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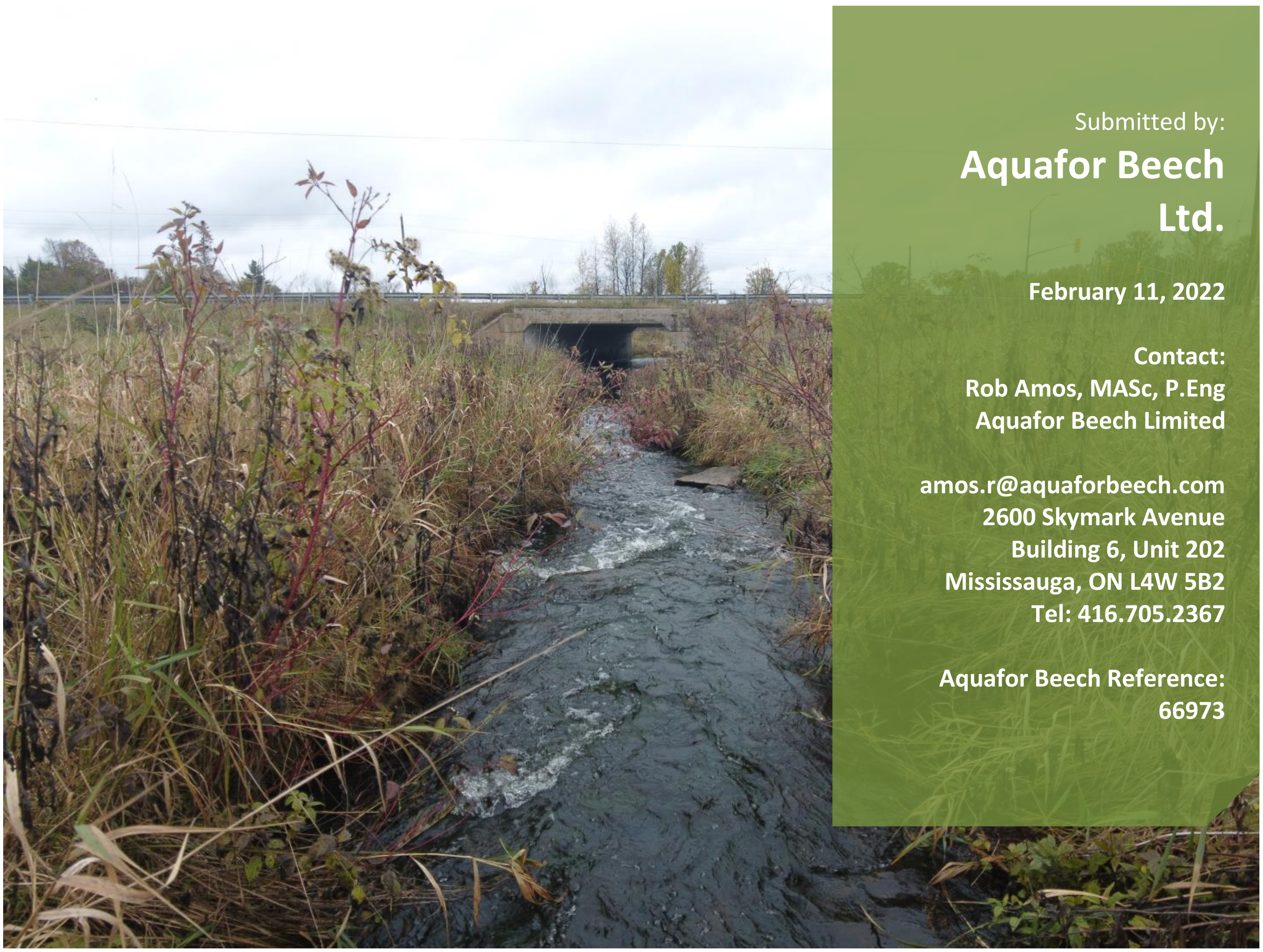
Floodplain Mapping – Deer Creek, Village of Madoc, Municipality of Centre Hastings

Hydrological Modelling Report



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February 11, 2022

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1 INTRODUCTION

Deer Creek drains a watershed area of approximately 77 km^2 in size. As illustrated in **Figure 1-1**, the creek is characterized by a main drainage branch and two tributaries – Madoc Creek and an unnamed tributary – which converge at the northern edge of the Village of Madoc (herein referred to as Madoc), after which the creek passes through Madoc and outlets into Moira Lake. The main branch extends approximately 18.6 km from its headwaters to its outlet at Moira Lake.

The purpose of this study is to establish updated Regulatory floodplain mapping for the Deer Creek watershed within Madoc, through detailed hydrological and hydraulic modelling, and related analyses of any flood hazards. The 100-year flood profile is used by Quinte Conservation to regulate development within the floodplain, to protect developed areas through structural land acquisition measures, and to identify properties at risk within the study area. The present report details the methodology and results of the hydrological component.

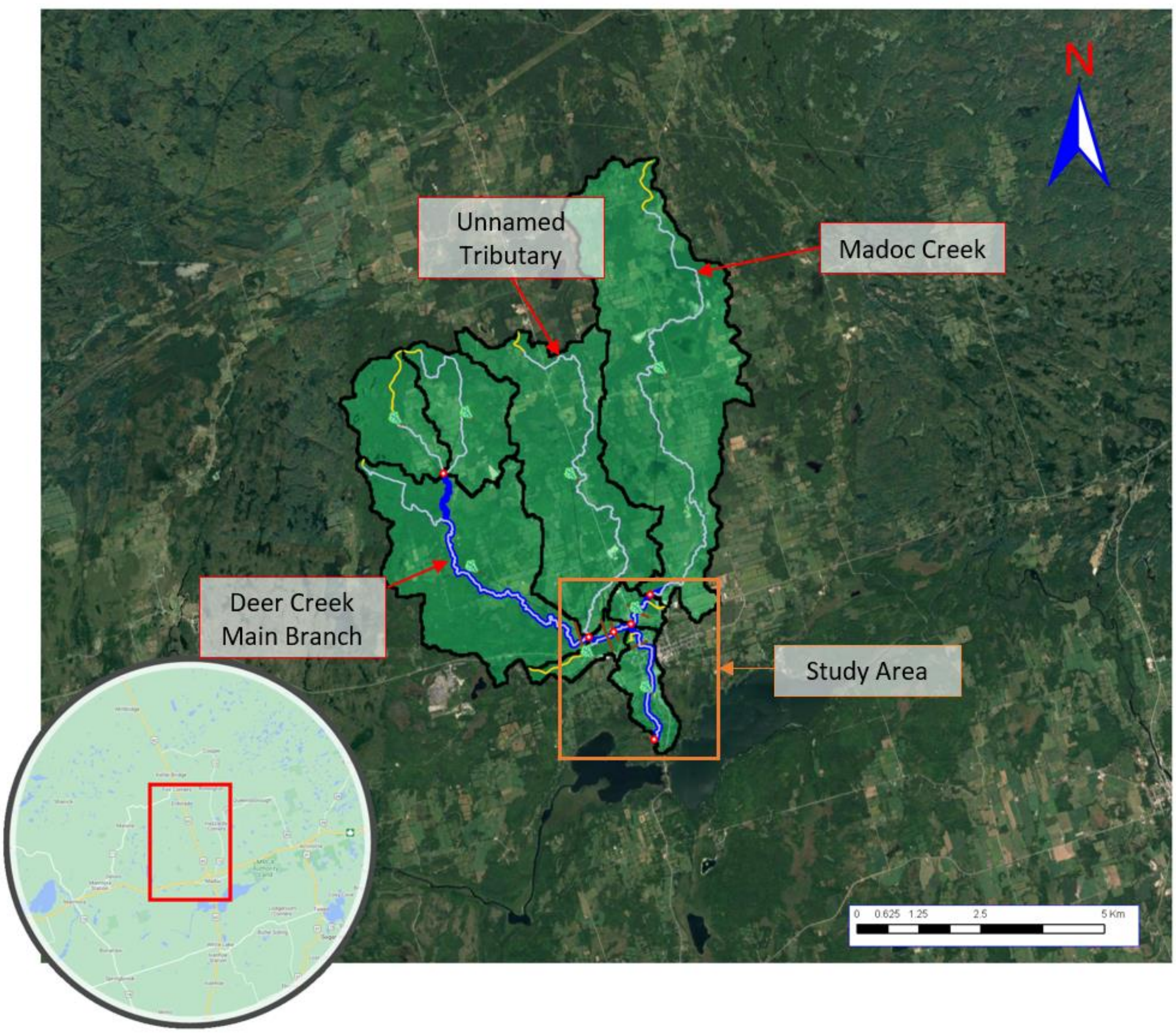

Figure 1-1: Deer Creek Watershed and Subbasins. Dark blue lines show reaches, yellow lines show longest flowpath shallow concentrated flow, and pale blue lines show longest flowpath channel flow.

