone 18.

The Jewell bathymetric survey comprised of 44 cross sections with their locations highlighted in Figure 5-1. Twelve (12) additional cross sections were surveyed for the bathymetry within the expanded study area that extends from the trail bridge near Ridge Road upstream to Goods Road.

Historically, 1-dimensional hydraulic models have been used for floodplain mapping. This type of model requires cross-section data to be set up by the user to represent the geometry data applied in the hydraulic model calculations. With recent advancements in the HEC-RAS modelling software that is developed and distributed freely by the U.S. Army Corps of Engineers, 2-dimensional modelling presents an alternative that can provide added benefits depending on the creek of interest.

A 2-dimensional model was selected for Rawdon Creek for the following reasons:

- To simulate the flow in the overbank areas that are located within the Village of Stirling, including several buildings located within the regulatory flood limit.
- To accommodate the storage area imposed by the James Street Dam.
- To achieve more realistic modeling results in local spill areas or low-lying areas.
- To take advantage of detailed terrain and survey data that provide opportunity to use HEC-RAS software to produce accurate output results for depth, velocity, and water surface elevations at any georeferenced location within the flood study area.

The terrain layer was used to develop a computational mesh that ultimately controls the movement of water through Rawdon Creek and the surrounding overbank areas. For each computation cell, an elevation-volume relationship is calculated to produce a single water surface elevation.

The Rawdon Creek model is comprised of 85,000 grid cells (not all are utilized), with smaller cells applied for the channel and specific areas of interest, such as road crossings or spill areas. The purpose of the customized mesh is to ensure accurate flow movement while utilizing a 5-second computational time step with output results set at 5-minute mapping intervals. The detailed 2D flow area established in the geometry editor provides the foundation for the dynamic mapping output. An example of the grid applied in the model is shown in Figure 5-2.

With the 2D modeling approach, cross sections are not needed to run the simulation. However, cross section water surface elevation (WSEL) plots for the 50-, 100-, 200-yr, Timmins and Timmins plus climate change events are shown for ten (10) cross sections within the study area per the map and cross section plots shown in Appendix J. These cross-section plots are useful for reviewing results to view WSELs as they relate to the channel cross sections for several storm events.

Figure 5-1: Locations of Surveyed Bathymetry Sections within Rawdon Creek Study Area

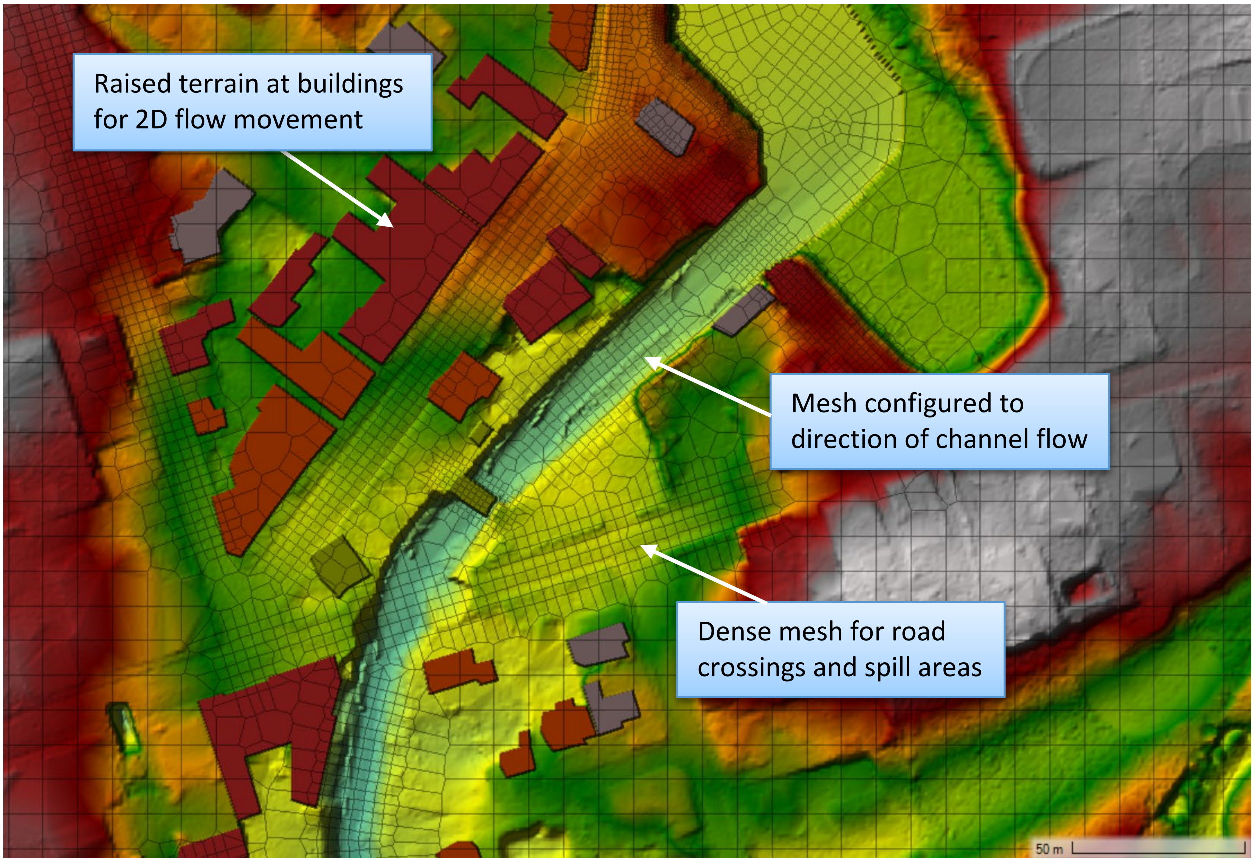


Figure 5-2: Example of Geometry Configuration for Model Setup

5.2 Internal and External Boundary Conditions

There are four (4) boundary conditions (BCs) for the 2D model (see Figure 5-3). Three of these are inflow BCs and the other is an outflow BC.

The 2D unsteady flow model received its flow data from an inflow hydrograph where the incoming flows change with time. The inflow hydrograph was obtained by the tabular output in the Visual OTTHYMO model; each inflow BC corresponds to an inflow hydrograph. The table below summaries the inflow peaks and their corresponding receiving catchments as shown in Appendix B. Inflow BC 1 represents the inflow hydrograph produced by Catchments 201, 301, 401, and 402. Inflow BC 2 represents the inflow hydrograph from Catchments 101, 501, and 502. Inflow BC 3 represents the inflow hydrograph from Catchment 100. Inflow BC 3 is conservatively located upstream of the James Street dam since Catchment 100 includes all of the Stirling downtown area.

The outflow boundary condition is established by normal depth since Rawdon Creek is governed by channel flow downstream of the study area. Consideration was given to potential impacts from the Trent River. Water level data for the Trent River was obtained from Parks Canada, however the nearest station in their records was too far upstream for application at the Rawdon Creek outlet. Therefore, a review of the detailed LiDAR was completed to identify an approximate 3-4m difference in normal water level between the Trent River and the downstream limit of the study area. Therefore, the Trent River would not influence the Rawdon Creek flow behaviour in the regulatory event.

Table 5-1: Inflow Boundary Condition Peak Flows

Inflow BC	Receiving Catchments	Peak Flow (m^3/s)				
		50-Yr	100-Yr	200-Yr	Timmins	Timmins + CC
1	201-301-401-402	23.0	26.4	30.0	47.2	64.1
2	101-501-502	18.3	21.1	24.0	37.9	46.8
3	100	3.1	3.5	3.8	6.0	7.5

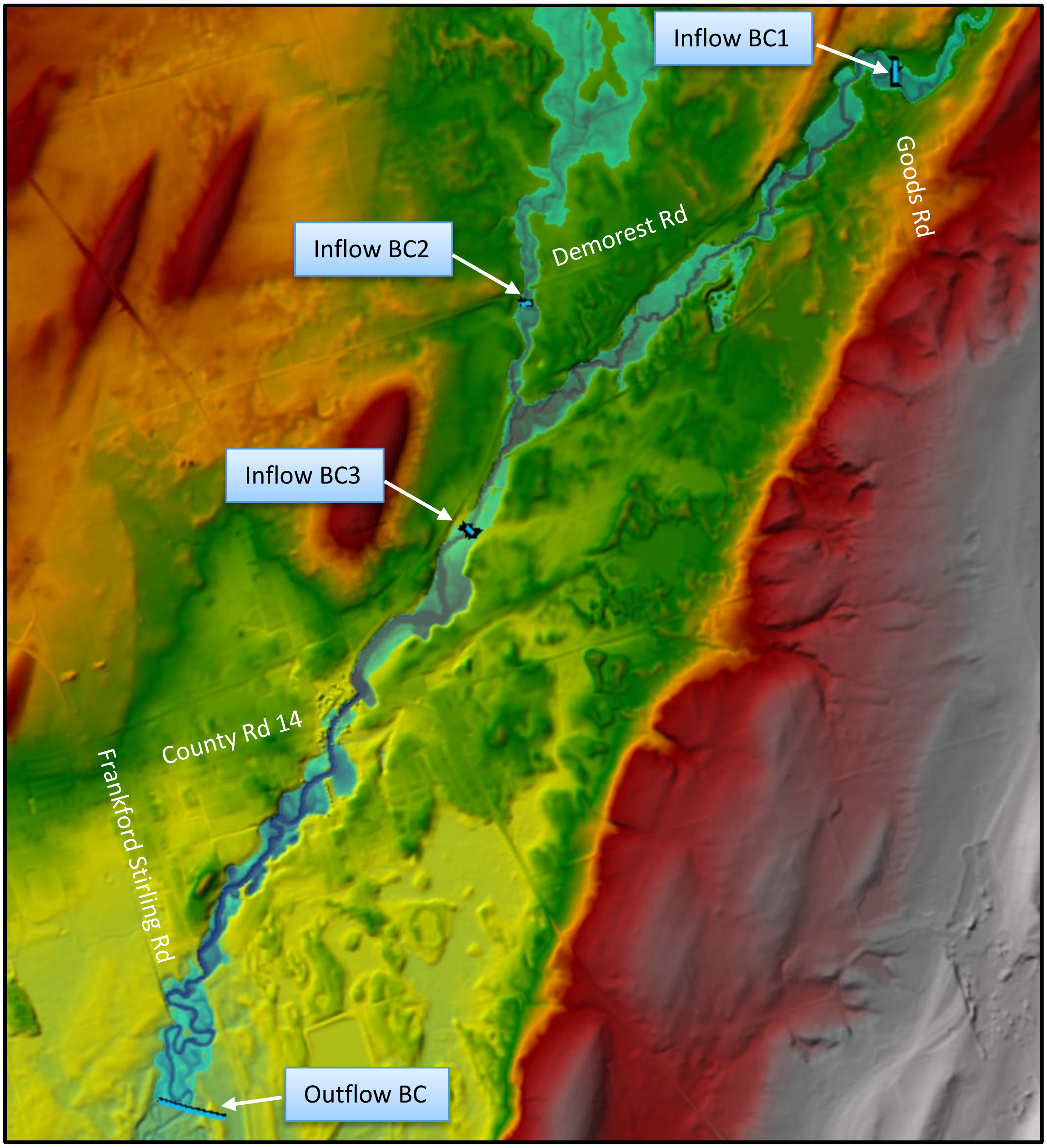


Figure 5-3: Locations of Inflow and Outflow Boundary Conditions

5.3 Storage Impacts

Due to the size of the bridge openings and availability of spill routes when constrictions do occur, local storage areas provide no appreciable flow attenuation within the Rawdon Creek study area.

In a large storm event such as the regulatory storm, there is temporary storage on the upstream side of the dam although no meaningful flow attenuation is provided due to the hydraulic efficiency of the weir as discussed further in Section 5.4.5.

The other noticeable temporary storage area is on the upstream side of Henry Street in the vicinity of the local ball diamond (see Figure 5-4). However, this storage area also provides no appreciable flow reduction to downstream lands.

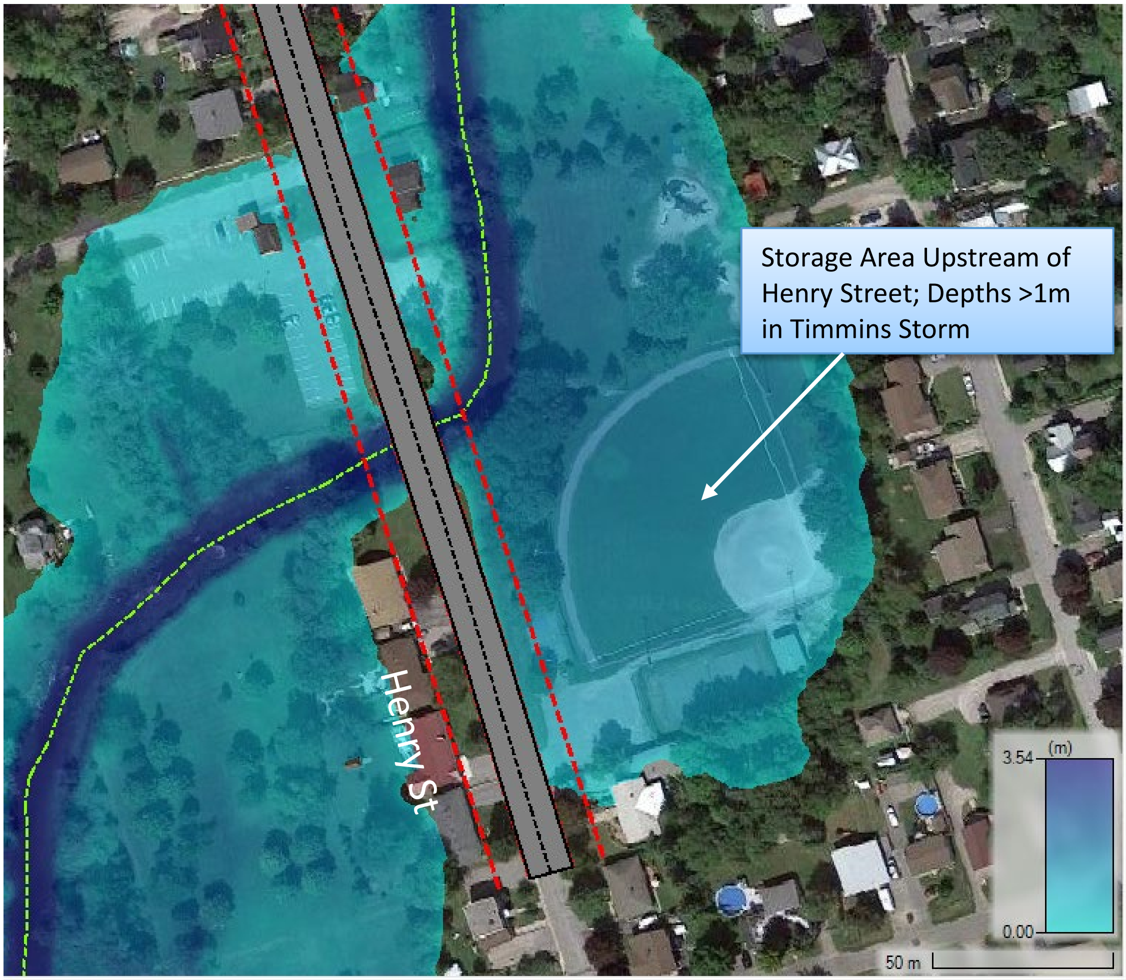


Figure 5-4: Storage Area Upstream of Henry Street (Timmins Event)

5.4 Bridge Crossings

The hydraulic model simulates the effects of the bridges on the water surface elevations at each crossing. Each of the nine (9) Rawdon Creek crossings within the study area are bridges or a weir (i.e. no culverts). This section summarizes the existing crossing configurations, stage-discharge curves, and the maximum water surface elevations at each road crossing. The purpose of the section is to address the impacts of the existing infrastructure on the overall floodplain delineation discussed in Section 7.

5.4.1 Goods Road

Goods Road is the first road crossing within the Rawdon Creek study area and consists of a 9.2m span bridge (see Figure 5-5). It is part of the supplemental scope of work that includes the stretch of Rawdon Creek between Goods Road and the trail bridge near Ridge Road. The bridge at Goods Road is upstream of the confluence that receives the Rawdon Creek tributary to the northwest (Inflow BC2) and subsequently has a lesser flow than the flows within the main study area.

A summary of the Goods Road bridge is provided in Table 5-2. The stage and discharge hydrographs for this crossing are provided in Appendix I. The chart in Appendix I shows that the difference between headwater (HW) and tailwater (TW) elevations is relatively minor. This suggests that the bridge is efficient in conveying its regulatory peak flow of 47.2 m^3/s. As a result, there is no overtopping of this bridge in the Timmins event and safe access is expected to be available in the regulatory storm.

Figure 5-5: Elevation View of Goods Road Bridge

Table 5-2: Goods Road Crossing Summary

Road Name:		Goods Road	
Coordinates:		44.322604, -77.521115	
Span (m) =	9.2	[1]Soffit (m) =	124.7
[2]Upstream Invert (m)		Downstream Invert (m)	
122.54		122.50	
	Low Point of Road =	125.41	m
	[3]Timmins WSEL =	125.39	m
Maximum Relief Flow Depth (m)		Recommended Limit = 0.3m	
0		✓	
Depth*Velocity Calculated (m^2/s)		Recommended Limit = 0.8 (m^3/s)	
0		✓	

[1]Soffit measured as highest point of bridge opening.

[2]Invert taken as creek inverts at upstream and downstream of bridge opening.

[3]Timmins WSEL measured at immediate upstream side of bridge.

Figures 5-6 and 5-7 illustrate the extents of the Timmins floodplain with satellite and terrain background imagery. Evidently, the flow contracts and expands effectively through the bridge opening in the Timmins event.

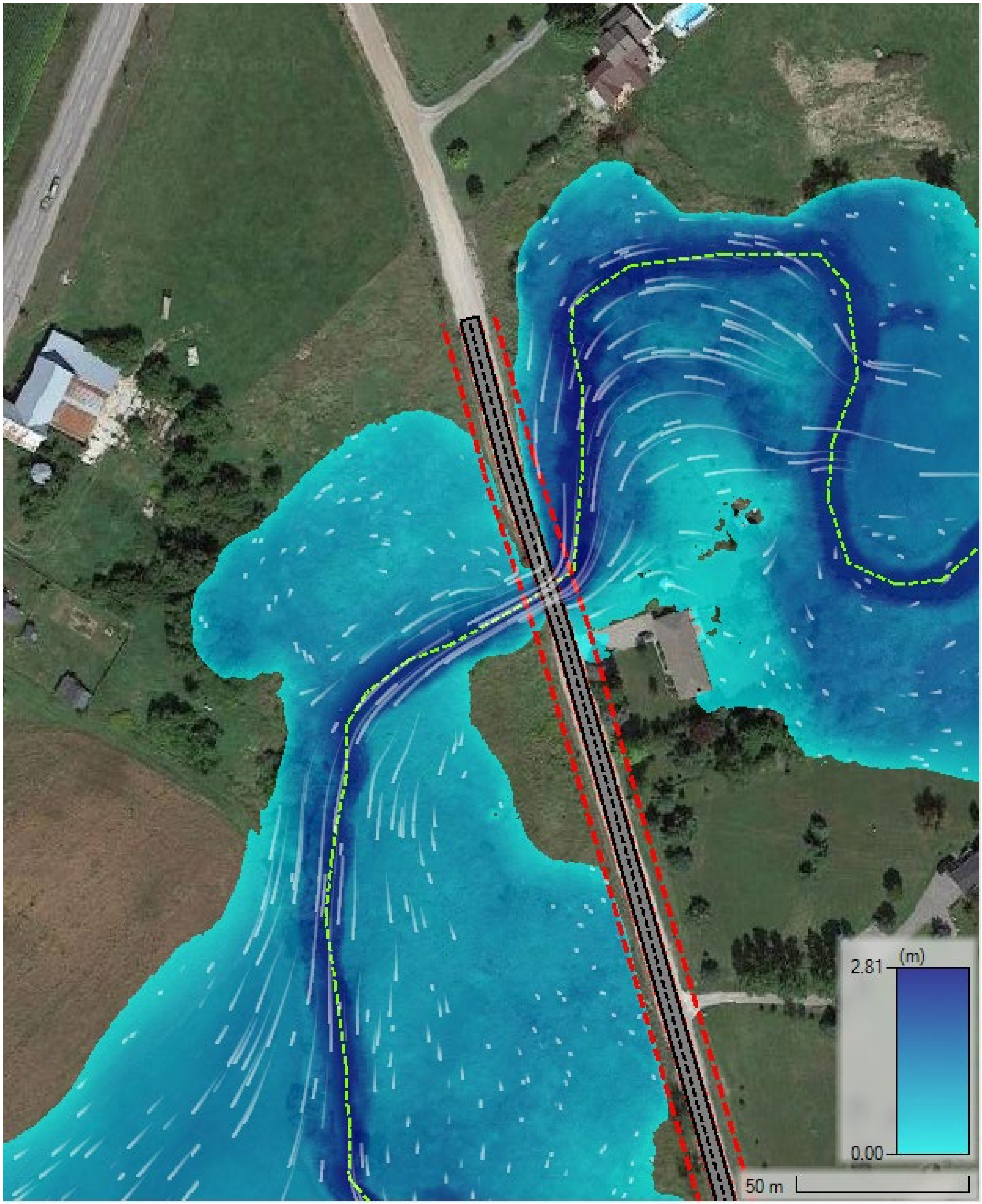


Figure 5-6: Schematic of Timmins Floodplain at Goods Rd – Satellite Background

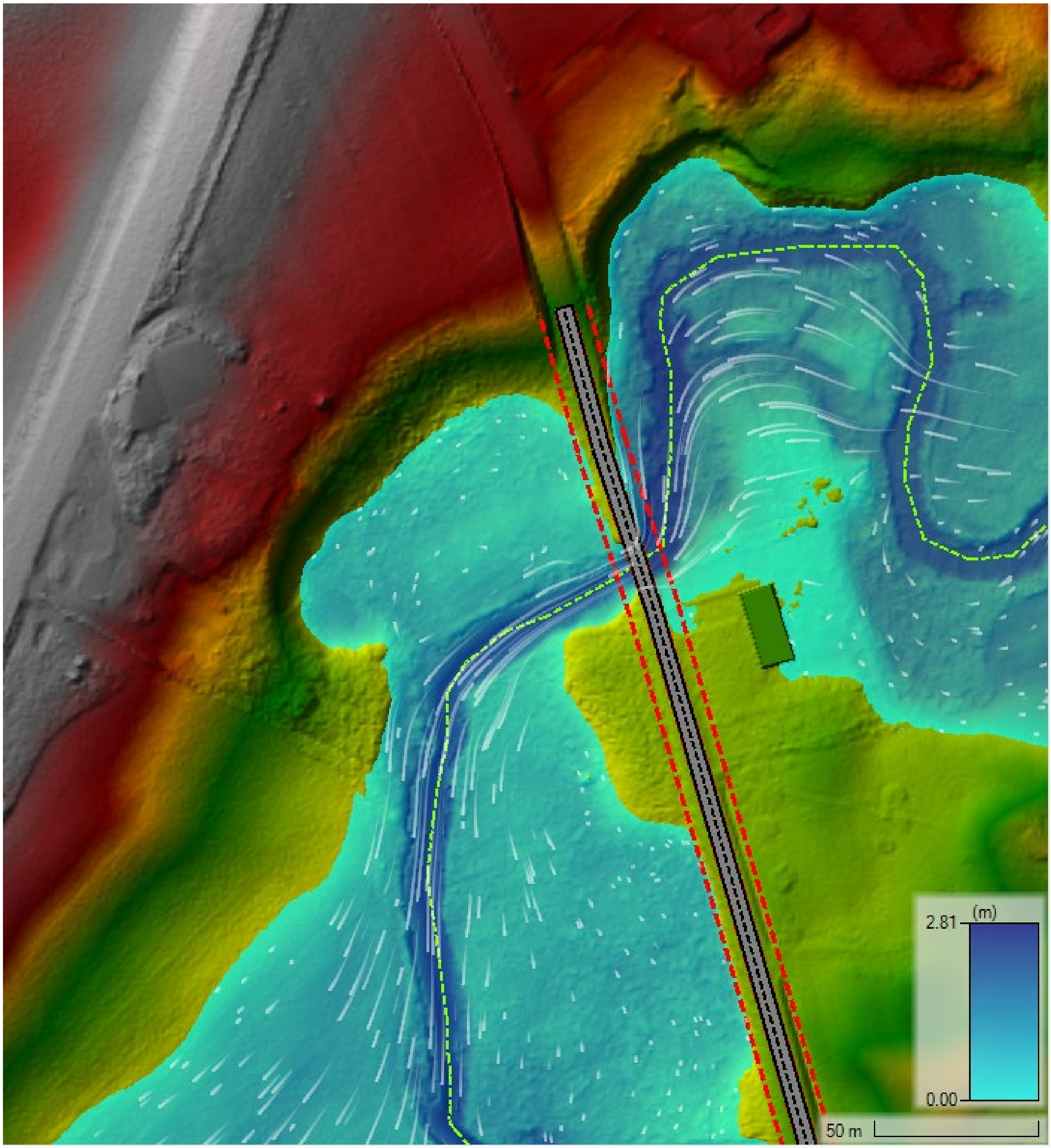


Figure 5-7: Schematic of Timmins Floodplain at Goods Rd – Terrain Background

5.4.2 Evergreen Road

Evergreen Road is the next crossing downstream of Goods Rd and consists of a 13.8m span bridge (see Figure 5-8). It is also part of the supplemental scope of work that includes the stretch of Rawdon Creek between Goods Road and the trail bridge near Ridge Road. The bridge at Evergreen Road is upstream of the confluence that receives the Rawdon Creek tributary to the northwest (Inflow BC2) and subsequently has a lesser flow than the flows within the main study area.

A summary of the bridge crossing at Evergreen Road is provided in Table 5-3. The stage and discharge hydrographs for this crossing are provided in Appendix I. The chart in Appendix I shows a minimal difference in headwater (HW) and tailwater (TW) elevations. Similar to Goods Road, this suggests that the bridge efficiently conveys its regulatory peak flow. As a result, there is no overtopping of this bridge in the Timmins event and safe access is expected to be available in the regulatory storm.

Figure 5-8: Image of Evergreen Road Bridge Opening

Table 5-3: Evergreen Road Crossing Summary

Road Name:		Evergreen Rd	
Coordinates:		44.316368, -77.524742	
Span (m) =	13.8	[1]Soffit (m) =	124.18
[2]Upstream Invert (m)		Downstream Invert (m)	
121.48		121.3	
Low Point of Road =		125.17	m
[3]Timmins WSEL =		124.57	m
Maximum Relief Flow Depth (m)		Recommended Limit = 0.3m	
0		✓	
Depth*Velocity Calculated (m^2/s)		Recommended Limit = 0.8 (m^3/s)	
0		✓	

[1]Soffit measured as highest point of bridge opening.

[2]Invert taken as creek inverts at upstream and downstream of bridge opening.

[3]Timmins WSEL measured at immediate upstream side of bridge.

Figures 5-9 and 5-10 illustrate the extents of the Timmins floodplain with satellite and terrain background imagery. Evidently, the flow contracts and expands effectively through the bridge opening.

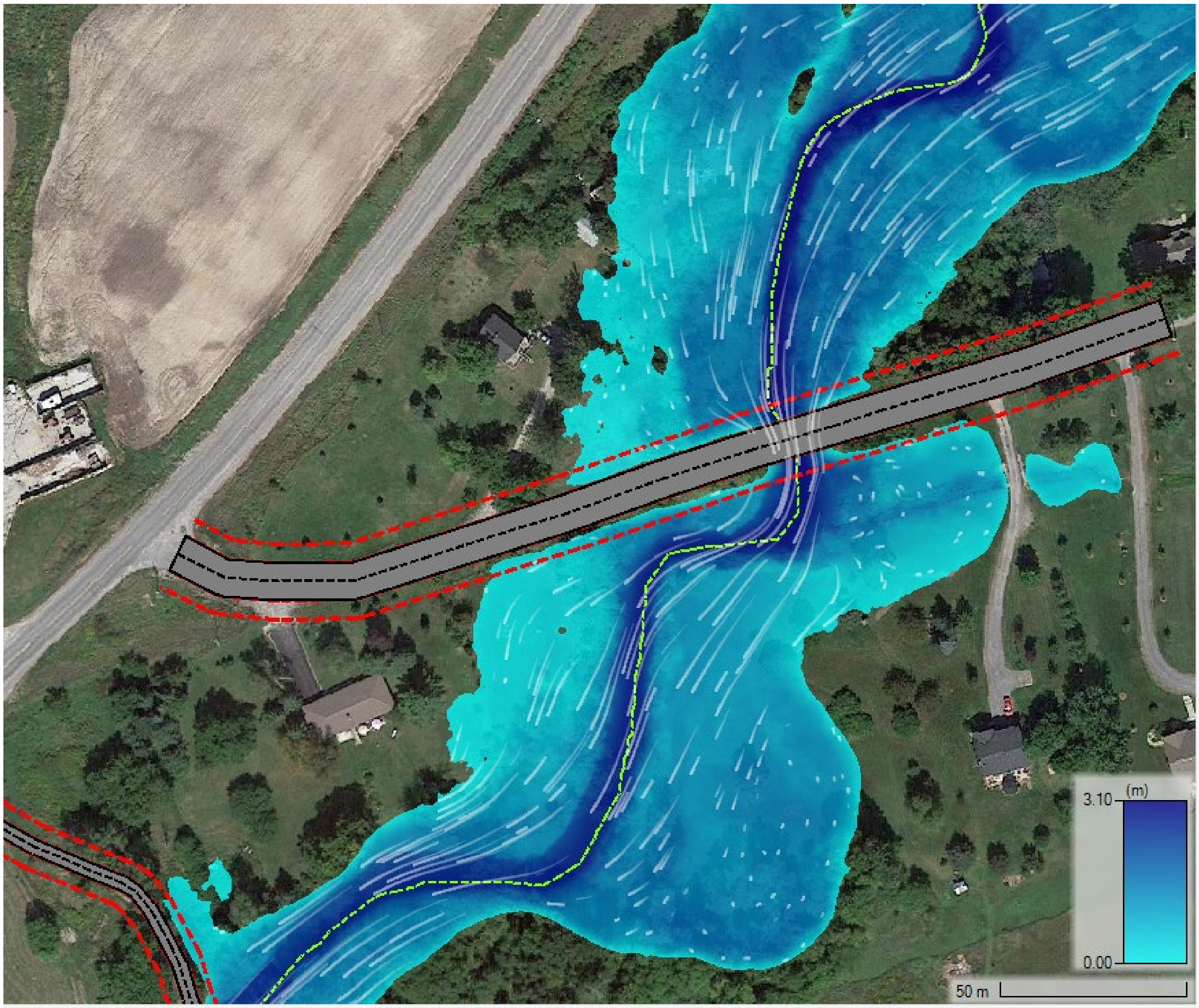


Figure 5-9: Schematic of Timmins Floodplain at Evergreen Rd – Satellite Background

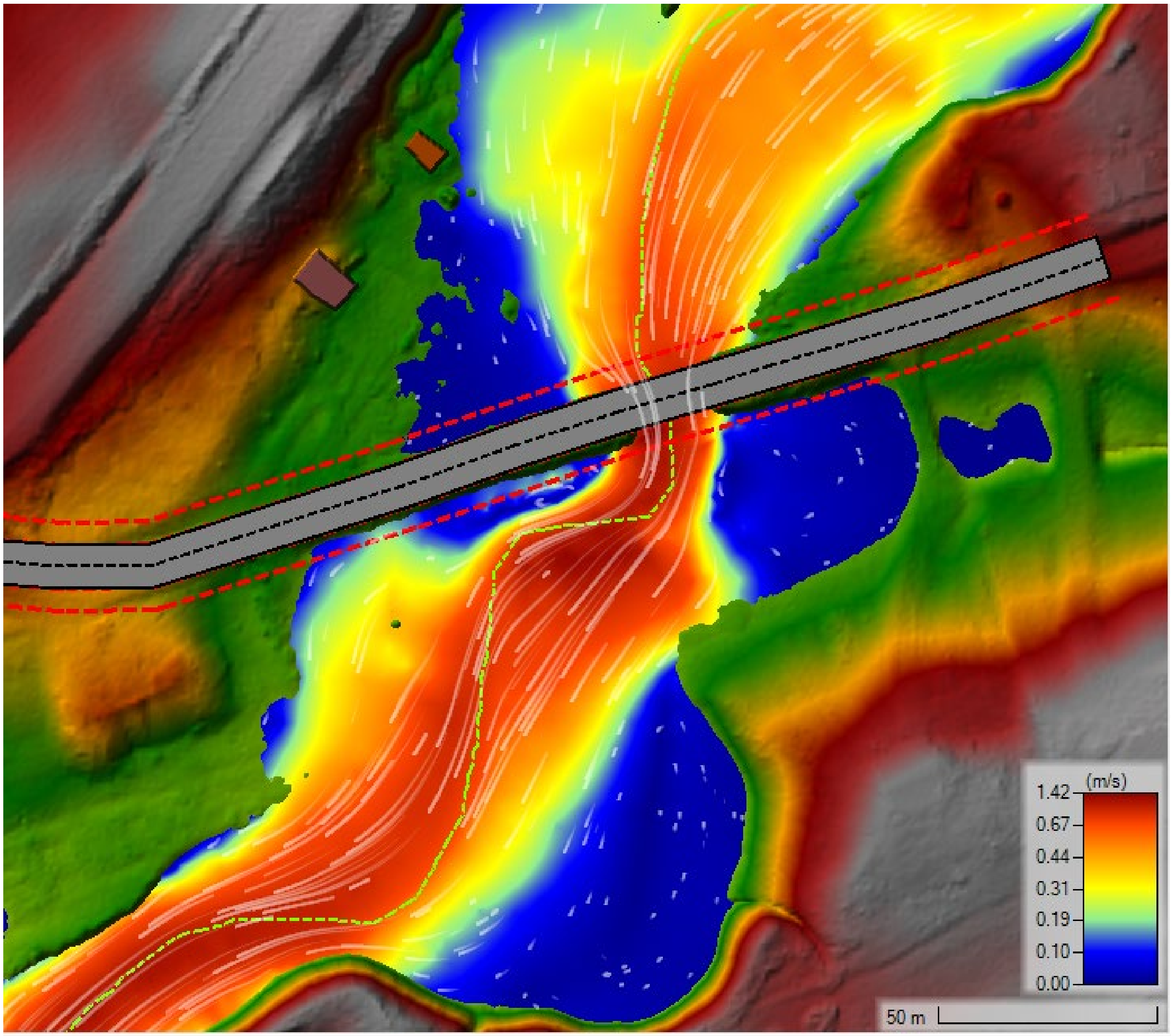


Figure 5-10: Schematic of Timmins Floodplain at Evergreen Rd – Terrain Background

5.4.3 Aggregate Site Driveway Crossing

Shortly downstream (~230m) of Evergreen Rd is a driveway crossing to what is believed to be an aggregate site. This crossing is within the expanded study area and upstream of the confluence with Rawdon Creek's northwest tributary.

