# HYDROLOGY AND HYDRAULICS REPORT

**Lower Trent Region Conservation Authority**

**And**

**The City of Quinte West**

**FHIMP ON22-008**

**For the**

## Mayhew and Cold Creek Floodplain Mapping Update

**June 10, 2024**

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# 1 Introduction

Lower Trent Region Conservation Authority (LTC) has partnered with the City of Quinte West (Quinte West) along with provincial and federal partners to lead the Mayhew & Cold Creek Floodplain Mapping Update.

With acquired funding through the federal Flood Hazard Identification Mapping Program (FHIMP) and Quinte West, LTC has undertaken a leadership role in the production of updated mapping for Mayhew & Cold Creek. The objective is to provide a floodplain mapping update that will allow LTC and Quinte West 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 is 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.

Both Mayhew & Cold Creek were previously mapped in 1980s, making the data no longer current.

# 2 Background

## 2.1 Mayhew Creek

Several previous studies (see list below) were commissioned with the intent to reduce the flood risk at certain locations along Mayhew 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.
- *1981 Flood Control Study* prepared by Crysler & Lathem Ltd.
- *1983 Mayhew Creek Two-Zone Concept* prepared by Totten Sims Hubicki Associates Consultants
- *2009 Draft Mayhew Creek Master Drainage Plan* prepared by WaterPlan Associates
- *2015 Tremur Lake Dam Safety Review* prepared by Sanchez Engineering

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 Mayhew Creek, this report was intended to provide "floodplain mapping of Mayhew Creek from its confluence with the Trent River to the Mill Dam and review a flooding problem at the CN railway underpass at Wooler Road". The floodplain mapping shows the regional storm flood plain and the location of the fill line. The peak flow during the regional storm was determined to be 4671 c.f.s (132 c.m.s) at the mouth of Mayhew Creek.

### *1981 Flood Control Study; Crysler & Lathem Ltd.*

The purpose of the *1981 Flood Control Study* was to determine flood flows and flood levels for the 5-, 10-, 25-, 50-, and 100-year design storms, to assess the flood protection at these return frequencies. Additionally, the author was requested to provide solutions for reducing flooding during these storms, and to assess the structural condition of the Old Mill Dam. It was concluded that Old Mill Dam was not susceptible to failure under existing conditions, flooding depths were anticipated to be 0.3-1.0ft under regional storm conditions, the 100-yr peak flow was 12.8 $m^3/s$, the north and south branch were unable to accommodate the 100-yr flow without impacting structures, and that flooding at the Wooler Road area was largely due to the accumulation of ice in/around culverts on the south side of Wooler Road. The author recommended that culverts and berms be constructed in various locations within the Mayhew Creek watershed to safely convey flood flows, and that ice be removed routinely at the culvert on the south side of Wooler Road.

### *1983 Two-Zone Concept Draft Report; Totten Sims Hubicki Associates Consultants*

This report includes the latest floodplain mapping update since the 1975 mapping prepared by Kilborn Engineering Ltd. The 1983 mapping includes a combination of one-zone and two-zone policy areas separated by boundaries between the former City of Trenton and Murray Township. The investigation presumed future development mitigation measures that were not fully implemented, and therefore, the conservation authority has been regulating the existing conditions mapping to the present day.

### *2009 Draft Mayhew Creek Master Drainage Plan; WaterPlan Associates*

The *2009 Mayhew Creek Master Drainage Plan* is a draft report that was created with the purpose of providing guidelines for practitioners to follow as a basis for development approvals. The work applied updated the hydrologic model using HEC-HMS and IDF inputs from the Trenton EC flow monitoring station at 2nd Dug Hill Road. The hydrologic analysis established a 100-yr peak outflow of 12.2 $m^3/s$. In response to the results from both the hydrologic and hydraulic analyses, various strategies, guidelines, and recommendations were made for best management practices.

### *2015 Tremur Lake Dam Safety Review; Sanchez Engineering Inc.*

LTC provided the *2015 Tremur Lake Dam Safety Review Final Report* to be used as part of the assessment of the outlet structure for Tremur Lake. The Dam Safety Review (DSR) was completed as a comprehensive review of the safety of the Tremur Lake Dam. The report provides relevant background information considering that in 2015 Sanchez Engineering prepared an update of the hydrologic and hydraulic data for the dam. Although the hydrologic model from the DSR was relatively simplistic since the drainage area to Tremur Lake was modeled as a single catchment, its peak flow is useful for comparison to historical peak flow estimates in addition to the current 2023 peak flow estimates described herein.

## 2.2 Cold Creek

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

- *1978 Fill and Floodline Mapping* prepared by Totten Sims Hubicki Associates Limited Consultants
- *1981 Assessment of Flood Control Works* Totten Sims Hubicki Associates Limited Consultants
- 1983 *Floodplain Assessment and Policy Formulation for a Two Zone Concept Application in the Village of Frankford* prepared by Totten Sims Hubicki Associates Limited

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

### *1978 Fill and Floodline Mapping Study; Totten Sims Hubicki Associates Limited Consultants*

The *Fill and Floodline Mapping Study* was intended to determine the floodline based on regional storm criteria, and to establish fill and construction control lines. The flood and fill lines resulting from the study were to be used to guide development within the study area. Additionally, recommendations to reduce flooding were provided, including increasing the discharge capacity of the secondary channel of Cold Creek, and ensuring proper maintenance to bridge structures during ice conditions.

### *1981 Assessment of Flood Control Works; Totten Sims Hubicki Associates Limited Consultants*

The 1981 report was prepared to analyze the impact of flooding at Cold Creek, during the 25-, 50- and 100-yr storm events. In addition, part of the scope was to include the assessment of flooding due to ice jams. Upon doing so, Totten Sims Hubicki Associates recommended remedial work to minimize the impact of flooding, and included a cost-benefit analysis of alternative methods. The 25-, 50-, and 100-yr peak flows were determined to be 79 $m^3/s$, 103 $m^3/s$, and 142 $m^3/s$, respectively.

### *1983 Floodplain Assessment and Policy Formulation for a Two Zone Concept Application in the Village of Frankford; Totten Sims Hubicki Associates Limited*

The 1983 study was prepared in response to the findings of the 1978 *Fill and Floodline Mapping Study* and the *1981 Assessment of Flood Control Works* report. In said reports, the impacts of the regional storm and 100-yr storm on the Village of Frankford were analyzed. The 100-yr and regional storm peak flows used in the analysis were 142 $m^3/s$ and 210 $m^3/s$, respectively. The purpose of the 1983 report was to suggest precautionary measures as well as policies to enforce with the intent of reducing the flood hazard risk.

# 3 Study Area

The study area for the Mayhew and Cold Creek floodplain mapping updates were outlined by LTC at the beginning of the project. The study areas are described below.

## 3.1 Mayhew Creek

The study area focuses on the communities adjacent to Mayhew Creek primarily located within the urban area of Trenton. An excerpt of the study area from LTC documentation is provided in Figure 3-1, where the red line is the historical floodline. Due to development pressures near Telephone Road, Quinte West expressed interest in an extension of the floodplain mapping update to include the area shown in Figure 3-2.

The Mayhew Creek watershed has a total area of 38.7km$^2$. The watershed generally slopes towards the east and flows into Trenton from the west side of the city, and outlets into the Trent River. The Mayhew Creek watershed includes three reservoirs: the Glenburnie Reservoir, Tremur Lake, and the Old Mill Dam reservoir. This is discussed further in Section 4.3.4.

Existing and future build-out conditions were considered. Guidance for future development was obtained from Schedule B of the City of Quinte West Official Plan that outlines land use designations for the Trenton Urban Area. Schedule B1 is included in Appendix A.


**Figure 3-1: Excerpt of Existing Mayhew Creek Mapping Extent from LTC Documentation**

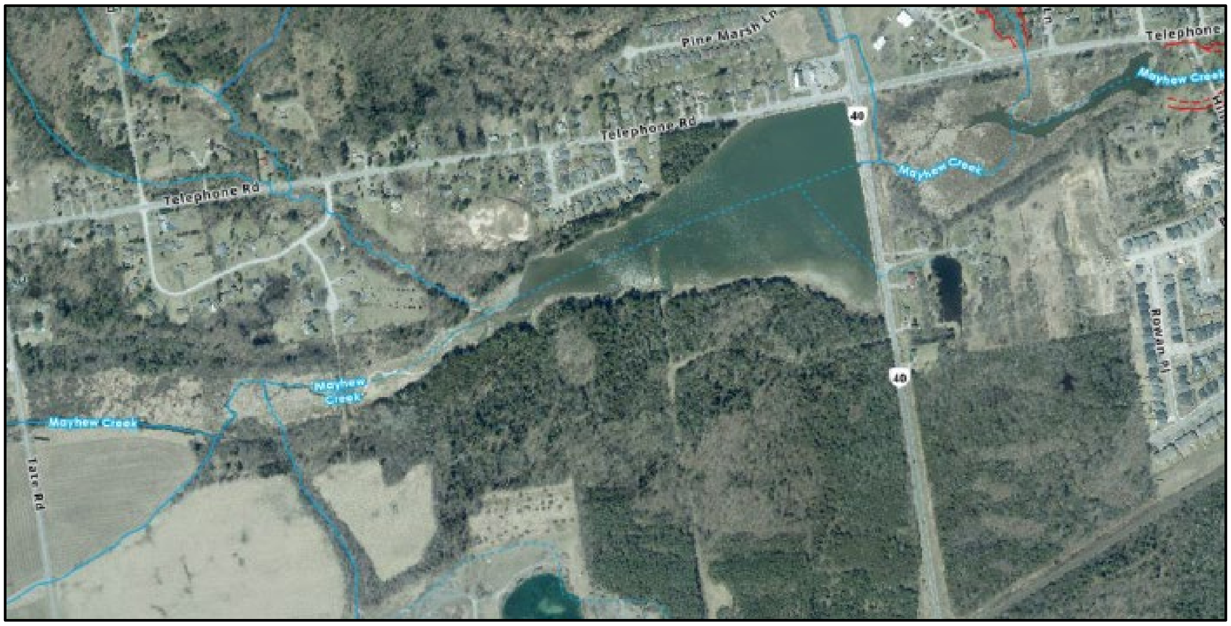



**Figure 3-2: Excerpt of Extension Area of Floodplain Mapping Area - Mayhew Creek**

## 3.2 Cold Creek

The study area focuses on the community adjacent to Cold Creek primarily located within the urban area of Frankford. An excerpt of the study area from LTC documentation is provided in Figure 3-3, where the red line is the existing floodline. The 1978 report identified a large spill from Cold Creek as illustrated in Figure 3-4 and has been re-evaluated in this floodplain mapping update.

The Cold Creek watershed has a total area of 257.3km$^2$. The Cold Creek watershed flows into Frankford from the west and outlets into the Trent River. There are no large flood control reservoirs in the Cold Creek watershed.

Existing and future build-out conditions were considered. Guidance for future development was obtained from Schedule B of the City of Quinte West Official Plan that outlines land use designations for the Frankford Urban Area. Schedule B2 is included in Appendix A.


**Figure 3-3: Excerpt of Existing Cold Creek Mapping Extent from LTC Documentation**

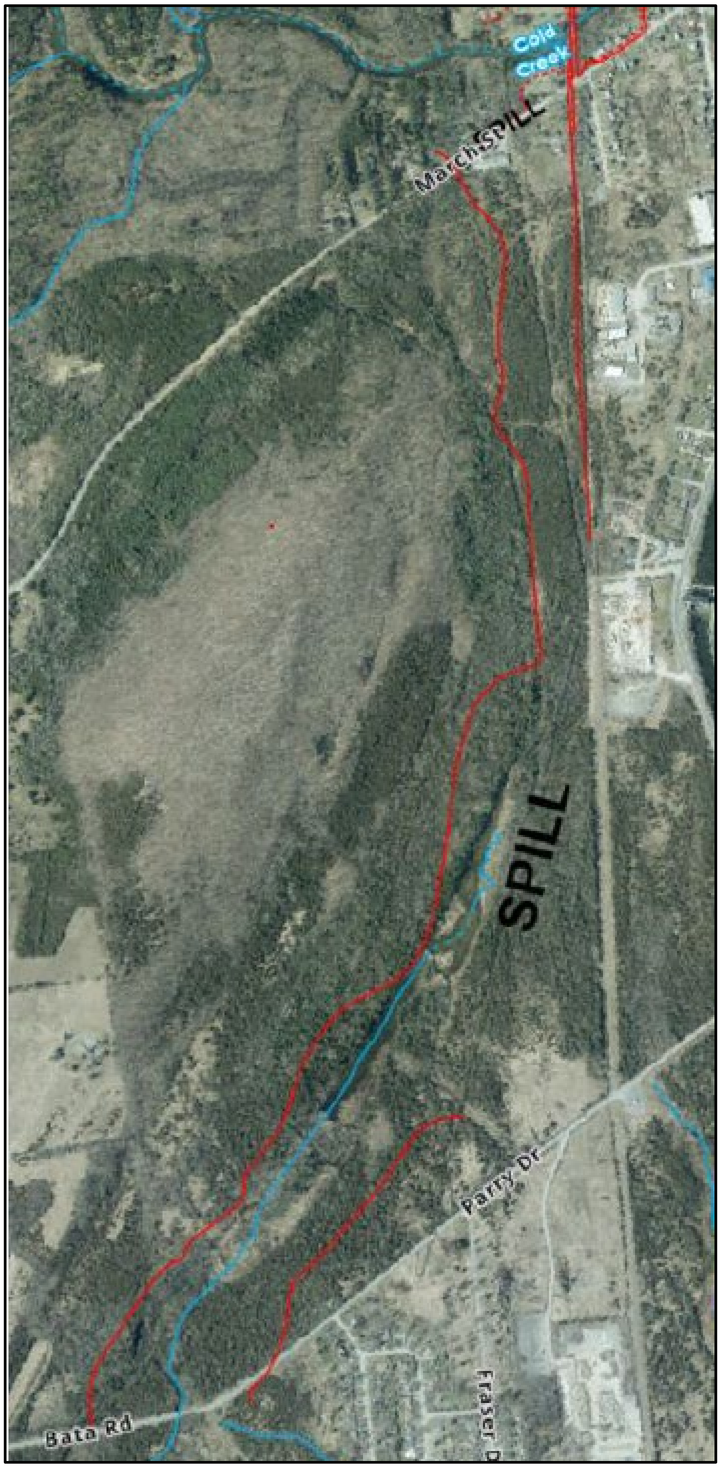



**Figure 3-4: Excerpt from LTC Documentation Showing the Historical Mapping for the Cold Creek Spill at March St**

# 4 Hydrology

The hydrology assessment was prepared for several nodes of interest throughout the Mayhew and Cold Creek watersheds. Various methodologies were applied and compared to determine representative peak flows. Each methodology was carefully considered prior to the selection of the peak flows for use in the hydraulic model, including potential increases in flows due to spring-melt conditions.

The Mayhew and Cold Creek watersheds are 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 Mayhew Creek watershed has a total area of 38.7km$^2$ and traverses the west portion of Trenton before it outlets to the Trent River. Catchment boundaries are identified in Appendix B-1.

Cold Creek has a much larger watershed at 257.3 km$^2$ and similarly traverses the small community of Frankford before it outlets to the Trent River. Catchment boundaries are identified in Appendix B-2.

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 previous studies, however, Jewell completed a detailed review of the contour information and updated the sub-catchment boundaries accordingly.

2) Jewell completed a topographic and bathymetric survey and performed inspections within the relevant urban areas of Quinte West to confirm elevations that may not be identified in LiDAR such as pipe outlets and culverts of interest. Refer to Section 5.1 for more information on the survey.

### *4.1.2 Soils and Land Cover*

A soils map is provided in Appendix C for both watersheds. Soils 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*. Some soil texture information was obtained from the *Soil Map of Northumberland County*.

The Mayhew Creek watershed is primarily comprised of Brighton Sandy Loam and Bondhead Sandy Loam (Ontario Ministry of Agriculture and Food, 1974). The Cold Creek watershed is primarily comprised of Dundonald Sandy Loam, Brighton Sandy Loam, Bondhead Sandy Loam, and Pontypool Sand (Ontario Ministry of Agriculture and Food, 1974).

The soils are predominantly classified as Hydrologic Soils Groups (HSG) A and B for each watershed. 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 Mayhew Creek watershed has 52% HSG A coverage as shown in Appendix B and Table 4-1. Significant portions of the watershed are comprised of HSG B and HSG C soils, with 27% and 14% coverage, respectively. The Cold Creek watershed has 46% HSG A coverage as shown in Table 4-2. Significant coverage of HSG B is present at 40% coverage, with a much smaller amount attributed to soil types C and D.

**Table 4-1: Mayhew Creek HSG Summary**

| HSG Soils Group | Area ($km^2$) | Land coverage (%) |
|---|---|---|
| A | 20.3 | 52 |
| B | 10.4 | 27 |
| C | 5.51 | 14 |
| D | 2.47 | 6 |

**Table 4-2: Cold Creek HSG Summary**

| HSG Soils Group | Area ($km^2$) | Land coverage (%) |
|---|---|---|
| A | 109.4 | 46 |
| B | 101.7 | 40 |
| C | 15.4 | 5 |
| D | 20.0 | 8 |

> 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 run off 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 run off the lands. CNs are representative of the pervious portion of the watershed. Jewell followed the guidance in *MTO Design Chart 1.09* to determine CNs for the discretized catchments.

Land cover information was obtained from the Ontario Land Cover Compilation (OLCC), a database owned by *Land Information Ontario,* and provided by the *Ontario Ministry of Natural Resources and Forestry*. A review of land coverage for both watersheds shows that the land use is predominantly cultivated land, woods, and water. A summary of land coverage percentage is provided in the tables below. Land cover maps are provided in Appendix C.

**Table 4-3: Mayhew Creek Land Cover Summary**

| Land Cover | Area (km$^2$) | Land Coverage (%) |
|---|---|---|
| Woods | 7.31 | 19 |
| Meadows | 0.08 | 0.2 |
| Cultivated | 16.45 | 43 |
| Urban | 4.82 | 12 |
| Water | 9.94 | 26 |
| Bedrock | 0.05 | 0.1 |

**Table 4-4: Cold Creek Land Cover Summary**

| Land Cover | Area (ha) | Land Coverage (%) |
|---|---|---|
| Woods | 49.6 | 19 |
| Meadows | 0.2 | 0.1 |
| Cultivated | 149.0 | 58 |
| Urban | 10.4 | 4 |
| Water | 45.7 | 18 |
| Bedrock | 0.6 | 0.2 |

### *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 was the selected station since it yields the longest record of data and is in closest proximity to Mayhew and Cold Creek.

Environment and Climate Change Canada (ECCC) provided additional precipitation and stream flow gauge data for the Mayhew Creek near Trenton (02HK011) and Cold Creek Station at Orland (02HK007) stations. 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. Therefore, the precipitation data was used with caution and only to help in understanding of the distribution of the daily totals from the Trenton Airport station.

On September 8$^{th}$ and 9$^{th}$ 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 between Cobourg and Kingston, including Quinte West. 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.

For context, the statistical 100-yr event for the Trenton Airport station has a projected depth of 96.5mm and 108.1mm for the 12hr and 24hr durations respectively. The Hurricane Frances event produced rainfall volumes in excess of a 100-yr statistical storm for a similar duration. Therefore, it is a historic event that provides opportunity to calibrate the Mayhew and Cold Creek hydrologic models to known data values. This is discussed further in Section 4.3.

An additional major rainfall event occurred locally in September of 2021. This rainfall event was used to verify the results of the model after calibration. The rainfall distribution for the September 2021 event is shown in Figure 4-23. These events are discussed further in Section 4.3.

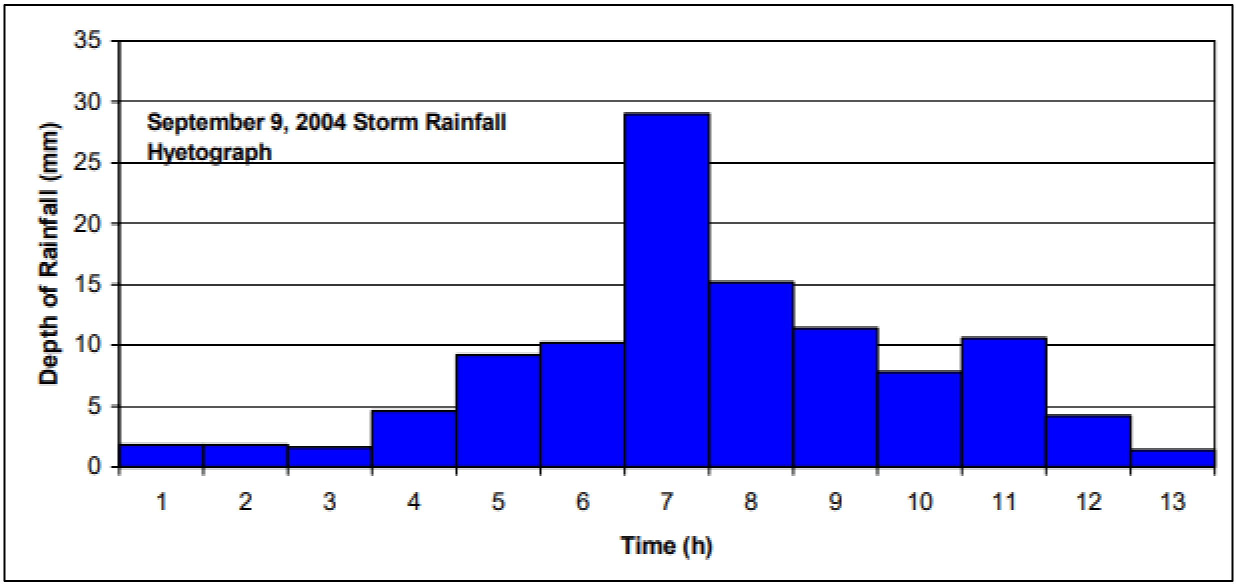



**Figure 4-2: Excerpt from 2008 Potter Creek MDP Illustrating Hurricane Franes Rainfall Hyetograph**

An important consideration in the precipitation data is the potential impacts on 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 used 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 calculate 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. A rainfall depth of less than 2.5 standard deviations would be within the 95% confidence interval; the previously mentioned 111.8mm rain event in 2004 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 our analysis. However, we note that the Frances event is a statistical outlier and could be omitted for statistical correctness. 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. Since the Timmins event is more severe than the 2004 rainfall, it would continue to govern in an assessment of historical storms.

As directed by the *MNR 2002 Technical Guide*, an areal reduction factor of 94% was applied to the Timmins precipitation data for Mayhew Creek. This reduction factor was selected from Table D-4 of the Guide using an equivalent circular area of 70.7km$^2$. The equivalent circular area was derived from an equivalent circular diameter of 9.5km$^2$, when measured at the point of interest at the outlet to the Trent River.

An areal reduction factor of 70% was applied to the Timmins precipitation data for Cold Creek. This reduction factor was derived from an equivalent circular diameter of 34.75km, resulting in an equivalent circular area of 948.1km$^2$.

**Table 4-5: Unadjusted vs. Adjusted Trenton Airport Runoff Volumes for 24-Hr Duration Storm**

| Storm Event | Rainfall Volume (mm) | | % Difference |
|---|---|---|---|
| | Unadjusted | 2004 Outlier Removed | |
| 50-yr | 98.6 | 87.7 | 12.4% |
| 100-yr | 108.1 | 96.4 | 12.1% |
| 200-yr | 117.6 | 104.1 | 13.0% |
| 500-yr | 130.1 | 114.3 | 13.8% |
| *Timmins | 193.0 | - | - |

**Timmins Storm from MTO Design Chart 1.04*

**Table 4-6: Mayhew Creek Timmins Rainfall with Areal Reduction**

| | | | |
|---|---|---|---|
| **Equivalent Circular Diameter** | | **9.5** | **km** |
| **Equivalent Circular Area** | | **70.7** | **km$^2$** |
| **% of Timmins Storm Required** | | **94%** | |
| **Hour** | **Depth (mm) No Reduction** | **Depth (mm) With Reduction** | ***Depth (mm) Climate Change** |
| 1 | 15 | 14.1 | 17.6 |
| 2 | 20 | 18.8 | 23.5 |
| 3 | 10 | 9.4 | 11.8 |
| 4 | 3 | 2.82 | 3.5 |
| 5 | 5 | 4.7 | 5.9 |
| 6 | 20 | 18.8 | 23.5 |
| 7 | 43 | 40.42 | 50.5 |
| 8 | 20 | 18.8 | 23.5 |
| 9 | 23 | 21.62 | 27.0 |
| 10 | 13 | 12.22 | 15.3 |
| 11 | 13 | 12.22 | 15.3 |
| 12 | 8 | 7.52 | 9.4 |
| **TOTAL** | **193.0** | **181.4** | **226.8** |

**See Section 4.5 for Climate Change discussion.*

**Table 4-7: Cold Creek Timmins Rainfall with Areal Reduction**

| | | | |
|---|---|---|---|
| **Equivalent Circular Diameter** | | **34.7** | **km** |
| **Equivalent Circular Area** | | **948** | **$km^2$** |
| **% of Timmins Storm Required** | | **70%** | |
| **Hour** | **Depth (mm) No Reduction** | **Depth (mm) With Reduction** | ***Depth (mm) Climate Change** |
| 1 | 15 | 10.5 | 13.1 |
| 2 | 20 | 14 | 17.5 |
| 3 | 10 | 7 | 8.8 |
| 4 | 3 | 2.1 | 2.6 |
| 5 | 5 | 3.5 | 4.4 |
| 6 | 20 | 14 | 17.5 |
| 7 | 43 | 30.1 | 37.6 |
| 8 | 20 | 14 | 17.5 |
| 9 | 23 | 16.1 | 20.1 |
| 10 | 13 | 9.1 | 11.4 |
| 11 | 13 | 9.1 | 11.4 |
| 12 | 8 | 5.6 | 7.0 |
| **TOTAL** | **193** | **135.1** | **168.9** |

The recommended return period storms for floodplain mapping are derived from SCS and AES distributions with varying durations. In an assessment of the critical return period storm, Jewell compared the peak outflows from the HEC-HMS hydrologic model for the 6-, 12-, and 24-hr duration events with both distributions. Any event less than 6 hours is not recommended since shorter duration events do not produce significant enough rainfall volumes to govern as the regulatory storm event. The AES distribution was not selected since it produced lesser flows than the SCS Type II distribution.

For Mayhew and Cold Creek, the 24-hr duration with an SCS distribution produces the largest peak runoff rate.

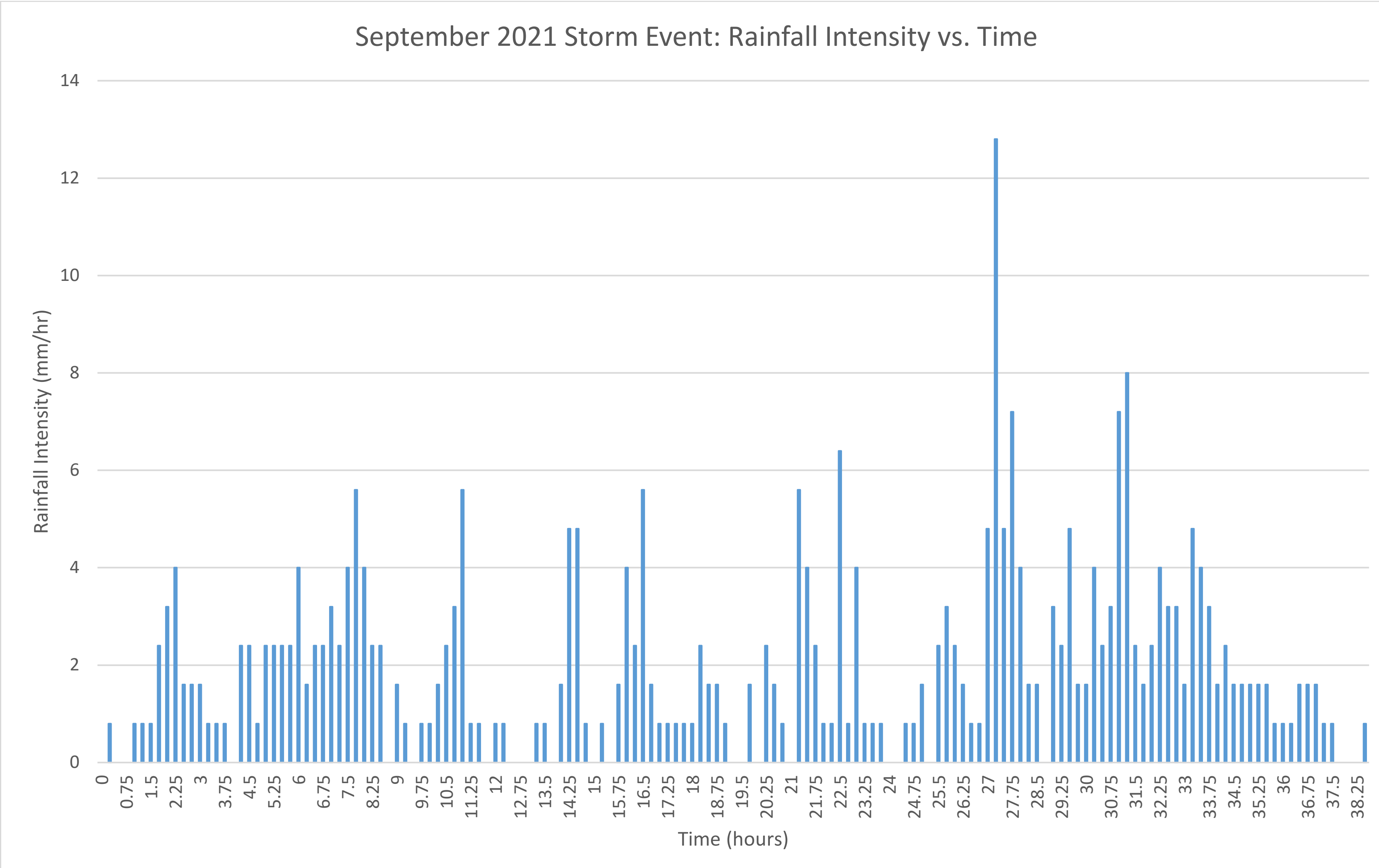



**Figure 4-3: Measured Rainfall Input for September 2021 Validation Event**

### *4.1.4 Water Survey of Canada Stream Flows*

There is a stream flow gauge located along Mayhew Creek at *Water Survey of Canada (WSOC)* Station 02HK011 titled 'Mayhew Creek Near Trenton'. The gauge is located near the intersection of County Road 42 and Hill Road with a receiving drainage area of 33.0 $km^2$, or 85% of the total 38.7 $km^2$ Mayhew Creek watershed.

The flow data of interest is the *Annual Maximum Instantaneous Peak Discharge.* The length of record for the gauge is from 1994 to 2021, with 27 years of annual instantaneous maximum peaks.

Similarly, there is a stream flow gauge located along Cold Creek at *Water Survey of Canada (WSOC)* Station 02HK007 titled 'Cold Creek at Orland'. The flow gauge has a receiving drainage area of 161 $km^2$, or 63% of the total 257.3 $km^2$ Cold Creek watershed.

The flow data of interest is the *Annual Maximum Instantaneous Peak Discharge.* The record length for the gauge is from 1982 to 2021, with 36 years of annual instantaneous maximum peaks.

Discharge data was received from ECCC in 5-minute intervals for use in the calibration and validation model runs. Table 4-8 summarizes the station data; gauge locations are shown in Figure 4-4.

**Table 4-8: List of Local Stream Flow Gauges for Extended Data Record**

| Name | Station ID | Length of Record | Gross Drainage Area ($km^2$) |
|---|---|---|---|
| Cold Creek at Orland | 02HK007 | 1982 - 2021 | 161.0 |
| Mayhew Creek near Trenton | 02HK011 | 1994 - 2021 | 33.0 |

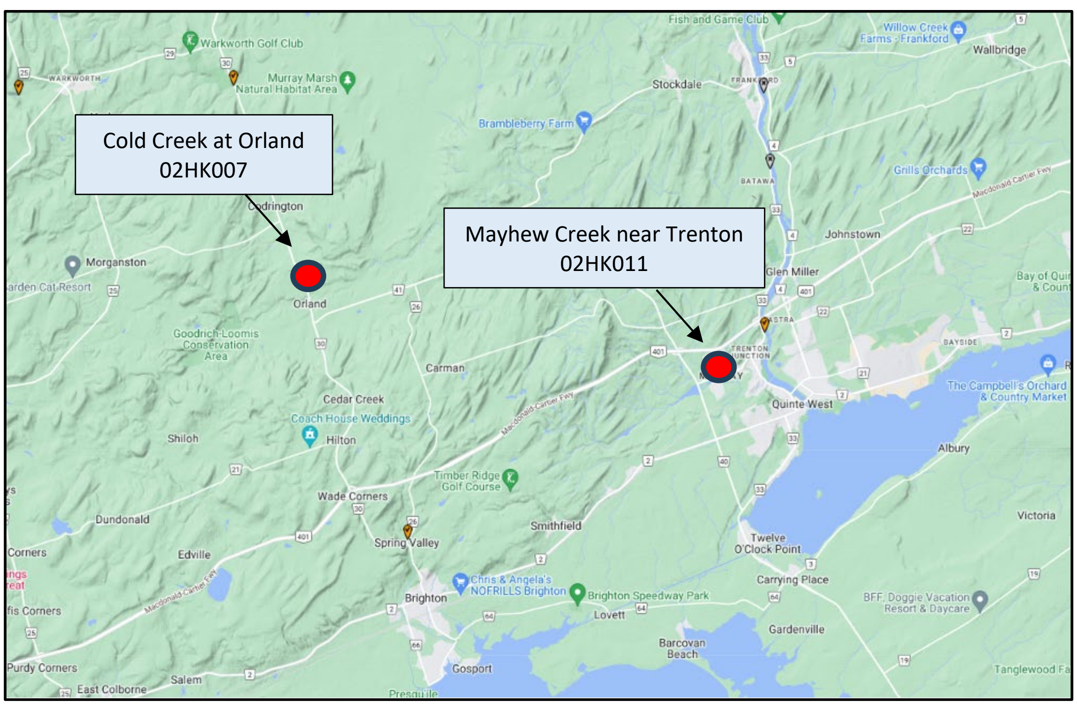



**Figure 4-4: Mayhew and Cold Creek Water Survey of Canada Stream Flow Gauge Locations**

## 4.2 Flood Frequency Analysis

The Consolidated Frequency Analysis V3.1 was used to complete the general frequency analysis (GFA) 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.

The Mayhew Creek analysis was completed on station 02HK011 that contained instantaneous peak flow data from 1994 to 2022, representing 27 years of record.

**Table 4-9: General Frequency Analysis – Mayhew Creek**

| Return Period (yr) | Peak Flow (cms) |
|---|---|
| 2 | 6.8 |
| 5 | 9.5 |
| 10 | 11.2 |
| 20 | 12.8 |
| 50 | 14.9 |
| 100 | 16.5 |
| 200 | 18.0 |
| 500 | 20.0 |

The Cold Creek analysis used the entire period of record for which annual extremes are reported (1982 – 2021). This represented 40 years of record and peak flows for 36 years were reported.

**Table 4-10: General Frequency Analysis Results – Cold Creek**

| Return Period (yr) | Peak Flow (cms) |
|---|---|
| 2 | 21.5 |
| 5 | 27.4 |
| 10 | 30.7 |
| 20 | 33.6 |
| 50 | 37.1 |
| 100 | 39.4 |
| 200 | 41.7 |
| 500 | 44.5 |

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 27-yr data record of annual instantaneous peaks at the

WSOC Mayhew Creek flow gauge, only six (6) years had instantaneous peaks outside the months of January to April. For the 36-yr data record of annual instantaneous peaks at the WSOC Cold Creek flow gauge, only seven (7) years had instantaneous peaks outside the months of January to April. This suggests a 78-81% probability that a severe flood 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 produced in a spring melt or spring melt plus rainfall event.

## 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 HEC-HMS 4.11.

All modelling programs are simplifications of reality and are limited in their capabilities. While HEC-HMS is a widely used and trusted hydrologic model, the model results are limited by input parameters and the uncertainty associated in the data sets and calculations used to produce these inputs. The modelling program is acceptable for simulating peak flows to be used in the hydraulic model. The most recent software publication was used for this project.

HEC-HMS was selected for the Mayhew Creek and Cold Creek hydrologic models.

Notable input parameters for the hydrologic model include:

- Precipitation – intensity, duration, and frequency as well as distribution.
- Catchment area.
- Percent imperviousness – runoff volume, time to peak, and peak flow increase with percent imperviousness.
- 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. CN was used.
- 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.

### *4.3.1 CN Loss Method*

The CN loss method was selected 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 GIS software applications 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. AMC II was used in the return period and Timmins peak flow estimates to be used in the updated floodplain mapping as presented in Section 4.6. 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 at the flow gauge consistently occur during the spring snow-melt season.

The initial abstraction parameter was set to 5mm for all catchments for both watersheds. Conversations with ECCC led to the conclusion that an adjustment to sub-catchment curve numbers, as opposed to initial abstraction, was more appropriate to account for the pond and wetland coverage and for calibration of the model.

### *4.3.2 Lag Time*

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 Mayhew and Cold Creek are within the upper limit. The largest sub-catchment between the two creeks is 37 $km^2$, and majority of the sub-catchments have a drainage area of 24 $km^2$ or less. 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.

### *4.3.3 Channel Routing*

Channel routing was completed using the Muskingum-Cunge method. This method is applicable for reaches with relatively shallow slopes. This routing method allows the user to input a cross-section to represent ground surface data for the channel and overbank areas. Cross-sections were obtained from the terrain data and then simplified into an eight-point cross-section.

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 4-11: Muskingum-Cunge Channel Routing Dimensions – Mayhew Creek**

| Route | Length (m) | Slope (%) | Manning's n | | | Index Celerity |
|---|---|---|---|---|---|---|
| | | | Left | Middle | Right | |
| Junction A – Junction AB 2-1 | 877 | 1.63 | 0.05 | 0.05 | 0.05 | 0.5 |
| Junction AB 2-1 – Junction B | 925 | 1.83 | 0.05 | 0.05 | 0.05 | 0.5 |
| Junction B - Glenburnie Reservoir | 996 | 0.97 | 0.05 | 0.05 | 0.05 | 0.5 |

**Table 4-12: Muskingum-Cunge Channel Routing Dimensions - Cold Creek**

| Route | Length (m) | Slope (%) | Manning's n | | | Index Celerity |
|---|---|---|---|---|---|---|
| | | | Left | Middle | Right | |
| Junction B – Junction BC 4-3 | 906 | 0.27 | 0.05 | 0.05 | 0.05 | 0.6 |
| Junction BC 4-3 – Junction BC 3-2 | 921 | 0.16 | 0.05 | 0.05 | 0.05 | 0.6 |
| Junction BC 3-2 – Junction BC 2-1 | 1399 | 0.06 | 0.05 | 0.05 | 0.05 | 0.6 |
| Junction BC 2-1 – Junction C | 1298 | 0.37 | 0.05 | 0.05 | 0.05 | 0.6 |
| Junction C – Junction CD 5-4 | 1583 | 0.43 | 0.05 | 0.05 | 0.05 | 0.6 |
| Junction CD 5-4 – Junction CD 4-3 | 1851 | 0.30 | 0.05 | 0.05 | 0.05 | 0.6 |
| Junction CD 4-3 – Junction CD 3-2 | 1365 | 0.29 | 0.05 | 0.05 | 0.05 | 0.6 |
| Junction CD 3-2 – Junction CD 2-1 | 318.5 | 0.33 | 0.05 | 0.05 | 0.05 | 0.6 |
| Junction CD 2-1 – Junction D | 1211 | 0.41 | 0.05 | 0.05 | 0.05 | 0.6 |
| Junction D – Junction DE 1-2 | 1107 | 0.21 | 0.05 | 0.05 | 0.05 | 0.6 |
| Junction DE 1-2 – Junction DE 2-3 | 675 | 0.15 | 0.05 | 0.05 | 0.05 | 0.6 |
| Junction DE 2-3 – Junction DE 3-4 | 946 | 0.11 | 0.05 | 0.05 | 0.05 | 0.6 |
| Junction DE 3-4 – Junction E | 811 | 0.09 | 0.05 | 0.05 | 0.05 | 0.6 |
| Junction E – Junction F | 8487 | 0.05 | 0.05 | 0.05 | 0.05 | 0.6 |
| Junction F – Junction G | 4798 | 0.03 | 0.05 | 0.05 | 0.05 | 0.6 |
| Junction G – Junction H | 4711 | 0.21 | 0.05 | 0.05 | 0.05 | 0.6 |
| Junction H – Junction I | 7893 | 0.03 | 0.05 | 0.05 | 0.05 | 0.6 |
| Junction I – Junction J | 4256 | 0.22 | 0.05 | 0.05 | 0.05 | 0.6 |

### *4.3.4 Reservoir Routing*

Reservoir routing was included in the Mayhew Creek hydrologic model to account for the Glenburnie Reservoir, Tremur Lake, and the Old Mill Dam. The stage-storage-discharge relationship for each reservoir was established based on field survey, GIS applications, and a review of background documents. The field survey is used to identify the elevation and size of the outlet structures, the GIS applications are used to establish the footprint area of the reservoir, and the background documents identified the normal operating procedures for Tremur Lake as described in the *2015 Tremur Lake Dam Safety Review*.

The storage-discharge relationship is based on the assumption that all stop logs are in place. Quinte West Work Orders for monthly dam inspections since 2017 were reviewed for current operating procedures. The monthly inspections records were used to match the stoplog settings in the 2021 validation event, ensuring the modeled Tremur Lake Dam is consistent with the 2021 inspection form.

The Glenburnie Reservoir is controlled by a 2.5m long x 0.55m rise sharp-crested weir for low flows, and a 300mm diameter v-notch orifice becomes engaged once the water reaches the top of the sharp-crested weir opening. In large storm events runoff would overtop the access road as broad-crested weir flow.

Tremur Lake is controlled by two stop log bays with a 2.52m openings; large storm events would spill over Wooler Road as broad-crested weir flow.

The Old Mill Dam is controlled by a 3m sharp-crested weir; in large events a second-stage of the sharp-crested weir would be utilized. The storage-discharge relationships applied to each reservoir are shown in the figures below.

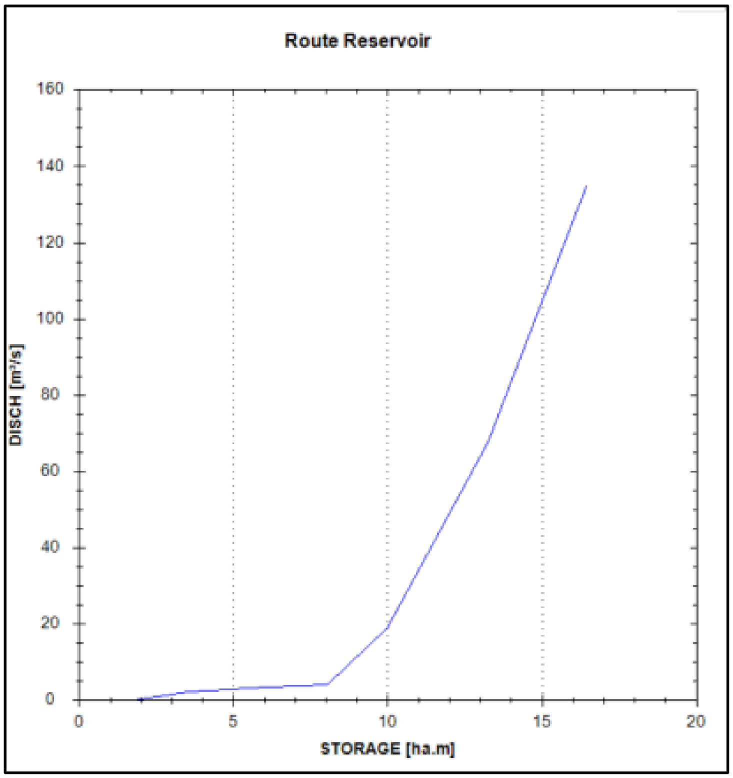



**Figure 4-5: Glenburnie Reservoir Storage-Discharge Relationship**

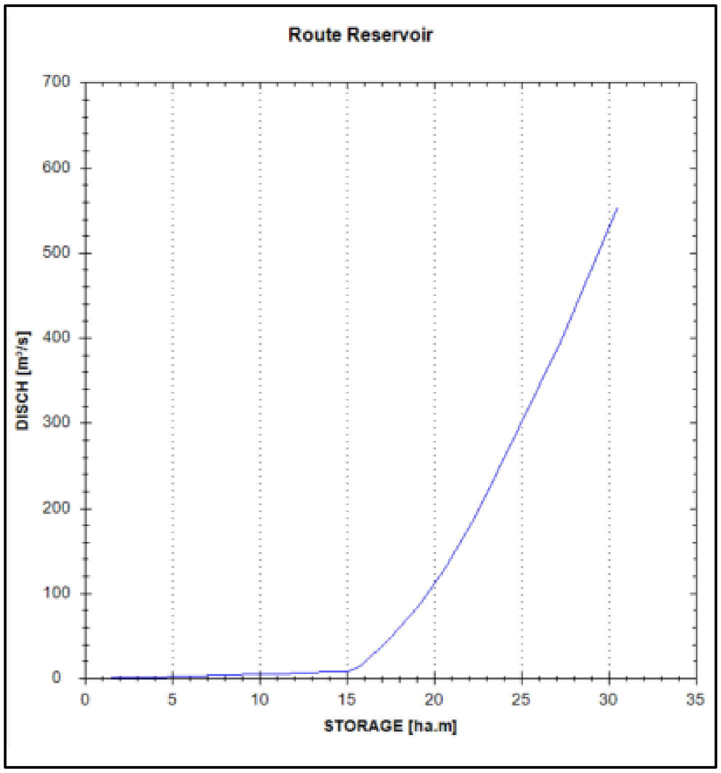



**Figure 4-6: Tremur Lake Storage-Discharge Relationship**

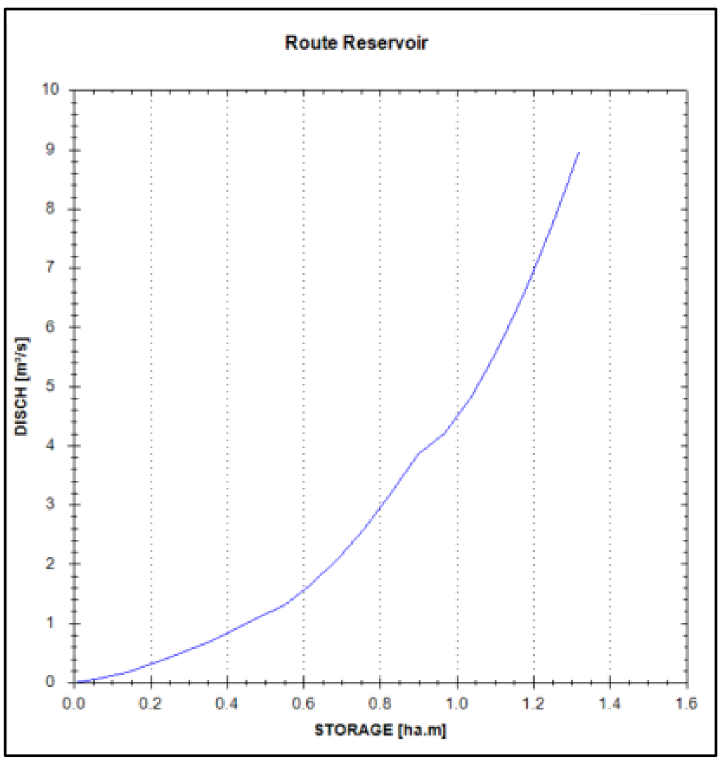



**Figure 4-7: Old Mill Dam Storage-Discharge Relationship**

### *4.3.5 Calibration*

Calibration of the model was completed through a 3-step process.

*Step 1:* Calibrate HEC-HMS model to Frances event by adjusting CN, storage coefficient and time of concentration.

*Step 2:* Calibrate model to 1% AEP using global adjustment to curve number.

*Step 3:* Calibrate to remaining return period events with dynamic AMC adjustment.

#### 4.3.5.1 Step 1: Calibration to Hurricane Frances

Firstly, the HEC-HMS model was calibrated to the 2004 Hurricane Frances storm event. Calibration parameters included the CN, storage coefficient and time of concentration. Parameters were adjusted until the shape of the modelled hydrograph was fitted reasonable with the recorded data.

On September 9 of 2004, the stream flow gauge for Mayhew Creek recorded its maximum peak flow of 16.1 $m^3/s$ and the stream flow gauge for Cold Creek recorded its maximum peak flow of 33.9 $m^3/s$.

The Hurricane Frances event occurred 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.

The Frances event for each watershed is shown in Figure 4-8 and Figure 4-9.

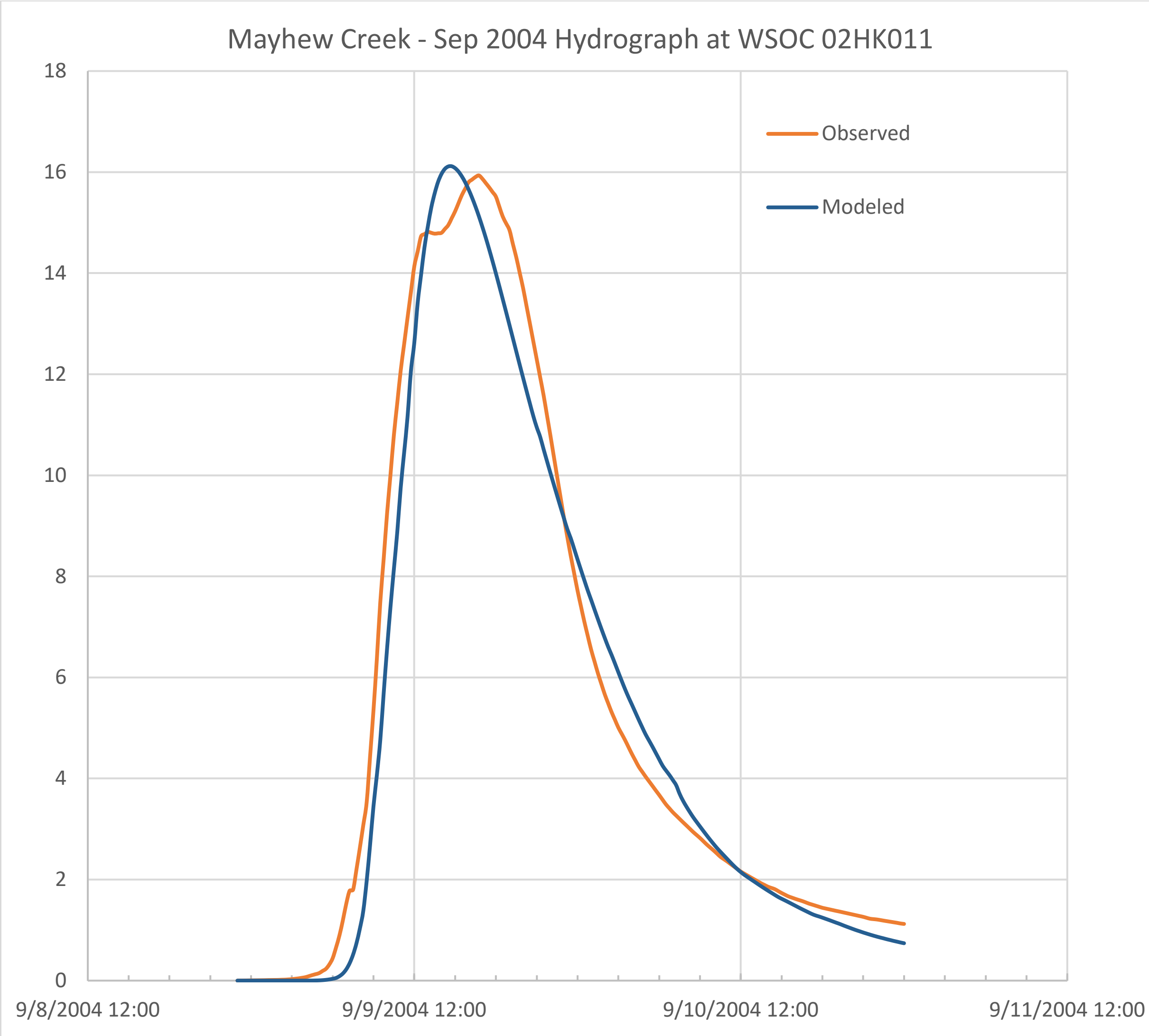



**Figure 4-8: Mayhew Creek Calibration to Frances**

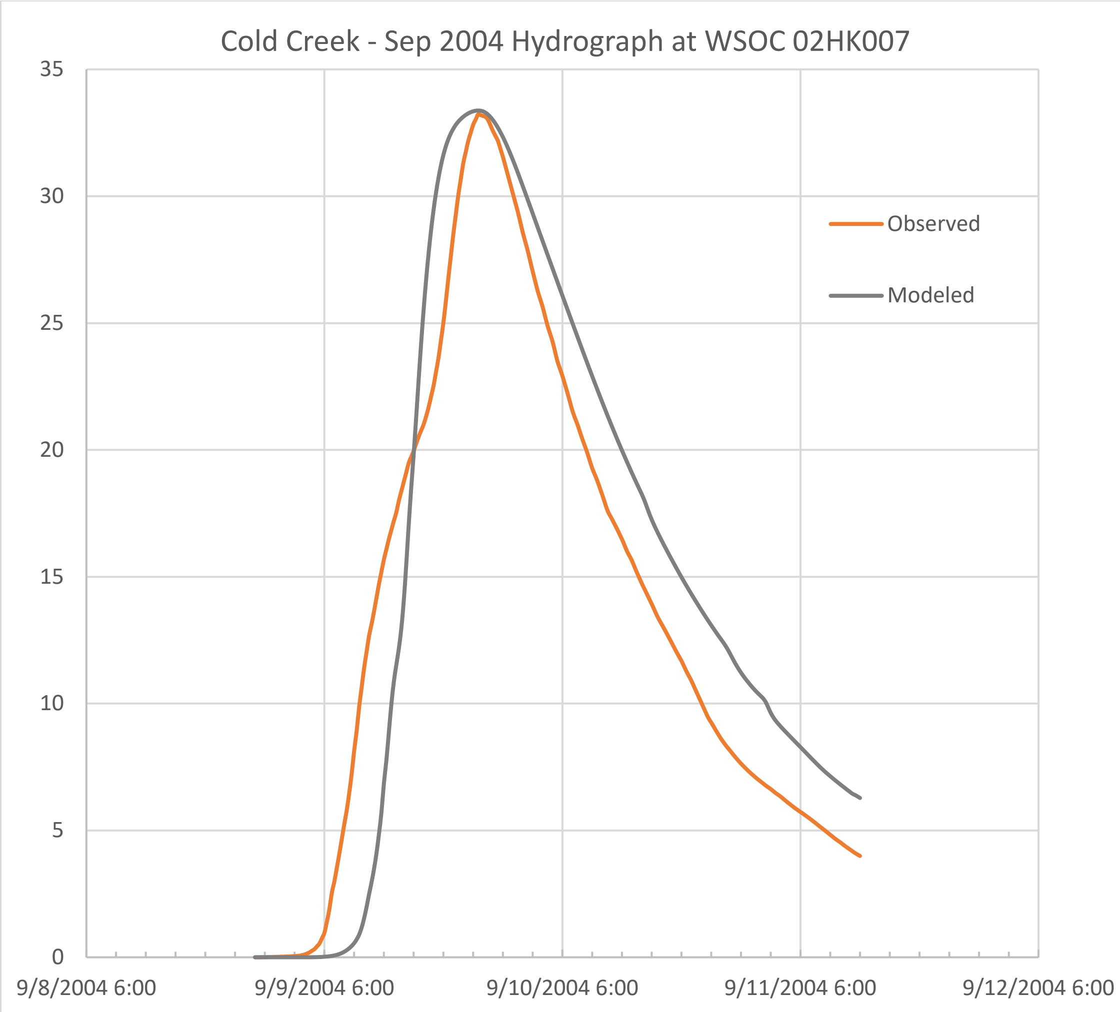



**Figure 4-9: Model Calibration to Frances – Cold Creek**

### 4.3.5.2 Calibration to 1% AEP

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 AMC II conditions such that the adjusted model represents the AMC II conditions. Input parameters for each creek are listed in Tables 4-13 and Table 4-14.

A summary of peak flow results is plotted on semi-log scales in the following two figures.

The yellow line represents the model outputs with calibration to the 100-yr event. While providing acceptable agreement with the GFA results in the 50-yr to 100-yr range, the HEC-HMS model underestimates the more frequent return period events and overestimates the less frequent events. It follows that the HEC-HMS model will overestimate the larger Timmins precipitation event that exceeds the return period precipitation depths.

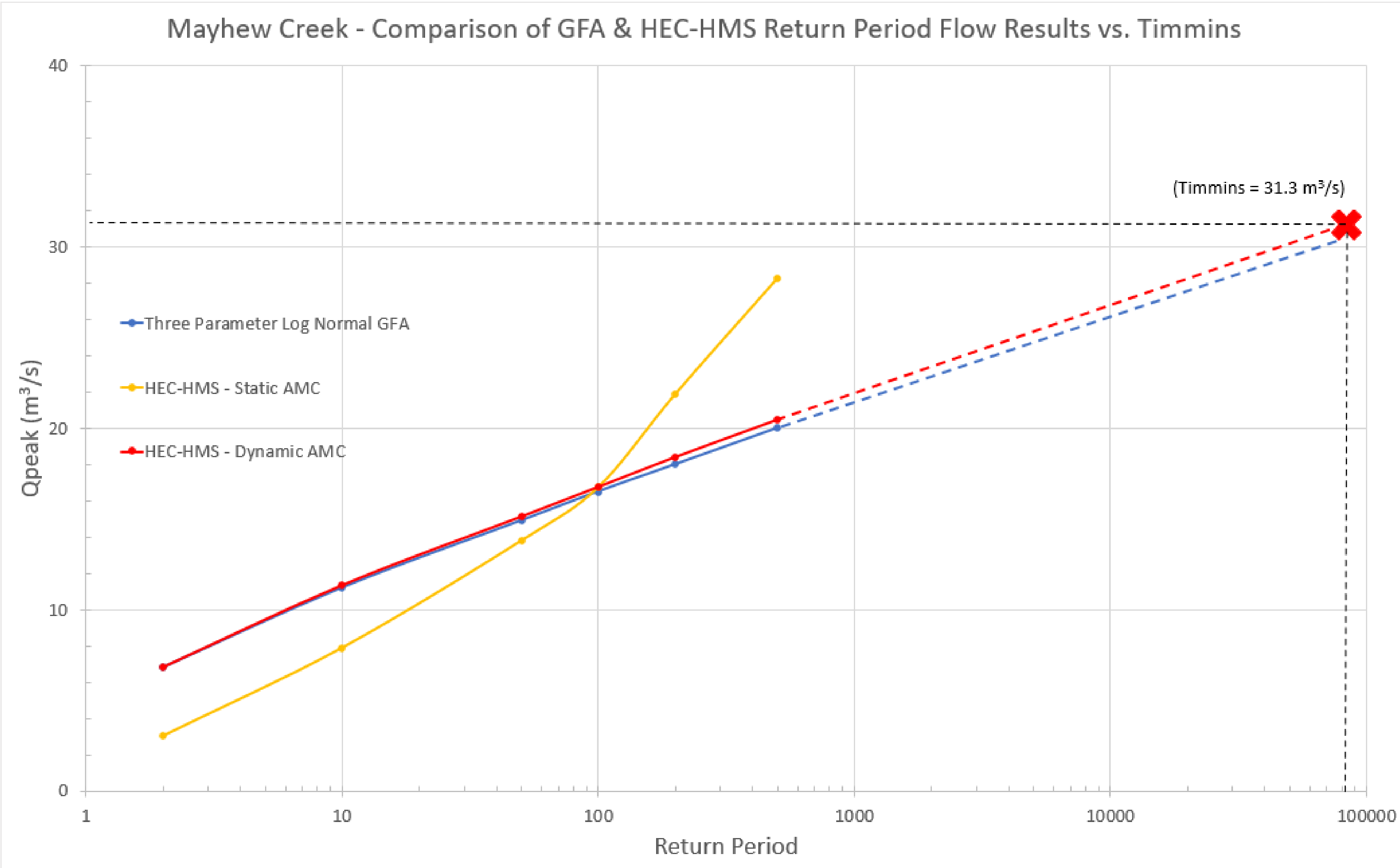
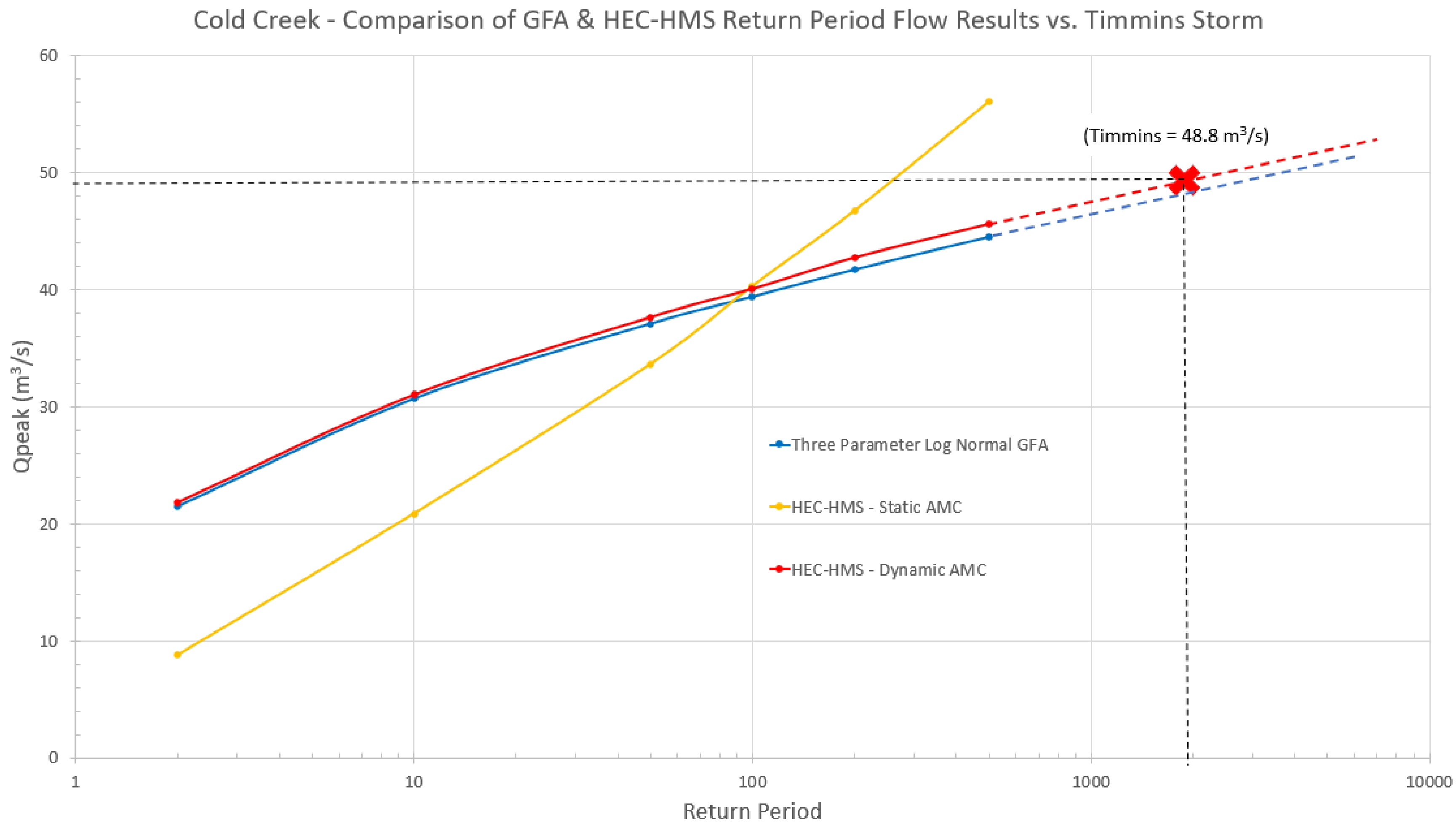



**Figure 4-10: Calibration of Mayhew Creek Model to Return Period Events with Dynamic AMC**




**Flow results in above chart measured at Cold Creek Stream Flow Gauge Location (02HK007)*

**Figure 4-11: Calibration of Cold Creek Model to Return Period Events with Dynamic AMC**

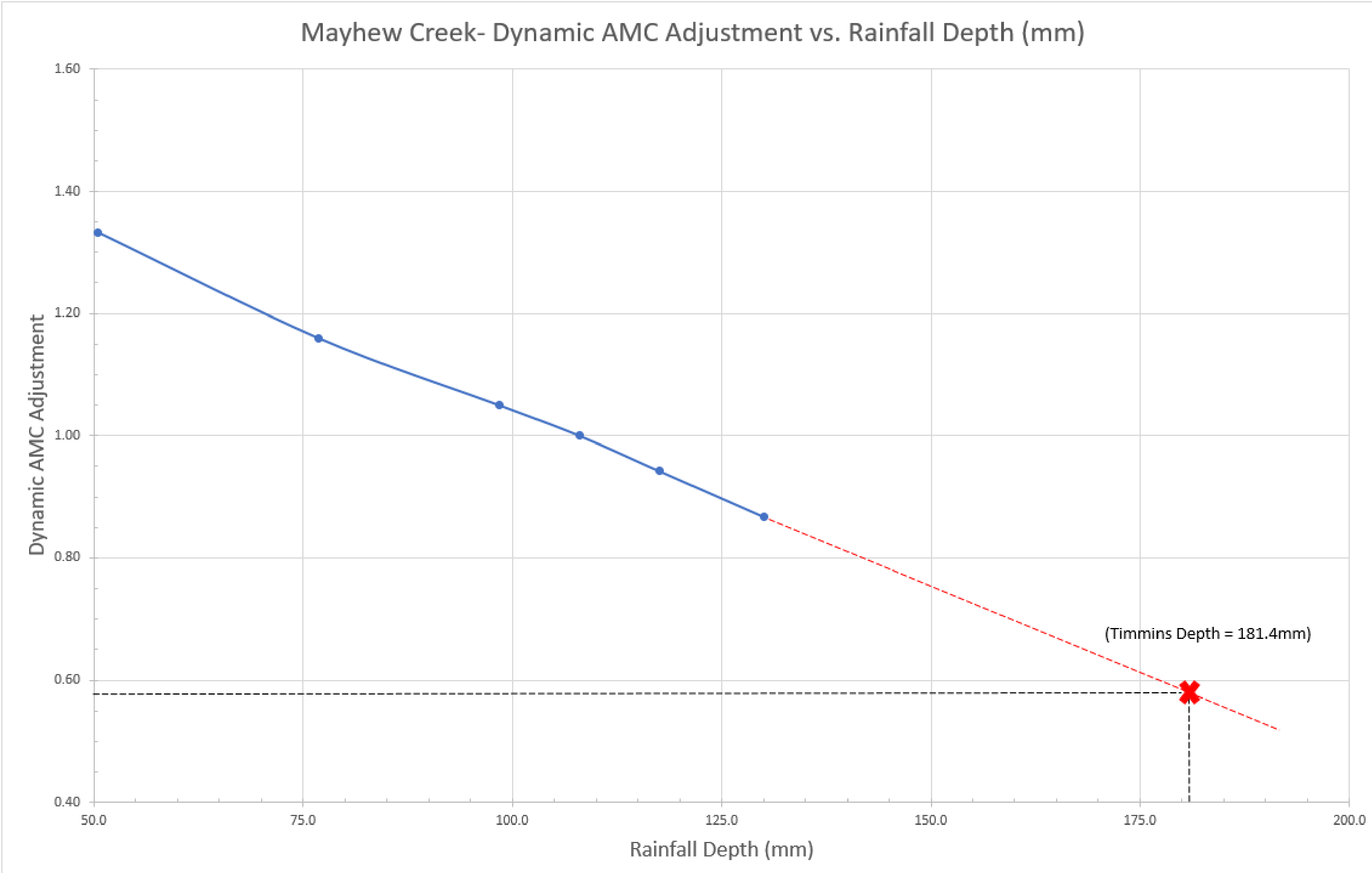



**Figure 4-12: Mayhew Creek Dynamic AMC Adjustments**

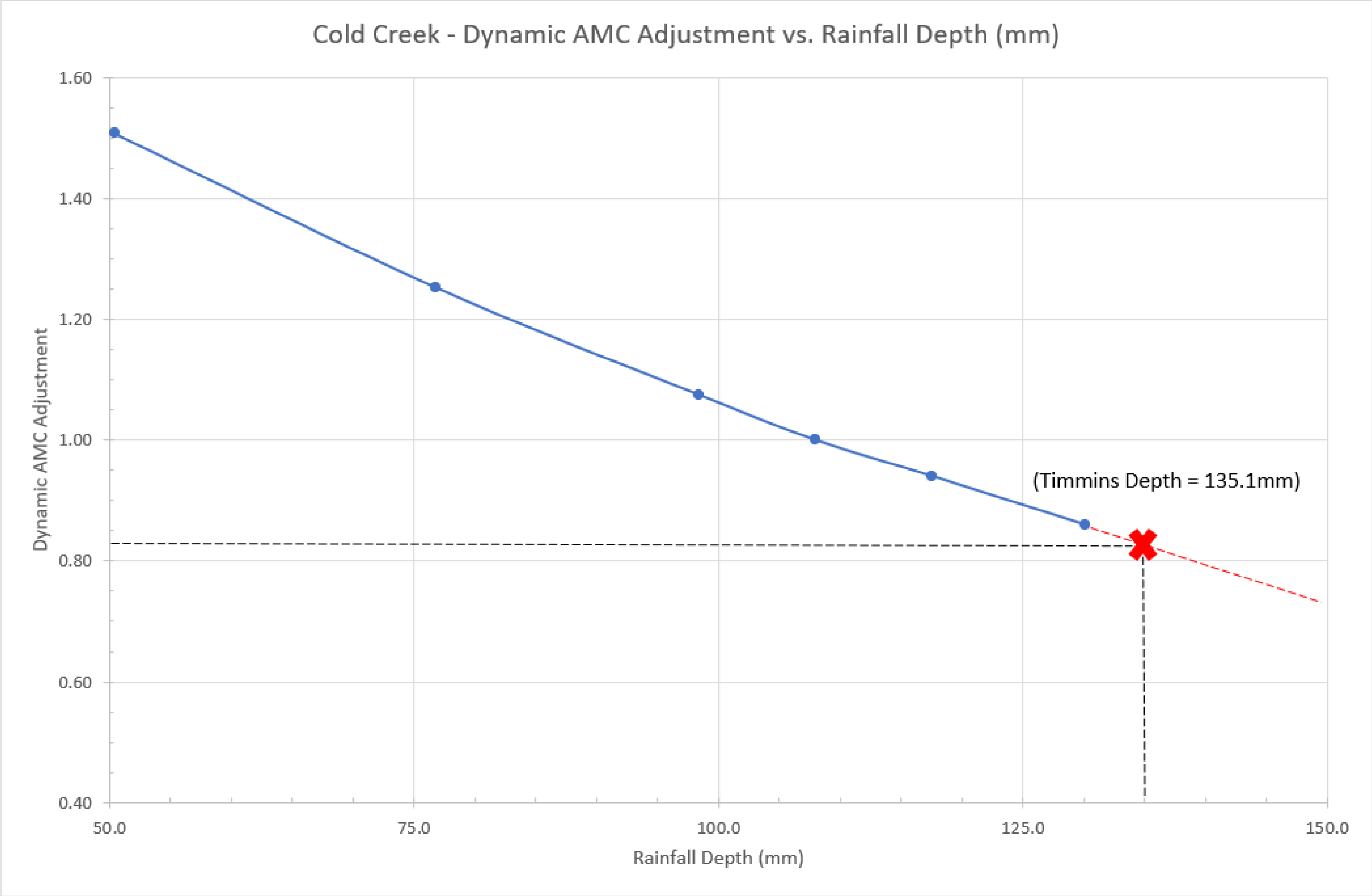



**Figure 4-13: Cold Creek Dynamic AMC Adjustments**

**Table 4-13: Mayhew Creek Input Parameters for 1% AEP**

| Catchment ID | Area ($km^2$) | Initial Abstraction | CN | Impervious (%) | Time of Concentration (hr) | Storage Coefficient (hr) |
|---|---|---|---|---|---|---|
| 100 | 5.27 | 5.0 | 59.3 | 0.0 | 2.8 | 5.2 |
| 200 | 5.31 | 5.0 | 59.5 | 0.0 | 3.3 | 6.2 |
| 300 | 1.10 | 5.0 | 57.6 | 0.0 | 0.7 | 1.2 |
| 301 | 2.42 | 5.0 | 57.6 | 0.0 | 2.3 | 4.3 |
| 400 | 3.51 | 5.0 | 65.3 | 0.0 | 2.9 | 5.5 |
| 401 | 4.44 | 5.0 | 57.4 | 0.0 | 3.5 | 6.6 |
| 500 | 10.29 | 5.0 | 62.1 | 0.0 | 5.1 | 9.4 |
| 600 | 6.34 | 5.0 | 52.2 | 23.0 | 2.4 | 4.5 |

**Table 4-14: Cold Creek Input Parameters for 1% AEP**

| 1% AEP Input Parameters | | | | | | |
|---|---|---|---|---|---|---|
| Catchment ID | Area (km2) | Initial Abstraction | CN | Impervious (%) | Time of Concentration (hr) | Storage Coefficient (hr) |
| 100 | 7.0 | 5.0 | 52.9 | 5.0 | 6.5 | 9.7 |
| 200 | 12.4 | 5.0 | 50.5 | 3.0 | 11.7 | 17.5 |
| 300 | 34.0 | 5.0 | 49.9 | 3.0 | 16.1 | 24.0 |
| 400 | 16.8 | 5.0 | 53.3 | 3.0 | 12.4 | 18.5 |
| 500 | 24.0 | 5.0 | 49.2 | 3.0 | 16.2 | 24.3 |
| 600 | 9.0 | 5.0 | 45.4 | 0.0 | 9.0 | 13.5 |
| 701 | 23.6 | 5.0 | 51.9 | 0.0 | 8.3 | 12.4 |
| 702 | 11.7 | 5.0 | 44.6 | 0.0 | 7.6 | 11.3 |
| 800 | 37.2 | 5.0 | 51.9 | 0.0 | 12.9 | 19.3 |
| 900 | 13.6 | 5.0 | 53.1 | 0.0 | 6.2 | 9.2 |
| 1,000 | 26.4 | 5.0 | 49.4 | 0.0 | 18.0 | 26.9 |
| 1,100 | 16.9 | 5.0 | 45.8 | 0.0 | 20.6 | 30.9 |
| 1,200 | 11.6 | 5.0 | 47.7 | 0.0 | 12.7 | 19.0 |
| 1,300 | 13.2 | 5.0 | 55.7 | 0.0 | 10.5 | 15.7 |

#### 4.3.5.3 Calibration to 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 CNs, 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. The corrected model results were presented in Figure 4-10 and Figure 4-11 and as the red line.