2 DESCRIPTION OF THE WATERSHED

Deer Creek drains a watershed area of approximately 77 km^2 in size, with the main branch extending roughly 18.6 km from its headwaters to its outlet at Moira Lake. Channel slope along the main branch of Deer Creek is summarized in **Table 2-1**. The main branch and the unnamed tributary are dominated by high slopes exceeding 0.0085 m/m, but become flatter (<0.0035 m) upstream of their confluence. Conversely, the Madoc Creek tributary is characterized by a moderate upstream slope of 0.002 m/m, which increases sharply to 0.033 m/m near the confluence with the main branch of Deer Creek. After converging with the two tributaries, the slope of Deer Creek becomes increases to 0.010 m/m as it passes through Madoc, then flattens out before discharging into Moira Lake.

Immediately downstream of the confluence, the average slope is 0.00055 m/m. Riparian wetlands exist along the majority of the main branch and tributaries, becoming particularly wide near the confluence of the main branch and the unnamed tributary. Wetlands immediately adjacent to the streams are composed of marshes that are vegetated primarily by Typha. Bogs, fens, and swamps are also present near the streams in some areas, enveloping marshes to create wetland complexes. The presence of these wetland complexes suggests that frequent flooding occurs in these areas, highlighting the importance of floodplain mapping to avoid flooding of new developments and to preserve the natural peak flow attenuation provided by the floodplains.

Table 2-1: Summary of Channel Slope Variation along the Main Branch of Deer Creek

Chainage (US to DS)	Channel Slope (m/m)
0 m - 12,580 m	0.006256
12,580 m - 13,730 m	0.003268
13,730 m - 14,420 m	0.00055
14,420 m - 15,900 m	0.000601
15,900 m - 17,180 m	0.00996
17,180 m - 18,600 m	0.000519

The Deer Creek watershed receives an average of ~920 mm per year and has an average annual temperature of 6.8°C. Loam (61%) and rockland (26%) comprise the majority of the watershed soil cover classification, with the remaining area consisting of wetlands (6%), clay (4%), soil classified as “urban” (<2%), and open water (<1%). As illustrated in **Figure 2-1**, the majority of the soil is classified as Hydrological Soil Group B. Precambrian bedrock dominates the geology in this area, with outcrops of shale, sandstone, limestone, and dolostone. **Figure 2-2** shows that land uses within the watershed are predominantly rural in nature, consisting primarily of forests (52%) and agriculture (39%). Urbanization is concentrated in Madoc and only accounts for ~2% of the total watershed area. The use of soil hydrological classification and land use for calculating runoff is described in **Section 4.7**, with CN values shown in **Table 4-7** and **Table 4-8**.