A summary of the bridge crossing at Evergreen Road is provided in Table 5-4. The stage and discharge hydrographs for this crossing are provided in Appendix I.

Table 5-4 shows that the maximum relief flow depth over the road is greater than the generally preferred 0.3m depth limit for safe access per the *2008 MTO Highway Drainage Design Standards*. This crossing is unique in that it has two Timmins water surface elevations depending on where the measurement is taken along the driveway. This is due to the spill that occurs south of the crossing due to the backwater that occurs from the bridge opening. The bridge does not have a rectangular opening as its opening replicates sloping abutment characteristics as shown in Figure 5-12. As a result, it is not as effective as the upstream crossings at Goods Road and Evergreen Road.

Figures 5-13 and 5-14 illustrate the location of the spill towards the south portion of the driveway. It is uncommon to have a spill within a crossing, however, this is the case due to the excessively long driveway (~250m total) and the local terrain characteristics. As the backwater occurs upstream of the bridge crossing, a spill of 11.4 m^3/s is conveyed towards the low-lying area south of the bridge.

The water from this 11.4 m^3/s spill ponds in the low-lying area with relatively low velocities and spills over the south portion of the driveway with a maximum depth of 0.76m. Therefore, safe access for standard motor vehicles is not available at this driveway in the regulatory storm event.

Figure 5-11: Image of Crossing to Aggregate Site

Figure 5-12: Image of Sloping Ground at Bridge Opening Simulating a Sloping Abutment

Table 5-4: Aggregate Site Entrance Crossing Summary

Road Name:		Aggregate Site Entrance	
Coordinates:		44.314894, -77.526694	
Span (m) =	12.2	[1]Soffit (m) =	123.93
[2]Upstream Invert (m)		Downstream Invert (m)	
121.93		121.77	
Low Point of Road =		124.46 / 122.93 m	
[3]Timmins WSEL =		124.12 / 123.69 m	
Maximum Relief Flow Depth (m)		Recommended Limit = 0.3m	
0.76		x	
Depth*Velocity Calculated (m^2/s)		Recommended Limit = 0.8 (m^3/s)	
0.46		✓	

[1]Soffit measured as highest point of bridge opening.

[2]Invert taken as creek inverts at upstream and downstream of bridge opening.

[3]Timmins WSEL measured at immediate upstream side of bridge. Two elevations shown due to change in elevation due to south spill.

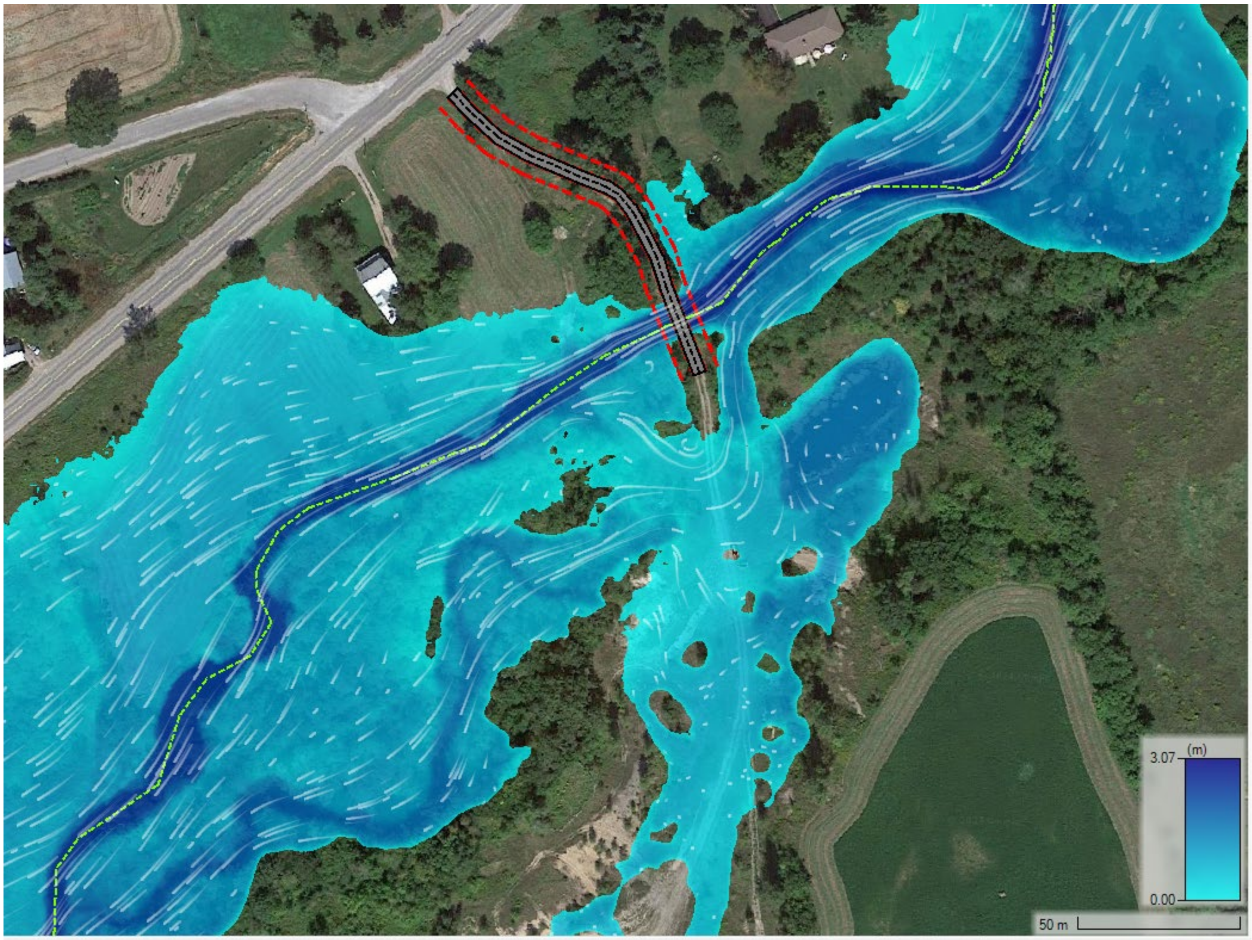


Figure 5-13: Schematic of Timmins Floodplain at Aggregate Crossing Entrance – Satellite Background

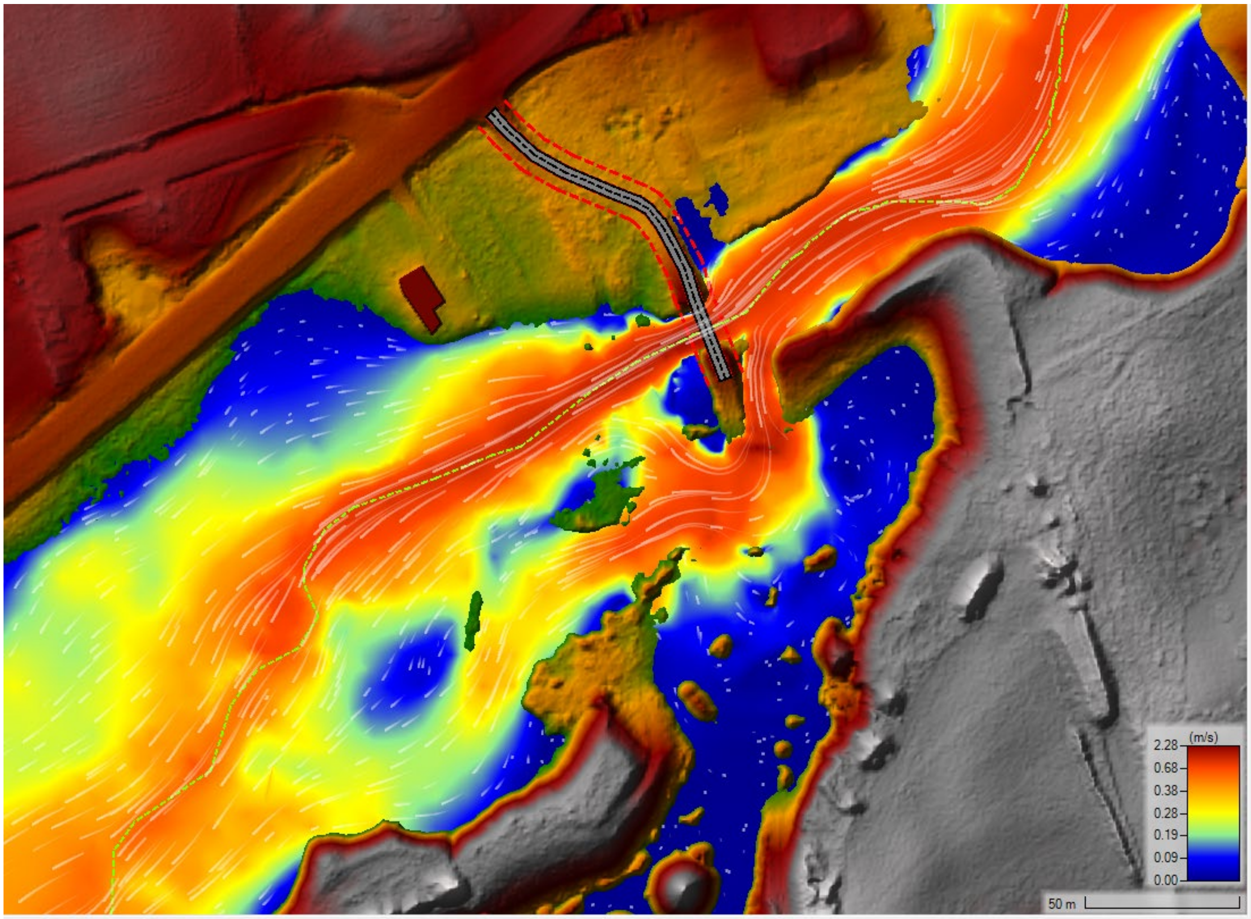


Figure 5-14: Schematic of Timmins Floodplain at Aggregate Crossing Entrance – Terrain Background

5.4.4 Trail Off of Ridge Road

The Trail Bridge near Ridge Road is immediately downstream of the confluence between Rawdon Creek and its northwest tributary. With the inclusion of the northwest branch, the Timmins flows increases substantially to 84.5 m^3/s.

A summary of the bridge crossing at the trail bridge is provided in Table 5-5. The stage and discharge hydrographs for this crossing are provided in Appendix I.

Figure 5-15: Elevation View of Trail Bridge Off of Ridge Road

Figures 5-16 and 5-17 illustrate the extents of the Timmins floodplain with satellite and terrain background imagery. Evidently, the flow contracts and expands effectively through the bridge opening with no anticipated damages to the nearest dwellings in the Timmins event.

Table 5-5: Trail Bridge Crossing Summary

Road Name:		Trail Bridge near Ridge Rd	
Coordinates:		44.306484, -77.538652	
Span (m) =	14.7	[1]Soffit (m) =	122.09
[2]Upstream Invert (m)		Downstream Invert (m)	
119.60		119.80	
	Low Point of Road =	123.55	m
	[3]Timmins WSEL =	122.89	m
Maximum Relief Flow Depth (m)		Recommended Limit = 0.3m	
0		✓	
Depth*Velocity Calculated (m^2/s)		Recommended Limit = 0.8 (m^3/s)	
0		✓	

[1]Soffit measured as highest point of bridge opening.

[2]Invert taken as creek inverts at upstream and downstream of bridge opening.

[3]Timmins WSEL measured at immediate upstream side of bridge.

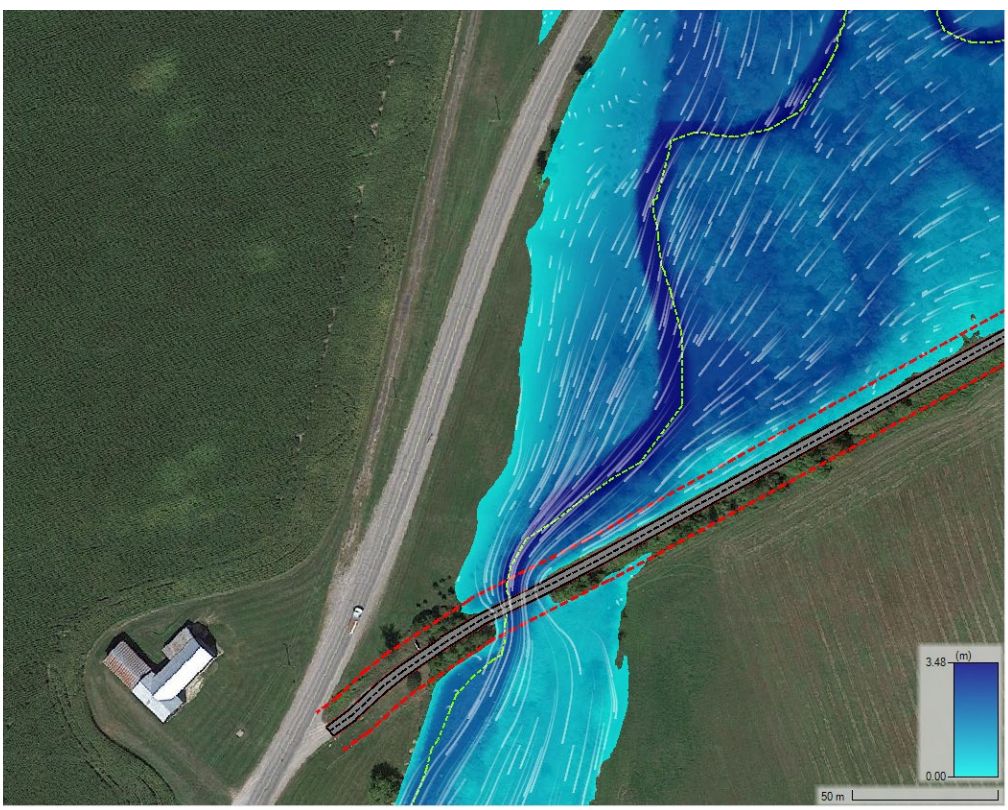


Figure 5-16: Schematic of Timmins Floodplain at Trail Bridge near Ridge Rd – Satellite Background

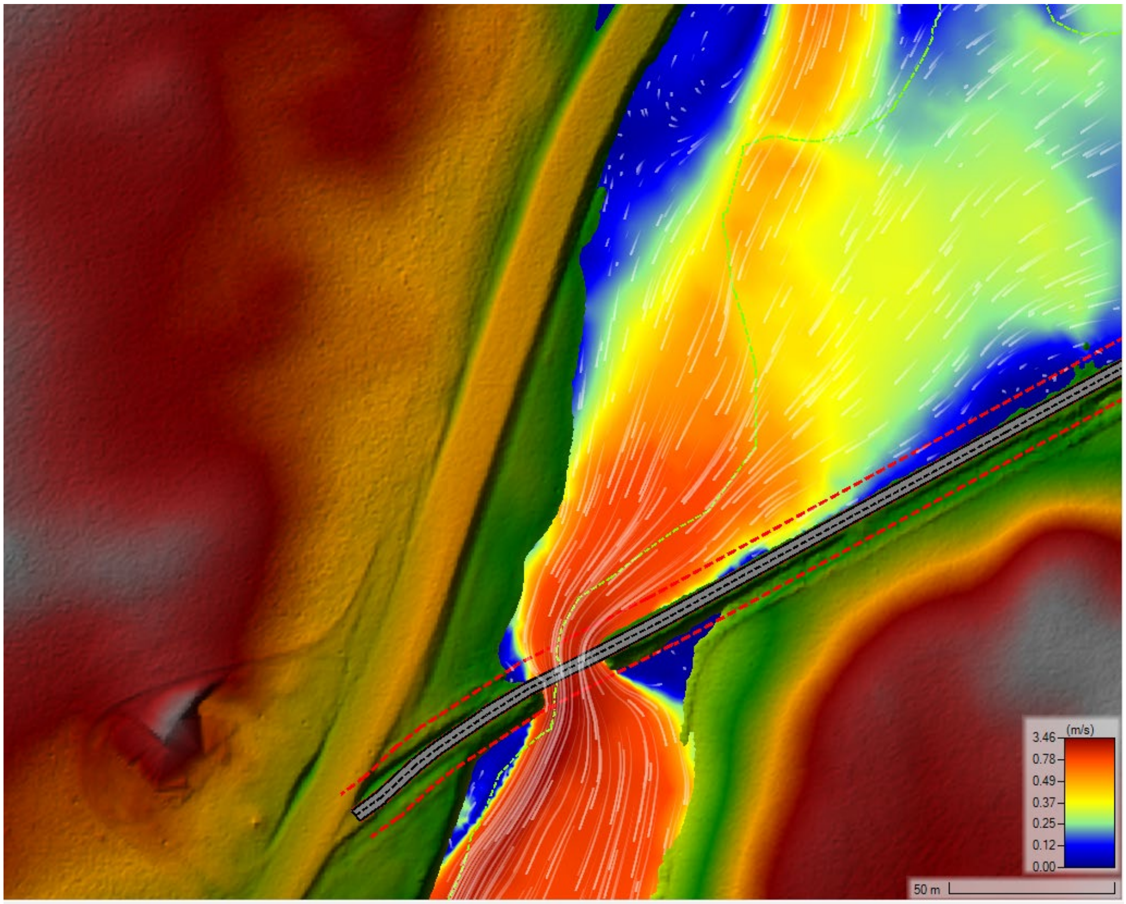


Figure 5-17: Schematic of Timmins Floodplain at Trail Bridge near Ridge Rd – Terrain Background

5.4.5 James Street Dam

The James Street Dam is the first crossing within the urban core of Stirling. The dam is comprised of an 18.5m long weir with a large opening height of 1.85m (see Figure 5-18). The dam is simulated as a gate opening with a constant height and a closed top to allow the weir and road overtopping to both be included in the crossing configuration.

A summary of James Street Dam is provided in Table 5-6. The stage and discharge hydrographs for this crossing are provided in Appendix I. A review of the stage-discharge hydrographs indicates that the entire regulatory flow of 89.1 m^3/s flows through the large weir. With the weir invert at 117.73m, the depth of flow over the weir is substantial at 1.90m and the water level is equal to the low point of road at James Street.

Note that no dead storage (i.e. no storage below the weir invert) was included in the model simulations.

Figure 5-18: Elevation View of James Street Dam

Table 5-6: James St Dam Summary

Road Name:		James St Dam	
Coordinates:		44.297241, -77.545699	
Weir length (m)	18.5	[1]Weir height (m)	1.85
Weir Invert (m)		[2]Downstream Invert (m)	
117.73		114.9	
Low Point of Road =		119.63	m
[3]Timmins WSEL =		119.63	m
Maximum Relief Flow Depth (m)		Recommended Limit = 0.3m	
0		✓	
Depth*Velocity Calculated (m^2/s)		Recommended Limit = 0.8 (m^3/s)	
0		✓	

[1]Weir height is maximum height of opening from weir invert to closed top.

[2]Downstream invert measured at creek at bottom of spillway.

[3]Timmins WSEL measured at immediate upstream side of bridge.

Figures 5-19 and 5-20 illustrate the extents of the Timmins floodplain upstream of the dam with satellite and terrain background imagery. The mill pond upstream of the dam offers some storage, however, the hydraulically efficient weir and high ground in the overbank areas upstream of the dam limit the floodplain extents on upstream side of this hydraulic structure. As a result, no buildings are within the regulatory flood limit on the immediate upstream side of the James Street dam in the Timmins event.

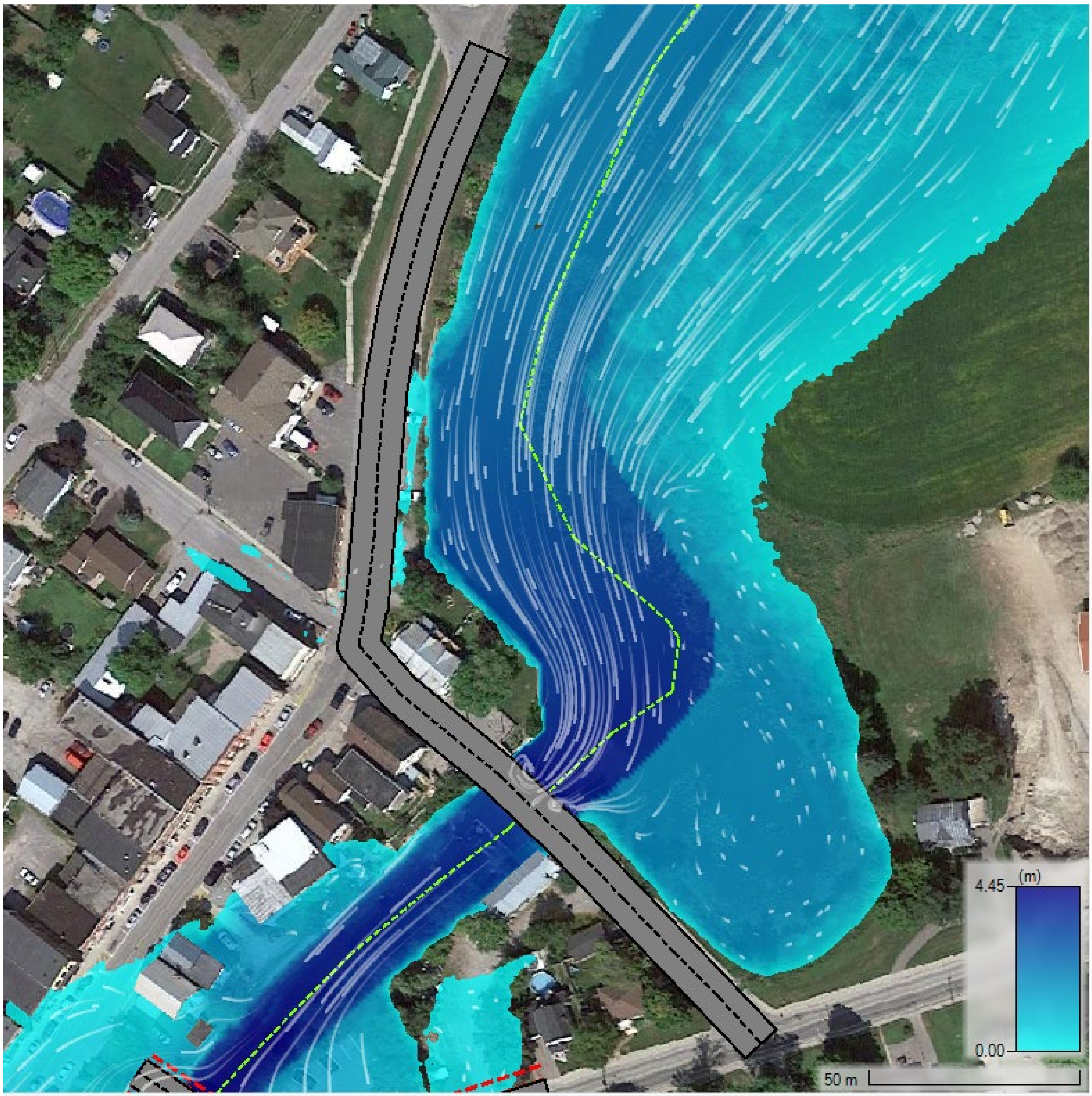


Figure 5-19: Schematic of Timmins Floodplain at James St Dam – Satellite Background

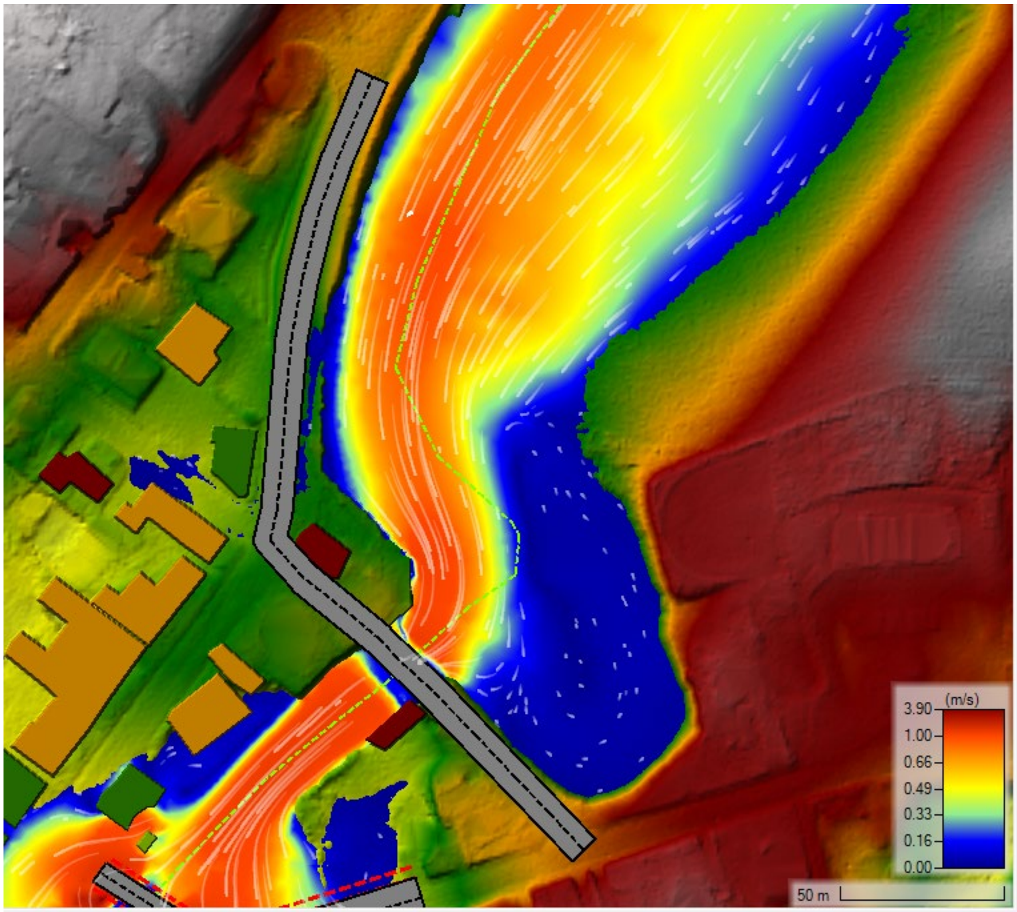


Figure 5-20: Schematic of Timmins Floodplain at James St Dam – Terrain Background

5.4.6 Pedestrian Bridge

The pedestrian bridge is located 106m downstream of the James Street dam and only 30m upstream of East Front Street. The pedestrian bridge has a span of over 10m (see Figure 5-21); however, the overbank areas are immediately within existing developed properties within the Stirling urban core. As a result, several buildings are susceptible to the flood hazard in the regulatory storm event.

Table 5-7 summarizes the crossing information for the pedestrian bridge. The relief flow depth and depth-velocity product are included although of less importance considering the bridge is not intended for vehicular traffic. The flow and stage hydrographs in Appendix I show that the flows through or over the pedestrian bridge only account for 29.3 m^3/s, or 33% of the Timmins flow. The remaining 59.8 m^3/s spills overland on either side of the pedestrian bridge.

The spill on either side of the pedestrian bridge is evident in Figures 5-22 and 5-23 that show the inundation area for both the pedestrian bridge and the crossing at East Front Street due to their proximity to one another. The spill on the north side of the pedestrian bridge contributes to a relief flow depth of 0.35m on the west side of the East Front Street bridge. The spill on the south side of the pedestrian bridge contributes to a significant flow depth over the road of 1.07m on the east side of the East Front Street bridge.

The East Front Street bridge is summarized in the following subsection.

Figure 5-21: Image of Bridge Opening for Pedestrian Crossing Between James St and E Front St.

Table 5-7: Pedestrian Bridge Crossing Summary

Road Name:		Pedestrian Bridge	
Coordinates:		44.296654, -77.546573	
Span (m) =	10.7	[1]Soffit (m) =	116.95
[2]Upstream Invert (m)		Downstream Invert (m)	
114.31		114.33	
Low Point of Road =		117.42	m
[3]Timmins WSEL =		118.49	m
Maximum Relief Flow Depth (m)		Recommended Limit = 0.3m	
1.07		x	
Depth*Velocity Calculated (m^2/s)		Recommended Limit = 0.8 (m^3/s)	
1.96		x	

[1]Soffit measured as highest point of bridge opening.

[2]Invert taken as creek inverts at upstream and downstream of bridge opening.

[3]Timmins WSEL measured at immediate upstream side of bridge.

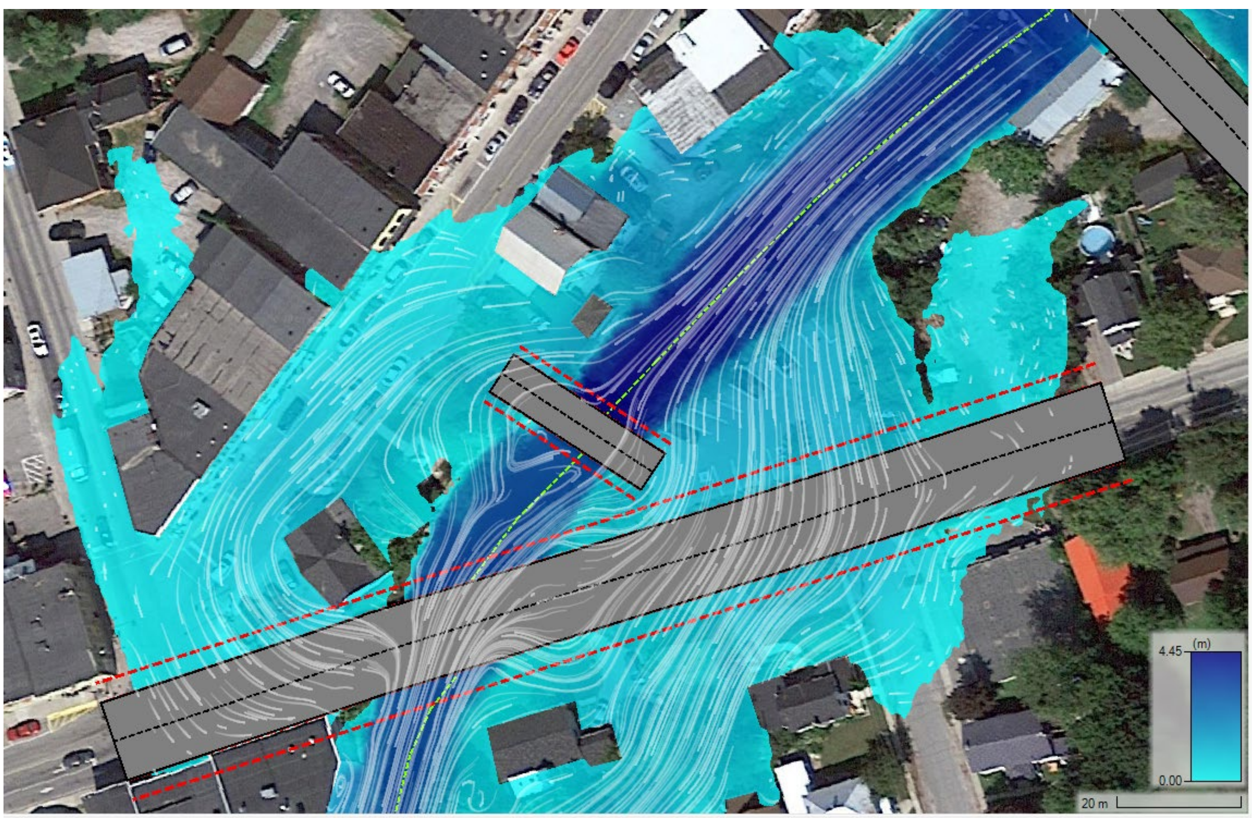


Figure 5-22: Schematic of Timmins Floodplain at Pedestrian Bridge and E Front Street Bridge – Satellite Background

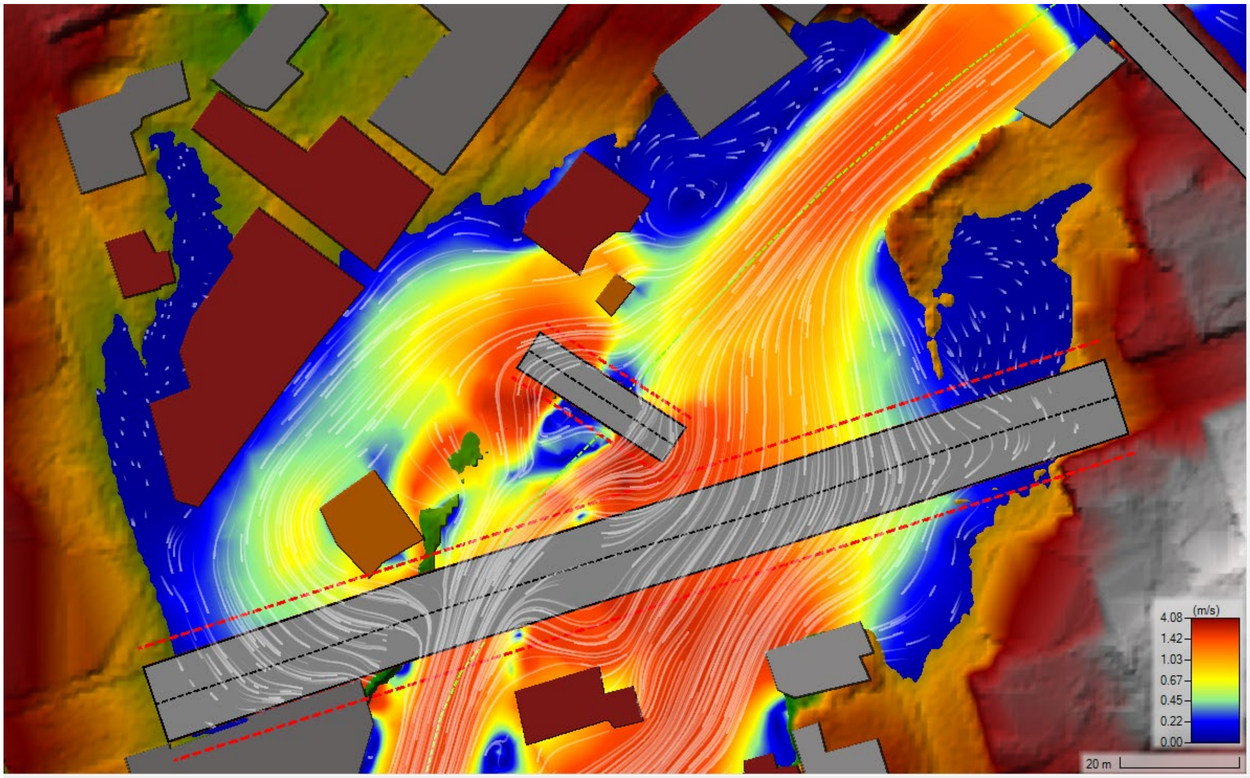


Figure 5-23: Schematic of Timmins Floodplain at Pedestrian Bridge and E Front Street Bridge – Terrain Background

5.4.7 East Front Street

The East Front Street bridge has a wide span of 17.1m (see Figure 5-24) that provides effective flow conveyance capacities. However, the spills from the pedestrian bridge a short distance upstream create spill that by-pass the bridge opening and spill directly over the roadway. The result is the high relief flow depths and depth-velocity products at this crossing that do not meet recommended criteria for safe access in the regulatory event.