The Timmins events for each of Mayhew and Cold creeks will follow the trajectory of the dashed red lines with peak flows determined from the corrected models as indicated by the red X.

The dynamic adjustments are indicated in Figure 4-12 and Figure 4-13.

The calibrated HMS model compares well with the observed flow records (see below). The *validation* component is described in the following subsection. The observed values were corrected for base flow.

**Table 4-15: Mayhew Creek – Comparison of Observed vs. Modeled Results for Calibration / Validation Events**

| Parameter | | Sep. 2004 | | Sep. 2021 | |
|---|---|---|---|---|---|
| | | Observed | Modeled | Observed | Modeled |
| 1 | $Q_{peak}$ ($m^3/s$) | 15.9 | 16.1 | 5.4 | 6.5 |
| 2 | Time to Peak (hr) | 17.7 | 15.8 | 38.5 | 37.5 |
| 3 | Volume (ha-m) | 88.5 | 84.8 | 43.5 | 51.4 |
| 4 | Soil Conditions | Dry | AMC I | Dry | AMC I |

**Table 4-16: Cold Creek – Comparison of Observed vs. Modeled Results for Calibration / Validation Events**

| Parameter | | Sep. 2004 | | Sep. 2021 | |
|---|---|---|---|---|---|
| | | Observed | Modeled | Observed | Modeled |
| 1 | $Q_{peak}$ ($m^3/s$) | 33.2 | 33.4 | 23.5 | 25.5 |
| 2 | Time to Peak (hr) | 22.5 | 22.2 | 45.4 | 42.1 |
| 3 | Volume (ha-m) | 287 | 306 | 327 | 295 |
| 4 | Soil Conditions | Dry | AMC I | Dry | AMC I |

### *4.3.6 Validation*

LTC provided precipitation and flow gauge data for a large rainfall event that recently occurred in September of 2021; given the high rainfall depths this event was selected as the validation event for both Mayhew and Cold Creek. Recall the precipitation data for this event was illustrated in Figure 4-3.

Shortly after the September 2021 storm event, LTC's Water Resources Manager issued a memo (**see Appendix I**) 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. Cold Creek was one of the two creeks reaching its 2-yr flow.

Similar to Frances, an AMC I condition was applied given that the minimum instantaneous discharge for the year 2021 occurred a few weeks prior on the 4$^{th}$ of September, suggesting dry moisture conditions leading up to the September 22$^{nd}$ rainfall event. There was also no significant rainfall in the days prior to the storm event.

Although small, the flow measurement preceding the validation storm is considered baseflow for the purpose of the calibration, and is subtracted from the flow readings associated with the September 22-23$^{rd}$ rainfall event.

A comparison of peak flow results between observed and modeled outputs for the validation storm is provided below. The comparisons show that the modeled response matches the general shape of the observed readings and provides a reasonable representation for both creek systems.

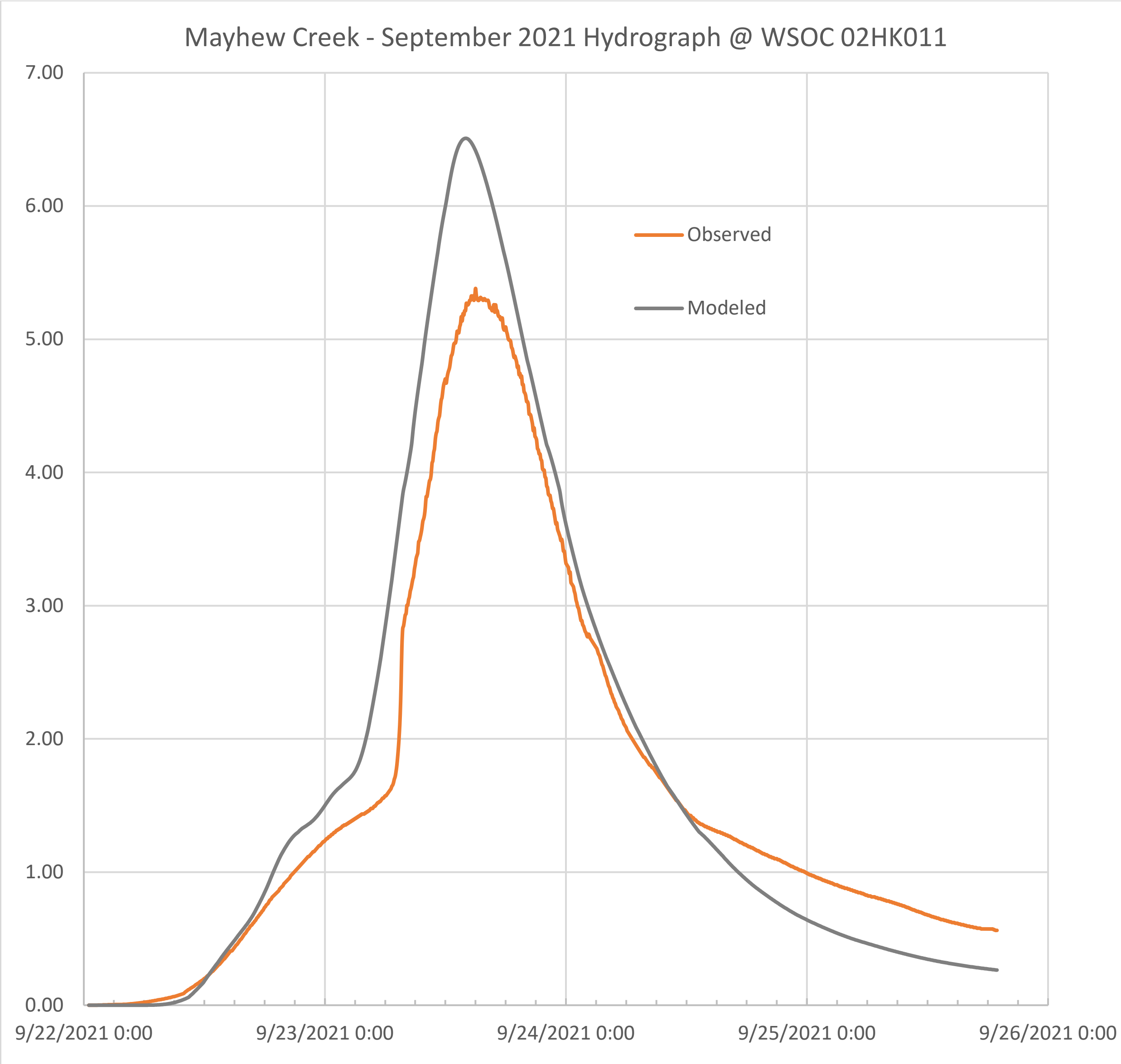



**Figure 4-14: Mayhew Creek Validation Event Hydrograph Comparison**

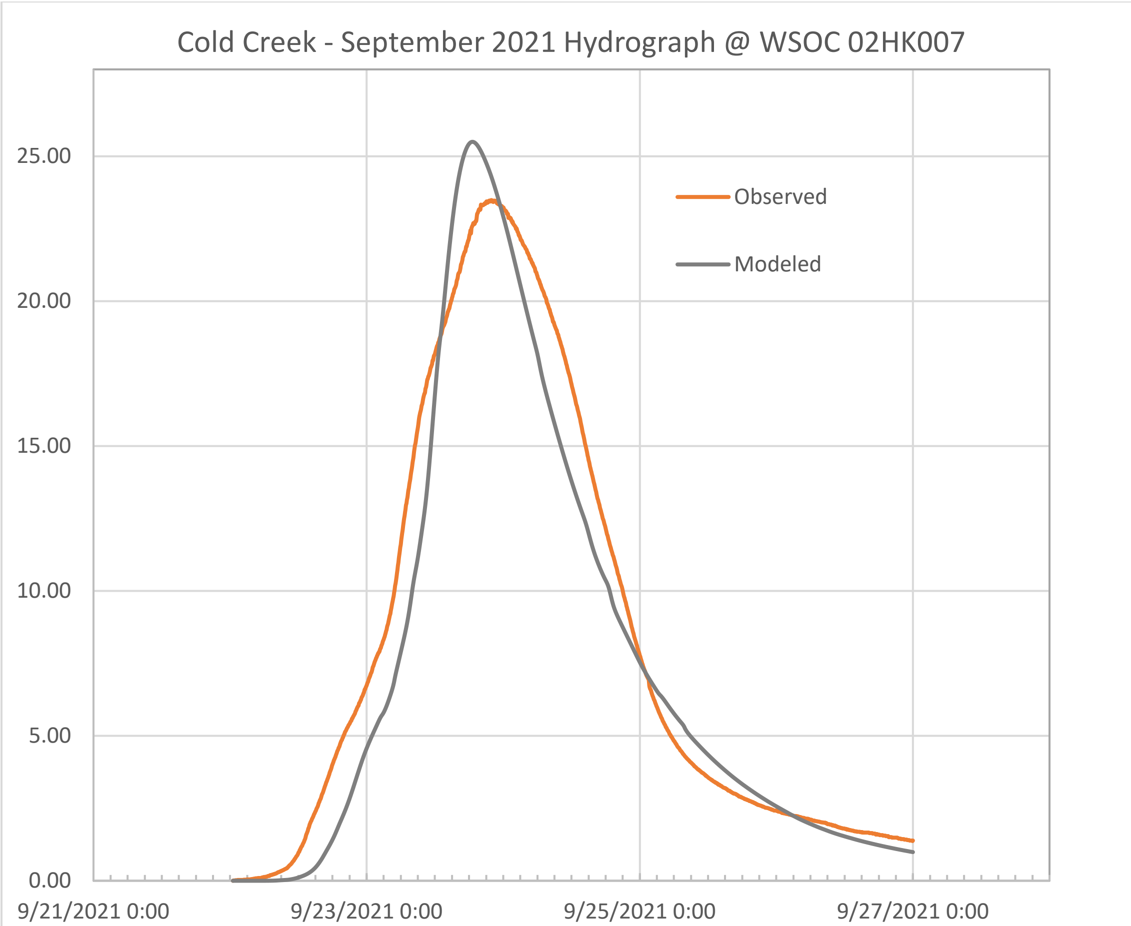



**Figure 4-15: Cold Creek Validation Event Hydrograph Comparison**

## 4.4 Index Flood Analysis

Jewell employed the Index Flood Analysis 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. Twelve regions across the province were identified as having similar characteristics and a regression curve was developed for each region (see Figure 4-16). Note that the Mayhew and Cold Creek watersheds are located near the boundary of Regions 1, 8 and 9. Regions 1 and 9 produced peak flows that compare well with the GFA analysis and were therefore included in the presentation of peak flows in Section 4.6.

The 2-yr flows are resolved directly from the equation using the constant and exponent from Table 4-17. Other return period flows may be derived from the 2-yr flow by multiplying with the factors provided in Table 4-18.

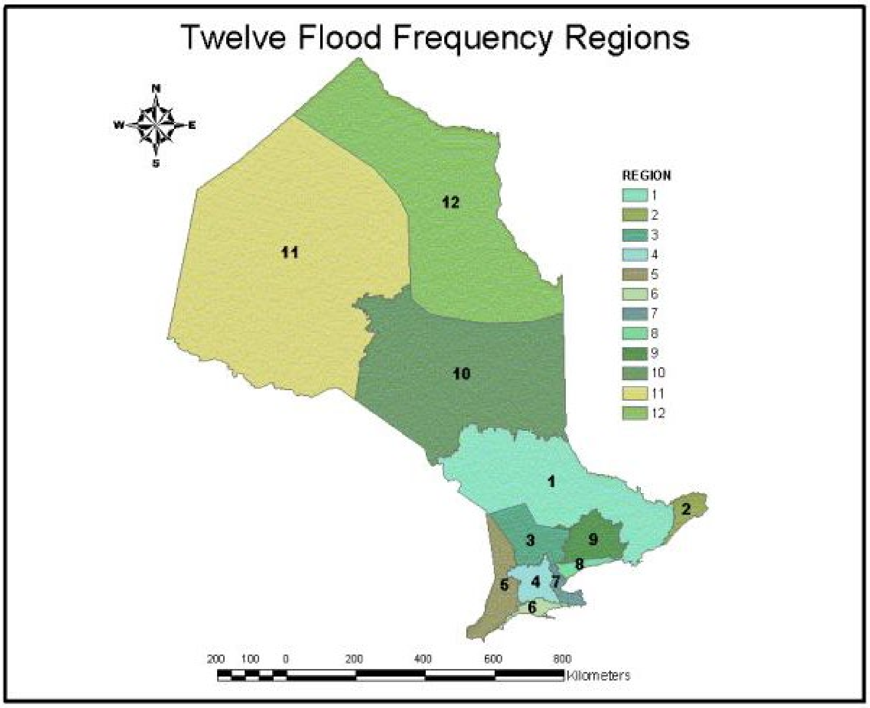



**Figure 4-16: Index Flood Regions**

**Equation: 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 4-17: Table of Constant (C) and Exponent (n) for use in the Modified Index Flood Equation (Ministry of Natural Resources and Forestry, 2020)**

| 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 4-18: Ratio of Various Flood Frequencies to $Q_2$ (Ministry of Natural Resources and Forestry, 2020)**

| 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 |
| **12** | 0.94 | 1.00 | 1.22 | 1.38 | 1.52 | 1.68 | 1.80 | 1.90 | 2.05 |

**Table 4-19: Limitation of Application of Index Flood Method Based on Drainage Area (Ministry of Natural Resources and Forestry, 2020)**

| 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.*

Both Mayhew and Cold Creek are 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 Quinte West. Jewell followed the Ontario MNRFs recommendation of obtaining the value for the 50$^{th}$ percentile of the mean annual temperature change based on the Coupled Model Intercomparison Project Phase 5 (CMIP5), Representative Concentration Pathways (RCPs) 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.

The increase in temperature results in a significant (25%) increase in precipitation volume (see Table 4-20 and Table 4-21).

Determine future estimated rainfall intensity value ($R_P$), according to the historic estimated rainfall intensity ($R_C$) and the long term (30-year mean) annual mean temperature change ($\Delta T$) using equation (1):

$$R_p = R_c \, x \, 1.07^{\Delta T} \qquad (1)$$

**Figure 4-17: Excerpt from Technical Memo with Equation for Future Estimated Rainfall Intensities**

**Table 4-20: Future Estimated Rainfall Intensities for Timmins (Regulatory) Storm for Mayhew Creek**

| 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/hr** | |
| 1 | 60 | 15 | 14.1 | 8 | 17.6 | 25.0% |
| 2 | 120 | 20 | 18.8 | 10 | 23.5 | 25.0% |
| 3 | 180 | 10 | 9.4 | 6 | 11.8 | 25.0% |
| 4 | 240 | 3 | 2.8 | 1 | 3.5 | 25.0% |
| 5 | 300 | 5 | 4.7 | 3 | 5.9 | 25.0% |
| 6 | 360 | 20 | 18.8 | 10 | 23.5 | 25.0% |
| 7 | 420 | 43 | 40.4 | 23 | 50.5 | 25.0% |
| 8 | 480 | 20 | 18.8 | 10 | 23.5 | 25.0% |
| 9 | 540 | 23 | 21.6 | 12 | 27.0 | 25.0% |
| 10 | 600 | 13 | 12.2 | 6 | 15.3 | 25.0% |
| 11 | 660 | 13 | 12.2 | 7 | 15.3 | 25.0% |
| 12 | 720 | 8 | 7.5 | 4 | 9.4 | 25.0% |
| **Total** | | **193** | **181** | **100** | **227** | **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 global 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 assessment of the data, Jewell expects that climate change would have a more noticeable impact on precipitation-driven runoff events rather than a snowmelt-driven runoff event.

**Table 4-21: Future Estimated Rainfall Intensities for Timmins (Regulatory) Storm for Cold Creek**

| Time | Historic Intensity | Historic Adjusted ($R_c$) | Percent of 12 hour | Future Estimated Intensity ($R_p$) | % Increase in Intensity |
|---|---|---|---|---|---|
| Hour | mm/hr | mm/hr | % | mm/hr | |
| 1 | 15 | 10.5 | 8 | 13.1 | 25.0% |
| 2 | 20 | 14.0 | 10 | 17.5 | 25.0% |
| 3 | 10 | 7.0 | 6 | 8.8 | 25.0% |
| 4 | 3 | 2.1 | 1 | 2.6 | 25.0% |
| 5 | 5 | 3.5 | 3 | 4.4 | 25.0% |
| 6 | 20 | 14.0 | 10 | 17.5 | 25.0% |
| 7 | 43 | 30.1 | 23 | 37.6 | 25.0% |
| 8 | 20 | 14.0 | 10 | 17.5 | 25.0% |
| 9 | 23 | 16.1 | 12 | 20.1 | 25.0% |
| 10 | 13 | 9.1 | 6 | 11.4 | 25.0% |
| 11 | 13 | 9.1 | 7 | 11.4 | 25.0% |
| 12 | 8 | 5.6 | 4 | 7.0 | 25.0% |
| **Total** | **193** | **135.1** | **100** | **169** | **25.0%** |

The stream flow gauge data presented in Section 4.2 generally illustrates that the expected return period flows would occur during a freeze-thaw/snowmelt condition. This is because most instantaneous annual peaks occur in the spring months. These 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. Therefore, climate change is expected to have a greater impact on precipitation-driven flood events rather than spring-melt events. With this understanding, Jewell followed the guidance and information from the federate climate data portal.

## 4.6 Presentation of Peak Flows

### *4.6.1 General Frequency Analysis –Peak Flows*

The CFA general frequency analysis results for each return period event is summarized in Table 4-22.

The results in Table 4-22 represent the expected peak flows at the Mayhew and Cold Creek flow gauge locations. For return period flows that include the entire watershed, a transposition of flows is required (see Figure 4-18). The transposed return period flows for the full watersheds are summarized in Table 4-23.

**Table 4-22: Summary of Maximum Peak Flows from General Frequency Analysis**

| Return Period | Peak Flow ($m^3/s$) | |
|---|---|---|
| | Mayhew Creek | Cold Creek |
| 2-yr | 6.8 | 21.5 |
| 10-yr | 11.2 | 30.7 |
| 50-yr | 14.9 | 37.1 |
| 100-yr | 16.5 | 39.4 |
| 200-yr | 18.0 | 41.7 |
| 500-yr | 20.0 | 44.5 |

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 4-18: Excerpt from MTO Online Drainage Manual**

**Table 4-23: Summary of Maximum Peak Flows with Transposition of Flows Applied to Account for Full Watershed Area**

| Return Period | Peak Flow ($m^3/s$) | |
|---|---|---|
| | Mayhew Creek | Cold Creek |
| 2-yr | 7.7 | 30.6 |
| 10-yr | 12.6 | 43.6 |
| 50-yr | 16.8 | 52.7 |
| 100-yr | 18.6 | 56.0 |
| 200-yr | 20.3 | 59.3 |
| 500-yr | 22.5 | 63.3 |

### *4.6.2 Hydrologic Model – Peak Flow Summary*

The peak flows simulated in HEC-HMS for each storm event at their respective node of interest are summarized in Tables 4-24 and 4-25. Recall that the node locations are illustrated in the catchment drawings in Appendix B.

Future full build-out conditions for the Trenton and Frankford urban areas (see Appendix A) were considered in a review of the regulatory peak flows. Since the regulatory storm is the Timmins event, the flows would not receive full quantity controls in future development scenarios.

The communities of Trenton and Frankford are located at the downstream end of the Mayhew and Cold Creek watersheds. In a full build-out scenario, the increase in hardened surfaces within the urban boundary increases the peak flows from local developments. However, the peak runoff from these development areas have shorter times to peak relative to their existing condition. The result is a separation between the earlier peak from the urbanized areas relative to the larger peak from the remainder of the watershed, creating a lesser peak flow in the regulatory storm event for Mayhew Creek. The future development area within the Cold Creek watershed is minimal and produces no appreciable change in flows. Therefore, the existing conditions are used for the floodplain mapping update since this condition produces equivalent or slightly larger peak flows in the Mayhew and Cold Creek drainage systems.

**Table 4-24: Mayhew Creek - Modelled Peak Flows at Each Hydrologic Node of Interest ($m^3/s$)**

| Hydrologic Node | Annual Exceedance Probability | | | | | | Timmins | Timmins + Climate Change |
|---|---|---|---|---|---|---|---|---|
| | 50% | 10% | 2% | 1% | 0.5% | 0.2% | | |
| A | 2.5 | 3.8 | 4.9 | 5.3 | 5.4 | 5.7 | 6.7 | 9.8 |
| B | 4.8 | 7.2 | 9.2 | 10.0 | 10.2 | 10.8 | 12.9 | 19.0 |
| C | 3.8 | 7.2 | 9.7 | 10.6 | 10.9 | 11.6 | 14.0 | 20.7 |
| D | 4.4 | 8.4 | 11.4 | 12.5 | 13.0 | 13.8 | 17.0 | 25.3 |
| E | 6.9 | 11.4 | 15.2 | 16.8 | 18.5 | 20.5 | 32.9 | 54.8 |
| F | 6.9 | 11.4 | 15.2 | 16.8 | 18.5 | 20.5 | 33.0 | 54.2 |
| G | 9.6 | 14.6 | 19.6 | 21.0 | 22.9 | 25.4 | 40.2 | 69.9 |

**Table 4-25: Cold Creek – Modelled Peak Flows at Each Hydrologic Node of Interest ($m^3/s$)**

| Hydrologic Node | Annual Exceedance Probability | | | | | | Timmins | Timmins + Climate Change |
|---|---|---|---|---|---|---|---|---|
| | 50% | 10% | 2% | 1% | 0.5% | 0.2% | | |
| A | 4.6 | 6.3 | 7.4 | 7.8 | 8.3 | 8.8 | 9.8 | 14.5 |
| B | 8.2 | 11.6 | 14.1 | 14.9 | 15.9 | 17.0 | 17.9 | 26.6 |
| C | 15.0 | 21.1 | 25.5 | 27.1 | 28.9 | 30.8 | 32.7 | 49.1 |
| D | 20.7 | 29.3 | 35.5 | 37.7 | 40.2 | 42.9 | 45.8 | 69.0 |
| E | 21.8 | 31.0 | 37.6 | 40.0 | 42.7 | 45.6 | 48.8 | 73.6 |
| F | 24.6 | 35.1 | 42.7 | 45.5 | 48.5 | 51.8 | 55.5 | 83.3 |
| G | 27.0 | 38.6 | 46.8 | 49.9 | 53.3 | 56.9 | 60.8 | 91.4 |
| H | 31.1 | 44.7 | 54.4 | 58.0 | 61.9 | 66.2 | 70.9 | 106.3 |
| I | 32.1 | 46.3 | 56.4 | 60.1 | 64.3 | 68.8 | 73.0 | 109.8 |
| J | 32.7 | 47.3 | 57.7 | 61.6 | 65.9 | 70.5 | 74.7 | 112.6 |

The selected peak flows for the Mayhew and Cold Creek floodplain mapping updates are summarized in Table 4-26 and Table 4-27. 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 tables 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 flow can only be calculated using rainfall-runoff software programs.

**Table 4-26: Summary of Peak Flows Applied in 2024 Mayhew Creek Floodplain Mapping Update**

| Return Period | Peak Flow ($m^3/s$) |
| --- | --- |
| 50-yr | 19.6 |
| 100-yr | 21.0 |
| 200-yr | 22.9 |
| 500-yr | 25.4 |
| ***Timmins** | **40.2** |
| Timmins + Climate Change | 69.9 |

**Denotes regulatory storm event*

**Table 4-27: Summary of Peak Flows Applied in 2024 Cold Creek Floodplain Mapping Update**

| Return Period | Peak Flow ($m^3/s$) |
| --- | --- |
| 50-yr | 57.7 |
| 100-yr | 61.6 |
| 200-yr | 65.9 |
| 500-yr | 70.5 |
| ***Timmins** | **74.7** |
| Timmins + Climate Change | 112.6 |

**Denotes regulatory storm event*

Table 4-28 and Table 4-29 provide a comparison of peak flows for each of the hydrologic modeling methods. These tables summarize results from the following methodologies in addition to historical rates:

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

**Table 4-28: Summary of Peak Flows from Alternative Methods for Mayhew Creek Outlet ($m^3/s$)**

| Return Period | 1981 Crysler & Lathem | 2015 Tremur Lake DSR * | GFA | Index Flood | | HEC-HMS |
|---|---|---|---|---|---|---|
| | | | | Region 1 | Region 9 | |
| 50 | 9.1 | 16.3 | 16.8 | 14.4 | 14.4 | 19.6 |
| 100 | 12.8 | 19.1 | 18.6 | 15.8 | 16.1 | 21.0 |
| 200 | - | - | 20.3 | 17.2 | 17.7 | 22.9 |
| 500 | - | - | - | 19.1 | 19.9 | 25.4 |
| **Timmins** | 132.9 | 45.5 | - | - | **-** | **40.2** |
| Timmins + Climate Change | | | | | | 69.9 |

** Tremur Lake values Tremur Lake Dam*

**Table 4-29: Summary of Peak Flows from Alternative Methods for Cold Creek Outlet ($m^3/s$)**

| Return Period | 1981 TSH | GFA | Index Flood | | HEC-HMS |
|---|---|---|---|---|---|
| | | | Region 1 | Region 9 | |
| 50 | 103.0 | 52.7 | 68.8 | 72.5 | 57.7 |
| 100 | 142.0 | 56.0 | 75.5 | 81.1 | 61.6 |
| 200 | - | 59.3 | 82.5 | 89.4 | 65.9 |
| 500 | - | 63.3 | 91.8 | 100.5 | 70.5 |
| ***Timmins** | 210.0 | - | - | - | **74.7** |
| Timmins + Climate Change | | | | | 112.6 |

# 5 Hydraulics – Mayhew Creek

The hydraulic analysis was prepared using HEC-RAS version 6.4.1. The hydrology results from the hydrology model were applied in the HEC-RAS model to delineate the flood hazard limits for the Mayhew Creek floodplain mapping update.

The Mayhew Creek floodplain study area is defined by a well-defined channel, two reservoirs imposed by the Tremur Lake Dam and Old Mill Dam, a man-made diversion channel, and a potential flood risk predominantly within the urban areas near the outlet to the Trent River. This section describes the following as they relate to Mayhew Creek:

- bathymetry,
- cross-sections,
- storage impacts,
- bridge/culvert crossings, and
- spill locations.

Model sensitivities and a comparison of historical and 2024 flood limits are provided in Sections 7 and 8. The identification of buildings within the flood hazard limit is also discussed in Section 8.

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

The LiDAR data described in Section 4.1.1 was supplemented by site-specific topographic and bathymetric 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 datum CGVD 2013 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.

Topographic survey of crossings (bridges and culverts) and weirs were used to supplement the LiDAR data.

The Jewell bathymetric survey comprised of 132 cross sections for Mayhew Creek (see Figures 5-1 and 5-2). The bathymetry surface was derived using these cross sections in CAD and imported as a TIF file into the terrain layer of the HEC-RAS model. The main (natural) channel from Old Mill Dam to the Trent River includes 92 of these cross sections, the diversion route has 29 cross sections, and the extension area between Tate Road and Tremur Lake includes 11 cross sections. While the bathymetry for the eleven cross sections within the extension area are available, they are replaced by the starting water level imposed by the Tremur Lake Dam.

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

- To simulate the flow in the overbank areas that are located within the Town of Trenton, including flow movement around the buildings located within the flood hazard limit.
- To investigate the spill quantities, diversion routes, and the reservoirs (Tremur Lake and Old Mill Dam) within the study area.

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

The Mayhew Creek model is comprised of 68,000 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 high number of cells is applied due to the density of the cells required to accurately represent the flood characteristics in the urban inundation areas.

The purpose of the customized mesh is to ensure accurate flow movement using a 5 second computational time step and output results set at 10-minute mapping intervals. The detailed 2D flow area established in the geometry editor of the hydraulic model provides the foundation for the dynamic mapping output. An example of the grid applied in the model is shown in Figure 5-3.

With the 2D modeling approach, cross sections are not needed to run the simulation. However, cross section water surface elevation (WSEL) plots are shown for a set of representative cross sections within the study area per the map and cross section plots shown in Appendix K. These cross-section plots are useful for reviewing WSEL results as they relate to the channel cross sections.


**Figure 5-1: Locations of Surveyed Bathymetry Sections within Main Mayhew Creek Study Area**

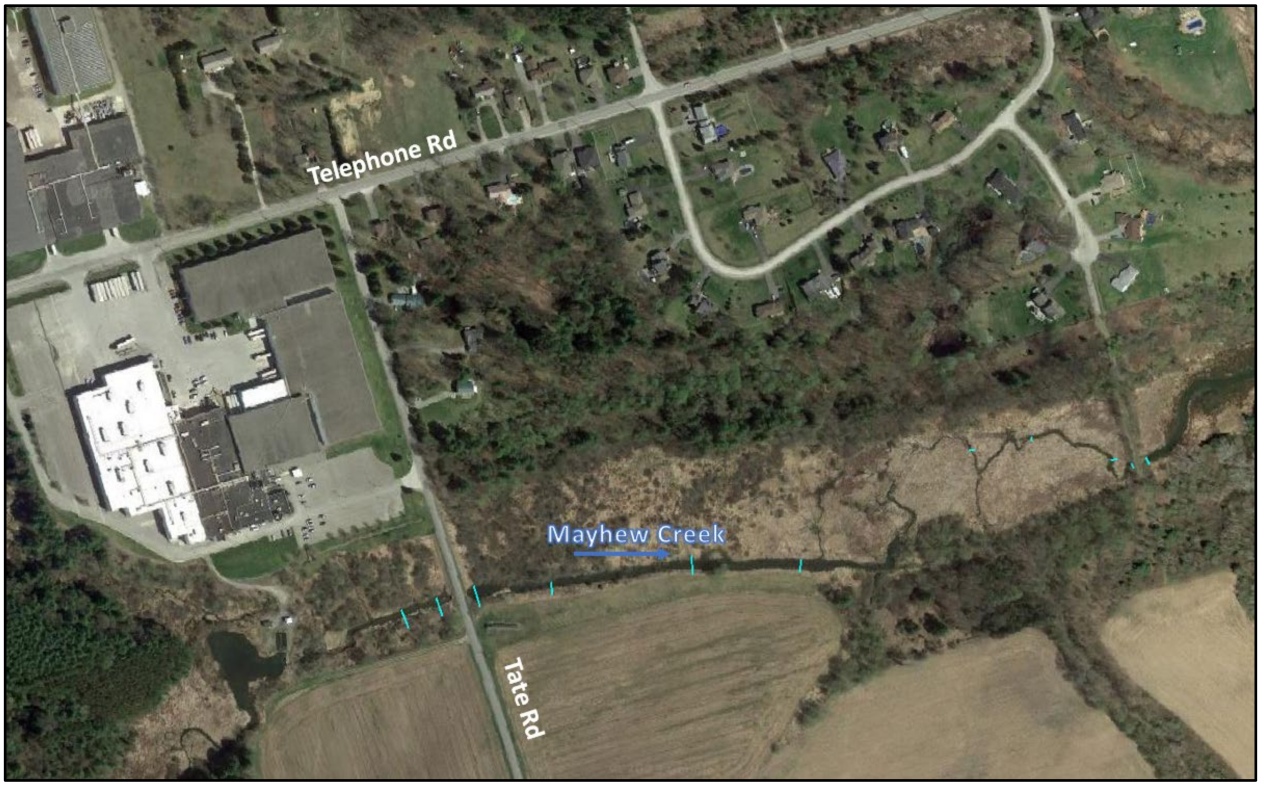



**Figure 5-2: Locations of Surveyed Bathymetry Sections within Mayhew Creek Extended Study Area**

## 5.2 Internal and External Boundary Conditions

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

The 2D unsteady flow model receives its flow data from inflow hydrographs where the incoming flows change with time. The inflow hydrographs were obtained by the tabular output in the hydrology model; each inflow BC corresponds to an inflow hydrograph. The table below summarizes the inflow peaks and their corresponding receiving catchments from Appendix B.

Inflow BC1-A and BC1-B represent the upstream inflow hydrographs. These two separate inflow boundary conditions were used to separate the hydraulic into two parts; **1)** the main study area (County Road 40 to Trent River) and **2)** the extension area (Tate Road to County Road 40).

The reason for the separation is to ensure minimal impact due to minor differences in reservoir routing between HEC-HMS and HEC-RAS software programs. The peak flows for each event were carefully calibrated using the HEC-HMS model as described in Section 4. In testing, it was found that while the HEC-RAS model routing produced very similar outflow results, it had a longer time to peak. With consideration of the importance of the timing of the hydrographs between the Tremur Lake Dam outflow and the two major tributaries from the north that connect to Mayhew Creek immediately downstream of the Tremur Lake Dam, the separation of the hydraulic model was completed to ensure peak flows were not inadvertently reduced by an artificial lag.

**Table 5-1: Inflow Boundary Condition Peak Flows**

| Inflow BC | Receiving Catchments | Peak Flow ($m^3/s$) | |
|---|---|---|---|
| | | 1% AEP | Timmins |
| 1 | 100-200-300-301-400 | 16.0 | 21.8 |
| 2 | 401 | 3.5 | 4.8 |
| 3 | 500 | 7.0 | 9.5 |
| 4 | 600 | 9.2 | 13.2 |

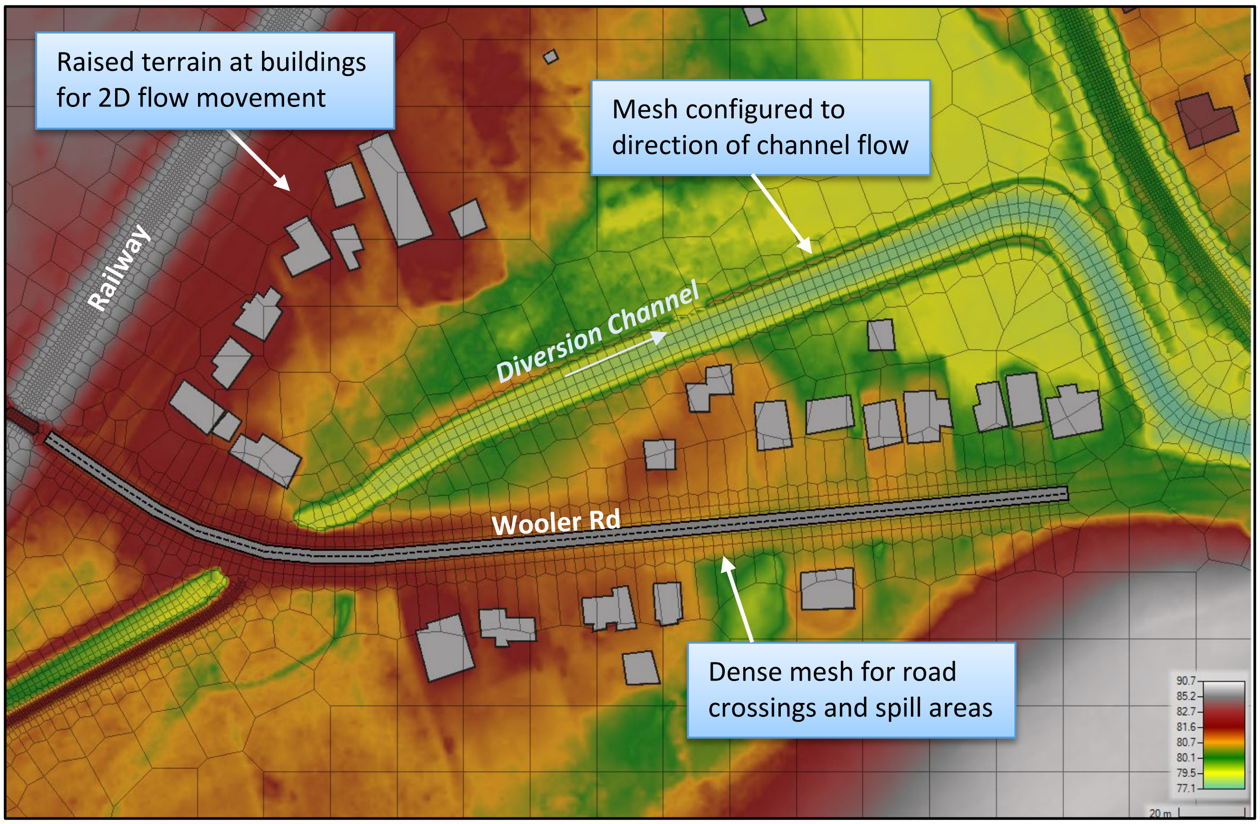



**Figure 5-3: Example of Geometry Configuration for Mayhew Creek Model Setup**

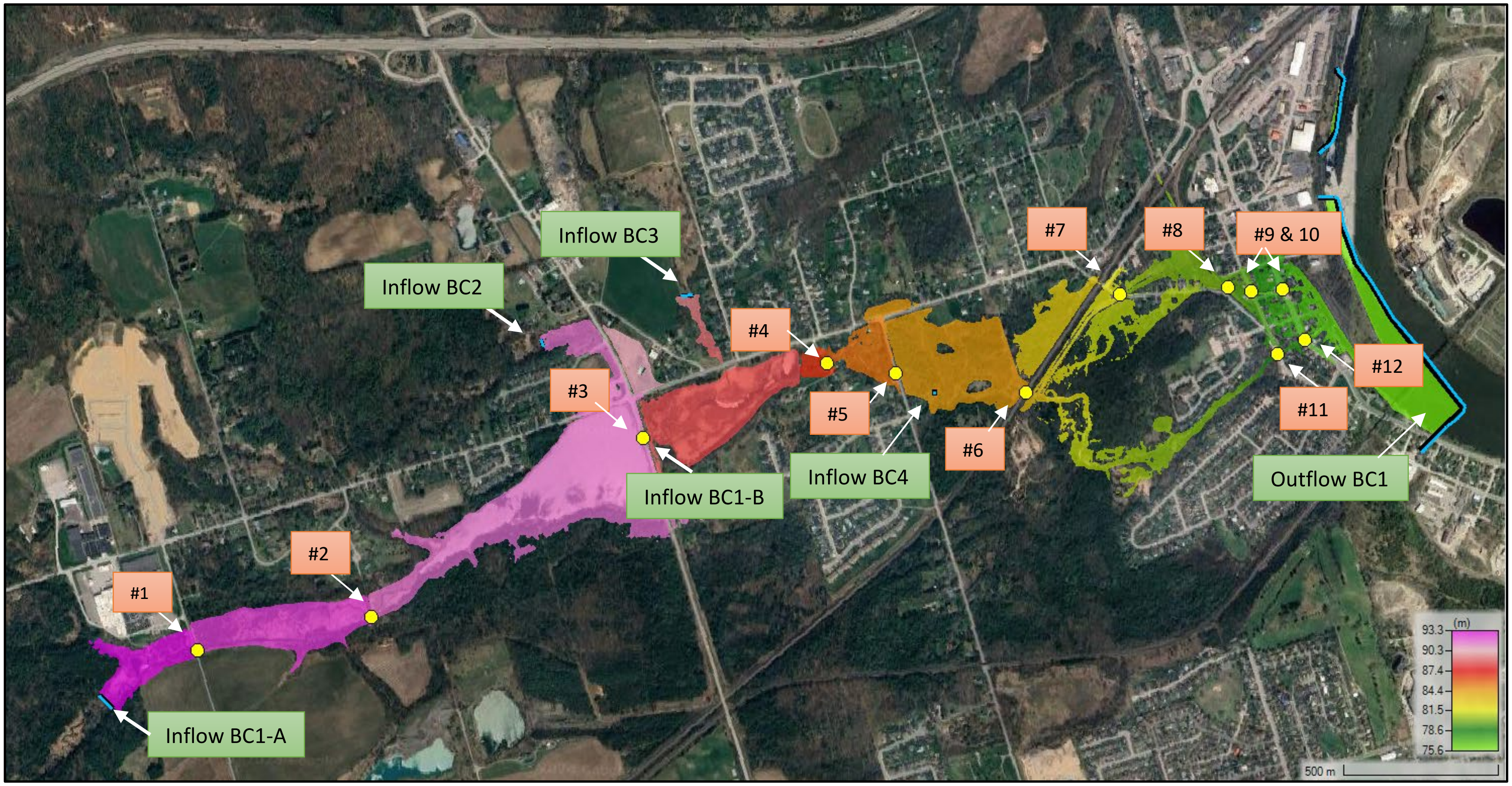



**Figure 5-4: Locations of Culvert/Bridge Crossings and Inflow/Outflow Boundary Conditions (Timmins Water Level Overlay)**

## 5.3 Culvert & Bridge Crossings

There are sixteen (16) hydraulic structures within the subject study area. Twelve (12) of these are culvert or bridge crossings as described below with their locations shown in Figure 5-4. Five (5) are flood control structures discussed separately in Section 5.4. The Tremur Lake Dam is both a crossing and a flood control structure.

The hydraulic model simulates the effects of culvert and bridge crossings on the water surface elevations at each crossing. This subsection summarizes the existing crossing configurations, stage-discharge curves, and the maximum water surface elevations at each location. The purpose of this subsection is to address the impacts of the existing road and railway infrastructure on the overall floodplain delineation. Stage and flow hydrographs for the Mayhew Creek Crossings are provided in Appendix J-2.

Note that excluding the above culverts of interest, an additional four (4) culverts were measured and included in the hydraulic model to accommodate the inflows from the tributaries for Sub-catchments 401 and 500. The tributary for Sub-Catchment 401 crosses Pine Marsh Lane, County Road 40, and Telephone Road before it outlets into the wetland/storage area upstream of the Old Mill Dam. Similarly, the tributary for Sub-Catchment 500 crosses a separate box culvert crossing of Telephone Road and outlets to the wetland/storage area upstream of Old Mill Dam (see Figure 5-5).

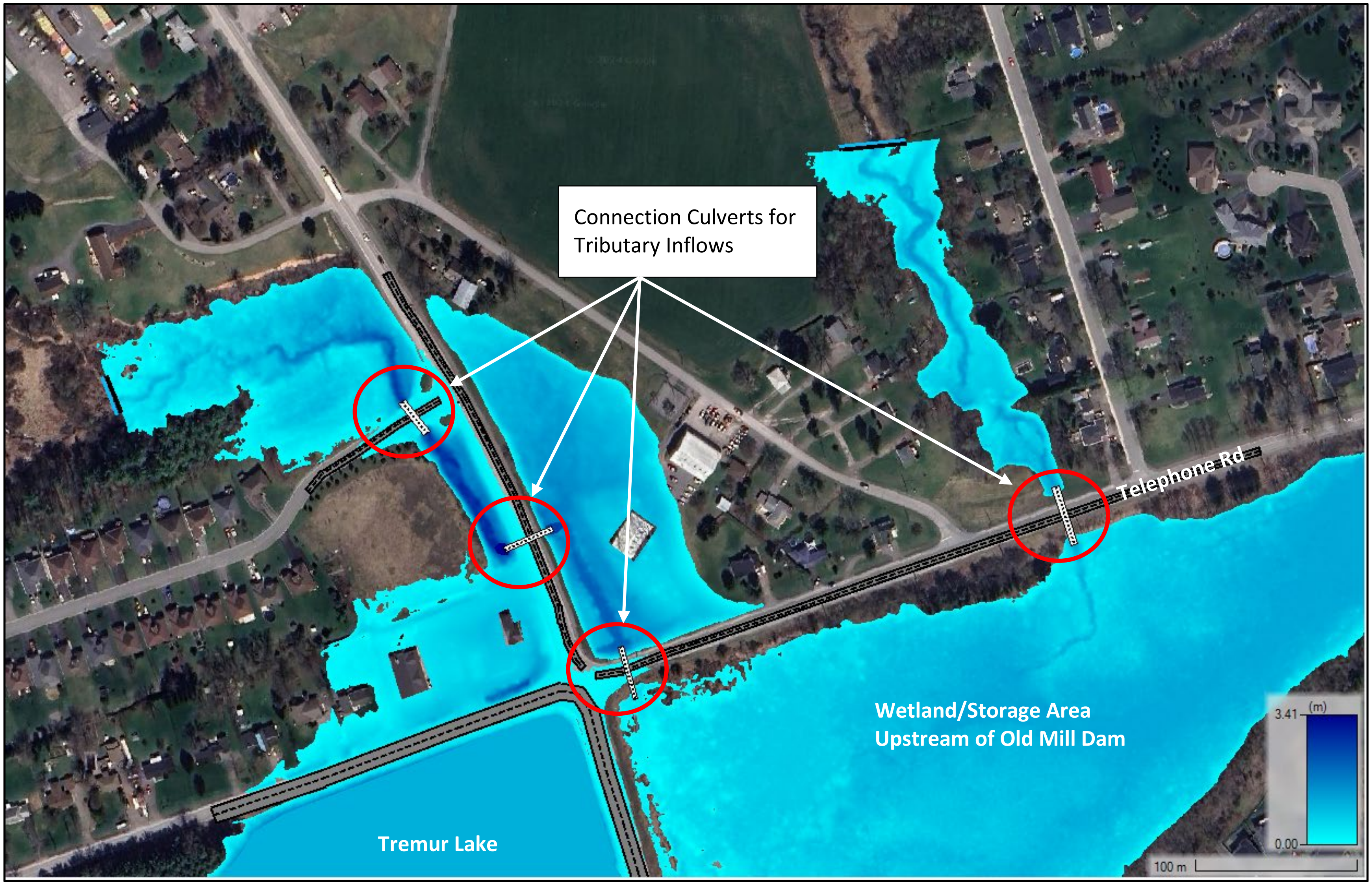



**Figure 5-5: Connection Culverts for Inflows from Tributaries from Sub-Catchments 401 and 500 (Timmins Depth Overlay)**

## *Crossing #1: Tate Road*

The Tate Road crossing is the first road crossing within the Mayhew Creek study area (upstream to downstream) and consists of a 2.5m span by 1.9m rise CSP arch culvert (see Figure 5-6).

A summary of the Tate Road culvert is provided in Table 5-2. The stage and discharge hydrographs for this crossing are provided in Appendix J-1. In the Timmins event, majority of the runoff occurs as relief flow over the road with a depth of 0.19m. The culvert can convey slightly less than 10 $m^3/s$ in an extreme storm event.

There is a large industrial building upstream of Tate Road and north of the creek as shown in Figure 5-7. The finished floor elevation is approximately 100.30m CGVD 2013 based on the LiDAR data. The Timmins water level adjacent to the building is much lower at 93.25m. Therefore, the Timmins event presents a flood risk to road travel but the upstream building is well elevated above the backwater elevations imposed by the Tate Road crossing.

**Figure 5-6: Elevation View of Tate Road Culvert Crossing**

**Table 5-2: Tate Road Crossing Summary**

| | | | |
|---|---|---|---|
| Road Name: | | Tate Rd | |
| Span (m) = | 2.5 | Rise (m) = | 1.90 |
| Upstream Invert (m) | | Downstream Invert (m) | |
| 90.52 | | 90.48 | |
| Low Point of Road = | | 92.81 | m |
| Timmins WSEL = | | 93.22 | m |
| Maximum Relief Flow Depth (m) | | Recommended Limit = 0.3m | |
| 0.41 | | x | |
| Depth*Velocity Calculated ($m^2/s$) | | Recommended Limit = 0.8 ($m^3/s$) | |
| 0.18 | | ✓ | |

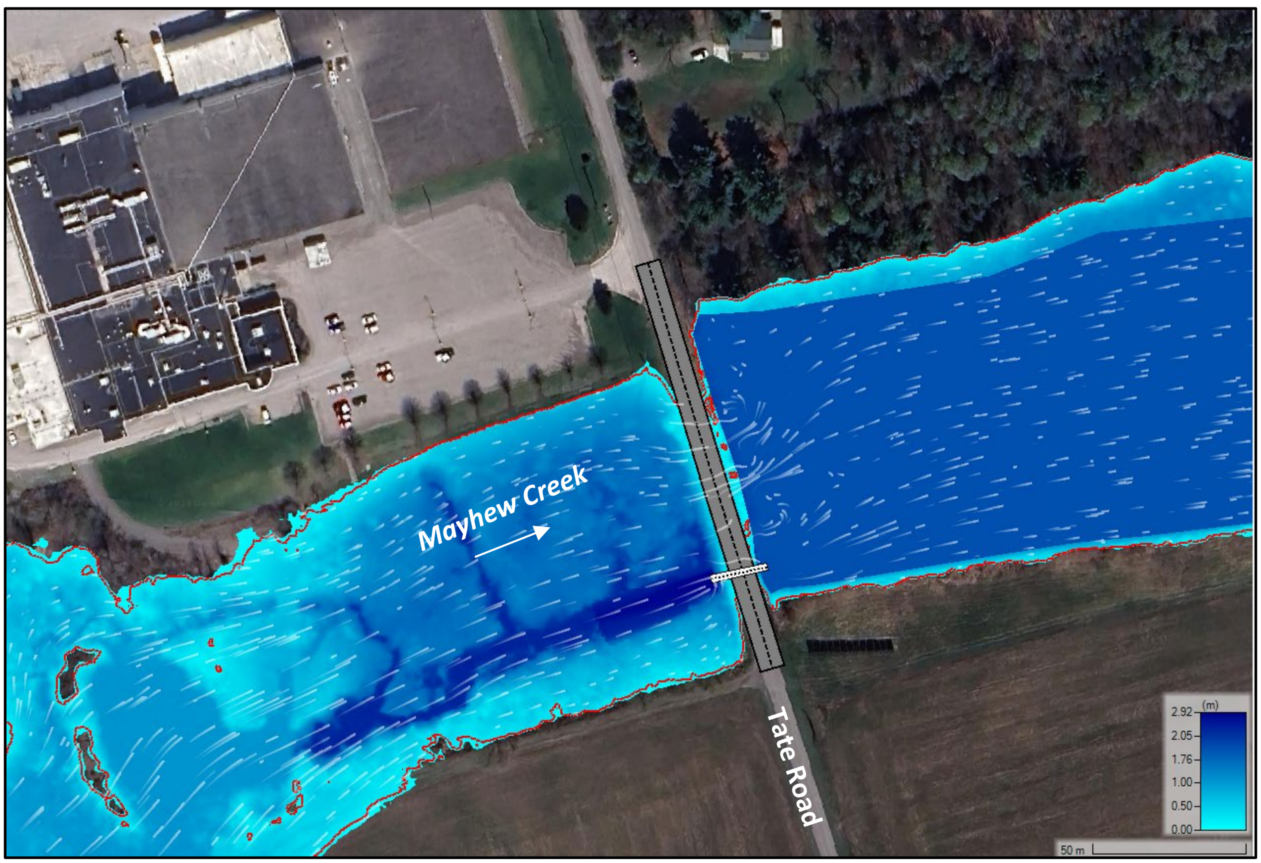



**Figure 5-7: Schematic of Timmins Inundation Area (Blue) with 1% AEP (Red) at Tate Road; Depth Overlay**

## *Crossing #2: Access Road*

There is an access road upstream of Tremur Lake that appears to be a private crossing. It is a 2.3m diameter CSP culvert as shown in Figures 5-8 and 5-9. The road provides access to a small agricultural field south of Mayhew Creek. In the Timmins event, there would be a spill over the road and the culvert has minimal hydraulic contribution since the water levels at this location are controlled by the Tremur Lake Dam. This is evident as there is minimal contraction and expansion in the flood limits on the upstream and downstream sides of the crossing. There is a natural narrowing of the channel immediately downstream of the crossing that contributes to higher velocities before runoff enters Tremur Lake (see Figure 5-10). There are no buildings within the flood hazard limits imposed by the access road.

A summary of the access road crossing is provided in Table 5-3. The stage and discharge hydrographs for this crossing are provided in Appendix J-1. The TW is influenced by the backwater from the Tremur Lake Dam. A discussion of the Tremur Lake Dam controls is provided in Section 5.4.

**Figure 5-8: View from Crossing #2: Access Road Looking Towards Tremur Lake**

**Figure 5-9: Culvert Opening for Access Road Crossing**

**Table 5-3: Crossing Summary of Private Access Road**

| | | | |
|---|---|---|---|
| Road Name: | | Access Rd | |
| Diameter (m) = | 2.3 | # of Openings | 1 |
| Upstream Invert (m) | | Downstream Invert (m) | |
| 90.48 | | 90.48 | |
| Low Point of Road = | | 91.73 | m |
| Timmins WSEL = | | 92.36 | m |
| Maximum Relief Flow Depth (m) | | Recommended Limit = 0.3m | |
| 0.63 | | x | |
| Depth*Velocity ($m^2/s$) | | Recommended Limit = 0.8 ($m^3/s$) | |
| 0.70 | | ✓ | |

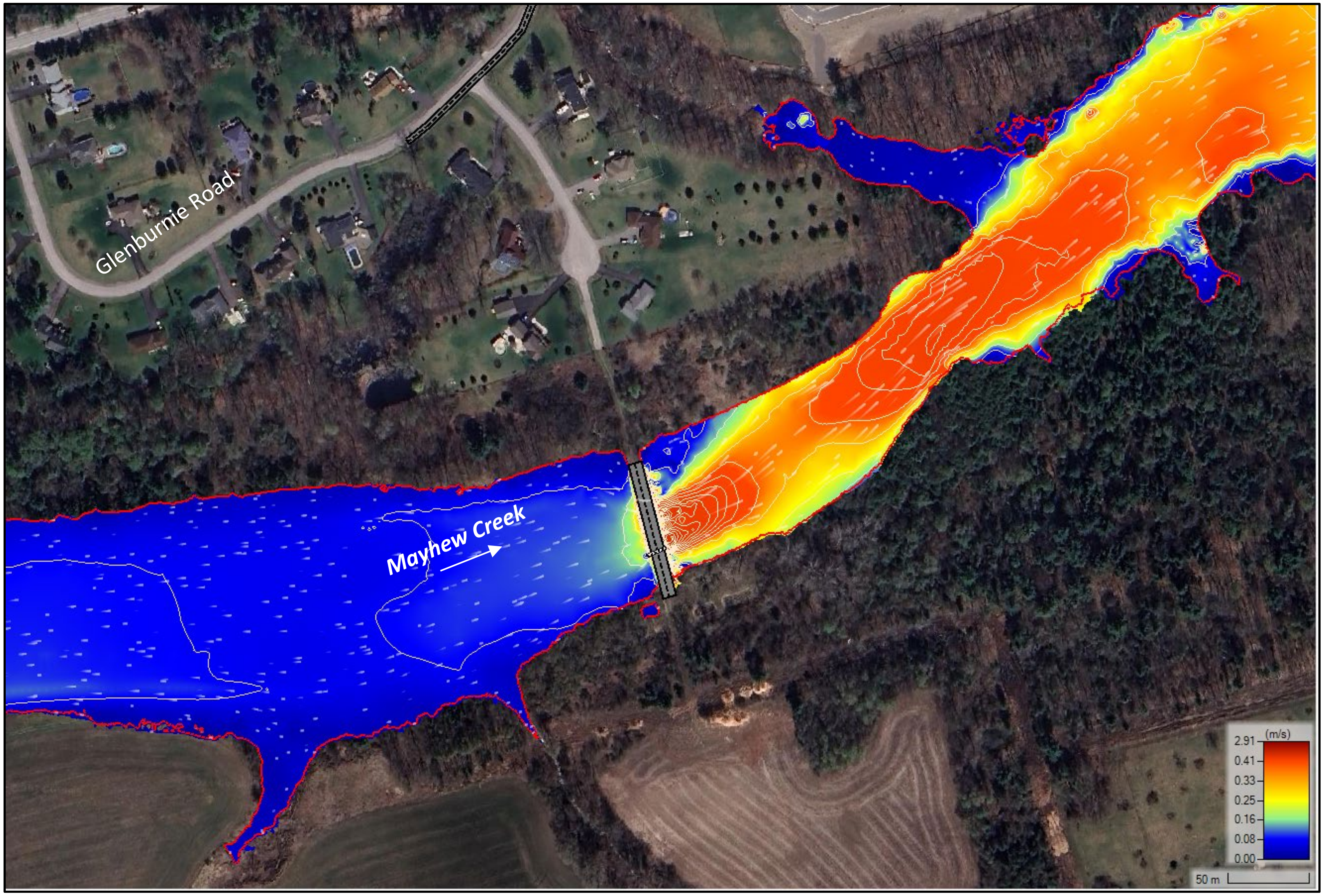



**Figure 5-10: Timmins Inundation Boundary (Blue) with 1% AEP Limits (Red) near Crossing #2: Access Road; Velocity Overlay at 0.1m Contours**

## *Crossing #3: Tremur Lake Dam*

The Tremur Lake Dam crosses Wooler Road (County Road 40) and is considered a flood control structure. It is comprised of two stoplogs bays with a 2.52m span that control the water level. The normal operating water level of 90.43m was applied in the model with a height of 1.47m of weir opening above the invert of the stoplog invert (see Figure 5-11 and 5-12). The normal operating level was established based on review of the 2015 Dam Safety Review in addition to the operation record received from the City of Quinte West.

The water levels, storage volumes, and flow attenuation from the Tremur Lake Dam are summarized in Section 5.4. For the context of the road crossing, there is widespread relief flow in the Timmins event. Approximately 85m of relief flow occurs over County Road 40 (in the natural direction of the creek). Another 230m length of relief flow occurs north, over Telephone Road (see Figures 5-13 and 5-14).

The maximum relief flow depth is 0.17m in the Timmins event. There are no houses in the floodplain of Tremur Lake itself, however the relief flow route over County Road 40 and Telephone Road puts six (6) existing buildings within the regulatory flood limit.

A summary of the Tremur Lake Dam crossing is provided in Table 5-4.