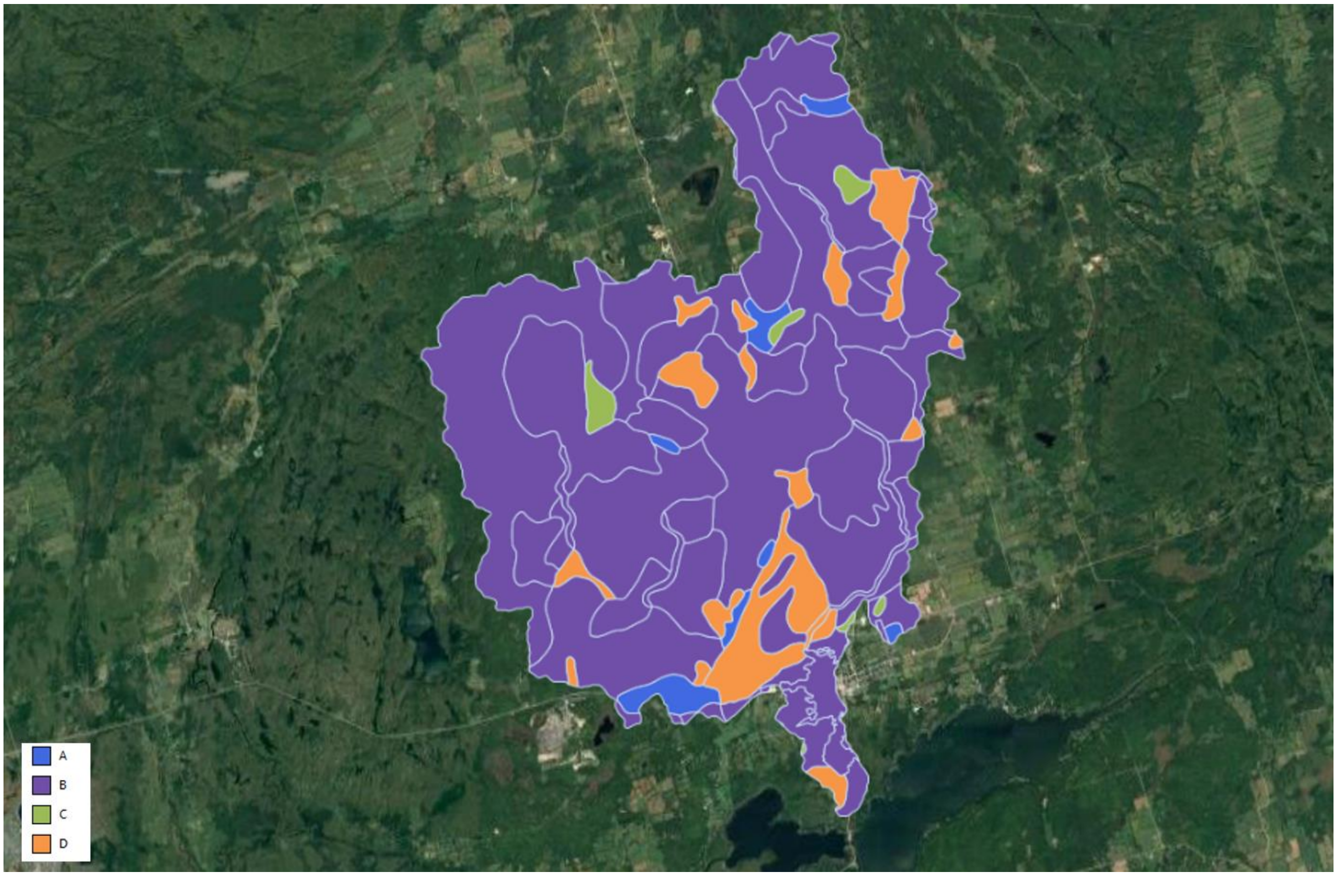

Figure 2-1: Soil Hydrological Group Distribution within the Deer Creek Watershed

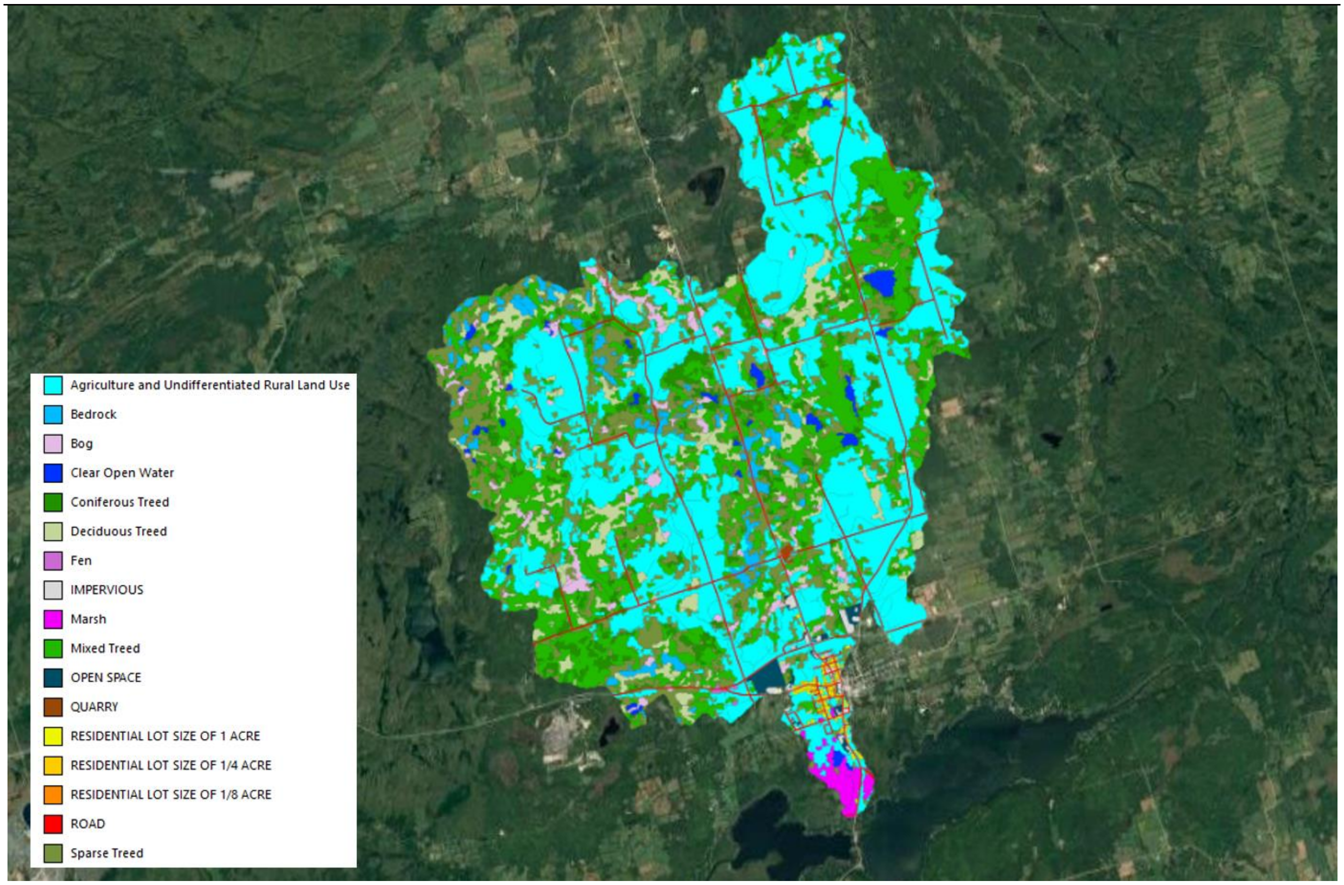

Figure 2-2: Land Use within the Deer Creek Watershed

3 PREVIOUS STUDIES

Two hydrological/hydraulic studies have previously been undertaken within the Deer Creek watershed:

Water Management Study Deer Creek: Village of Madoc (1986)

This study was undertaken by Garatech Inc. to estimate peak flows and flooding risk within the Village of Madoc. Peak flows were estimated using 4 methods, 3 of which were based on empirical relationships, and 1 of which was creating using the HYMO synthetic unit hydrograph method. The synthetic hydrograph method produced the most conservative flows, which were estimated to have a downstream peak of 82.26 m^3/s for a 7-day, 100-year snowmelt event. The HYMO model incorporated 6 sub-basins and 3 reaches; geographical partitioning and setup are shown in **Figure 3-1** and **Figure 3-2**, respectively.

The report highlighted that Deer Creek is different from most watersheds because it is drained by a main branch and two tributaries that converge just upstream of Madoc. This drains the Deer Creek watershed more quickly than most similarly-sized watersheds and produces higher peak flows.

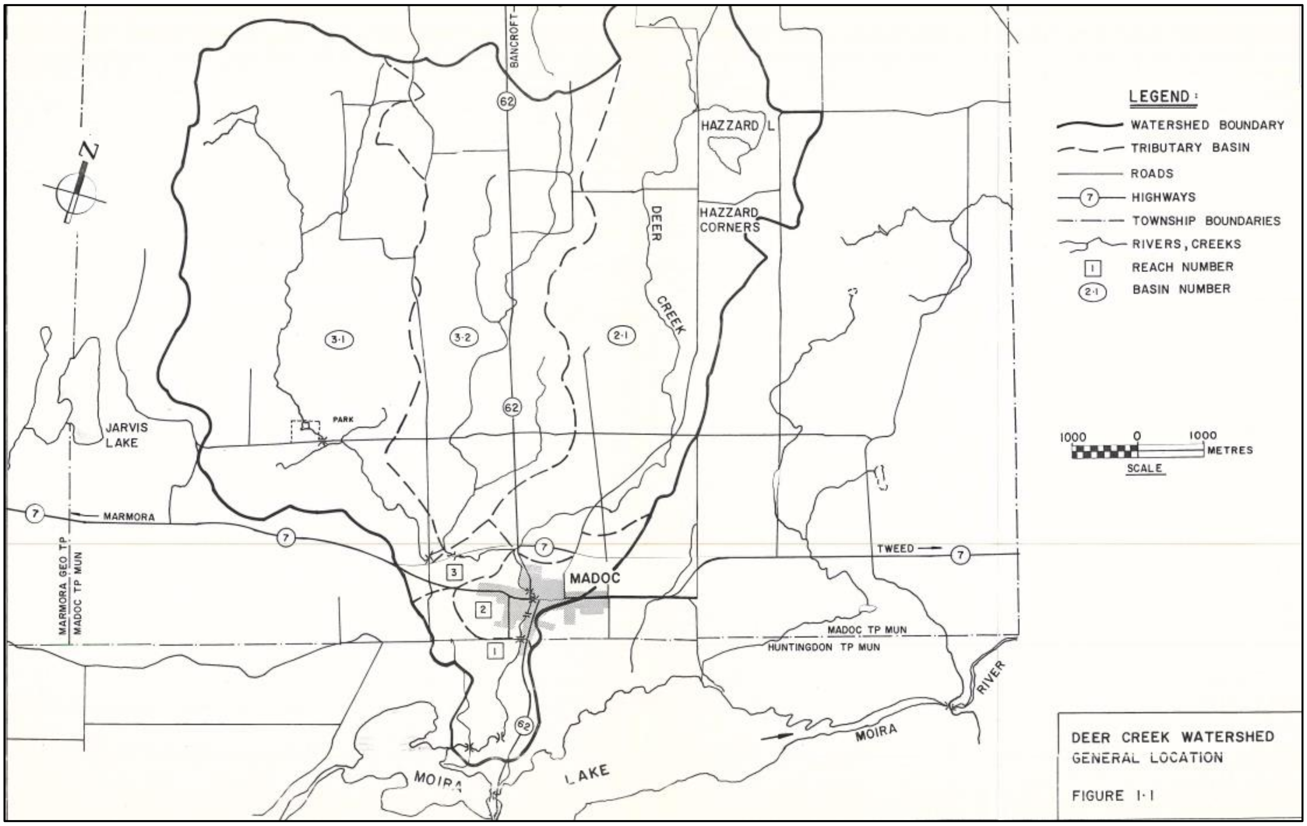

Figure 3-1: Deer Creek Partitioning in the HYMO Model (Garatech Inc., 1986)

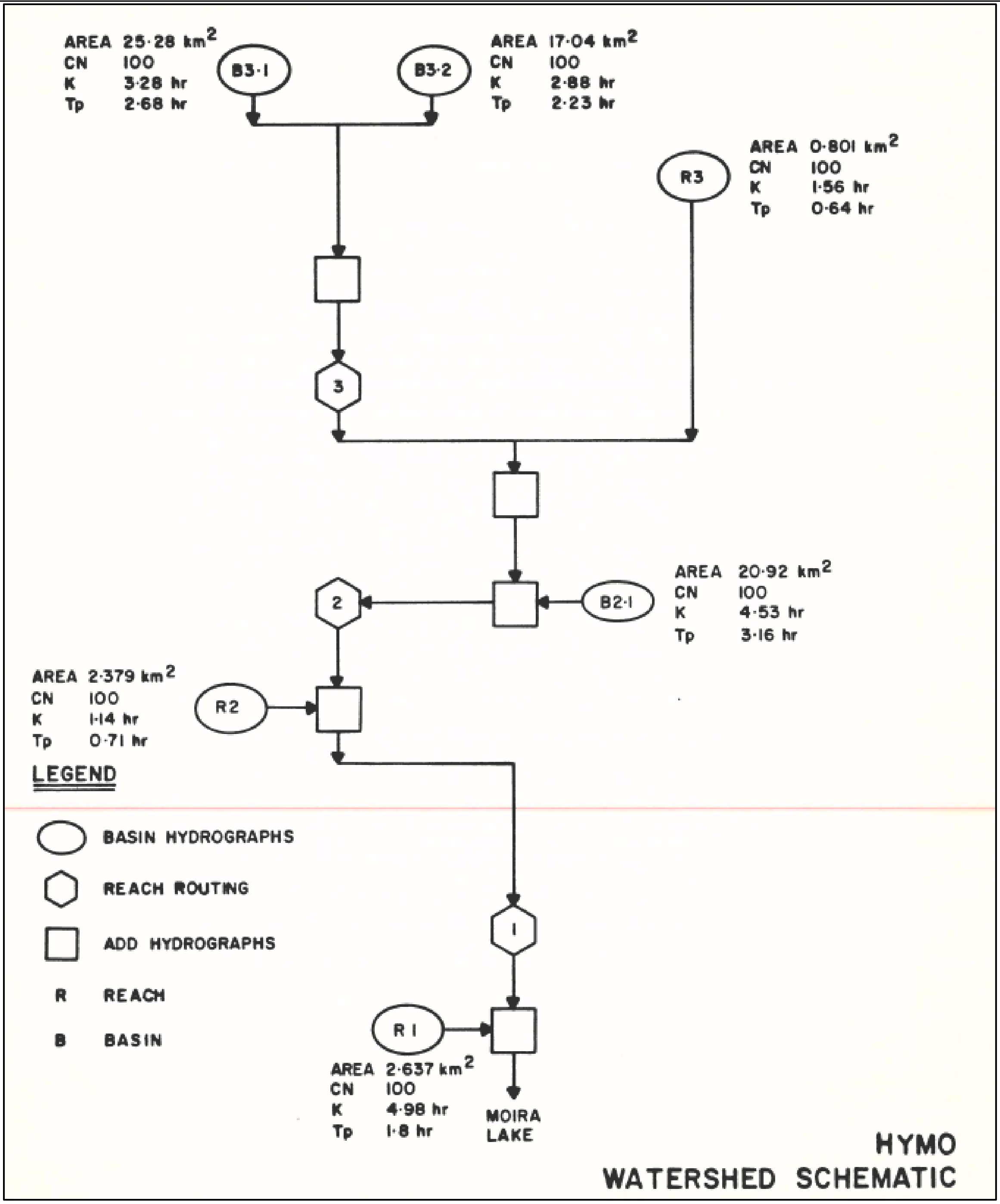

Figure 3-2: Deer Creek HYMO Model Components (Garatech Inc., 1986)

Quinte Conservation Watershed Hydrology Model (2010)

This model was built for the entire Quinte Conservation watershed using the GAWSER (Guelph All-Weather Sequential-Events Runoff model) program. Both event-based and continuous modelling approaches were employed, and the model was calibrated using several streamflow gauges. However, the hydrological model did not provide a sufficient level of detail for accurately modelling the hydrology of the Deer Creek watershed, nor did the report provide peak flow estimates for various design storms within Madoc.