At a first glance, Table 5-8 implies the bridge is undersized, however it is likely that the removal (or improvements) of the pedestrian bridge would be a more effective mitigation measure (and more cost-effective) relative to an upsizing of the E Front St bridge. We are not recommending this is a mitigation measure as the model results are currently unknown and it is presumed this is a popular walking path for locals and tourists alike, but we note it as a potential mitigation opportunity. With numerous buildings within the floodplain between the James St dam and the E Front St bridge in the Timmins event (see Section 7.3), this may be a worthwhile investigation.

Figure 5-24: Elevation View of Bridge at E Front St

Table 5-8: E Front St Crossing Summary

Road Name:		E Front St	
Coordinates:		44.296283, -77.546970	
Span (m) =	17.1	[1]Soffit (m) =	116.76
[2]Upstream Invert (m)		Downstream Invert (m)	
114.09		113.93	
Low Point of Road =		117.47	m
[3]Timmins WSEL =		118.63	m
Maximum Relief Flow Depth (m)		Recommended Limit = 0.3m	
1.16		x	
Depth*Velocity Calculated (m^2/s)		Recommended Limit = 0.8 (m^3/s)	
1.38		x	

[1]Soffit measured as highest point of bridge opening.

[2]Invert taken as creek inverts at upstream and downstream of bridge opening.

[3]Timmins WSEL measured at spill on east side of E Front St.

5.4.8 Henry Street

The Henry Street bridge has a 16.6m span (see Figure 5-25) and is located 220m downstream of E Front Street.

Table 5-9 provides a crossing summary and Appendix I summarizes its flow and stage hydrographs. The relief flow depth of 0.83m is due to a road sag elevation on the north side of the bridge that is very low; lower than the highest point of the bridge soffit by 1.15m.

Figures 5-26 and 5-27 illustrate the particle tracing for this crossing. The low-lying area on the north side of the bridge is evident and it results in several dwellings being susceptible to the regulatory flood hazard including the existing pumping station. A separate image is provided in Section 7.3 to illustrate the location of the dwellings within the floodplain.

Figure 5-25: Elevation View of Henry St Bridge

Table 5-9: Henry St Crossing Summary

Road Name:		Henry St	
Coordinates:		44.294402, -77.547219	
Span (m) =	16.6	[1]Soffit (m) =	116.90
[2]Upstream Invert (m)		Downstream Invert (m)	
113.80		113.80	
	Low Point of Road =	115.75	m
	[3]Timmins WSEL =	116.58	m
Maximum Relief Flow Depth (m)		Recommended Limit = 0.3m	
0.83		x	
Depth*Velocity Calculated (m^2/s)		Recommended Limit = 0.8 (m^3/s)	
0.35		✓	

[1]Soffit measured as highest point of bridge opening.

[2]Invert taken as creek inverts at upstream and downstream of bridge opening.

[3]Timmins WSEL measured at immediate upstream side of bridge.

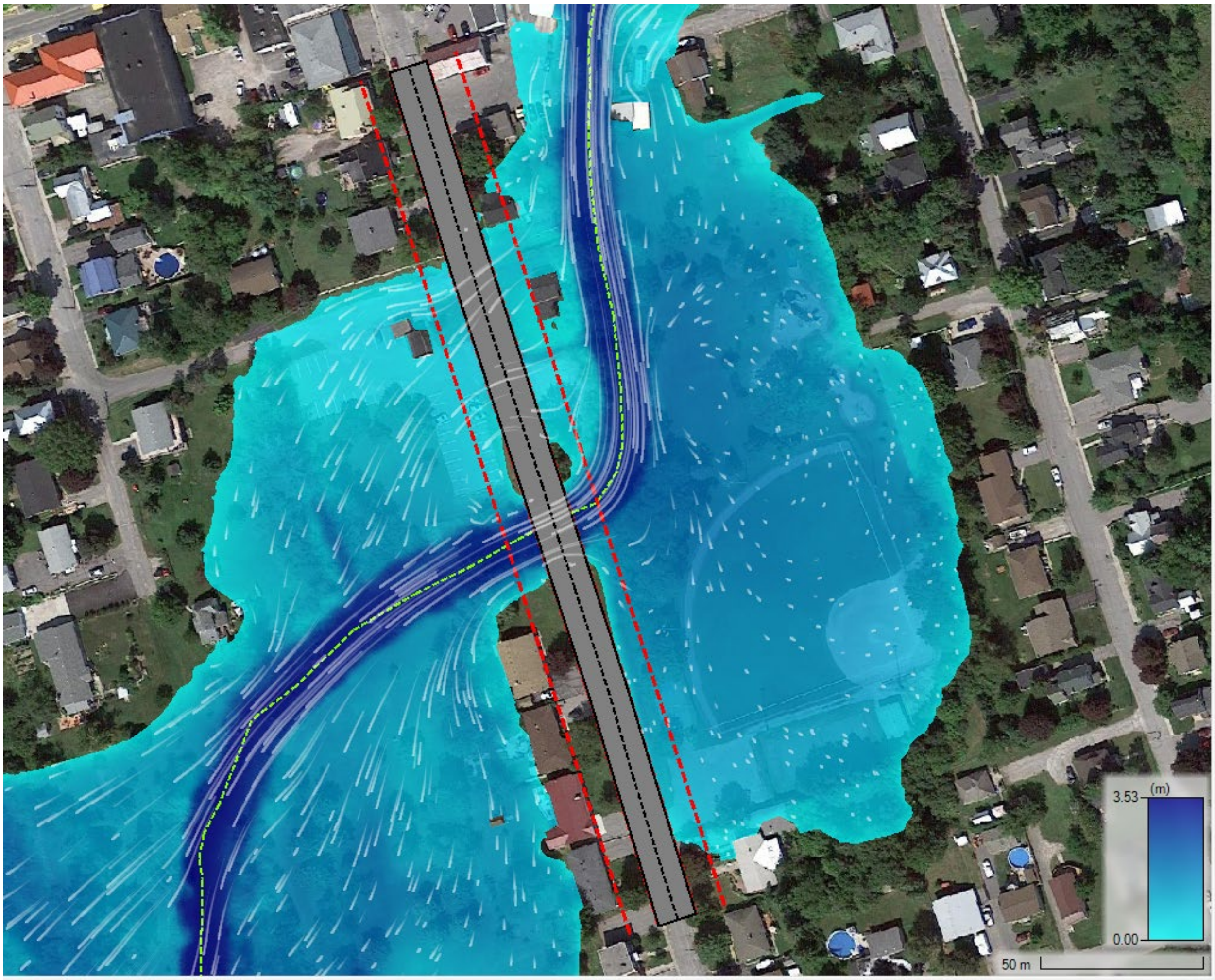


Figure 5-26: Schematic of Timmins Floodplain at Henry St Bridge– Satellite Background

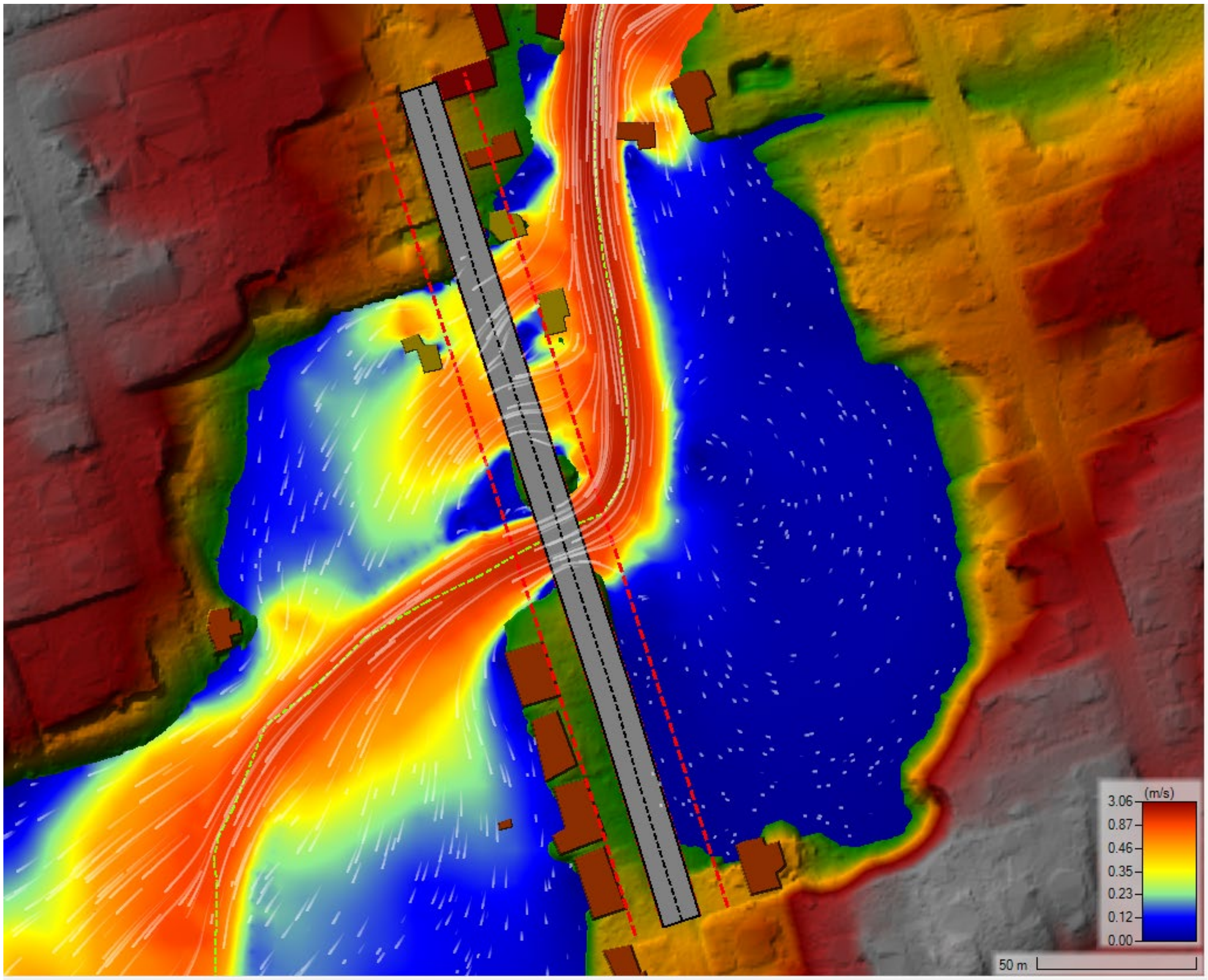


Figure 5-27: Schematic of Timmins Floodplain at Henry St Bridge – Terrain Background

5.4.9 Frankford Stirling Road

Frankford-Stirling Road is the most downstream crossing within the study area and consists of a 14.9m span bridge (see Figure 5-28). It is located a far distance (1km as the crow flies) from the nearest upstream crossing.

A summary of the bridge crossing at Frankford-Stirling Road is provided in Table 5-10. The stage and discharge hydrographs for this crossing are provided in Appendix I. The chart in Appendix I shows a minimal difference in headwater (HW) and tailwater (TW) elevations. Similar to Goods Road and Evergreen Road, this suggests that the bridge is efficient in conveyance its regulatory peak flow of 63.7 m^3/s. As a result, there is no overtopping of this bridge in the Timmins event and safe access is expected to be available in the regulatory storm.

Figure 5-28: Elevation View of Bridge at Frankford Stirling Road

Table 5-10: Frankford Stirling Road Crossing Summary

Road Name:		Frankford Stirling Rd	
Coordinates:		44.286505, -77.553394	
Span (m) =	14.9	[1]Soffit (m) =	115.68
[2]Upstream Invert (m)		Downstream Invert (m)	
112.35		112.37	
Low Point of Road =		116.30	m
[3]Timmins WSEL =		115.66	m
Maximum Relief Flow Depth (m)		Recommended Limit = 0.3m	
0		✓	
Depth*Velocity Calculated (m^2/s)		Recommended Limit = 0.8 (m^3/s)	
0		✓	

[1]Soffit measured as highest point of bridge opening.

[2]Invert taken as creek inverts at upstream and downstream of bridge opening.

[3]Timmins WSEL measured at immediate upstream side of bridge.

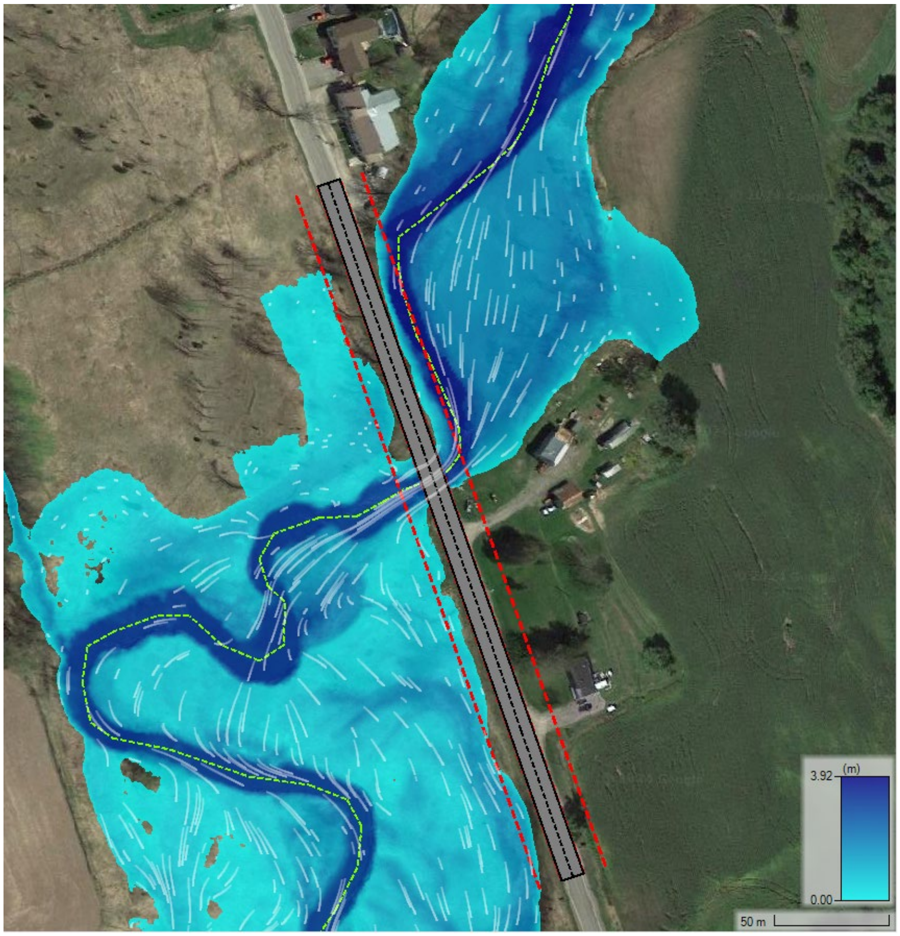


Figure 5-29: Schematic of Timmins Floodplain at Frankford Stirling Rd Bridge – Satellite Background

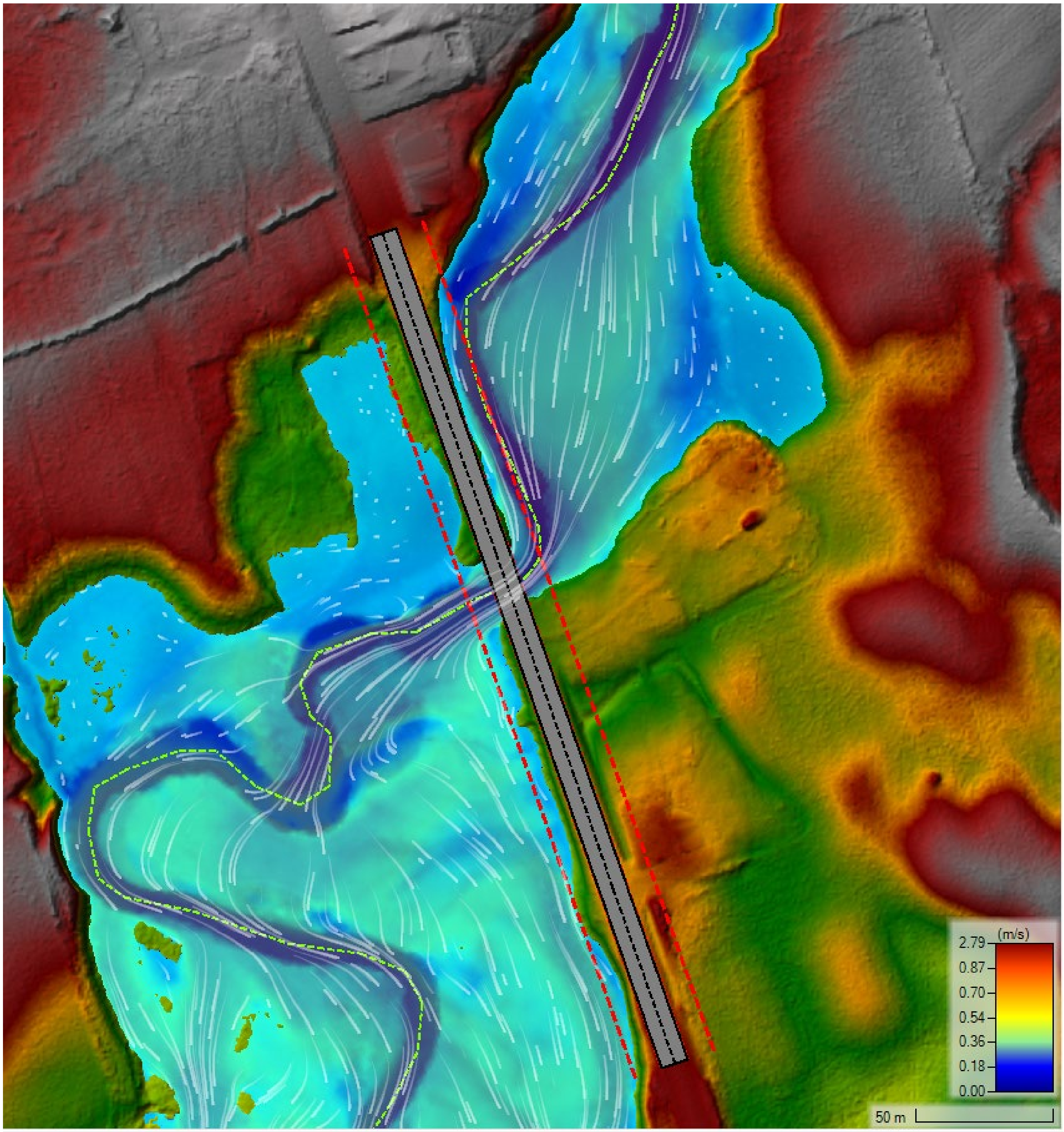


Figure 5-30: Schematic of Timmins Floodplain at Frankford Stirling Rd – Terrain Background

6 Sensitivity Analysis

The Flood hazard limits are derived from two separate modelling studies. Firstly, the peak flow rates are developed from hydrologic models, which estimate peak flows at various points of interest within the study area. Secondly, the hydraulic models incorporate the peak flows and estimate the water surface elevations within the study area.

The two models, in concert, serve as simplified predictive tools that emulate the watershed response to given precipitation events and estimate the resulting area of land that would be inundated by the flooding. The models have very simplistic inputs that attempt to represent the complex watershed conditions including slope, soils, land cover, land use, storage and surface roughness.

The objective for this sensitivity analysis is to attempt to answer the question – can we rely on the modelling results? That question is further refined to – how accurate is the estimate of the floodplain limits?

In this section, both uncertainty in the data and sensitivity of the model to the data and modelling techniques are explored.

6.1 Hydrologic Modelling

6.1.1 Precipitation Uncertainty and Sensitivity

Some hydrologic inputs have large uncertainties. An example is the precipitation depth. While many years of precipitation records are available at the selected precipitation station, uncertainty in the data arises from the method of collection, the maintenance of the gauge and siting of stations. It is reported that standard TB3 tipping bucket rain gauges underreport the precipitation depth by 3.5% and total depth gauges such as the Geonor T-200B underreport 4.7%. Older Type B rain gauges underreport by just 0.6% against the standard WMO pit gauge[1]. This would represent systematic losses in the data collection.

Return period precipitation depths are derived statistically from the data and estimates of return period depths are subject to the selection of statistical method and the period of record.

Precipitation used in the current is the data directly from the Environment Canada Intensity Duration Frequency (IDF) curves. EC reports the 95% confidence (equivalent to 2 standard deviations) for the precipitation intensities. As an example, the 1-hr intensities are reported with (+ / -) values in mm/h. These vary from 2.3mm to 10mm, or 11.7% to 20.4% of the stated intensities for the 2-yr to 100-yr respectively. While some estimate of confidence is provided for the statistical intensities, there is no direct statement within the station report on the confidence of the total depth estimates for return period.

[1] Field Accuracy of Canadian Rain Measurements, Kenneth A. Devine and Eva Mekis, Atmosphere-Ocean, 2008

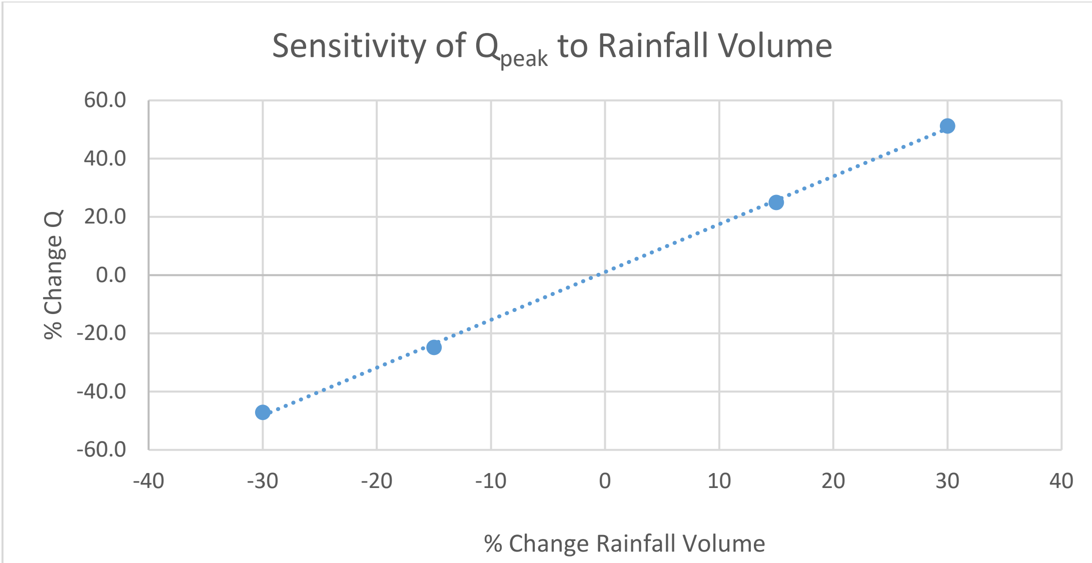


Figure 6-1: Sensitivity of Peak Runoff Rates to Rainfall Volume

6.2 Curve Number

The sensitivity analysis for the CN was completed to determine the impact this value has on peak flows. Figure 6-2 shows a comparison of CN values to the resulting peak flows from the hydrologic model. As expected, there is a strong correlation between CN and peak flows values. With a 15% increase in CN, there is approximately a 33% increase in peak flow. Similarly, for a 15% decrease in CN, there is approximately a 24% reduction in peak flow.

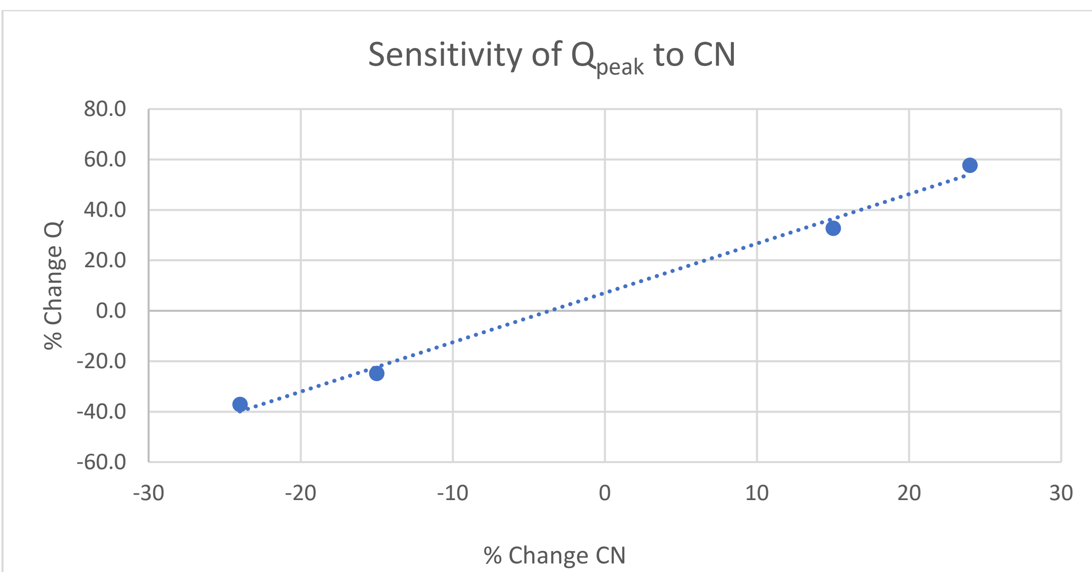


Figure 6-2: Sensitivity of Peak Runoff Rates to CN

6.2.1 Lag Time

The lag time has the least influence on peak flows of the three hydrologic inputs discussed, but still has moderate impacts on the model results. Figure 6-3 shows a comparison of lag time values to the resulting peak flows in the hydrologic model. With a 15% increase in lag time, there is a 10% decrease in peak flow. Similarly, for a 15% decrease in lag time, there is a 12% increase in peak flow.

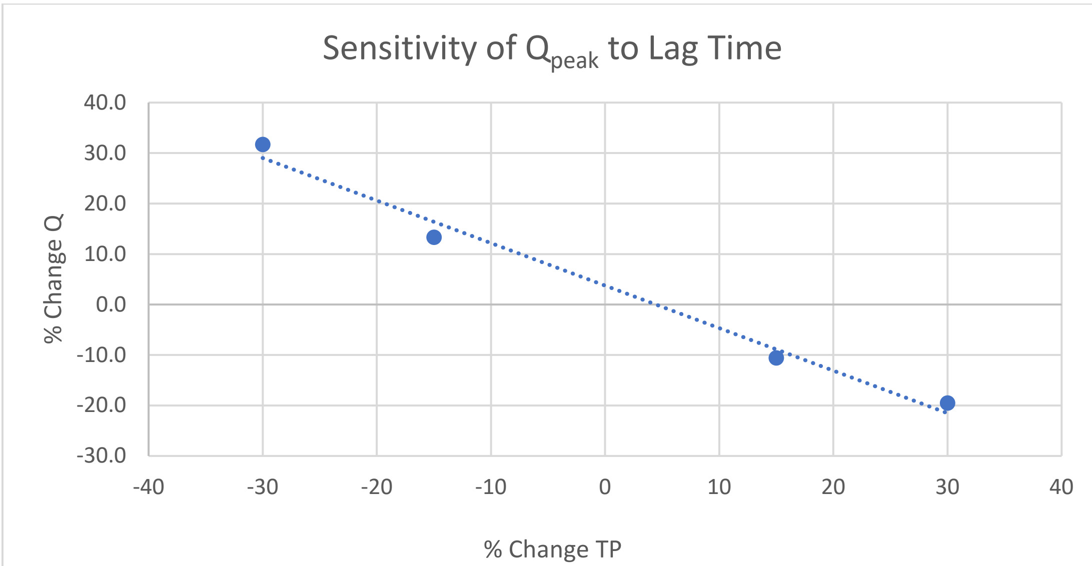


Figure 6-3: Sensitivity of Peak Runoff Rates to Lag Time

6.3 Hydraulic Modelling

The hydraulic model requires inputs for Manning's n values. The *HEC-RAS User's Manual* and *MTO Drainage Management Manual* provide ranges of roughness coefficient values for varying surface cover such as crop overbank areas, treed areas, and channel bottoms for natural watercourses. Mid-range, high, and low Manning's values were tested in different simulations to determine the effect of these values on the floodplain limits. Mid-range values were selected and applied in the regulatory floodplain mapping. A comparison of the flood limits for low, mid, and high-range values is shown in Figure 6-4. Evidently, the model is not overly sensitive to the Manning's n values.

Table 6-1: Manning's n Values Applied in Hydraulic Model Sensitivity Tests

Land Cover	Low	Medium	High
Swamp	0.035	0.045	0.06
Clear open water	0.028	0.032	0.035
Community infrastructure	0.035	0.05	0.12
Tree upland	0.05	0.07	0.09
Marsh	0.035	0.045	0.06
Deciduous treed	0.05	0.07	0.09
Mixed treed	0.05	0.07	0.09
Coniferous treed	0.05	0.07	0.09
Agriculture and undifferentiated rural	0.035	0.05	0.07
Plantations - treed cultivated	0.035	0.05	0.07
Hedge rows	0.04	0.05	0.07
Sand gravel mine tailings extraction	0.017	0.025	0.033

Figure 6-4: Comparison of Tested Flood Limits with Low (Cyan), Mid (Red), and High (Green) Manning's n Values Overlapping Due to Minimal Influence on Flood Extents

7 Flood Hazard Limit Delineation

The regulatory floodplain maps are included in the final deliverables package. The limits of the floodplain for the 50-, 100-, Timmins, and Timmins plus Climate Change events are also included.

7.1 Comparison of Historical Flood Limit to 2024 Mapping Update

A comparison of the historical flood limit and the 2024 flood limit is shown below. Recall that the previous flood limit is based on a significantly over-estimated Timmins peak flow of 292 m^3/s when compared to a frequency analysis of the Rawdon Creek stream flow gauge results. With the methodology described in Section 4 that considers the stream flow gauge results, the Timmins peak flow is within a more defensible range of 90.2 m^3/s. This difference in peak flows results in narrower flood extents relative to the previous mapping although one can see that the Rawdon Creek study area is relatively resilient to high peak flows for majority of the study area. The most notable difference between the past vs current draft flood limit is the reduced flood extents in the vicinity of Mill Street and upstream of the James St dam.

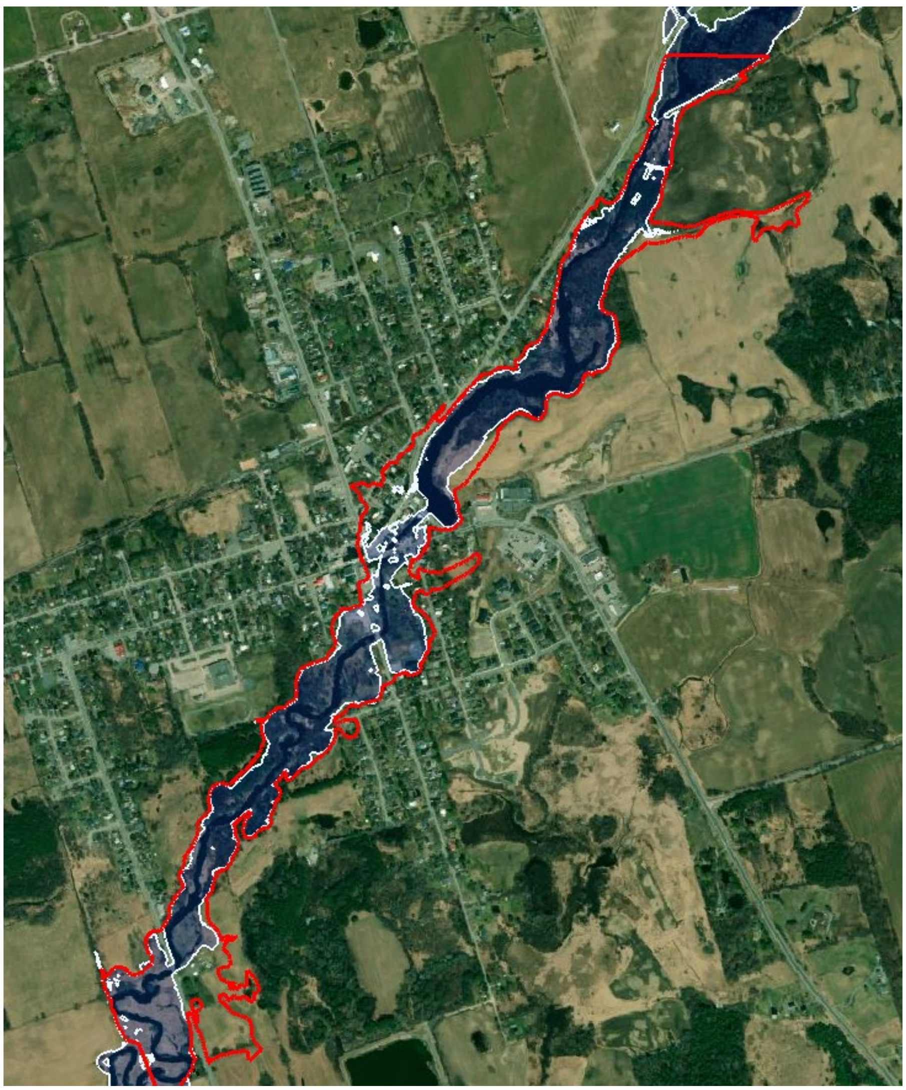


Figure 7-1: Comparison of Existing Timmins Flood Hazard (Red) to 2024 Draft Timmins Flood Limit (White with Blue Fill)

7.2 Water Surface Profiles

A plot of water surface profiles extending the full study area from Goods Road to Frankford Stirling Road is provided in Figure 7-2. The floodplain maps will include detailed station and water level data in addition to the georeferenced flood hazard limits.