**Figure 5-11: View of Upstream Side of Tremur Lake Dam Opening**

**Figure 5-12: View from Downstream Side of Tremur Lake Dam Opening**

**Table 5-4: County Road 40 / Tremur Lake Dam Crossing Summary**

| | | | |
|---|---|---|---|
| Road Name: | | County Road 40 / Tremur Lake Dam | |
| Height of Openings (m) = | 1.47 | # of Stoplog Bays | 2 |
| Span of Openings (m) = | 2.52 | # of Stoplogs (Normal) | 10 |
| Low Point of Road = | | 91.27 | m |
| Timmins WSEL = | | 91.44 | m |
| Maximum Relief Flow Depth (m) | | Recommended Limit = 0.3m | |
| 0.17 | | ✓ | |
| Depth*Velocity ($m^2/s$) | | Recommended Limit = 0.8 ($m^3/s$) | |
| 0.08 | | ✓ | |

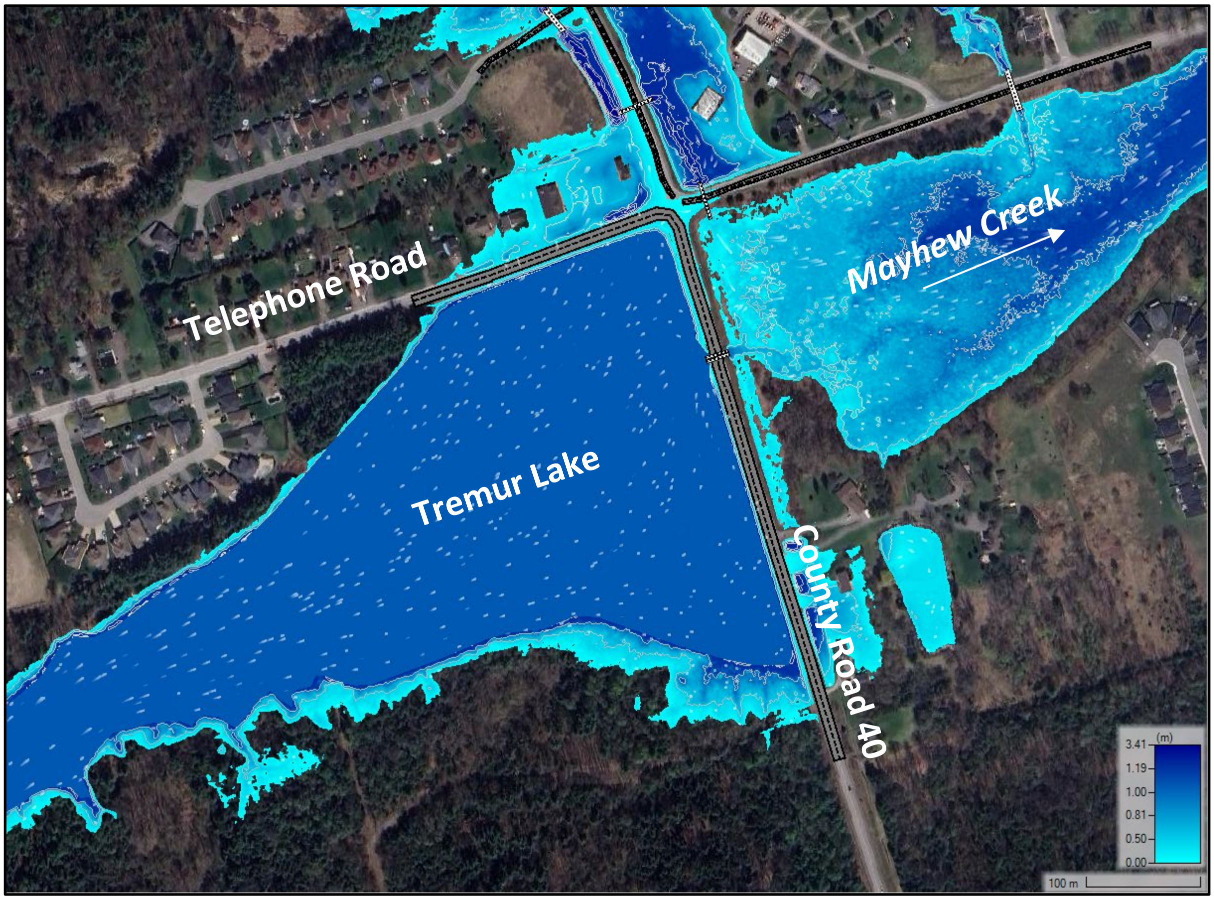



**Figure 5-13: Tremur Lake Dam with Depth Overlay in Timmins event**

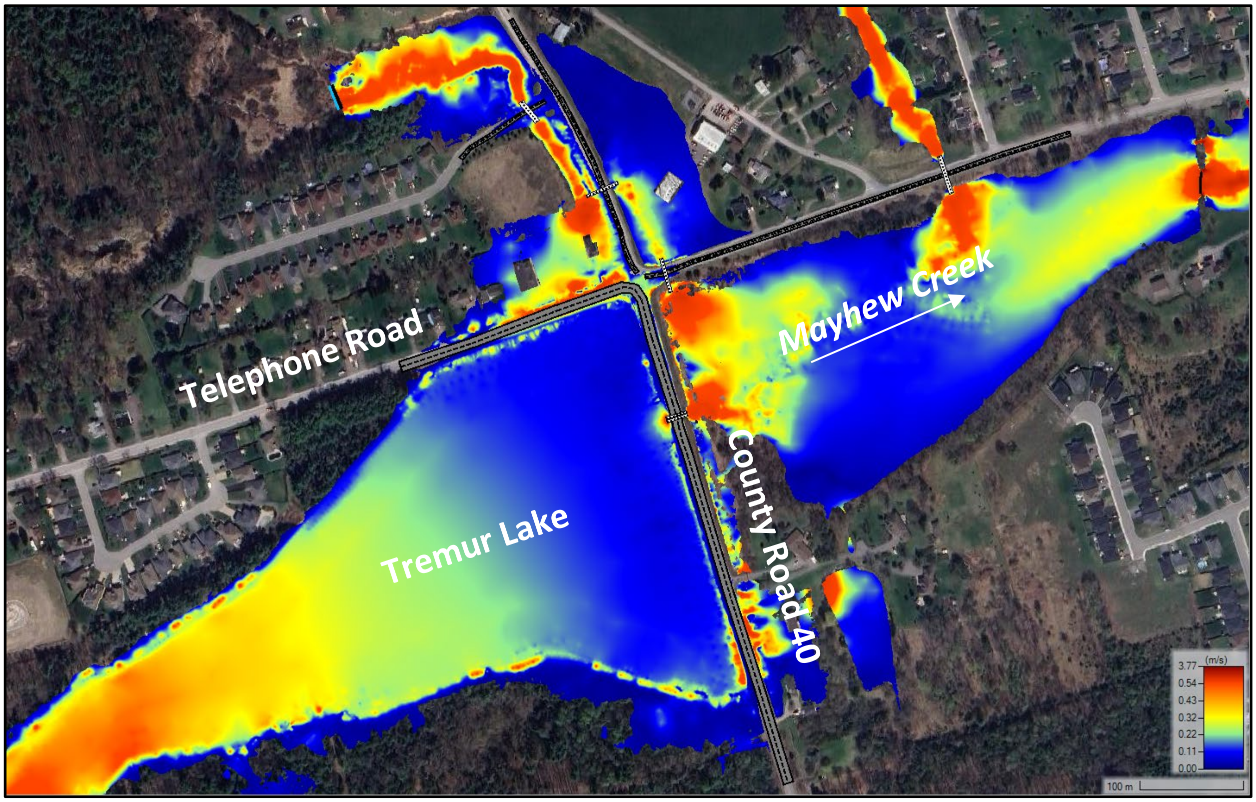



**Figure 5-14: Tremur Lake Dam with Velocity Overlay in Timmins Event**

## *Crossing #4: Hill Road*

Shortly downstream (90m) of the Old Mill Dam is the Hill Road crossing. This crossing is a 5.2m span bridge that provides local access to residents (see Figure 5-15). It is an old bridge that has been the subject of discussions at the City of Quinte West regarding its imminent replacement or rehabilitation.

Hydraulically, the bridge is relatively efficient and conveys the 1% AEP event with no overtopping and no existing buildings within its backwater area. The Timmins event overtops the road due to its extreme flows and puts one building within the flood hazard limit. The subject building is near the intersection of Hill Road and Telephone Road.

Due to the relatively deep sag of the road profile between Telephone Road and the new subdivision to the south, its relief flow depth is relatively high at 0.80m in the Timmins event despite its hydraulically efficient bridge opening.

A summary of the Hill Road bridge is provided in Table 5-5.

**Figure 5-15: Elevation View of Hill Road Bridge**

**Table 5-5: Hill Road Bridge Summary**

| Road Name: | | Hill Rd | |
|---|---|---|---|
| Span (m) = | 5.2 | Soffit (m) = | 85.96 |
| | Low Point of Road = | 86.07 | m |
| | Timmins WSEL = | 86.87 | m |
| Maximum Relief Flow Depth (m) | | Recommended Limit = 0.3m | |
| 0.80 | | x | |
| Depth*Velocity ($m^2/s$) | | Recommended Limit = 0.8 ($m^3/s$) | |
| 0.35 | | ✓ | |

## *Crossing #5: 2nd Dug Hill Road*

The 2nd Dug Hill Road crossing consists of three (3) 1.6m diameter culverts (see Figure 5-16). The low point in the road occurs just south of the intersection with Telephone Road, and is located 164m north of the 2nd Dug Hill culverts. In the 1% AEP event, there is no overtopping of the road but there are six (6) buildings within the flood hazard limit upstream of the crossing. Some of this flood risk is due to the existing buildings being located near a sharp bend in the creek and close to the creek overbank.

In the Timmins event, the flood risk is more severe and overtopping occurs over 2nd Dug Hill Road. There is additional overtopping north over Telephone Road into a low-lying area before this flood water drains back over Telephone Road on the opposite (south) side of 2nd Dug Hill and returns towards the Mayhew Creek channel (see Figure 5-17).

A summary table for the 2nd Dug Hill crossing is provided below.

**Figure 5-16: Elevation View of 2nd Dug Hill Road Culverts**

**Table 5-6: 2nd Dug Hill Road Crossing Summary**

| | | | |
|---|---|---|---|
| Road Name: | | 2nd Dug Hill Rd | |
| Diameter (m) = | 1.6 | # of Openings | 3 |
| Upstream Invert (m) | | Downstream Invert (m) | |
| 83.02 | | 83.02 | |
| Low Point of Road = | | 84.76 | m |
| Timmins WSEL = | | 85.27 | m |
| Maximum Relief Flow Depth (m) | | Recommended Limit = 0.3m | |
| 0.51 | | x | |
| Depth*Velocity ($m^2/s$) | | Recommended Limit = 0.8 ($m^3/s$) | |
| 0.34 | | ✓ | |

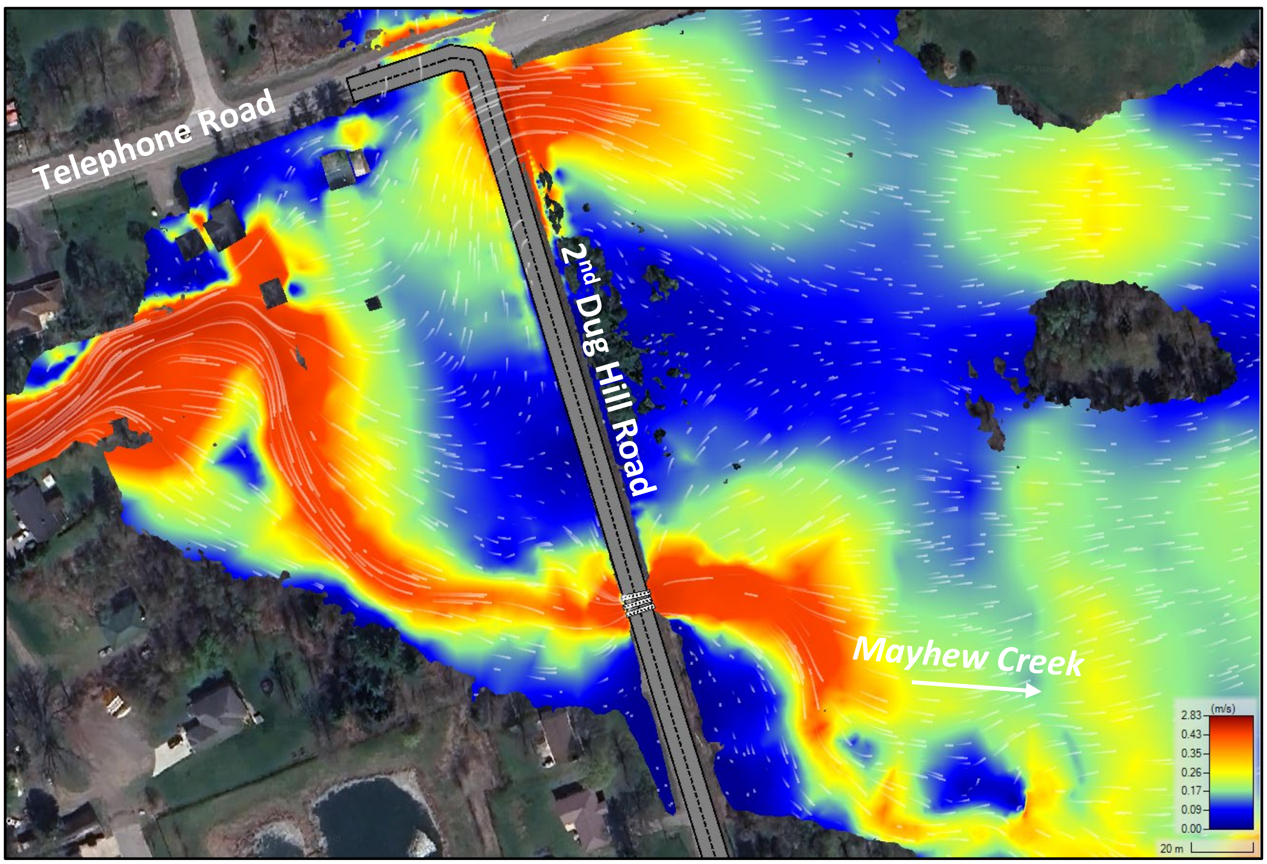



**Figure 5-17: Schematic of Timmins Floodplain at 2nd Dug Hill Road –Velocity Overlay**

## *Crossing #6: Railway Tracks*

The railway crossing is a 4.6m span by 3.7m rise CSP arch culvert as summarized in Table 5-7. This large culvert size allows events with one percent or greater annual exceedance probabilities to be conveyed without an internal spill to the Wooler Road (B) underpass or railway underpass at Trenton Junction. In the 1% AEP or more frequent events, the runoff is partially or fully contained by the berm and temporary ponding occurs upstream of the railway tracks. This is due to the large size of the culvert *in addition* to the flow attenuation received from the Tremur Lake reservoir. The Timmins event on the other hand overwhelms the culvert capacity and spills eastward due to its extreme peak flow rates (see Figure 5-18).

The railway crossing is a critical component of the Mayhew Creek study area for extreme flow events. In the Timmins event, backwater from this crossing causes flows to breach the earth berm (see Flood Control Structure #5 in Section 5.4) and seek relief flow routes at three underpass locations.

**Table 5-7: Railway Crossing Summary**

| Road Name: | | Railway Tracks | |
|---|---|---|---|
| Span (m) = | 4.6 | Rise (m) = | 3.7 |
| Upstream Invert (m) | | Downstream Invert (m) | |
| 81.46 | | 81.28 | |
| | Low Point of Road = | 87.42 | m |
| | Timmins WSEL = | 84.59 | m |
| Maximum Relief Flow Depth (m) | | Recommended Limit = 0.3m | |
| 0.00 | | ✓ | |
| Depth*Velocity ($m^2/s$) | | Recommended Limit = 0.8 ($m^3/s$) | |
| 0.00 | | ✓ | |

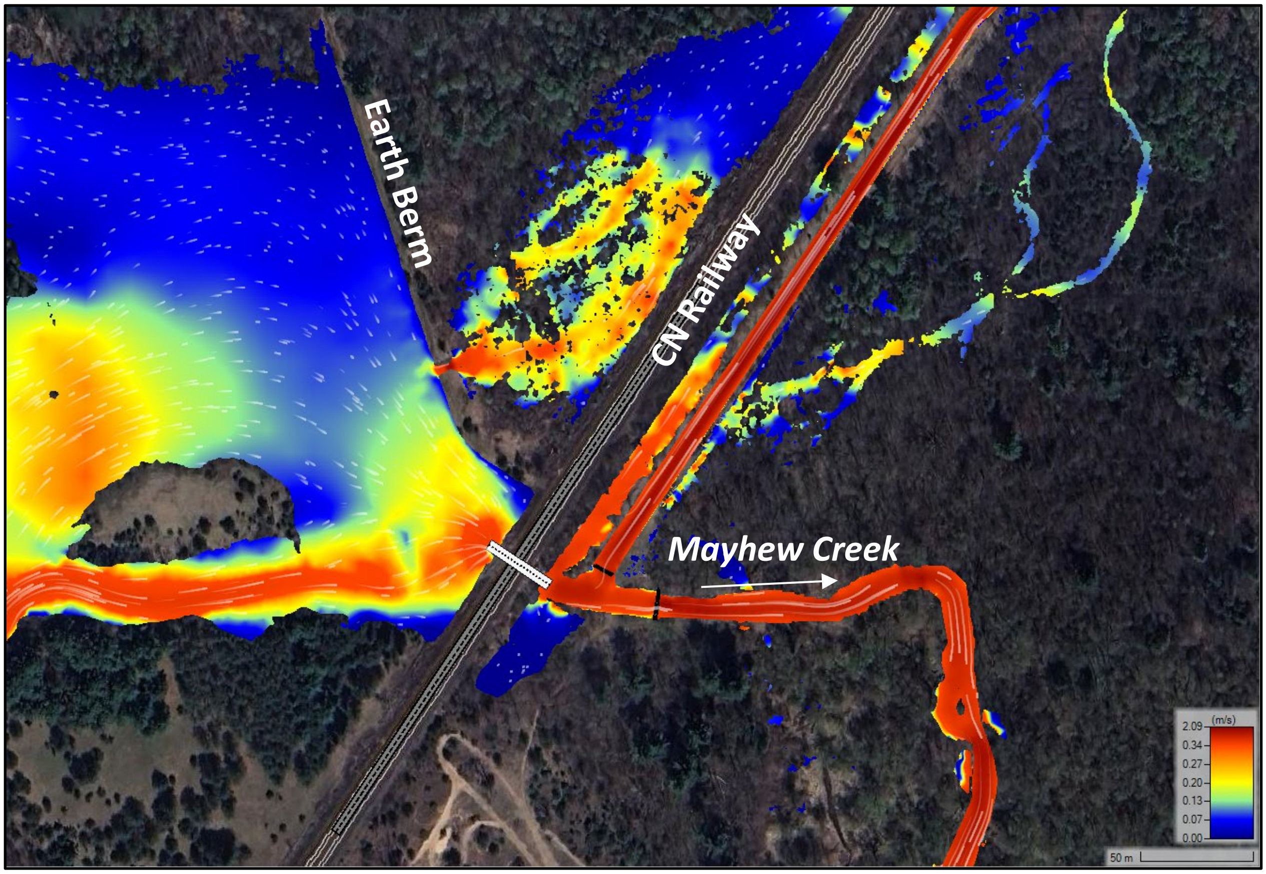



**Figure 5-18: Schematic of Timmins Floodplain at Railway Tracks – Velocity Overlay**

## *Crossing #7: Front Street (A) – Main Channel*

The main channel crossing at Front Street is a 4.9m span open footing concrete box culvert (see Figure 5-19). The height of opening is 1.9m from the creek bottom.

A summary of Crossing #7 is provided in Table 5-8. The stage and discharge hydrographs are provided in Appendix I. The flood limits upstream of the culvert opening are relatively narrow in both the 1% AEP and Timmins events (see Figure 5-20). In the Timmins event, the relief flow over Front Street is due to the amount of flow that is conveyed to the diversion channel and subsequently to Crossing #11: Front Street (B).

Fortunately, the channel upstream of Crossing #7 is relatively deep (1.5 – 2.0m) and well-defined giving the main channel high capacity to receive flows. An example cross section for the channel is presented in Figure 5-21. The houses immediately north of Mayhew Creek are well protected in the 1% AEP event despite their close proximity to Mayhew Creek. In the Timmins event, the inundation area creeps up nearly to the edge of the houses.

**Figure 5-19: Elevation View of Front Street (A) Crossing**

**Table 5-8: Front Street (A) Crossing Summary**

| Road Name: | | Front St (A) | |
|---|---|---|---|
| Span (m) = | 4.5 | Rise (m) = | 1.90 |
| [2]Upstream Invert (m) | | Downstream Invert (m) | |
| 76.90 | | 76.90 | |
| | Low Point of Road = | 78.96 | m |
| | Timmins WSEL = | 78.76 | m |
| Maximum Relief Flow Depth (m) | | Recommended Limit = 0.3m | |
| 0.0 | | ✓ | |
| Depth*Velocity ($m^2$/s) | | Recommended Limit = 0.8 ($m^3$/s) | |
| 0.0 | | ✓ | |


**Figure 5-20: Schematic of Timmins Floodplain Upstream of Crossing #7; Timmins Water Level Overlay**


**Figure 5-21: Sample Cross Section of Channel Upstream of Crossing #7: Front Street (A)**

## *Crossing #8: Water Street (A) – Main Channel*

Crossing #8 is an open footing concrete box culvert (see Figure 5-22) with a 3.6m span and height of opening of 1.25m (see Table 5-9). The profile of the road at Crossing #8 extends from Store Street to Nicholas Street.

The road profile has a low point south of the crossing that causes some relief flow to travel in a southeast direction parallel to the road (see Figure 5-23). The relief flow has more standard behaviour north of the crossing as it is perpendicular to the road centerline. There are several buildings in the flood hazard limit upstream of Crossing #8 in the Timmins event.

**Table 5-9: Water Street (A) Crossing Summary**

| Road Name: | | Water St (A) | |
|---|---|---|---|
| Span (m) = | 3.6 | Rise (m) = | 1.25 |
| Upstream Invert (m) | | Downstream Invert (m) | |
| 75.62 | | 75.06 | |
| | Low Point of Road = | 76.95 | m |
| | Timmins WSEL = | 77.61 | m |
| Maximum Relief Flow Depth (m) | | Recommended Limit = 0.3m | |
| 0.66 | | x | |
| Depth*Velocity ($m^2/s$) | | Recommended Limit = 0.8 ($m^3/s$) | |
| 0.22 | | ✓ | |

**Figure 5-22: Water Street (A) Crossing**


**Figure 5-23: Timmins Inundation Boundary in Vicinity of Crossings #7 and 8**

## *Crossing #9: Wooler Road (B) – Diversion Channel*

The diversion channel (see Section 5.5) drains to Crossing #9: Wooler Road (B). This is not to be confused with County Road 40 (Crossing #3) near Tremur Lake that is also referred to locally as Wooler Road.

Crossing #9 includes two CSP arch culverts with a 2.6m span and 1.6m rise (see Figure 5-24). A summary table for this crossing is shown in Table 5-10.

The depth*velocity product exceeds the general 0.8 $m^2/s$ limit along Wooler Road (B) in the Timmins event. This is due to the spill route at the underpass as shown in Section 5.6 rather than relief flow from the diversion channel.

**Figure 5-24: Elevation View of Wooler Road Crossing of Diversion Channel**

**Table 5-10: Wooler Road (B) Crossing Summary**

| | | | |
|---|---|---|---|
| Road Name: | | Wooler Rd (B) | |
| Span (m) = | 2.6 | Rise (m) = | 1.60 |
| Upstream Invert (m) | | Downstream Invert (m) | |
| 79.63 | | 79.34 | |
| Low Point of Road = | | 81.46 | m |
| Timmins WSEL = | | 81.95 | m |
| Maximum Relief Flow Depth (m) | | Recommended Limit = 0.3m | |
| 0.49 | | x | |
| Depth*Velocity ($m^2/s$) | | Recommended Limit = 0.8 ($m^3/s$) | |
| 1.11 | | x | |

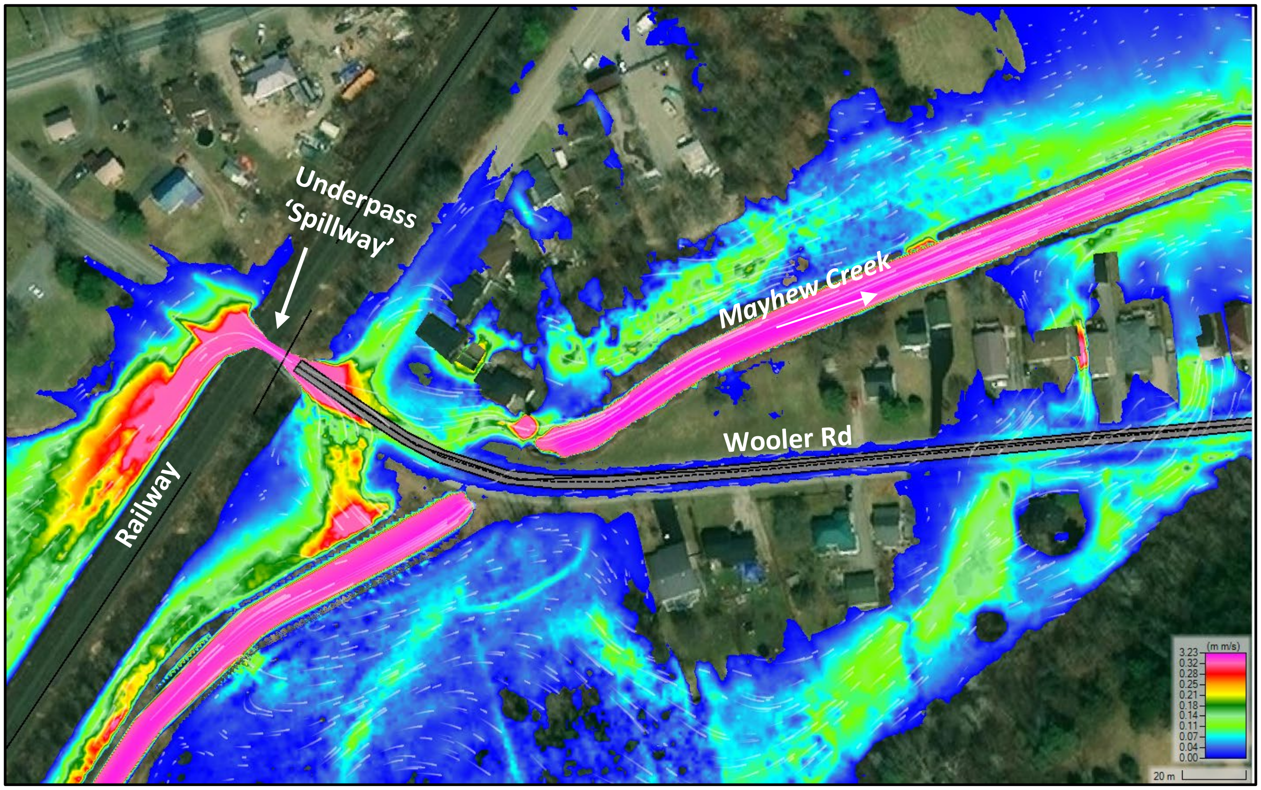



**Figure 5-25: Depth*Velocity Layer in Peak of Timmins Event with Hazardous Values Due to Spill Through Underpass at Wooler Road**

## *Crossing #10: Old Railway – Diversion Channel*

Crossing #10 is a bridge for an abandoned railway that crosses the diversion channel downstream of Wooler Road (B). It has a 3.8m span (see Figure 5-26) and is broadly overtopped in the Timmins event.

A summary table for the old railway bridge is provided below. Figures 5-27 and 5-28 show the bridge location and the surrounding inundation boundaries for Crossings #10, 11, and 12 due to their close proximity. There are several buildings within the Timmins flood limit upstream and downstream of Crossing #10.

**Figure 5-26: View of Bridge Opening at Old Railway Crossing of Diversion Channel**

**Table 5-11: Old Railway Diversion Channel Crossing Summary**

| | | | |
|---|---|---|---|
| Road Name: | | Railway (B) | |
| Span (m) = | 3.8 | Soffit (m) = | 79.64 |
| | Low Point of Road = | 79.67 | m |
| | Timmins WSEL = | 80.06 | m |
| Maximum Relief Flow Depth (m) | | Recommended Limit = 0.3m | |
| 0.39 | | x | |
| Depth*Velocity ($m^2$/s) | | Recommended Limit = 0.8 ($m^3$/s) | |
| 0.14 | | ✓ | |

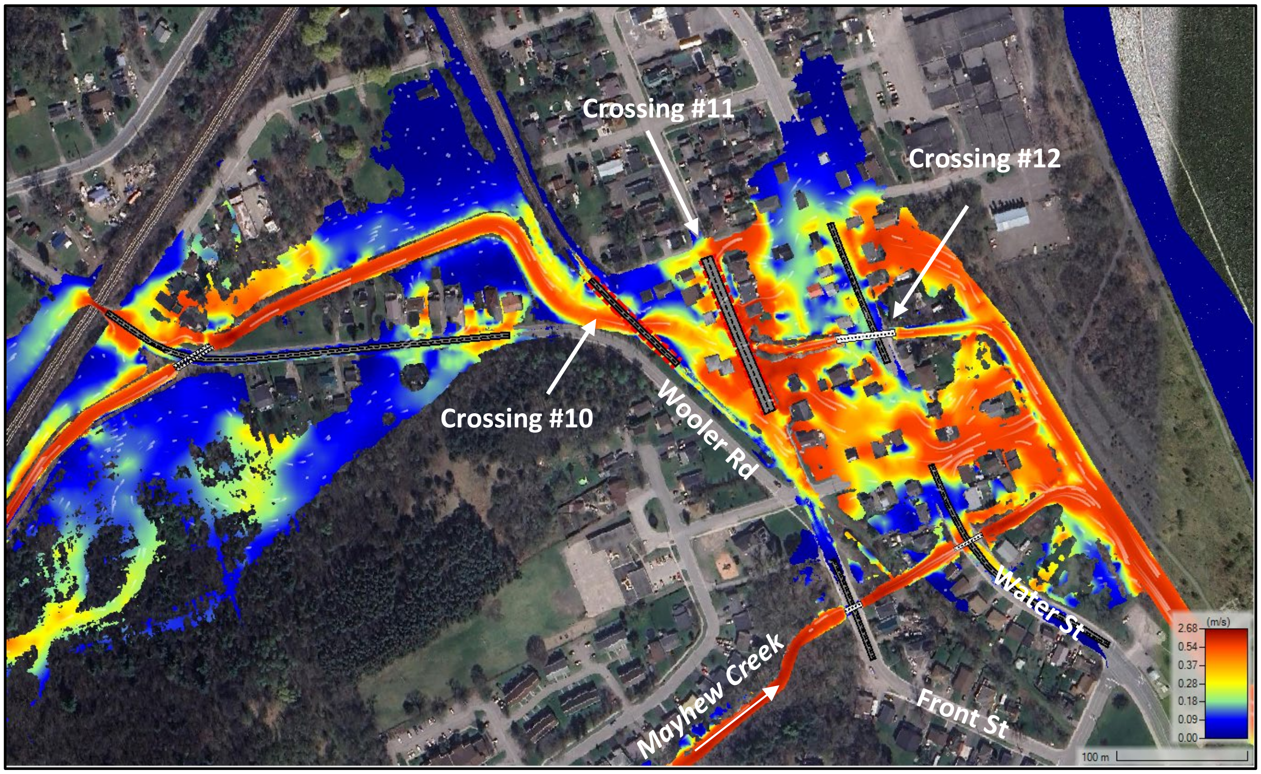



**Figure 5-27: Timmins Inundation Area for Crossings #10, 11, and 12; Velocity Overlay**

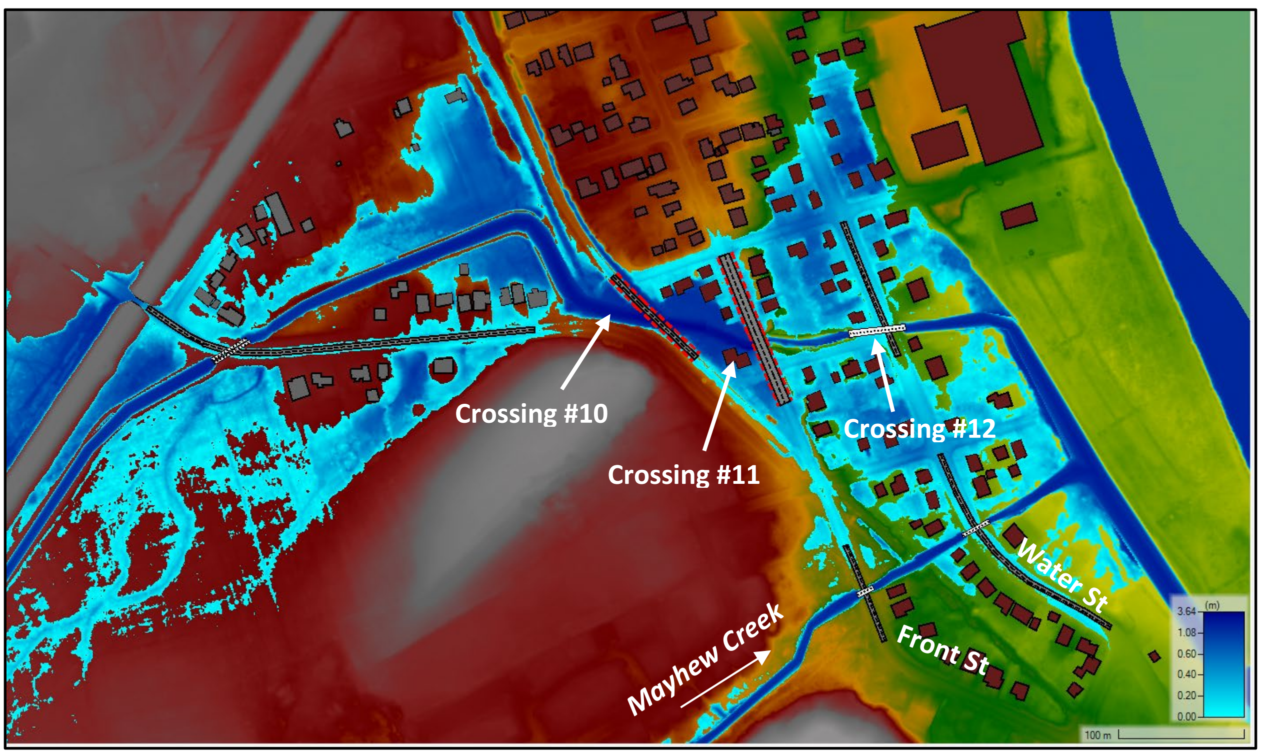



**Figure 5-28: Timmins Inundation Area for Crossings #10, 11, and 12; Depth Overlay**

## *Crossing #11: Front Street (B) – Diversion Channel*

The Front Street (B) crossing of the diversion channel is located 75m downstream of the abandoned railway. It has a 4.7m span and similar to Crossing #10, it is an old bridge with widespread overtopping in the regulatory event.

A summary table for Crossing #11 is provided below. Figure 5-29 is an image of its opening area.

**Figure 5-29: View of Bridge Opening at Front Street Diversion Channel Crossing**

**Table 5-12: Front Street (B) Diversion Channel Crossing Summary**

| | | | |
|---|---|---|---|
| Road Name: | | Front St (B) | |
| Span (m) = | 4.7 | Soffit (m) = | 78.88 |
| | Low Point of Road = | 79.02 | m |
| | Timmins WSEL = | 79.46 | m |
| Maximum Relief Flow Depth (m) | | Recommended Limit = 0.3m | |
| 0.44 | | x | |
| Depth*Velocity ($m^2$/s) | | Recommended Limit = 0.8 ($m^3$/s) | |
| 0.23 | | ✓ | |

## *Crossing #12: Water Street (B) – Diversion Channel*

The Water Street (B) crossing of the diversion channel has a 5.3m span and its opening is shown in Figure 5-30. A summary table is provided in Table 5-13. Similar to Crossings #10 and 11, there are buildings within the flood limit upstream and downstream of the crossing (see Figures 5-27 and 5-28). Stage and flow hydrographs for Crossings #10-12 are provided Appendix J-1.

**Figure 5-30: View of Culvert Opening at Front Street Diversion Channel Crossing**

**Table 5-13: Water St (B) Diversion Channel Crossing Summary**

| | | | |
|---|---|---|---|
| Road Name: | | Water St (B) | |
| Span (m) = | 5.3 | Rise (m) = | 1.80 |
| Upstream Invert (m) | | Downstream Invert (m) | |
| 75.17 | | 74.93 | |
| | Low Point of Road = | 77.30 | m |
| | Timmins WSEL = | 77.81 | m |
| Maximum Relief Flow Depth (m) | | Recommended Limit = 0.3m | |
| 0.51 | | x | |
| Depth*Velocity ($m^2/s$) | | Recommended Limit = 0.8 ($m^3/s$) | |
| 0.09 | | ✓ | |

## 5.4 Flood Control Structures

Mayhew Creek has five (5) flood control structures and a diversion channel. The flood control structure locations are presented in Figure 5-31 and correspond to the numbering in the following subsections.

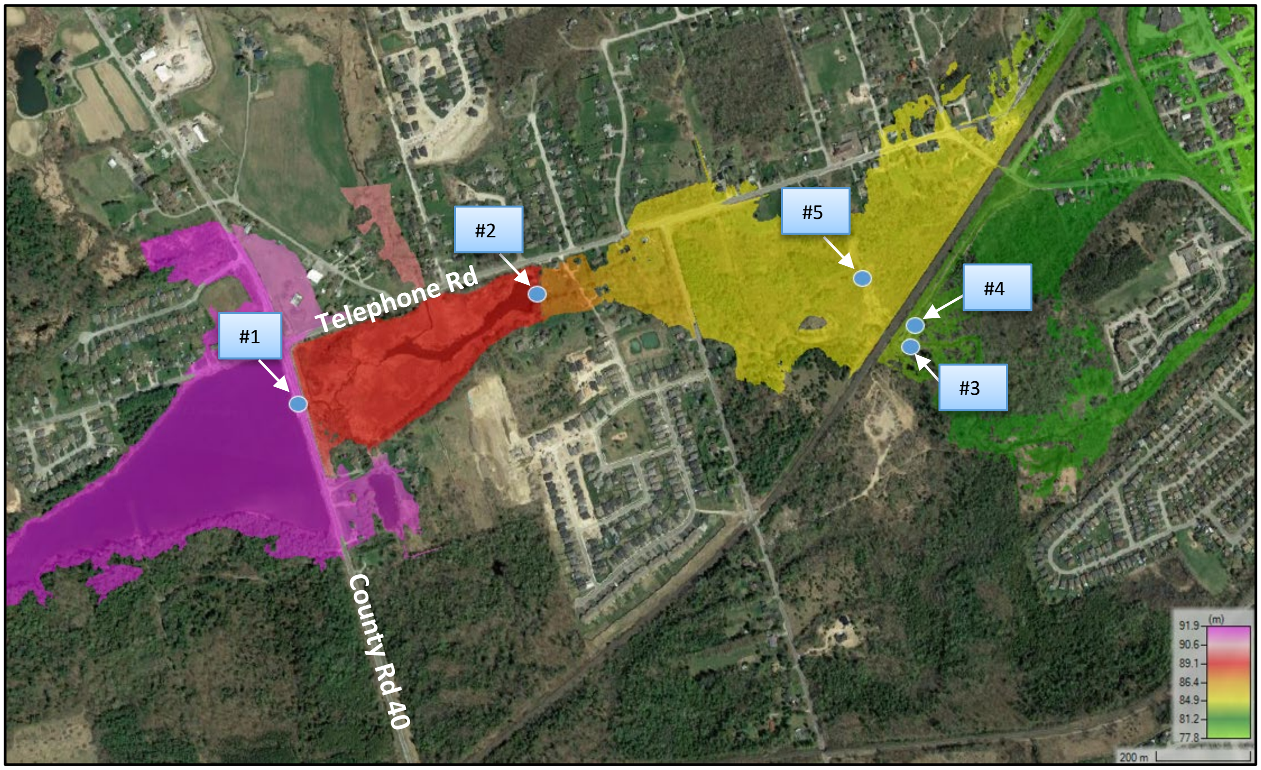



**Figure 5-31: Mayhew Creek Flood Control Structure Locations**

### *Flood Control Structure #1: Tremur Lake Dam*

The Tremur Lake Dam flood control structure is located at Crossing #3 at County Road 40 (Wooler Road) as discussed in Section 5.3. It controls the water level for Tremur Lake using the two stoplog bays that are part of the same concrete box culvert.

The structure provides significant flow attenuation in the 1% AEP storm event. In the Timmins event the dam openings offer minimal flow control as the peak flow rates in that event would cross County Road 40 primarily as relief flow over the road.

Table 5-14 summarizes the water levels, peak flow conveyance contributions from the stoplogs bays versus the relief overflows, maximum storage values, and the reduction in peak flows due to the dam and its reservoir.

In the draft hydraulic model the stoplogs were assumed to be placed at their normal operating level per the 2015 Tremur Lake Dam Safety Report and converted to 90.43m for datum CGVD 2013. With this condition, the height of opening is 1.47m. The length of each stoplog opening is 2.52m.

**Table 5-14: Tremur Lake Dam Results Summary**

| Parameter | Unit | 1% AEP | Timmins | Timmins + CC |
|---|---|---|---|---|
| Water Level | m | 91.37 | 91.47 | 91.57 |
| $Q_{peak}$ Stoplogs | $m^3/s$ | 7.4 | 8.5 | 9.8 |
| $Q_{peak}$ Relief Flow | $m^3/s$ | 1.9 | 10.2 | 22.2 |
| Max. Storage | ha-m | 14.8 | 16.3 | 18.0 |
| $Q_{peak}$ Inflow | $m^3/s$ | 16.0 | 21.8 | 32.4 |
| $Q_{peak}$ Outflow | $m^3/s$ | 9.3 | 18.7 | 32.0 |
| $Q_{peak}$ Reduction | % | 42% | 14% | 1.2% |

*Water Level in datum CGVD 2013.

## *Flood Control Structure #2: Old Mill Dam*

The Old Mill Dam has a 3m bottom opening for low flows flanked by concrete spillways on both sides (see Figure 5-32).

Table 5-15 summarizes the water levels, peak flow conveyance contributions from the total weir length versus the overflows beyond the weir, and the maximum storage values for its upstream reservoir.

**Table 5-15: Old Mill Dam Reservoir Summary**

| Parameter | Unit | 1% AEP | Timmins |
|---|---|---|---|
| Water Level | m | 88.27 | 88.60 |
| $Q_{peak}$ Weir | $m^3/s$ | 14.2 | 26.5 |
| $Q_{peak}$ Relief Flow | $m^3/s$ | 1.0 | 5.4 |
| Max. Storage (ha-m) | ha-m | 4.9 | 7.7 |

**Figure 5-32: Old Mill Dam**

## *Flood Control Structure #3: Two-Stage Weir Downstream of Railway Tracks*

There is a two-stage concrete weir located 65m downstream of the railway tracks (see Figure 5-33). The bottom stage has a weir length of 0.93m, a height of 0.7m, and an invert of 81.2m. The 2nd stage has a weir length of 3.9m, a height of 1.0m, and an invert of 81.9m. The total length of concrete weir with the two-stage structure and flanking concrete spillway is 13.9m.

The effects of the two-stage weir are shown in Figure 5-33 by the change in colour in the water surface elevation map between the immediate upstream and downstream sides of the weir. This difference in colour represents a drop in water level from 83.25m on the upstream side of the control structure to 82.50m on the downstream side as flow continues down the main channel of Mayhew Creek.

The two-stage weir works in tandem with the diversion weir discussed in the next subsection.

## *Flood Control Structure #4: Trapezoidal Diversion Weir*

The diversion weir is a trapezoidal concrete weir with a 5.2m bottom length and side slopes of approximately 2:1 on each side. The height of the structure from the invert of the concrete weir to the top of the concrete side slopes is 1.0m.

The invert of the weir is 81.77m, slightly greater than 0.5m higher than the adjacent two-stage weir, meaning the two-stage weir conveys low flows and is engaged prior to the diversion weir. The table in *Section 5.5: Diversion Channel* compares the flow conveyed by each structure in the 1% AEP, Timmins and Timmins plus climate change events.

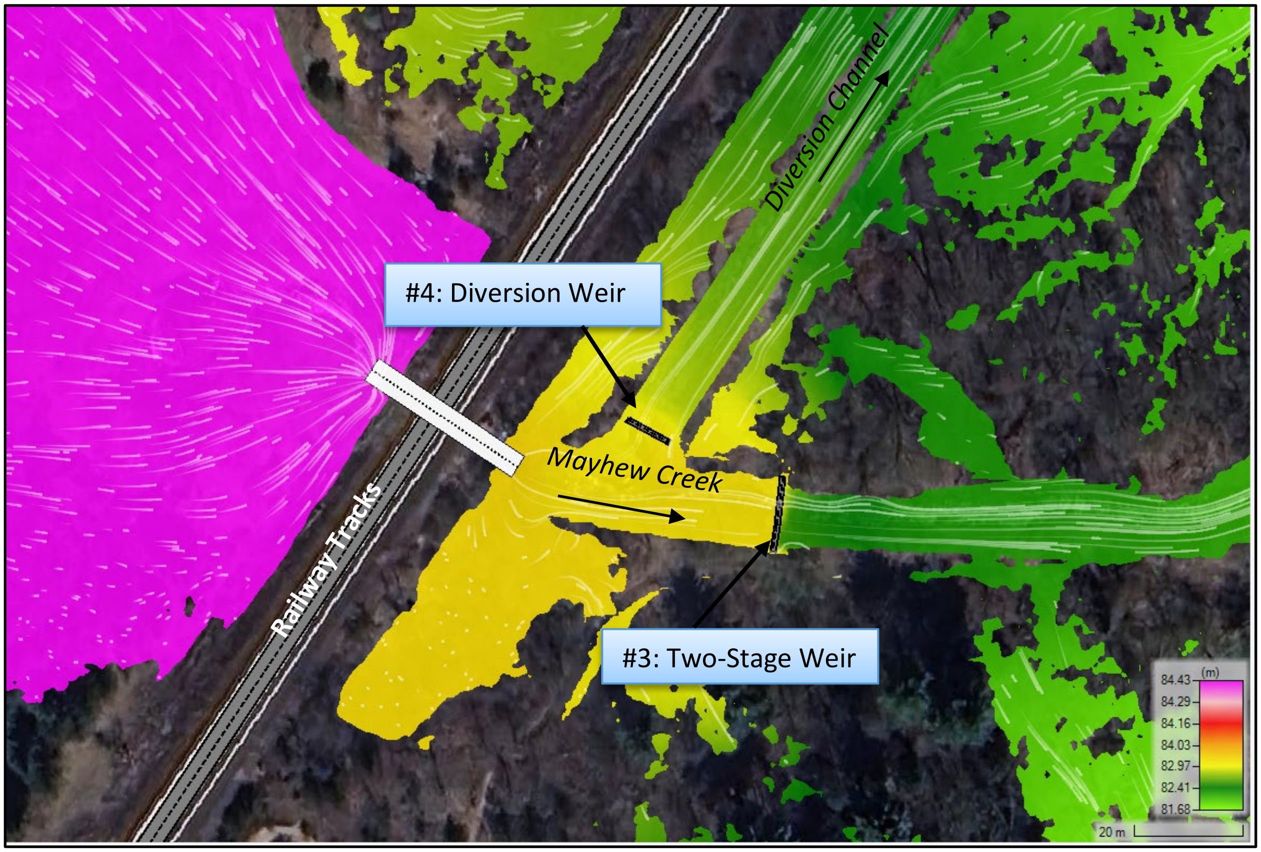



**Figure 5-33: Two-Stage Weir and Diversion Weir Downstream of Railway Tracks, Timmins WSEL Overlay**

## *Flood Control Structure #5: Earth Berm Upstream of Railway Tracks*

A 230m long earth berm is located upstream of the railway tracks that cross the Mayhew Creek main channel. The berm is north of the main channel and aligned in a north-south direction. A profile of the berm is depicted in Figure 5-34 with stationing from left to right looking in the direction of flow overtopping (see Figures 5-35 and 5-36). There is a depression in the berm approximately 70m north of the railway tracks. The depression is approximately 0.9m deep with a 2m bottom width. The berm is identifiable from the LiDAR and terrain data. Its location was also confirmed in the field by Jewell survey staff.

The berm is breached in the 1% AEP and Timmins events and results in the spill discussed in Section 5.6. Figures 5-35 and 5-36 show the location of the berm and flow through the spillway at a snapshot of the Timmins event.

A review of the particle tracing and inundation mapping in the Timmins storm event shows that the backwater imposed by the CSP arch culvert crossing of the railway tracks would eventually spill through the berm east of the Mayhew Creek main channel before ultimately flowing east towards the Wooler Road underpass.

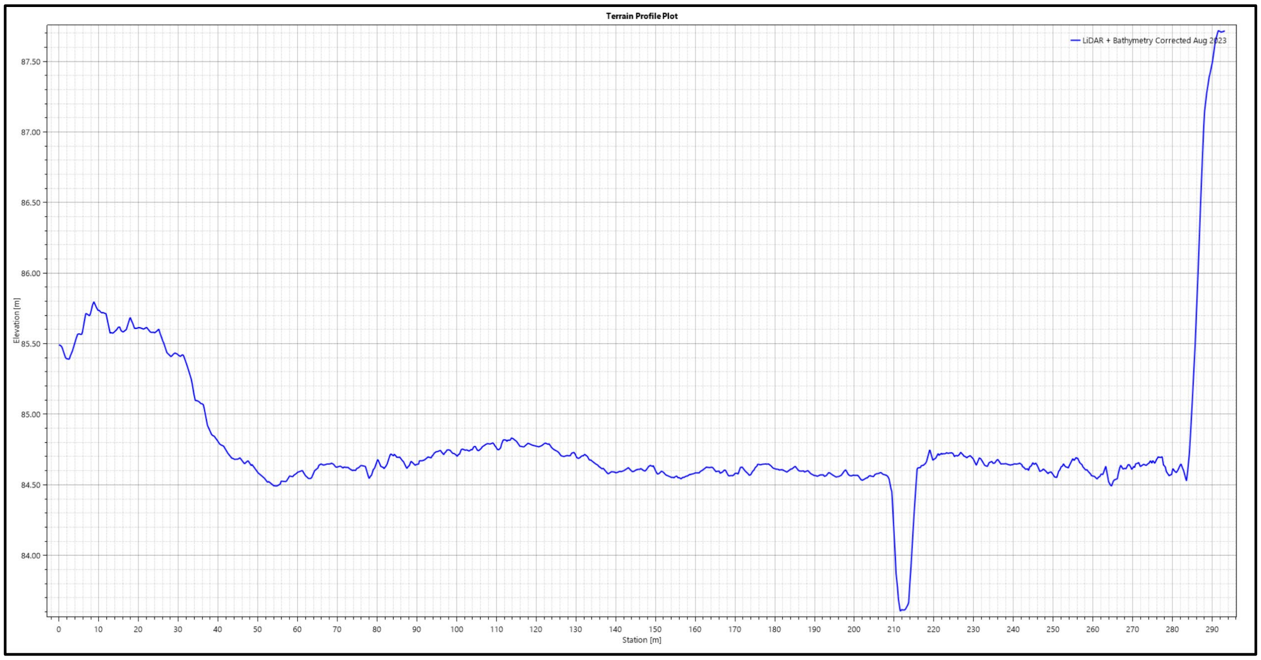



**Figure 5-34: Earth Berm Profile Upstream of Railway Tracks**

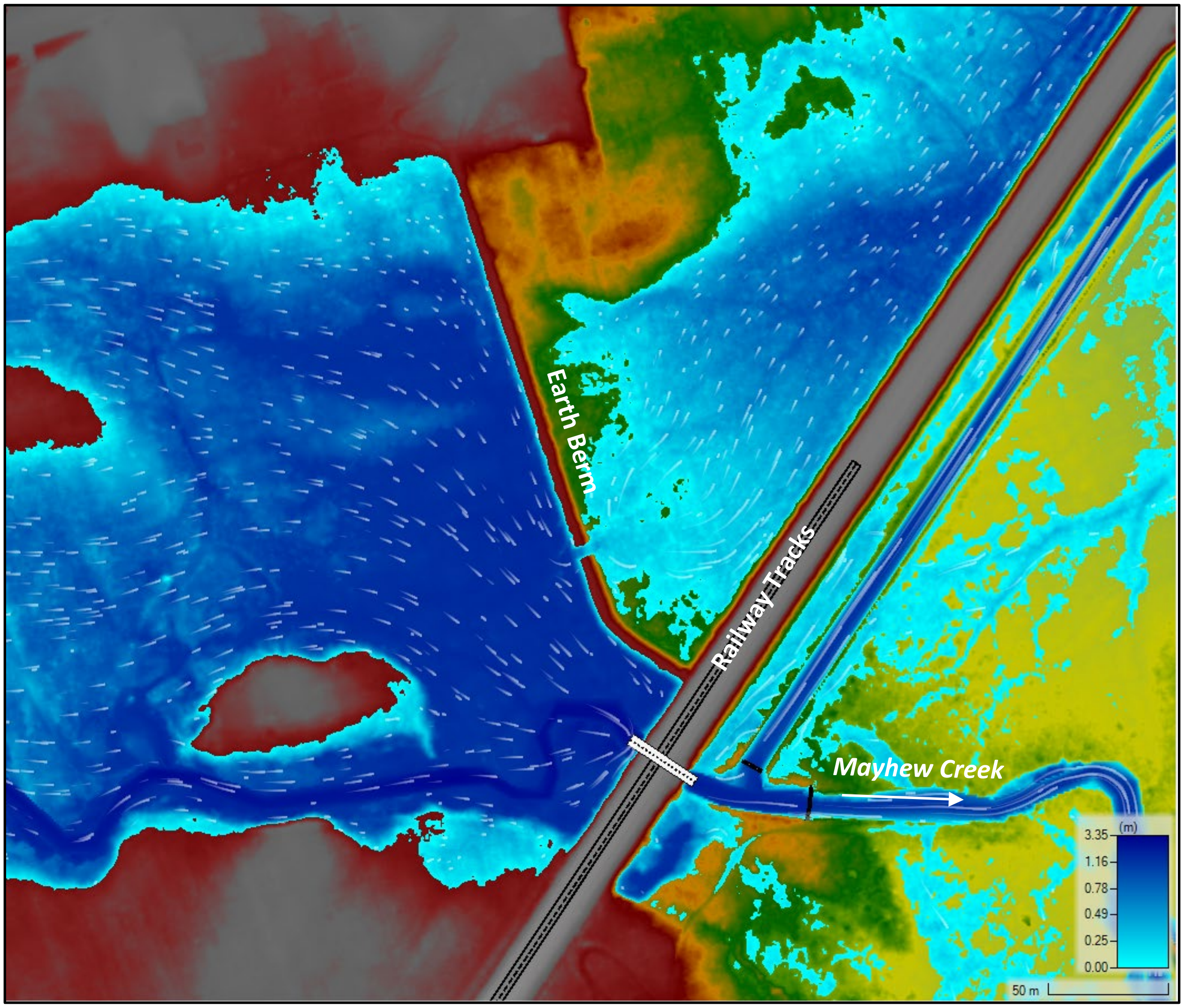



**Figure 5-35: Earth Berm Location Upstream of Railway; Snapshot of Timmins Event; Depth Overlay**

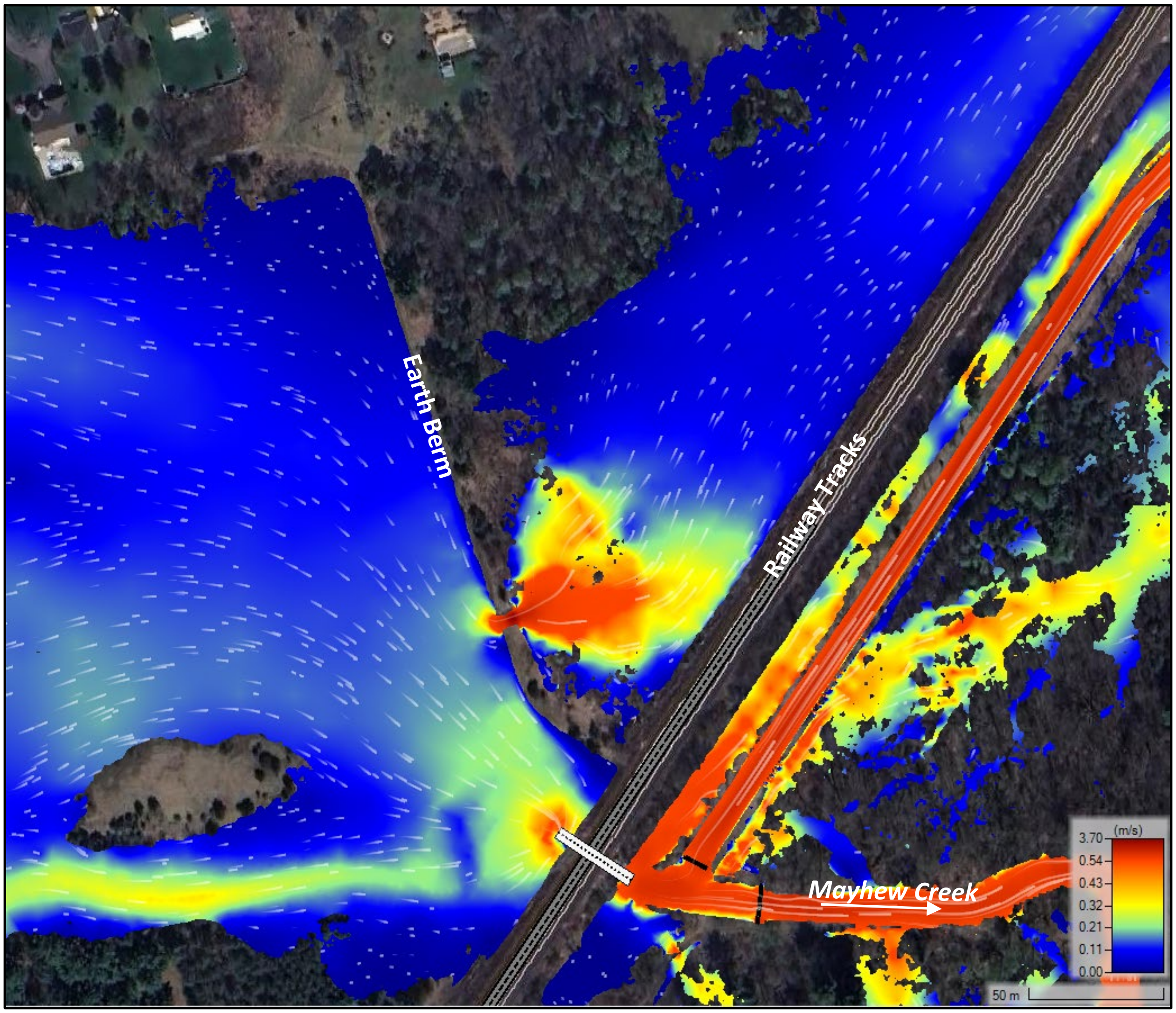



**Figure 5-36: Earth Berm Location Upstream of Railway; Snapshot of Timmins Event Before Berm Overtopping; Velocity Overlay**

## 5.5 Diversion Channel

There is a well-defined human-constructed diversion channel that begins immediately downstream of the railway tracks (see Figure 5-37) that continues until the outlet to the Trent River. Flood Control Structures #3 and #4 are used to establish the flows to the diversion channel (see Figure 5-33).

The diversion channel receives a significant amount of flow in a major storm event. A comparison of flow that drains to the diversion channel versus the main channel is provided below.

**Table 5-16: Hydraulic Conveyance in Mayhew Creek and Diversion Channels**

| Storm Event | Diversion Channel | Mayhew Creek |
|---|---|---|
| 1% AEP | 9.1 | 11.1 |
| Timmins | 10.5 | 13.8 |
| Timmins + CC | 10.7 | 14.3 |

**Remainder of flow spills outside of weir edges, mostly in same direction as diversion channel.*


**Figure 5-37: Mayhew Creek Natural Channel and Constructed Diversion Channel Downstream of Railway Tracks**

An example cross section of the diversion channel alongside the railway tracks is provided in Figure 5-38 to illustrate its representative channel dimensions.

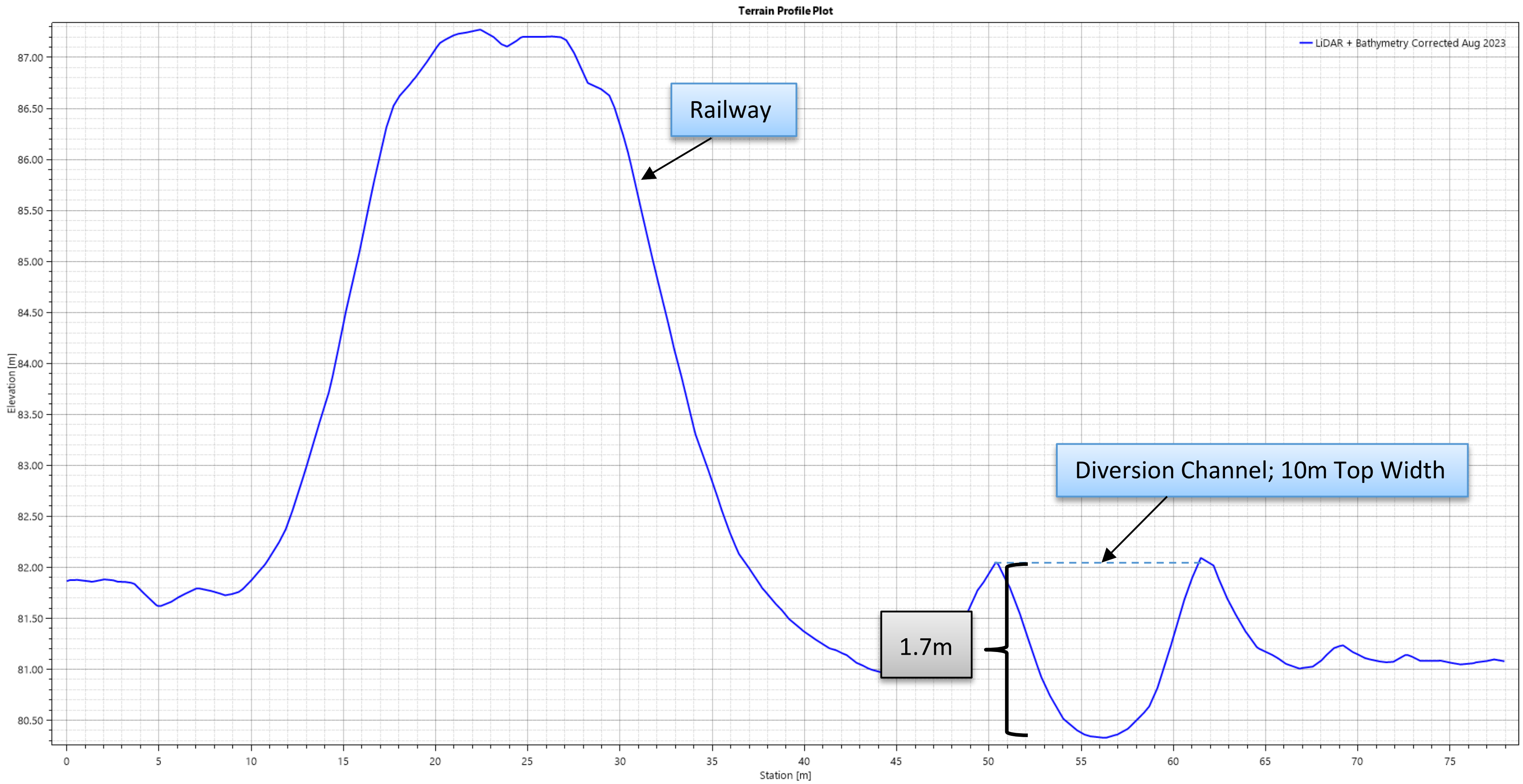



**Figure 5-38: Example Cross Section of Mayhew Creek Diversion Channel Parallel to Railway Tracks**

# 6 Hydraulics – Cold Creek

A similar hydraulic approach to Mayhew Creek was applied to Cold Creek and the hydraulic analysis was prepared using HEC-RAS version 6.4.1.

The Jewell bathymetric survey comprised of 37 cross sections for Cold Creek (see Figure 6-1). The Cold Creek hydraulic model is comprised of 31.000 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.

Cross-section water surface elevation plots are provided in Appendix K-2 for a series of representative cross sections of the Cold Creek channel and its overbanks.

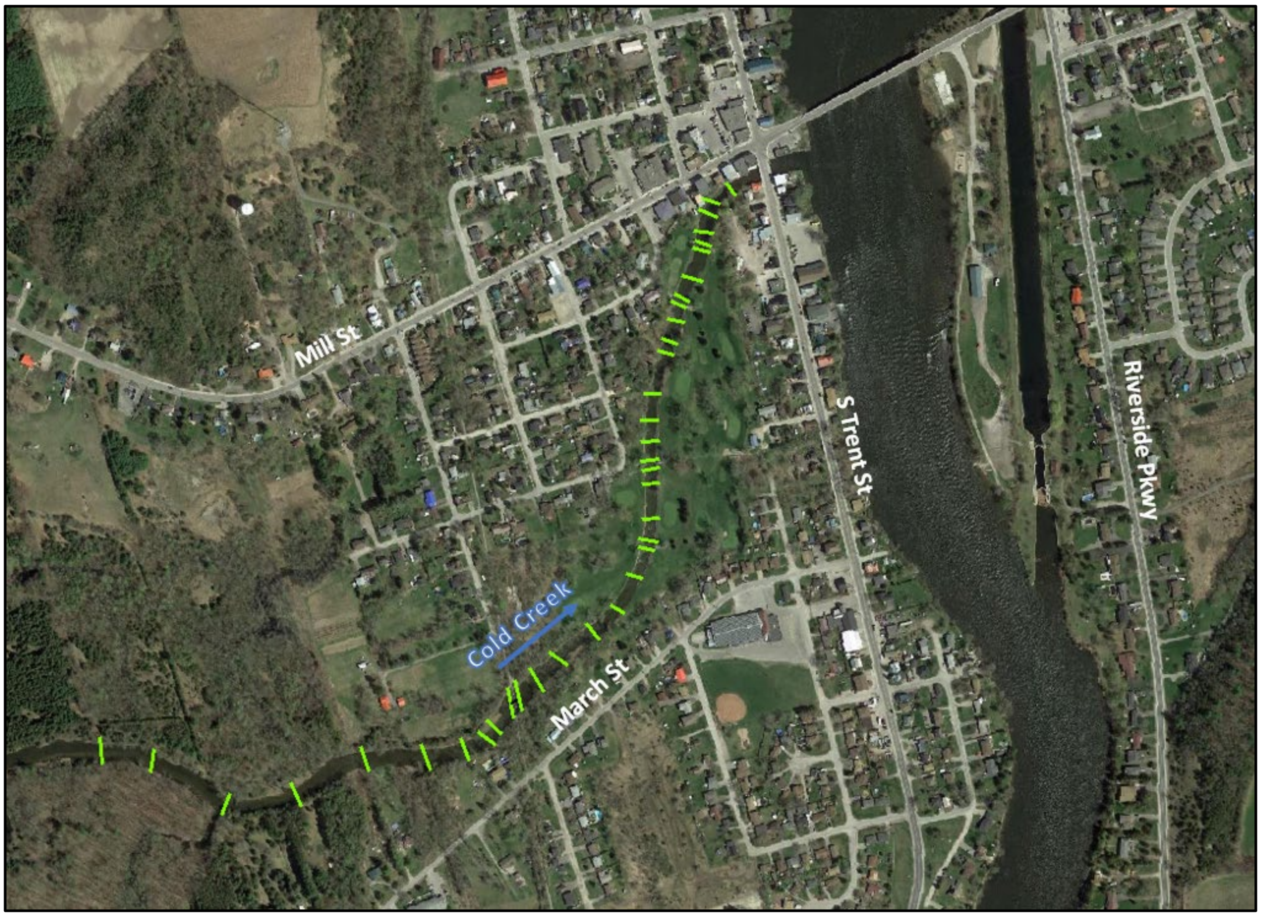



**Figure 6-1: Cold Creek Bathymetry Survey Section Locations**

## 6.1 Internal and External Boundary Conditions

There are three (3) boundary conditions (BCs) for the 2D model (see Figure 6-2). One is an inflow boundary condition and the other two are outflow BCs (see Table 6-1). The inflow hydrograph is from the hydrology model output. There are no major tributaries within the study area and subsequently a singular inflow boundary condition was applied.