4 HYDROLOGICAL MODEL

A hydrological model was created using the US Army Corps of Engineers HEC-HMS software (Ver. 4.8). A schematic of the model is presented in Error! Reference source not found.. When possible, standard hydrological p arameters were used as defined in the “Technical Guidelines for Flood Hazard Mapping” (Environmental Water Resources Group Ltd., 2017) that was prepared for Central Lake Ontario Conservation, the Grand River Conservation Authority, the Toronto and Region Conservation Authority, Credit Valley Conservation, Ganaraska Conservation, and the Nottawasaga Valley Conservation Authority.

The Deer Creek watershed is ungauged. However, baseflow data derived from manual velocity measurements was provided by Quinte Conservation. This data showed that baseflow at the downstream portion of the main branch was low (~1 m^3/s) and could be considered negligeable during a large storm. As such, baseflow was omitted from the hydrological model.

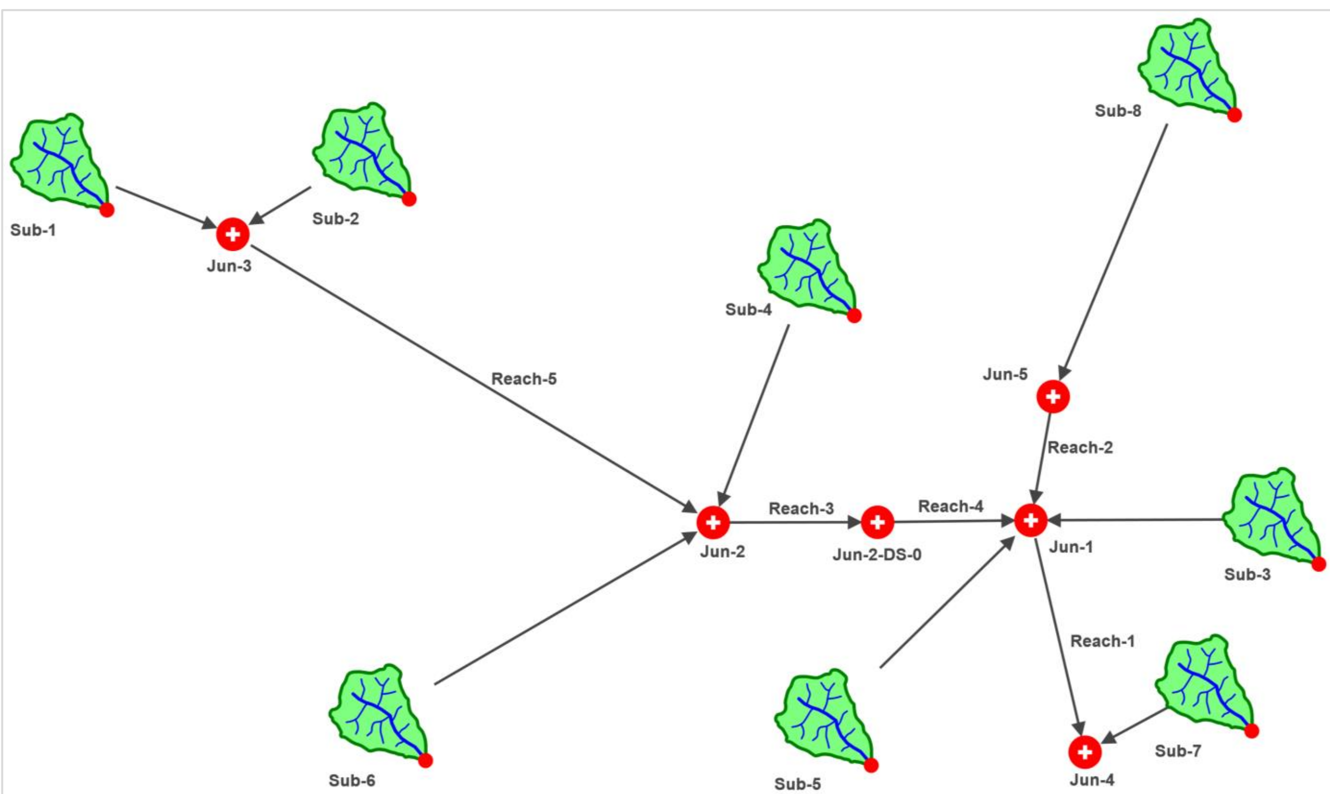

Figure 4-1: Schematic of the Deer Creek HEC-HMS Model

4.1 Design Storms

Intensity-duration-frequency (IDF) curves for the Deer Creek watershed were retrieved using the Ontario Ministry of Transportation (MTO) IDF Curve Lookup Tool (MTO, 2021). AES 30% storm distributions for Southern Ontario were applied because they are typically more accurate than SCS or Chicago distributions (OMNR, 2002). Storm duration was selected to be 12 hrs because 12-hr storms have previously been used in other studies undertaken for Quinte Conservation, such as the 2011 Bay of Quinte Regional Master Drainage Planning Project. Longer duration storms (e.g. 24 hrs) were not considered since AES storms are not defined for durations greater than 12 hrs. An areal reduction factor of 0.96 was applied to all design storms, as per the World Meteorological Organization method described in the MTO Drainage Management Manual (MTO, 1997). The intensities and associated rainfall depths for various storm durations under the 2-year, 5-year, 25-year, 50-year, and 100-year (Regulatory) storms are summarized in **Table 4-1** and **Table 4-2**. The design storm hyetographs used for hydrological modelling are shown in Appendix A.

Table 4-1: Rainfall Intensities (mm/hr) for Return Periods & Various Storm Durations

	Storm Duration				
Return period	**1-hr**	**2-hr**	**6-hr**	**12-hr**	**24-hr**
2-yr	20.3	12.5	5.8	3.6	2.2
5-yr	26.9	16.5	7.7	4.7	2.9
25-yr	36.7	22.6	10.5	6.4	3.9
50-yr	40.7	25.1	11.6	7.2	4.4
100-yr	44.7	27.6	12.8	7.9	4.9

Table 4-2: Rainfall Depths (mm) for Return Periods & Various Storm Durations

	Storm Duration				
Return period	**1-hr**	**2-hr**	**6-hr**	**12-hr**	**24-hr**
2-yr	20.3	25	34.8	42.8	52.7
5-yr	26.9	33.1	46.1	56.8	70
25-yr	36.7	45.2	62.9	77.5	95.4
50-yr	40.7	50.1	69.8	86	106
100-yr	44.7	55.1	76.7	94.6	116.4

4.2 Digital Elevation Model

LiDAR elevation data was available for the downstream study area, but not for the remainder of the watershed, which comprised the majority of the total watershed area. Wherever possible, LiDAR data was used for determining elevations and slopes of model components. In areas where LiDAR data was not available, Aquafor opted to use a 30 m hydrology-enforced digital elevation model (DEM) that was produced by the Ontario Ministry of Natural Resources and Forestry (MNRF). The DEM was created by first compiling various elevation maps throughout the province, then forcing the DEM such that flow accumulated along mapped watercourses. The enforced DEM was selected instead of MNRF's 2 m imagery-derived DEM because Aquafor's assessment of the imagery-derived DEM showed that it could not reliably predict watershed boundaries and flow accumulation.

4.3 Timestep

The control specification time step was selected to be 6 mins (0.1 hrs) because it is a relatively small timestep that can improve accuracy. The time interval for Muskingum-Cunge routing is dependent on reach index flow, which was selected as approximately the average between baseflow (assumed to be negligeable, i.e., 0 m^3/s) and peak flow within each reach, as per the HEC-HMS Technical Reference Manual (US Army Corps of Engineers, 2000).