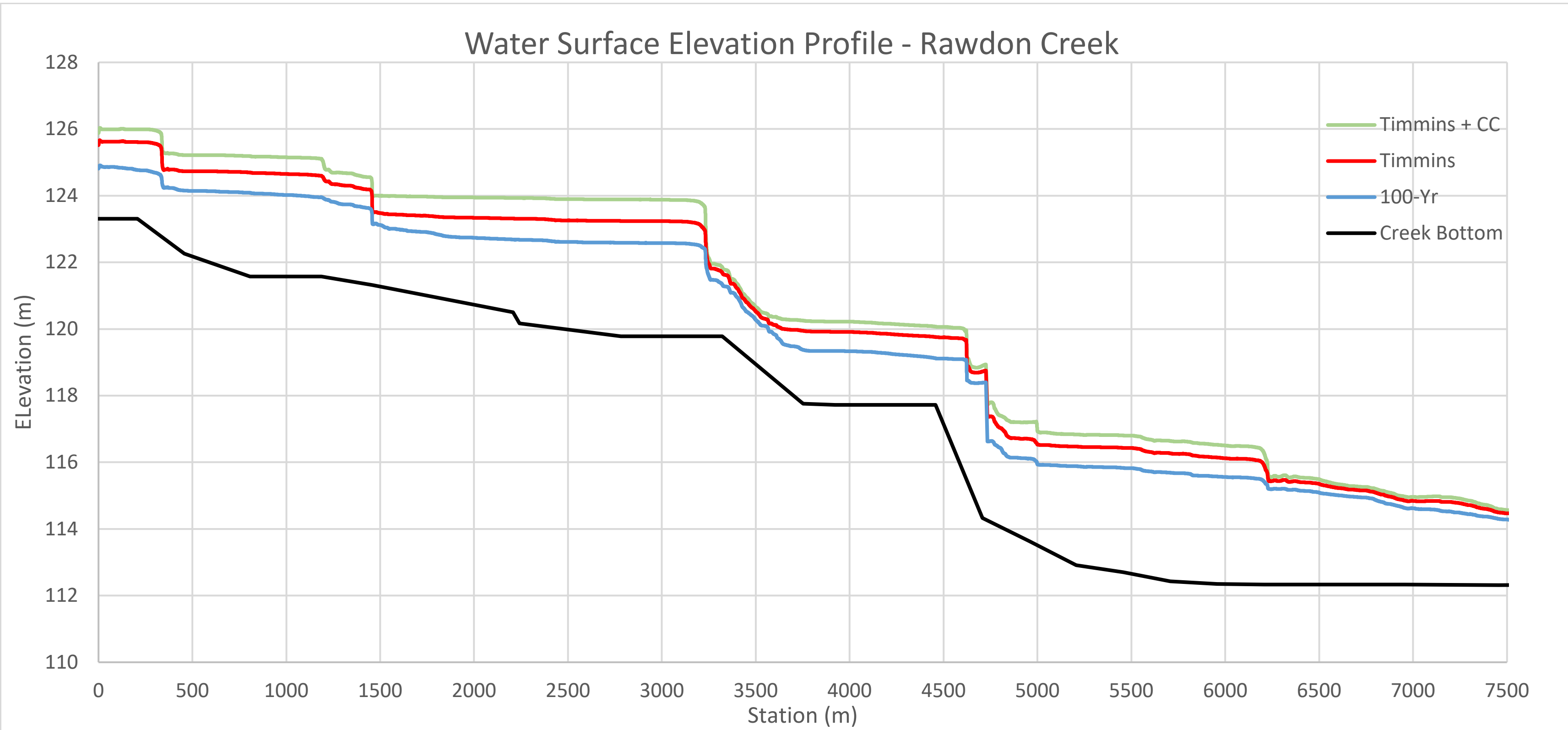


Figure 7-2: Water Surface Profile for 100-Yr, Timmins, and Timmins + Climate Change Events

7.3 Buildings within Flood Limit

The buildings within the flood hazard limit are predominantly in the vicinity of Mill Street and Henry Street within the community of Stirling. For the purpose of this estimate, a building is considered within the floodplain if the flood limit touches any point on the perimeter of the dwelling. There are two dwellings outside of the main urban area that may be susceptible to the Timmins regulatory flood hazard limit. Figure 7-3 illustrates the building locations that are within the floodplain in the 100-yr, Timmins, and/or Timmins plus climate change events.

The buildings that are within the floodplain in the 100-yr storm are illustrated by a yellow node. There are twenty (20) yellow nodes.

The buildings that are within the floodplain in the Timmins storm are illustrated by a pink node. There are sixteen (16) pink nodes.

The buildings that are within the floodplain in the Timmins plus climate change scenario are illustrated by a green node. There are twenty-two (22) green nodes.

The total number of buildings within the flood hazard limit for each event is summarized below.

Table 7-1: Number of Buildings within Flood Hazard Limit for Respective Storm Events

Storm Event	No. of Buildings
100-Yr	20
Timmins	36
Timmins + CC	58

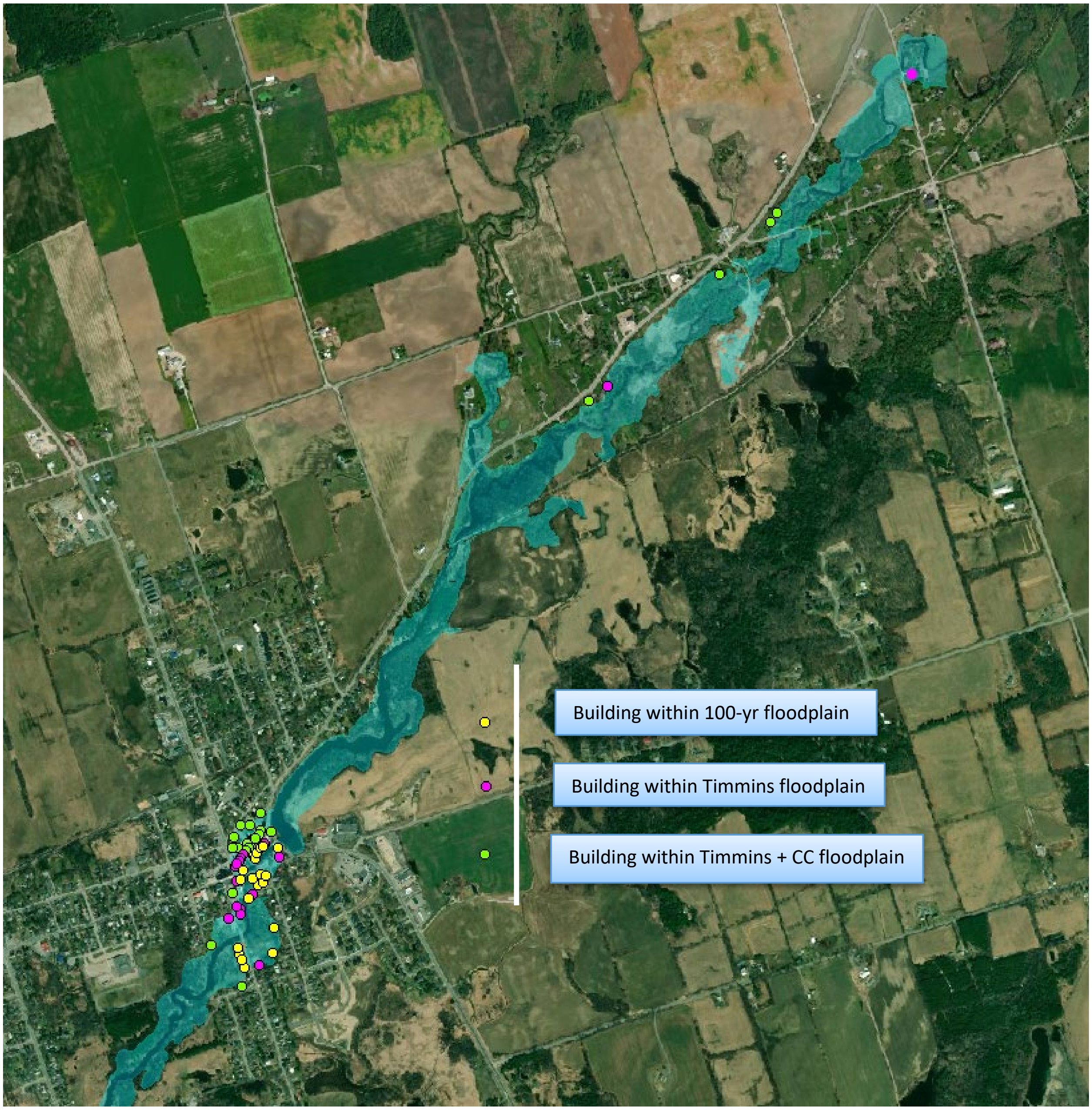


Figure 7-3: Illustration of Buildings within Floodplain

8 Conclusions

The Flood Hazard Identification Mapping Program has provided the opportunity for Lower Trent Conservation Authority, in partnership with the Township of Stirling-Rawdon and the provincial and federal partners, to complete the *2024 Rawdon Creek Floodplain Mapping Update*.

The finer details of this report provide an overview of the rigorous testing of the hydrology and hydraulics that has been completed to ensure reliable flood hazard limits are presented in the 2024 Rawdon Creek floodplain maps. The current mapping will allow Conservation and Township staff to make informed planning and regulatory decisions to help mitigate the flood risk to life and property, with emphasis on the urban core throughout the Village of Stirling.

Section 7.3 identifies the buildings currently within the 1% AEP, Timmins, and/or climate change storm events. The findings in this report provide the foundation and modelling tools to support a detailed investigation of mitigation alternatives in the event that a mitigation assessment is completed in the future.

We commend the Lower Trent Conservation staff and project partners for their efforts in preparing the *2024 Rawdon Creek Floodplain Mapping Update* that will benefit the local community within the Township of Stirling-Rawdon for many years to come.

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Jewell Engineering Inc.

Project Director:

Bryon Keene, P.Eng.
Jewell Engineering Inc.

9 References

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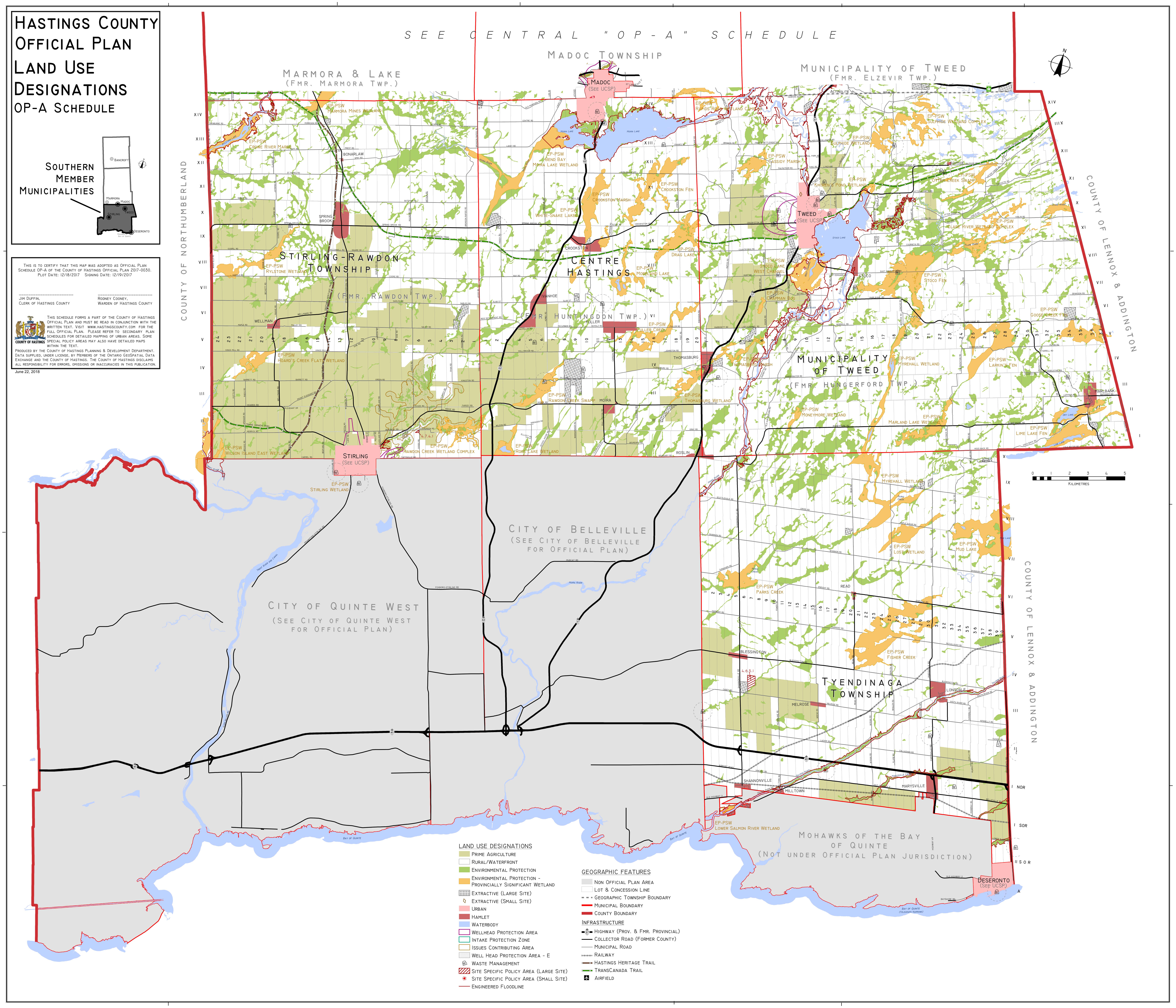
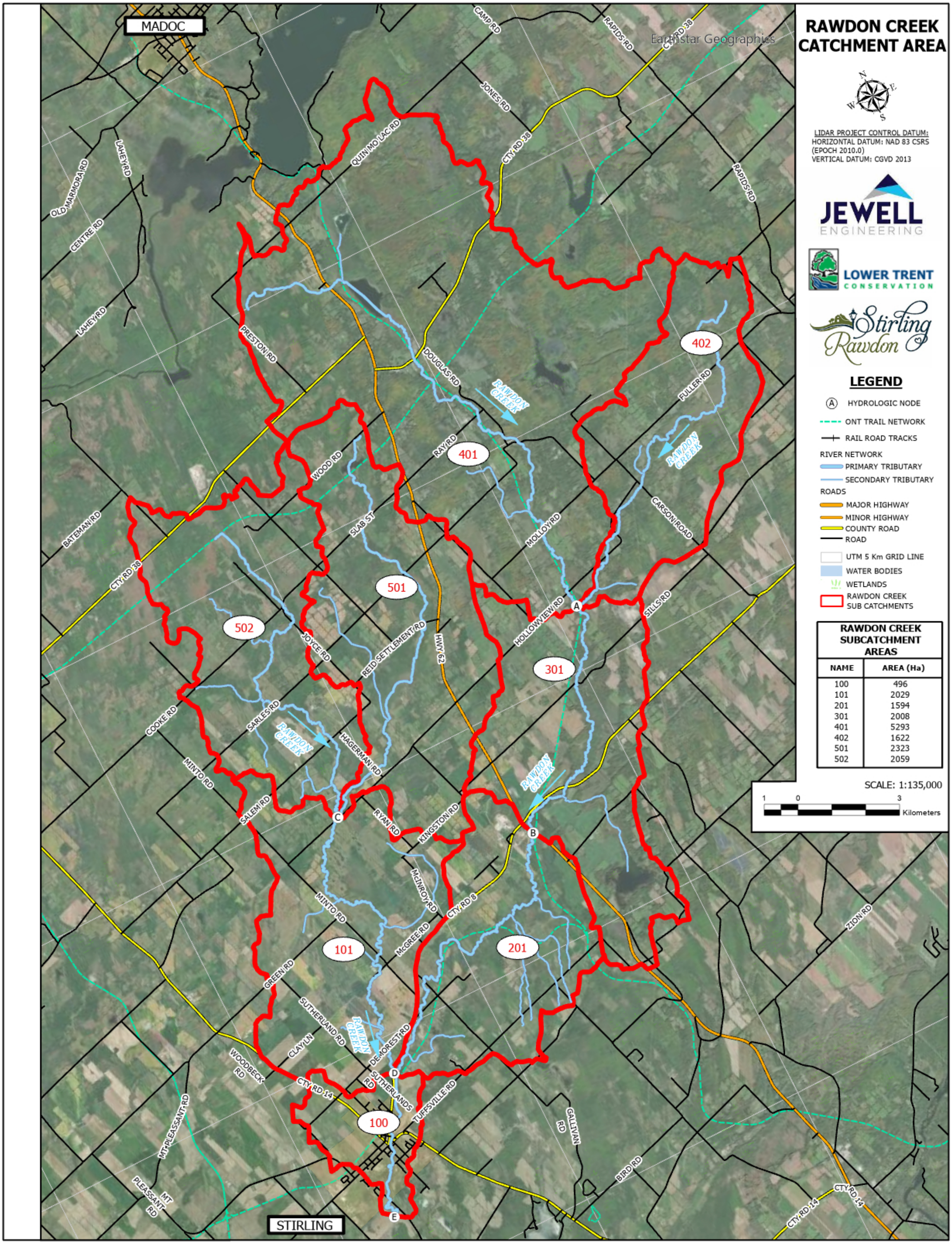
Appendix A:
Official Plan – Schedule A - South



Hastings County
Official Plan
Land Use
Designations
OP-A Schedule
Southern
Member
Municipalities
Bancroft
Marmora
Madoc
Tweed
Stirling
Deseronto
Mohawks of the Bay of Quinte
This is to certify that this map was adopted as Official Plan Schedule OP-A of the County of Hastings Official Plan 2017-0030.
Plot Date: 12/18/2017 Signing Date: 12/19/2017
Jim Duffin,
Clerk of Hastings County
Rodney Cooney,
Warden of Hastings County
Nisi Dominus Frustra
County of Hastings
This schedule forms a part of the County of Hastings Official Plan and must be read in conjunction with the written text. Visit www.hastingscounty.com for the full Official Plan. Please refer to secondary plan schedules for detailed mapping of urban areas. Some special policy areas may also have detailed maps within the text.
Produced by the County of Hastings Planning & Development Department. Data supplied, under license, by Members of the Ontario GeoSpatial Data Exchange and the County of Hastings. The County of Hastings disclaims all responsibility for errors, omissions or inaccuracies in this publication.
June 22, 2018
See Central "OP-A" Schedule
Madoc Township
Marmora & Lake
(Fmr. Marmora Twp.)
Municipality of Tweed
(Fmr. Elzevir Twp.)
County of Northumberland
County of Lennox & Addington
Madoc
(See UCSP)
Tweed
(See UCSP)
Stirling
(See UCSP)
Deseronto
(See UCSP)
Stirling-Rawdon Township
(Fmr. Rawdon Twp.)
Centre Hastings
(Fmr. Huntingdon Twp.)
Municipality of Tweed
(Fmr. Hungerford Twp.)
Tyendinaga Township
City of Belleville
(See City of Belleville for Official Plan)
City of Quinte West
(See City of Quinte West for Official Plan)
Mohawks of the Bay of Quinte
(Not under Official Plan Jurisdiction)
Bay of Quinte
Kilometres
Land Use Designations
Prime Agriculture
Rural/Waterfront
Environmental Protection
Environmental Protection - Provincially Significant Wetland
Extractive (Large Site)
Extractive (Small Site)
Urban
Hamlet
Waterbody
Wellhead Protection Area
Intake Protection Zone
Issues Contributing Area
Well Head Protection Area - E
Waste Management
Site Specific Policy Area (Large Site)
Site Specific Policy Area (Small Site)
Engineered Floodline
Geographic Features
Non Official Plan Area
Lot & Concession Line
Geographic Township Boundary
Municipal Boundary
County Boundary
Infrastructure
Highway (Prov. & Fmr. Provincial)
Collector Road (Former County)
Municipal Road
Railway
Hastings Heritage Trail
TransCanada Trail
Airfield

Appendix B:
Rawdon Creek Catchment Area Drawings



RAWDON CREEK SUBCATCHMENT AREAS

NAME	AREA (Ha)
100	496
101	2029
201	1594
301	2008
401	5293
402	1622
501	2323
502	2059

Node Descriptions

As shown in Appendix B, there are five points of interest (nodes) throughout the Rawdon Creek watershed where peak flows were determined.

Node A is the northernmost node where the main tributaries of Sub-catchments 401 and 402 converge. Node A is located just south of Hollowview Road, as seen in Appendix B

Node B corresponds to the location of the 'Rawdon Creek Near West Huntingdon' stream flow gauge (02HK008). Node B is just downstream of sub-catchment 301.

Node C is located along Hollowview Rd, East of Highway 62. The main tributaries of sub-catchments 501 and 502 converge at Node C.

Node D is located Northeast of the downtown Stirling area, just south of Ridge Rd. Rawdon Creek becomes a single, defined channel at Node D.

Node E is at the southernmost point of the watershed, where Rawdon Creek discharges to the Trent River. The entire Rawdon Creek watershed drains to Node E.

Catchment Descriptions

Sub-catchment 100 has a drainage area of 4.96 km^2, covering approximately 3% of the watershed. The soils are predominately soils group C, meaning the soils have a slow infiltration and transmission rate when wet. The largest land cover in sub-catchment 100 is cultivated land, which makes up 66% of the area. Sub-catchment 100 is the southern-most catchment and is at the downstream limit of the Rawdon Creek watershed.

Sub-catchment 101 has a drainage area of 20.3 km^2, covering approximately 12% of the watershed. The soils here are predominately soils group C, meaning they have slow infiltration and transmission rates when wet. The dominant land cover in Sub-catchment 101 is cultivated land, with 82% coverage.

Sub-catchment 201 has a drainage area of 15.9 km^2, covering approximately 9% of the watershed. The soils here are predominately soils group B, which suggests that they have moderate infiltration and transmission rates when wet. Approximately 62% of the sub-catchment has cultivated land cover.

Sub-catchment 301 has a drainage area of 20.8 km^2, covering approximately 12% of the watershed. The soils here are predominately soils group B, having moderate infiltration and transmission rates when wet. Cultivated land is the dominant land cover with 65% land coverage.

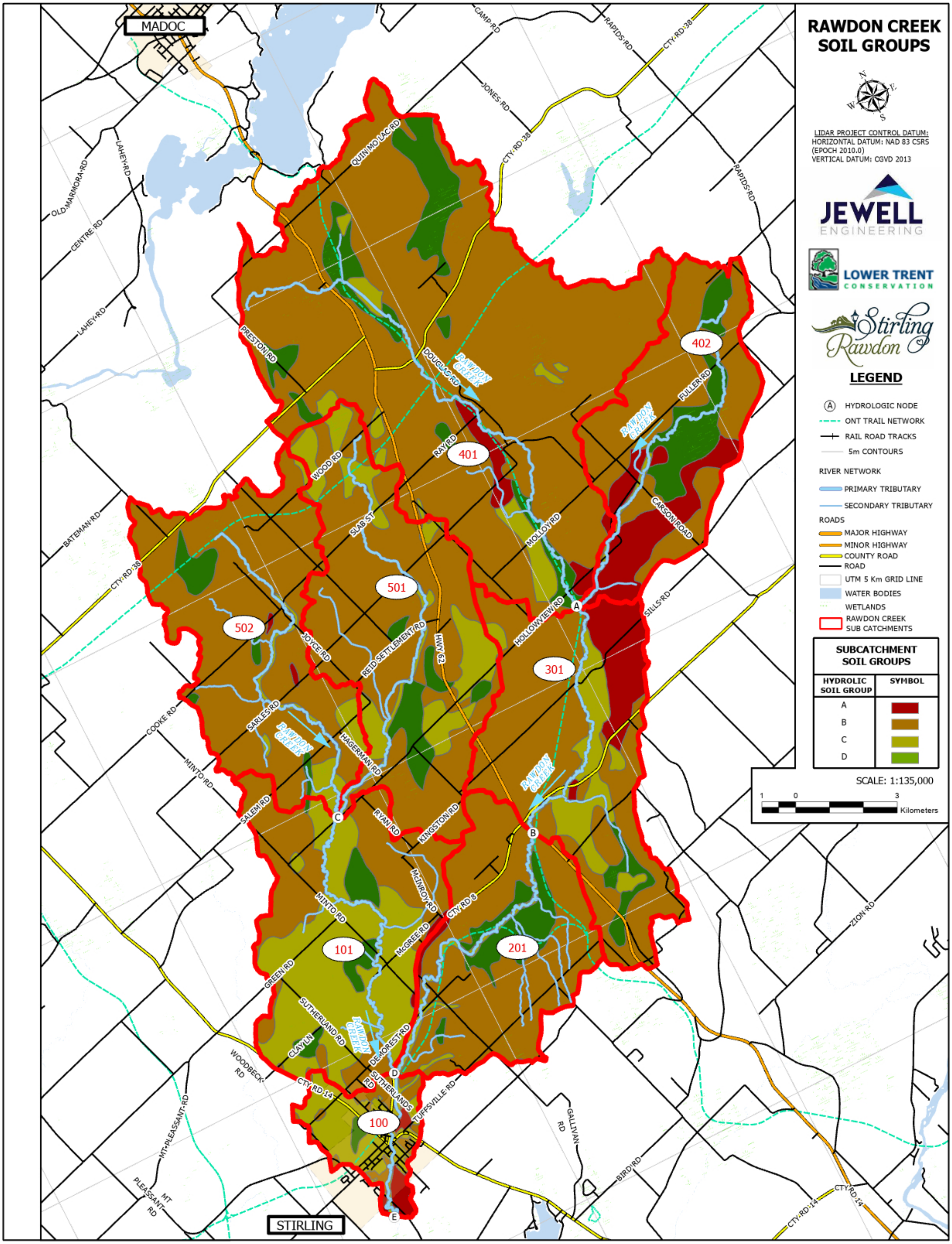
Sub-catchment 401 has a drainage area of 53.0 km^2, covering approximately 30% of the watershed. The soils in sub-catchment 401 are predominately soils group B, meaning they have moderate infiltration and transmission rates when wet. Woods and water have the greatest land coverage at 37% and 35%, respectively.

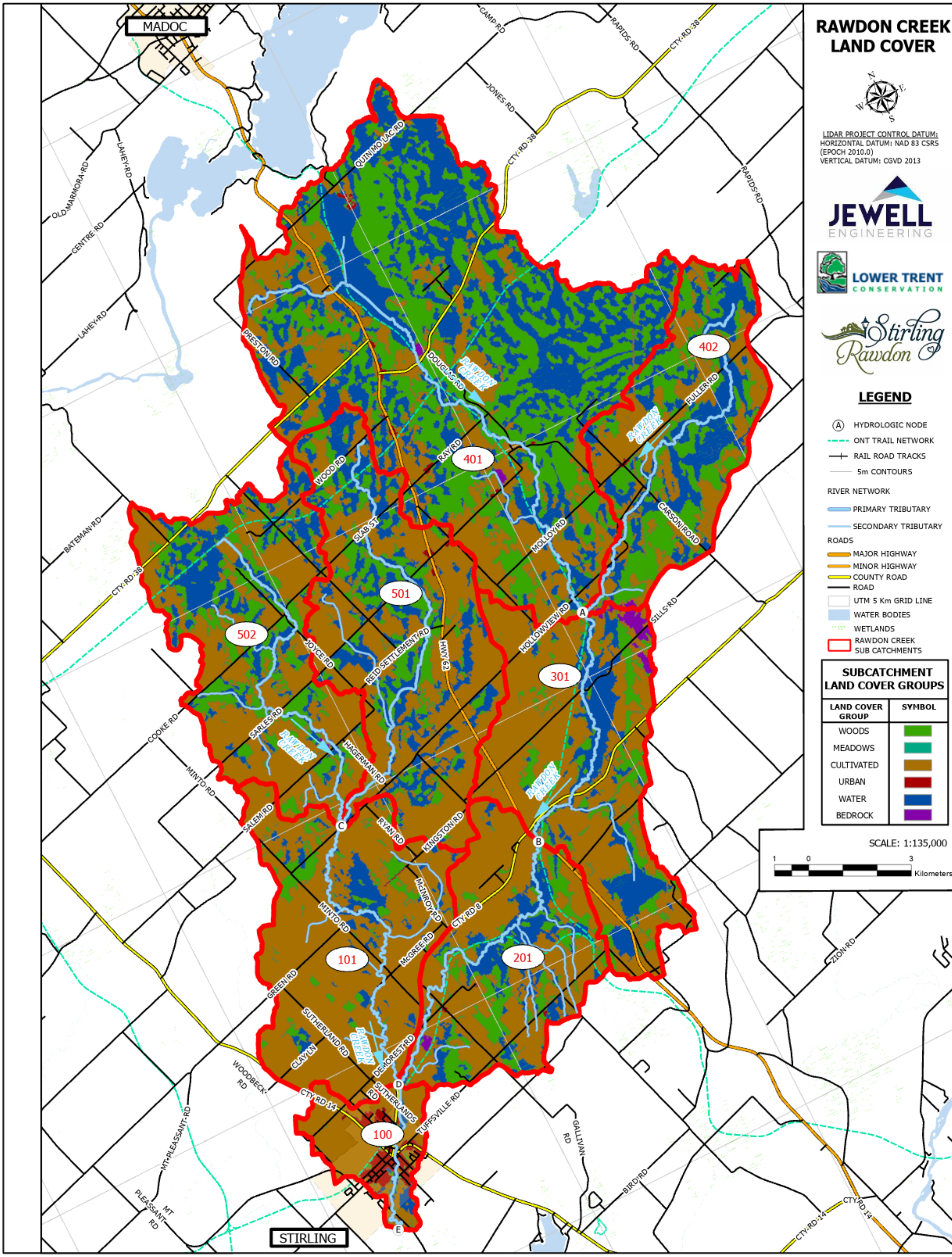
Sub-catchment 402 has a drainage area of 16.2 km^2, covering approximately 9% of the watershed. The soils are predominately soils group B, which have moderate infiltration and transmission rates when wet. Cultivated land, woods, and water cover the largest areas in the sub-catchment at 36%, 34%, and 29%, respectively.

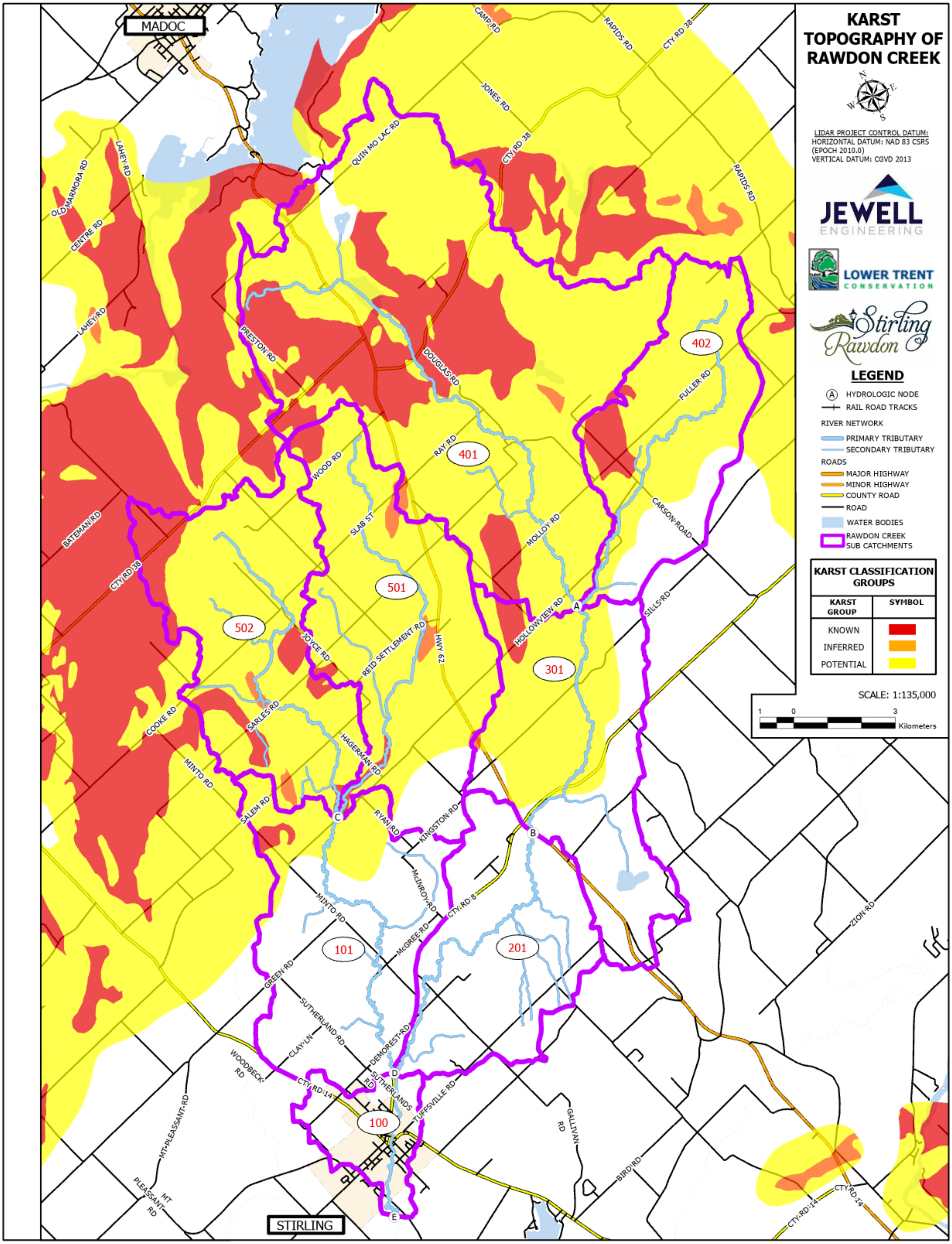
Sub-catchment 501 has a drainage area of 23.2 km^2, covering approximately 13% of the watershed. The soils are predominately soils group B, meaning they have moderate infiltration and transmission rates when wet. Cultivated land is the most prevalent land cover in sub-catchment 501, with 55% land coverage.

Sub-catchment 502 has a drainage area of 20.6 km^2, covering approximately 12% of the watershed. The soils in sub-catchment 502 are predominately soils group B, which suggests that the soils have moderate infiltration and transmission rates when wet. Cultivated land has the greatest land coverage at 54%.