**Table 6-1: Cold Creek Boundary Condition Summary**

| Boundary Condition | Type | Peak Flow ($m^3/s$) | |
|---|---|---|---|
| | | 1% AEP | Timmins |
| 1 | Inflow Hydrograph | 61.6 | 74.7 |
| 2 | Normal Depth | N/A | |
| 3 | Stage Hydrograph | N/A | |

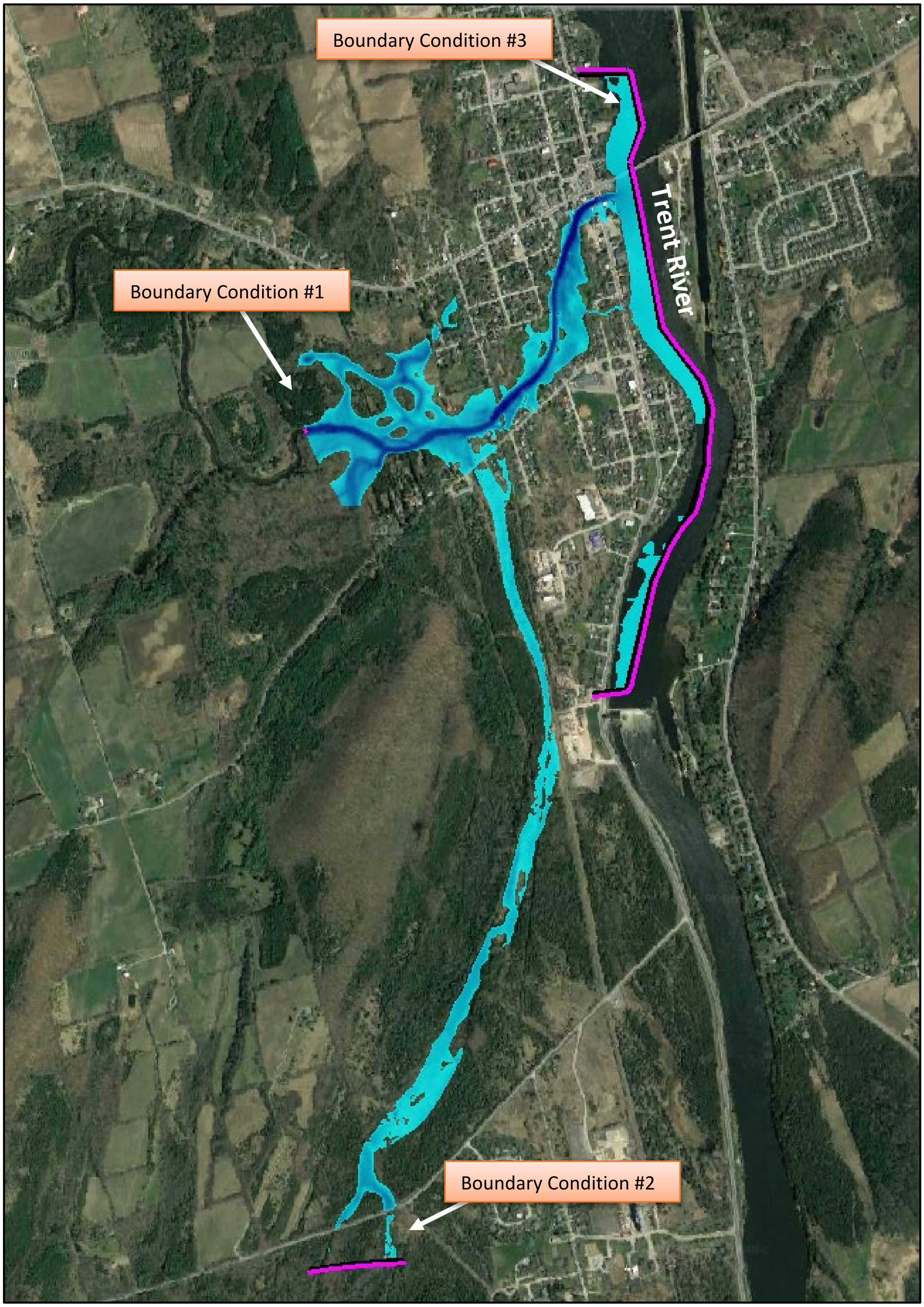



**Figure 6-2: Cold Creek Boundary Condition Locations**

## 6.2 Culvert & Bridge Crossings

There are six (6) culvert and bridge crossings within the subject study area as shown in Figure 6-3.

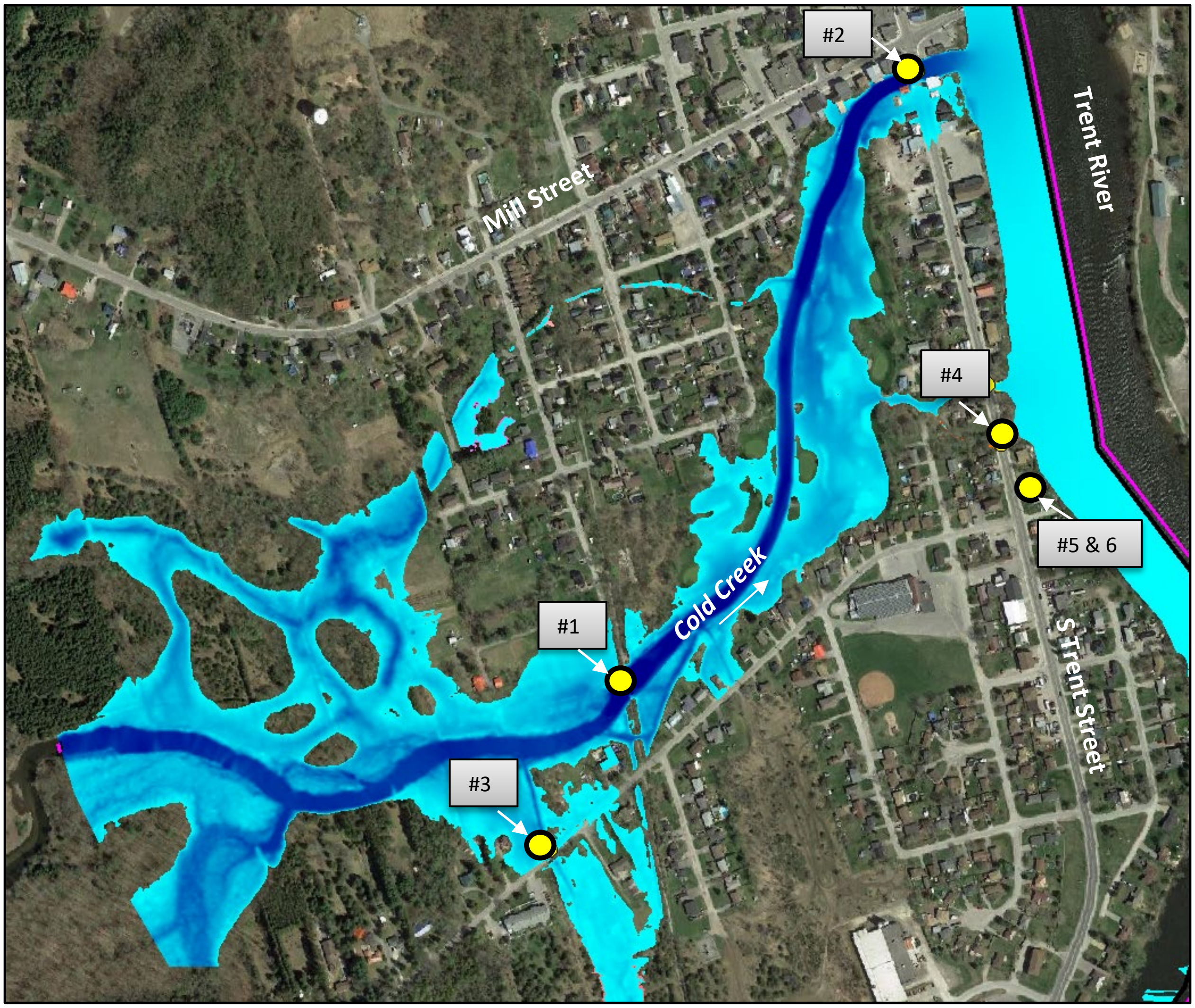



**Figure 6-3: Cold Creek Bridge/Culvert Locations**

The hydraulic model simulates the effects of culvert and bridge crossings on the water surface elevations at each crossing and this subsection summarizes the existing crossing configurations, stage-discharge curves, and the maximum water surface elevations at each location. The purpose of the section is to address the impacts of the existing road and railway infrastructure on the overall floodplain delineation.

Stage and flow hydrographs for each crossing in the Timmins event are provided in Appendix J-2.

Note that excluding the above culverts of interest, an additional six (6) culverts were surveyed and included in the hydraulic model to accommodate the inflows from the culvert opening at the Wallace Street berm north of the Cold Creek main channel (see Figure 6-4). These connection culverts were included to assess the internal spill that occurs upstream of the old railway and north of the main channel.

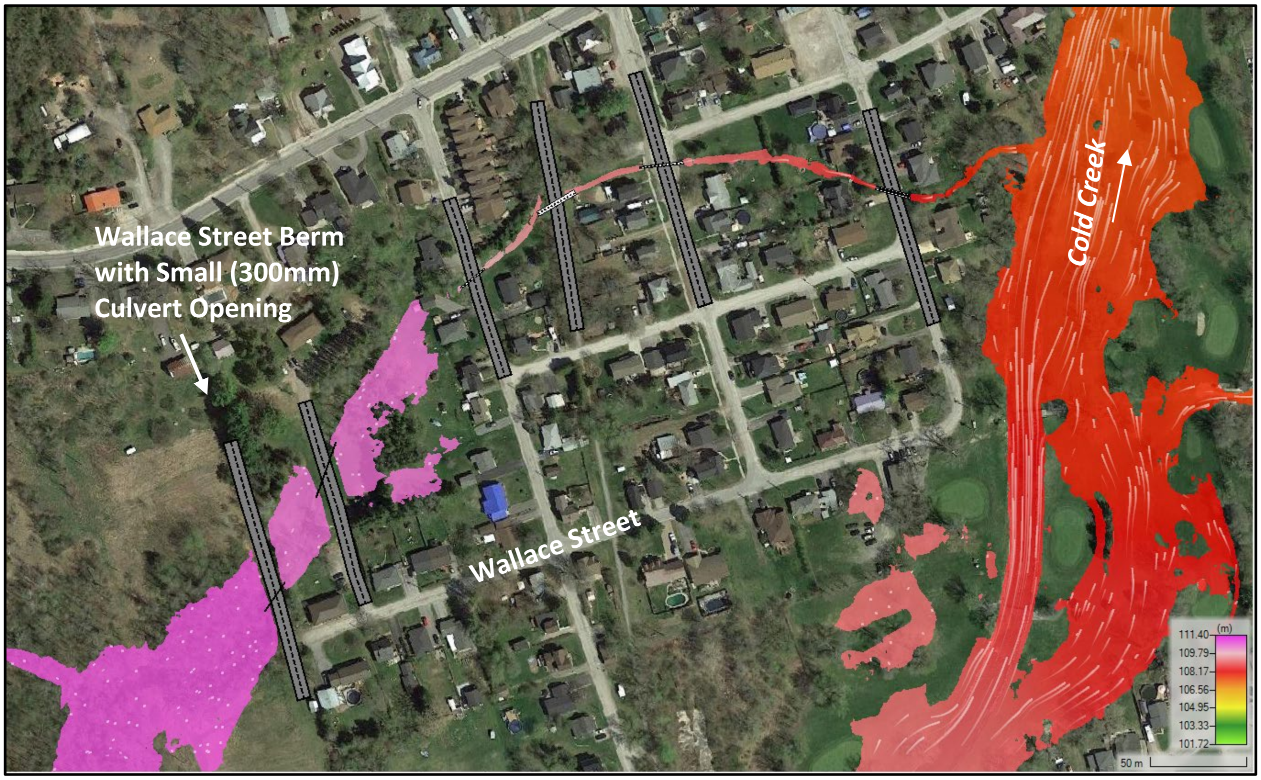



**Figure 6-4: Connection Culverts near Wallace Street North of Cold Creek Main Channel**

### *6.2.1 Crossing #1: Old Railway Bridge & Overflow Opening*

The first crossing is an old railway bridge with three (3) piers and a 28-degree skew (see Figure 6-5). After accounting for the skew, the bridge has an effective span of 28.2m. With the piers ranging from 2.0 to 2.4m in width, the total span of *opening* area is 21.8m.

In addition to the large bridge crossing, there is an overflow opening that can be considered part of the same trail crossing configuration (see Figure 6-6). This overflow bridge opening is located 57m south of the centerline of the main channel and has an invert that is approximately 1.5m higher than the main channel.

The bridge openings for Crossing #1 are summarized in Tables 6-2 and 6-3. The Timmins flood behaviour in the vicinity of Crossing #1 is demonstrated in Figure 6-7.

**Figure 6-5: Elevation View of Old Railway Bridge**

**Figure 6-6: Old Railway Bridge Overflow Opening**

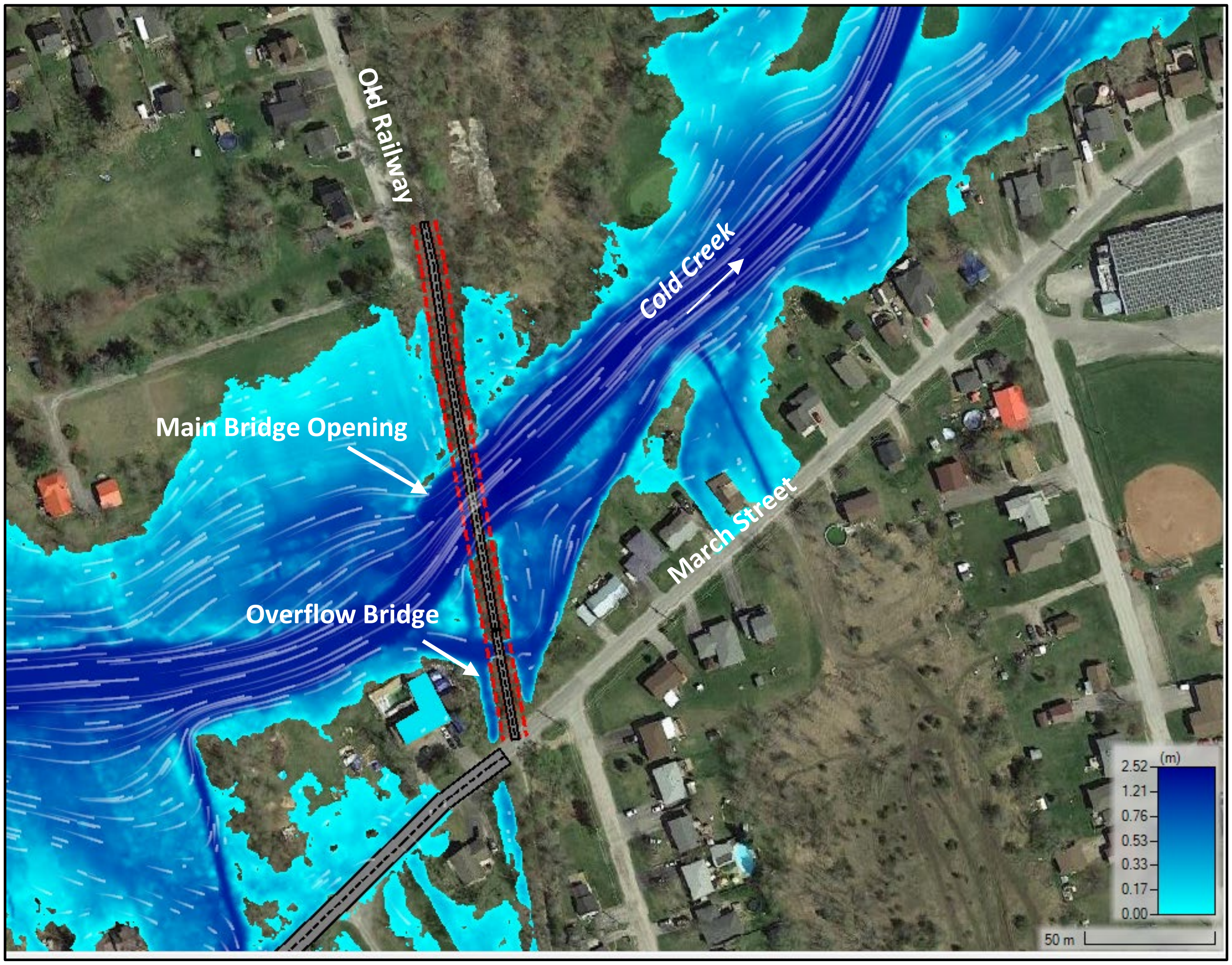



**Figure 6-7: Timmins Inundation Area at Old Railway Bridge; Depth Overlay**

**Table 6-2: Old Railway; Main Bridge Summary**

| Road Name: | | Old Railway (A) | |
|---|---|---|---|
| Span (m) = | 28.2 | [1]Soffit (m) = | 110.52 |
| Low Point of Road = | | 111.24 | m |
| Timmins WSEL = | | 109.93 | m |
| Maximum Relief Flow Depth (m) | | Recommended Limit = 0.3m | |
| 0.0 | | ✓ | |
| Depth*Velocity ($m^2/s$) | | Recommended Limit = 0.8 ($m^2/s$) | |
| 0.0 | | ✓ | |

**Table 6-3: Old Railway; Overflow Bridge Summary**

| Road Name: | | Railway Overflow Bridge | |
|---|---|---|---|
| Span (m) = | 4.8 | Soffit (m) = | 110.74 |
| | Low Point of Road = | 111.08 | m |
| | Timmins WSEL = | 110.00 | m |
| Maximum Relief Flow Depth (m) | | Recommended Limit = 0.3m | |
| 0.0 | | ✓ | |
| Depth*Velocity ($m^2/s$) | | Recommended Limit = 0.8 ($m^2/s$) | |
| 0.0 | | ✓ | |

### *6.2.2 Crossing #2: South Trent Street*

The bridge crossing of the main channel at South Trent Street is also the outlet to the Trent River (see Figure 6-8). The bridge has a 16.4m span and is summarized in Table 6-4.

Crossing #2 is overtopped south of the bridge opening in the Timmins event. There are several buildings within the flood hazard limit in the vicinity of this bridge (see Figures 6-9 and 6-10).

**Table 6-4: Summary of South Trent Street Bridge Crossing at Trent River Outlet**

| Road Name: | | S Trent St 1 | |
|---|---|---|---|
| Span (m) = | 16.4 | Soffit (m) = | 106.83 |
| | Low Point of Road = | 107.22 | m |
| | Timmins WSEL = | 107.53 | m |
| Maximum Relief Flow Depth (m) | | Recommended Limit = 0.3m | |
| 0.31 | | ✓ | |
| Depth*Velocity ($m^2/s$) | | Recommended Limit = 0.8 ($m^3/s$) | |
| 0.06 | | ✓ | |


**Figure 6-8: Crossing #2 - South Trent Street Bridge Outlet to Trent River**

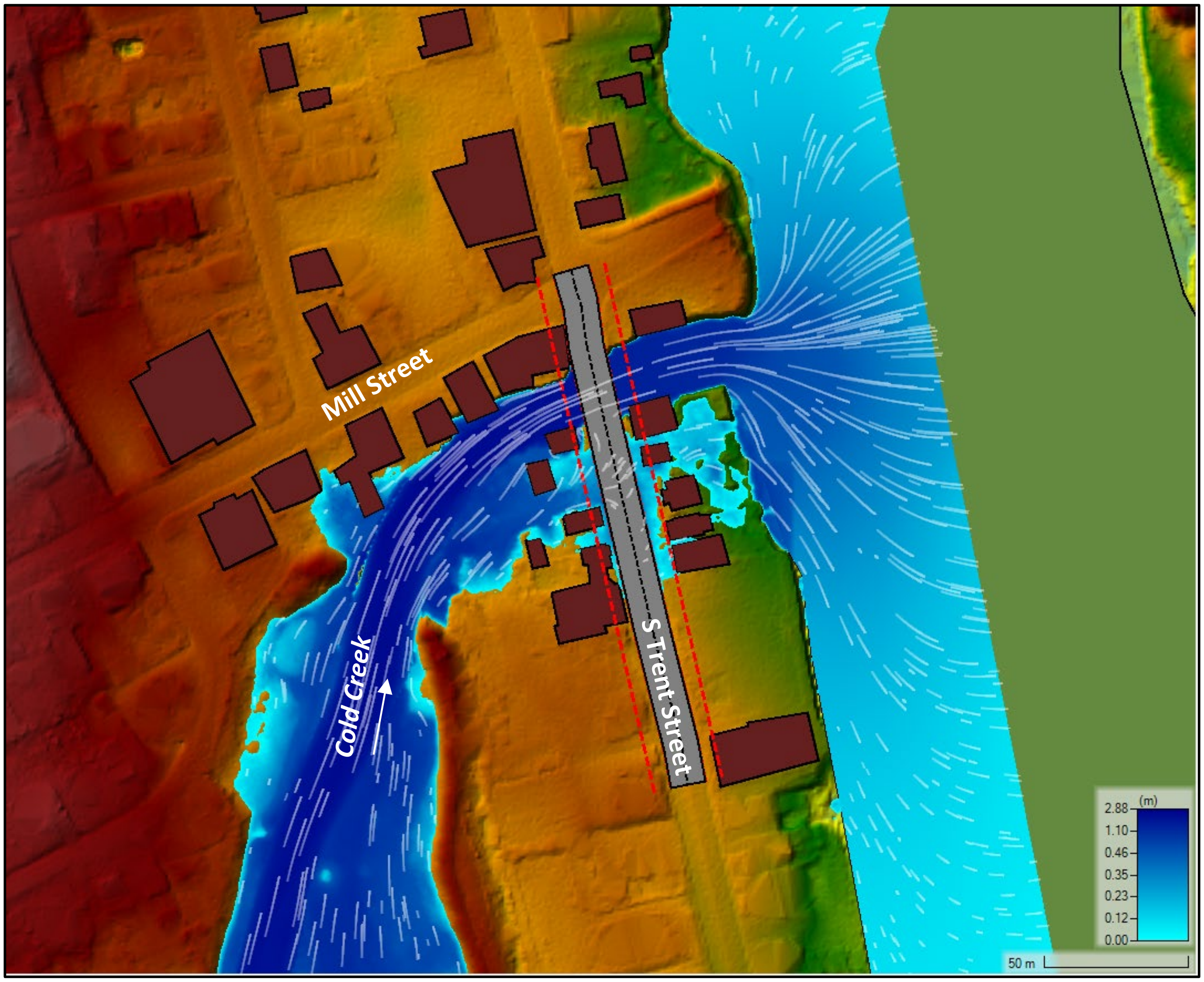



**Figure 6-9: Timmins Inundation Area in Vicinity of South Trent Street Main Crossing; Depth Overlay**


**Figure 6-10: Timmins Inundation Area in Vicinity of South Trent Street Bridge; Water Level Overlay**

### *6.2.3 Crossing #3: March Street (Spill Crossing)*

The March Street crossing is utilized as a spill crossing upstream of the old railway in large storm events (see Figures 6-11 and 6-12). The culvert is relatively small compared to the amount of flow that is conveyed to the crossing in the Timmins event.

*Section 6.3: Spill Areas & Storage Impacts* notes the spill to March Street is 2.7 $m^3/s$ in the Timmins event. The consequence is a relief flow depth of 0.22m (see Table 6-5).

**Figure 6-11: March Street Spill Culvert**

**Table 6-5: March Street Crossing Summary**

| | | | |
|---|---|---|---|
| Road Name: | | March St | |
| Span (m) = | 1.8 | Rise (m) = | 1.1 |
| Upstream Invert (m) | | Downstream Invert (m) | |
| 109.20 | | 109.15 | |
| Low Point of Road = | | 110.26 | m |
| [3]Timmins WSEL = | | 110.48 | m |
| Maximum Relief Flow Depth (m) | | Recommended Limit = 0.3m | |
| 0.22 | | ✓ | |
| Depth*Velocity ($m^2/s$) | | Recommended Limit = 0.8 ($m^2/s$) | |
| 0.02 | | ✓ | |

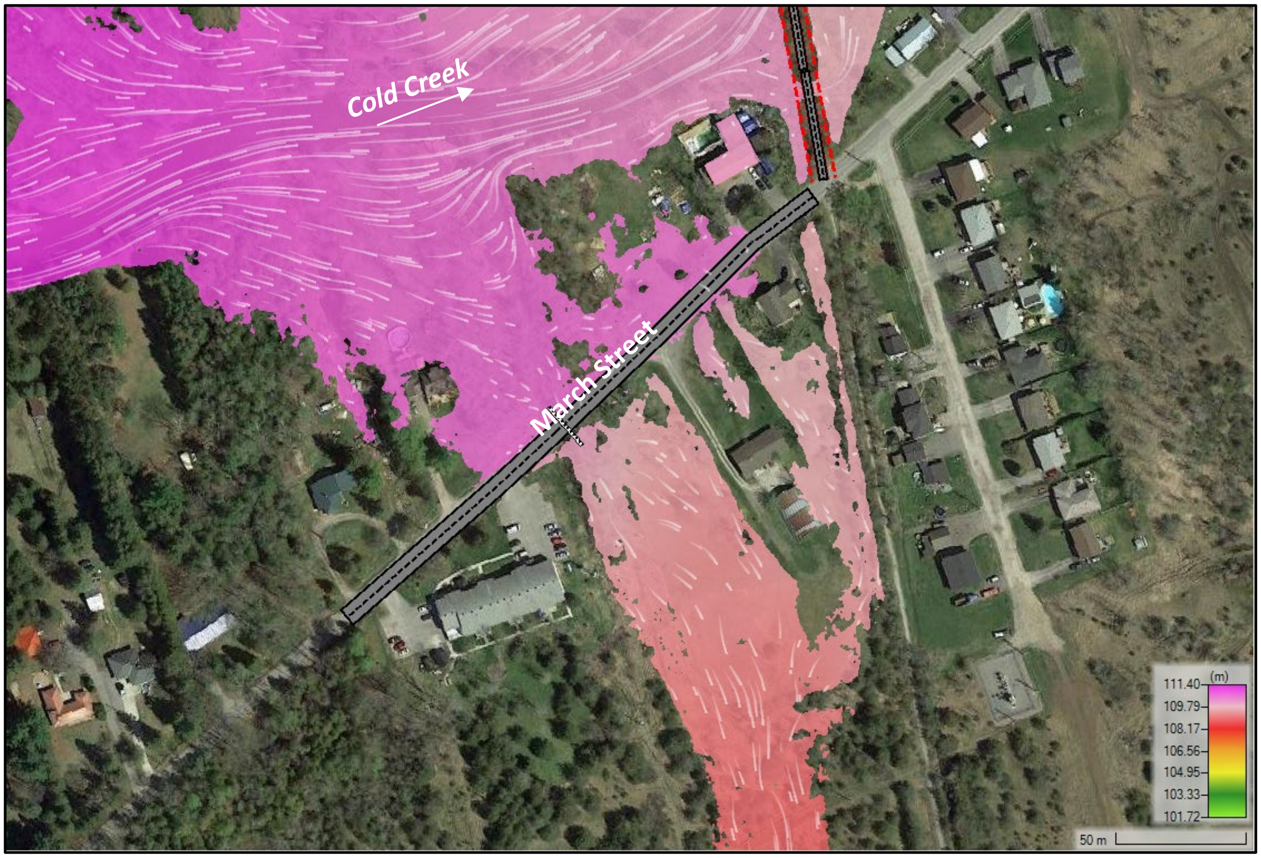



**Figure 6-12: Timmins Inundation Area in Vicinity of March Street Spill Crossing; Water Level Overlay**

### *6.2.4 Crossing #4: South Trent Street – North Bridge Opening for Berm Spillway*

There is a spillway in the berm east of the Cold Creek main channel on the west side of South Trent Street (see Figure 6-17 of Section 6.3). This spillway outlets to three (3) crossings of South Trent Street. These crossings are referred to as Crossings #4, 5, and 6.

Crossing #4 is the main spill crossing of South Trent Street. It is a 9.2m span bridge as shown in Figure 6-12. The location of the crossing as it relates to the berm spillway is shown in Figure 6-13. The crossing conveys a peak flow of 3.6 $m^3/s$ in the Timmins event.

A summary table for Crossing #4 is provided below.

**Figure 6-13: North Spill Crossing at South Trent Street**

**Table 6-6: Crossing #4 Bridge Summary**

| | | | |
|---|---|---|---|
| Road Name: | | S Trent St North Spillway Crossing | |
| Span (m) = | 9.2 | Soffit (m) = | 108.3 |
| | Low Point of Road = | 107.57 | m |
| | Timmins WSEL = | 106.93 | m |
| Maximum Relief Flow Depth (m) | | Recommended Limit = 0.3m | |
| 0.0 | | ✓ | |
| Depth*Velocity ($m^2$/s) | | Recommended Limit = 0.8 ($m^2$/s) | |
| 0.0 | | ✓ | |

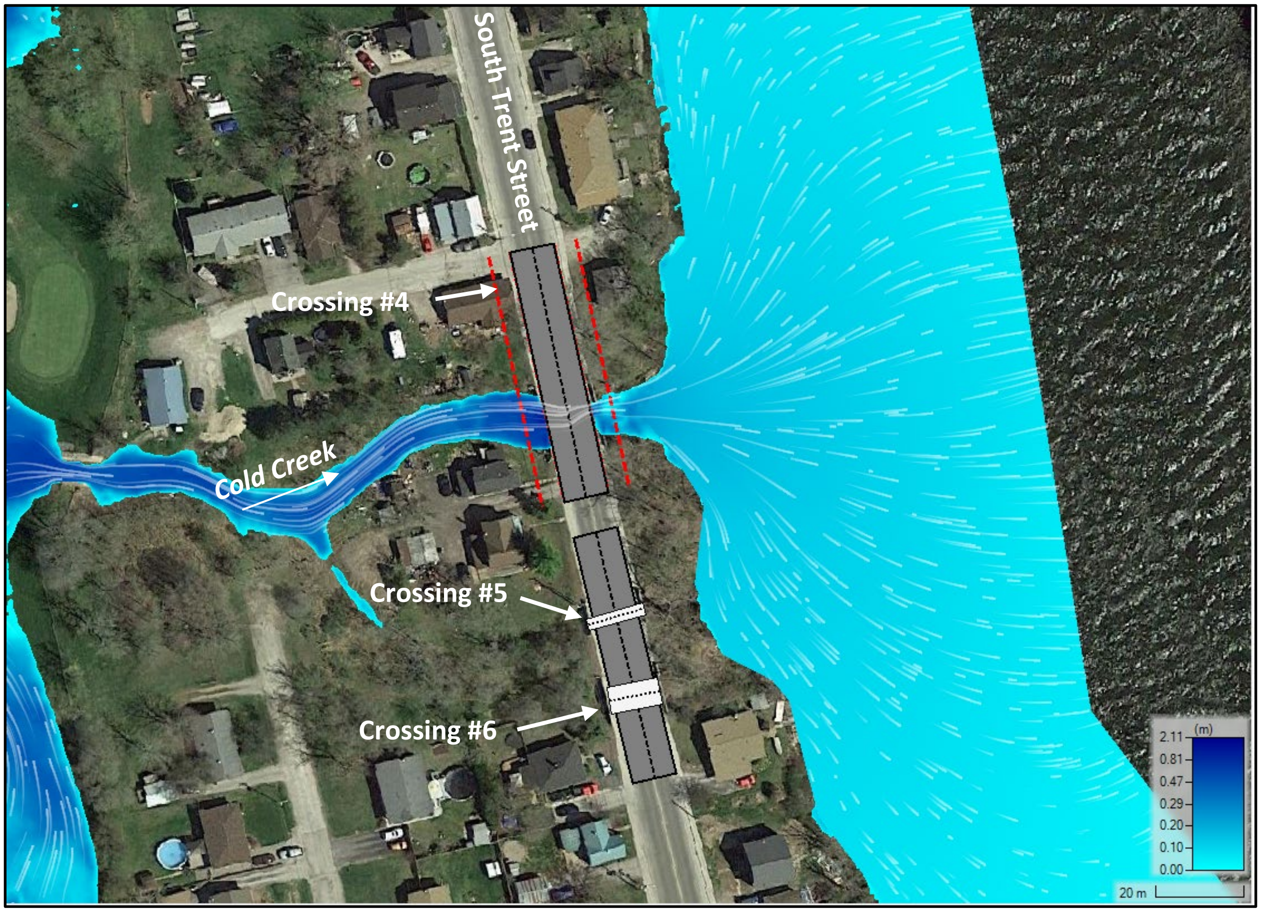



**Figure 6-14: Crossing Locations for Cold Creek Spill Towards South Trent Street**

## *6.2.5 Crossing #5 & 6: South Trent Street – Middle & South Spill Crossing*

Crossings #5 and 6 are the middle and south spill crossings of South Trent Street as shown in Figure 6-13. Both are open footing concrete structures as shown in Figures 6-14 and 6-15. These two structures were modelled as a single crossing due to their proximity. These crossings are not utilized in the Timmins event. In the climate change scenario, they provide no appreciable conveyance with a receiving flow of only 0.03 $m^3$/s.

**Figure 6-15: Middle Spill Crossing at South Trent Street**

**Figure 6-16: South Spill Crossing at South Trent Street**

## 6.3 Spill Areas & Storage Impacts

There are three spill locations for the Cold Creek study area (see Figure 6-17). The peak flow contributing to each spill location for the 1% AEP and Timmins events is summarized in Table 6-9.

**Spill #1** is the largest spill in terms of peak flow rates and occurs at March Street. It is an external spill in that the flows crossing March Street do not return to the Cold Creek system. In the Timmins event, 44.3 $m^3/s$ occurs at Spill #1. This is significant as it represents 23% of the peak flow produced by the overall Cold Creek watershed.

**Spill #2** is a minimal internal spill and occurs at the Wallace Street berm north of Cold Creek. Flow does not breach the berm in the Timmins event. There is a small 300mm culvert in the berm that results in local drainage through the residential area before returning to Cold Creek.

**Spill #3** occurs at the spillway in the flood berm bound by Cold Creek to the west and South Trent Street to the east. It conveys 10.5 $m^3/s$ in the Timmins event and outlets to Crossings #4, 5, and 6. The flood berm associated with Spill #3 is 400m in length and effectively protects the houses along South Trent Street and Centre Street assuming it maintains structural stability in the Timmins event. The location of the flood berm and its spillway is evident in Figure 6-18.

**Table 6-7: Cold Creek Peak Spill Rates in 1% AEP and Timmins Event**

| Spill | 1% AEP | Timmins |
|---|---|---|
| 1 | 1.3 | 2.7 |
| 2 | 0.08 | 0.10 |
| 3 | 1.2 | 3.6 |


**Figure 6-17: Cold Creek Spills in Timmins Event**

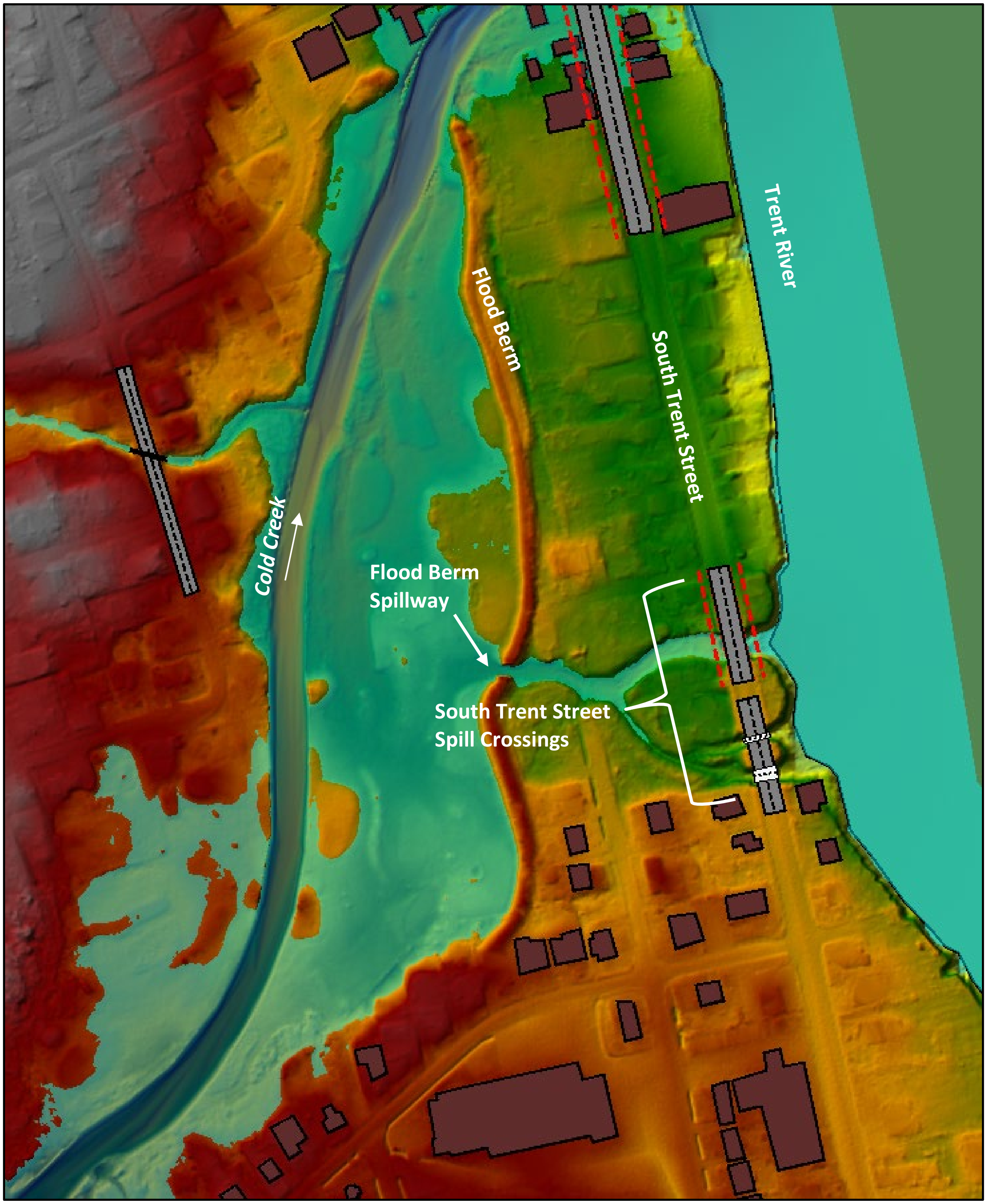



**Figure 6-18: Berm Between Cold Creek and South Trent Street with Spillway to South Trent Street Spill Crossings**

# 7 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.

## 7.1 Hydrologic Modelling

### *7.1.1 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.

The selection of precipitation depth estimate has been tested in the hydrologic model in relation to the peak discharge. For Mayhew Creek, it was found that a 30% increase in precipitation depth would lead

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

to a 71% increase in peak discharge (see Figure 7-1). Similarly, a 30% reduction would result in a 56% reduction in peak discharge.

At Cold Creek, a 30% increase in precipitation depth generated a 61% increase in peak flows and a 30% decrease resulted in a 59% reduction (see Figure 7-2).

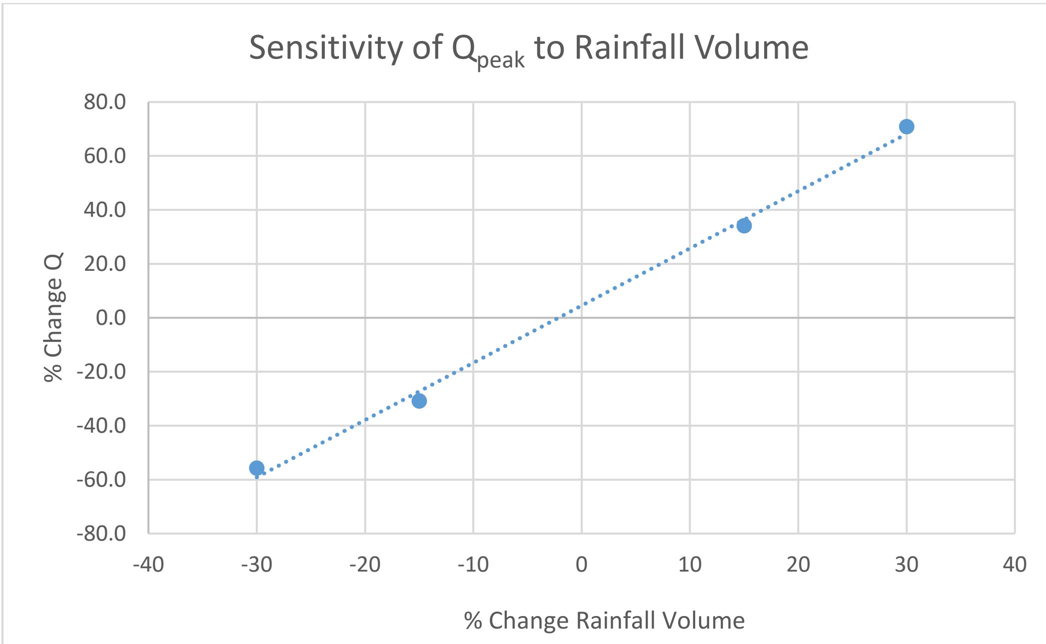



**Figure 7-1: Mayhew Creek - Sensitivity of Peak Runoff Rates to Rainfall Volume**

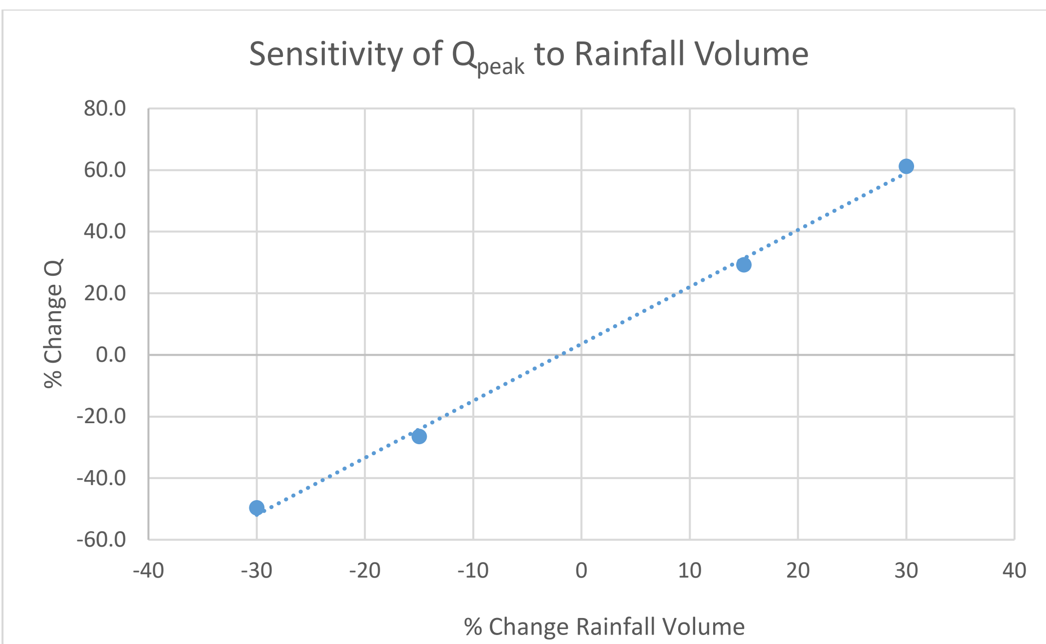



**Figure 7-2: Cold Creek - Sensitivity of Peak Runoff Rates to Rainfall Volume**

Rainfall volume is of particular interest. It is evident that there is an appreciable increase in peak flows with an increase in the rainfall volume. Since climate change considerations include increasing the rainfall depth by 25%, the increase in the peak runoff rate is significant.

### *7.1.2 Curve Number*

Variations in curve number produced the greatest impact on the modelled peak flows. The CN affects the losses and unsurprisingly has a significant influence on the peak runoff. A 29% increase in CN at Mayhew Creek results in a 78% increase in peak runoff and a 29% reduction results in a 45% reduction in peak runoff (see Figure 7-3).

The same effect was also observed at Cold Creek a 30% increase in CN resulted in a 50% increase in peak flow, while a 30% reduction resulted in a 38% reduction in peak flow (see Figure 7-4).

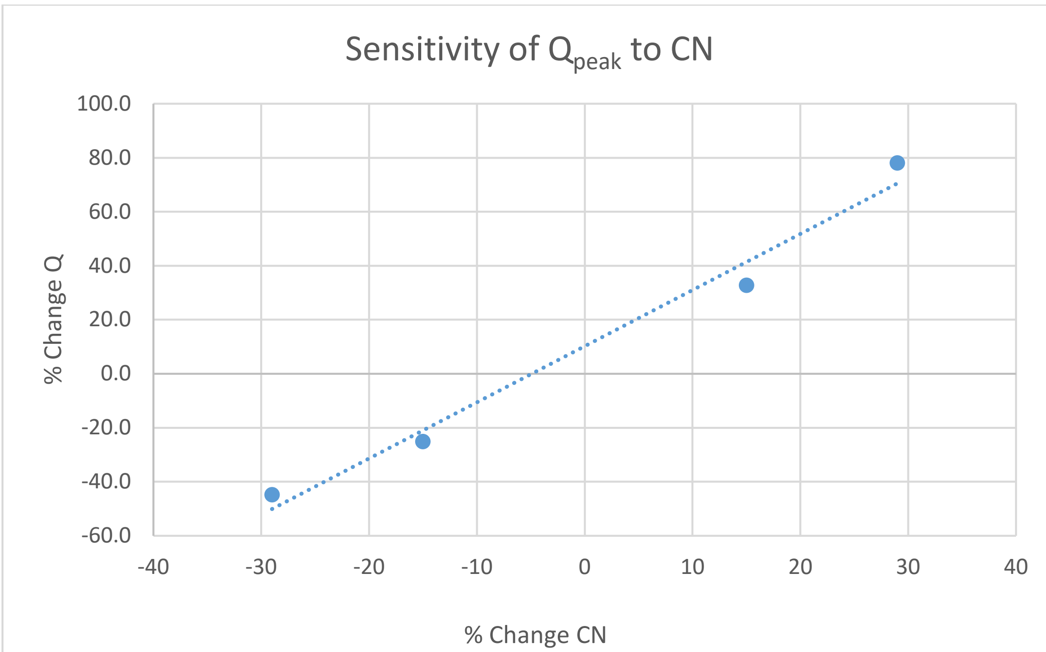



**Figure 7-3: Mayhew Creek - Sensitivity of Peak Runoff Rates to CN**

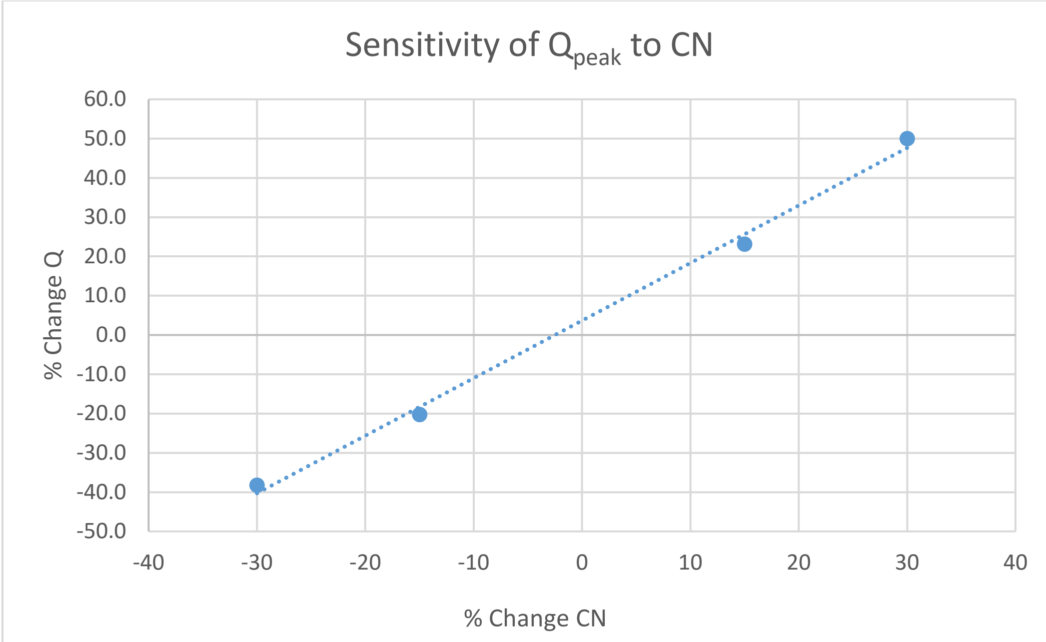



**Figure 7-4: Cold Creek - Sensitivity of Peak Runoff Rates to Curve Number**

### *7.1.3 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. At Mayhew Creek a 50% increase in lag time resulted in a 23% reduction in peak flow. Whereas a 50% reduction in lag time corresponded to a 53% reduction in peak flow (see Figure 7-5).

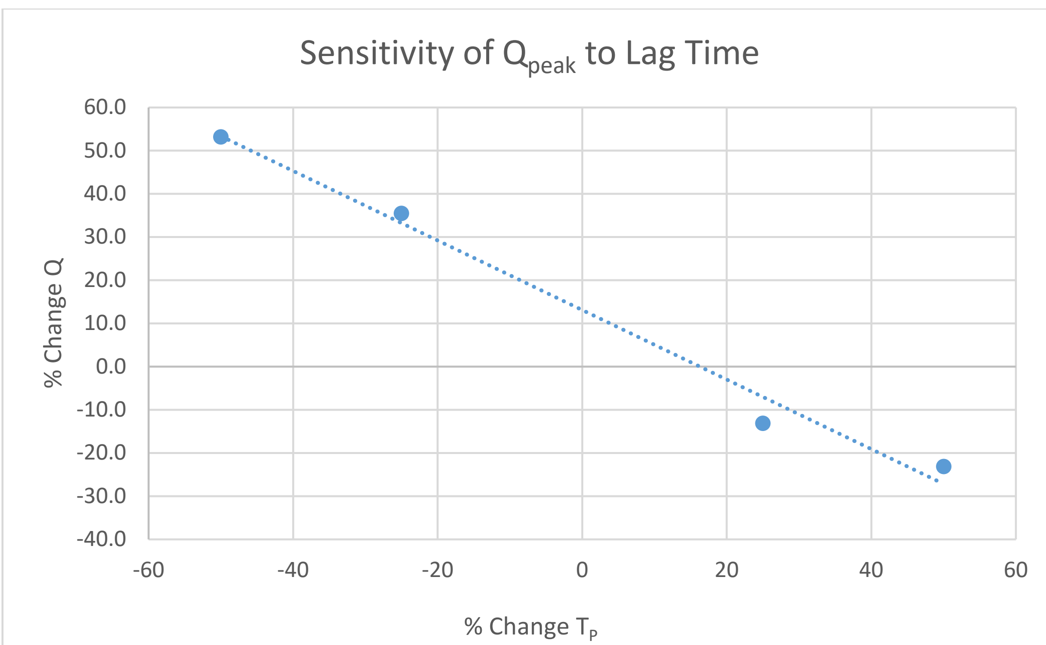



**Figure 7-5: Mayhew Creek - Sensitivity of Peak Runoff Rates to Lag Time**

At Cold Creek a 50% increase in lag time produced a 30% reduction in peak flow and a 50% reduction in lag time increased the peak flow 72% (see Figure 7-6).

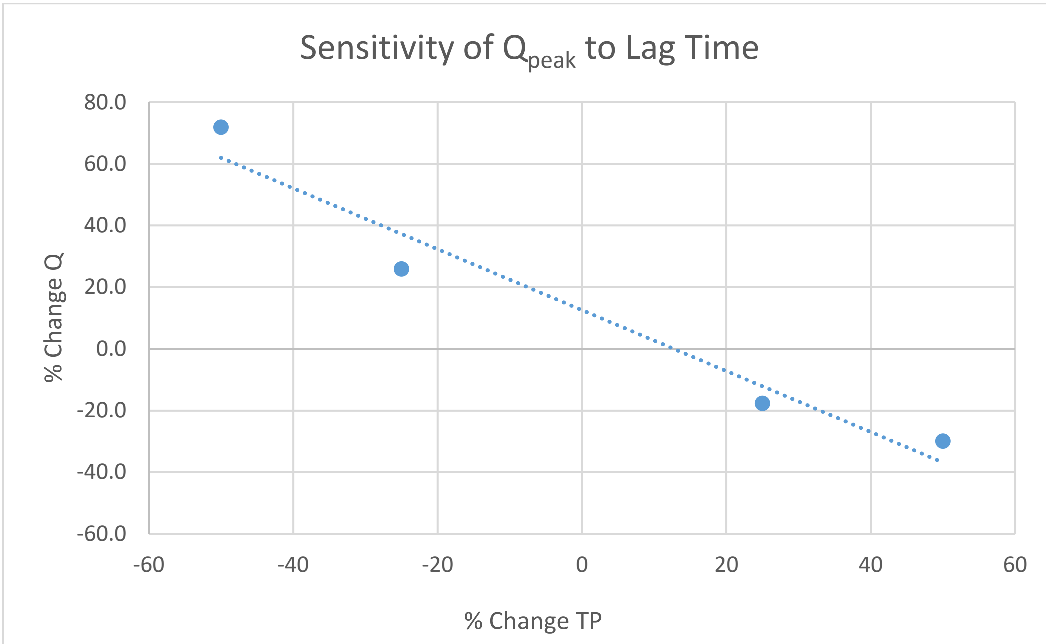



**Figure 7-6: Cold Creek - Sensitivity of Peak Runoff Rates to Lag Time**

## 7.2 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 (see Table 7-1). Mid-range values were selected and applied in the regulatory floodplain mapping. Both hydraulic models have moderate sensitivity to the Manning's n values. An example of the comparison of the flood limits for low, mid, and high-range values is shown in Figure 7-7 for the Timmins event.

**Table 7-1: 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 7-7: Example of Cold Creek Timmins Flood Hazard Limits for Low (Cyan), Mid (Red), and High (Green) Manning's n Values**

# 8 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.

## 8.1 Comparison of Historical Flood Limit to 2024 Mapping Update

A comparison of the historical flood limit and the 2024 flood limit is shown below. The white boundary represents the raw output from the hydraulic model that is used to produce the Timmins flood hazard limits identified in the 2024 floodplain mapping set (see Figures 8-1 to 8-3). The red line represents the historical flood hazard limit.

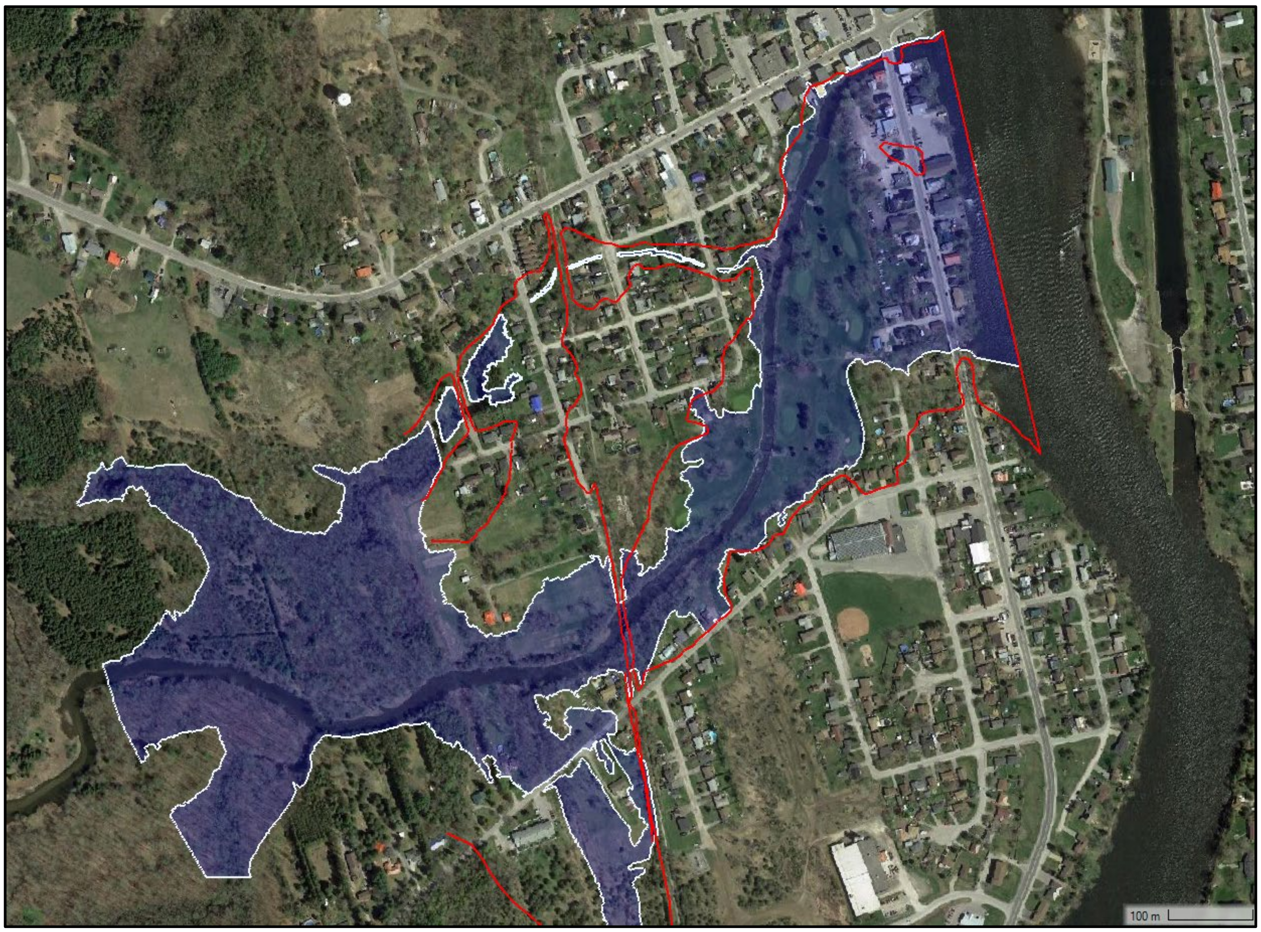



**Figure 8-1: Comparison of Historic vs. Updated Timmins Flood Hazard Limit for Cold Creek Main Study Area**

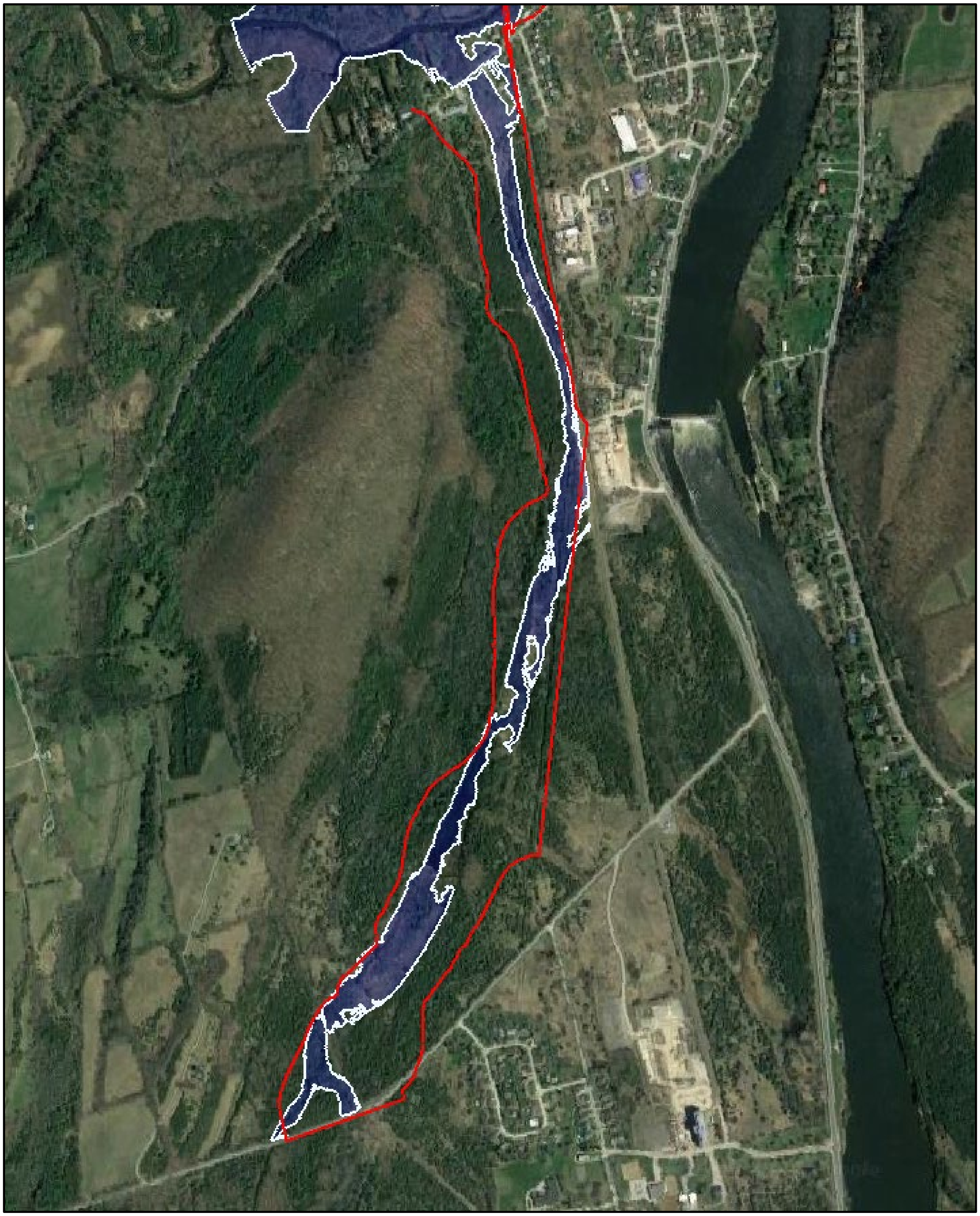


**Figure 8-2: Comparison of Historic vs. Updated Timmins Flood Hazard Limit for Cold Creek Spill Area**

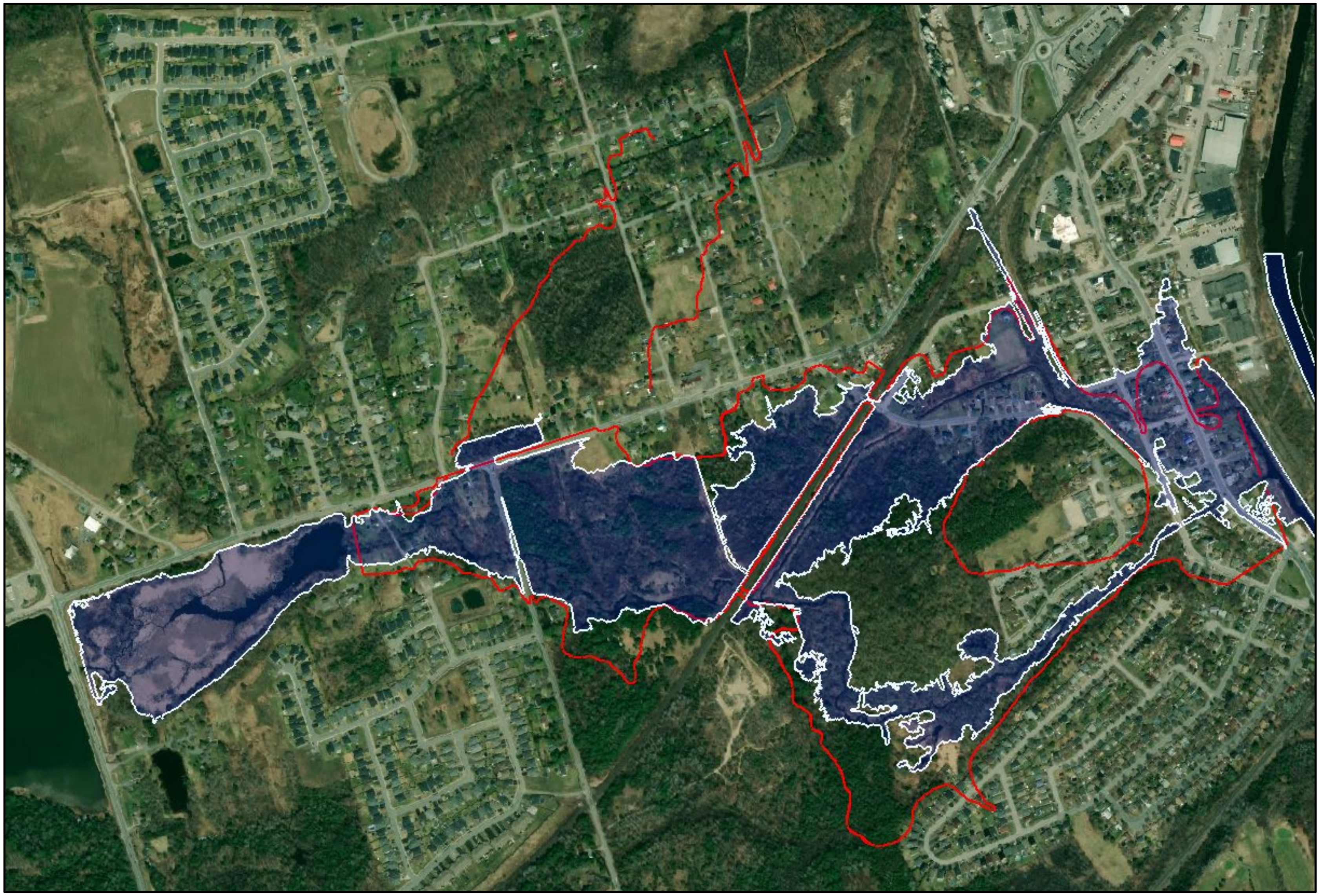


**Figure 8-3: Comparison of Historic vs. Updated Timmins Flood Hazard Limit for Mayhew Creek Main Study Area**

## 8.2 Water Surface Profiles

A plot of water surface profiles extending the full study area for both Mayhew and Cold Creeks is provided in Figures 8-3 and 8-4.