4.4 Subbasins

Eight subbasins were created from the enforced DEM based on a stream delineation process that required a minimum drainage area of 5 km^2, as shown in **Figure 4-2**. Originally, only 7 subbasins were delineated, but the western subbasin drained by Madoc Creek was split upstream of the study area into Sub-3 and Sub-8, so as to better estimate flows entering the study area. Subbasin properties and model inputs are shown in **Table 4-3** below.

Table 4-3: Subbasin Properties and Inputs

Subbasin ID	Area (km^2)	Slope (m/m)	Initial Abstraction (mm)	Composite CN	Impervious Surface (%)	Lag Time (min)
Sub-1	5.117	0.071907	28.74	63.87	0	155.98
Sub-2	5.811	0.067197	23.87	68.03	0	55.62
Sub-3	1.069	0.031997	14.14	78.23	0	38.71
Sub-4	16.88	0.055292	25.23	66.82	0	114.41
Sub-5	1.439	0.037712	20.82	70.93	0	210.91
Sub-6	16.73	0.06284	29.58	63.20	0	107.84
Sub-7	2.618	0.095826	16.78	75.17	11.74	35.51
Sub-8	27.36	0.050658	22.05	69.73	0	252.41

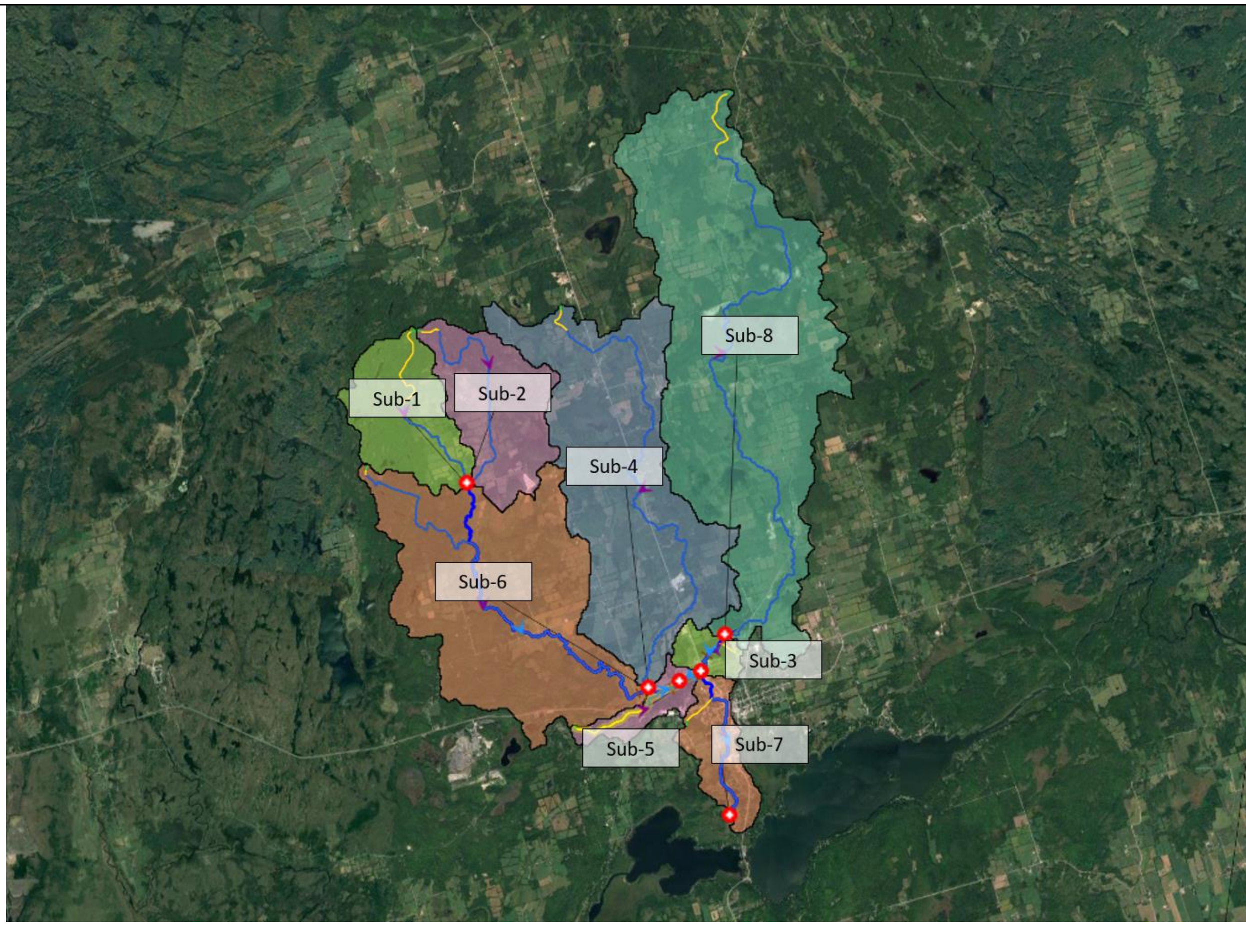

Figure 4-2: Delineated Subbasins in the HEC-HMS Model

4.5 Reach Routing

Since LiDAR cannot accurately capture stream bathymetry, the LiDAR DEM was augmented with stream topographic surveys conducted by Aquafor during the periods of October 25th-27th and November 25th-26th. This allowed for accurate cross-sections to be generated for each reach. Furthermore, in areas where LiDAR was not available, the 30 DEM could not adequately capture stream meandering; as such, reaches were manually defined to follow MNRF's mapped stream network.

Aquafor's field survey, along with a review of aerial imagery, revealed that the majority of the watershed stream floodplains consisted of riparian wetlands, which indicates that the stream banks are frequent overtopped. Because of this, it was important to capture overbank areas when routing flow, rather than assuming a simple trapezoidal channel geometry. In addition, the Deer Creek reaches are characterized by relatively low slopes, varying between 0.00055 - 0.00532 and having an average of 0.00302. Therefore, the Muskingum-Cunge routing method was selected and was applied using 8-point cross-sections that were defined for each reach. The location of routing cross-sections is shown in **Figure 4-3**, and cross-section graphs are shown in Appendix C. This approach accounts for overbank flow and is appropriate for modelling flow in subbasins with low slope (US Army Corps of Engineers, 2000, 2021). Manning's n was selected to be 0.08 for the floodplains given the dense, Typha-dominated vegetation in these areas. This is in agreement with the typical Manning's n coefficients put forth in the MTO Drainage Management Manual (MTO, 1997), which suggests values of 0.06-0.08 for light brush and trees in summer and 0.01-0.16 for medium to dense vegetation in summer. Manning's n was selected to be 0.035 for the channel, based on recommended values for open channels (**Table 4-4**).

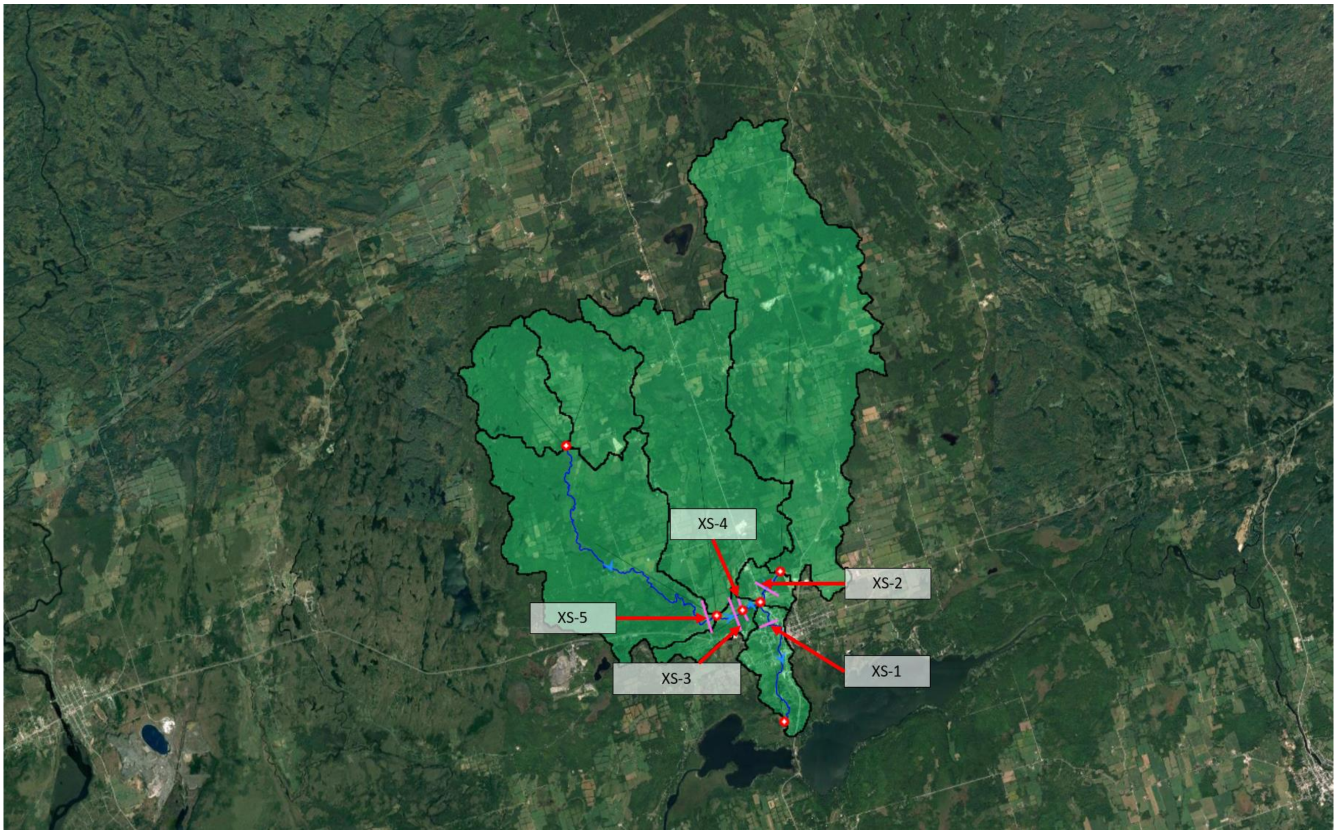

Figure 4-3: Location of Cross-Sections used for Routing Flows in the HEC-HMS Model