Appendix C:
Soil, Land Cover, and Karst Maps


RAWDON CREEK SOIL GROUPS
LIDAR PROJECT CONTROL DATUM:
HORIZONTAL DATUM: NAD 83 CSRS (EPOCH 2010.0)
VERTICAL DATUM: CGVD 2013
JEWELL ENGINEERING
LOWER TRENT CONSERVATION
Stirling Rawdon
LEGEND
A HYDROLOGIC NODE
ONT TRAIL NETWORK
RAIL ROAD TRACKS
5m CONTOURS
RIVER NETWORK
PRIMARY TRIBUTARY
SECONDARY TRIBUTARY
ROADS
MAJOR HIGHWAY
MINOR HIGHWAY
COUNTY ROAD
ROAD
UTM 5 Km GRID LINE
WATER BODIES
WETLANDS
RAWDON CREEK SUB CATCHMENTS
SUBCATCHMENT SOIL GROUPS
HYDROLIC SOIL GROUP
SYMBOL
A
B
C
D
SCALE: 1:135,000
1 0 3 Kilometers
MADOC
STIRLING
100
101
201
301
401
402
501
502
RAWDON CREEK


RAWDON CREEK
LAND COVER
LIDAR PROJECT CONTROL DATUM:
HORIZONTAL DATUM: NAD 83 CSRS
(EPOCH 2010.0)
VERTICAL DATUM: CGVD 2013
JEWELL
ENGINEERING
LOWER TRENT
CONSERVATION
Stirling
Rawdon
LEGEND
HYDROLOGIC NODE
ONT TRAIL NETWORK
RAIL ROAD TRACKS
5m CONTOURS
RIVER NETWORK
PRIMARY TRIBUTARY
SECONDARY TRIBUTARY
ROADS
MAJOR HIGHWAY
MINOR HIGHWAY
COUNTY ROAD
ROAD
UTM 5 Km GRID LINE
WATER BODIES
WETLANDS
RAWDON CREEK
SUB CATCHMENTS
SUBCATCHMENT
LAND COVER GROUPS
LAND COVER GROUP
SYMBOL
WOODS
MEADOWS
CULTIVATED
URBAN
WATER
BEDROCK
SCALE: 1:135,000
1
0
3
Kilometers
MADOC
STIRLING
RAWDON CREEK
402
401
501
502
301
101
201
100
A
B
C
D
E
QUIN MO LAC RD
PRESTON RD
DOUGLAS RD
RAY RD
WOOD RD
SLAB ST
MOLLOY RD
FULLER RD
CARSON ROAD
SILLS RD
HOLLOWVIEW RD
REID SETTLEMENT RD
HWY 62
JOYCE RD
SARLES RD
HAGERMAN RD
COOKE RD
MINTO RD
SALEM RD
RYAN RD
KINGSTON RD
McINROY RD
McGREE RD
CTY RD 8
GREEN RD
SUTHERLAND RD
CLAY LN
DEMOREST RD
SUTHERLANDS RD
CTY RD 14
TUFFSVILLE RD
WOODBECK RD
MT PLEASANT RD
GALLIVAN RD
BIRD RD
ZION RD
CAMP RD
RAPIDS RD
CTY RD 38
JONES RD
OLD MARMORA RD
LAHEY RD
CENTRE RD
BATEMAN RD


KARST TOPOGRAPHY OF RAWDON CREEK
LIDAR PROJECT CONTROL DATUM:
HORIZONTAL DATUM: NAD 83 CSRS (EPOCH 2010.0)
VERTICAL DATUM: CGVD 2013
JEWELL ENGINEERING
LOWER TRENT CONSERVATION
Stirling Rawdon
LEGEND
HYDROLOGIC NODE
RAIL ROAD TRACKS
RIVER NETWORK
PRIMARY TRIBUTARY
SECONDARY TRIBUTARY
ROADS
MAJOR HIGHWAY
MINOR HIGHWAY
COUNTY ROAD
ROAD
WATER BODIES
RAWDON CREEK SUB CATCHMENTS
KARST CLASSIFICATION GROUPS
KARST GROUP
SYMBOL
KNOWN
INFERRED
POTENTIAL
SCALE: 1:135,000
1 0 3
Kilometers
MADOC
STIRLING
402
401
501
502
301
101
201
100
A
B
C
D
E
CAMP RD
RAPIDS RD
CTY RD 38
JONES RD
QUIN MO LAC RD
OLD MARMORA RD
LAHEY RD
CENTRE RD
PRESTON RD
DOUGLAS RD
FULLER RD
RAY RD
WOOD RD
BATEMAN RD
SLAB ST
MOLLOY RD
CARSON ROAD
HOLLOWVIEW RD
SILLS RD
JOYCE RD
REID SETTLEMENT RD
HWY 62
COOKE RD
SARLES RD
HAGERMAN RD
MINTO RD
SALEM RD
RYAN RD
KINGSTON RD
McINROY RD
CTY RD 8
McGREE RD
ZION RD
GREEN RD
SUTHERLAND RD
CLAY LN
DEMOREST RD
WOODBECK RD
CTY RD 14
SUTHERLANDS RD
TUFFSVILLE RD
GALLIVAN RD
MT PLEASANT RD
BIRD RD

Appendix D:
Federal Climate Data Portal: ΔT Adjustment

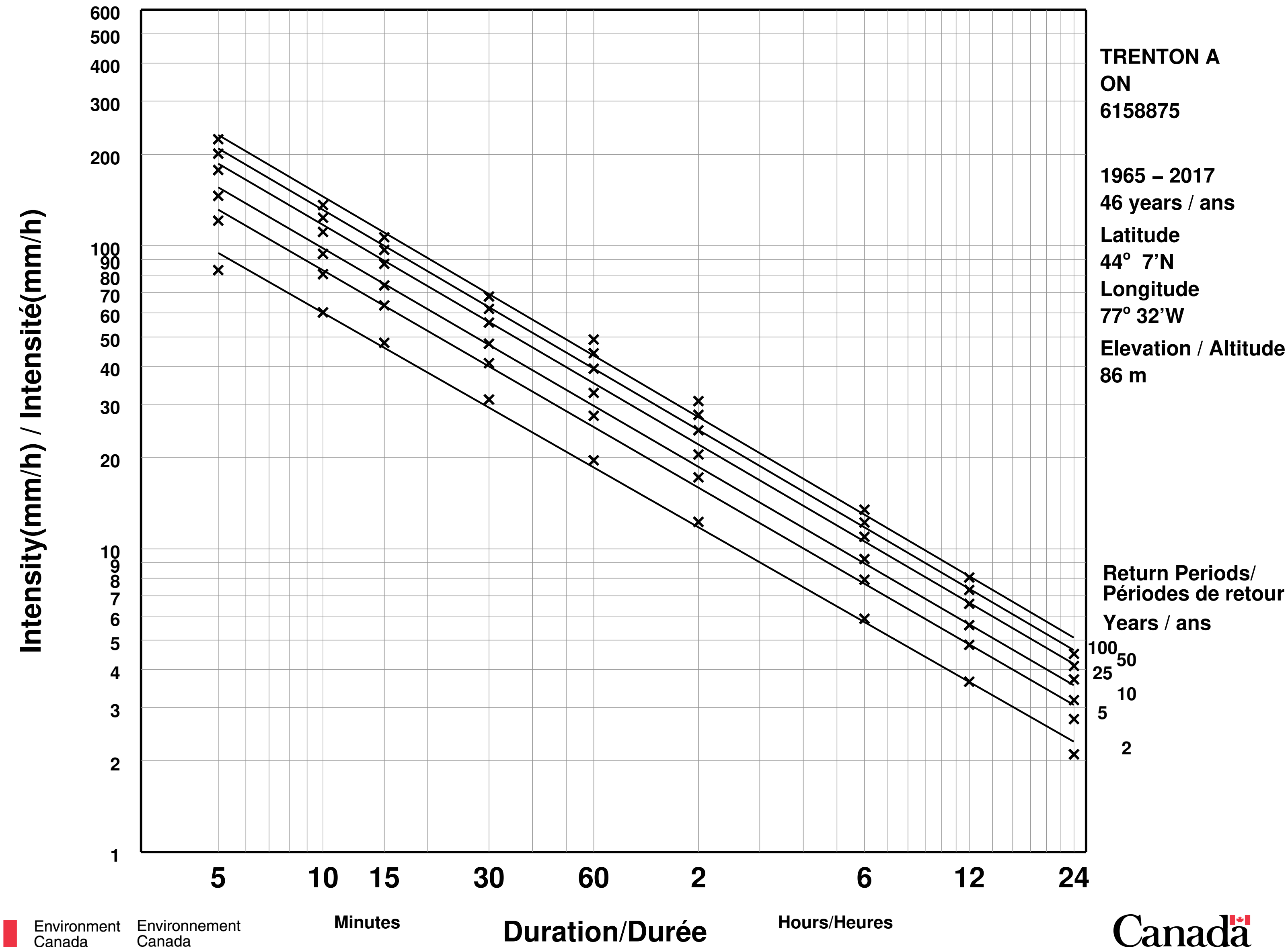
time	rcp26_tg_mean_p10	rcp26_tg_mean_p50	rcp26_tg_mean_p90	rcp45_tg_mean_p10	rcp45_tg_mean_p50	rcp45_tg_mean_p90	rcp85_tg_mean_p10	rcp85_tg_mean_p50	rcp85_tg_mean_p90	rcp26_tg_mean_delta7100_p10	rcp26_tg_mean_delta7100_p50	rcp26_tg_mean_delta7100_p90	rcp45_tg_mean_delta7100_p10	rcp45_tg_mean_delta7100_p50	rcp45_tg_mean_delta7100_p90	rcp85_tg_mean_delta7100_p10	rcp85_tg_mean_delta7100_p50	rcp85_tg_mean_delta7100_p90
1/1/1951	6.9	7	7.2	6.9	7	7.2	6.9	7	7.2	-0.6	-0.3	-0.1	-0.6	-0.3	-0.1	-0.6	-0.3	-0.1
1/1/1961	7	7.1	7.2	7	7.1	7.2	7	7.1	7.2	-0.5	-0.3	-0.1	-0.5	-0.3	-0.1	-0.5	-0.3	-0.1
1/1/1971	7.2	7.3	7.5	7.2	7.3	7.5	7.2	7.3	7.5	0	0	0	0	0	0	0	0	0
1/1/1981	7.5	7.7	7.9	7.5	7.7	7.9	7.5	7.7	7.9	0.2	0.3	0.4	0.2	0.4	0.5	0.2	0.3	0.5
1/1/1991	7.9	8	8.4	7.8	8.1	8.4	7.9	8.2	8.4	0.5	0.7	1	0.5	0.8	1	0.6	0.8	1
1/1/2001	8.2	8.4	9	8.2	8.6	9	8.2	8.7	9	0.8	1.2	1.6	0.7	1.2	1.6	0.9	1.3	1.6
1/1/2011	8.3	8.8	9.5	8.4	9	9.7	8.5	9.2	9.8	1	1.4	2.1	1.1	1.7	2.3	1.3	1.8	2.4
1/1/2021	8.5	9.2	10	8.8	9.3	10.2	8.9	9.6	10.3	1.2	1.8	2.6	1.4	2	2.9	1.7	2.2	3
1/1/2031	8.5	9.4	10.3	9.1	9.7	10.8	9.4	10.2	11.3	1.3	2	2.9	1.7	2.4	3.4	2.2	2.8	3.9
1/1/2041	8.6	9.4	10.6	9.2	10.1	11.3	10.1	10.9	12.2	1.3	2.1	3.3	1.8	2.8	4	2.8	3.5	4.8
1/1/2051	8.6	9.4	10.8	9.3	10.3	11.8	10.8	11.6	13.3	1.2	2.1	3.4	1.9	2.9	4.5	3.4	4.2	6
1/1/2061	8.6	9.4	10.8	9.4	10.4	12.2	11.4	12.4	14.2	1.2	2.1	3.4	2	3.2	4.8	4.1	5	6.8
1/1/2071	8.6	9.4	10.7	9.5	10.6	12.1	11.8	13.1	15.1	1.2	2.1	3.3	2.2	3.3	4.7	4.6	5.7	7.7

Appendix E:
Trenton A Environment Canada IDFs

Short Duration Rainfall Intensity–Duration–Frequency Data

2022/10/31

Données sur l'intensité, la durée et la fréquence des chutes de pluie de courte durée



Environment Canada Environnement Canada

Canada

Calculate Precipitation Frequency Curve using Gumbel

Year	Depth (mm)	Rank	q_i	p_i	T_p est	(x-u)/a	p theor	T_p theor
1988	28	48	0.9884	0.0116	1.0	-1.2771	0.0277	1.0
1976	30.2	47	0.9676	0.0324	1.0	-1.1158	0.0473	1.0
1991	32.8	46	0.9468	0.0532	1.1	-0.9251	0.0803	1.1
1975	34.3	45	0.9260	0.0740	1.1	-0.8151	0.1044	1.1
1989	34.7	44	0.9052	0.0948	1.1	-0.7857	0.1115	1.1
1994	34.8	43	0.8845	0.1155	1.1	-0.7784	0.1133	1.1
1971	35.1	42	0.8637	0.1363	1.2	-0.7564	0.1188	1.1
1978	36.6	41	0.8429	0.1571	1.2	-0.6464	0.1483	1.2
1982	39	40	0.8221	0.1779	1.2	-0.4704	0.2018	1.3
1985	39.7	39	0.8013	0.1987	1.2	-0.4190	0.2186	1.3
2001	40.4	38	0.7805	0.2195	1.3	-0.3677	0.2359	1.3
1968	40.9	37	0.7598	0.2402	1.3	-0.3310	0.2485	1.3
1984	42.2	36	0.7390	0.2610	1.4	-0.2357	0.2820	1.4
1987	42.4	35	0.7182	0.2818	1.4	-0.2210	0.2873	1.4
1992	42.8	32	0.6559	0.3441	1.5	-0.1917	0.2978	1.4
1992	42.8	32	0.6559	0.3441	1.5	-0.1917	0.2978	1.4
1992	42.8	32	0.6559	0.3441	1.5	-0.1917	0.2978	1.4
1965	43.9	31	0.6351	0.3649	1.6	-0.1110	0.3271	1.5
1966	45.7	30	0.6143	0.3857	1.6	0.0210	0.3756	1.6
2016	46.2	29	0.5935	0.4065	1.7	0.0577	0.3891	1.6
1972	47.2	28	0.5727	0.4273	1.7	0.1310	0.4160	1.7
2008	47.6	27	0.5520	0.4480	1.8	0.1604	0.4266	1.7
1970	48	26	0.5312	0.4688	1.9	0.1897	0.4373	1.8
1981	48.2	25	0.5104	0.4896	2.0	0.2044	0.4426	1.8
1990	50	24	0.4896	0.5104	2.0	0.3364	0.4895	2.0
2003	50.2	23	0.4688	0.5312	2.1	0.3511	0.4946	2.0
1973	53.6	22	0.4480	0.5520	2.2	0.6004	0.5778	2.4
1997	53.9	21	0.4273	0.5727	2.3	0.6224	0.5847	2.4
2005	54.1	20	0.4065	0.5935	2.5	0.6371	0.5893	2.4
1969	54.9	19	0.3857	0.6143	2.6	0.6958	0.6073	2.5
1979	55.8	18	0.3649	0.6351	2.7	0.7618	0.6270	2.7
1993	56	17	0.3441	0.6559	2.9	0.7764	0.6313	2.7
2010	59.1	16	0.3234	0.6766	3.1	1.0038	0.6932	3.3
1980	60	15	0.3026	0.6974	3.3	1.0698	0.7096	3.4
2007	62.1	14	0.2818	0.7182	3.5	1.2238	0.7452	3.9
1983	63.3	13	0.2610	0.7390	3.8	1.3118	0.7639	4.2
1995	64.9	12	0.2402	0.7598	4.2	1.4292	0.7870	4.7

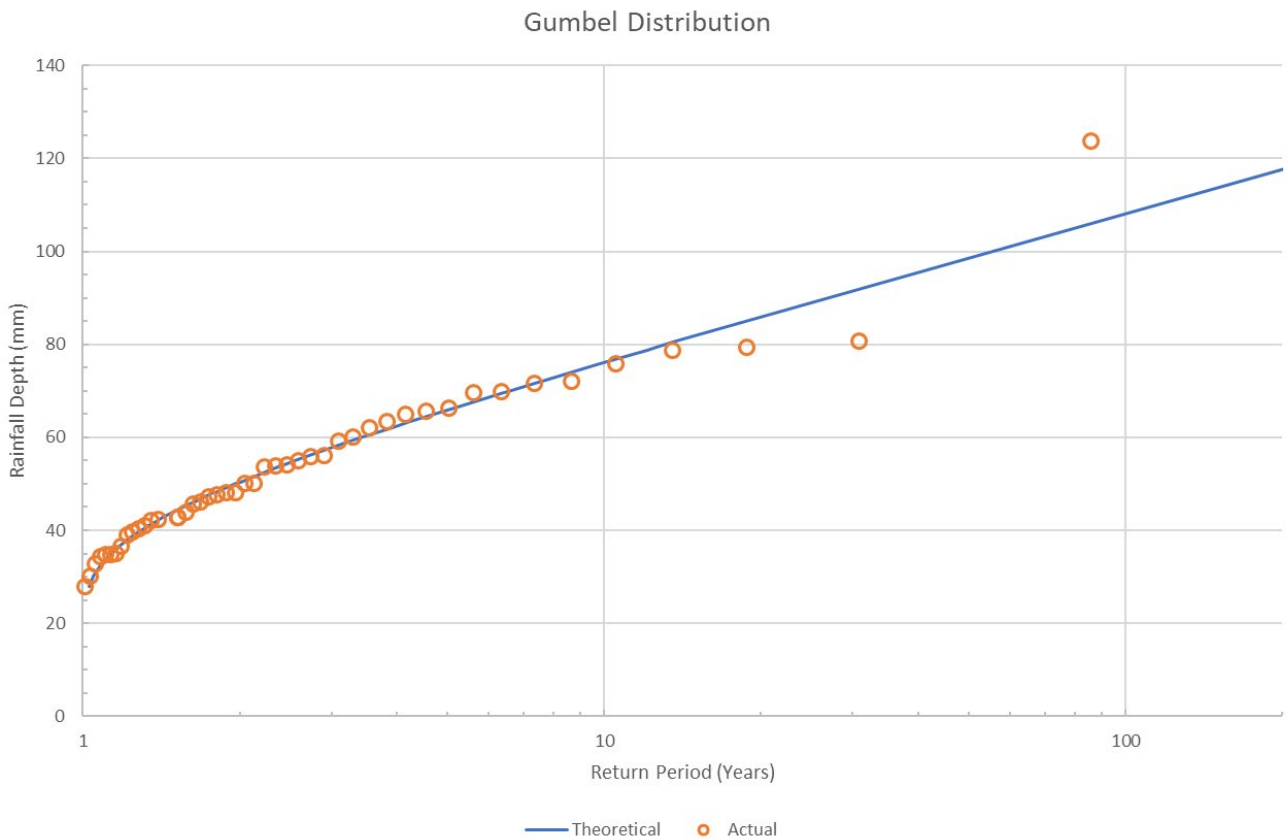
1986	65.6	11	0.2195	0.7805	4.6	1.4805	0.7965	4.9
2017	66.3	10	0.1987	0.8013	5.0	1.5319	0.8056	5.1
1967	69.6	9	0.1779	0.8221	5.6	1.7739	0.8439	6.4
2006	69.9	8	0.1571	0.8429	6.4	1.7959	0.8471	6.5
2000	71.6	7	0.1363	0.8637	7.3	1.9206	0.8637	7.3
1977	72.1	6	0.1155	0.8845	8.7	1.9572	0.8683	7.6
2009	75.8	5	0.0948	0.9052	10.6	2.2286	0.8979	9.8
2002	78.8	4	0.0740	0.9260	13.5	2.4486	0.9172	12.1
2014	79.4	3	0.0532	0.9468	18.8	2.4926	0.9206	12.6
2012	80.6	2	0.0324	0.9676	30.8	2.5806	0.9271	13.7
2004	123.7	1	0.0116	0.9884	85.9	5.7416	0.9968	**312.1**

$$Z\ Score = \frac{x - \mu}{\sigma}$$

		Return Period (Yr)	**Depth (mm)**
3.9019	0.98	**50**	**98.62**

Number of Obs (n) = 48

Min	28
Max	123.7
Average	53.2833
Std Dev	17.4875
Alpha	13.6350
mu	45.4132



Gumbel Distribution
140
120
100
80
60
40
20
0
Rainfall Depth (mm)
1
10
100
Return Period (Years)
Theoretical
Actual

Appendix F:
General Frequency Analysis Output – CFA

Rawdon Creek General Frequency Analysis

```
WSC STATION NO.=02HK008
WSC STATION NAME=Rawdon Creek Near West Huntingdon
-----------------------------------------------------------------------
HISTORIC INFORMATION:   TOTAL TIME SPAN=   38
                        CENSORING THRESHOLD=    20.000
                        HISTORIC PEAKS ABOVE THE THRESHOLD=    0
SEQ.NO.     YEAR   MON      FLOW
-------     ----   ---    --------
     1      1983     2       6.850
     2      1984     4      10.200
     3      1985     3       8.310
     4      1986     3       5.900
     5      1987     4       9.620
     6      1988     3      11.900
     7      1989     3       5.800
     8      1990     3       5.800
     9      1991     4       5.090
    10      1992     3      10.800
    11      1993     3       9.240
    12      1994     3       4.710
    13      1995     2       2.100

Press <RETURN> to continue
```

WSC STATION NO.=02HK008
WSC STATION NAME=Rawdon Creek Near West Huntingdon

SEQ.NO.	YEAR	MON	FLOW
14	1999	4	5.500
15	2000	2	4.530
16	2001	4	5.850
17	2002	6	10.100
18	2003	3	6.720
19	2004	3	7.240
20	2005	4	7.790
21	2006	12	5.680
22	2007	4	4.180
23	2008	4	13.400
24	2009	4	11.200
25	2010	1	9.940
26	2011	3	9.320
27	2012	3	5.160
28	2013	3	7.310
29	2014	4	13.000

Press <RETURN> to continue

WSC STATION NO.=02HK008
WSC STATION NAME=Rawdon Creek Near West Huntingdon

SEQ.NO.	YEAR	MON	FLOW
30	2015	10	4.770
31	2016	4	4.570
32	2017	5	12.900
33	2018	4	10.500
34	2019	3	9.870
35	2020	1	8.080
36	2021	3	5.610

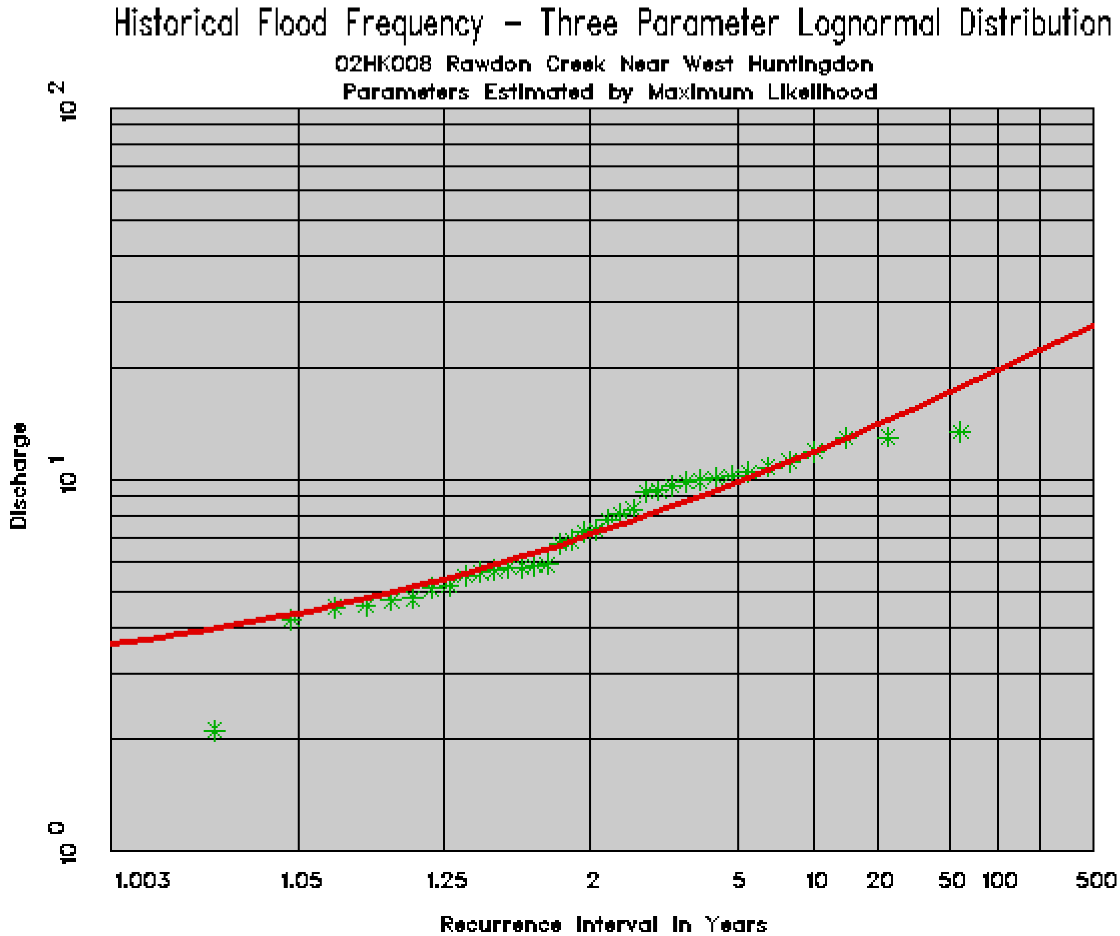
Press <RETURN> to continue

```
*** FREQUENCY ANALYSIS PROGRAM ***
    --- SAMPLE STATISTICS ---

WSC STATION NO.=02HK008
WSC STATION NAME=Rawdon Creek Near West Huntingdon
DRAINAGE AREA=     93.00
HISTORIC INFORMATION: TOTAL TIME SPAN=   38
                       CENSORING THRESHOLD=    20.000
                       HISTORIC PEAKS ABOVE THE THRESHOLD=    0
NUMBER OF OBSERVATIONS=   36

         X series   lnX series
MEAN        7.765       1.9763
S.D.        2.875        .4040
C.V.        .3703        .2044
C.S.        .2923       -.6365
C.K.       2.4586       4.0762

You should always check :
>  that the data are accurate
>  for historic information
>  that the data and historic information are up to date

Press <RETURN> to continue
```

```
*** FREQUENCY ANALYSIS PROGRAM ***

WSC STATION NO = 02HK008
WSC STATION NAME = Rawdon Creek Near West Huntingdon

HISTORIC INFORMATION:
TOTAL TIME SPAN =    38
                      CENSORING THRESHOLD =     20.000
                      HISTORIC PEAKS ABOVE THE THRESHOLD =     0

NUMBER OF OBSERVATIONS =    36

C.S. of lnX series =     -.6365
LOWER OUTLIER LIMIT of X =      2.485
NOTE:  1 LOW OUTLIER(S) DETECTED.

Do you want to alter the number of low outliers? : N
```

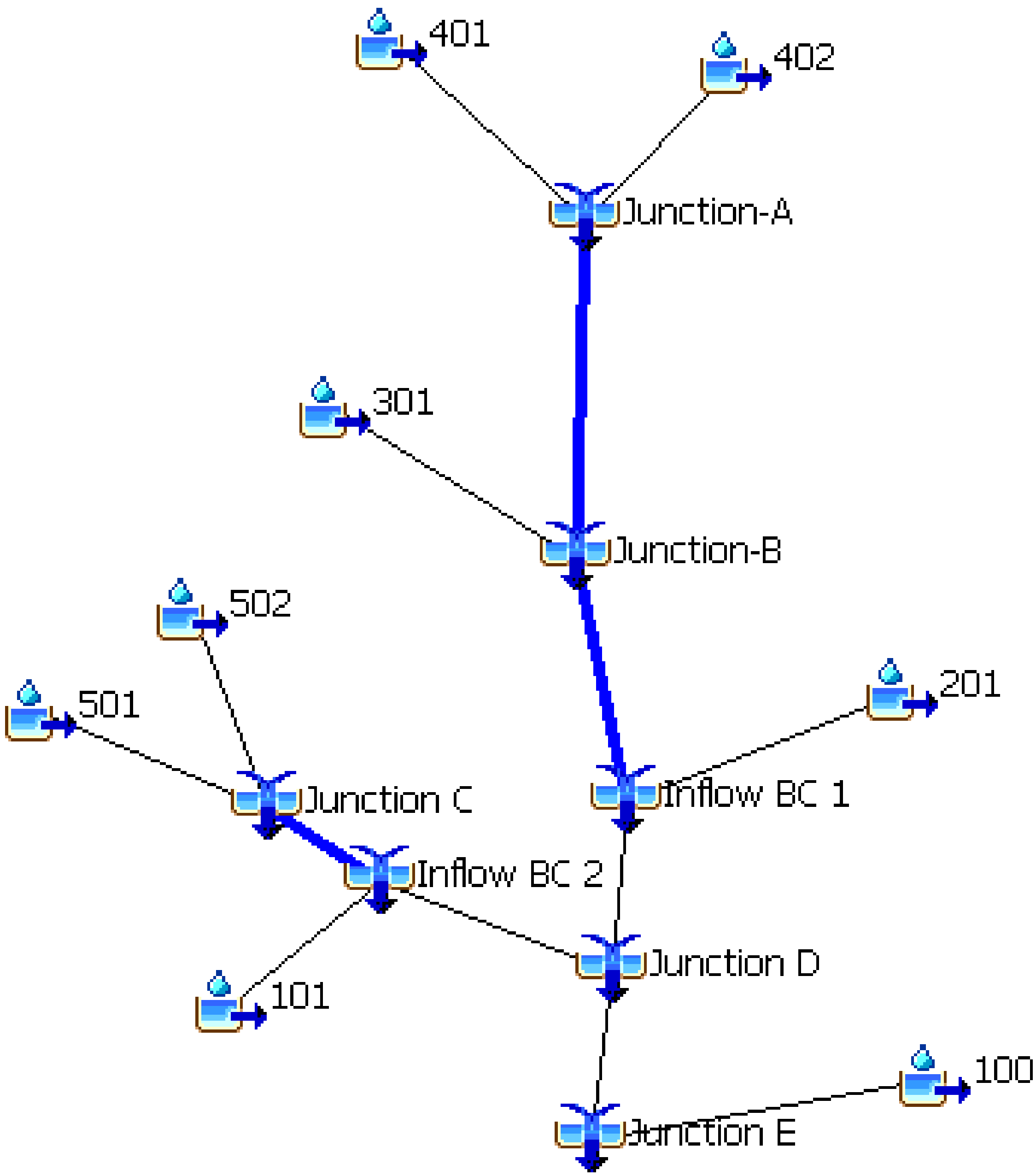
```
   HISTORICAL FREQUENCY ANALYSIS - THREE-PARAMETER LOGNORMAL DISTRIBUTION
   02HK008     Rawdon Creek Near West Huntingdon

                          SAMPLE STATISTICS
                 MEAN       S.D.        C.V       C.S.       C.K.
      X SERIES   7.765      2.875       .370       .292      2.459
   LN X SERIES   1.976       .404       .204      -.636      4.076

   X(MIN)=      2.100                         TOTAL SAMPLE SIZE=  36
   X(MAX)=     13.400                       NO. OF LOW OUTLIERS=   1
   LOWER OUTLIER LIMIT OF X=    2.485         NO. OF ZERO FLOWS=   0


             AFTER REMOVAL OF ZEROES AND/OR LOW OUTLIERS
                 MEAN       S.D.        C.V.       C.S.       C.K.
      X SERIES   7.927      2.746       .346       .432      2.283
   LN X SERIES   2.012       .349       .174       .039      1.987
LN(X-A) SERIES   1.503       .574       .382      -.216      2.112

Press <RETURN> to continue , <CTRL> P to obtain hard copy_
```

```
           SOLUTION OBTAINED VIA MAXIMUM LIKELIHOOD
     PARAMETERS OF THE 3LN WHICH DUPLICATES THE CONDITIONAL FUNCTION:
                  A=     2.708   M= 1.476   S=  .580

                        FLOOD FREQUENCY REGIME

                RETURN        EXCEEDANCE        FLOOD
                PERIOD        PROBABILITY

                 1.003           .997             3.60
                 1.050           .952             4.37
                 1.250           .800             5.39
                 2.000           .500             7.09
                 5.000           .200             9.84
                10.000           .100            11.9
                20.000           .050            14.1
                50.000           .020            17.1
               100.000           .010            19.6
               200.000           .005            22.2
               500.000           .002            26.0

Press <RETURN> to continue , <CTRL> P to obtain hard copy_
```



Historical Flood Frequency - Three Parameter Lognormal Distribution
02HK008 Rawdon Creek Near West Huntingdon
Parameters Estimated by Maximum Likelihood
Discharge
10 2
10 1
10 0
1.003
1.05
1.25
2
5
10
20
50
100
500
Recurrence Interval in Years

Appendix G:
HEC-HMS Schematic


401
402
Junction-A
301
Junction-B
502
501
201
Junction C
Inflow BC 1
Inflow BC 2
Junction D
101
100
Junction E

Appendix H:
LTC Memo – September 2021 Storm Event

Quick Event Summary - September 22-23, 2021

Janet Noyes <janet.noyes@ltc.on.ca>

Fri 9/24/2021 10:24 AM

To: Gage Comeau <gage.comeau@ltc.on.ca>; Rhonda Bateman <rhonda.bateman@ltc.on.ca>

The rainfall totals for this two-day event can be seen in the attached clip from our Daily Planning Cycle spreadsheet. Of note:

- two rain gauges are not working (Rawdon & Shelter Valley Creek)
- Butler (Proctor) Creek gauge appears to be double what it should - calculation factor entered wrong in logger perhaps?
- LTC manual rain gauge - read at 8:30 am each day: 26.8 (Sept 22); 94.4 (Sept 23); 16.6 (Sept 24) for a total of 137.8 mm
- Trenton data indicates 85.2 mm over Sept 22-23 - still waiting for Sept 24 to be included.

I think I'm comfortable saying that we saw between **75 mm and 120 mm** of rain (3 to 5 inches) across our watershed over the 48-hour period.