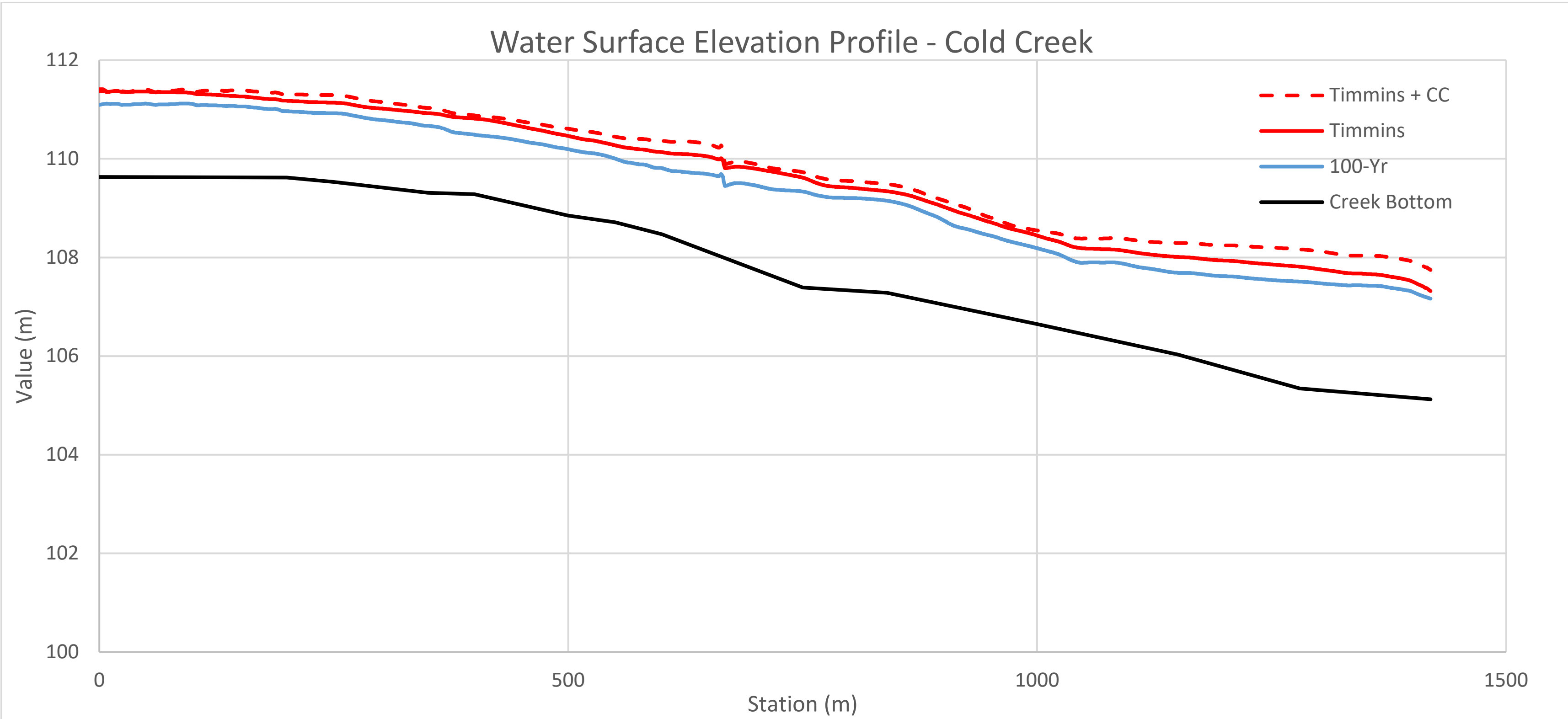



**Figure 8-4: Cold Creek Water Surface Profile for 100-Yr, Timmins, and Timmins + Climate Change Events**

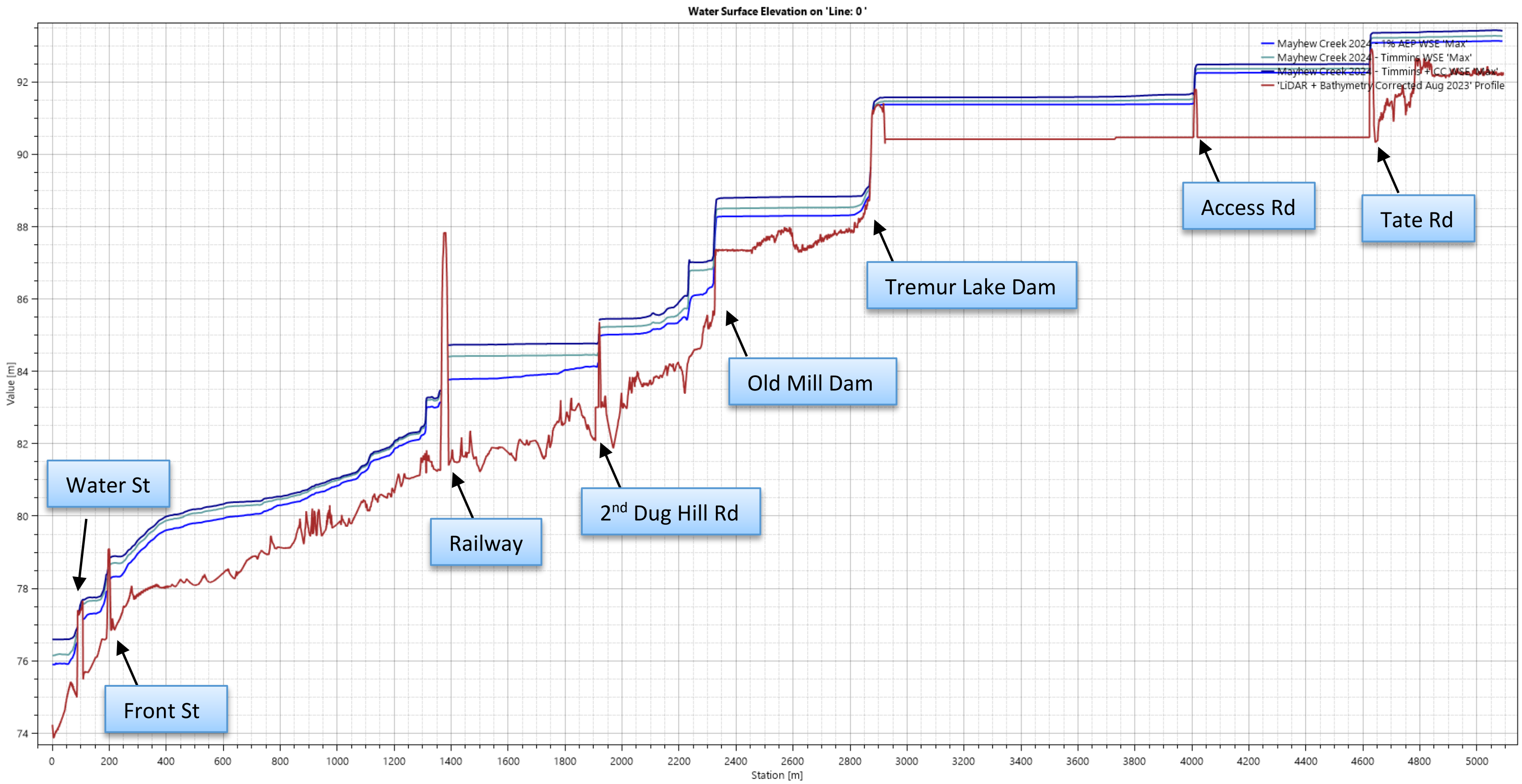



**Figure 8-5: Mayhew Creek Water Surface Profile for 100-Yr, Timmins, and Timmins + Climate Change Events**

## 8.3 Buildings within Flood Limit

The buildings within the Mayhew Creek flood hazard limit are predominantly between E Davis St and Store St, with many along Wooler Rd. The majority of buildings within the Cold Creek flood hazard limit are in the downtown Frankford area, near the intersection of Mill St and S Trent St.

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. Figure 8-5 and Figure 8-6 illustrates the building locations that are within the floodplain in the 100-yr, Timmins, and/or Timmins plus climate change events for Mayhew and Cold Creek, respectively.

The buildings that are within the floodplain in the 100-yr storm are illustrated by a yellow node. There are sixty-six (66) yellow nodes for Mayhew Creek and fourteen (14) yellow nodes for Cold Creek.

The buildings that are within the floodplain in the Timmins storm are illustrated by a pink node. There are twenty-four (24) pink nodes for Mayhew Creek and five (5) pink nodes for Cold Creek.

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 for Mayhew Creek and four (4) green nodes for Cold Creek.

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

**Table 8-1: Number of Buildings within Flood Hazard Limit for Respective Storm Events**

| Storm Event | No. of Buildings | |
|---|---|---|
| | Mayhew Creek | Cold Creek |
| 100-Yr | 66 | 14 |
| Timmins | 90 | 19 |
| Timmins + CC | 112 | 23 |

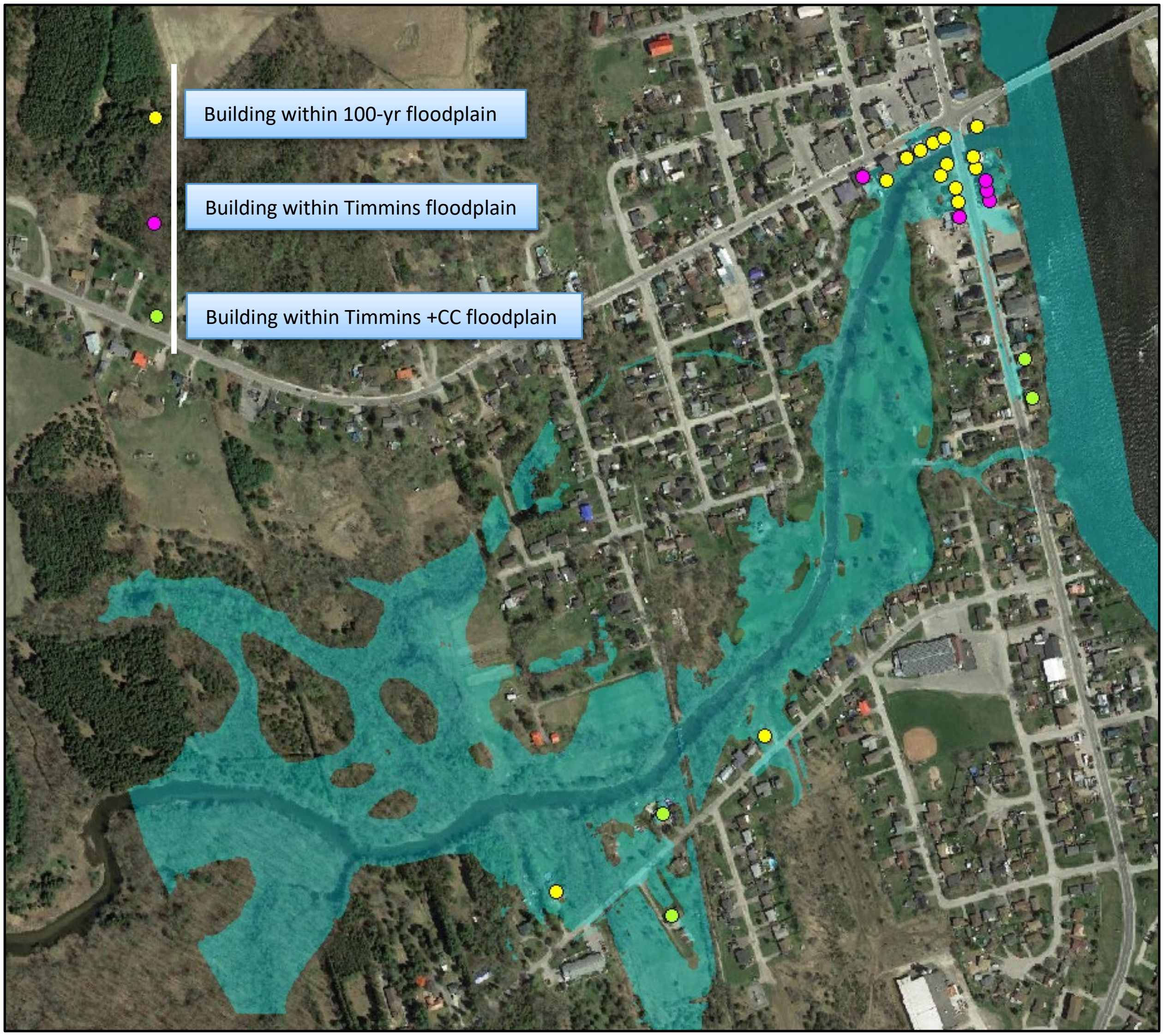



**Figure 8-6: Illustration of Buildings within Floodplain (Cold Creek)**

**Figure 8-7: Illustration of Buildings within Floodplain (Mayhew Creek)**

# 9 Conclusions

The Flood Hazard Identification Mapping Program has provided the opportunity for Lower Trent Conservation Authority, in partnership with the City of Quinte West and the provincial and federal partners, to complete the *2024 Mayhew and Cold 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 Mayhew and Cold Creek floodplain maps. The current mapping will allow Conservation and City 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 Frankford and Trenton.

Section 8.3 identifies the buildings currently within the 1% AEP, Timmins, and/or climate change storm events for both creeks. 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 Mayhew and Cold Creek Floodplain Mapping Update* that will benefit the local community within the City of Quinte West for many years to come.

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Project Director:


Bryon Keene, P.Eng.
Jewell Engineering Inc.

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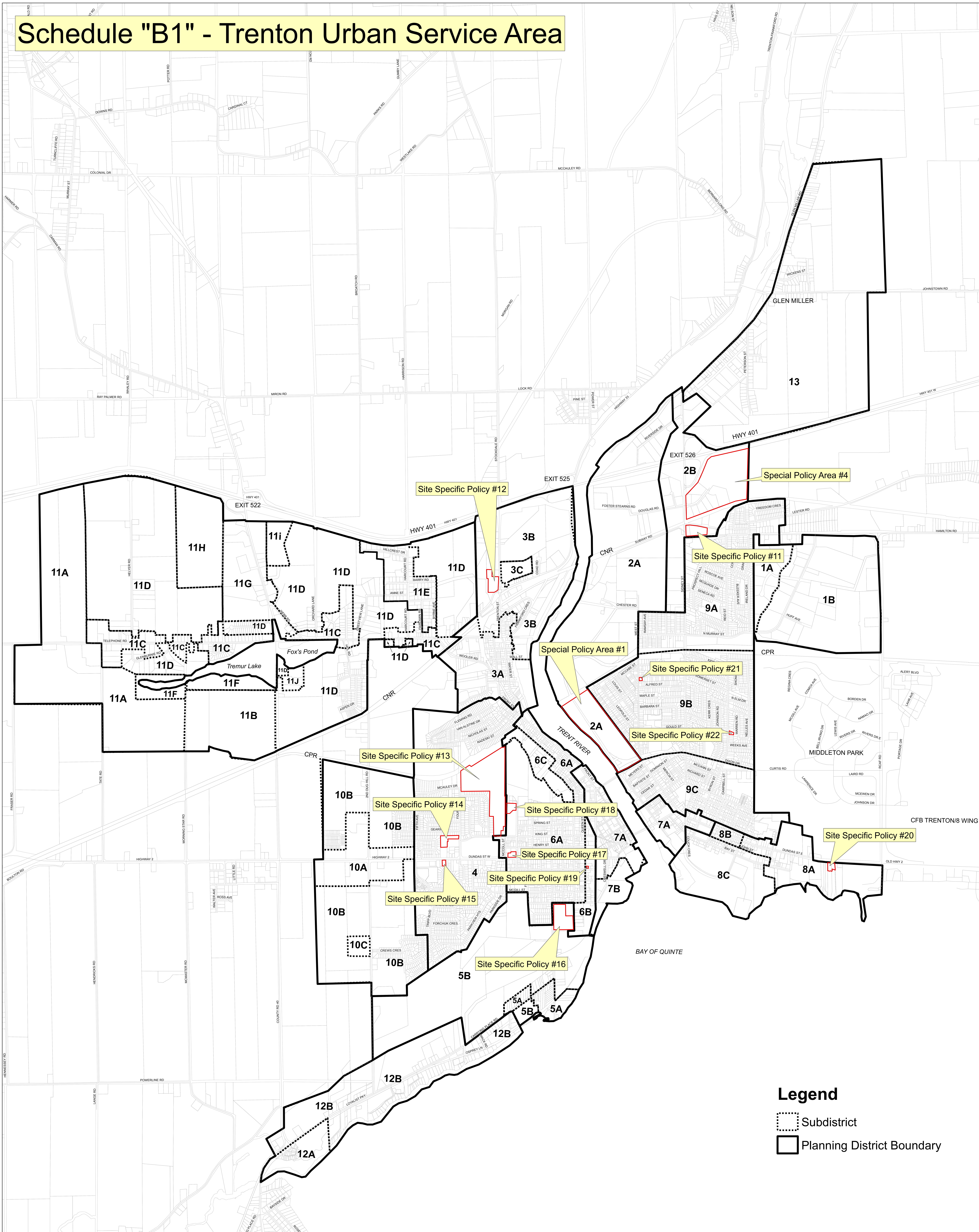
# Appendix A:
## Official Plan – Schedule B1, B2

# Schedule "B1" - Trenton Urban Service Area




Quinte West
A Natural Attraction

Produced by the City of Quinte West
GIS Division. Copyright 2015.
File: B1trenton_planning_area.mxd
Date: August 10, 2015

# Schedule "B2" - Frankford Urban Service Area

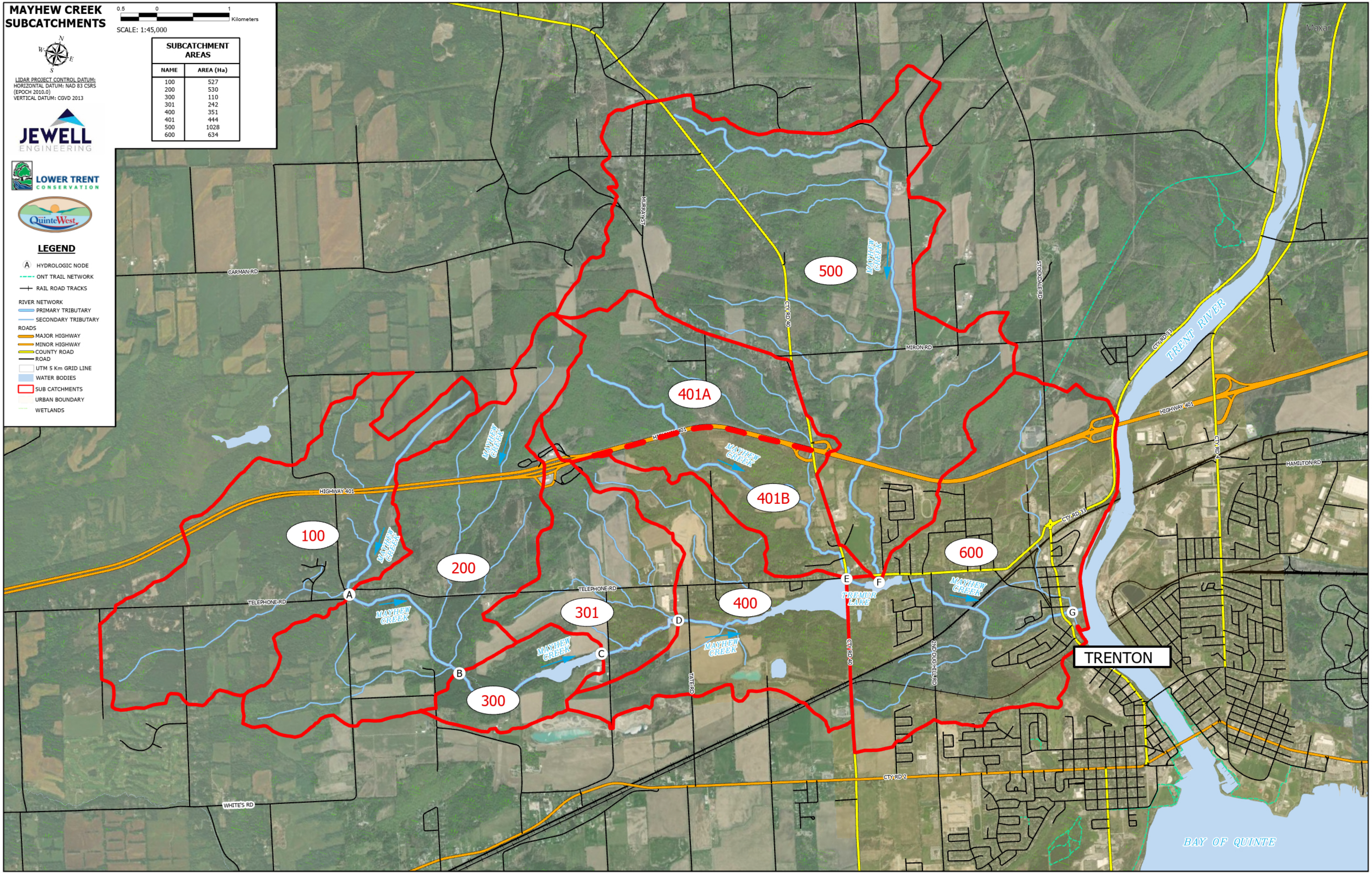

QuinteWest
A Natural Attraction

Produced by the City of Quinte West
GIS Division. Copyright, 2015.
File:B2frankford_planning_area.mxd
Date: May 28, 2015

0 100 200 400 600 800 Meters

N E S W

# Appendix B-1:
## Mayhew Creek Catchment Areas & Descriptions




**SUBCATCHMENT AREAS**

| NAME | AREA (Ha) |
|---|---|
| 100 | 527 |
| 200 | 530 |
| 300 | 110 |
| 301 | 242 |
| 400 | 351 |
| 401 | 444 |
| 500 | 1028 |
| 600 | 634 |

***Mayhew Creek Sub-Catchment Descriptions***

As shown in Appendix B, the Mayhew Creek watershed was divided into eight sub-catchments based on confluence points and nodes of interest.

Sub-catchment 100 has a drainage area of 527ha, covering approximately 14% of the watershed. This sub-catchment is the westernmost sub-catchment in the Mayhew Creek watershed. The soils within sub-catchment 100 are predominately soils group A, meaning they have high infiltration and transmission rates when wet. Cultivated land has the greatest land coverage at 49%.

Sub-catchment 200 has a drainage area of 530ha, covering approximately 14% of the watershed. This sub-catchment is due east of sub-catchment 100. The soils here are predominately soils group B, which suggests that they have moderate infiltration and transmission rates when wet. 43% of the sub-catchment is comprised of cultivated land.

Sub-catchments 300 and 301 have drainage areas of 110ha and 242ha, respectively, covering 9% of the watershed total. The soils within sub-catchments 300 and 301 are predominately soils group A, suggesting high infiltration and transmission rates when wet. Cultivated land covers 53% of the two sub-catchments.

Sub-catchment 400 has a drainage area of 351ha, covering approximately 9% of the watershed. The soils are predominately soils group A and B, meaning moderate to high infiltration and transmission rates when wet. Cultivated lands make up 52% of the area.

Sub-catchment 401 has a drainage area of 444ha, covering approximately 11% of the watershed. The soils are predominately soils group A, meaning high infiltration and transmission rates when wet. Cultivated land, water, and woods have similar coverage within the sub-catchment at 31%, 30%, and 30%, respectively.

Sub-catchment 500 has a drainage area of 1,028ha, covering approximately 37% of the watershed. Sub-catchment 500 is located at the northern most part of the Mayhew Creek watershed. The soils are predominately soils group A, which have high infiltration and transmission rates when wet. Cultivated land is the most dominant land cover, covering 46% of the sub-catchment area.

Sub-catchment 600 has a drainage area of 634ha, covering approximately 16% of the watershed. This sub-catchment is at the eastern end of the Mayhew Creek watershed, along the Trent River. The soils are predominately soils group C, which have slow infiltration and transmission rates when wet. 38% of sub-catchment 600 is impervious lands.

***Mayhew Creek Node Descriptions***

Node A is located at the intersection of Telephone Rd and Christiani Rd. All of sub-catchment 100 drains to Node A.

Node B is located southeast of Node A. Sub-catchments 100 and 200 contribute to the modeled peak flows at Node B.

Node C is downstream of the Glenburnie Reservoir near Fraser Rd. Sub-catchment 100, 200, and 300 drain to Node C.

Node D is northeast of Node C, near Tate Rd. Sub-catchments 100-301 contribute to the modeled peak flows at Node D.

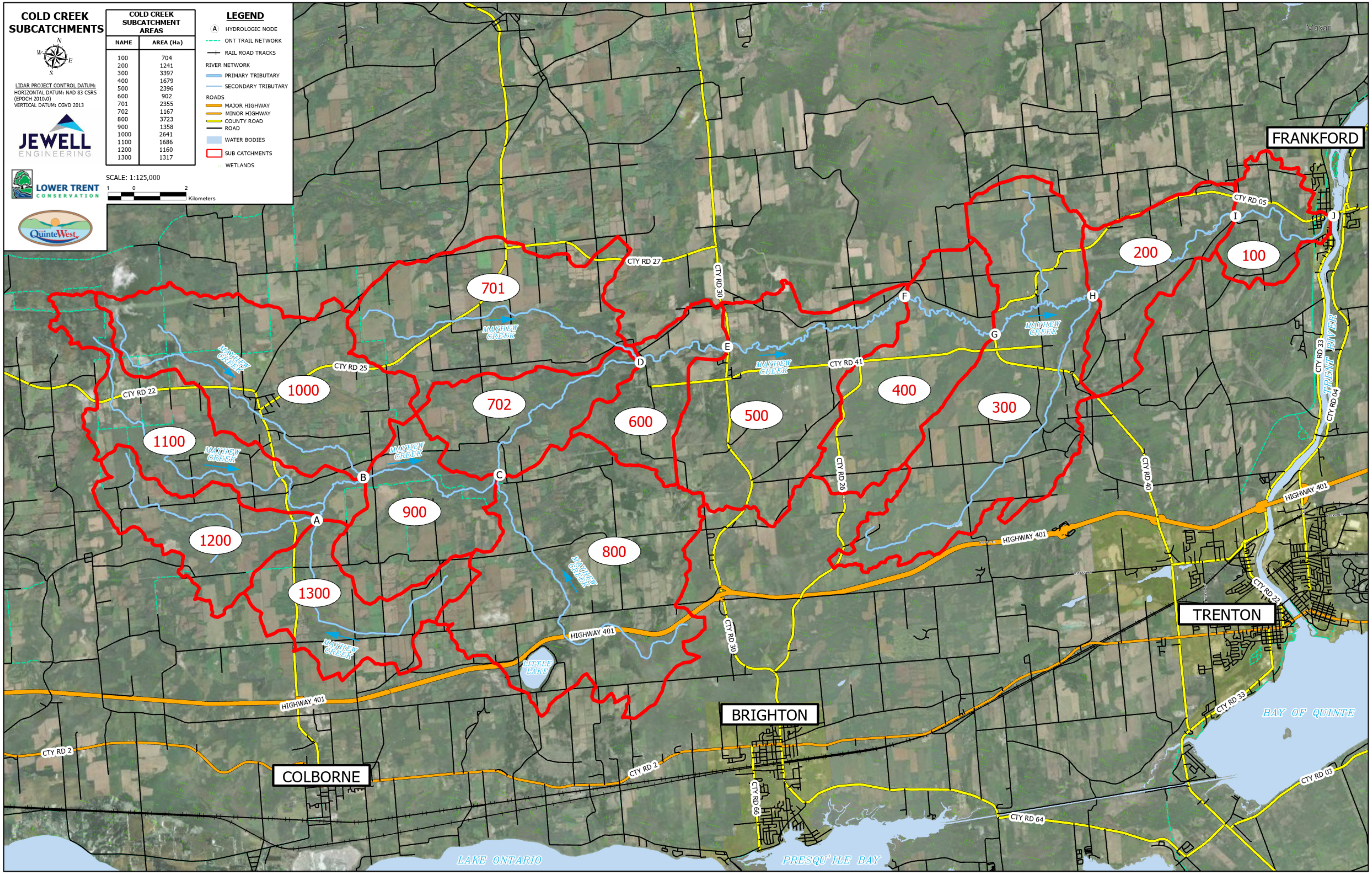
Node E is located at the intersection of Telephone Rd and County Rd 40, near the outlet of Tremur Lake. Sub-catchments 100-401 drain to Node E.

Node F is south of the intersection of Telephone Rd and Orchard Ln. The main tributary of sub-catchment 500 converges with the main branch of Mayhew Creek at Node F. Sub-catchments 100-500 contribute to the peak flows at Node F.

Node G is located at the very downstream end of the Mayhew Creek watershed and discharges to the Trent River.

# Appendix B-2:
## Cold Creek Catchment Areas & Descriptions

# COLD CREEK SUBCATCHMENTS




| COLD CREEK SUBCATCHMENT AREAS | |
|---|---|
| NAME | AREA (Ha) |
| 100 | 704 |
| 200 | 1241 |
| 300 | 3397 |
| 400 | 1679 |
| 500 | 2396 |
| 600 | 902 |
| 701 | 2355 |
| 702 | 1167 |
| 800 | 3723 |
| 900 | 1358 |
| 1000 | 2641 |
| 1100 | 1686 |
| 1200 | 1160 |
| 1300 | 1317 |

***Cold Creek Sub-Catchment Descriptions***

As shown in Appendix B, the Cold Creek watershed was divided into 14 sub-catchments based on confluence points and nodes of interest.

Sub-catchment 100 has a drainage area of 704ha, covering approximately 3% of the watershed. This sub-catchment is the easternmost sub-catchment in the Mayhew Creek watershed, and discharges into the Trent River. The soils within sub-catchment 100 are predominately soils group B, meaning they have moderate infiltration and transmission rates when wet. Cultivated land has the greatest land coverage at 50%.

Sub-catchment 200 has a drainage area of 1,241ha, covering approximately 5% of the watershed. This sub-catchment is due west of sub-catchment 100. The soils here are predominately soils group A and B, which suggests that they have moderate to high infiltration and transmission rates when wet. 53% of the sub-catchment is comprised of cultivated land.

Sub-catchment 300 has a drainage area of 3,397ha, covering approximately 13% of the watershed. The soils are predominately soils group A and B, meaning moderate to high infiltration and transmission rates when wet. Cultivated lands make up 59% of the area.

Sub-catchment 400 has a drainage area of 1,680ha, covering approximately 7% of the watershed. The soils are predominately soils group B, which have moderate infiltration and transmission rates when wet. Cultivated land is the most dominant land cover, covering 68% of the sub-catchment area.

Sub-catchment 500 has a drainage area of 2,397ha, covering approximately 9% of the watershed. The soils are predominately soils group A, which have high infiltration and transmission rates when wet. 61% of sub-catchment 500 is cultivated land.

Sub-catchment 600 has a drainage area of 902ha, covering approximately 4% of the watershed. The soils are predominately soils group A, which have high infiltration and transmission rates when wet. 49% of sub-catchment 600 is cultivated land.

Sub-catchment 701 has a drainage area of 2,355ha, covering approximately 9% of the watershed. The soils are predominately soils group A and B, which have moderate to high infiltration and transmission rates when wet. Cultivated land is the most dominant land cover, covering 66% of the sub-catchment area.

Sub-catchment 702 has a drainage area of 1,167ha, covering approximately 5% of the watershed. The soils are predominately soils group A, meaning high infiltration and transmission rates when wet. Cultivated land and woods cover the largest areas of land in sub-catchment 702.

Sub-catchment 800 has a drainage area of 3,723ha, covering approximately 14% of the watershed. The soils are predominately soils group B, which have moderate infiltration and transmission rates when wet. Cultivated land is the most dominant land cover, covering 61% of the sub-catchment area.

Sub-catchment 900 has a drainage area of 1,359ha, covering approximately 5% of the watershed. 60% of soils are soils group B, which are soils with moderate infiltration and transmission rates when wet. Cultivated land is the most dominant land cover, covering 60% of the sub-catchment area.

Sub-catchment 1000 has a drainage area of 2,642ha, covering approximately 10% of the watershed. The soils are predominately soils group A, which suggests that they have high infiltration and transmission rates when wet. 58% of the sub-catchment is comprised of cultivated land.

Sub-catchment 1100 has a drainage area of 1,686ha, covering approximately 7% of the watershed. The soils within sub-catchment 1100 are predominately soils group A, meaning they have high infiltration and transmission rates when wet. Cultivated land has the greatest land coverage at 48%.

Sub-catchment 1200 has a drainage area of 1,160ha, covering approximately 5% of the watershed. The soils are predominately soils group A and B, meaning moderate to high infiltration and transmission rates when wet. Cultivated lands make up 48% of the area.

Sub-catchment 1300 has a drainage area of 1,317ha, covering approximately 5% of the watershed. The soils are predominately soils group B, meaning moderate infiltration and transmission rates when wet. Cultivated land covers 67% of the sub-catchment.

***Cold Creek Node Descriptions***

Node A is located at the west end of the watershed, directly north of Colborne. Sub-catchments 1200 and 1300 drain to Node A.

Node B is located northeast of Node A. Sub-catchments 1000-1300 drain to Node B.

Node C is located due east of Node B at the of sub-catchment 702, 800, and 900. Sub-catchments 800-1300 contribute to the peak flows modeled at Node C.

Node D is located west of Orland. Sub-catchments 701-1300 drain to Node D.

Node E is just north of the intersection of Cty Rd 30 and Cty Rd 41 in Orland. Sub-catchments 600-1300 drain to Node E. Node E corresponds to the 'Cold Creek at Orland' stream flow gauge location.

Node F is located southeast of Old Wooler Rd's intersection with Gainsforth Rd. Sub-catchments 500-1300 contribute to the modeled peak flows at Node F.

Node G is located just north of the intersection of County Rd 40 and County Rd 28, and is south of Wooler. Sub-catchments 400-1300 drain to Node G.

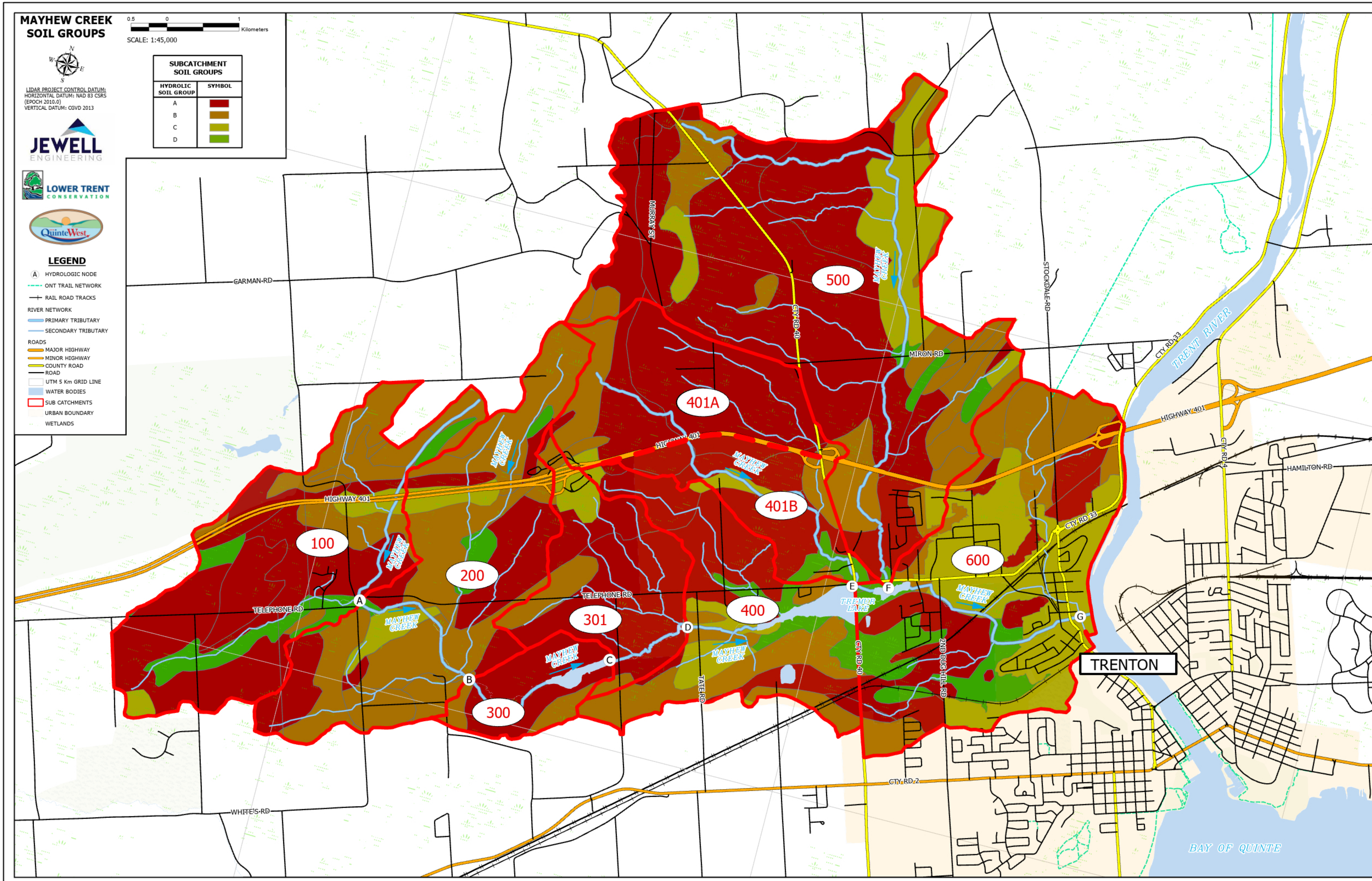
Node H is northeast of Node G. Node H receives drainage from Sub-catchments 300-1300.

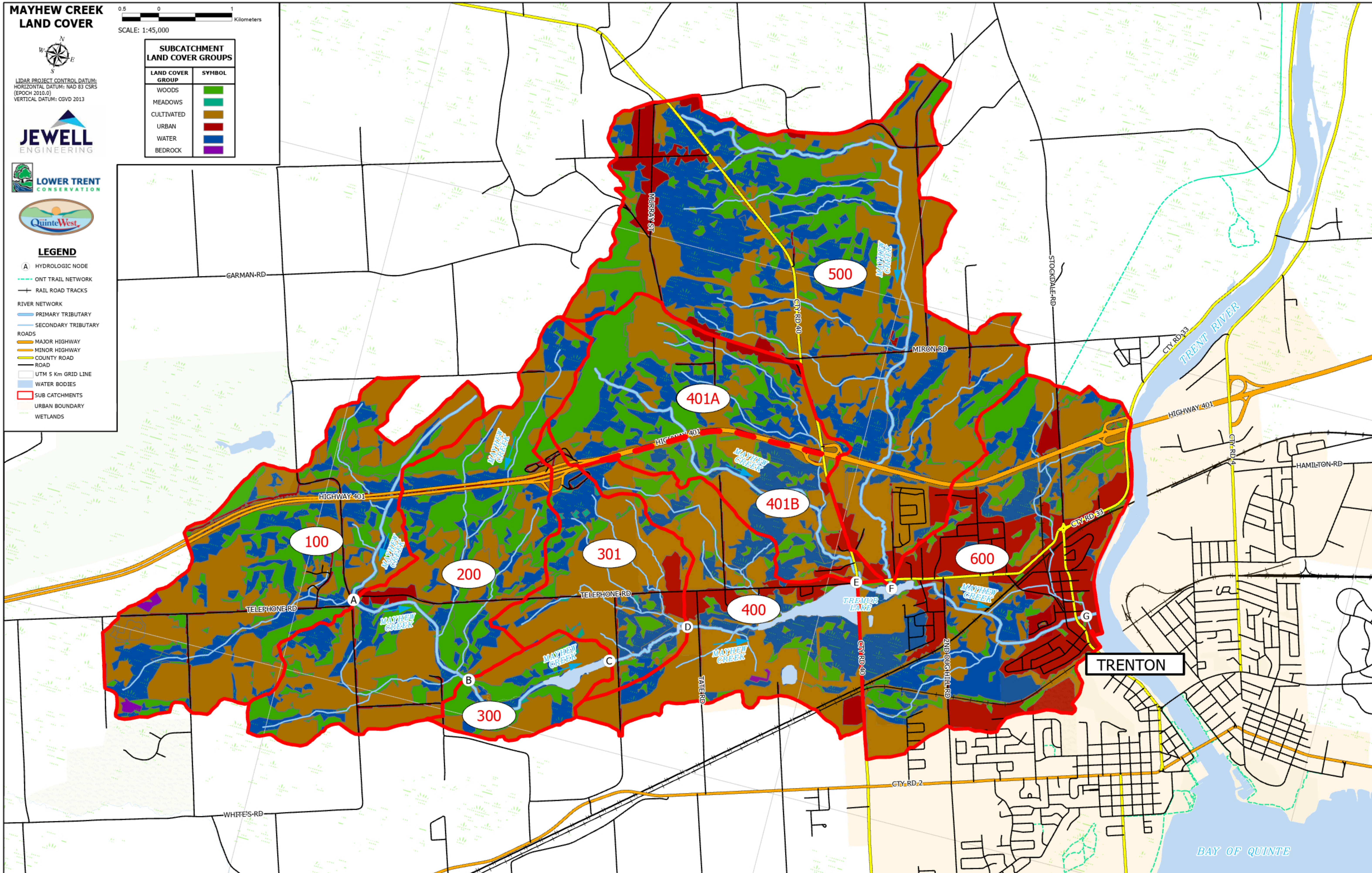
Node I is located at the eastern end of the watershed along Stockdale Rd at the Stockdale Mill. Sub-catchments 200-1300 contribute to the peak flows modeled at Node I.

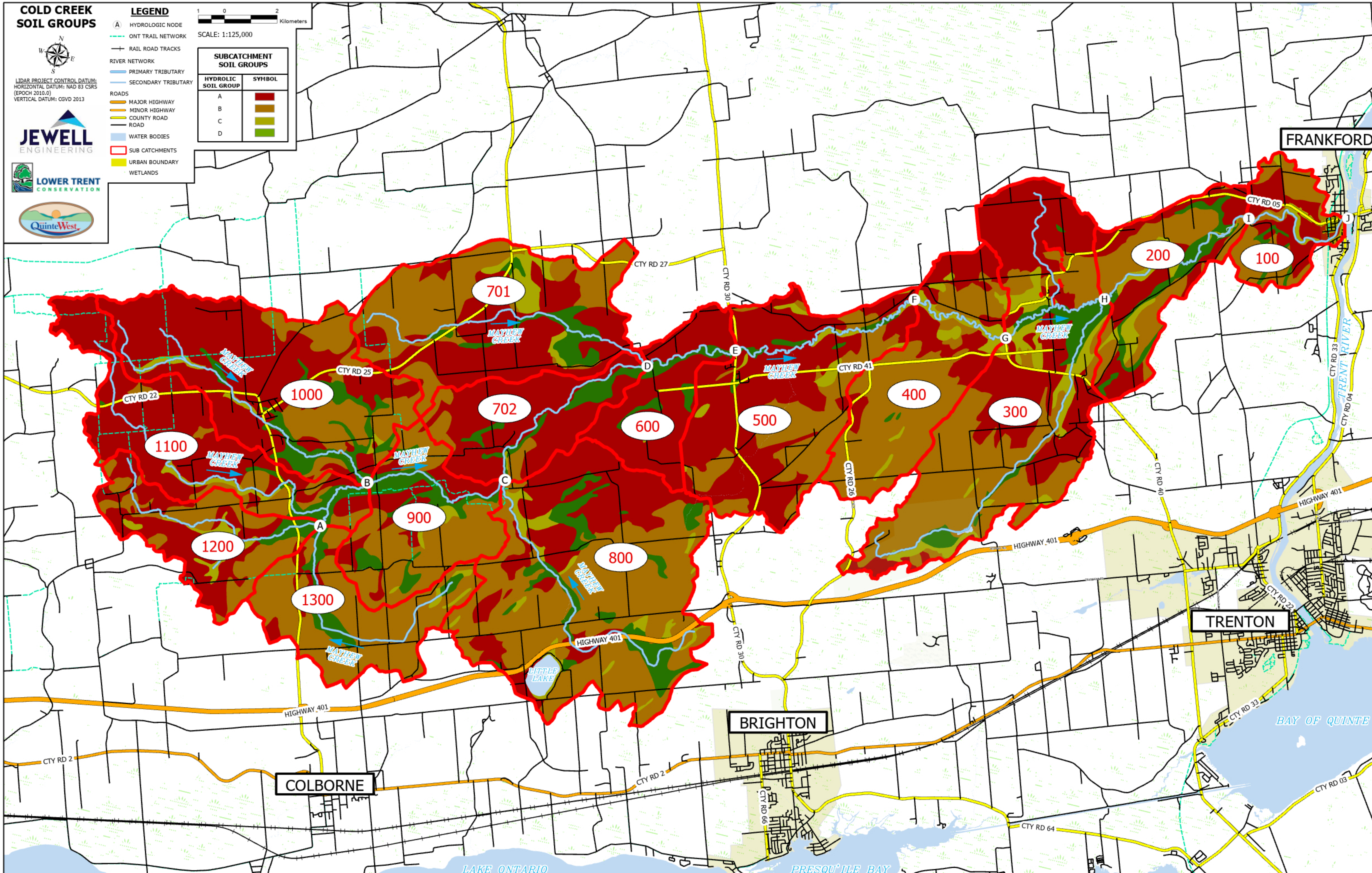
Node J is located at the easternmost point of the Cold Creek watershed. The entire watershed ultimately drains to Node J before outletting to the Trent River in Frankford.

# Appendix C:
## Soil and Land Cover Maps

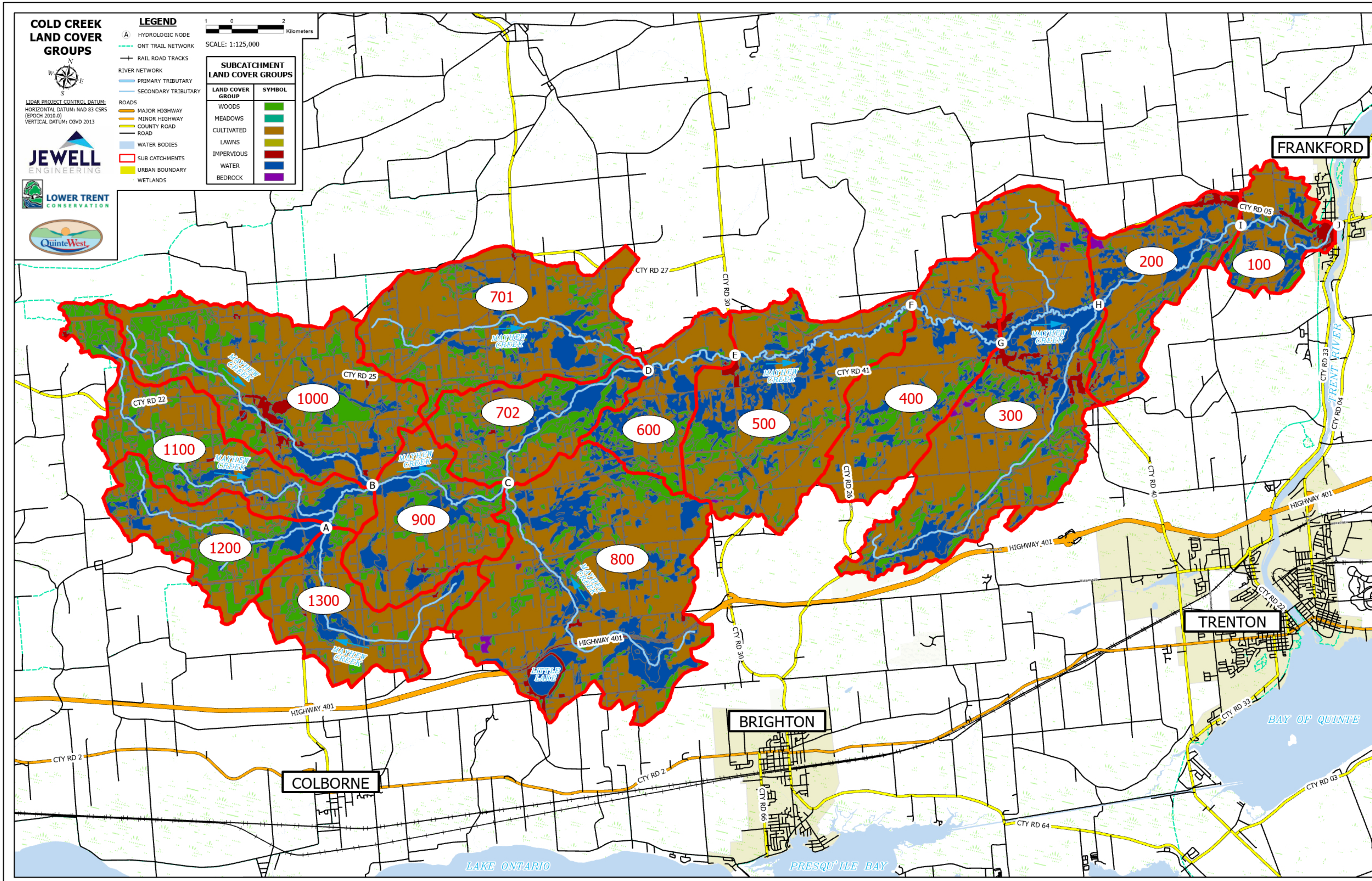
JEWELL ENGINEERING


MAYHEW CREEK SOIL GROUPS
SCALE: 1:45,000
Kilometers
LIDAR PROJECT CONTROL DATUM:
HORIZONTAL DATUM: NAD 83 CSRS (EPOCH 2010.0)
VERTICAL DATUM: CGVD 2013
JEWELL ENGINEERING
LOWER TRENT CONSERVATION
QuinteWest
SUBCATCHMENT SOIL GROUPS
HYDROLIC SOIL GROUP
SYMBOL
A
B
C
D
LEGEND
HYDROLOGIC NODE
ONT TRAIL NETWORK
RAIL ROAD TRACKS
RIVER NETWORK
PRIMARY TRIBUTARY
SECONDARY TRIBUTARY
ROADS
MAJOR HIGHWAY
MINOR HIGHWAY
COUNTY ROAD
ROAD
UTM 5 Km GRID LINE
WATER BODIES
SUB CATCHMENTS
URBAN BOUNDARY
WETLANDS
100
200
300
301
400
401A
401B
500
600
A
B
C
D
E
F
G
MAYHEW CREEK
TREMUR LAKE
TRENTON
TRENT RIVER
BAY OF QUINTE
HIGHWAY 401
TELEPHONE RD
GARMAN RD
MURRAY ST
CTY RD 40
MIRON RD
STOCKDALE RD
CTY RD 33
CTY RD 4
HAMILTON RD
TATE RD
2ND DUG HILL RD
CTY RD 2
WHITE'S RD


MAYHEW CREEK
LAND COVER
SCALE: 1:45,000
Kilometers
LIDAR PROJECT CONTROL DATUM:
HORIZONTAL DATUM: NAD 83 CSRS
(EPOCH 2010.0)
VERTICAL DATUM: CGVD 2013
JEWELL
ENGINEERING
LOWER TRENT
CONSERVATION
QuinteWest
SUBCATCHMENT LAND COVER GROUPS
LAND COVER GROUP
SYMBOL
WOODS
MEADOWS
CULTIVATED
URBAN
WATER
BEDROCK
LEGEND
HYDROLOGIC NODE
ONT TRAIL NETWORK
RAIL ROAD TRACKS
RIVER NETWORK
PRIMARY TRIBUTARY
SECONDARY TRIBUTARY
ROADS
MAJOR HIGHWAY
MINOR HIGHWAY
COUNTY ROAD
ROAD
UTM 5 Km GRID LINE
WATER BODIES
SUB CATCHMENTS
URBAN BOUNDARY
WETLANDS
100
200
300
301
400
401A
401B
500
600
A
B
C
D
E
F
G
HIGHWAY 401
TELEPHONE RD
GARMAN RD
MURRAY ST
CTY RD 40
MIRON RD
STOCKDALE RD
CTY RD 33
CTY RD 4
HAMILTON RD
TATE RD
2ND DUGHILL RD
CTY RD 2
WHITE'S RD
MAYHEW CREEK
TREMUR LAKE
TRENT RIVER
TRENTON
BAY OF QUINTE



COLD CREEK SOIL GROUPS
LEGEND
HYDROLOGIC NODE
ONT TRAIL NETWORK
RAIL ROAD TRACKS
RIVER NETWORK
PRIMARY TRIBUTARY
SECONDARY TRIBUTARY
ROADS
MAJOR HIGHWAY
MINOR HIGHWAY
COUNTY ROAD
ROAD
WATER BODIES
SUB CATCHMENTS
URBAN BOUNDARY
WETLANDS
Kilometers
SCALE: 1:125,000
SUBCATCHMENT SOIL GROUPS
HYDROLOGIC SOIL GROUP
SYMBOL
A
B
C
D
LIDAR PROJECT CONTROL DATUM:
HORIZONTAL DATUM: NAD 83 CSRS
(EPOCH 2010.0)
VERTICAL DATUM: CGVD 2013
JEWELL ENGINEERING
LOWER TRENT CONSERVATION
QuinteWest
FRANKFORD
TRENTON
BRIGHTON
COLBORNE
100
200
300
400
500
600
701
702
800
900
1000
1100
1200
1300
MAYHEW CREEK
LITTLE LAKE
CTY RD 05
CTY RD 27
CTY RD 30
CTY RD 25
CTY RD 22
CTY RD 41
CTY RD 26
CTY RD 40
CTY RD 33
CTY RD 04
CTY RD 2
CTY RD 66
CTY RD 64
CTY RD 03
HIGHWAY 401
TRENT RIVER
BAY OF QUINTE
LAKE ONTARIO
PRESQU'ILE BAY


COLD CREEK LAND COVER GROUPS
LEGEND
HYDROLOGIC NODE
ONT TRAIL NETWORK
RAIL ROAD TRACKS
RIVER NETWORK
PRIMARY TRIBUTARY
SECONDARY TRIBUTARY
ROADS
MAJOR HIGHWAY
MINOR HIGHWAY
COUNTY ROAD
ROAD
WATER BODIES
SUB CATCHMENTS
URBAN BOUNDARY
WETLANDS
SCALE: 1:125,000
Kilometers
SUBCATCHMENT LAND COVER GROUPS
LAND COVER GROUP
SYMBOL
WOODS
MEADOWS
CULTIVATED
LAWNS
IMPERVIOUS
WATER
BEDROCK
LIDAR PROJECT CONTROL DATUM:
HORIZONTAL DATUM: NAD 83 CSRS (EPOCH 2010.0)
VERTICAL DATUM: CGVD 2013
JEWELL ENGINEERING
LOWER TRENT CONSERVATION
QuinteWest
FRANKFORD
TRENTON
BRIGHTON
COLBORNE
100
200
300
400
500
600
701
702
800
900
1000
1100
1200
1300
MAYHEW CREEK
LITTLE LAKE
HIGHWAY 401
CTY RD 05
CTY RD 27
CTY RD 30
CTY RD 25
CTY RD 22
CTY RD 41
CTY RD 26
CTY RD 40
CTY RD 33
CTY RD 04
CTY RD 2
CTY RD 66
CTY RD 64
CTY RD 03
TRENT RIVER
BAY OF QUINTE
LAKE ONTARIO
PRESQU'ILE BAY

# 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 |

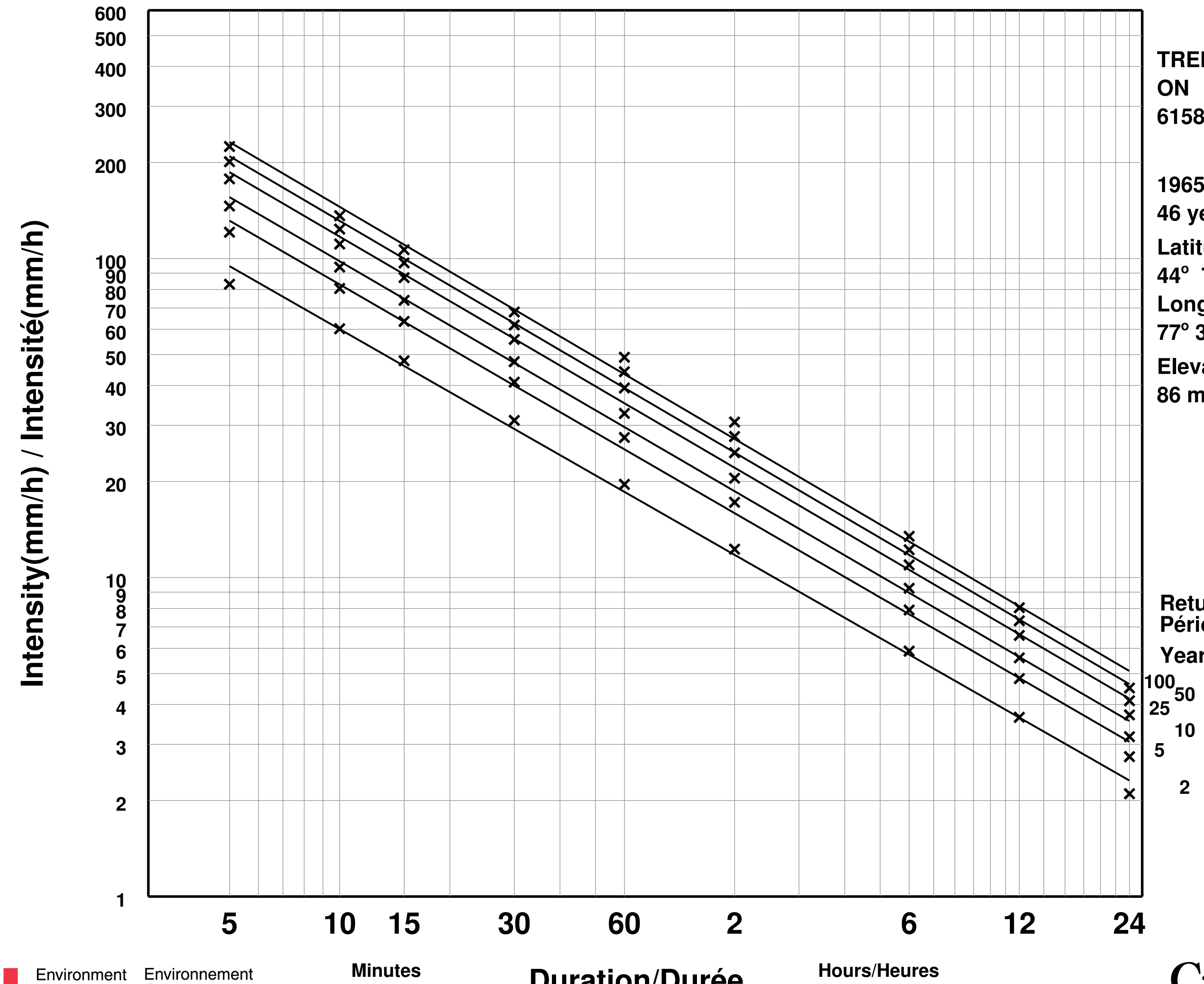
# Appendix E:
## Trenton A Environment Canada IDFs

JEWELL
ENGINEERING

# 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




TRENTON A
ON
6158875

1965 – 2017
46 years / ans

Latitude
44° 7'N
Longitude
77° 32'W
Elevation / Altitude
86 m

Environment Canada Environnement Canada

Canada

# Appendix F:
## Gumbel Distribution Precipitation Frequency Curve

**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 |

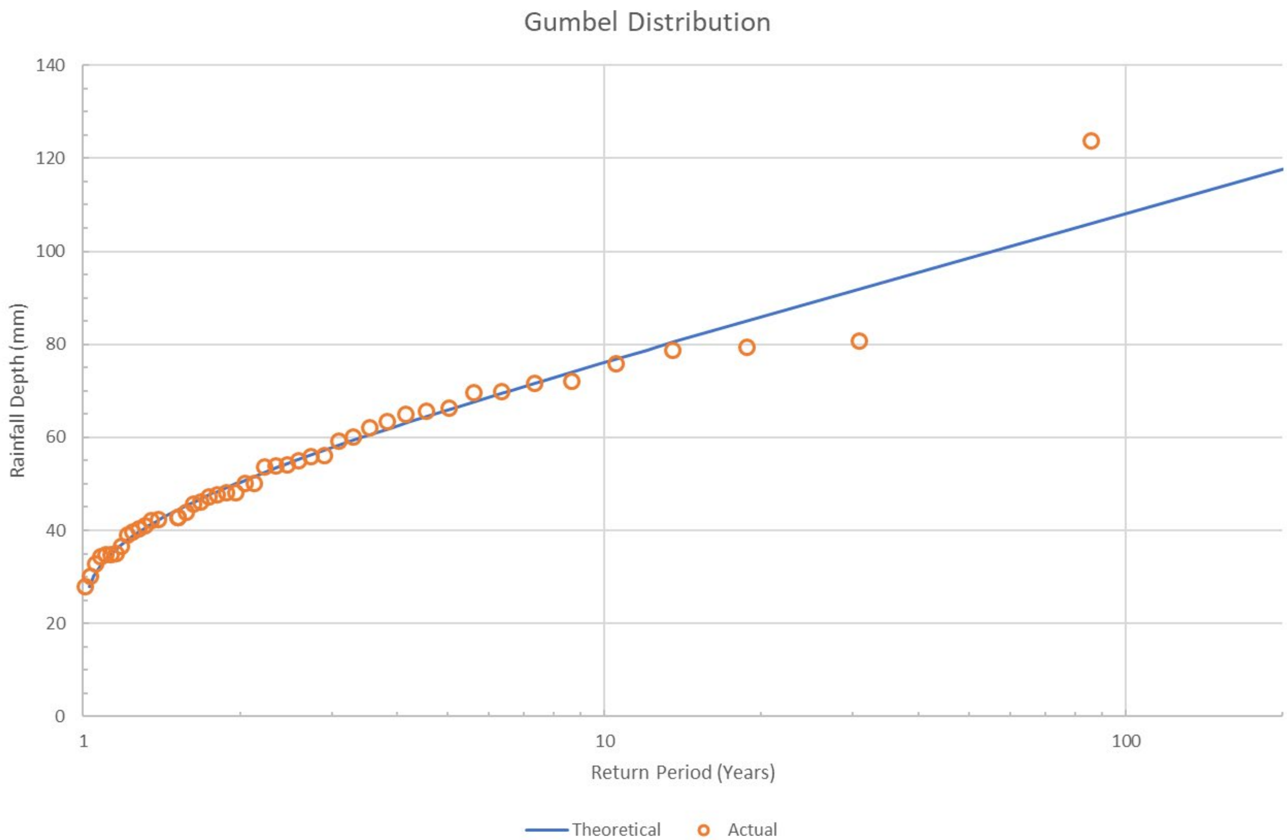
| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| 1986 | 65.6 | 11 | 0.2195 | 0.7805 | 4.6 | 1.4805 | 0.7965 | 4.9 |
| 2017 | 66.3 | 10 | 0.1987 | 0.8013 | 5.0 | 1.5319 | 0.8056 | 5.1 |
| 1967 | 69.6 | 9 | 0.1779 | 0.8221 | 5.6 | 1.7739 | 0.8439 | 6.4 |
| 2006 | 69.9 | 8 | 0.1571 | 0.8429 | 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 G:
## General Frequency Analysis Output - CFA

## General Frequency Analysis – Cold Creek

WSC STATION NO=02HK007
WSC STATION NAME=Cold Creek at Orland

TOTAL TIME SPAN, YT= 35 YRS.  FLOW THRESHOLD = 20.000
OBSERVED PEAKS, N= 36  HISTORIC PEAKS ABOVE THRESHOLD, NHA= 0

OBSERVED PEAKS ABOVE THRESHOLD, NA= 20
OBSERVED PEAKS BELOW THRESHOLD, NB= 16
MISSING PEAKS BELOW THRESHOLD, NC= -1

| MONTH (1) | YEAR (2) | FLOOD (3) | DESCENDING ORDER (4) | RANK M (5) | RANK ADJ. (6) | CUM. PROB. (7) | RET.PERIOD YEARS (8) |
|---|---|---|---|---|---|---|---|
| 4 | 1982 | 20.200 | 40.500 | 1 | 1.00 | 1.70 | 58.67 |
| 2 | 1983 | 24.000 | 33.900 | 2 | 2.00 | 4.55 | 22.00 |
| 4 | 1984 | 19.500 | 29.400 | 3 | 3.00 | 7.39 | 13.54 |
| 3 | 1985 | 19.600 | 29.300 | 4 | 4.00 | 10.23 | 9.78 |
| 4 | 1987 | 17.900 | 29.100 | 5 | 5.00 | 13.07 | 7.65 |
| 3 | 1988 | 28.700 | 28.700 | 6 | 6.00 | 15.91 | 6.29 |

Press <RETURN> to continue

WSC STATION NO=02HK007
WSC STATION NAME=Cold Creek at Orland

| MONTH (1) | YEAR (2) | FLOOD (3) | DESCENDING ORDER (4) | RANK M (5) | RANK ADJ. (6) | CUM. PROB. (7) | RET.PERIOD YEARS (8) |
|---|---|---|---|---|---|---|---|
| 3 | 1989 | 22.500 | 28.000 | 7 | 7.00 | 18.75 | 5.33 |
| 3 | 1990 | 40.500 | 26.600 | 8 | 8.00 | 21.59 | 4.63 |
| 3 | 1991 | 25.200 | 26.500 | 9 | 9.00 | 24.43 | 4.09 |
| 3 | 1992 | 29.400 | 25.400 | 10 | 10.00 | 27.27 | 3.67 |
| 1 | 1993 | 25.000 | 25.200 | 11 | 11.00 | 30.11 | 3.32 |
| 3 | 1994 | 14.800 | 25.000 | 12 | 12.00 | 32.95 | 3.03 |
| 1 | 1995 | 18.300 | 24.700 | 13 | 13.00 | 35.80 | 2.79 |
| 3 | 1997 | 18.700 | 24.300 | 14 | 14.00 | 38.64 | 2.59 |
| 3 | 1998 | 25.400 | 24.000 | 15 | 15.00 | 41.48 | 2.41 |
| 11 | 1999 | 9.550 | 23.600 | 16 | 16.00 | 44.32 | 2.26 |
| 2 | 2000 | 15.500 | 22.600 | 17 | 17.00 | 47.16 | 2.12 |
| 4 | 2001 | 13.000 | 22.500 | 18 | 18.00 | 50.00 | 2.00 |
| 5 | 2002 | 12.500 | 22.300 | 19 | 19.00 | 52.84 | 1.89 |