The locations of the reaches are shown in **Figure 4-4** below and reach profile plots are shown in Appendix D. A single reach was defined for each downstream subbasin, except for Sub-5, for which 2 reaches were defined. This was necessary because the upstream portion of Sub-5 was characterized by wide floodplains (~55 m along each bank) and a low slope (0.00055), whereas the downstream section had smaller floodplains (~25 m along the left bank, 0 m along the right bank) and a higher slope (0.00088). A summary of reach parameters is provided in **Table 4-5**.

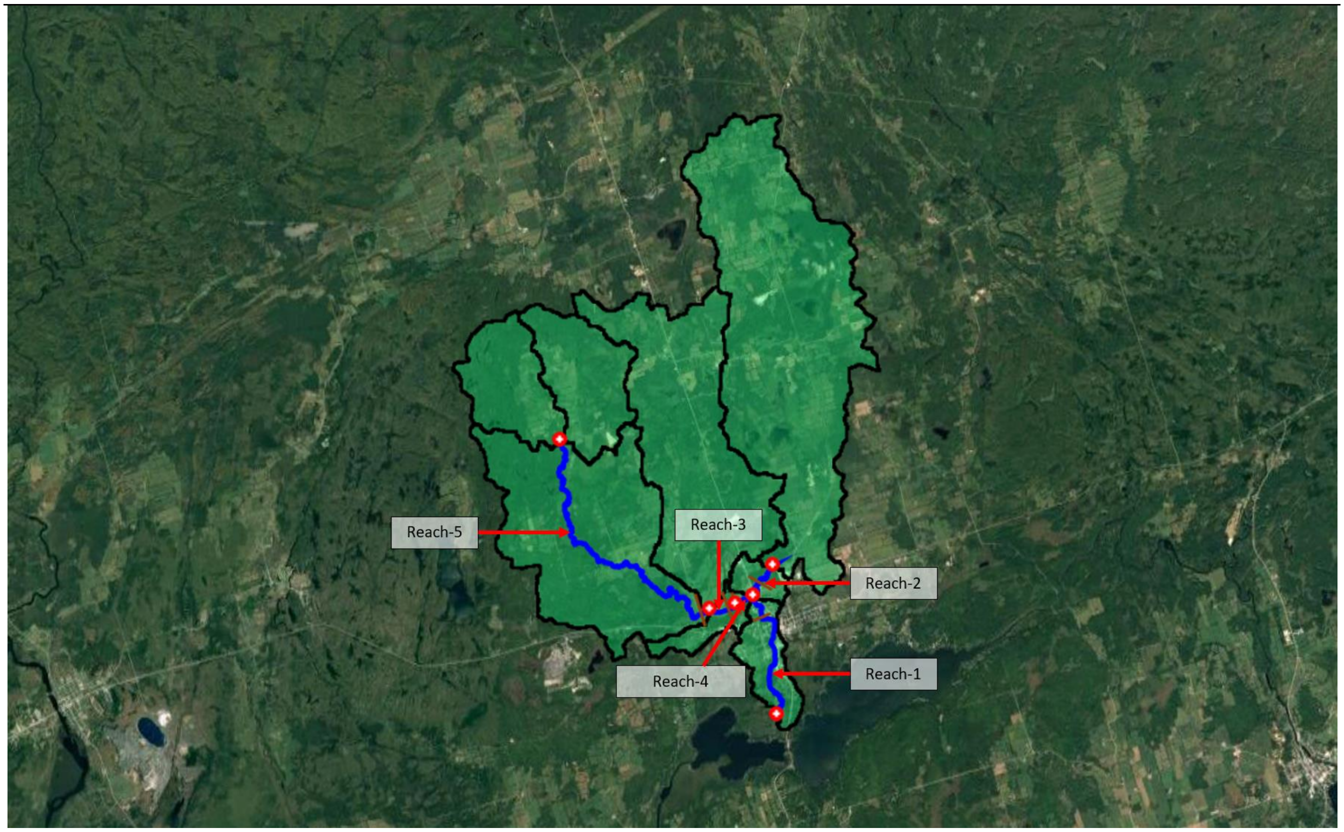

Figure 4-4: Delineated Reaches in the HEC-HMS Model

A small weir/drop structure with no retention capacity is located on Deer Creek, downstream of the Russel Street crossing, within the Town of Madoc (**Figure 4-5** and **Figure 4-6**). The structure was not considered when routing flows in the hydrologic model, because it is not deemed to have a significant impact on flow for large events, including the 2-year event. In addition, most routing methods, including Muskingum-Cunge, cannot account for complex elements such as control structures (US Army Corps of Engineers, 2000).

Table 4-4: Standard Manning Roughness Coefficients for Open Channels (from Environmental Water Resources Group Ltd., 2017)

Land cover	Standard 'n' Value
Overbank	
Woods	0.080
Meadows	0.055
Lawns	0.045
Wetlands*	0.080
Channel	
Natural	0.035
Grass	0.030
Natural Rock	0.035
Armour Stone	0.025
Concrete	0.015
Articulated Block	0.020
Gabions	0.025
Woods	0.012

*Modified by Aquafor as per the MTO Drainage Management Manual (MTO, 1997)

Table 4-5: Reach Properties

Reach ID	Length (m)	Slope (m/m)
Reach-1	3639.17	0.00382
Reach-2	1196.23	0.00452
Reach-3	686.86	0.00055
Reach-4	547.15	0.00087
Reach-5	8539.21	0.00532