Regarding streamflows:

- Only 2 local streams reached the 2-yr (bankfull) flow - just over - not close to 5-yr flow:
  - Cold Creek peaked at 24.265 m3/s (2-yr is 24 m3/s)
  - Salt Creek peaked at 15.667 m3/s (2-yr is 14 m3/s)
- 5 of our streams reached half of the 2-year:
  - Shelter Valley Creek peaked at 9.802 m3/s (2-yr is 19 m3/s)
  - Butler Creek peaked at 3.556 m3/s (2-yr is 5.4 m3/s)
  - Mayhew Creek peaked at 5.26 m3/s (2-yr is 6.7 m3/s)
  - Burnley Creek peaked at 9.952 m3/s (2-yr is 14 m3/s)
  - Trout Creek peaked at 4.683 m3/s (2-yr is 7 m3/s)
- 2 of our creeks did not even reach half of the 2-yr - in northeast area with lots of wetland storage and exhibits more drought conditions:
  - Rawdon Creek peaked at 3.528 m3/s (2-yr is 12 m3/s)
  - Hoards/Squires Creek peaked at 3.553 m3/s (2-yr is 17 m3/s)

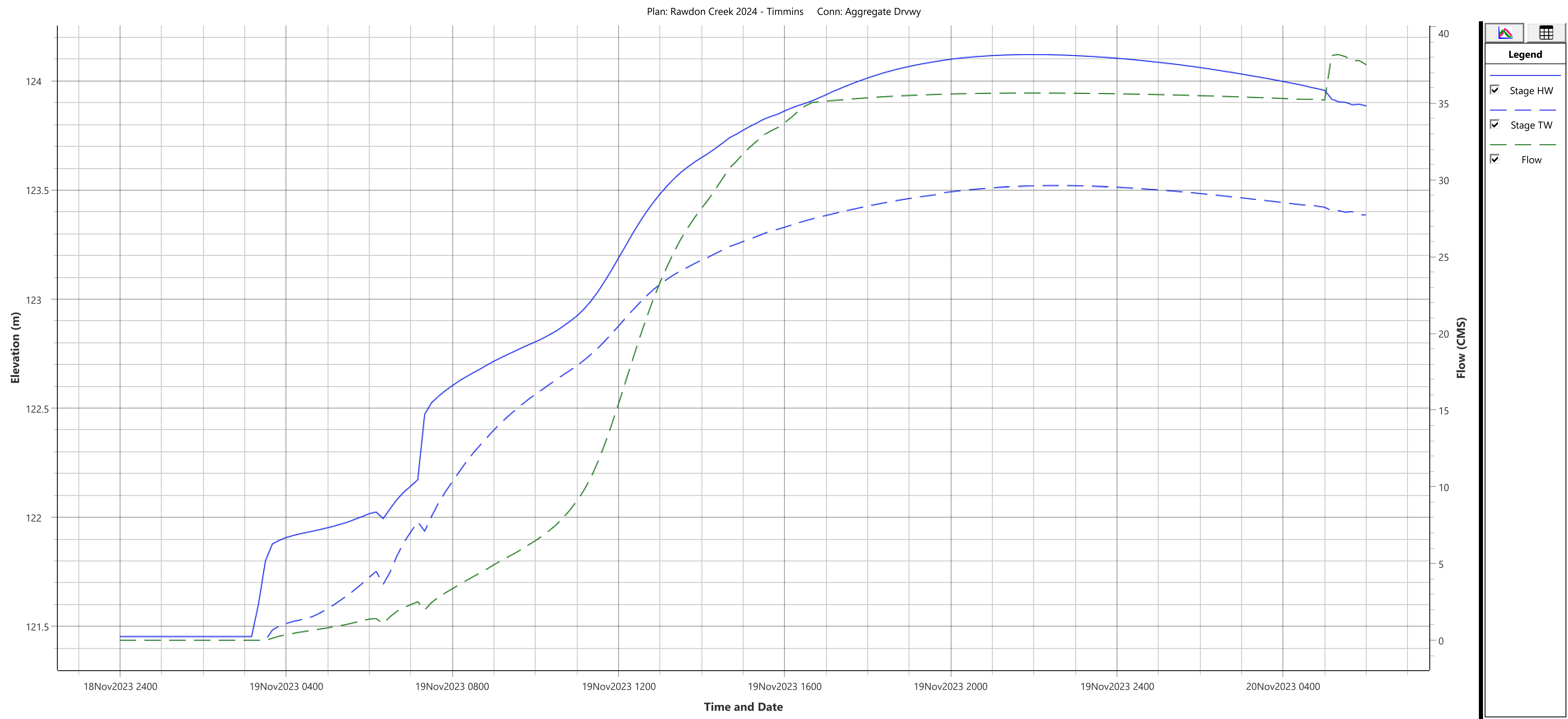
Lower Trent CA		Watershed Risk Assessment					
	Date: Sept 24, 2021	Time: 09:00		Prepared By: JKN			
Flow (cms)	**Current**	**24 hrs ago**	2 Yr Flow(OFAT)	**10 yr Flow (OFAT)**	**Trend**	**CONCERN**	
Cold	21.958	24.265	24	37	**Down**	CONCERN	
Mill	7.815	9.344	14	22	**Down**	CONCERN	
Rawdon	2.594	3.528	12	19	**Down**		
Butler	0.404	3.556	6.8	12	**Down**		
Mayhew	2.448	5.26	7	10	**Down**		
Shelter Valley	1.791	9.802	19	36	**Down**		
Salt	5.402	15.667	14	21	**Down**		
Squires	3.515	2.206	17	28	**Up**		
Trout	4.022	4.683	7	11	**Down**	CONCERN	
Healey Falls	137	142	**300 cms concern**		**Down**		
Stage (m)	**Current**	**24 Hrs ago**	**48 hrs ago**	**Trend**	**Last 24 hours Rise/Fall cm**		
Upper Ross	113.606	113.565	113.474	**Up**	4.1		
Lower Ross	110.771	110.667	110.507	**Up**	10.4	**111.6 concern**	
P-Boom		113.674	113.602	#VALUE!	#VALUE!	**114.2 concern**	#VALUE!
Harwood		186.838	186.751	#VALUE!	#VALUE!	**186.9 concern**	#VALUE!
Precipitation (mm)	**This 24 hrs**	**last 24 hrs**	**last 48 hrs**	**Total for 48 hours**			
Trenton		41	44.2	85.2			
Butler		101.1	121.9	223			
Mill		41.8	34.6	76.4			
Rawdon		0	0	0			
Cold		52.6	58.8	111.4			
Trout		71	48.8	119.8			
Squires		37	39.2	76.2			
Salt		65.6	40.8	106.4			
Shelter Valley		0	0	0			

Janet

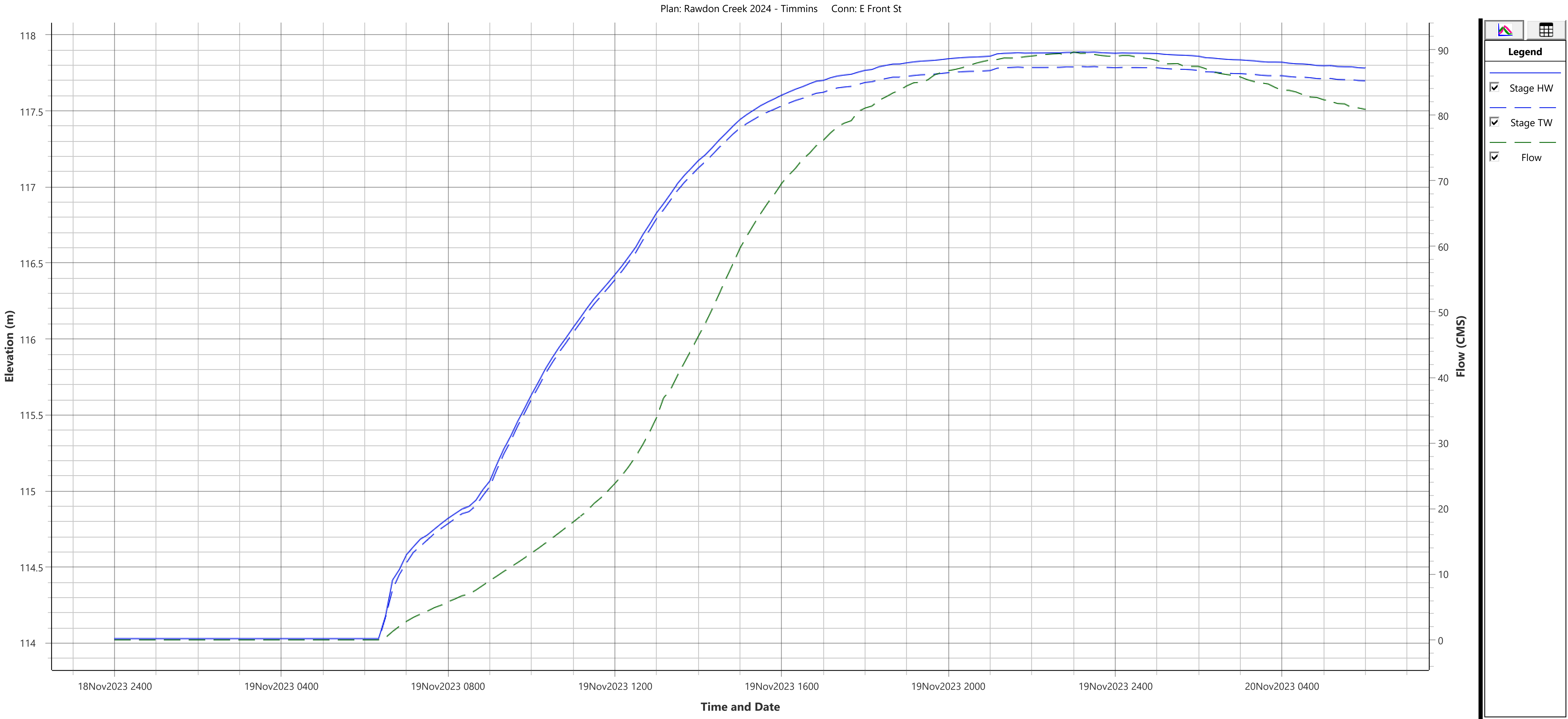
Janet Noyes, P.Eng.
Manager, Development Services & Water Resources
Lower Trent Conservation
613.394.3915 x211
janet.noyes@ltc.on.ca

**COVID-19 Notice: Lower Trent Conservation staff remain available to serve you virtually or by phone. To ensure your continued safety, our office is not open to the public at this time.

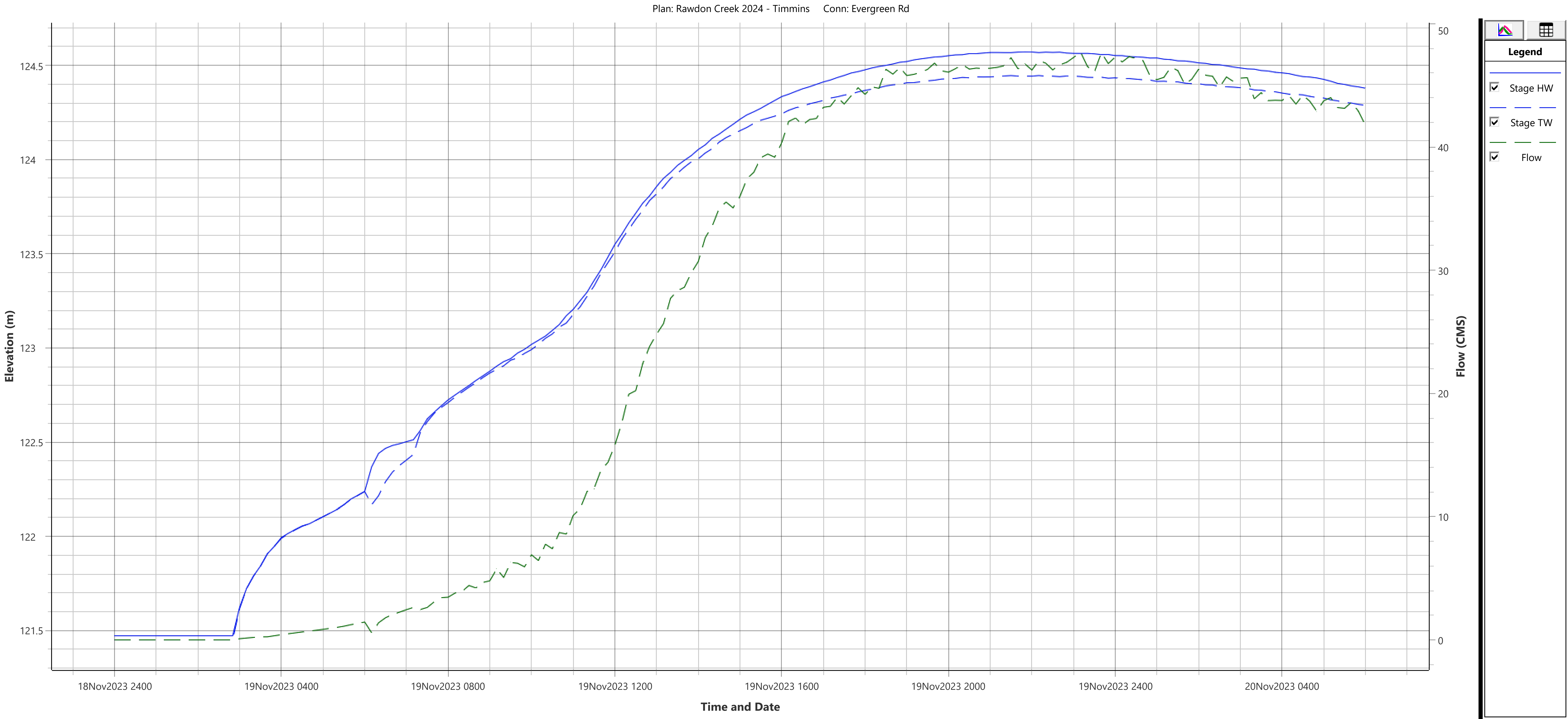
Appendix I:
Bridge Crossings Stage and Flow Hydrographs

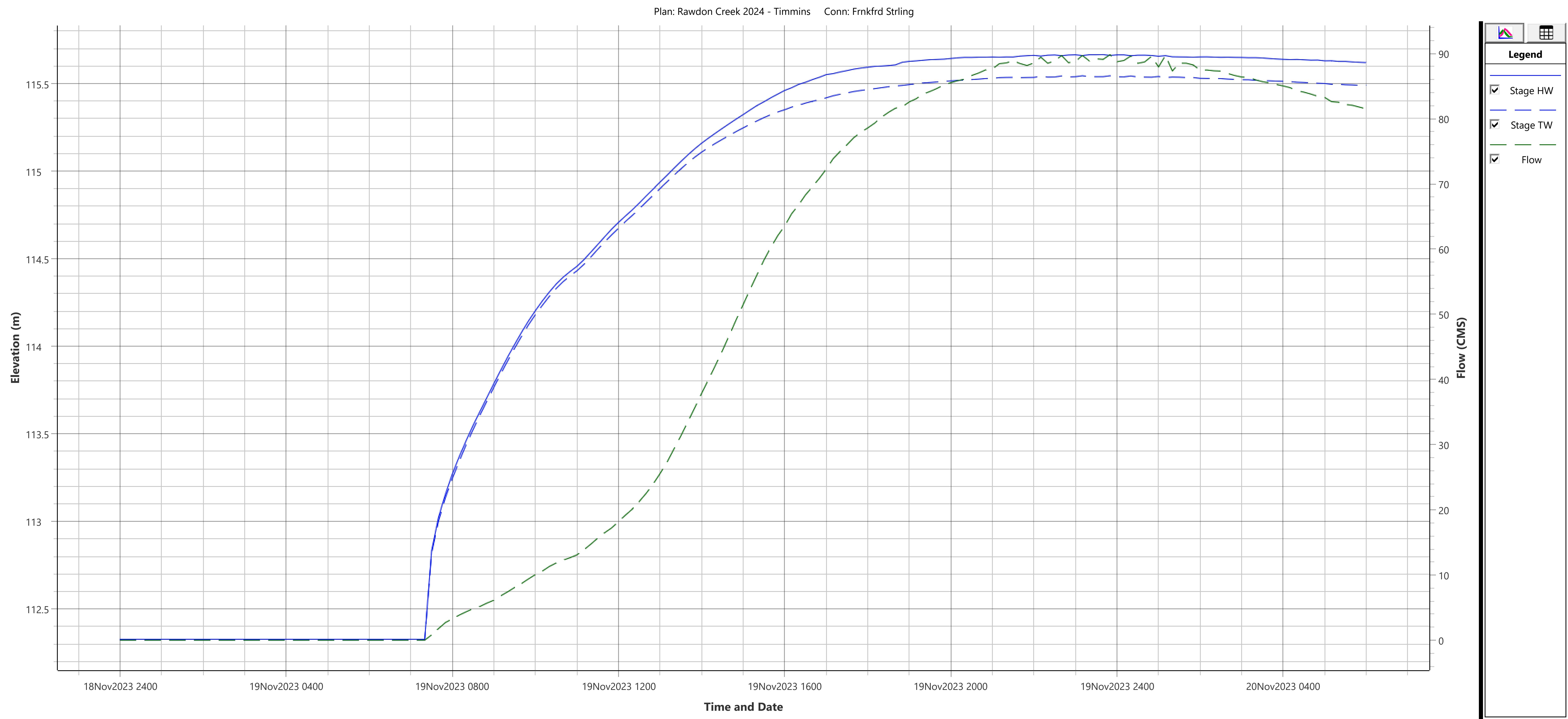


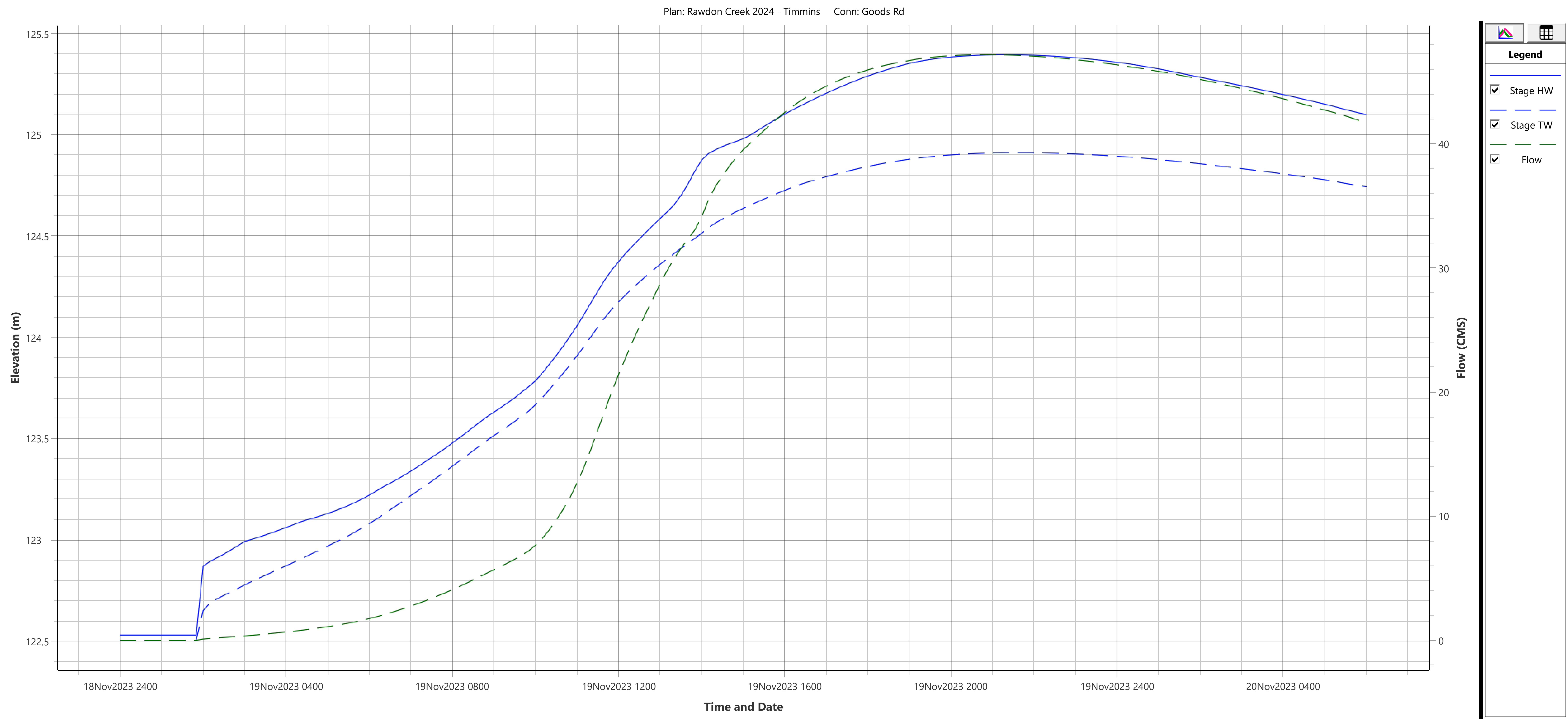
Plan: Rawdon Creek 2024 - Timmins   Conn: Aggregate Drvwy
Elevation (m)
124
123.5
123
122.5
122
121.5
Flow (CMS)
40
35
30
25
20
15
10
5
0
18Nov2023 2400
19Nov2023 0400
19Nov2023 0800
19Nov2023 1200
19Nov2023 1600
19Nov2023 2000
19Nov2023 2400
20Nov2023 0400
Time and Date
Legend
Stage HW
Stage TW
Flow



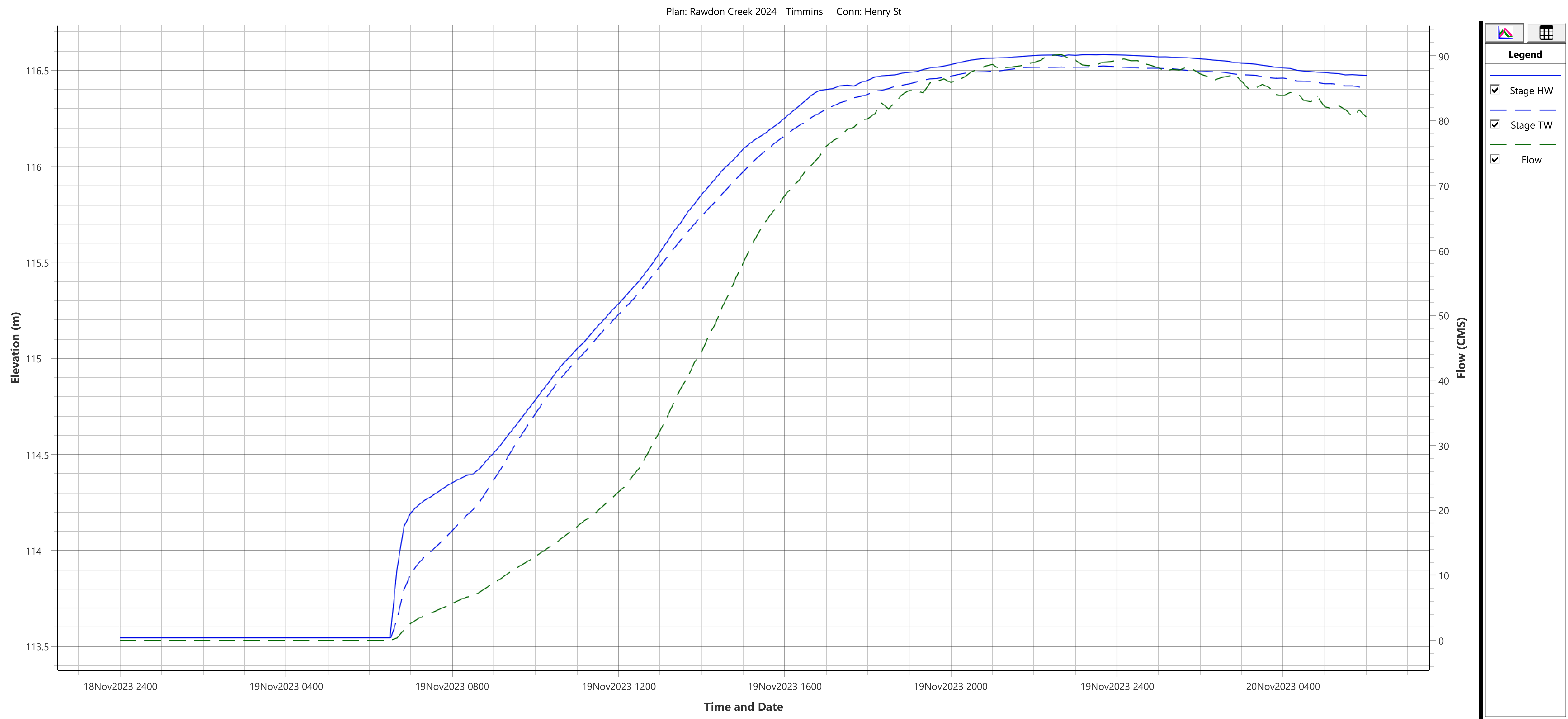
Plan: Rawdon Creek 2024 - Timmins
Conn: E Front St
Elevation (m)
Flow (CMS)
Time and Date
118
117.5
117
116.5
116
115.5
115
114.5
114
90
80
70
60
50
40
30
20
10
0
18Nov2023 2400
19Nov2023 0400
19Nov2023 0800
19Nov2023 1200
19Nov2023 1600
19Nov2023 2000
19Nov2023 2400
20Nov2023 0400
Legend
Stage HW
Stage TW
Flow

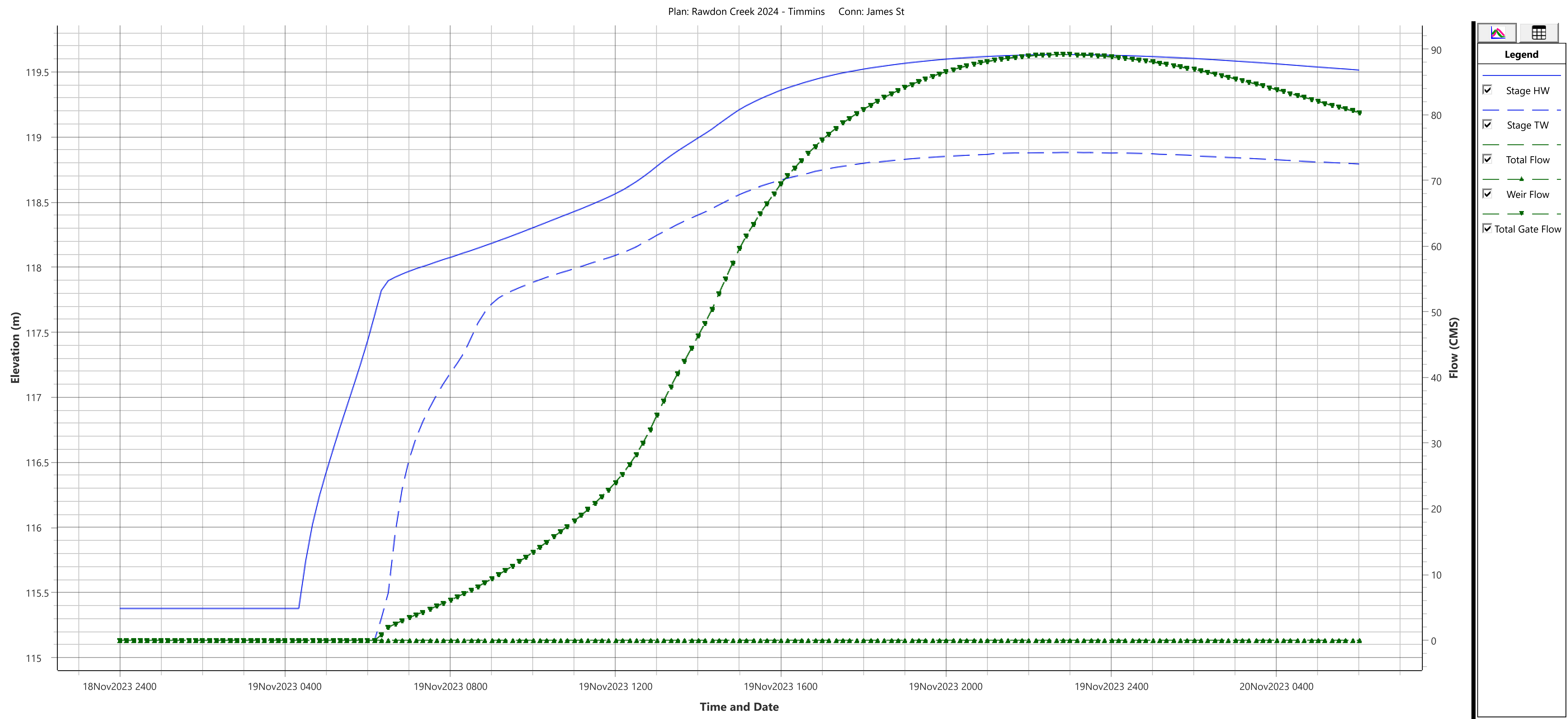

Plan: Rawdon Creek 2024 - Timmins   Conn: Evergreen Rd
Elevation (m)
124.5
124
123.5
123
122.5
122
121.5
Flow (CMS)
50
40
30
20
10
0
18Nov2023 2400
19Nov2023 0400
19Nov2023 0800
19Nov2023 1200
19Nov2023 1600
19Nov2023 2000
19Nov2023 2400
20Nov2023 0400
Time and Date
Legend
Stage HW
Stage TW
Flow


Plan: Rawdon Creek 2024 - Timmins Conn: Frnkfrd Strling
Elevation (m)
115.5
115
114.5
114
113.5
113
112.5
Flow (CMS)
90
80
70
60
50
40
30
20
10
0
18Nov2023 2400
19Nov2023 0400
19Nov2023 0800
19Nov2023 1200
19Nov2023 1600
19Nov2023 2000
19Nov2023 2400
20Nov2023 0400
Time and Date
Legend
Stage HW
Stage TW
Flow

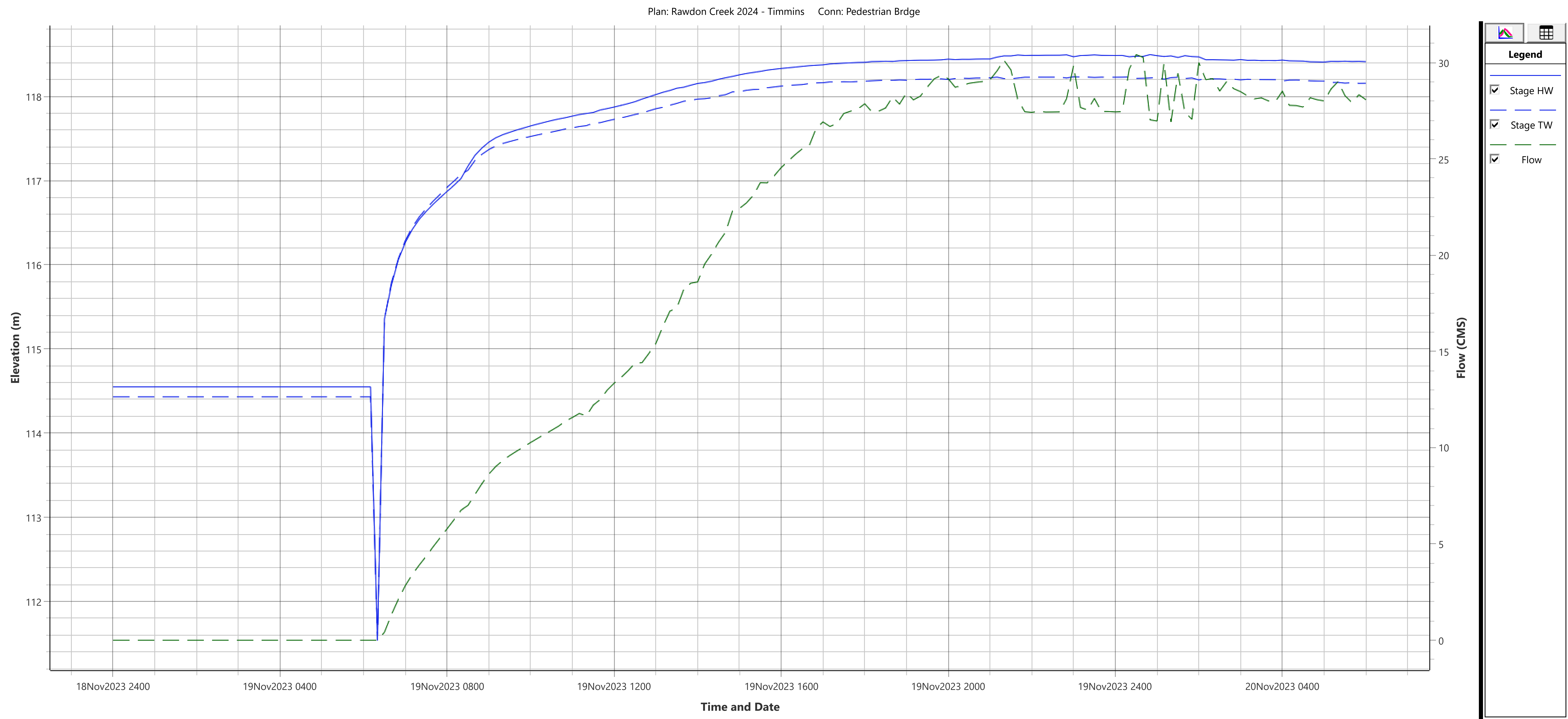


Plan: Rawdon Creek 2024 - Timmins Conn: Goods Rd
Elevation (m)
125.5
125
124.5
124
123.5
123
122.5
Flow (CMS)
40
30
20
10
0
18Nov2023 2400
19Nov2023 0400
19Nov2023 0800
19Nov2023 1200
19Nov2023 1600
19Nov2023 2000
19Nov2023 2400
20Nov2023 0400
Time and Date
Legend
Stage HW
Stage TW
Flow