Press <RETURN> to continue

WSC STATION NO=02HK007
WSC STATION NAME=Cold Creek at Orland

| MONTH (1) | YEAR (2) | FLOOD (3) | DESCENDING ORDER (4) | RANK M (5) | RANK ADJ. (6) | CUM. PROB. (7) | RET.PERIOD YEARS (8) |
|---|---|---|---|---|---|---|---|
| 3 | 2003 | 29.300 | 20.200 | 20 | 20.00 | 55.68 | 1.80 |
| | | | | THRESHOLD | | | |
| 9 | 2004 | 33.900 | 19.600 | 21 | 20.94 | 58.35 | 1.71 |
| 4 | 2005 | 22.300 | 19.500 | 22 | 21.88 | 61.01 | 1.64 |
| 3 | 2007 | 10.600 | 19.000 | 23 | 22.81 | 63.67 | 1.57 |
| 12 | 2008 | 15.900 | 18.700 | 24 | 23.75 | 66.34 | 1.51 |
| 4 | 2009 | 26.500 | 18.300 | 25 | 24.69 | 69.00 | 1.45 |
| 3 | 2011 | 26.600 | 17.900 | 26 | 25.63 | 71.66 | 1.40 |
| 1 | 2012 | 10.800 | 17.500 | 27 | 26.56 | 74.33 | 1.35 |
| 4 | 2013 | 15.800 | 15.900 | 28 | 27.50 | 76.99 | 1.30 |
| 4 | 2014 | 28.000 | 15.800 | 29 | 28.44 | 79.65 | 1.26 |
| 10 | 2015 | 22.600 | 15.500 | 30 | 29.38 | 82.32 | 1.21 |
| 1 | 2016 | 17.500 | 14.800 | 31 | 30.31 | 84.98 | 1.18 |

Press <RETURN> to continue

WSC STATION NO=02HK007
WSC STATION NAME=Cold Creek at Orland

| MONTH (1) | YEAR (2) | FLOOD (3) | DESCENDING ORDER (4) | RANK M (5) | RANK ADJ. (6) | CUM. PROB. (7) | RET.PERIOD YEARS (8) |
|---|---|---|---|---|---|---|---|
| 5 | 2017 | 23.600 | 13.000 | 32 | 31.25 | 87.64 | 1.14 |
| 4 | 2018 | 19.000 | 12.500 | 33 | 32.19 | 90.31 | 1.11 |
| 3 | 2019 | 29.100 | 10.800 | 34 | 33.13 | 92.97 | 1.08 |
| 1 | 2020 | 24.700 | 10.600 | 35 | 34.06 | 95.63 | 1.05 |
| 9 | 2021 | 24.300 | 9.550 | 36 | 35.00 | 98.30 | 1.02 |

Press <RETURN> to continue

HISTORICAL FREQUENCY ANALYSIS - THREE-PARAMETER LOGNORMAL DISTRIBUTION
02HK007 Cold Creek at Orland

SAMPLE STATISTICS

| | MEAN | S.D. | C.V | C.S. | C.K. |
|---|---|---|---|---|---|
| X SERIES | 21.688 | 6.864 | .316 | .350 | 3.571 |
| LN X SERIES | 3.025 | .337 | .111 | -.499 | 3.170 |
| LN(X-A) SERIES | 4.031 | .121 | .030 | .013 | 3.121 |

X(MIN)= 9.550 TOTAL SAMPLE SIZE= 36
X(MAX)= 40.500 NO. OF LOW OUTLIERS= 0
LOWER OUTLIER LIMIT OF X= 8.458 NO. OF ZERO FLOWS= 0

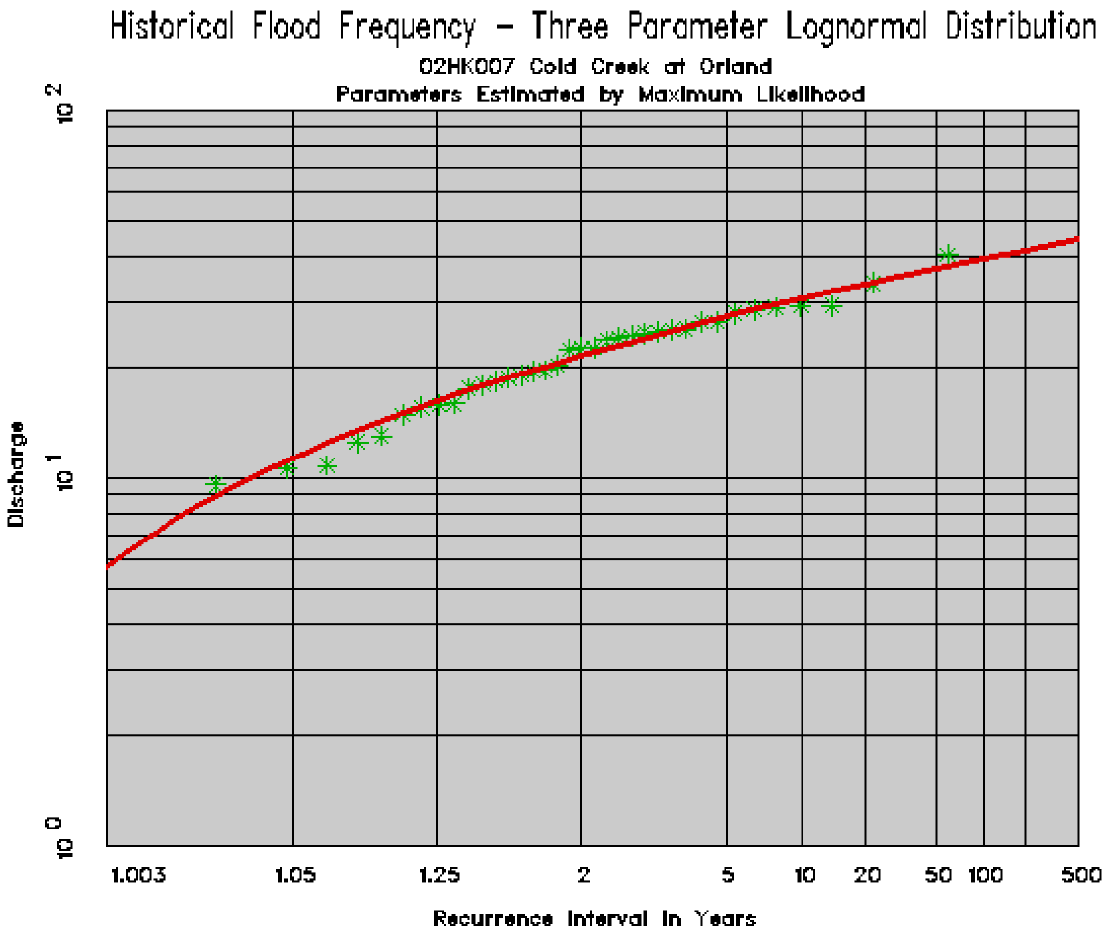
Press <RETURN> to continue , <CTRL> P to obtain hard copy_

SOLUTION OBTAINED VIA MAXIMUM LIKELIHOOD
3LN PARAMETERS: A= -35.007 M= 4.034 S= .119

FLOOD FREQUENCY REGIME

| RETURN PERIOD | EXCEEDANCE PROBABILITY | FLOOD |
|---|---|---|
| 1.003 | .997 | 5.74 |
| 1.050 | .952 | 11.3 |
| 1.250 | .800 | 16.1 |
| 2.000 | .500 | 21.5 |
| 5.000 | .200 | 27.4 |
| 10.000 | .100 | 30.7 |
| 20.000 | .050 | 33.6 |
| 50.000 | .020 | 37.1 |
| 100.000 | .010 | 39.4 |
| 200.000 | .005 | 41.7 |
| 500.000 | .002 | 44.5 |

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



Historical Flood Frequency - Three Parameter Lognormal Distribution
02HK007 Cold Creek at Orland
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

**General Frequency Analysis – Mayhew Creek**

WSC STATION NO.=02HK011
WSC STATION NAME=Mayhew Creek Near Trenton

HISTORIC INFORMATION: TOTAL TIME SPAN= 27
CENSORING THRESHOLD= 20.000
HISTORIC PEAKS ABOVE THE THRESHOLD= 0

| SEQ.NO. | YEAR | MON | FLOW |
|---|---|---|---|
| 1 | 1994 | 3 | 4.010 |
| 2 | 1995 | 11 | 7.750 |
| 3 | 1996 | 1 | 10.500 |
| 4 | 1997 | 3 | 4.880 |
| 5 | 1998 | 3 | 7.680 |
| 6 | 2000 | 2 | 6.650 |
| 7 | 2001 | 4 | 2.940 |
| 8 | 2002 | 5 | 6.190 |
| 9 | 2003 | 3 | 6.460 |
| 10 | 2004 | 9 | 16.100 |
| 11 | 2005 | 2 | 5.910 |
| 12 | 2006 | 12 | 8.140 |
| 13 | 2007 | 3 | 4.410 |

Press <RETURN> to continue

WSC STATION NO.=02HK011
WSC STATION NAME=Mayhew Creek Near Trenton

| SEQ.NO. | YEAR | MON | FLOW |
|---|---|---|---|
| 14 | 2008 | 4 | 6.620 |
| 15 | 2009 | 4 | 10.300 |
| 16 | 2011 | 3 | 9.890 |
| 17 | 2012 | 3 | 2.530 |
| 18 | 2013 | 4 | 7.140 |
| 19 | 2014 | 4 | 12.200 |
| 20 | 2015 | 10 | 5.030 |
| 21 | 2016 | 1 | 5.690 |
| 22 | 2017 | 5 | 10.100 |
| 23 | 2018 | 4 | 5.840 |
| 24 | 2019 | 3 | 9.730 |
| 25 | 2020 | 1 | 9.320 |
| 26 | 2021 | 9 | 5.440 |
| 27 | 2022 | 3 | 4.780 |

Press <RETURN> to continue

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

WSC STATION NO.=02HK011
WSC STATION NAME=Mayhew Creek Near Trenton
DRAINAGE AREA=      33.00
HISTORIC INFORMATION: TOTAL TIME SPAN=    27
                       CENSORING THRESHOLD=     20.000
                       HISTORIC PEAKS ABOVE THE THRESHOLD=     0
NUMBER OF OBSERVATIONS=    27

         X series    lnX series
MEAN        7.268       1.9006
S.D.        3.022        .4224
C.V.         .4159       .2223
C.S.         .9516      -.2719
C.K.        4.7588      3.6070

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 = 02HK011
WSC STATION NAME = Mayhew Creek Near Trenton

HISTORIC INFORMATION:
TOTAL TIME SPAN =     27
                      CENSORING THRESHOLD =      20.000
                      HISTORIC PEAKS ABOVE THE THRESHOLD =      0

NUMBER OF OBSERVATIONS =     27

C.S. of lnX series =      -.2719
LOWER OUTLIER LIMIT of X =       2.309
NOTE:  0 LOW OUTLIER(S) DETECTED.

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

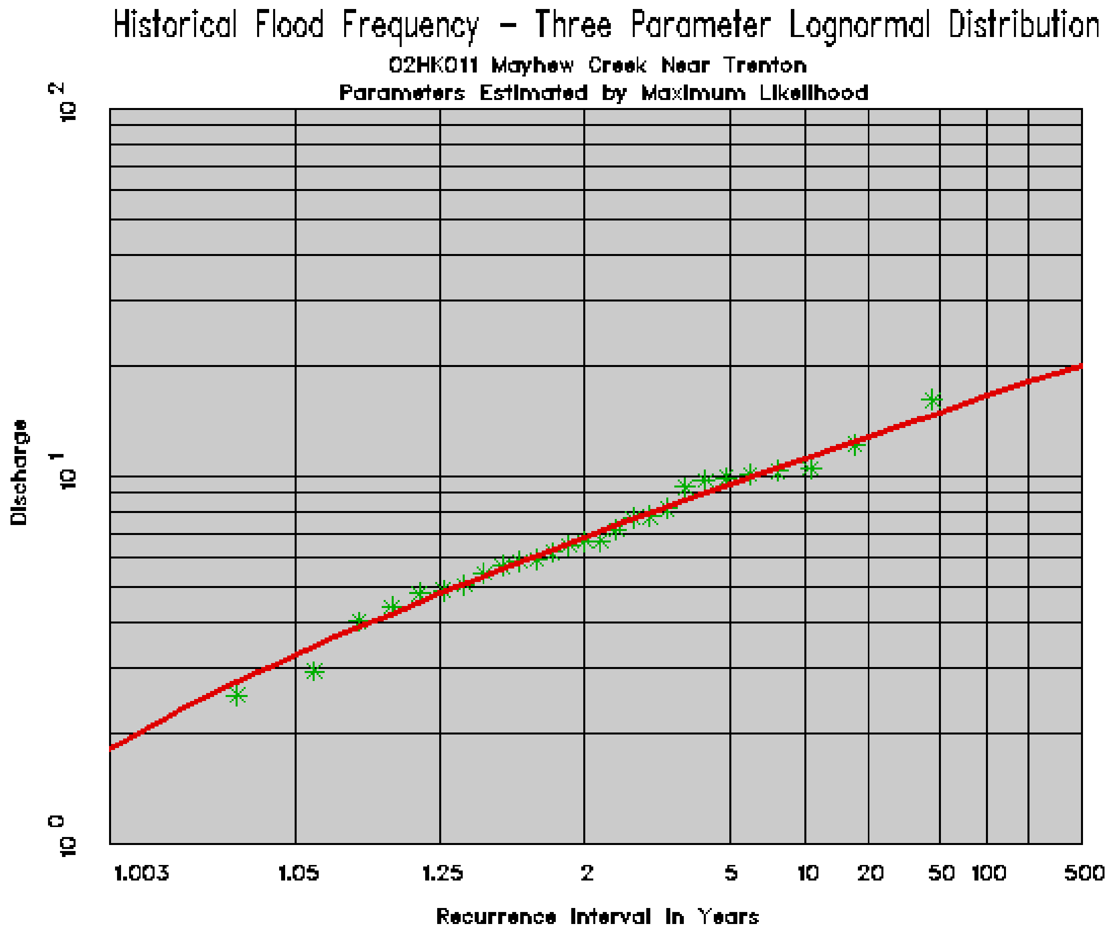
SOLUTION OBTAINED VIA MAXIMUM LIKELIHOOD

3LN PARAMETERS: A= -1.579 M= 2.126 S= .329

FLOOD FREQUENCY REGIME

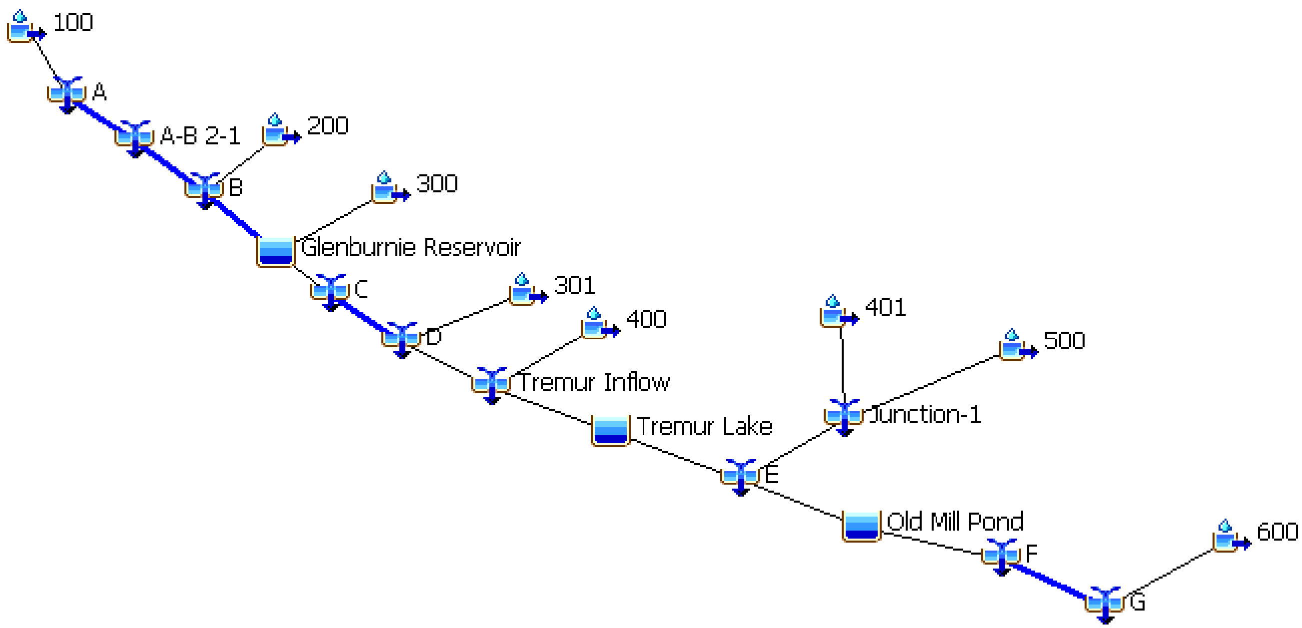
| RETURN PERIOD | EXCEEDANCE PROBABILITY | FLOOD |
|---|---|---|
| 1.003 | .997 | 1.81 |
| 1.050 | .952 | 3.26 |
| 1.250 | .800 | 4.77 |
| 2.000 | .500 | 6.80 |
| 5.000 | .200 | 9.48 |
| 10.000 | .100 | 11.2 |
| 20.000 | .050 | 12.8 |
| 50.000 | .020 | 14.9 |
| 100.000 | .010 | 16.5 |
| 200.000 | .005 | 18.0 |
| 500.000 | .002 | 20.0 |

Press <RETURN> to continue , <CTRL> P to obtain hard copy

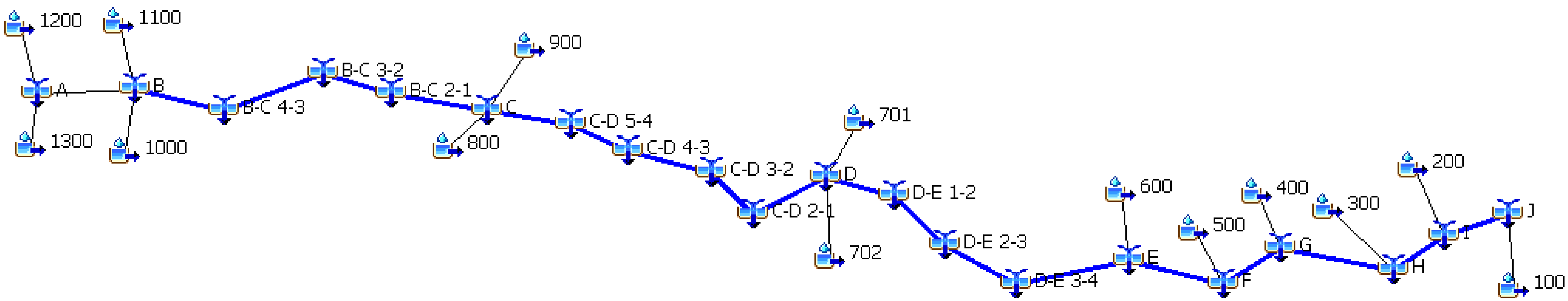

Historical Flood Frequency - Three Parameter Lognormal Distribution
02HK011 Mayhew Creek Near Trenton
Parameters Estimated by Maximum Likelihood
Discharge
10 2
10 1
10 0
1.003
1.05
1.25
2
5
10
20
50
100
500
Recurrence Interval in Years

# Appendix H:
## HEC-HMS Schematics

## Mayhew Creek



## Cold Creek


1200
1100
900
A
B
B-C 3-2
B-C 2-1
C
B-C 4-3
1300
1000
800
C-D 5-4
C-D 4-3
701
C-D 3-2
D
D-E 1-2
C-D 2-1
702
D-E 2-3
600
400
200
300
500
J
I
G
E
H
F
D-E 3-4
100

# Appendix I:
## LTC Memo – September 2021 Storm Event

JEWELL
ENGINEERING

## 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! |

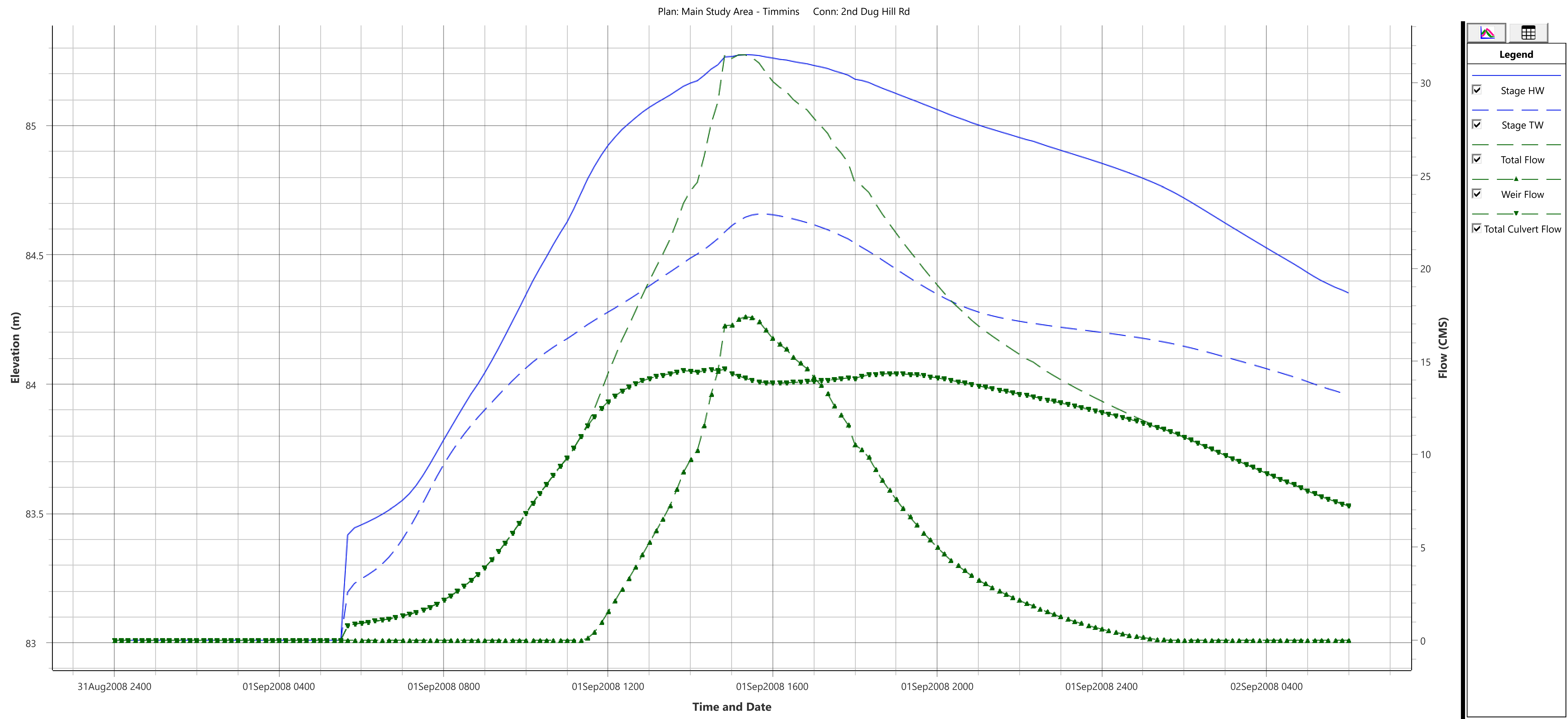
| Precipitation (mm) | This 24 hrs | last 24 hrs | last 48 hrs | Total for 48 hours |
|---|---|---|---|---|
| Trenton | | 41 | 44.2 | 85.2 |
| Butler | | 101.1 | 121.9 | 223 |
| Mill | | 41.8 | 34.6 | 76.4 |
| Rawdon | | 0 | 0 | 0 |
| Cold | | 52.6 | 58.8 | 111.4 |
| Trout | | 71 | 48.8 | 119.8 |
| Squires | | 37 | 39.2 | 76.2 |
| Salt | | 65.6 | 40.8 | 106.4 |
| Shelter Valley | | 0 | 0 | 0 |

Janet

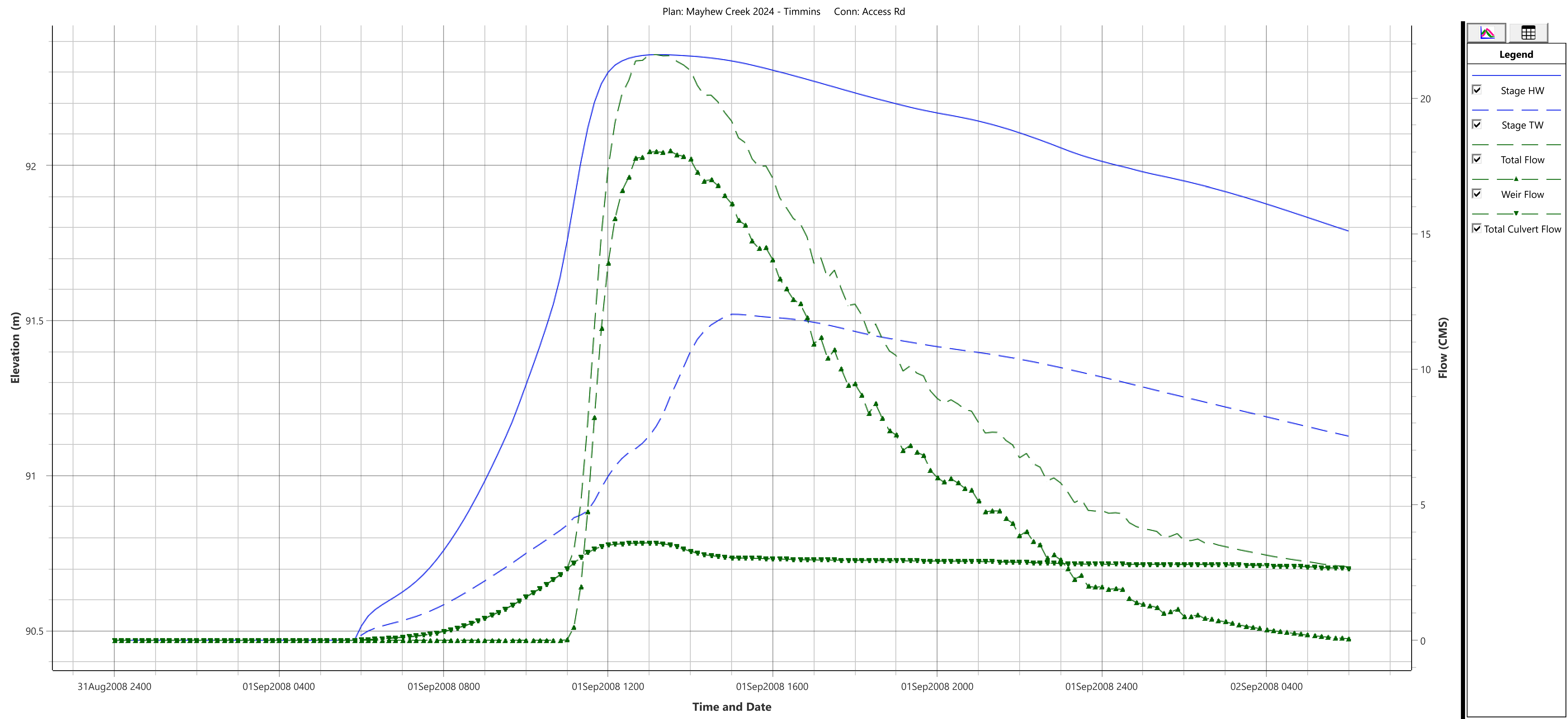
Janet Noyes, P.Eng.
Manager, Development Services & Water Resources
Lower Trent Conservation
613.394.3915 x211
janet.noyes@ltc.on.ca

**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.

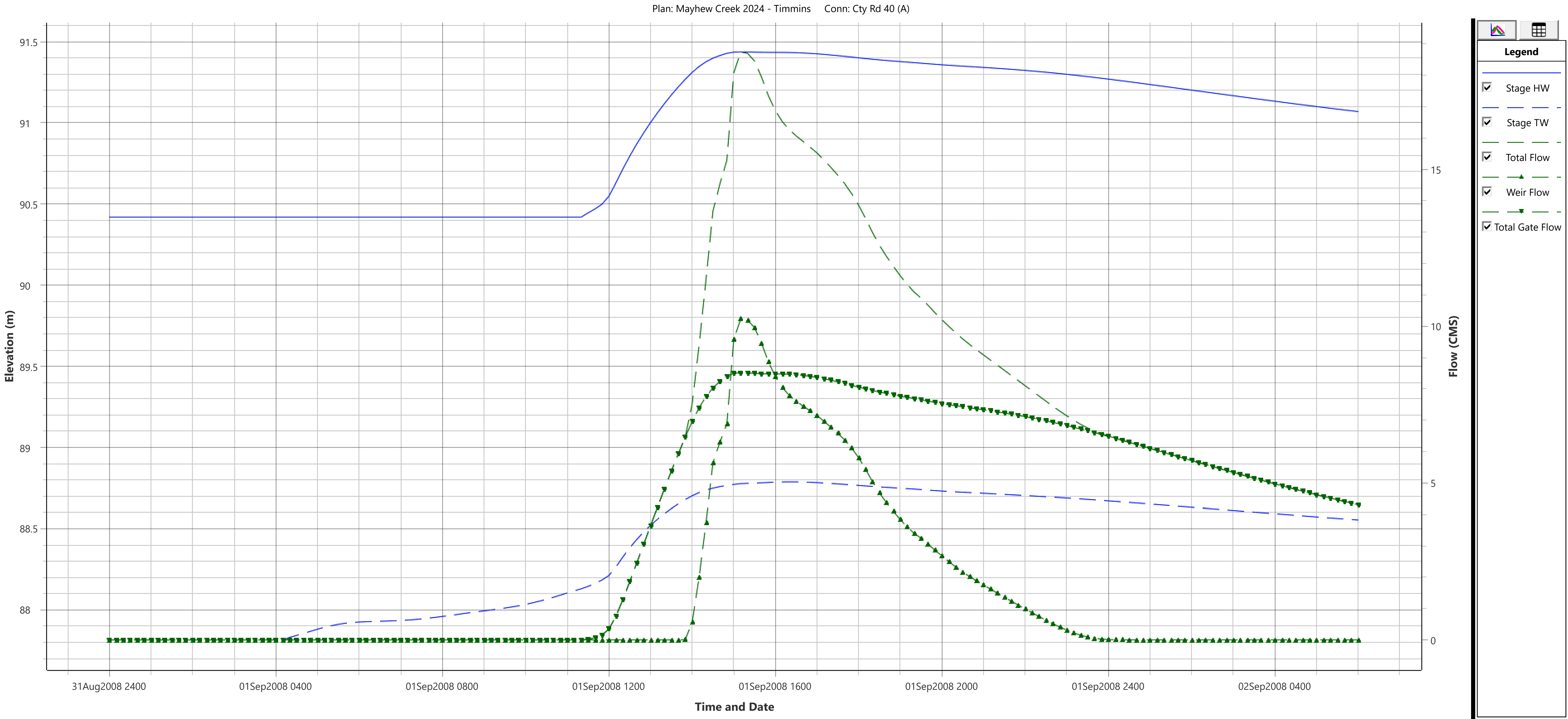
# Appendix J-1:
## Mayhew Creek - Bridge/Culvert Crossing Stage and Flow Hydrographs



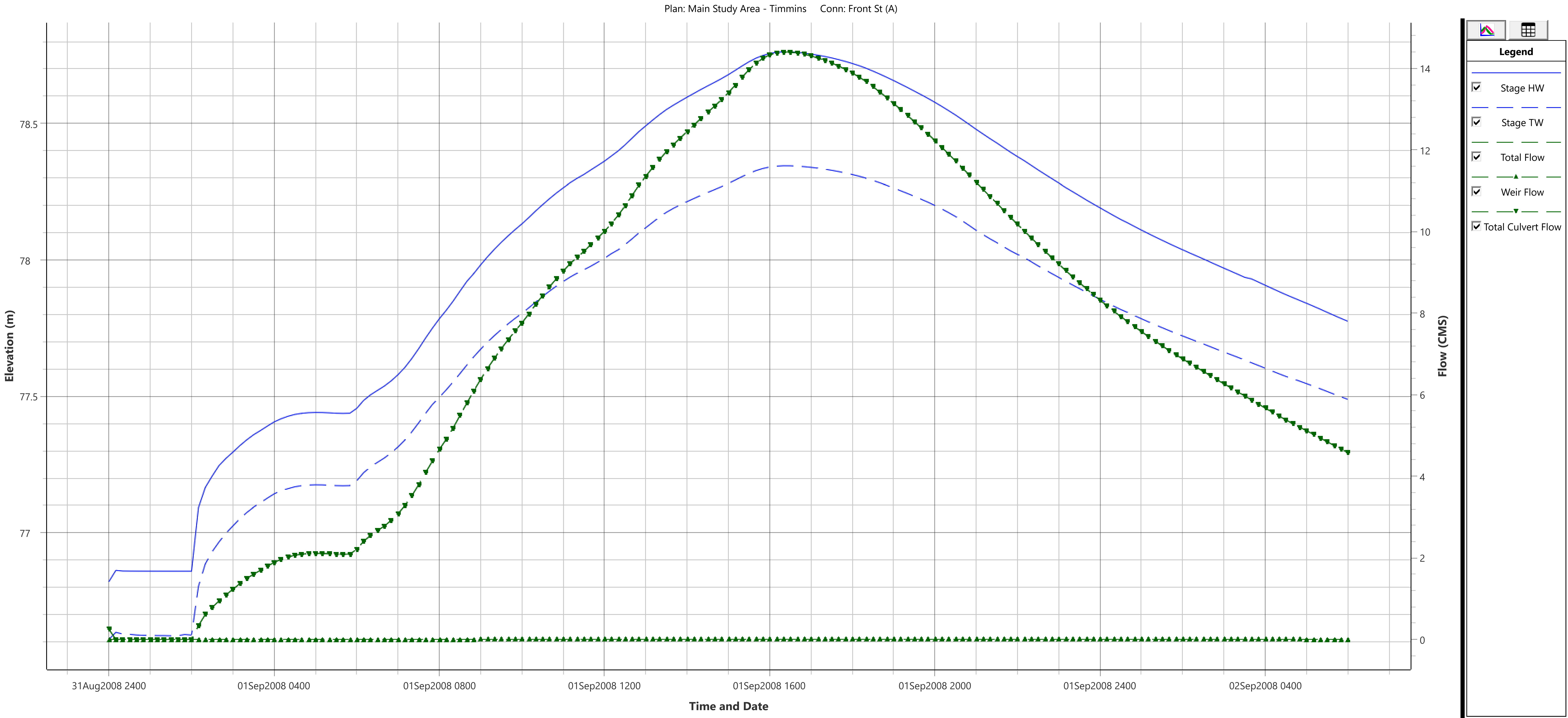
Plan: Main Study Area - Timmins Conn: 2nd Dug Hill Rd
Elevation (m)
85
84.5
84
83.5
83
Flow (CMS)
30
25
20
15
10
5
0
31Aug2008 2400
01Sep2008 0400
01Sep2008 0800
01Sep2008 1200
01Sep2008 1600
01Sep2008 2000
01Sep2008 2400
02Sep2008 0400
Time and Date
Legend
Stage HW
Stage TW
Total Flow
Weir Flow
Total Culvert Flow



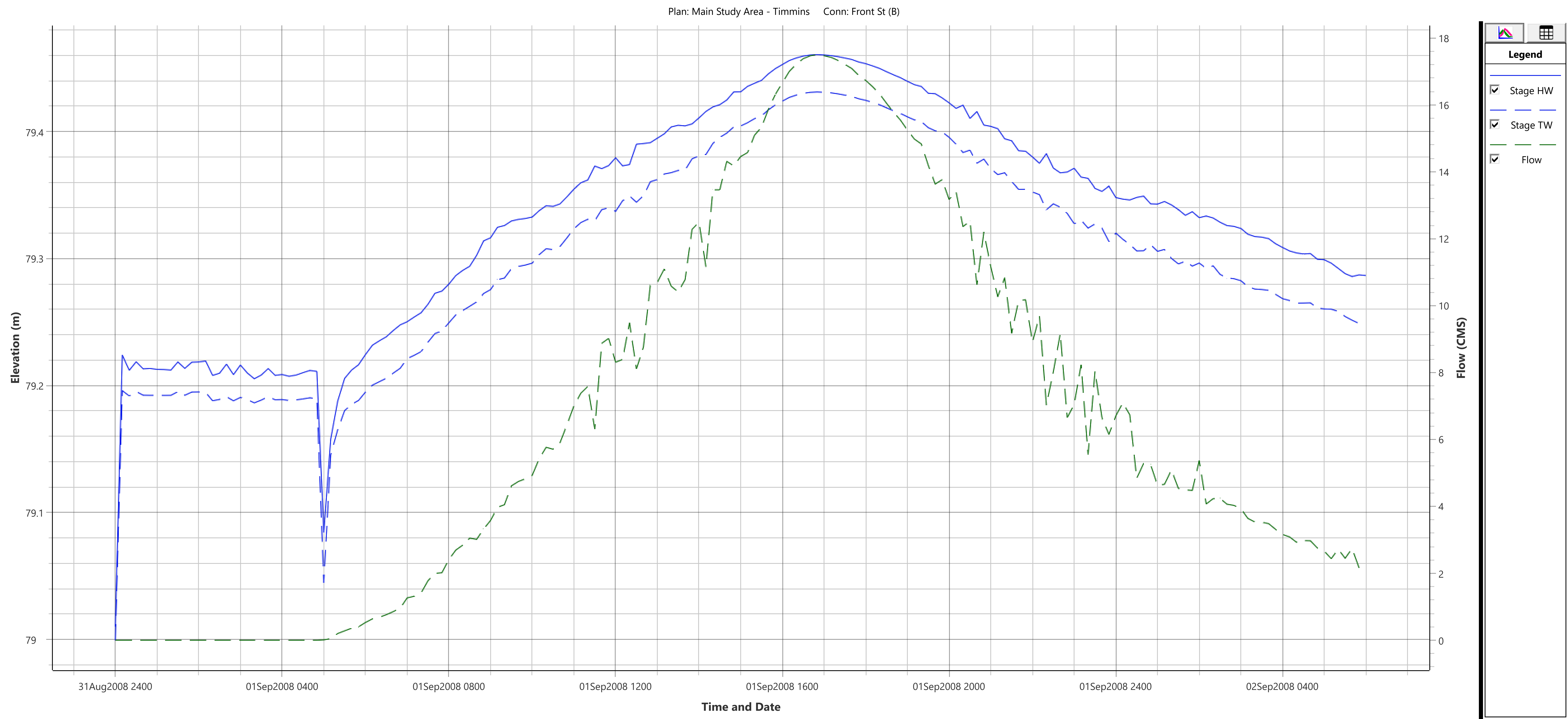
Plan: Mayhew Creek 2024 - Timmins Conn: Access Rd
Elevation (m)
92
91.5
91
90.5
Flow (CMS)
20
15
10
5
0
31Aug2008 2400
01Sep2008 0400
01Sep2008 0800
01Sep2008 1200
01Sep2008 1600
01Sep2008 2000
01Sep2008 2400
02Sep2008 0400
Time and Date
Legend
Stage HW
Stage TW
Total Flow
Weir Flow
Total Culvert Flow

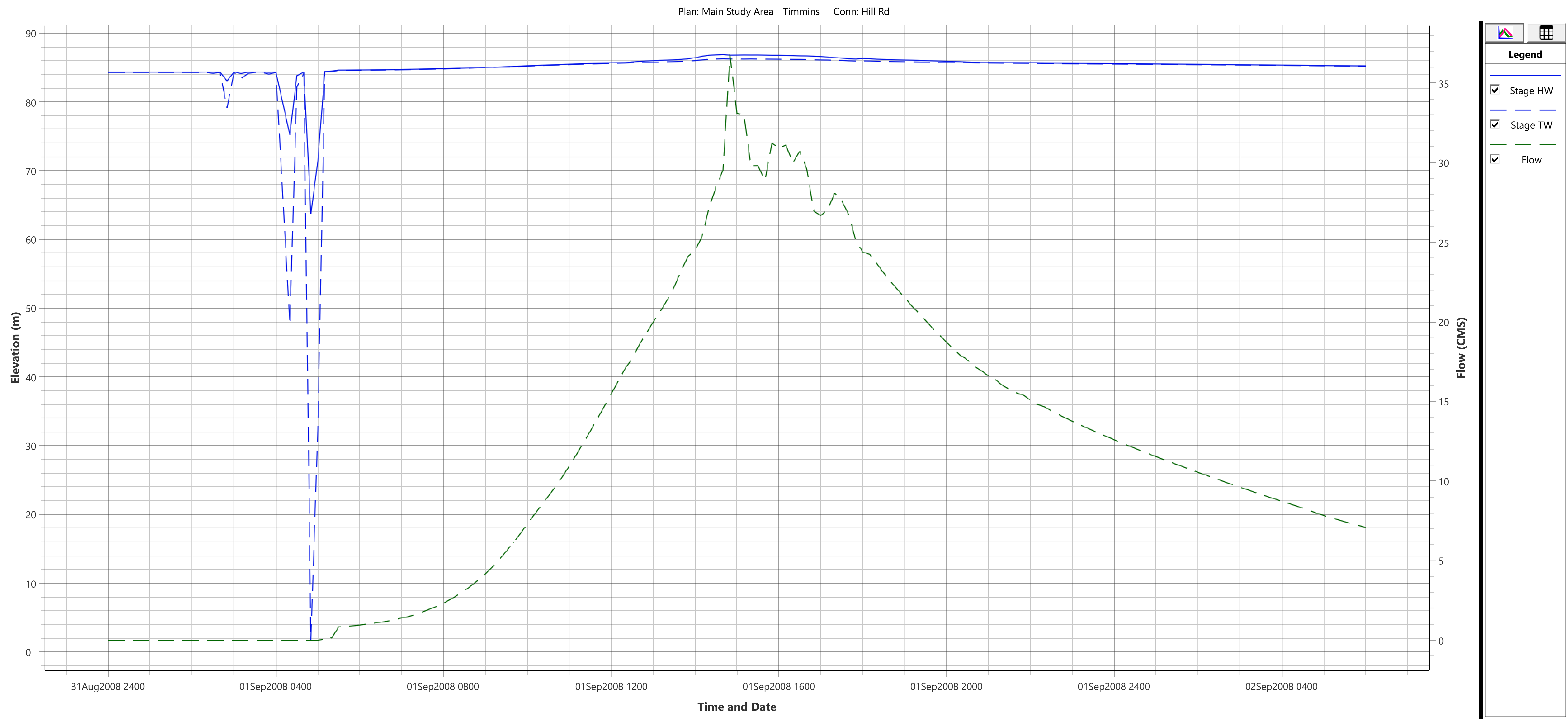


Plan: Mayhew Creek 2024 - Timmins Conn: Cty Rd 40 (A)
Elevation (m)
Flow (CMS)
Time and Date
91.5
91
90.5
90
89.5
89
88.5
88
15
10
5
0
31Aug2008 2400
01Sep2008 0400
01Sep2008 0800
01Sep2008 1200
01Sep2008 1600
01Sep2008 2000
01Sep2008 2400
02Sep2008 0400
Legend
Stage HW
Stage TW
Total Flow
Weir Flow
Total Gate Flow

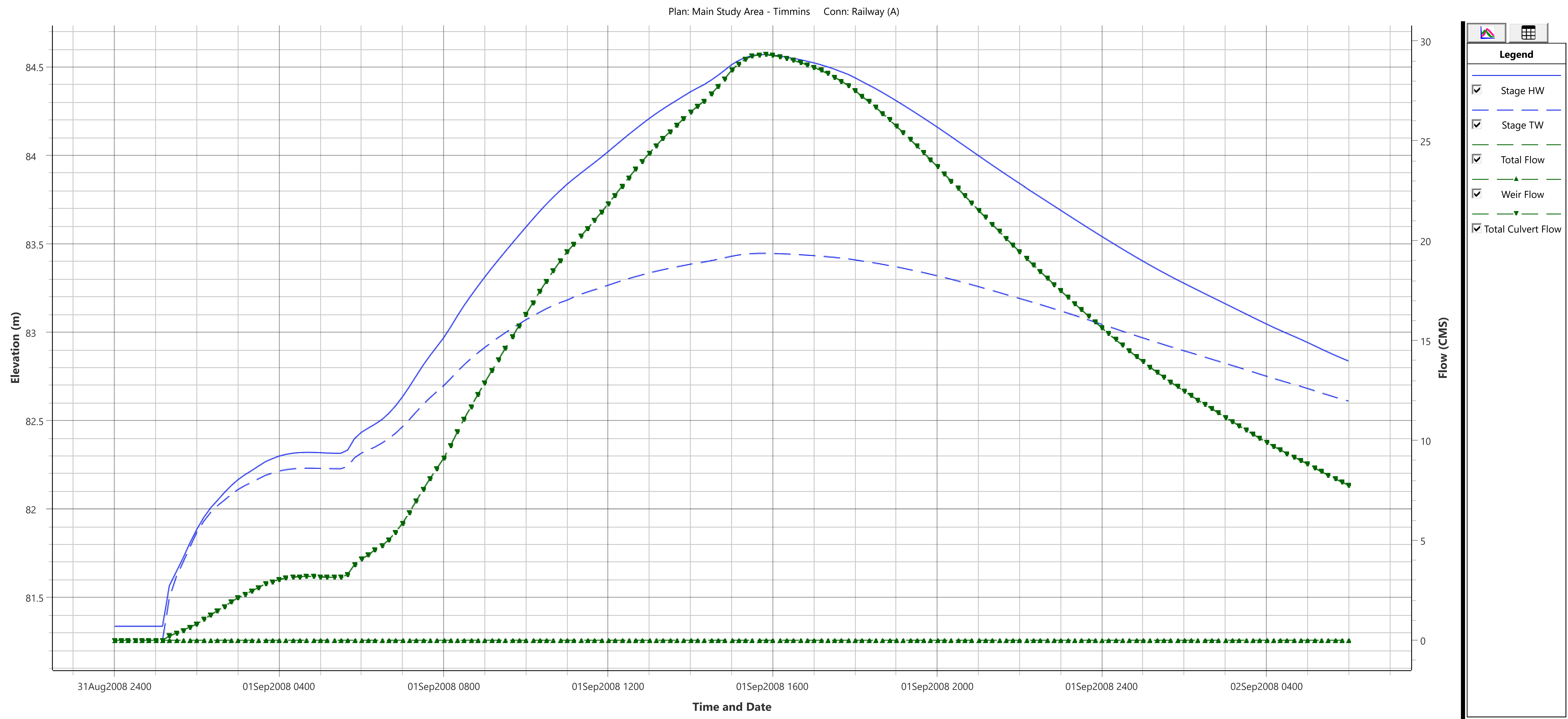


Plan: Main Study Area - Timmins Conn: Front St (A)
Elevation (m)
78.5
78
77.5
77
Flow (CMS)
14
12
10
8
6
4
2
0
31Aug2008 2400
01Sep2008 0400
01Sep2008 0800
01Sep2008 1200
01Sep2008 1600
01Sep2008 2000
01Sep2008 2400
02Sep2008 0400
Time and Date
Legend
Stage HW
Stage TW
Total Flow
Weir Flow
Total Culvert Flow

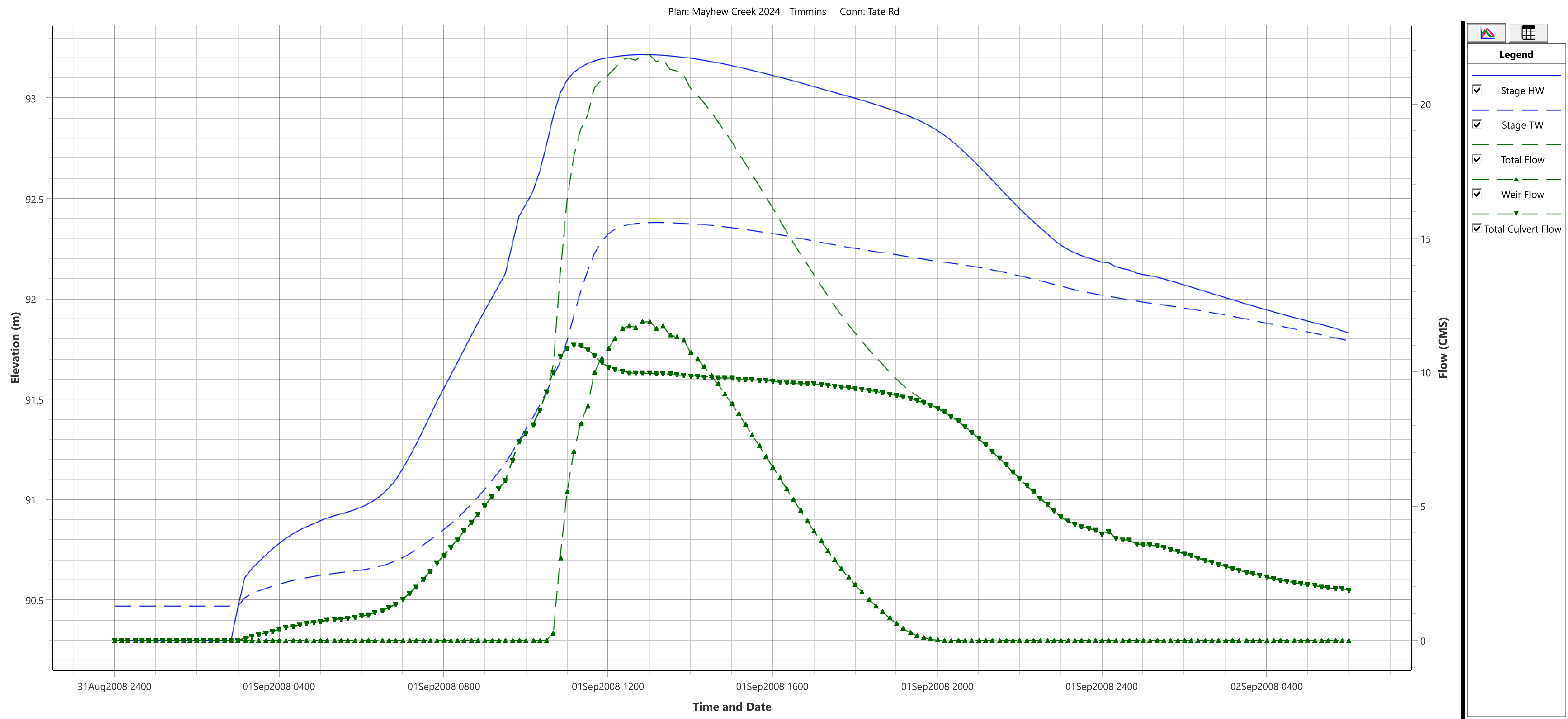

Plan: Main Study Area - Timmins Conn: Front St (B)
Legend
Stage HW
Stage TW
Flow
Elevation (m)
Flow (CMS)
Time and Date
79
79.1
79.2
79.3
79.4
0
2
4
6
8
10
12
14
16
18
31Aug2008 2400
01Sep2008 0400
01Sep2008 0800
01Sep2008 1200
01Sep2008 1600
01Sep2008 2000
01Sep2008 2400
02Sep2008 0400



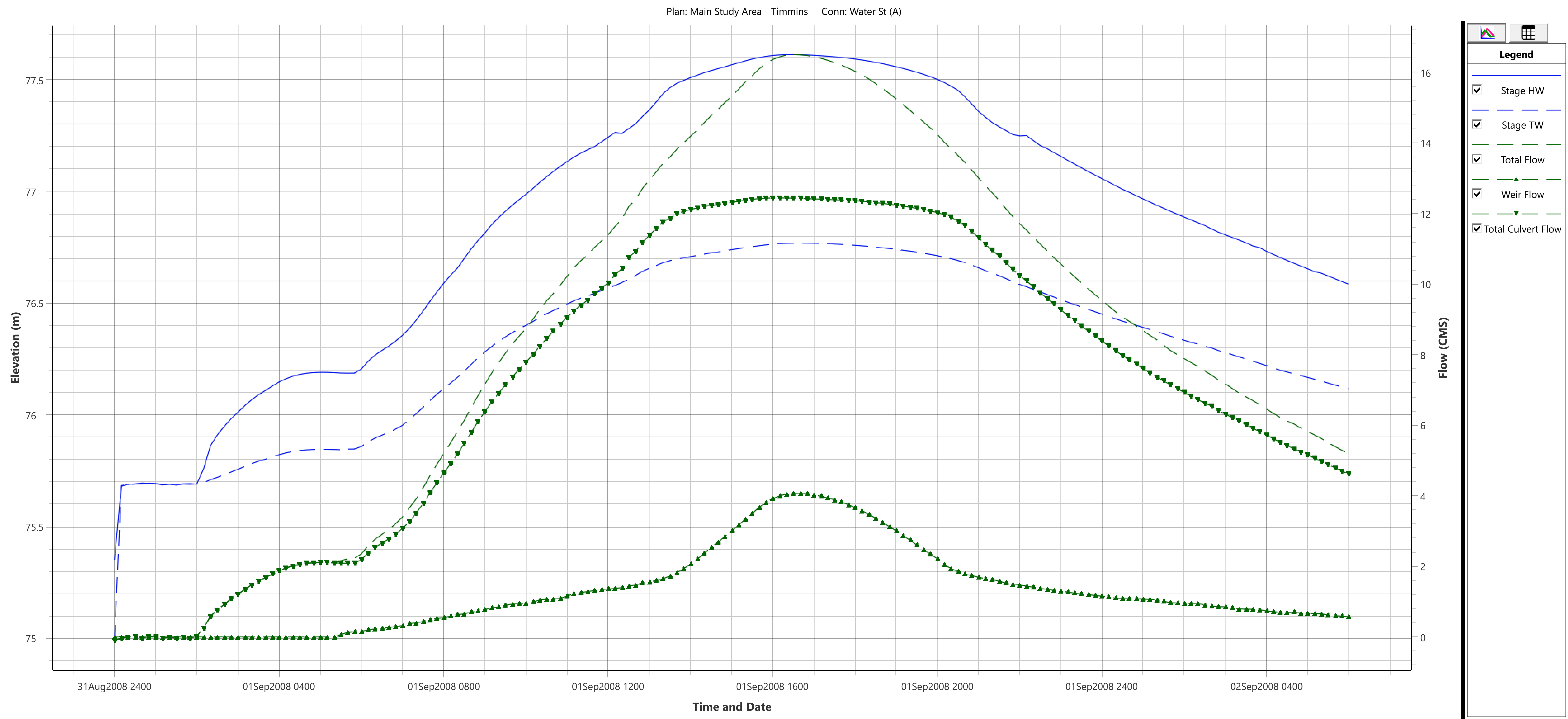
Plan: Main Study Area - Timmins Conn: Hill Rd
Elevation (m)
Flow (CMS)
Time and Date
31Aug2008 2400
01Sep2008 0400
01Sep2008 0800
01Sep2008 1200
01Sep2008 1600
01Sep2008 2000
01Sep2008 2400
02Sep2008 0400
0
10
20
30
40
50
60
70
80
90
0
5
10
15
20
25
30
35
Legend
Stage HW
Stage TW
Flow



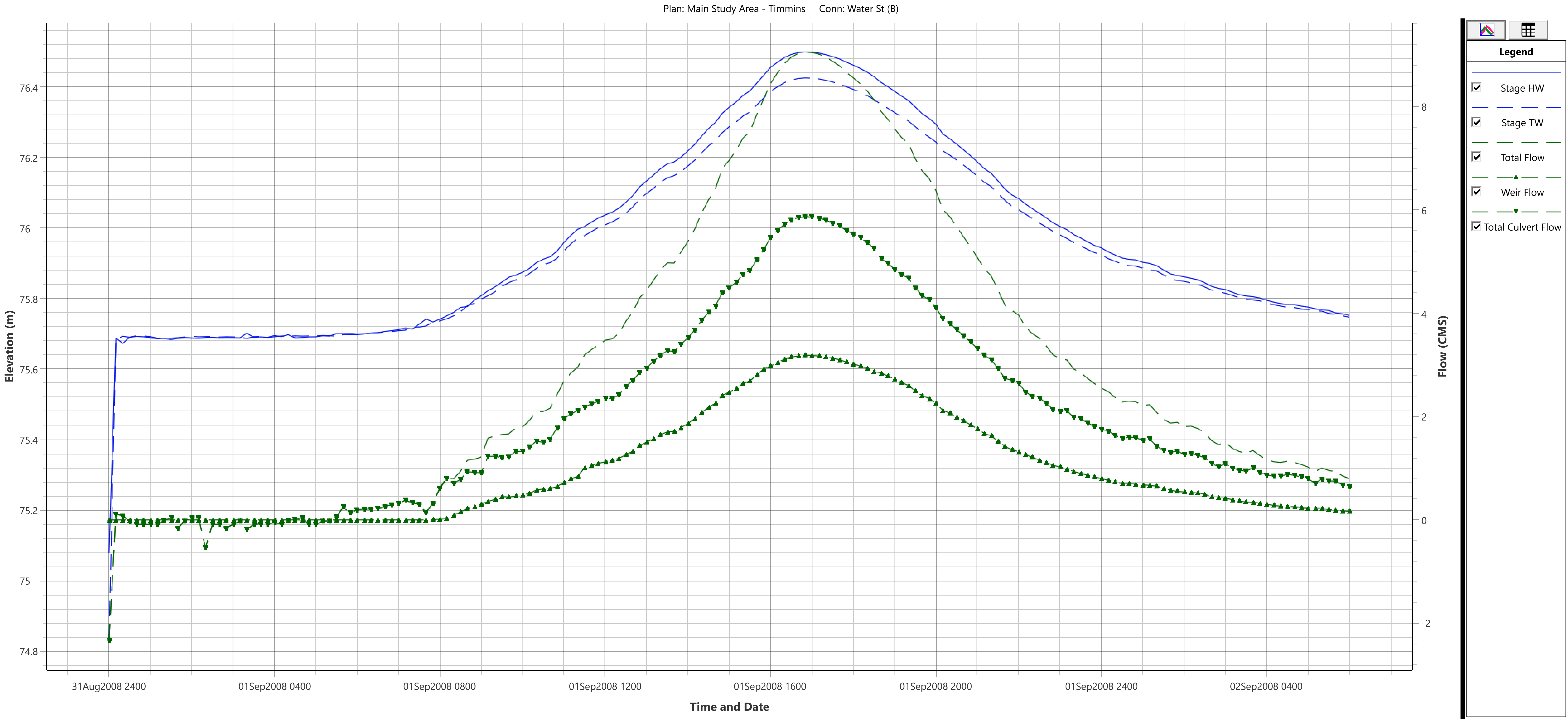
Plan: Main Study Area - Timmins Conn: Railway (A)
Elevation (m)
84.5
84
83.5
83
82.5
82
81.5
Flow (CMS)
30
25
20
15
10
5
0
31Aug2008 2400
01Sep2008 0400
01Sep2008 0800
01Sep2008 1200
01Sep2008 1600
01Sep2008 2000
01Sep2008 2400
02Sep2008 0400
Time and Date
Legend
Stage HW
Stage TW
Total Flow
Weir Flow
Total Culvert Flow



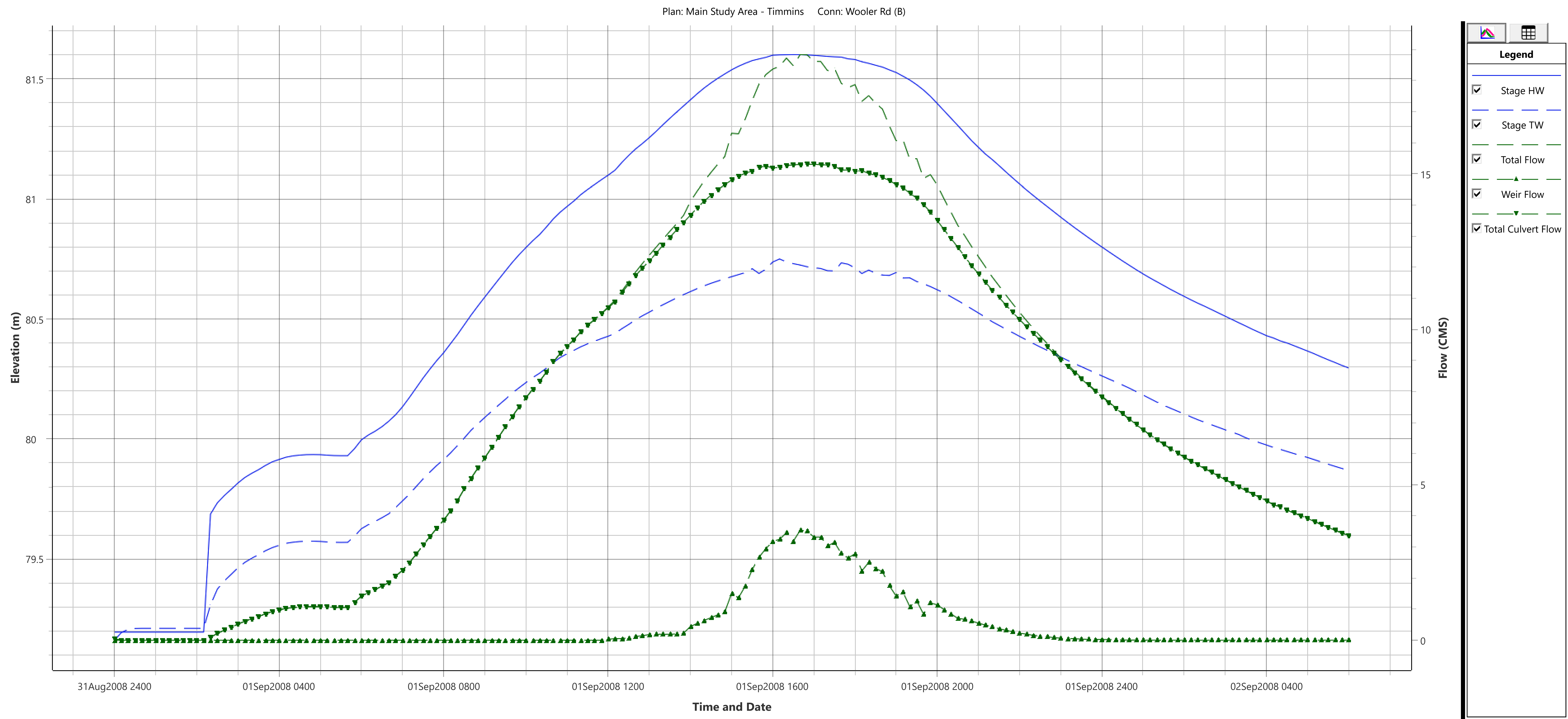
Plan: Mayhew Creek 2024 - Timmins Conn: Tate Rd
Elevation (m)
93
92.5
92
91.5
91
90.5
Flow (CMS)
20
15
10
5
0
31Aug2008 2400
01Sep2008 0400
01Sep2008 0800
01Sep2008 1200
01Sep2008 1600
01Sep2008 2000
01Sep2008 2400
02Sep2008 0400
Time and Date
Legend
Stage HW
Stage TW
Total Flow
Weir Flow
Total Culvert Flow



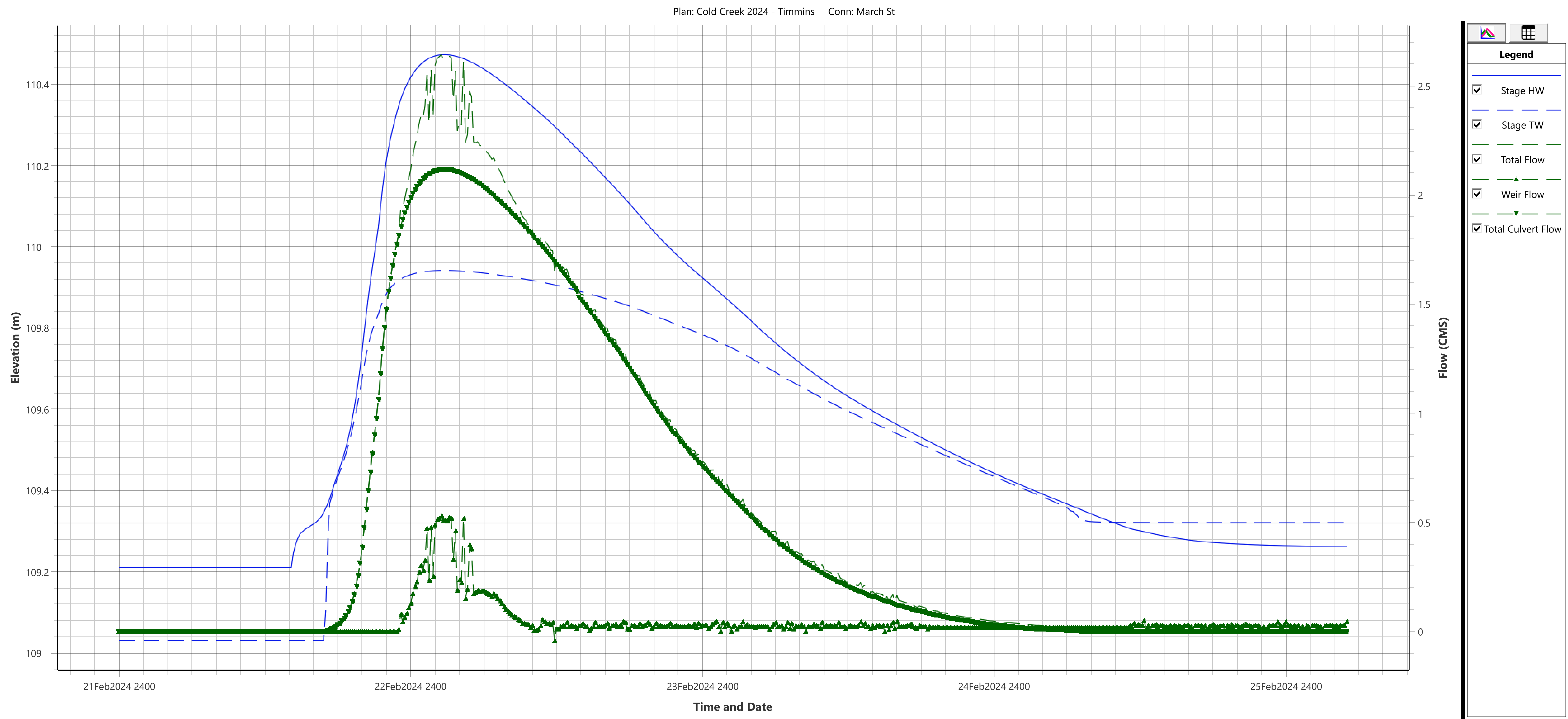
Plan: Main Study Area - Timmins Conn: Water St (A)
Elevation (m)
77.5
77
76.5
76
75.5
75
Flow (CMS)
16
14
12
10
8
6
4
2
0
31Aug2008 2400
01Sep2008 0400
01Sep2008 0800
01Sep2008 1200
01Sep2008 1600
01Sep2008 2000
01Sep2008 2400
02Sep2008 0400
Time and Date
Legend
Stage HW
Stage TW
Total Flow
Weir Flow
Total Culvert Flow