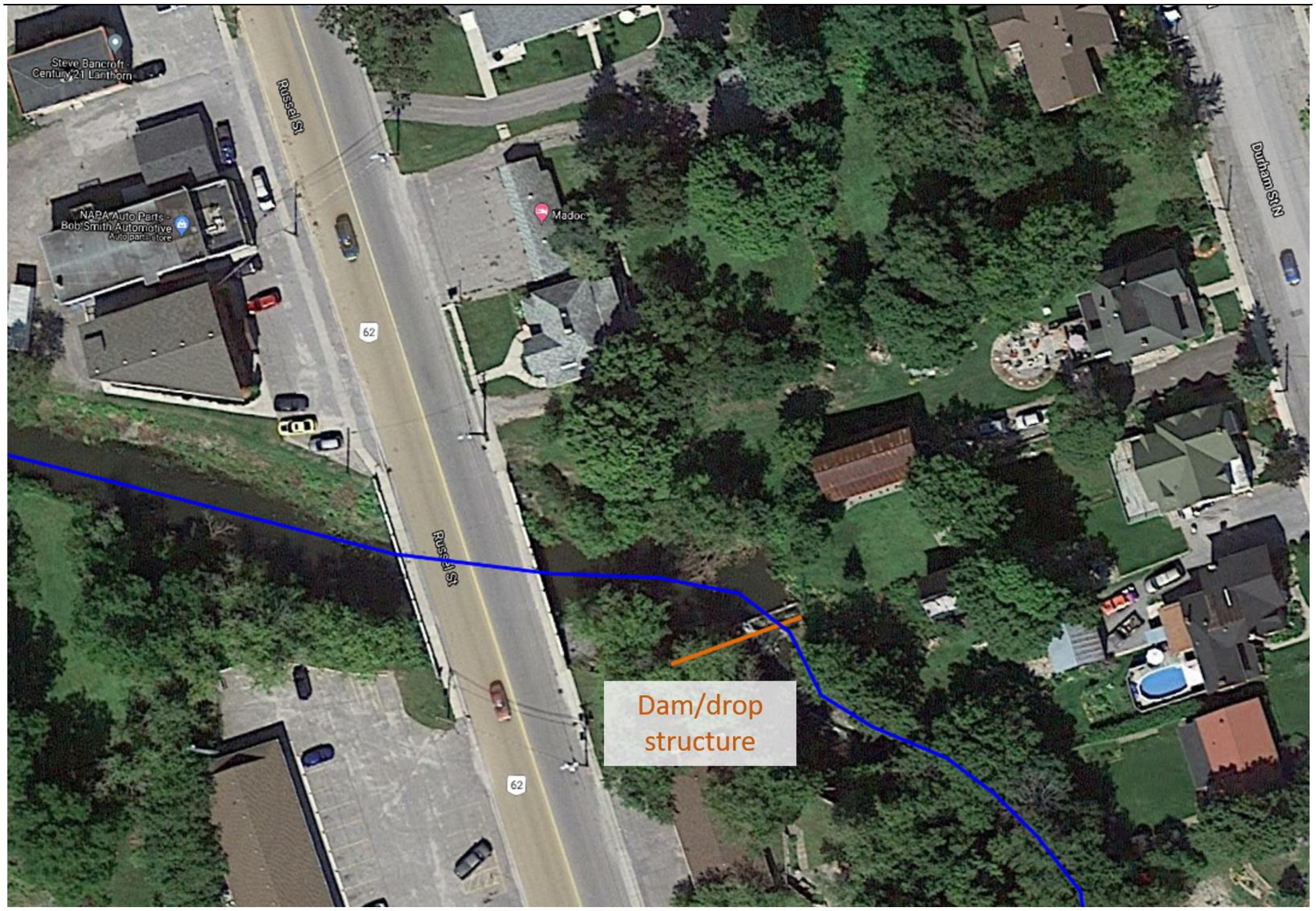

Figure 4-5: Location of Weir/Drop Structure Downstream of Russel Street

Figure 4-6: Photograph of the Weir/Drop Structure

4.6 Transform Method and Lag Time

The SCS unit hydrograph transform method was selected. Lag time, t_{lag} was calculated from time of concentration, t_c, using the equation:

$$t_{lag} = 0.6t_c$$

As per the recommendations in the United States Department of Agriculture (USDA) TR-55 report (USDA, 1986), time of concentration was calculated for each subbasin as the sum of travel times from sheet flow, shallow concentrated flow, and channel flow along the longest flowpath. Sheet flow length was estimated using the McCuen-Spiess equation, and travel time was estimated using the Manning's roughness values shown in **Table 4-6** that were recommended by Environmental Water Resources Group Ltd. (2017). Channel length coinciding with the longest flowpath was determined using the MNRF's stream network, and travel time was estimated by assuming using a natural channel roughness of 0.035, as per **Table 4-4**. Shallow concentrated flow length was calculated by subtracting channel length and sheet flow length, and travel time was estimated based on land use.

Table 4-6: Standard Manning Roughness Coefficients for Overland Flow (from Environmental Water Resources Group Ltd., 2017)

Land cover	Standard ‘n’ Value
Impervious	0.013
Lawns	0.250
Cultivated	0.300
Meadows	0.350
Woods	0.600

4.7 Runoff Calculation

The SCS curve number method for infiltration loss was adopted for estimating runoff. Typical CN values proposed by Environmental Water Resources Group Ltd. (2017), as shown in **Table 4-7**, were used for most land use types. The Ontario Land Classification layer was updated by Aquafor to include urbanization within Madoc. Coverage from roads was determined using the Ontario Road Network data, assuming an average road width of 8 m. Residential lots were identified based on land parcels and building points, from which the lots were classified according to size and assigned CN values (**Table 4-8**) as per TR-55 (USDA, 1986). Other impervious areas, such as large parking lots and buildings, were manually identified, along with a quarry located north of Madoc. Additional CN values were defined for wetlands and are recorded in **Table 4-8**. Missing soil and land use coverage, usually located within wetlands, was manually gap-filled based on data from the surrounding areas.

A single composite CN values was determined for each subbasin by calculating the weighted average of CN values, based on the area occupied by each CN value. Impervious areas located within Madoc were considered to be directly connected to the stream via storm drains and were not included in the calculation of composite CN values. Outside of Madoc, impervious areas were included in the calculation of composite CN values and were not considered to be directly connected.

The initial abstraction, I_a [mm], was calculated as:

$$I_a = 0.2S$$

where S [mm] is the maximum retention defined by:

$$S = \frac{25400 - 254CN}{CN}$$

Variations in CN based on antecedent moisture content was also taken into account, using **Table B.1** (Appendix B) provided in the report by Environmental Water Resources Group Ltd. (2017).

Table 4-7: Standard SCS Curve Numbers for each Land Use Type and Soil Hydrological Group (from Environmental Water Resources Group Ltd., 2017)

Land use	A	B	C	D
Woods	32	60	73	79
Meadows	38	65	76	81
Cultivated	62	74	82	86
Lawns	49	69	79	84
Impervious Areas	100	100	100	100

Table 4-8: Other SCS Curve Numbers for each Land Use Type and Soil Hydrological Group.

Land use	A	B	C	D
Residential Lot Size < 1 Acre and >1/4 Acre*	56	72	81	86
Residential Lot Size < 1/4 Acre and > 1/8 Acre*	69	80	87	90
Residential Lot Size < 1/8 Acre*	77	85	90	92
Bedrock	80	80	80	80
Bog	10	10	10	10
Fen	10	10	10	10
Marsh	80	80	80	80
Commercial*	89	92	94	95

*Values obtained from the TR-55 report (USDA, 1986)

4.8 Hydrological Model Results

For average antecedent moisture conditions, 100-year storm event produced a peak flow of 75.5 m^3/s downstream of Madoc at Jun-4 (**Figure 4-7**). **Table 4-9** provides a detailed summary of peak flows encountered throughout the watershed under the 2-year, 5-year, 25-year, 50-year, and 100-year events, for each antecedent moisture condition. The highest peak flow was found to be 141.6 m^3/s and occurs during the 100-year storm event at Jun-4 (downstream of Madoc) under wet antecedent conditions. Peak flows for each junction under average antecedent moisture conditions for the 100-year event are shown graphically in **Figure 4-8**. The peak flows at Jun-4 (downstream of Madoc) are plotted in **Figure 4-9**.

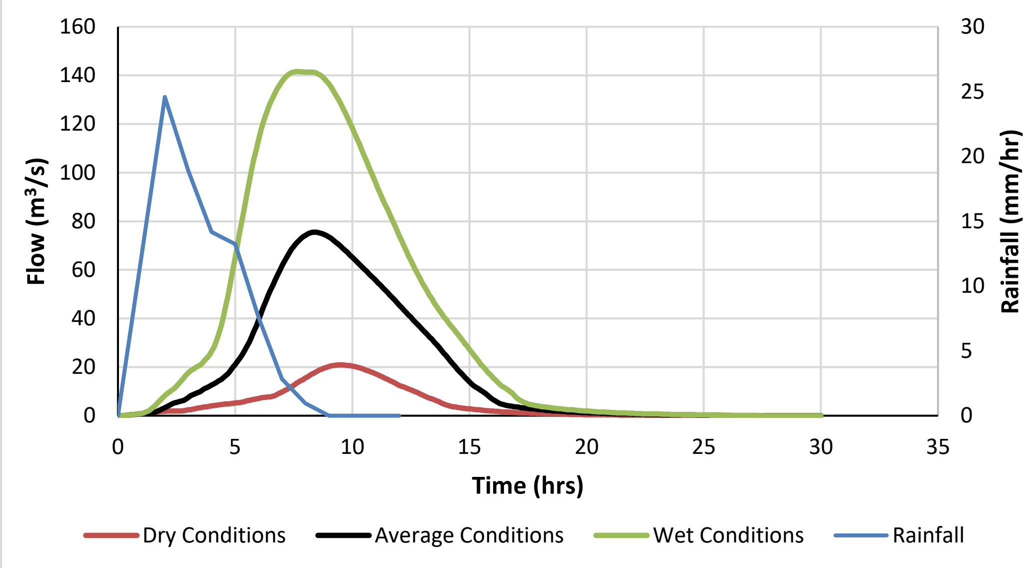

Figure 4-7: Hydrograph at Jun-4 (Downstream of Madoc) for the Average Moisture Condition, under the 100-yr Design Storm

Table 4-9: Maximum Inflow at Each Junction and for Each Antecedent Moisture Condition, under the 100-yr, 50-yr, 25-yr, 5-yr, and 2-yr Design Storms

	100-yr	50-yr	25-yr	5-yr	2-yr
Dry Antecedent Conditions					
Jun-1	21	14.1	8.2	0.6	0.1
Jun-2	14.1	9	4.9	0.2	0
Jun-2-DS-0	12.7	7.9	4.1	0.2	0
Jun-3	3.5	2.3	1.4	0.1	0
Jun-4	20.9	14	8.2	1.3	0.9
Jun-5	8.8	6.1	3.8	0.4	0
Average Antecedent Conditions					
Jun-1	75.2	61.8	49.2	23.4	8.7
Jun-2	50	41.3	33.1	16	5.4
Jun-2-DS-0	47.8	39.4	31.5	14.8	4.7
Jun-3	14.6	11.8	9.3	4	1.5
Jun-4	75.5	61.9	49.2	23.3	8.7
Jun-5	31.9	26.2	20.8	9.6	3.9
Wet Antecedent Conditions					
Jun-1	140.8	121.8	103.6	62.3	38.8
Jun-2	91.2	79.2	67.5	40.8	25.8
Jun-2-DS-0	88.7	76.9	65.4	39.3	24.8
Jun-3	27.5	23.8	20.2	11.9	6.8
Jun-4	141.6	122.7	104.2	62.4	38.8
Jun-5	59.2	51.4	43.7	26.2	15.6