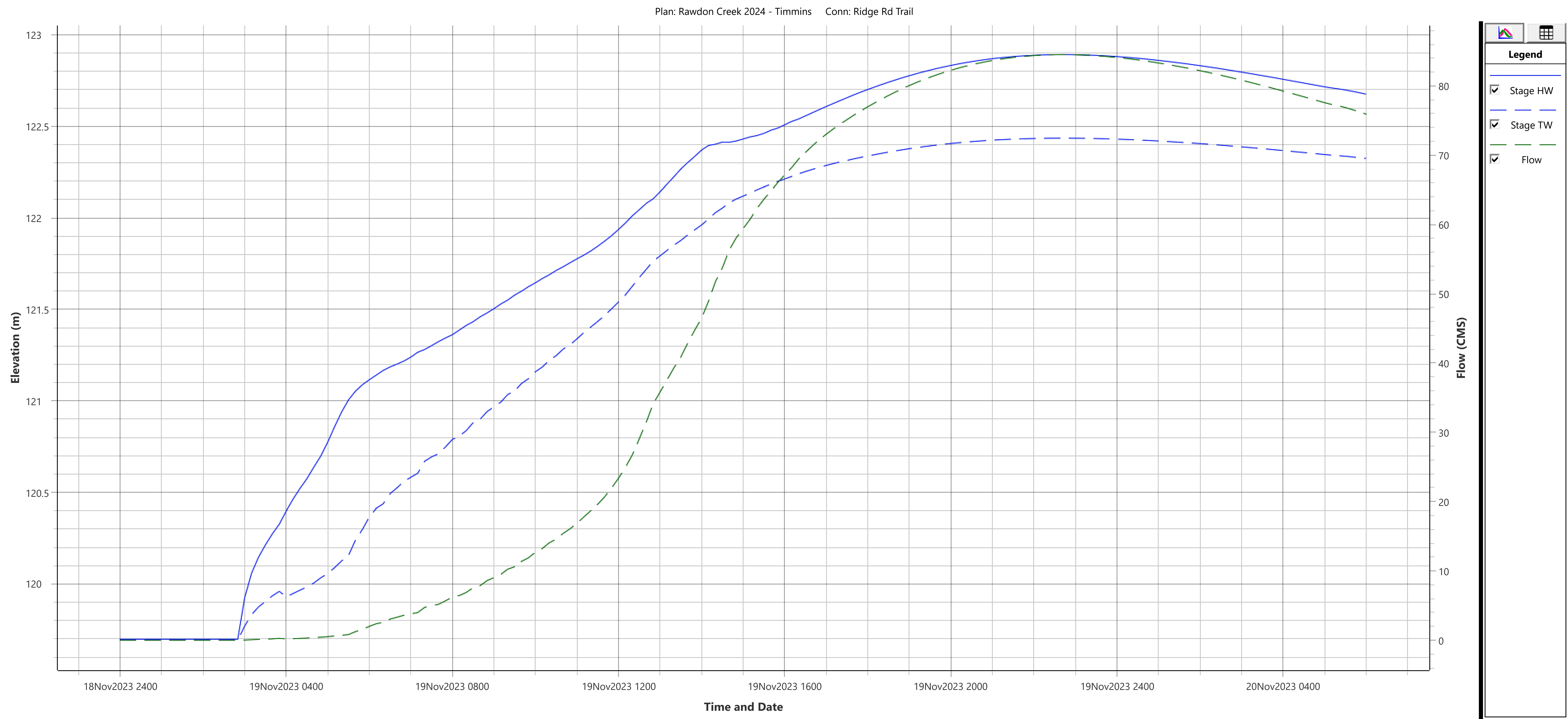

Plan: Rawdon Creek 2024 - Timmins   Conn: Henry St
Elevation (m)
116.5
116
115.5
115
114.5
114
113.5
Flow (CMS)
90
80
70
60
50
40
30
20
10
0
18Nov2023 2400
19Nov2023 0400
19Nov2023 0800
19Nov2023 1200
19Nov2023 1600
19Nov2023 2000
19Nov2023 2400
20Nov2023 0400
Time and Date
Legend
Stage HW
Stage TW
Flow



Plan: Rawdon Creek 2024 - Timmins   Conn: James St
Elevation (m)
Flow (CMS)
Time and Date
18Nov2023 2400
19Nov2023 0400
19Nov2023 0800
19Nov2023 1200
19Nov2023 1600
19Nov2023 2000
19Nov2023 2400
20Nov2023 0400
115
115.5
116
116.5
117
117.5
118
118.5
119
119.5
0
10
20
30
40
50
60
70
80
90
Legend
Stage HW
Stage TW
Total Flow
Weir Flow
Total Gate Flow



Plan: Rawdon Creek 2024 - Timmins
Conn: Pedestrian Brdge
Elevation (m)
Flow (CMS)
Time and Date
18Nov2023 2400
19Nov2023 0400
19Nov2023 0800
19Nov2023 1200
19Nov2023 1600
19Nov2023 2000
19Nov2023 2400
20Nov2023 0400
112
113
114
115
116
117
118
0
5
10
15
20
25
30
Legend
Stage HW
Stage TW
Flow

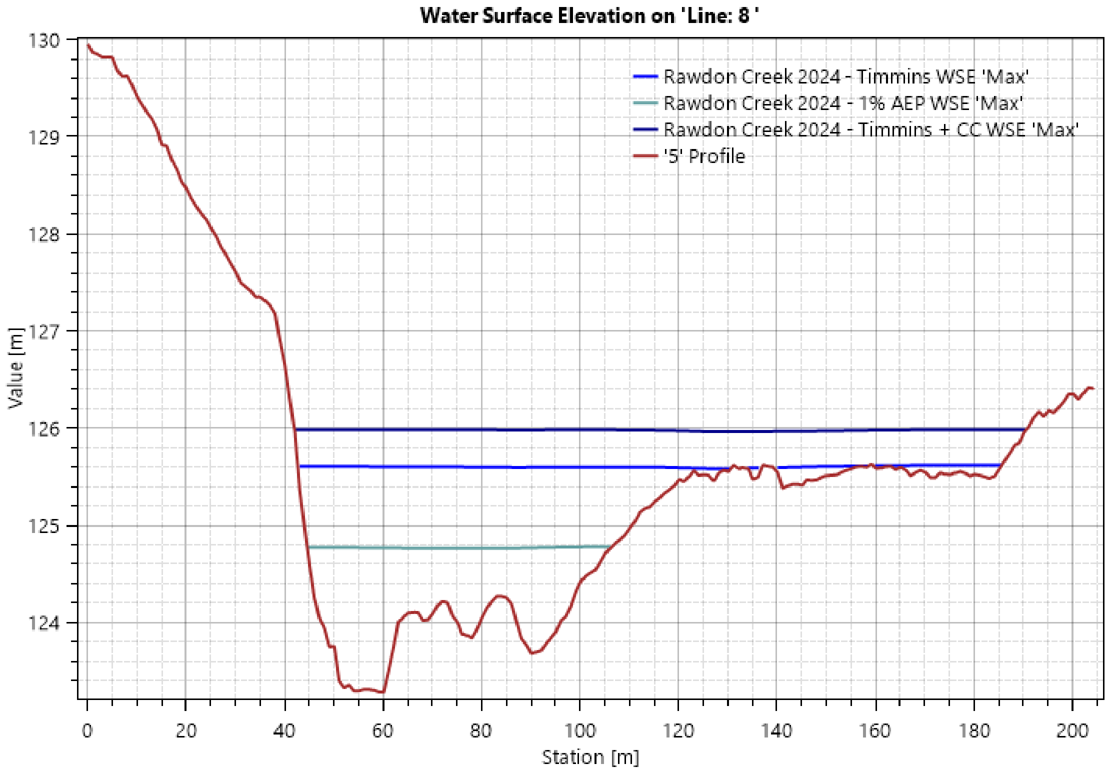


Plan: Rawdon Creek 2024 - Timmins Conn: Ridge Rd Trail
Elevation (m)
123
122.5
122
121.5
121
120.5
120
Flow (CMS)
80
70
60
50
40
30
20
10
0
18Nov2023 2400
19Nov2023 0400
19Nov2023 0800
19Nov2023 1200
19Nov2023 1600
19Nov2023 2000
19Nov2023 2400
20Nov2023 0400
Time and Date
Legend
Stage HW
Stage TW
Flow

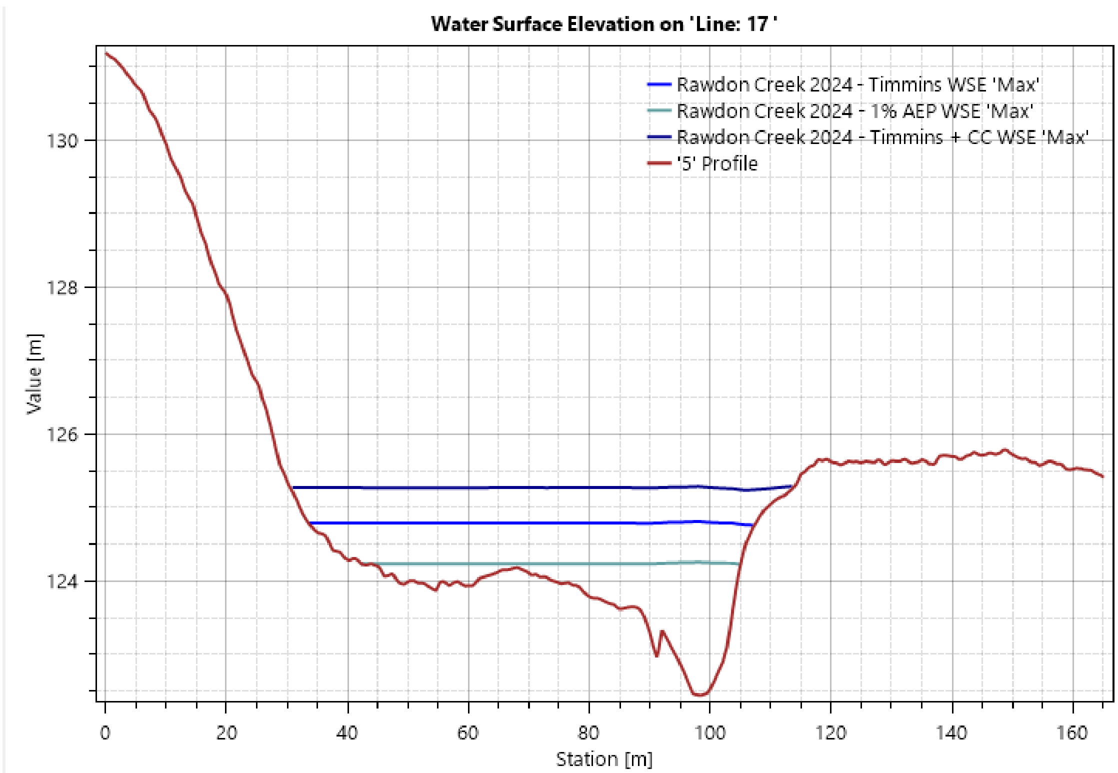
Appendix J:
Cross Section WSEL Plots for 100, Timmins, Timmins + Climate Change

Section Location Map for Cross Section WSEL Plots

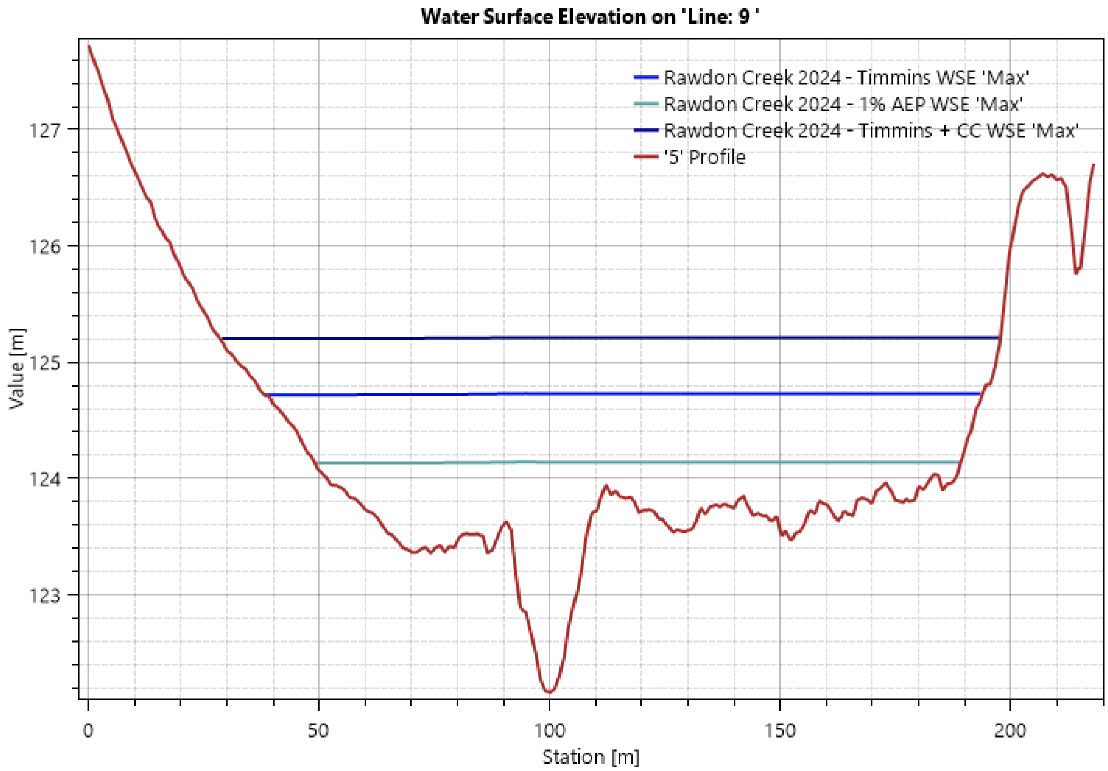
Section Location Map for Cross Section WSEL Plots