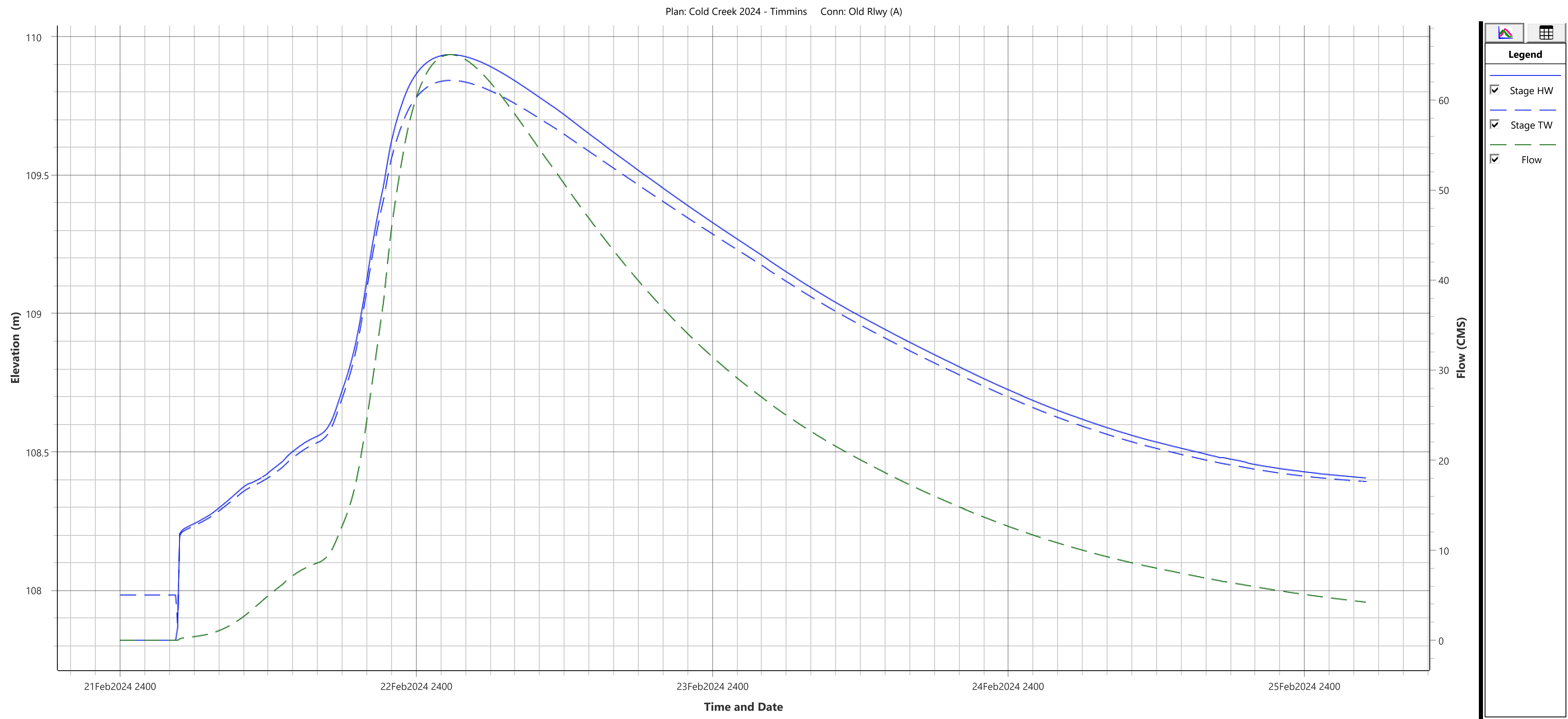
Plan: Main Study Area - Timmins Conn: Water St (B)
Legend
Stage HW
Stage TW
Total Flow
Weir Flow
Total Culvert Flow
Elevation (m)
Flow (CMS)
Time and Date
76.4
76.2
76
75.8
75.6
75.4
75.2
75
74.8
8
6
4
2
0
-2
31Aug2008 2400
01Sep2008 0400
01Sep2008 0800
01Sep2008 1200
01Sep2008 1600
01Sep2008 2000
01Sep2008 2400
02Sep2008 0400


Plan: Main Study Area - Timmins Conn: Wooler Rd (B)
Legend
Stage HW
Stage TW
Total Flow
Weir Flow
Total Culvert Flow
Elevation (m)
81.5
81
80.5
80
79.5
Flow (CMS)
15
10
5
0
31Aug2008 2400
01Sep2008 0400
01Sep2008 0800
01Sep2008 1200
01Sep2008 1600
01Sep2008 2000
01Sep2008 2400
02Sep2008 0400
Time and Date

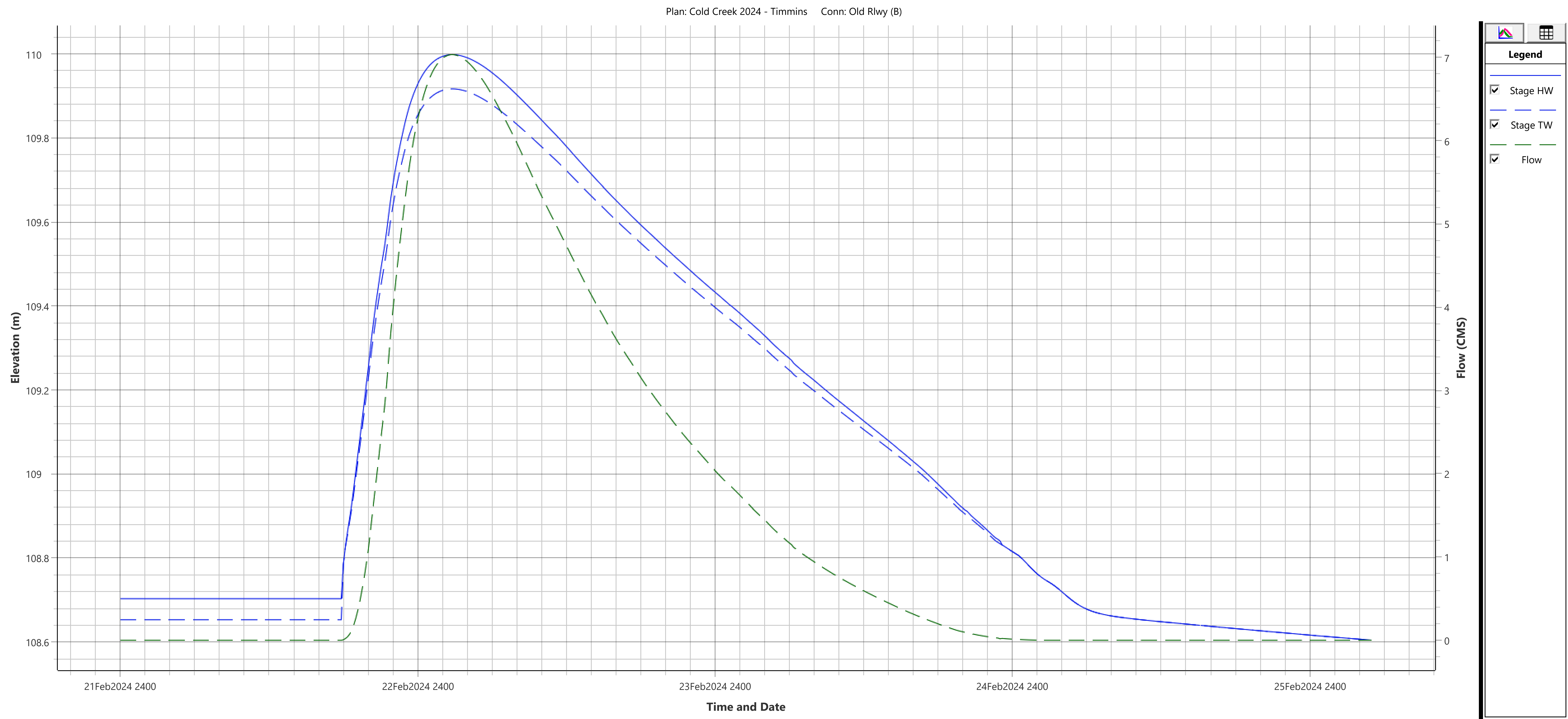
# Appendix J-2:
## Cold Creek - Culvert Crossing Stage and Flow Hydrographs

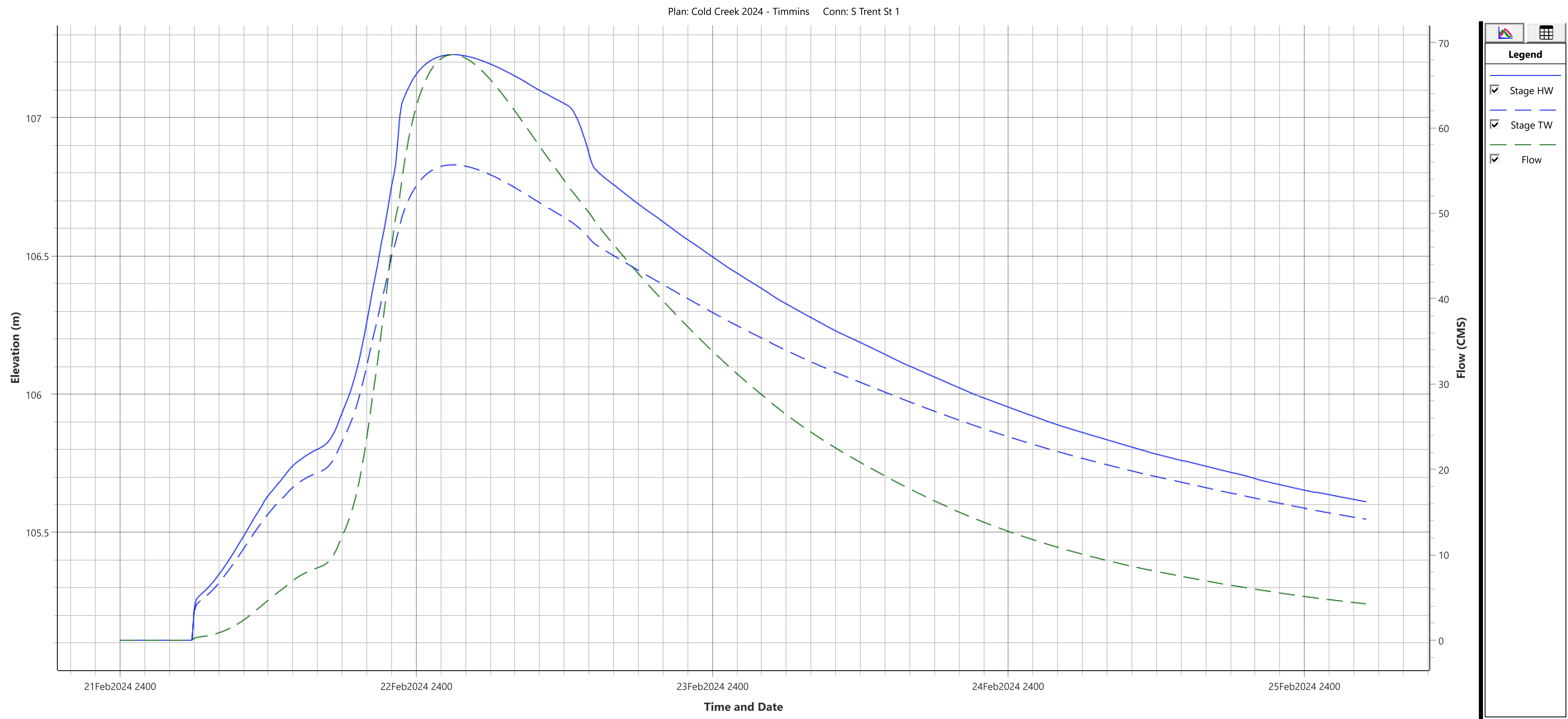


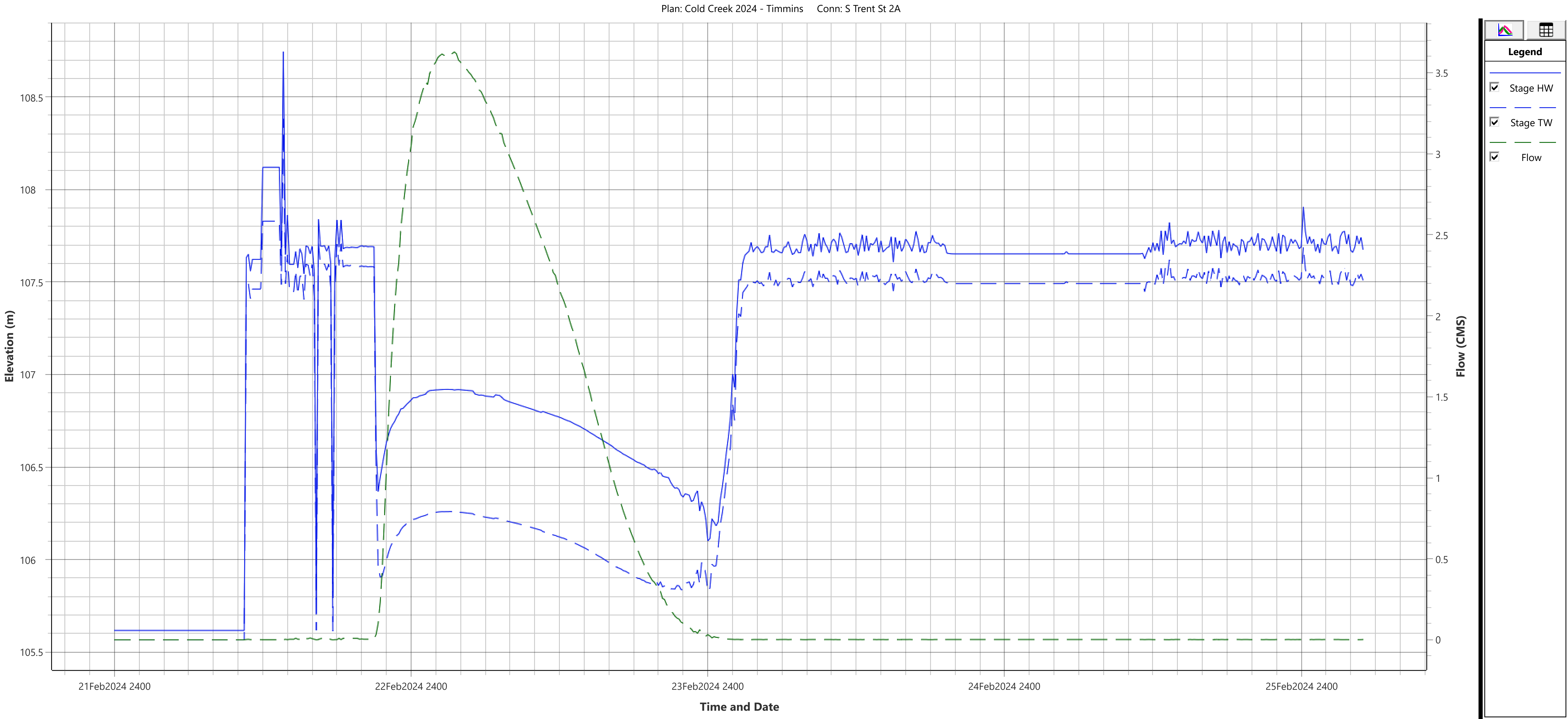
Plan: Cold Creek 2024 - Timmins     Conn: March St
Elevation (m)
110.4
110.2
110
109.8
109.6
109.4
109.2
109
Flow (CMS)
2.5
2
1.5
1
0.5
0
21Feb2024 2400
22Feb2024 2400
23Feb2024 2400
24Feb2024 2400
25Feb2024 2400
Time and Date
Legend
Stage HW
Stage TW
Total Flow
Weir Flow
Total Culvert Flow



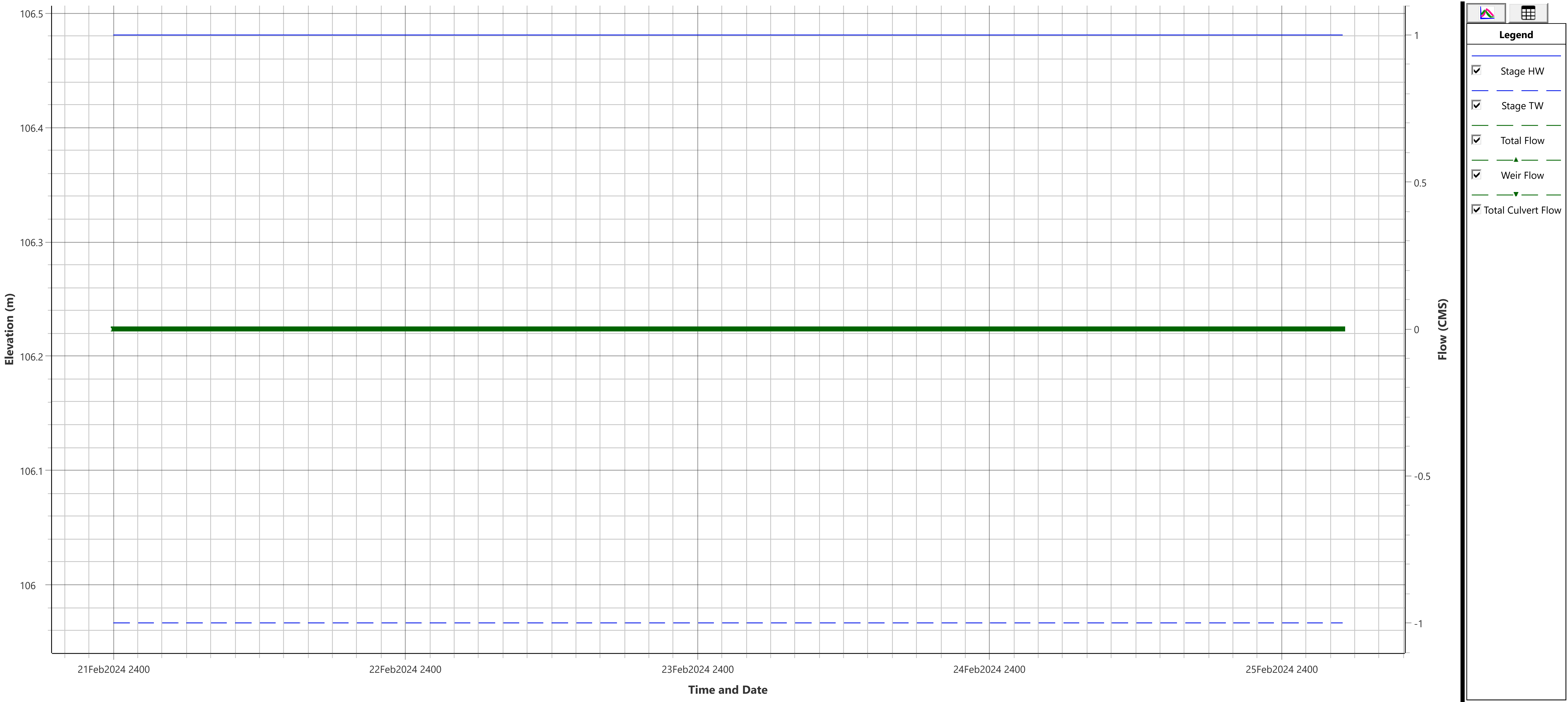
Plan: Cold Creek 2024 - Timmins Conn: Old Rlwy (A)
Elevation (m)
110
109.5
109
108.5
108
Flow (CMS)
60
50
40
30
20
10
0
21Feb2024 2400
22Feb2024 2400
23Feb2024 2400
24Feb2024 2400
25Feb2024 2400
Time and Date
Legend
Stage HW
Stage TW
Flow


Plan: Cold Creek 2024 - Timmins Conn: Old Rlwy (B)
Elevation (m)
110
109.8
109.6
109.4
109.2
109
108.8
108.6
Flow (CMS)
7
6
5
4
3
2
1
0
21Feb2024 2400
22Feb2024 2400
23Feb2024 2400
24Feb2024 2400
25Feb2024 2400
Time and Date
Legend
Stage HW
Stage TW
Flow


Plan: Cold Creek 2024 - Timmins    Conn: S Trent St 1
Elevation (m)
107
106.5
106
105.5
Flow (CMS)
70
60
50
40
30
20
10
0
21Feb2024 2400
22Feb2024 2400
23Feb2024 2400
24Feb2024 2400
25Feb2024 2400
Time and Date
Legend
Stage HW
Stage TW
Flow

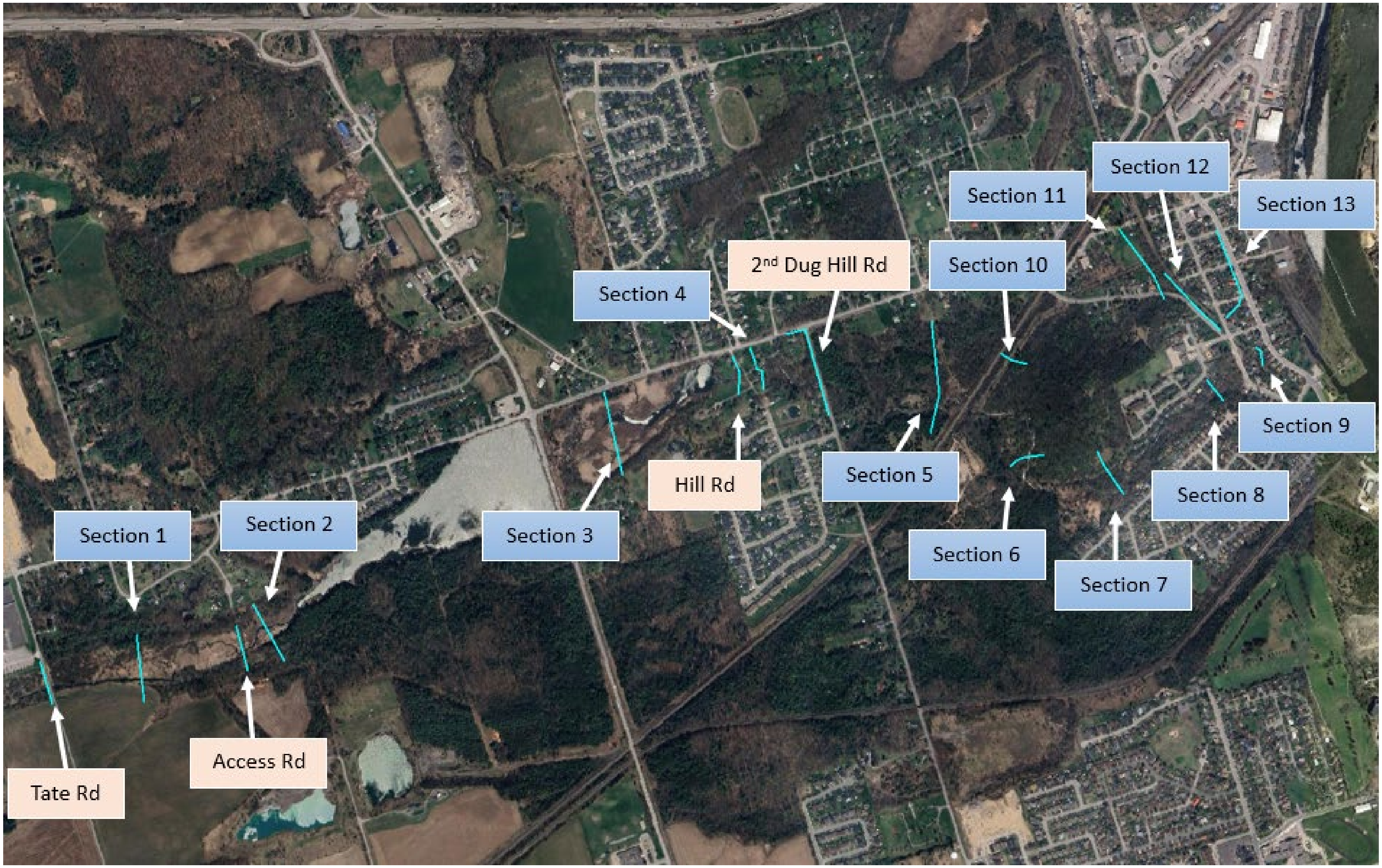

Plan: Cold Creek 2024 - Timmins Conn: S Trent St 2A
Elevation (m)
108.5
108
107.5
107
106.5
106
105.5
Flow (CMS)
3.5
3
2.5
2
1.5
1
0.5
0
21Feb2024 2400
22Feb2024 2400
23Feb2024 2400
24Feb2024 2400
25Feb2024 2400
Time and Date
Legend
Stage HW
Stage TW
Flow

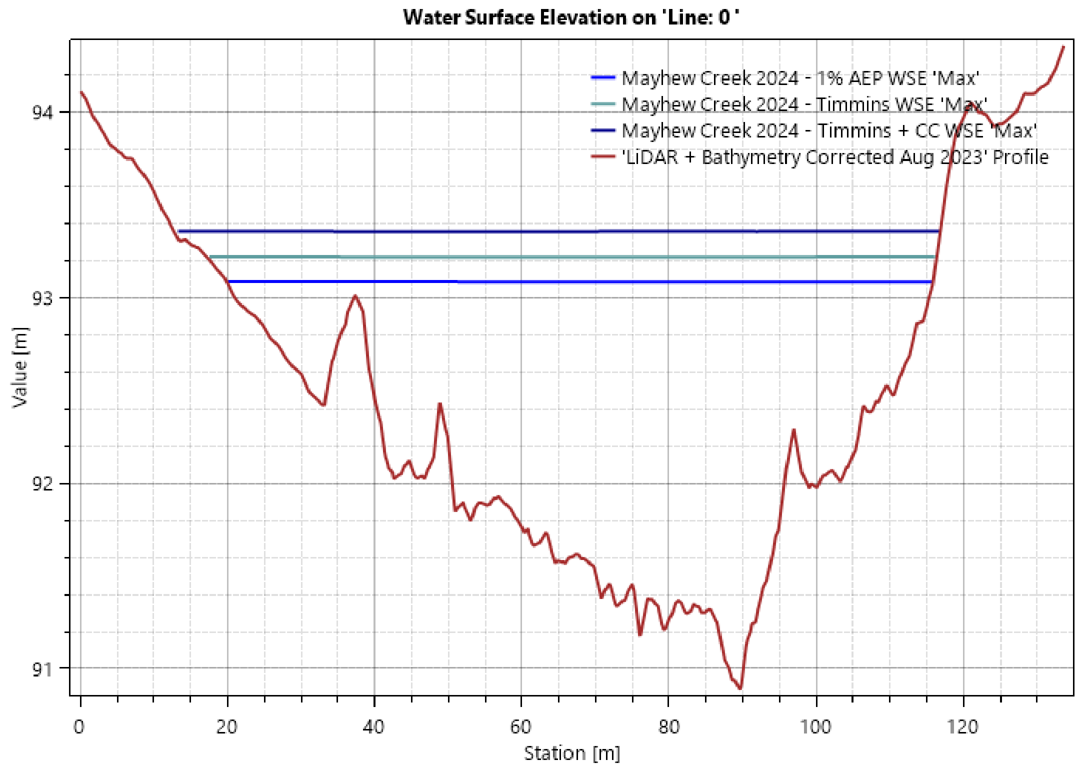
Plan: Cold Creek 2024 - Timmins     Conn: S Trent St 2B&C



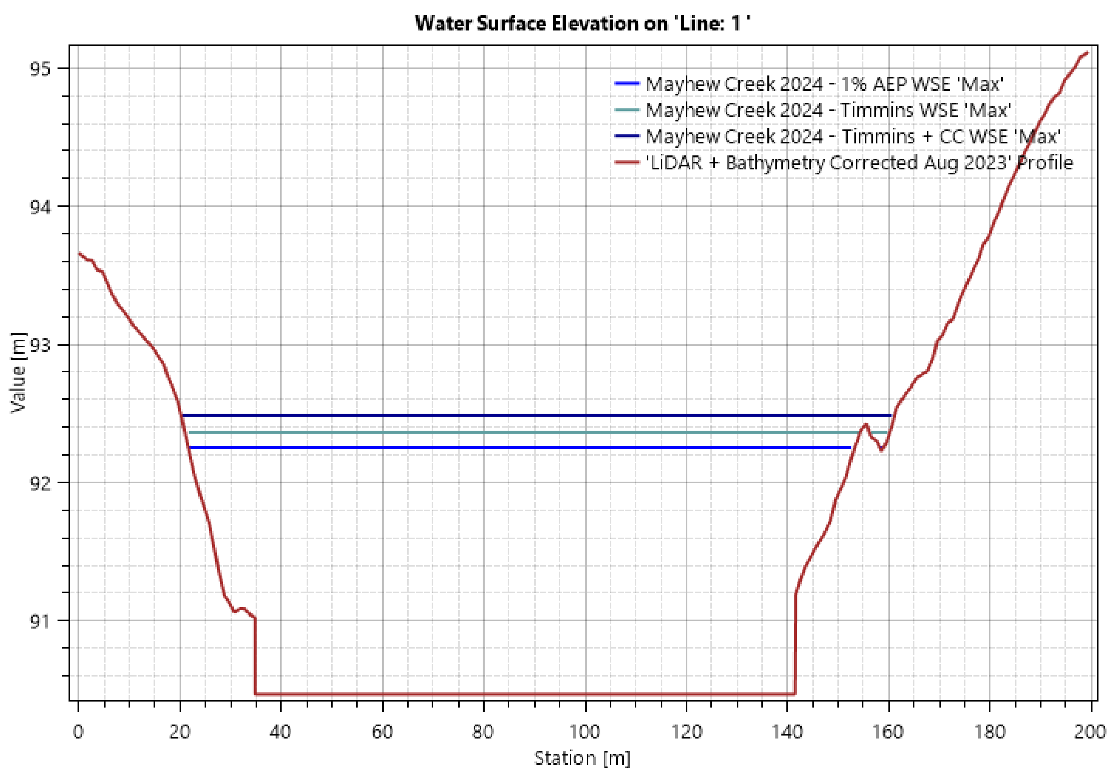
# Appendix K-1:
## Mayhew Creek Cross Section WSEL Plots

**Locations of Cross Section Profile Plots:**

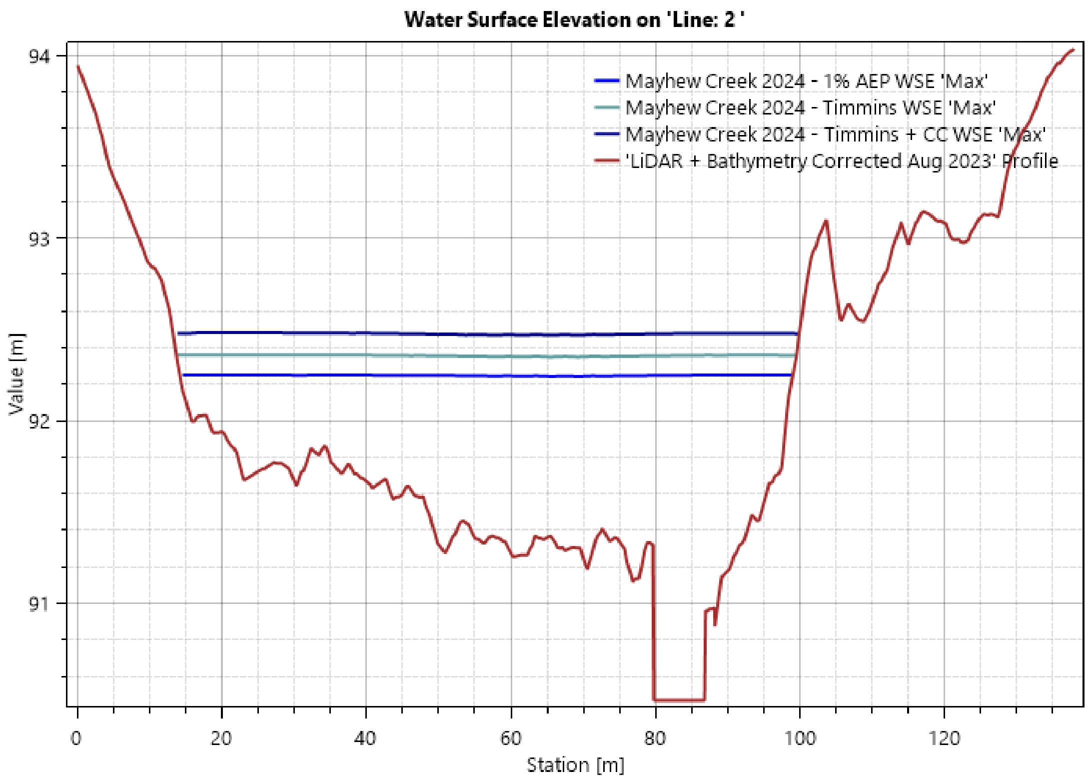




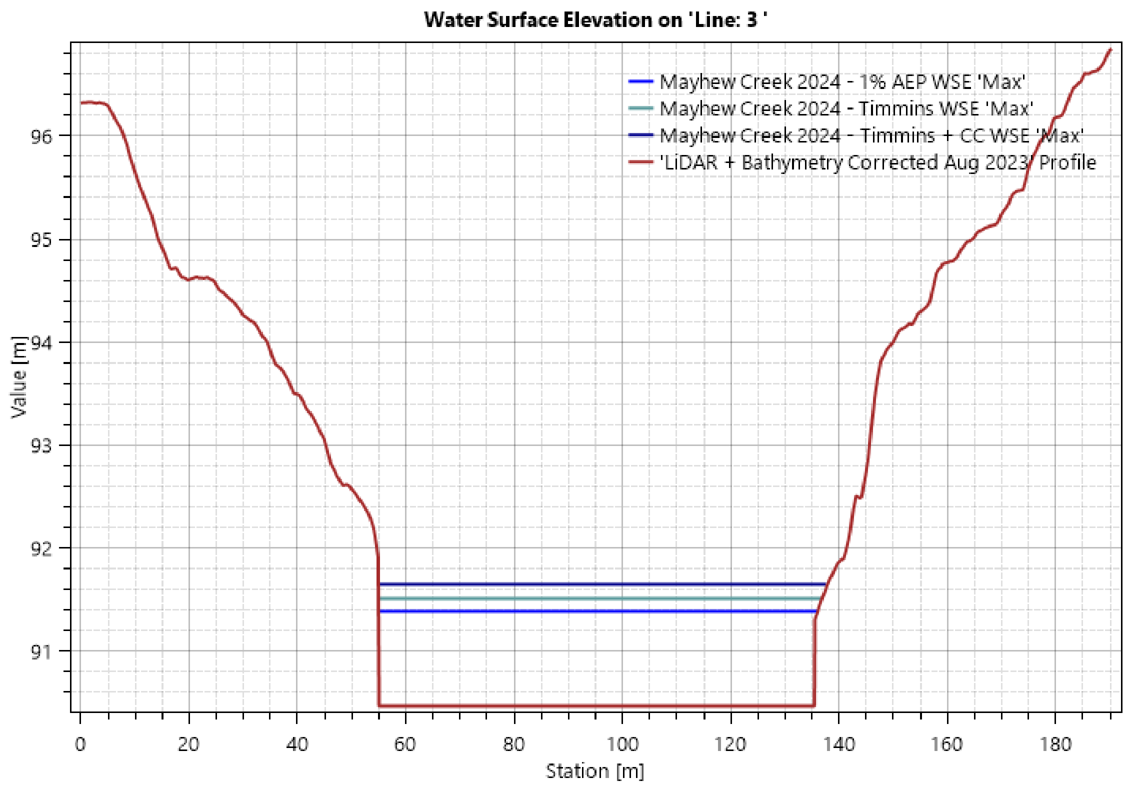

**Tate Rd, Station 5014**



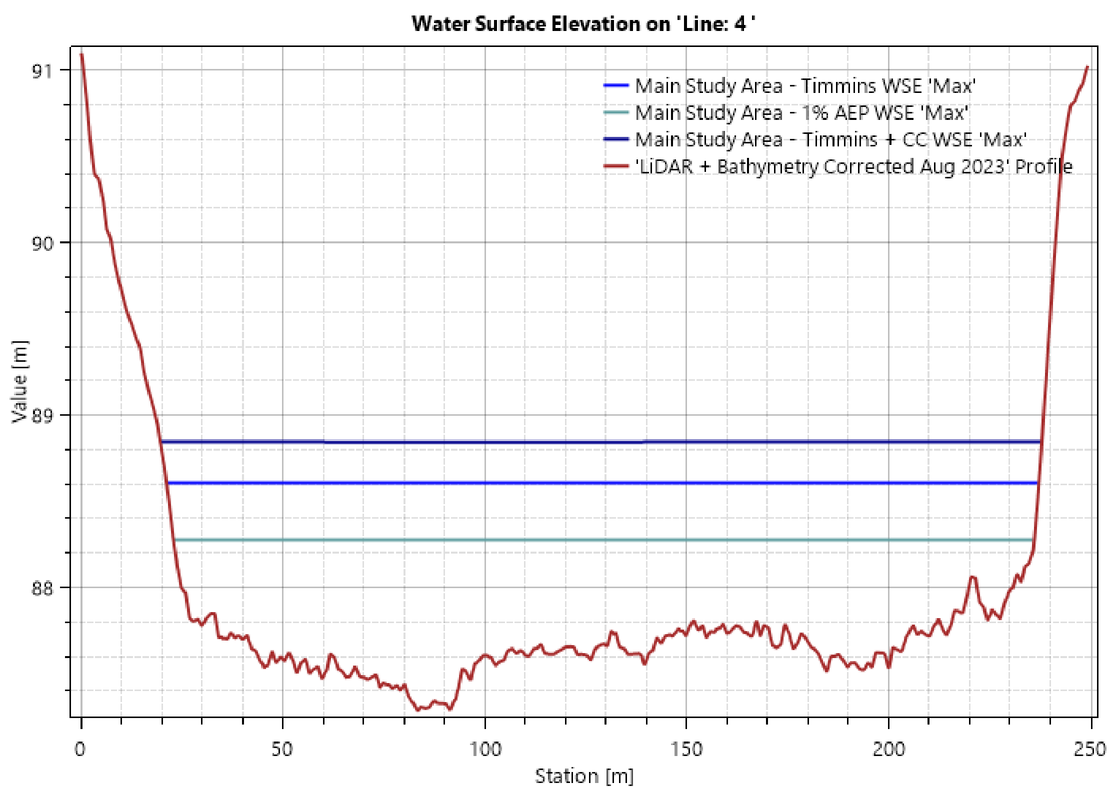

**Section 1, Station 4728**



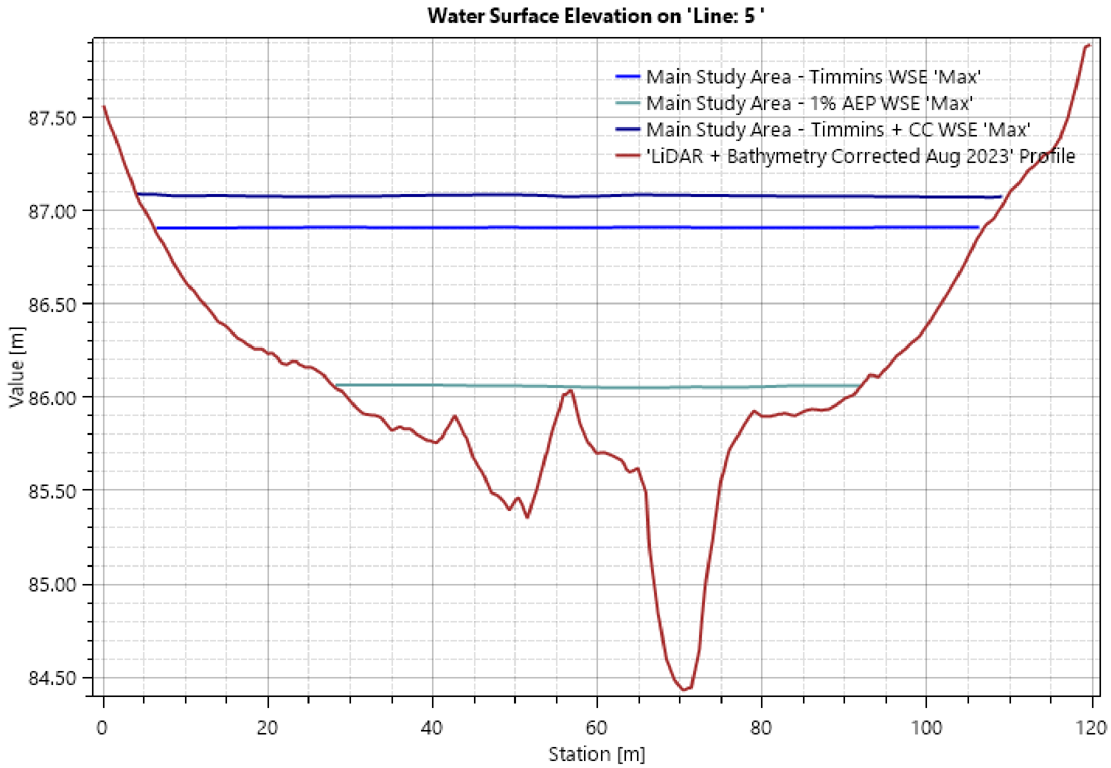

**Access Rd, Station 4302**



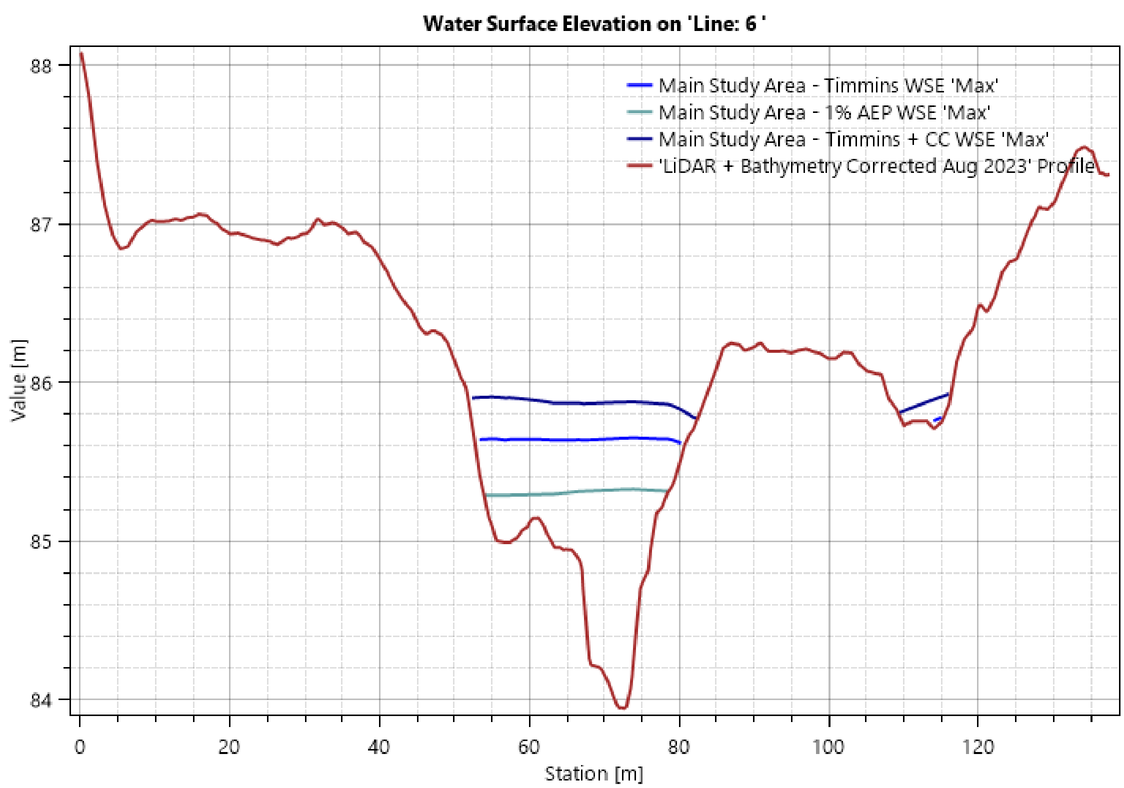

**Section 2, Station 4194**



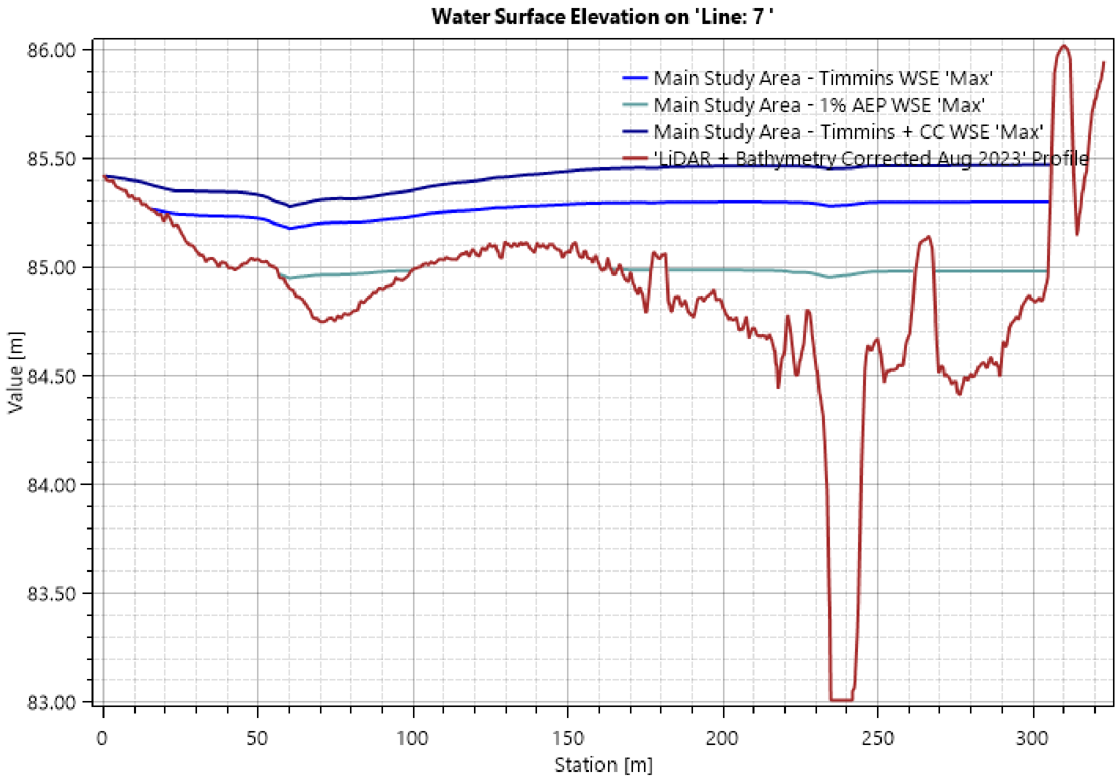

**Section 3, Station 2911**



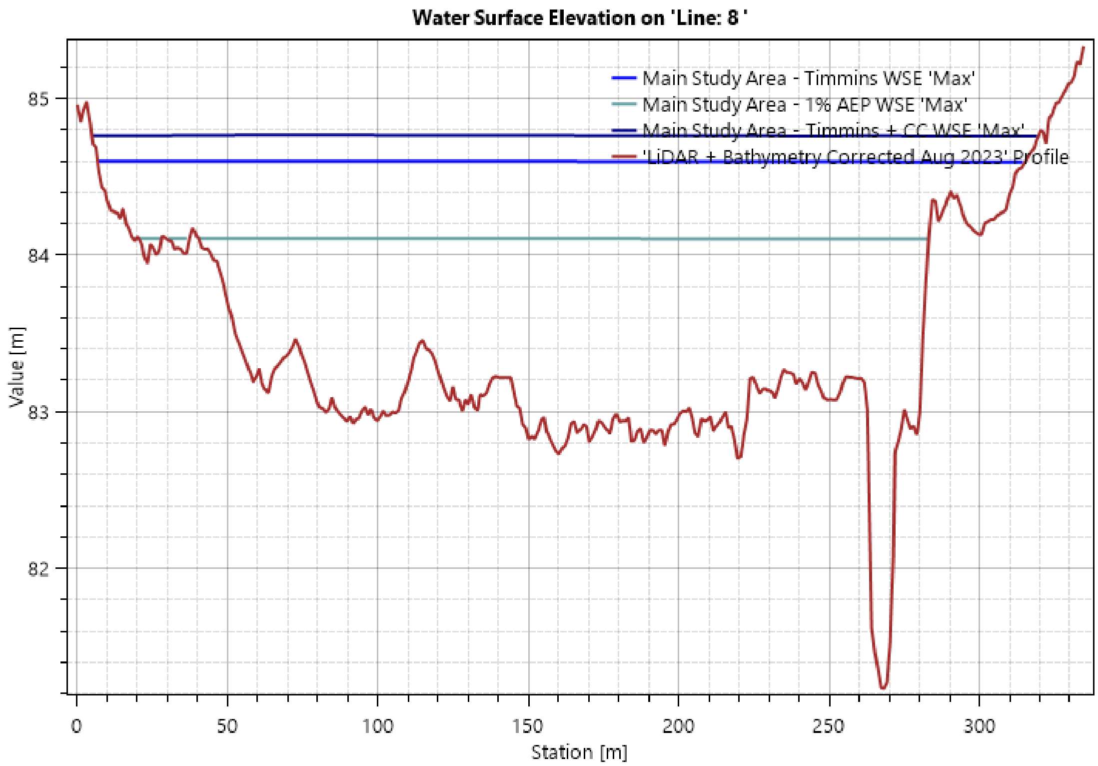

**Hill Rd, Station 2481**



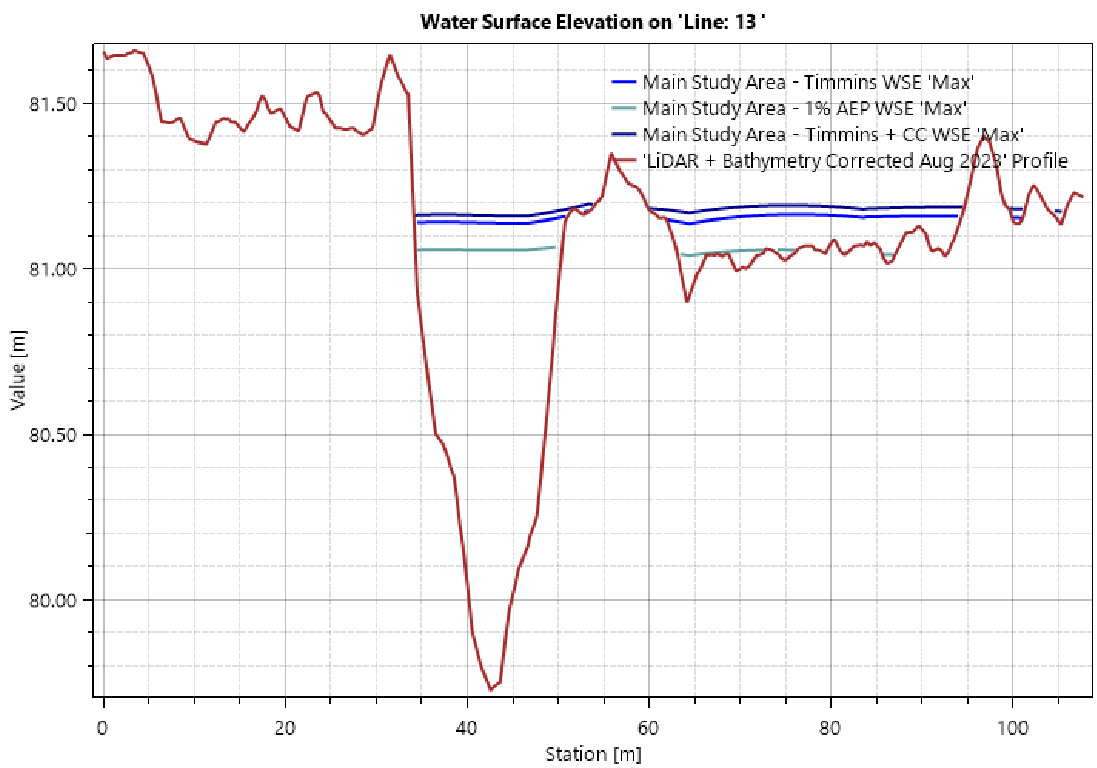

**Section 4, Station 2423**



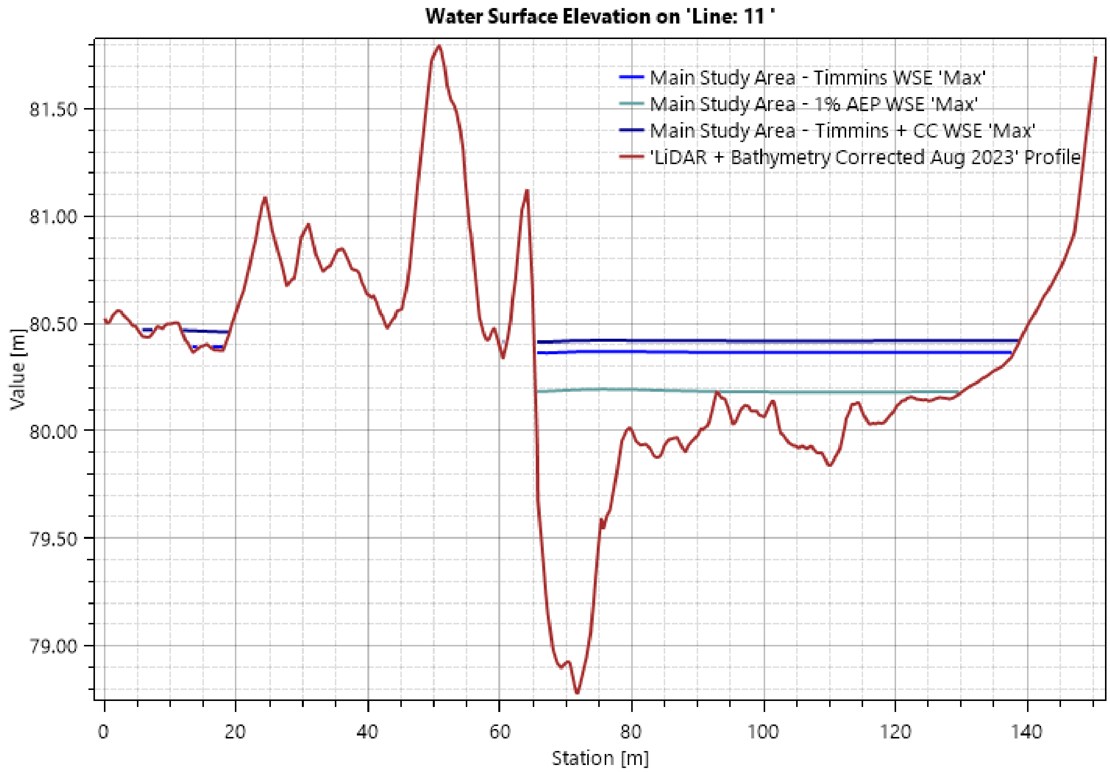

**2nd Dug Hill Rd, Station 2172**



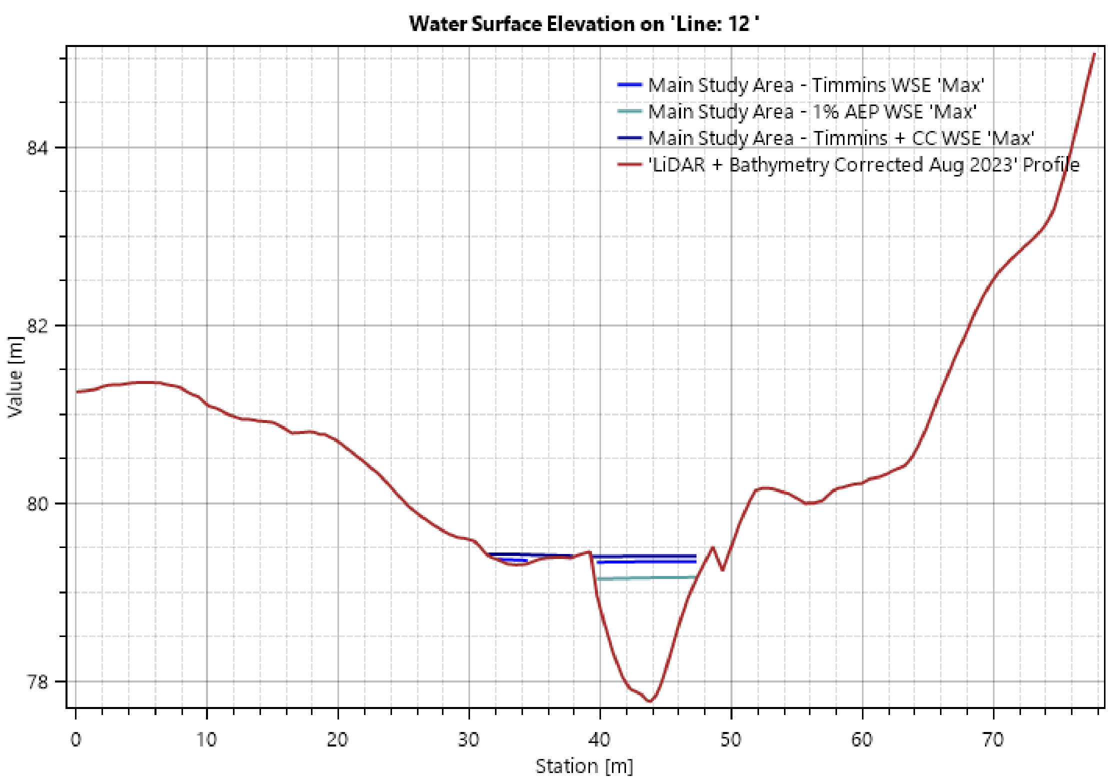

**Section 5, Station 1765**



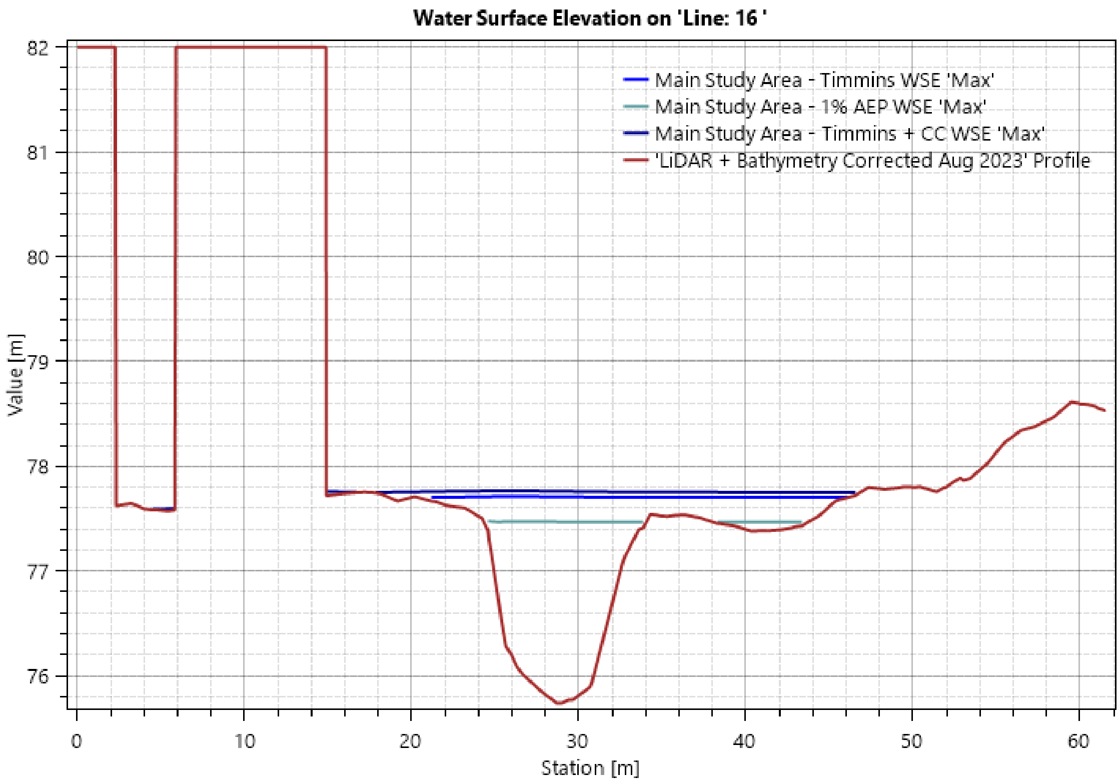

**Section 6, Station 1279**



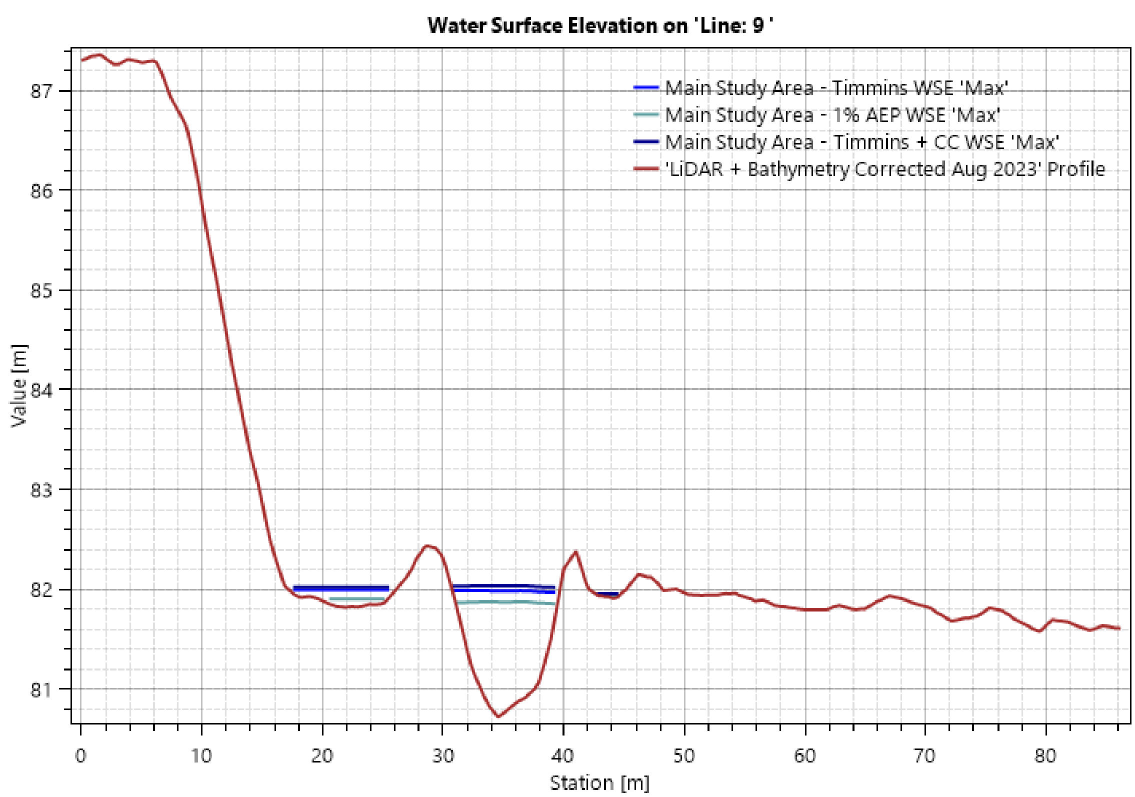

**Section 7, Station 932**



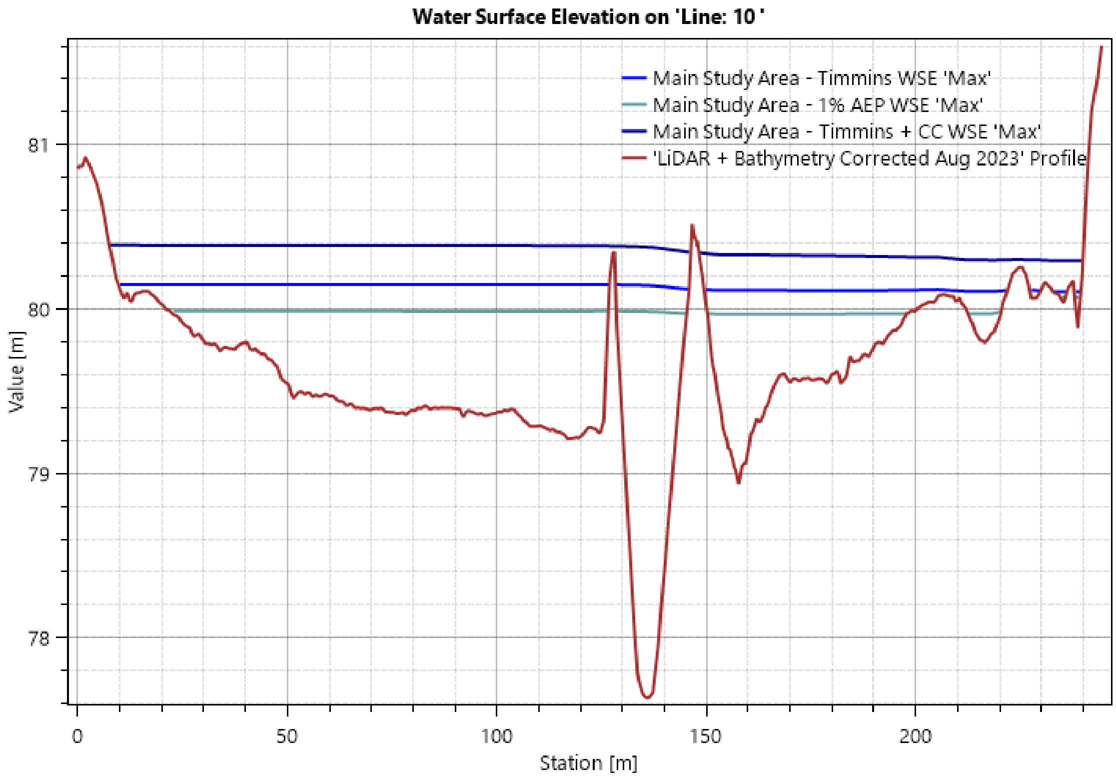

**Section 8, Station 512**



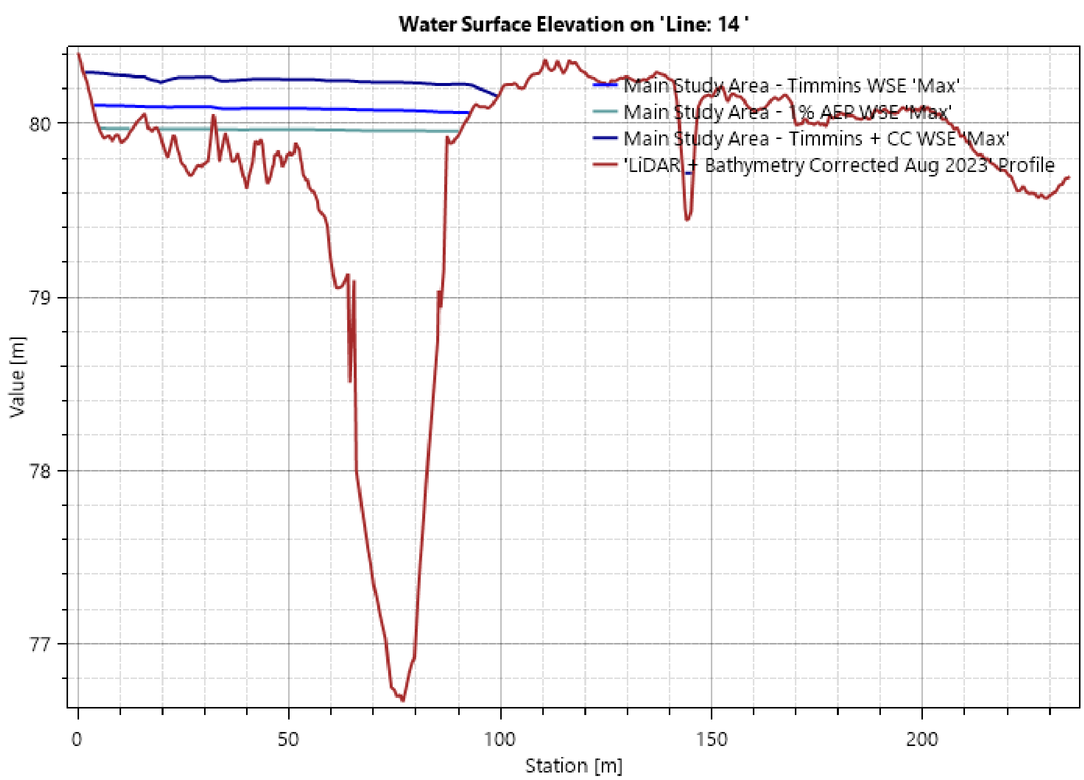

**Section 9, Station 333**



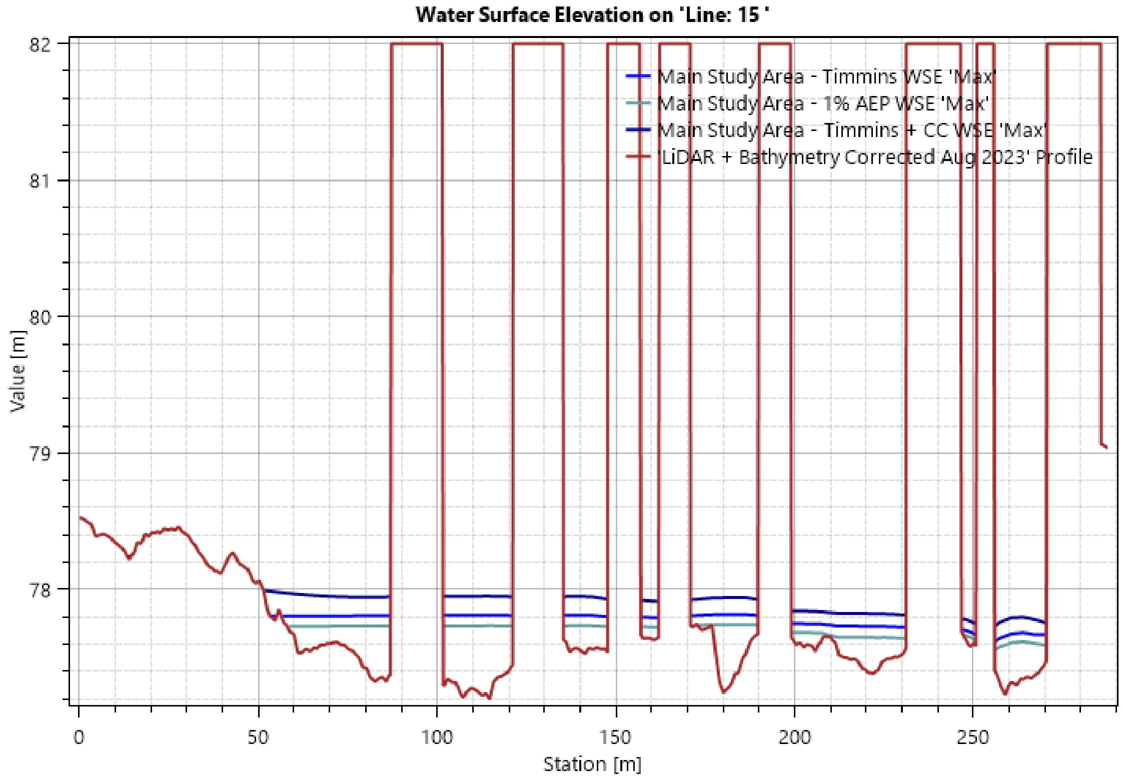

**Section 10, Station 1068**




**Section 11, Station 571**



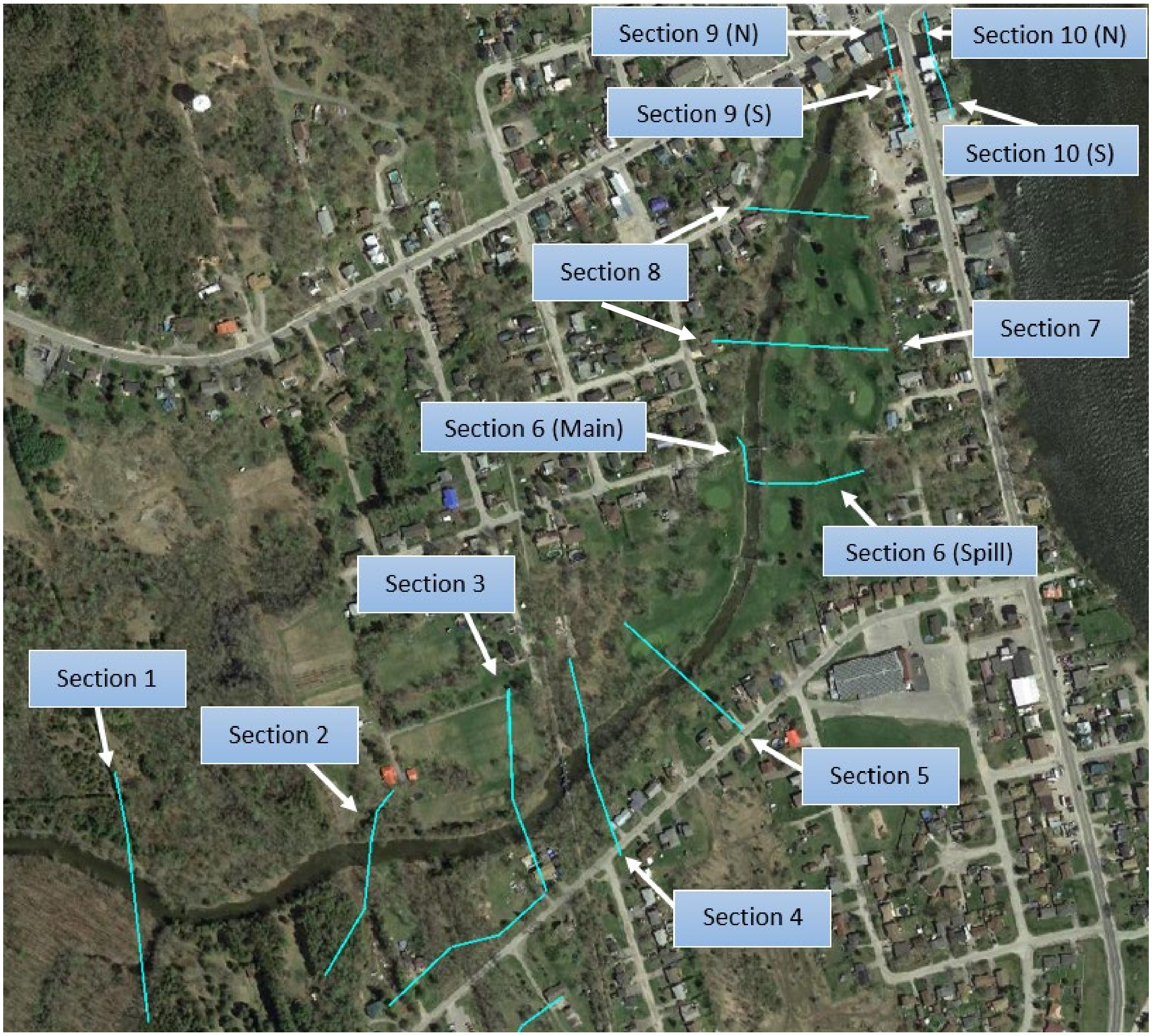

**Section 12, Station 434**

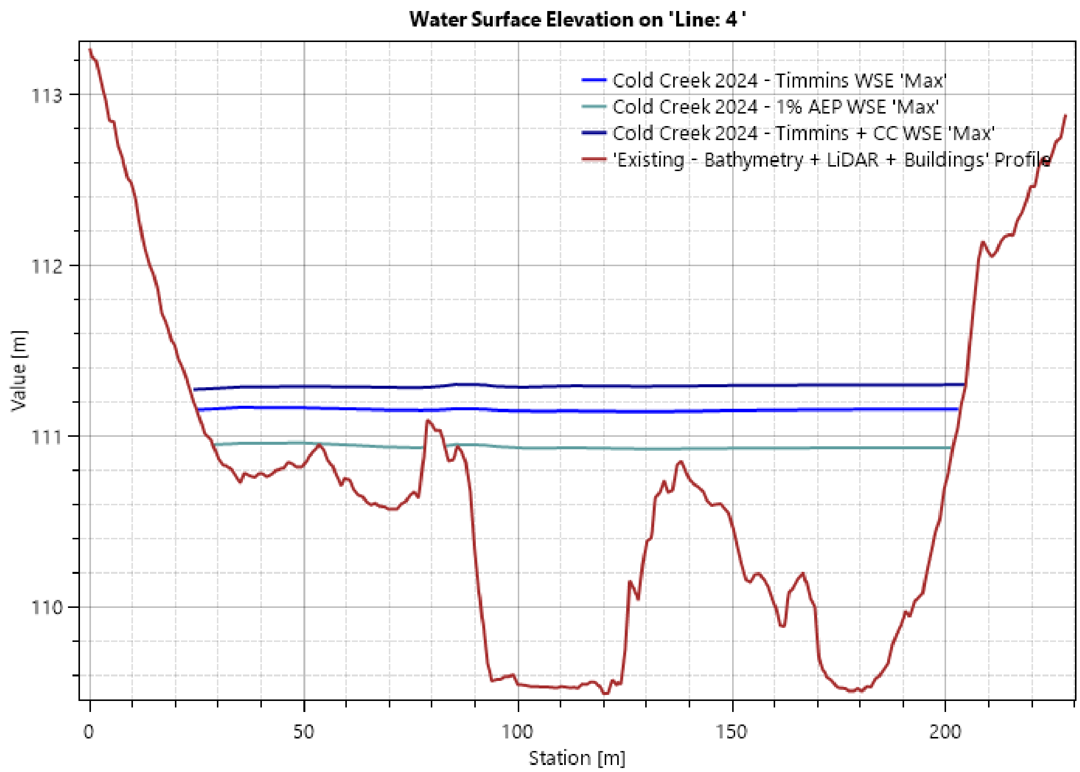



**Section 13, Station 249**

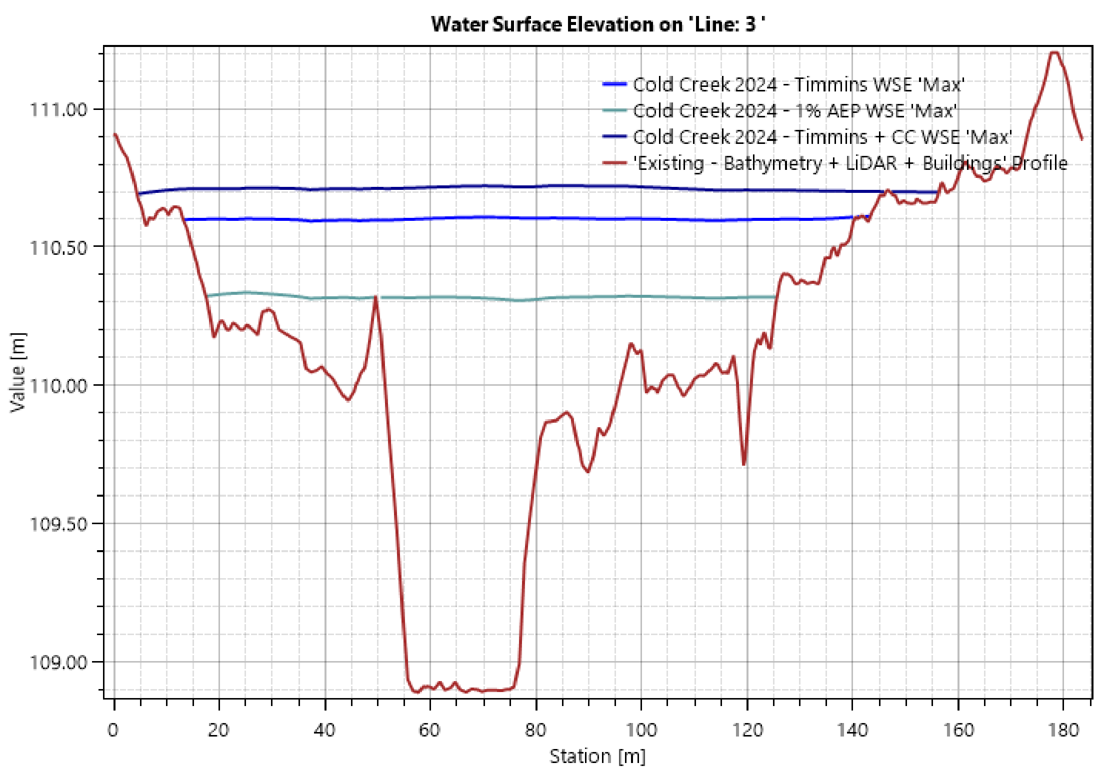
# Appendix K-2:
## Cold Creek Cross Section WSEL Plots

**Locations of Cross Section Profile Plots:**

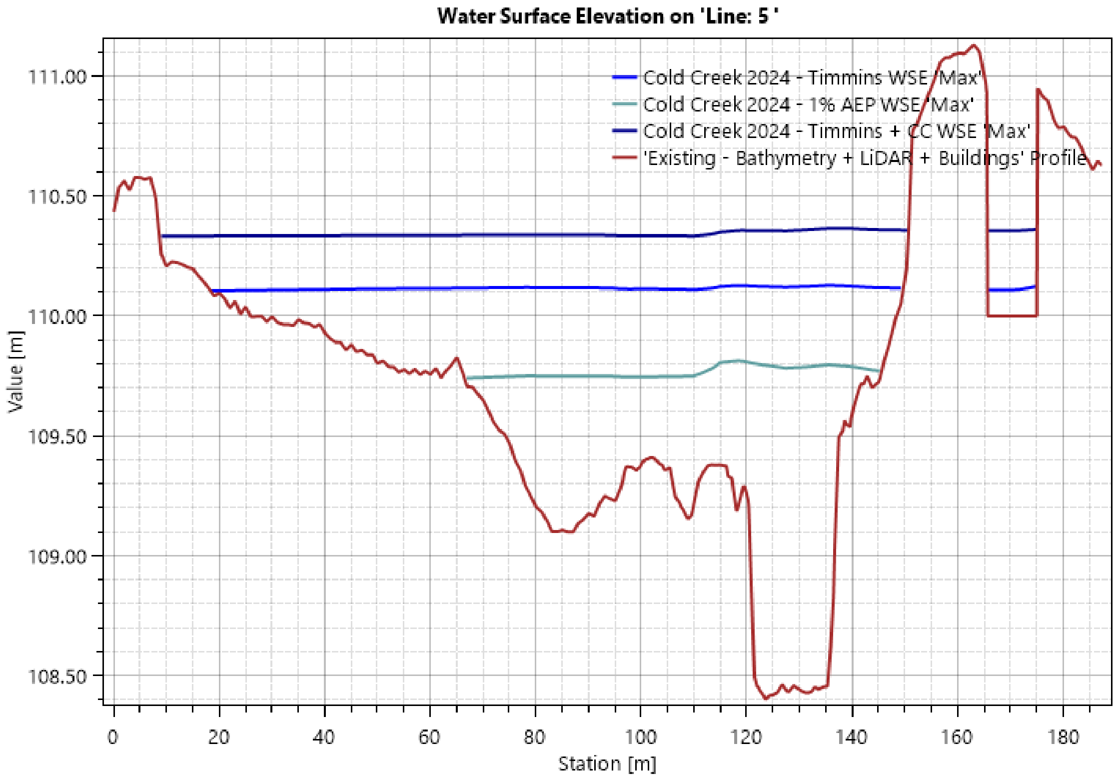




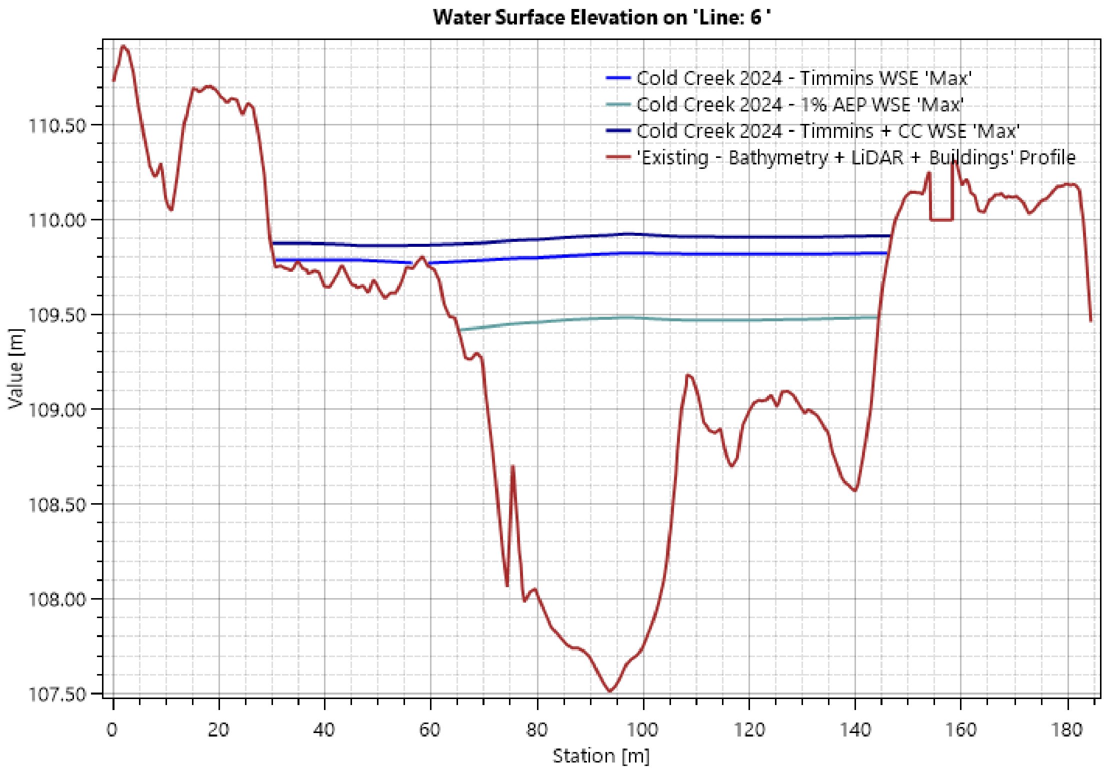

**Section 1, Station 1274**



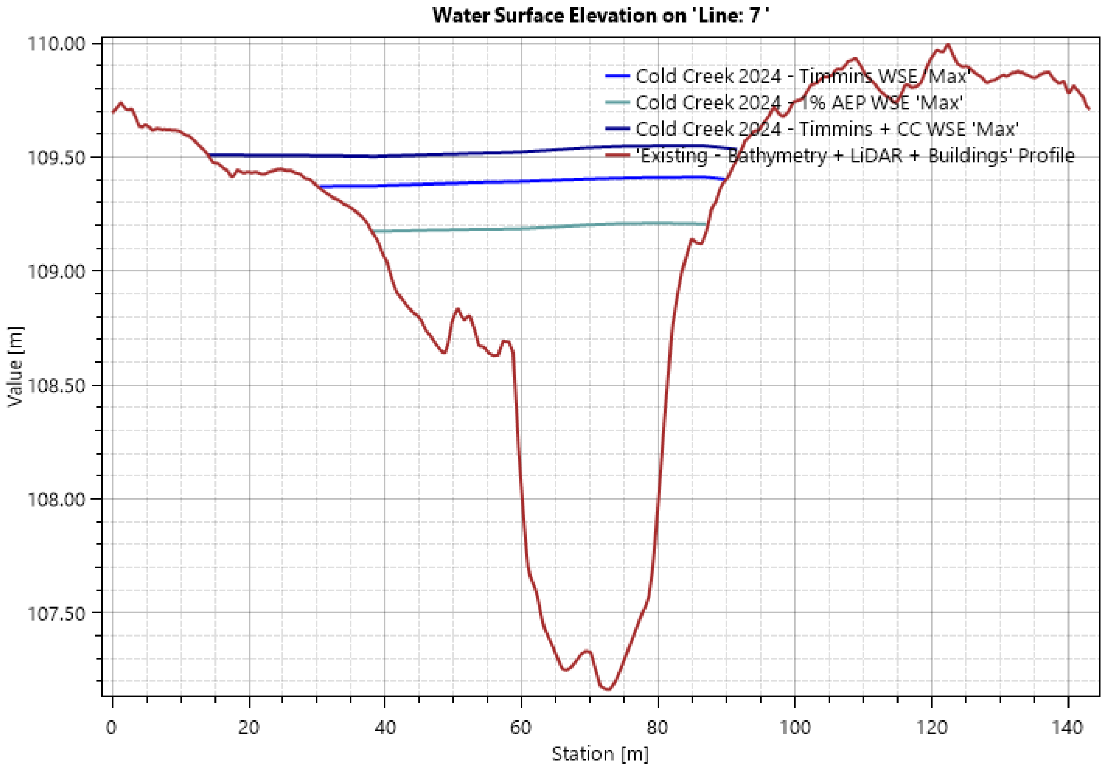

**Section 2, Station 1028**



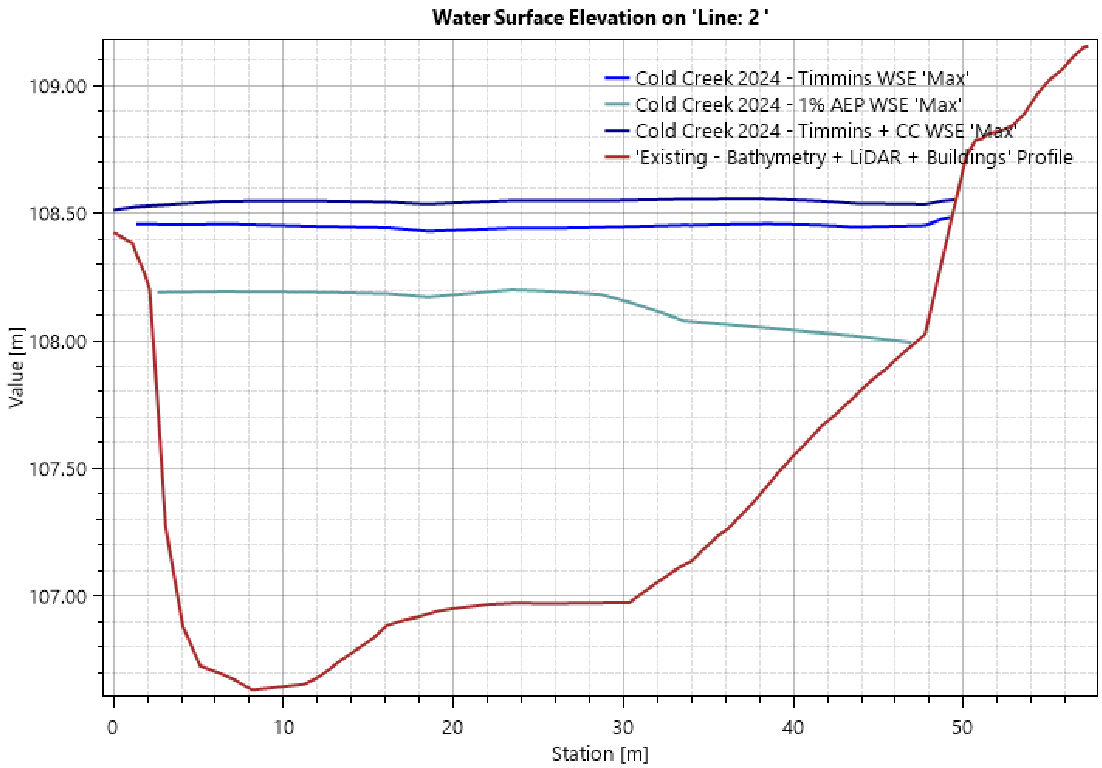

**Section 3, Station 876**



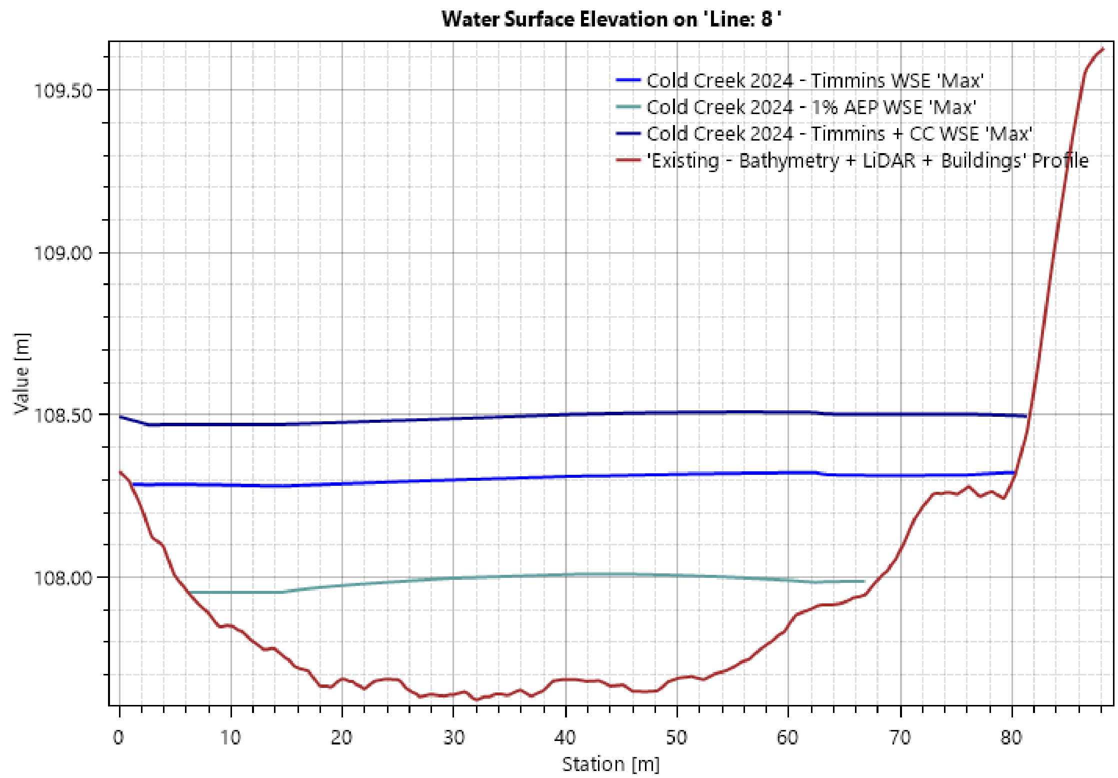

**Section 4, Station 808**



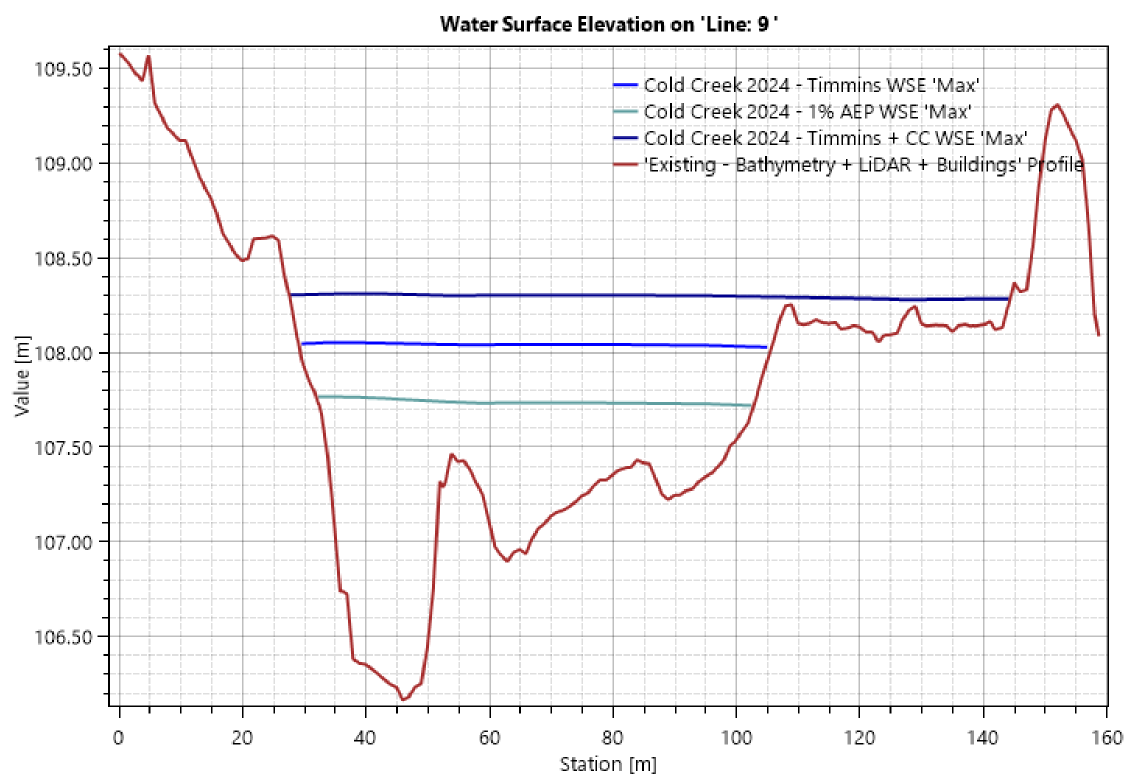

**Section 5, Station 726**



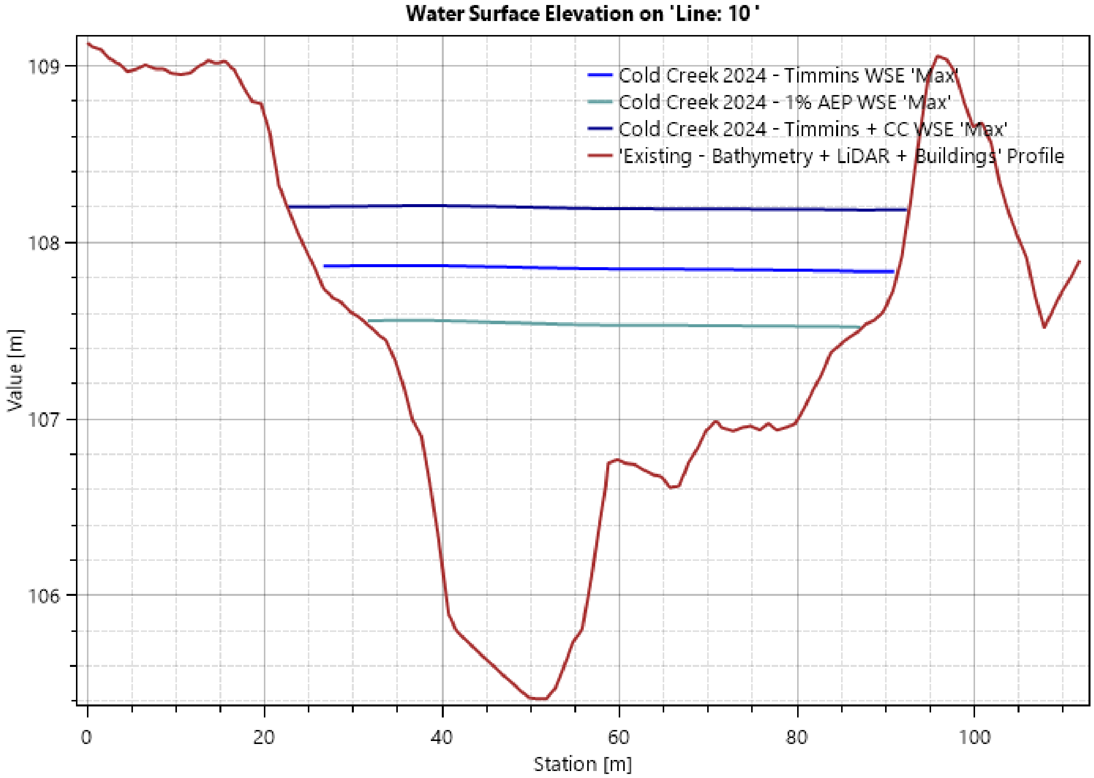

**Section 6 (main), Station 464**



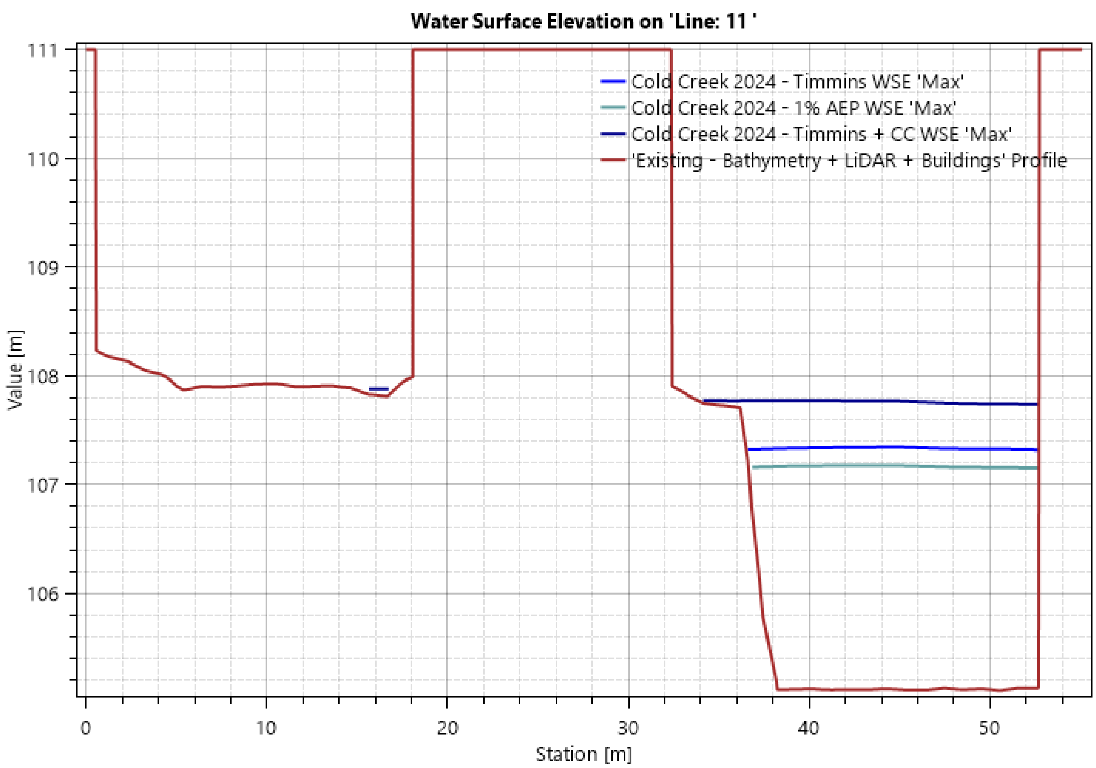

**Section 6 (spill), Station 464**



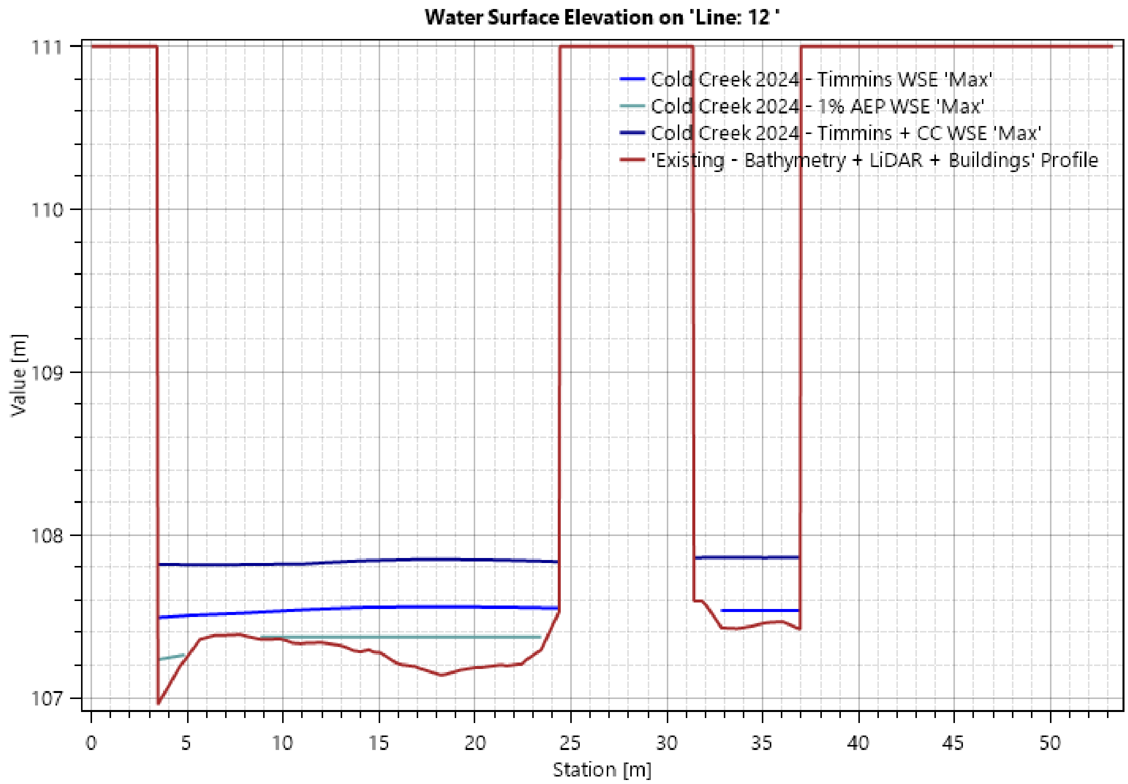

**Section 7, Station 349**



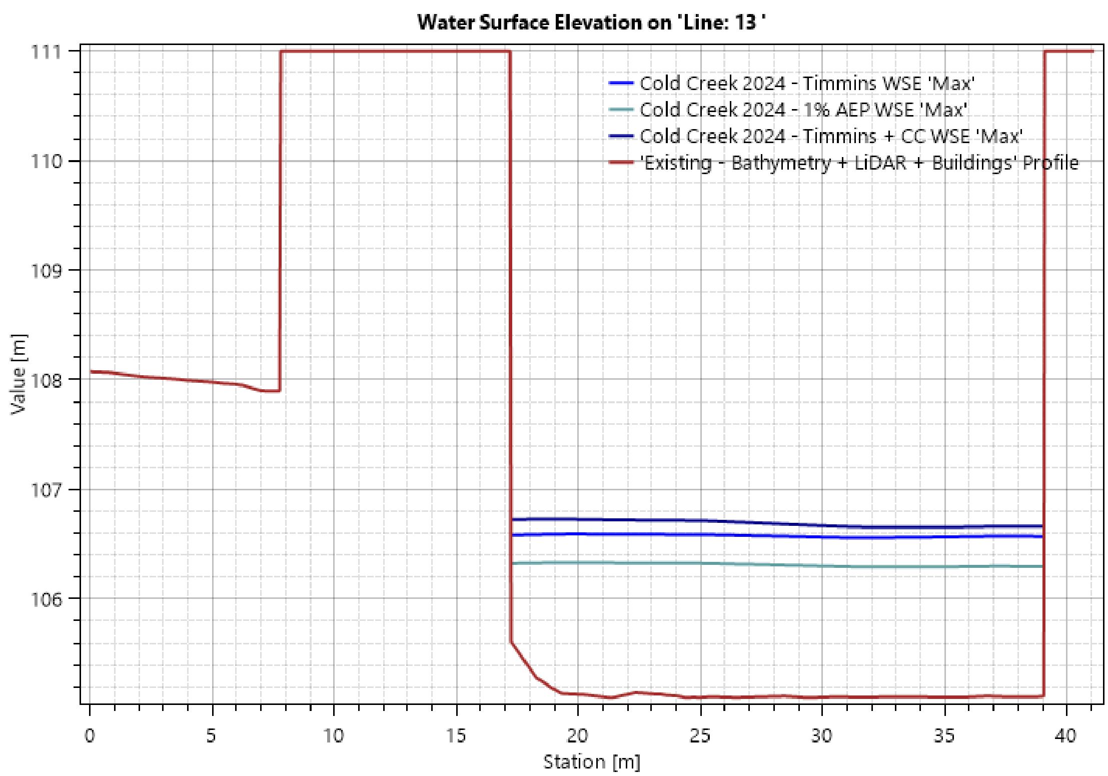

**Section 8, Station 226**



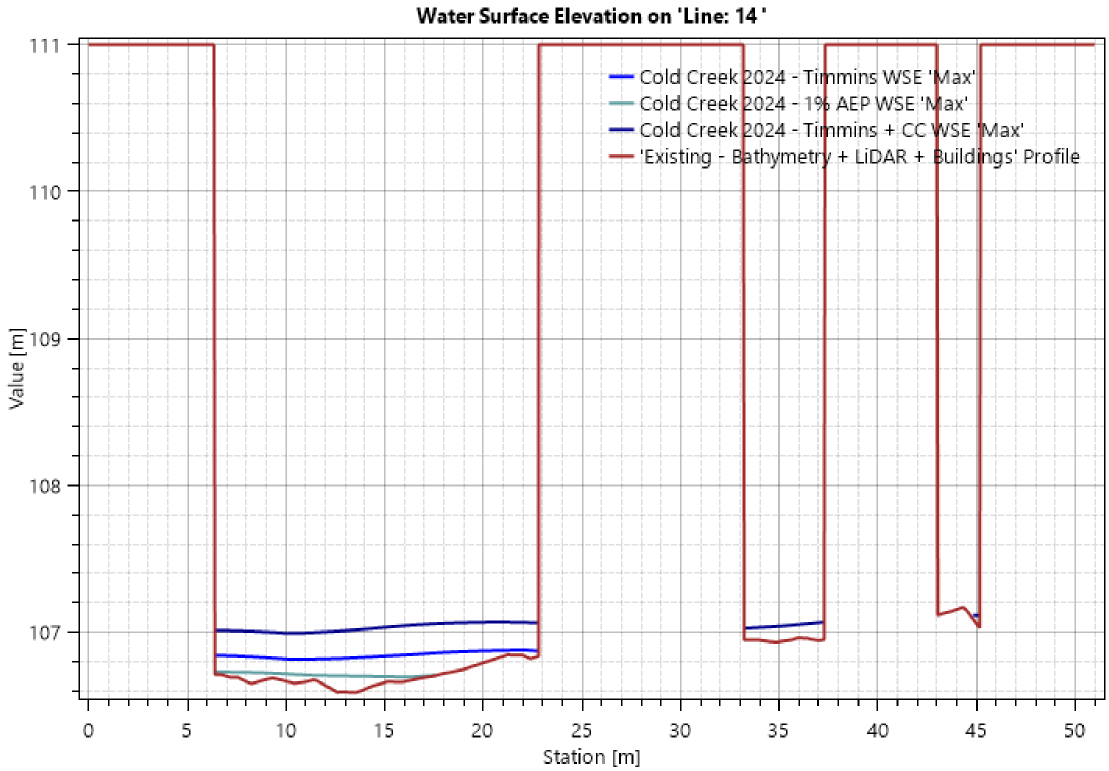

**Section 9 (N), Station 60**




**Section 9 (S), Station 97**

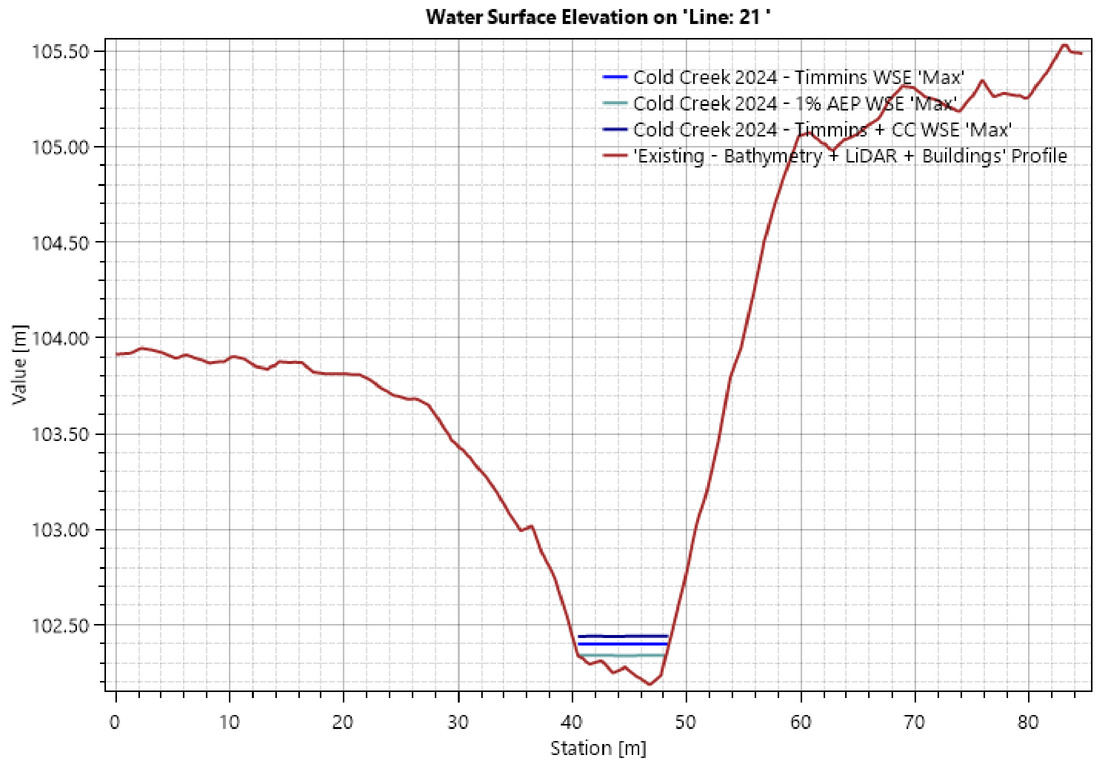



**Section 10 (N), Station 60**

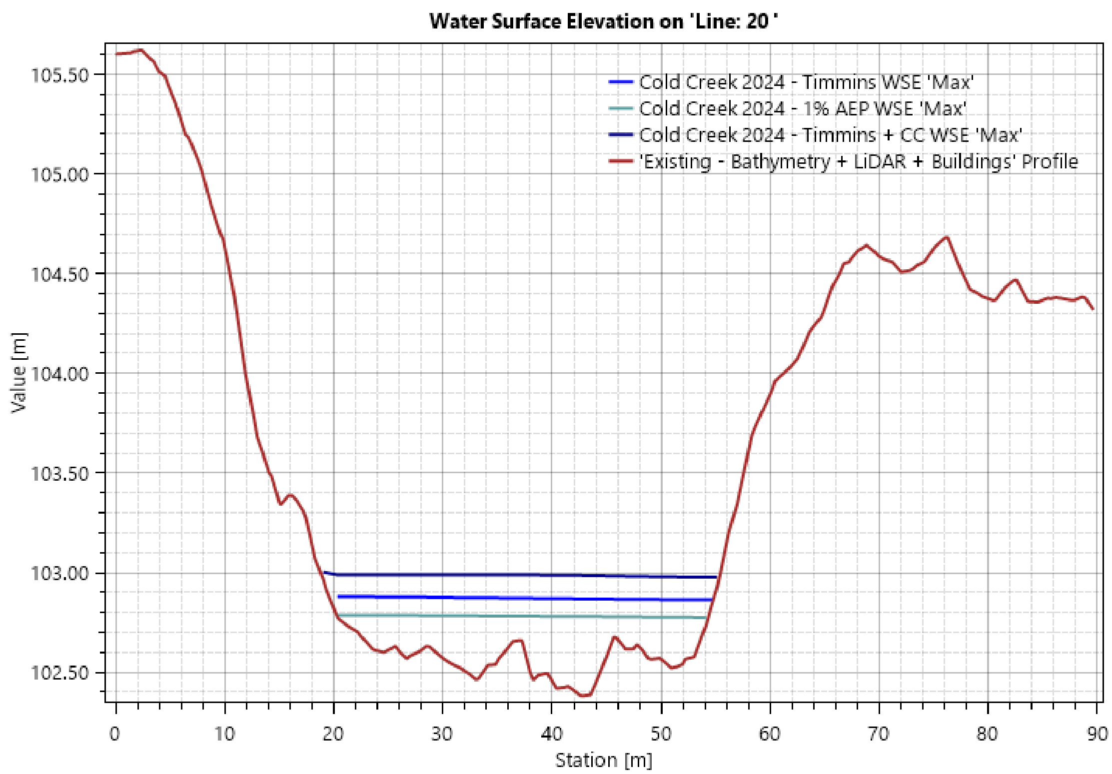



**Section 10 (S), Station 6**

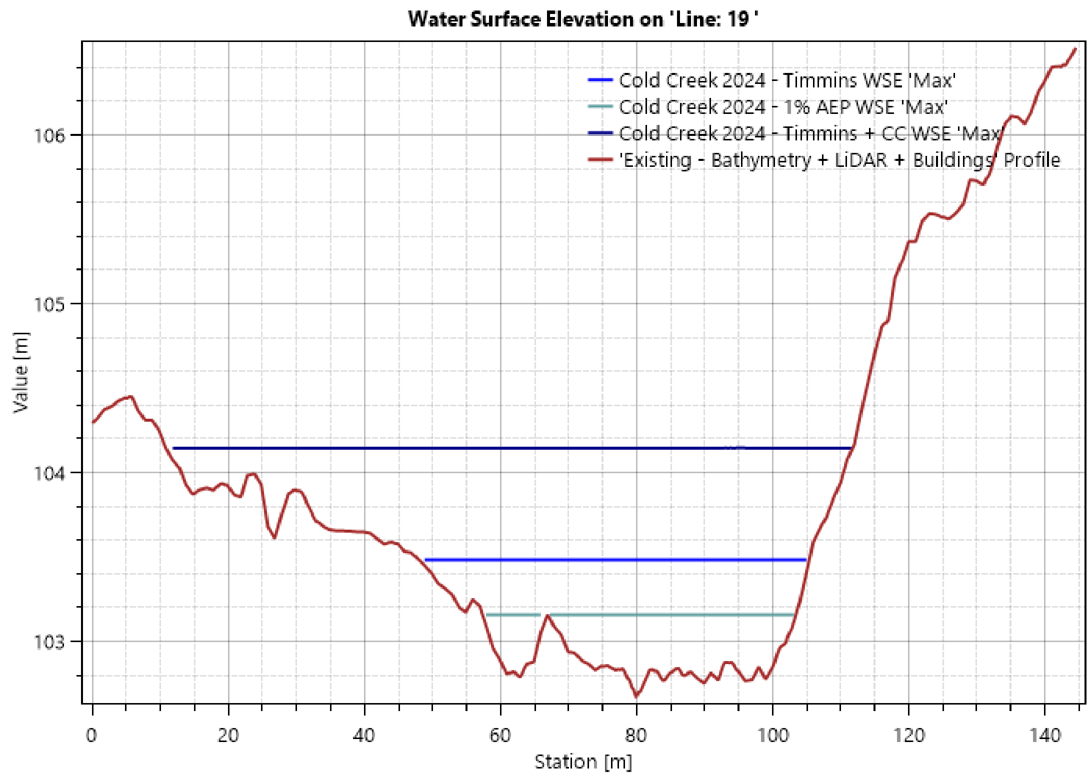
**Locations of Cross Section Profile Plots (Spill)**



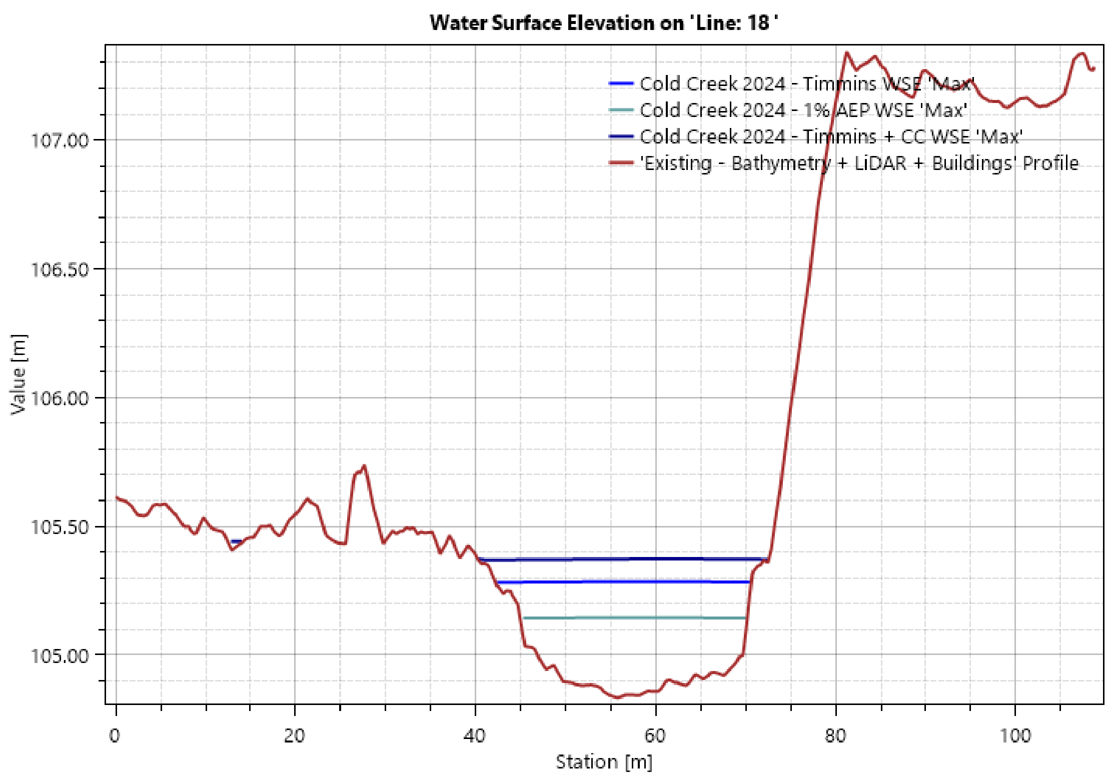

**Section 18, Station 149**



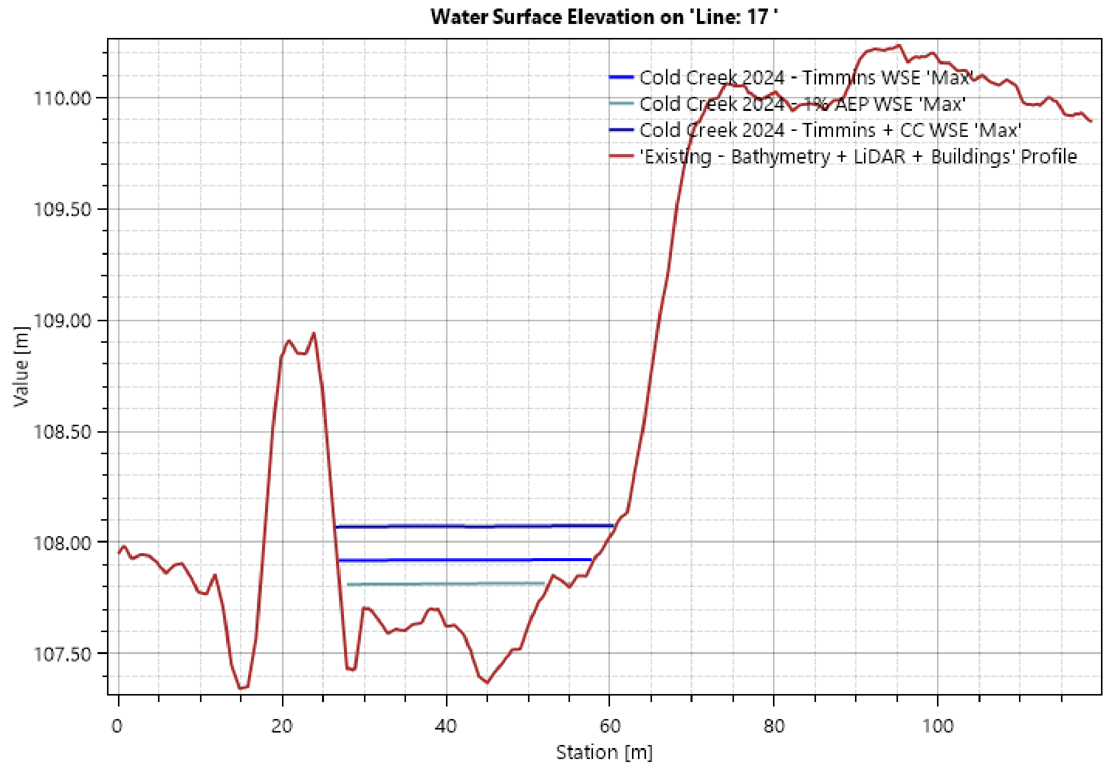

**Section 17, Station 149**



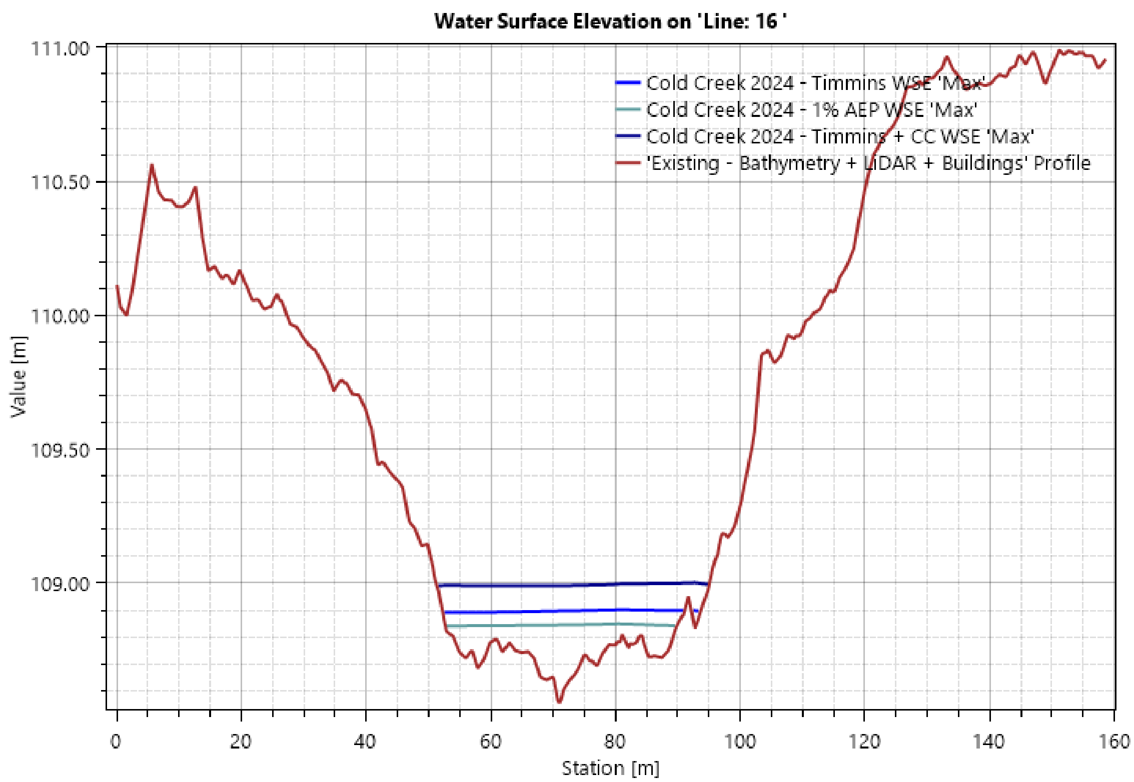

**Section 16, Station 305**



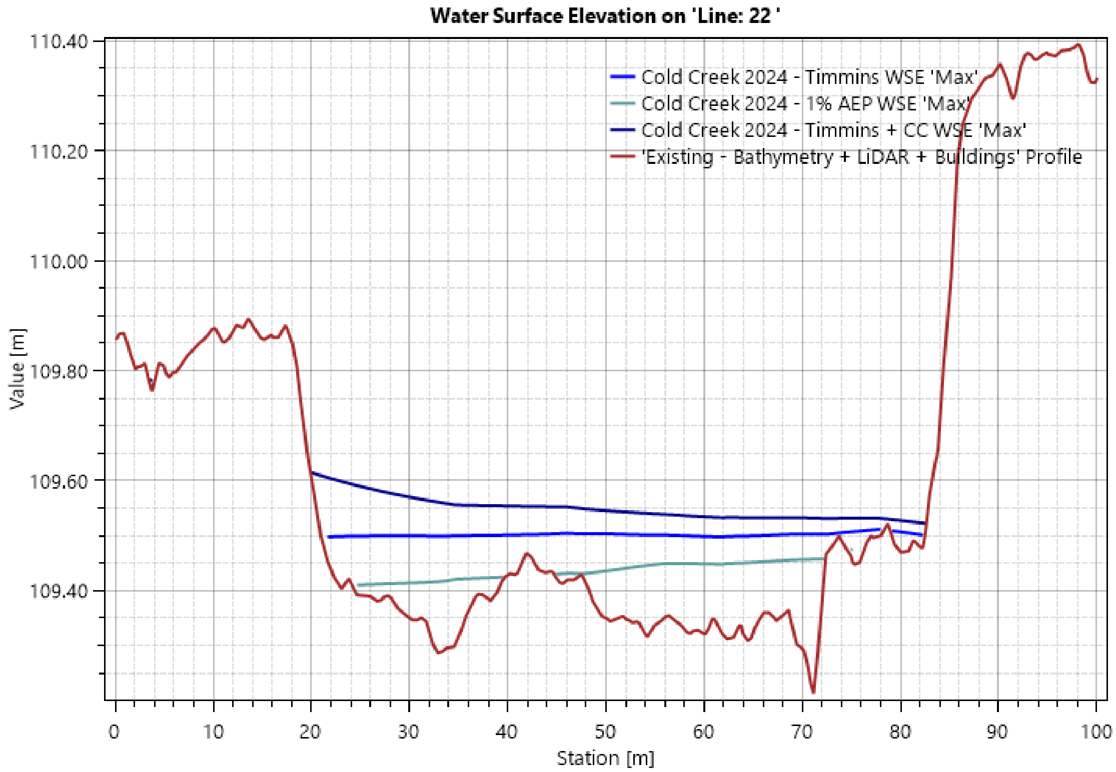

**Section 15, Station 1143**



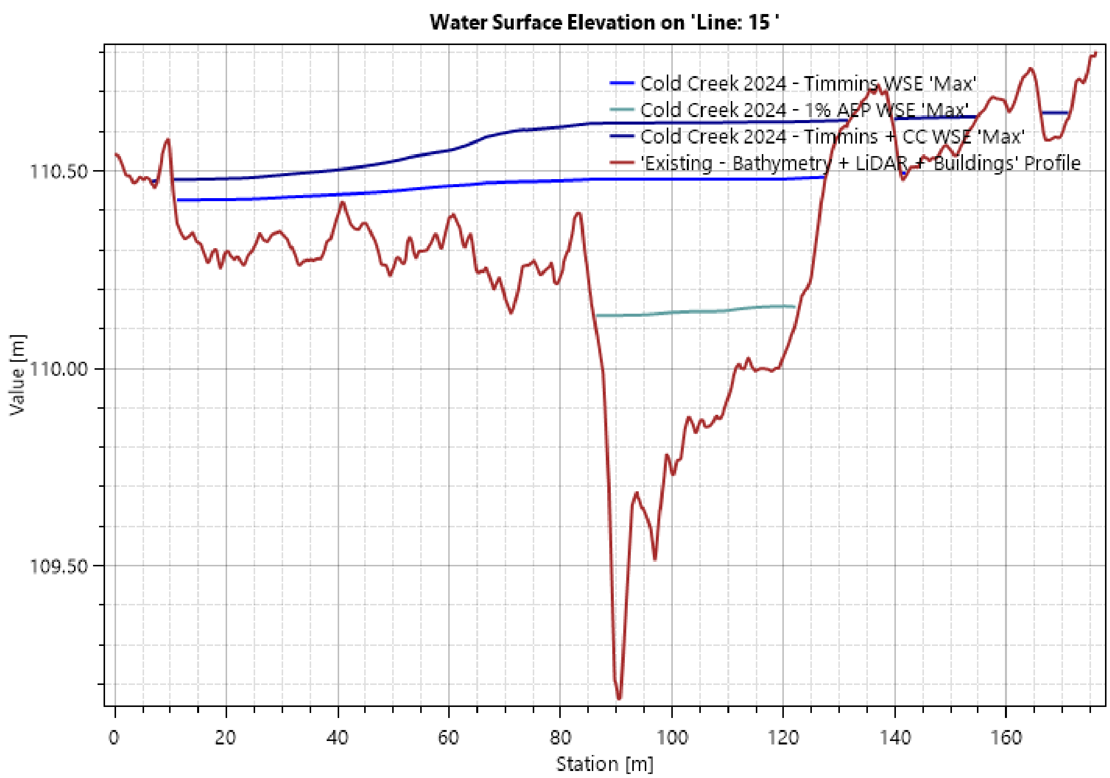

**Section 14, Station 2143**




**Section 13, Station 2564**




**Section 12, Station 2737**




**Section 11, Station 2839**