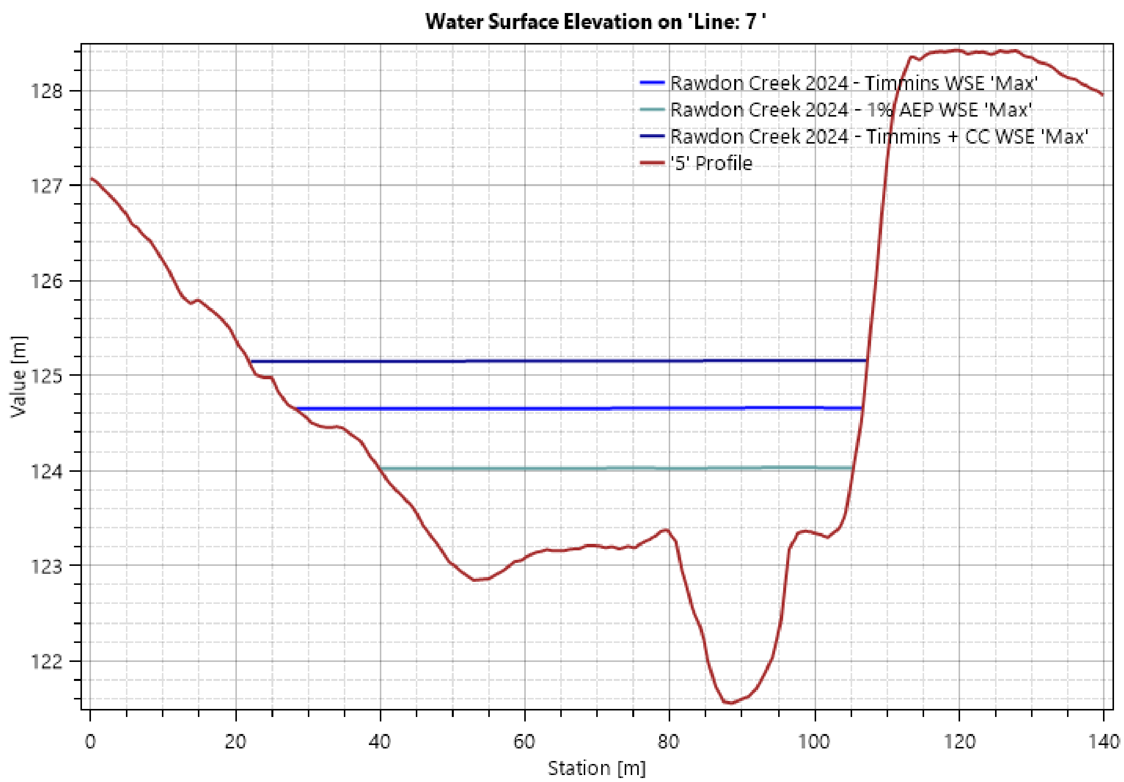
Section 1, Station 6750



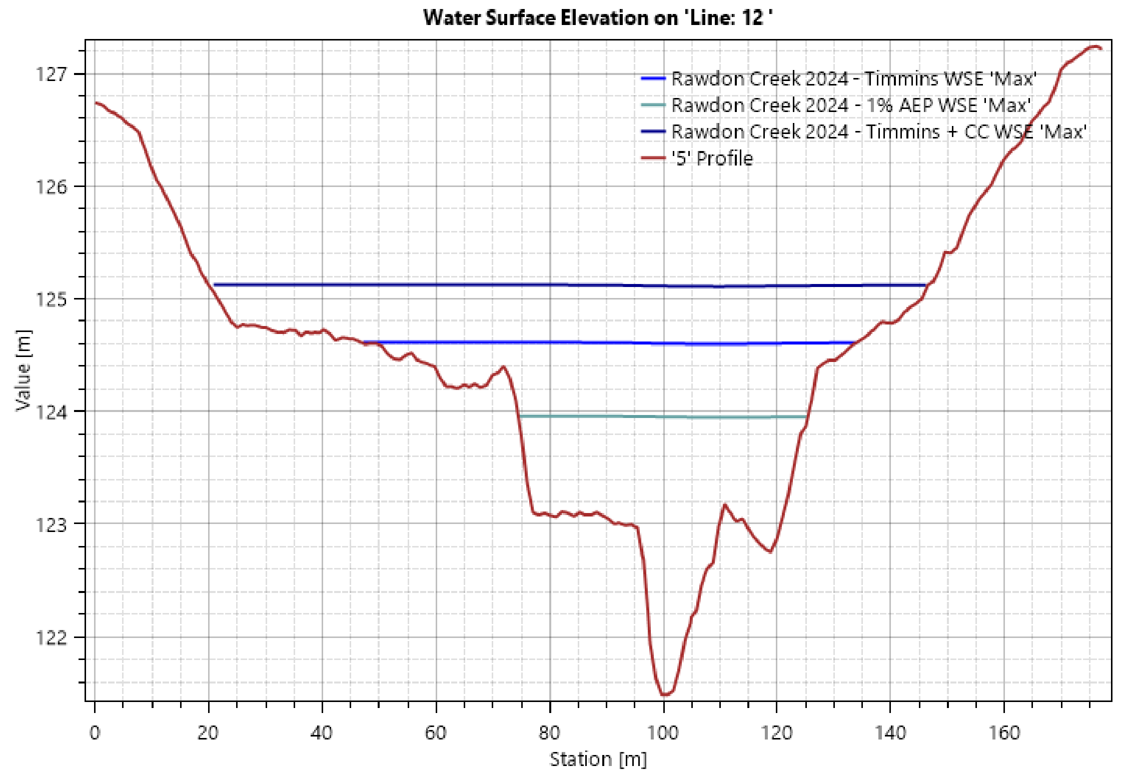
Section 2, Station 6585



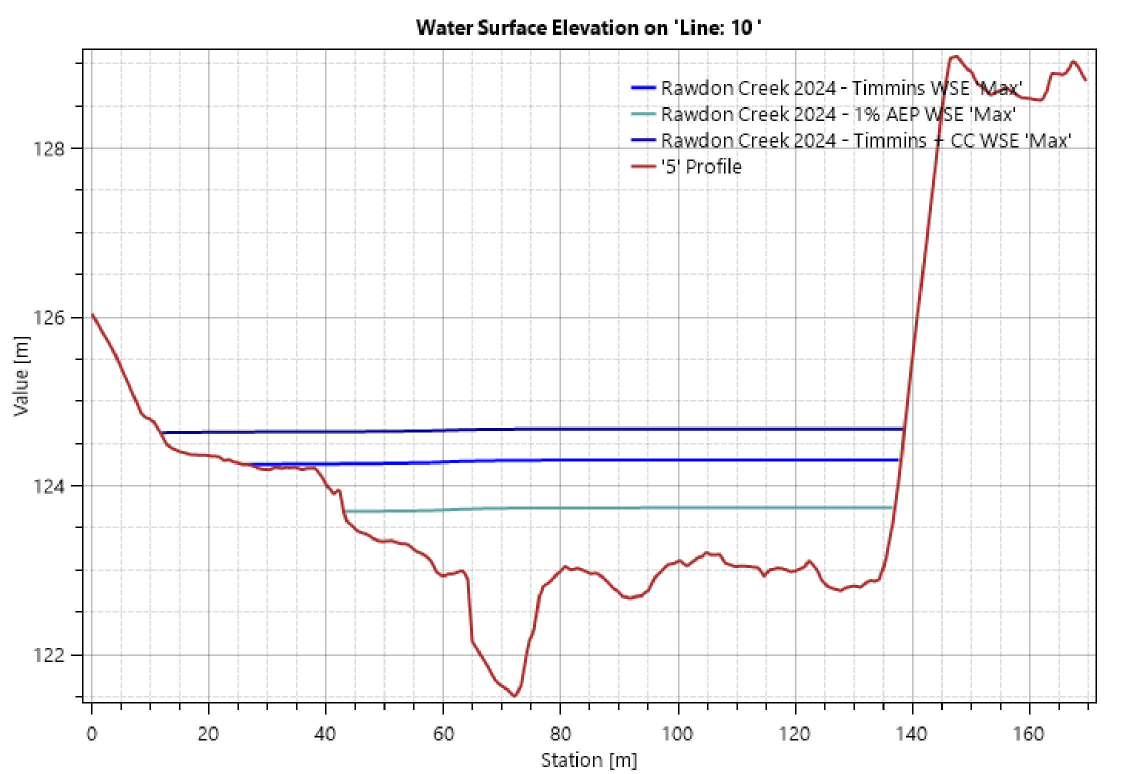
Section 3, Station 6451



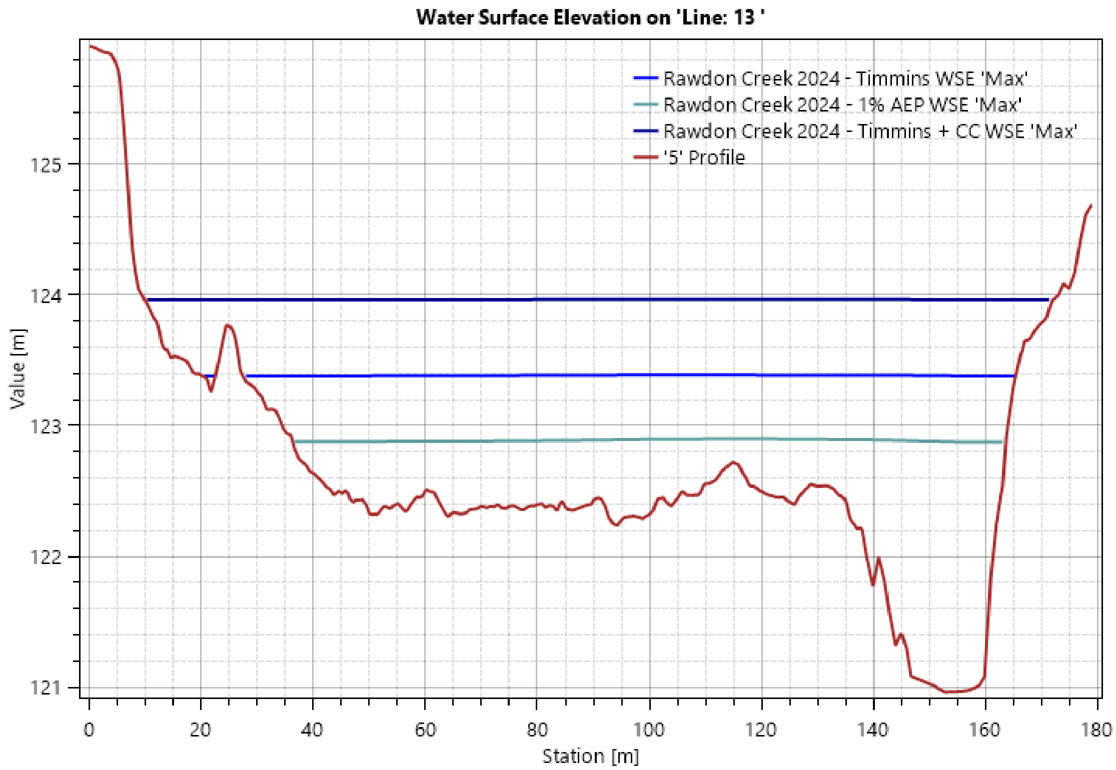
Section 4, Station 5993



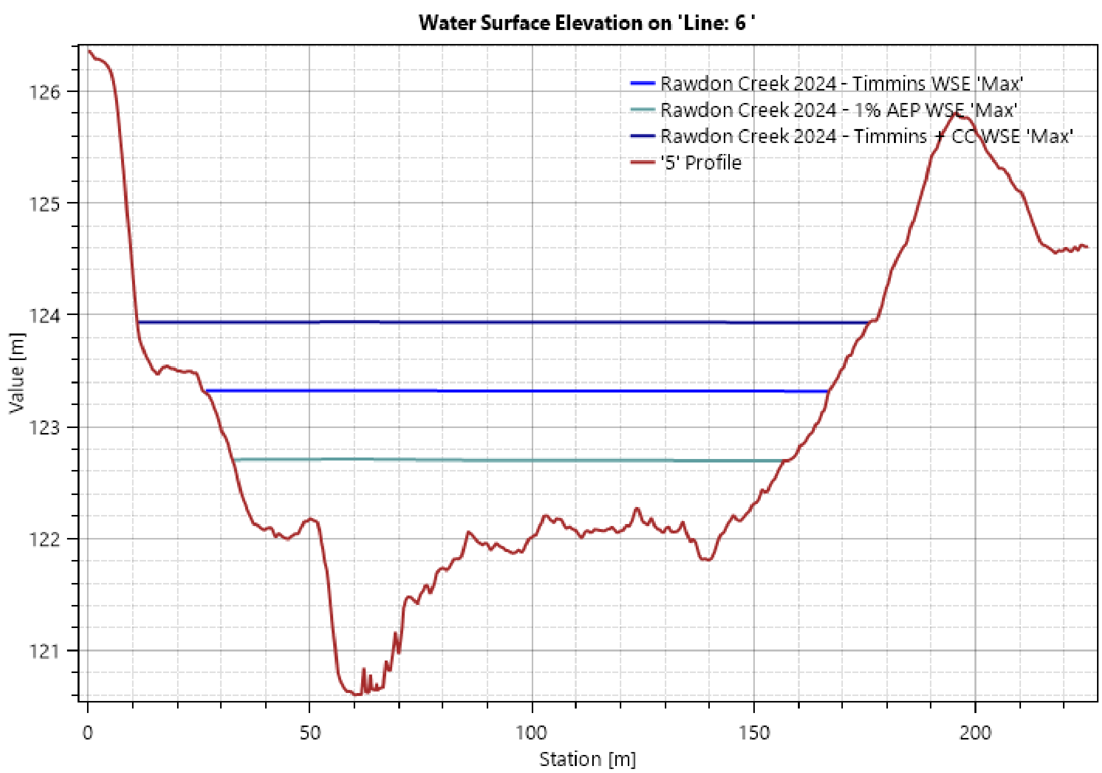
Section 5, Station 5772



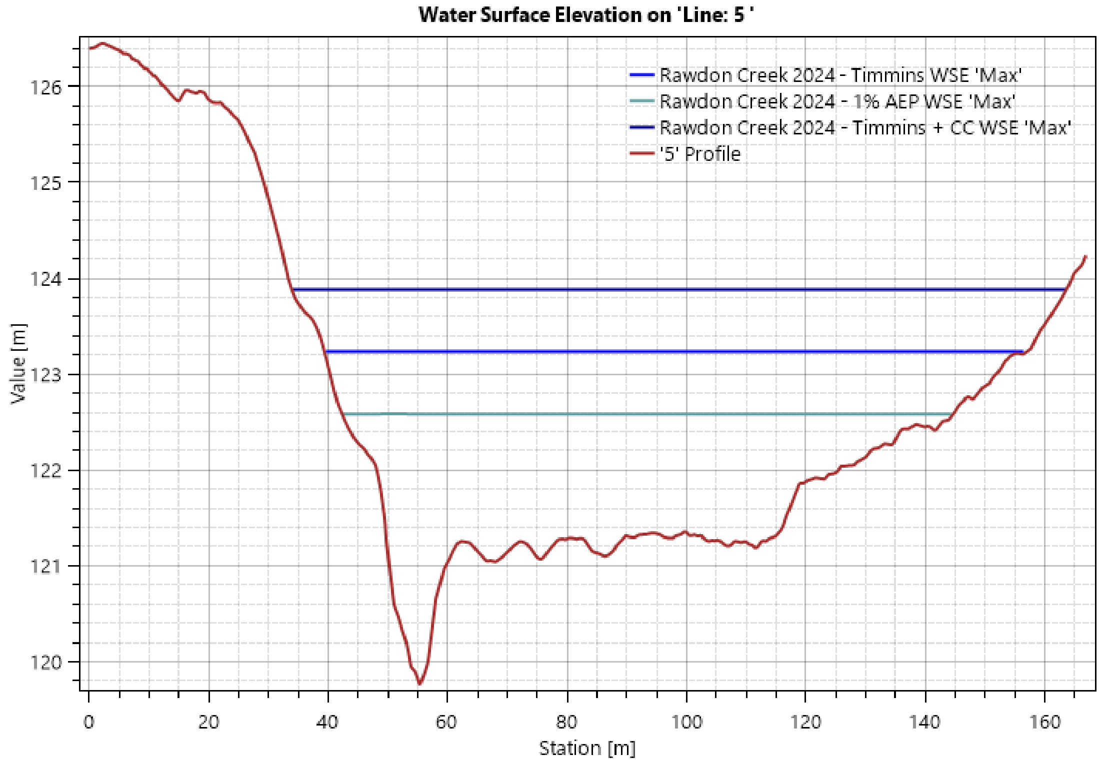
Section 6, Station 5617



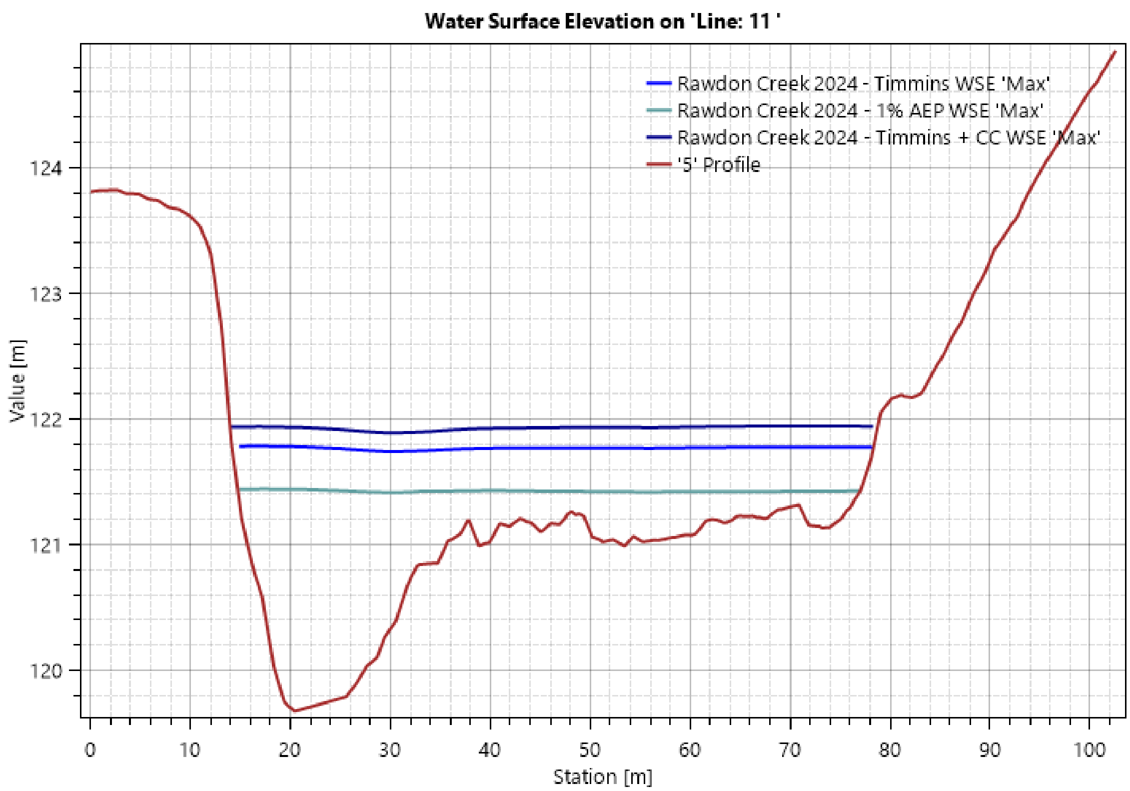
Section 7, Station 5169



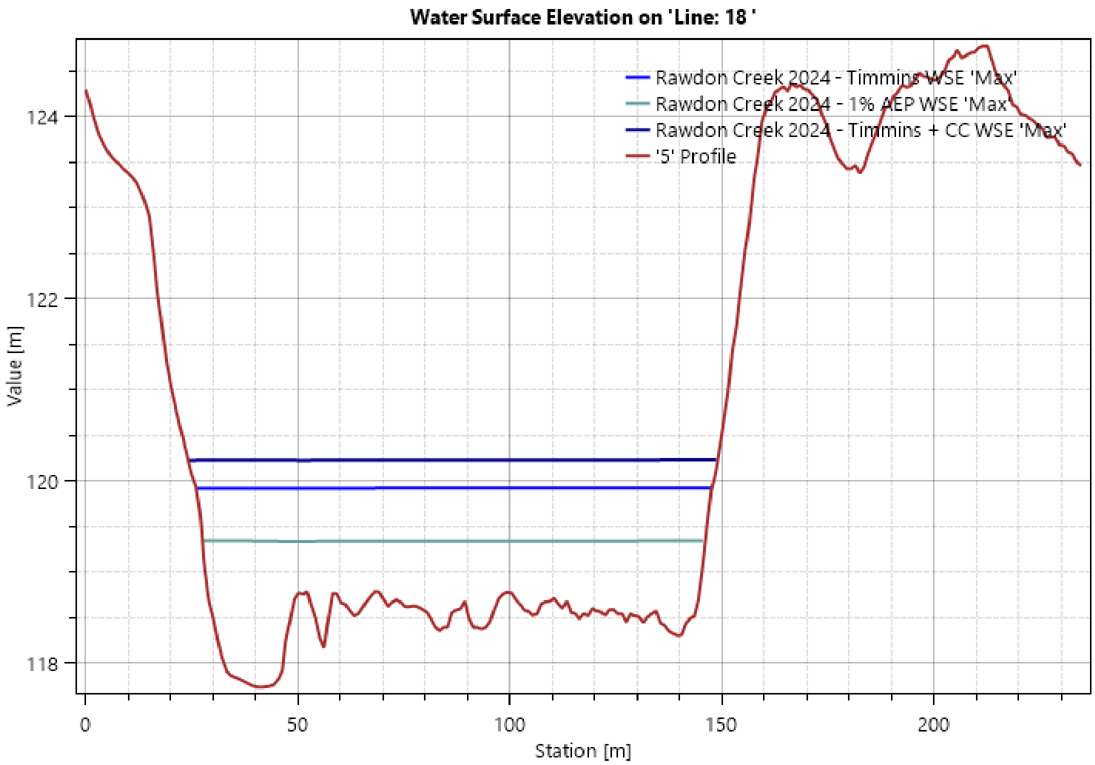
Section 8, Station 4840



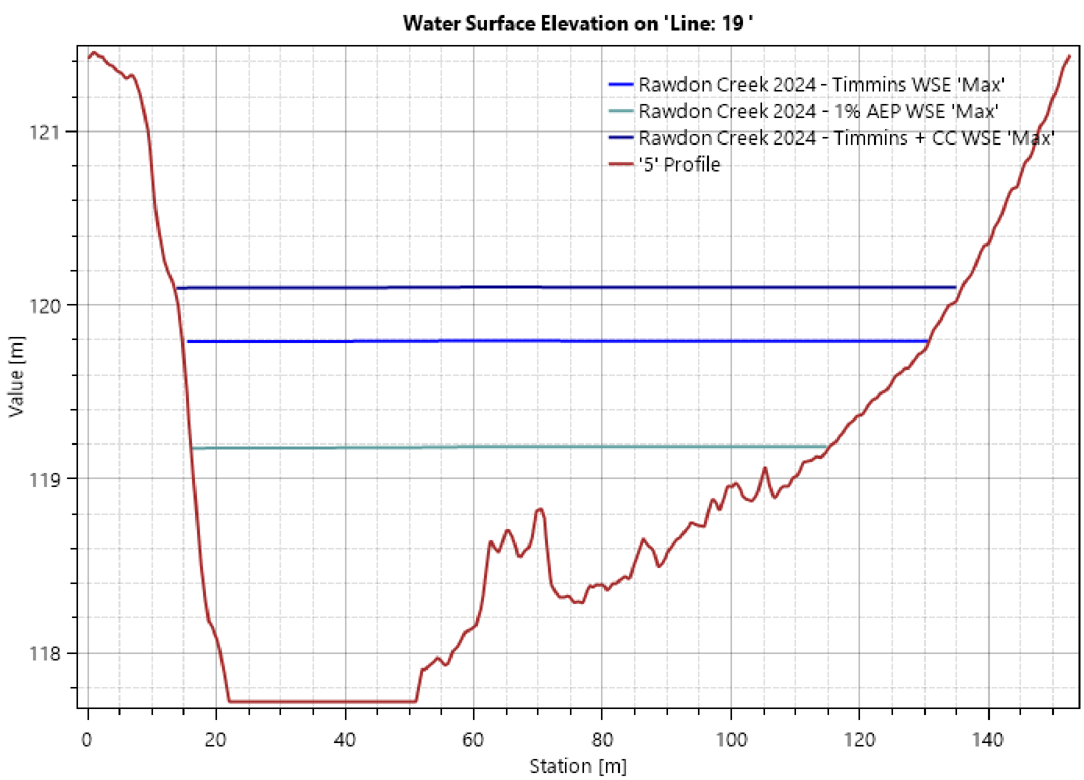
Section 9, Station 4134



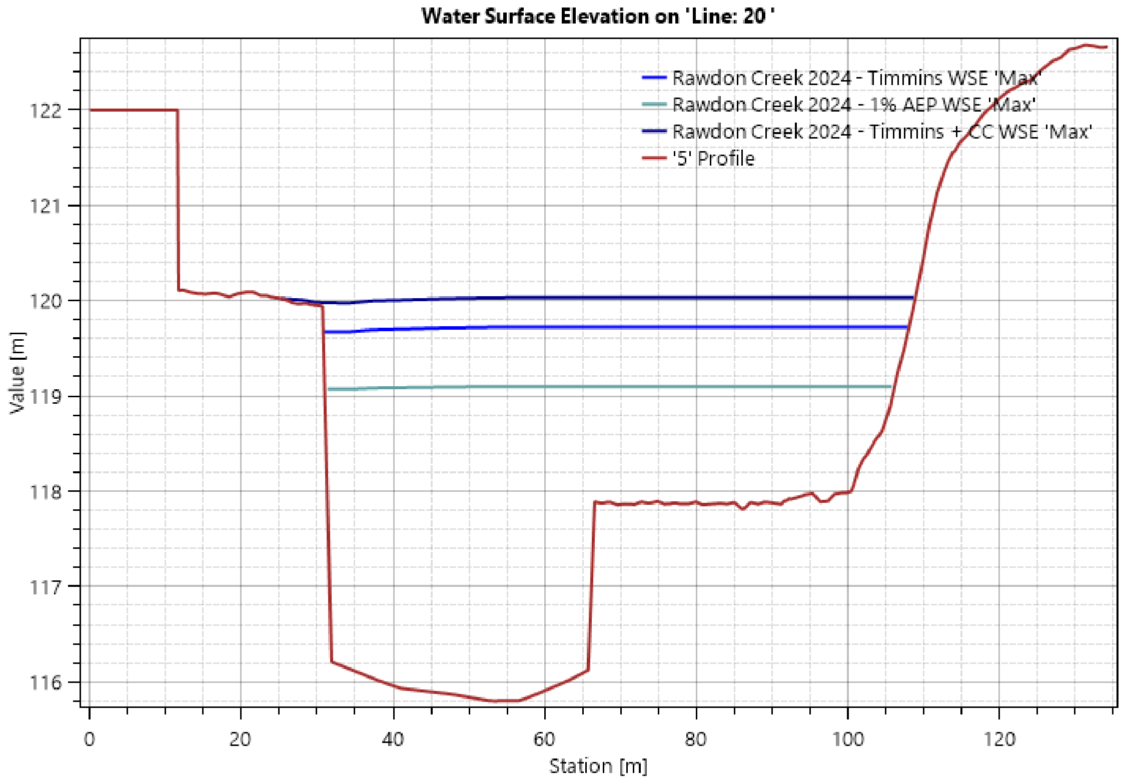
Section 10, Station 3641



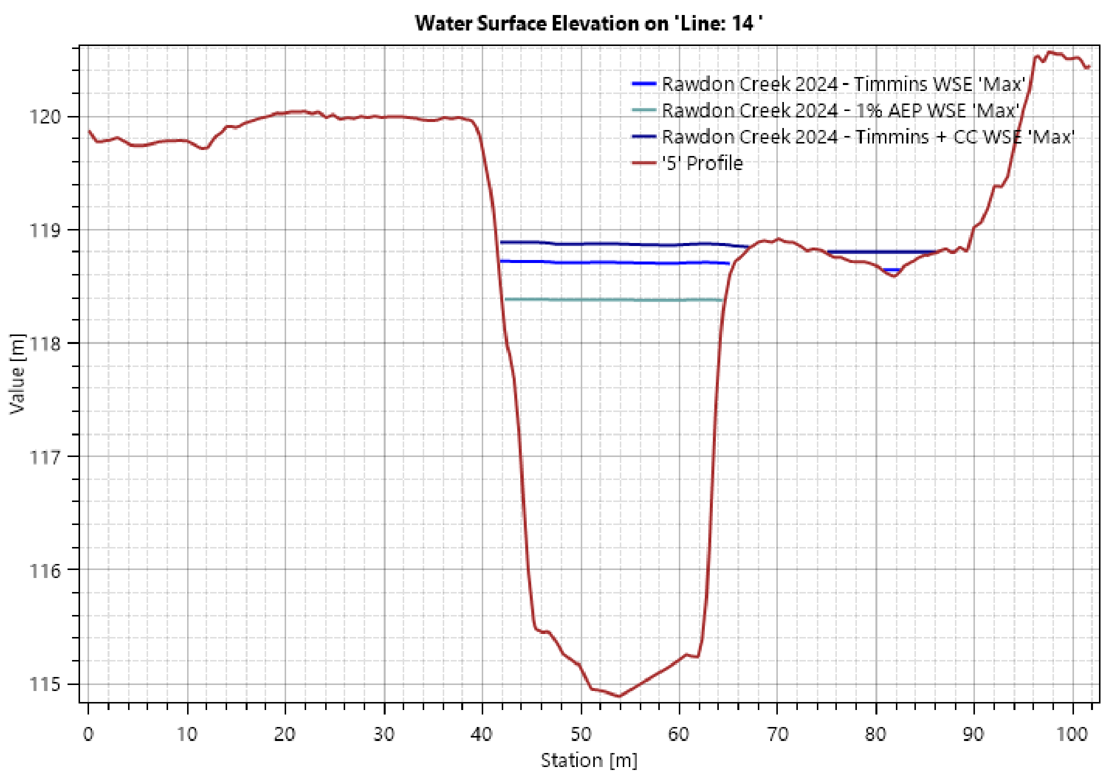
Section 11, Station 3162



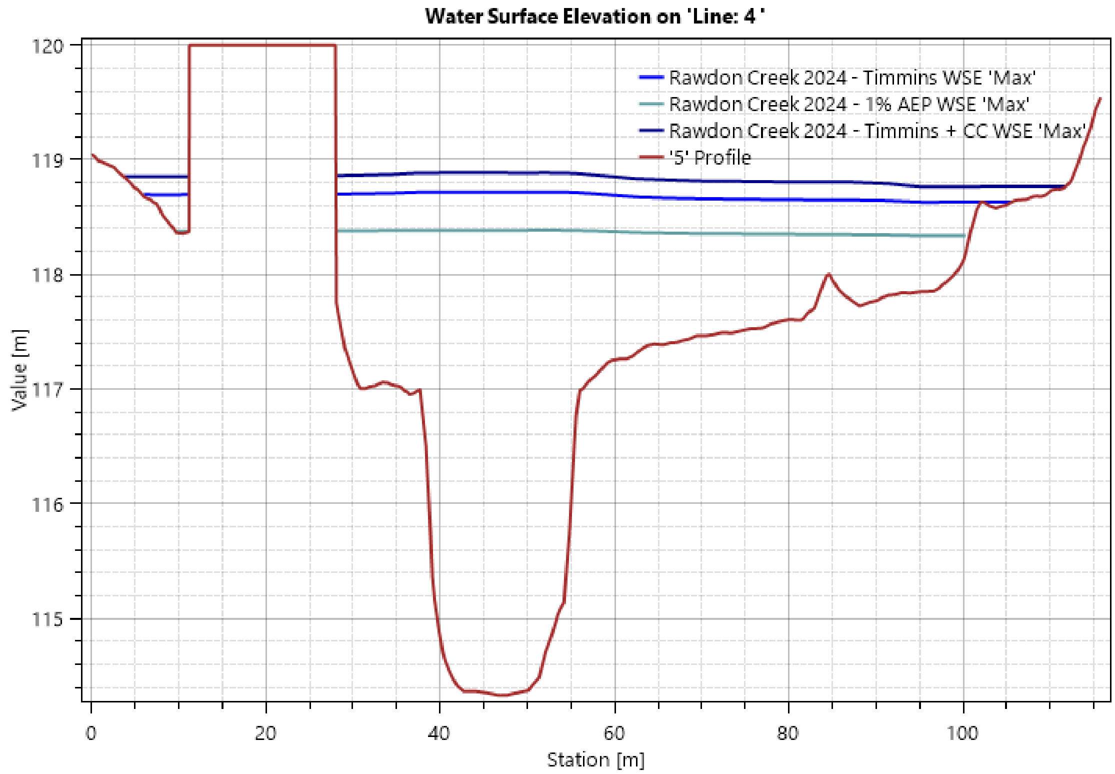
Section 12, Station 2570



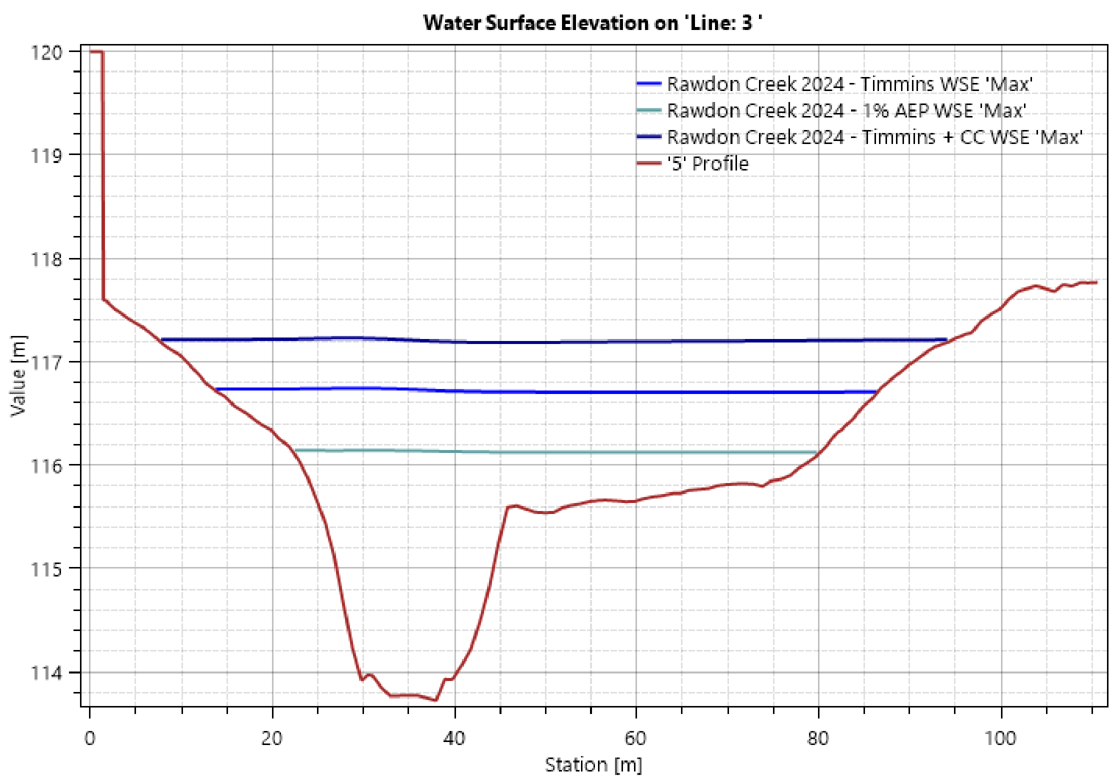
Section 13, Station 2357



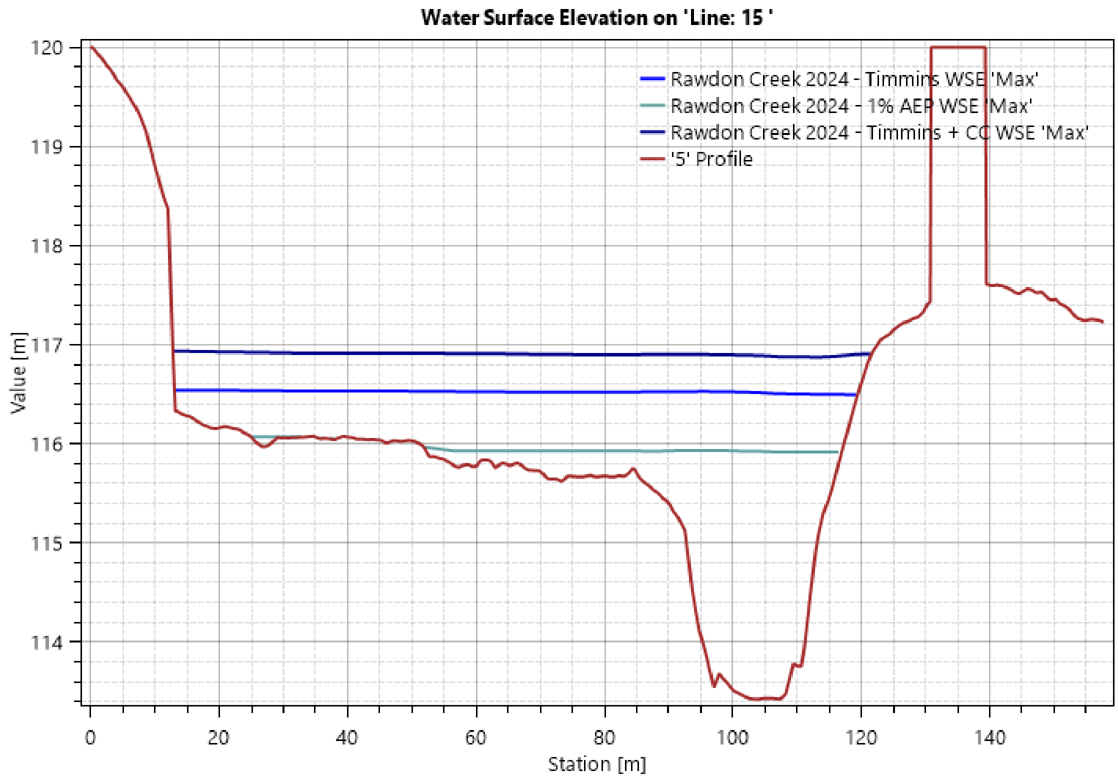
Section 14, Station 2292



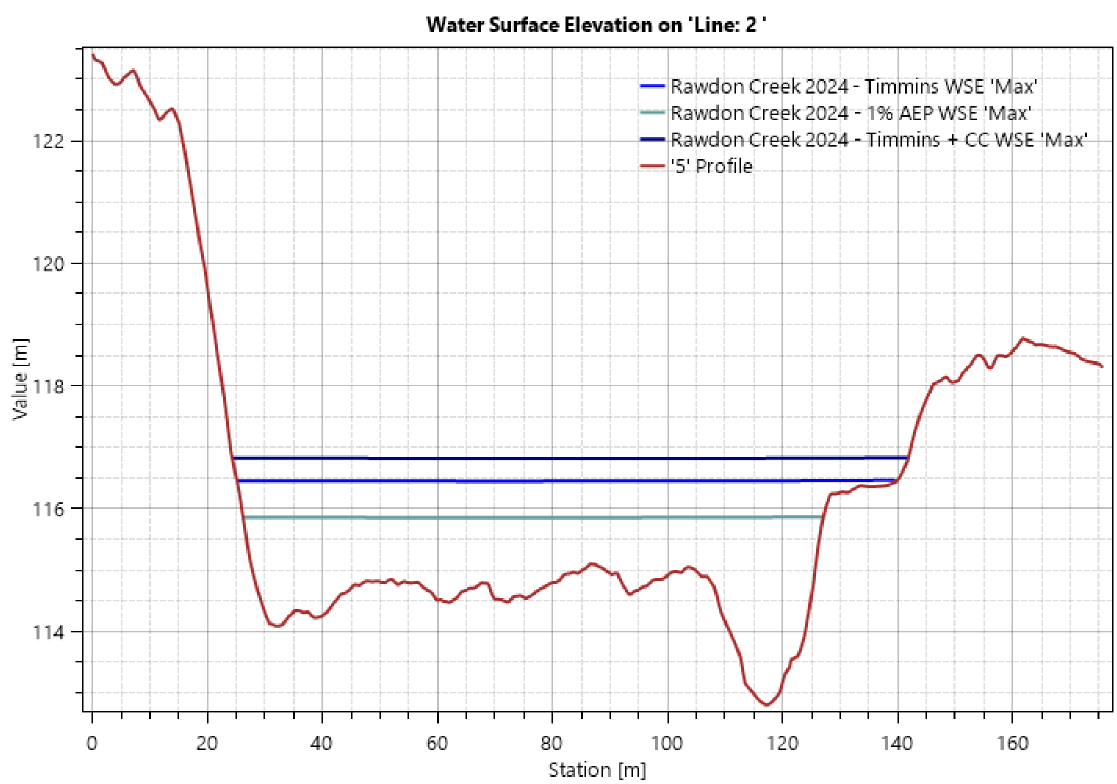
Section 15, Station 2237



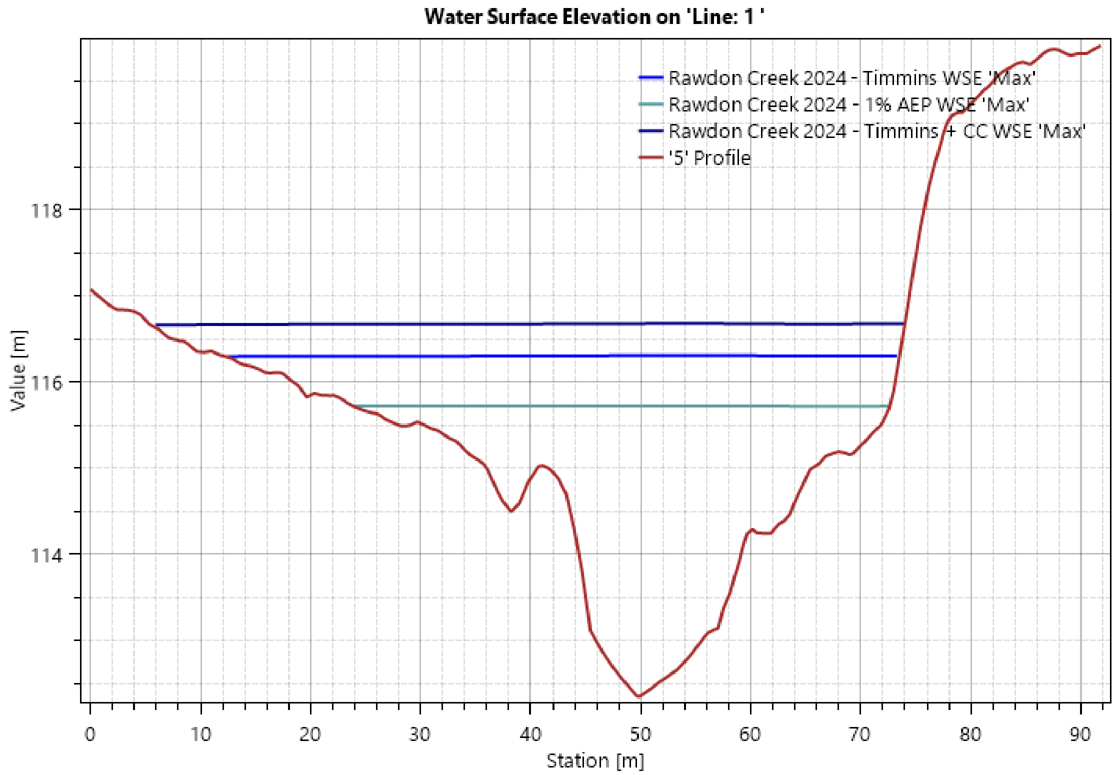
Section 16, Station 2073



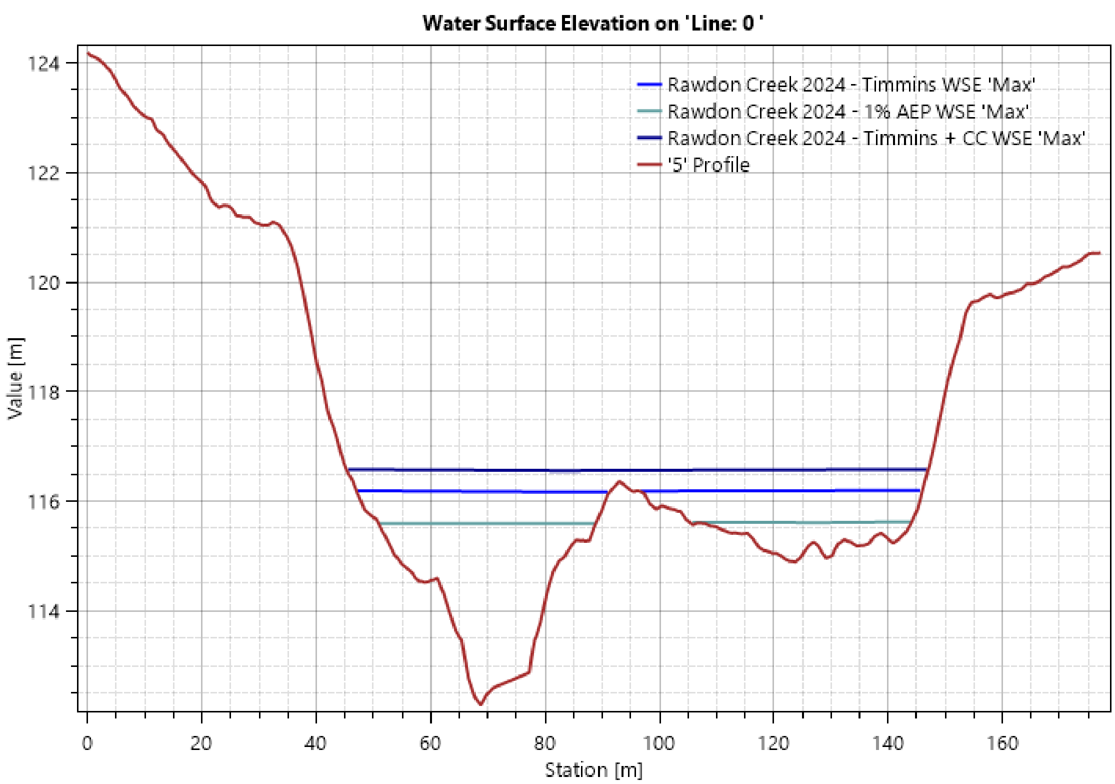
Section 17, Station 1925



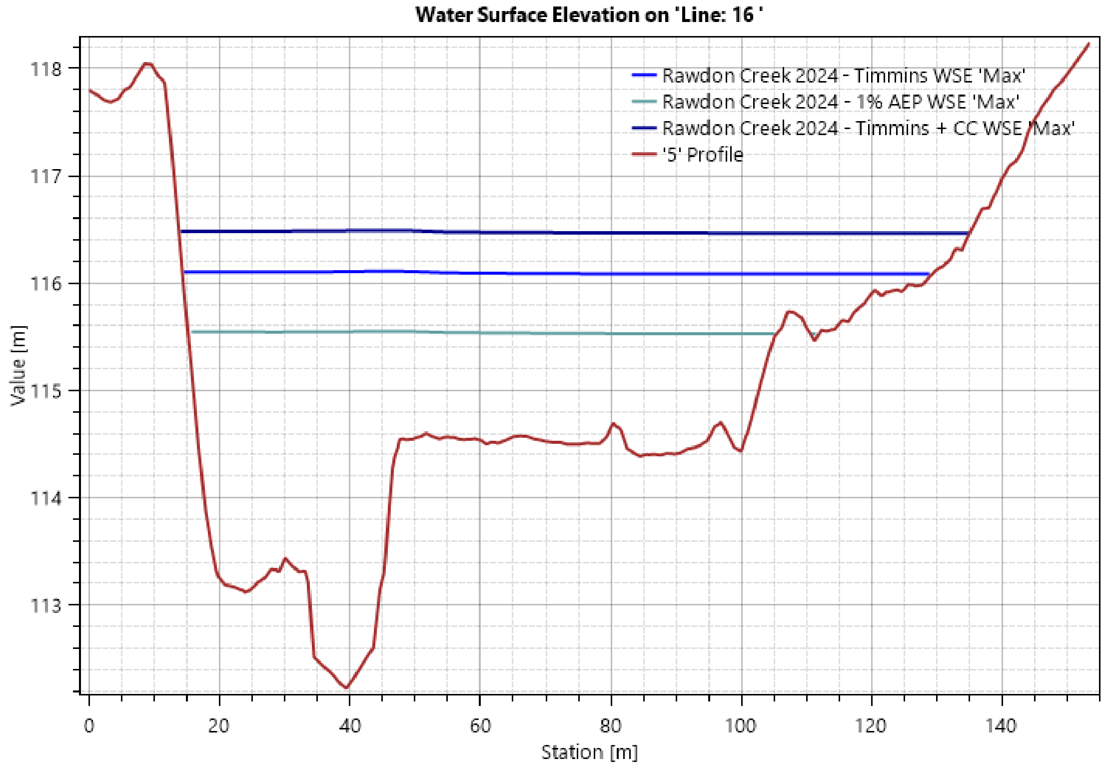
Section 18, Station 1699



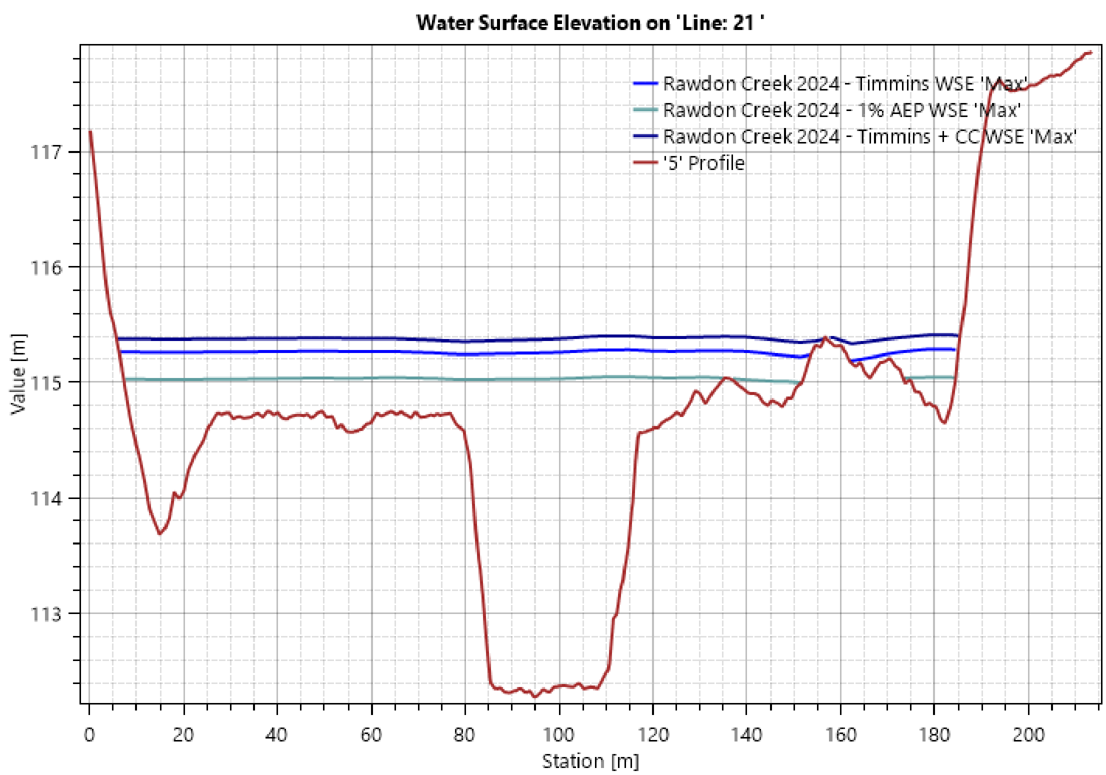
Section 19, Station 1346



Section 20, Station 1073

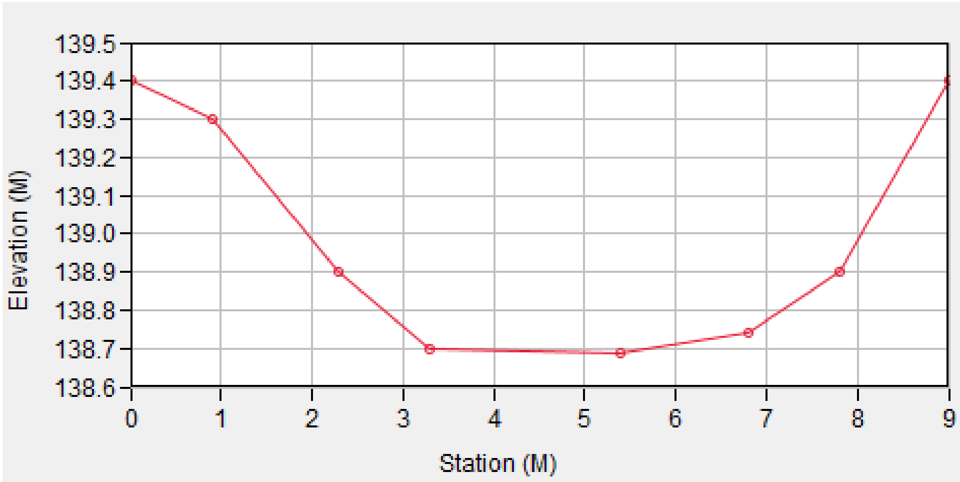


Section 21, Station 851

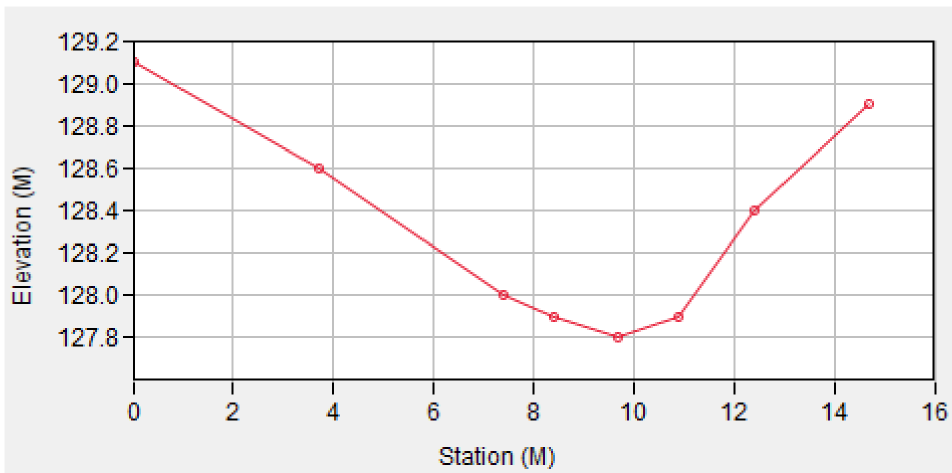


Section 22, Station 361

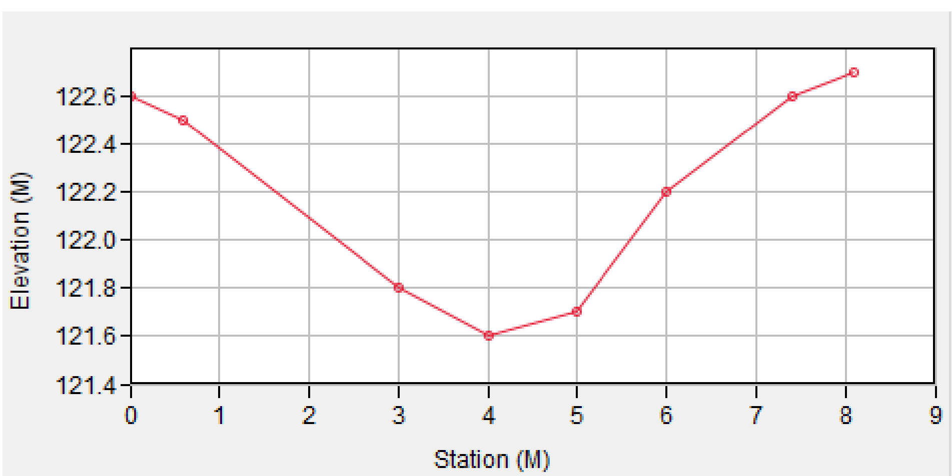
Appendix K:
Representative Eight-Point Cross Sections for Channel Routing



Reach 3 Cross-Section



Reach 4 Cross-Section



Reach 5 Cross-Section