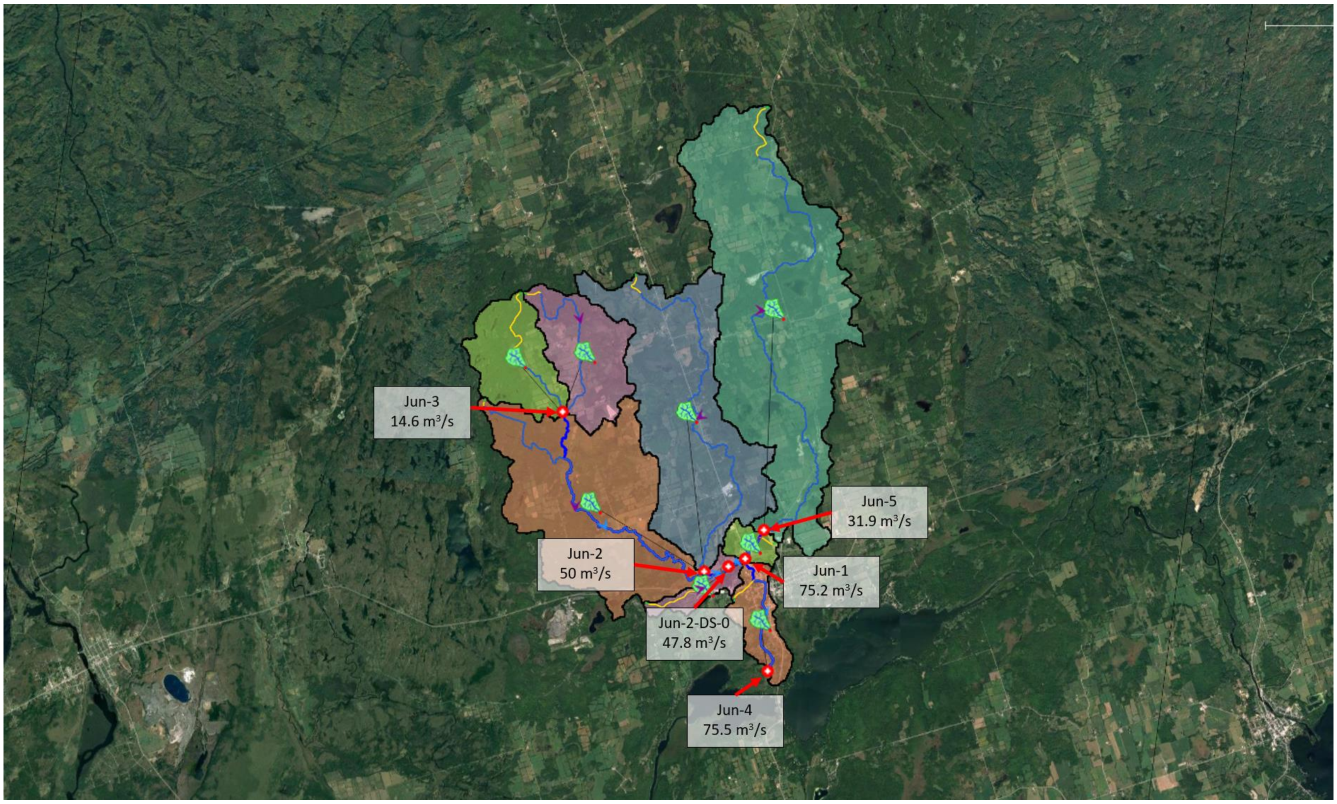

Figure 4-8: Peak Flows at each Junction for the 100-year Event under Average Antecedent Moisture Conditions

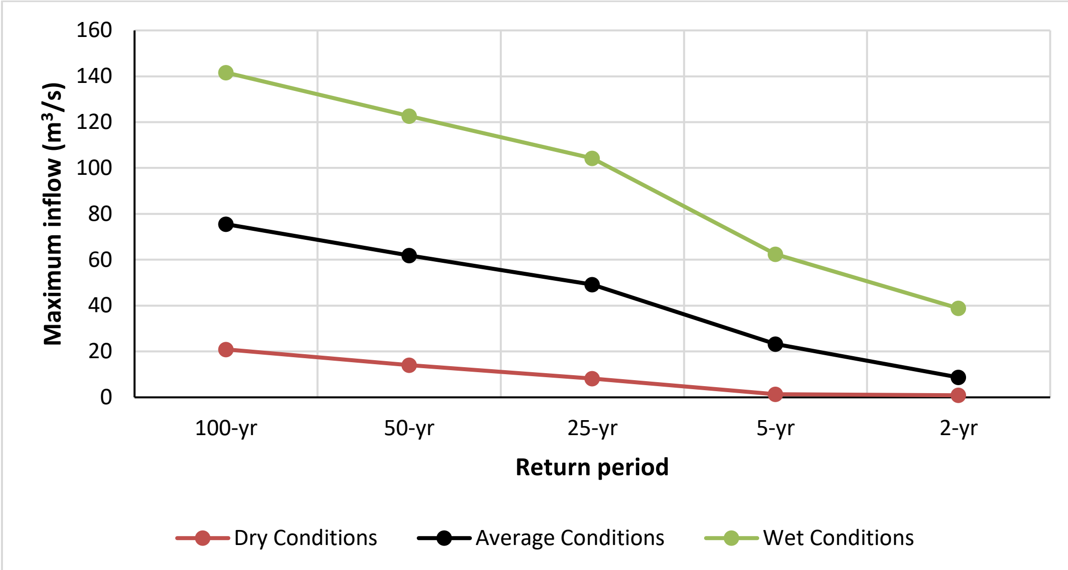

Figure 4-9: Maximum Inflow at Jun-4 (Downstream of Madoc) for Each Antecedent Moisture Condition, under the 100-yr, 50-yr, 25-yr, 5-yr, and 2-yr Design Storms

The peak flow of 75.5 m^3/s downstream of Madoc at Jun-4 under the 100-year event and average antecedent moisture conditions is similar to the peak flow of 82.26 m^3/s currently used by Quinte Conservation to define the regulatory/100-year flooding extents, which was determined in the 1982 Water Management Study for Deer Creek. Aquafor recommends using the values determined for average antecedent moisture conditions when undertaking hydraulic modelling for Deer Creek in Madoc and updating regulatory floodlines. The peak flow values estimated by Aquafor downstream of Madoc under the 100-year, 50-year, 25-year, 5-year, and 2-year events are compared against the peak flow values estimated by the HYMO model in the 1982 Water Management Study in **Table 4-10**. It should be noted that although the general setup of Aquafor's model and the 1986 HYMO model are similar, several key differences exist between the two models: HYMO is a synthetic unit hydrograph method, in contrast to the SCS unit hydrograph used by Aquafor; the HYMO model used a 7-day snowmelt event for modelling 100-year peak flows instead of a 12-hour rainfall event; runoff was calculated based on soil infiltration rates and storage instead of SCS curve numbers; and the HYMO model did not incorporate any attenuation or diffusion processes when routing flows.

Table 4-10: Comparison of Peak Flows between Aquafor's Model Outputs and the 1982 HYMO model Outputs for the 100-yr, 50-yr, 25-yr, 5-yr, and 2-yr events, Downstream of Madoc

	100-yr	50-yr	25-yr	5-yr	2-yr
Aquafor's Estimates	75.5	61.9	49.2	23.3	8.7
1982 HYMO Model Estimates	82.26	71.95	61.62	37.24	21.11

4.9 Future Land Use

Polygons showing future urban land use were obtained from Hastings County. Aquafor simplified the land use layer and removed environmental protection areas, such that all future land use was classified as either commercial, residential, or open space areas (**Figure 4-10**). All future development is planned to be within

Subbasins 3, 5, and 7. Sub-basin properties for the Future Land Use Scenario are shown in **Table 4-11**. It should be noted that the fraction of impervious surface did not change, since imperviousness was accounted for in the commercial and residential land use CN values provided in the TR-55 report (USDA, 1986). The Future Land Use Scenario was run using the 12-hr AES rainfall distribution under average antecedent moisture conditions for the 2-year, 5-year, 5-year, 50-year, and 100-year rainfall events. Peak flows at each junction are recorded in **Table 4-12**. The effect of future developments in the watershed had virtually no effect on peak flow, which increased by no more than 0.133% at any given junction. This is because the increase in CN value was small or nil in all subbasins and because proposed future land development is located within a relatively small area at the downstream extents of the watershed, meaning that the majority of excess runoff attributed to increased urbanization would discharge into Moira Lake before runoff from the large upstream rural areas arrived downstream.

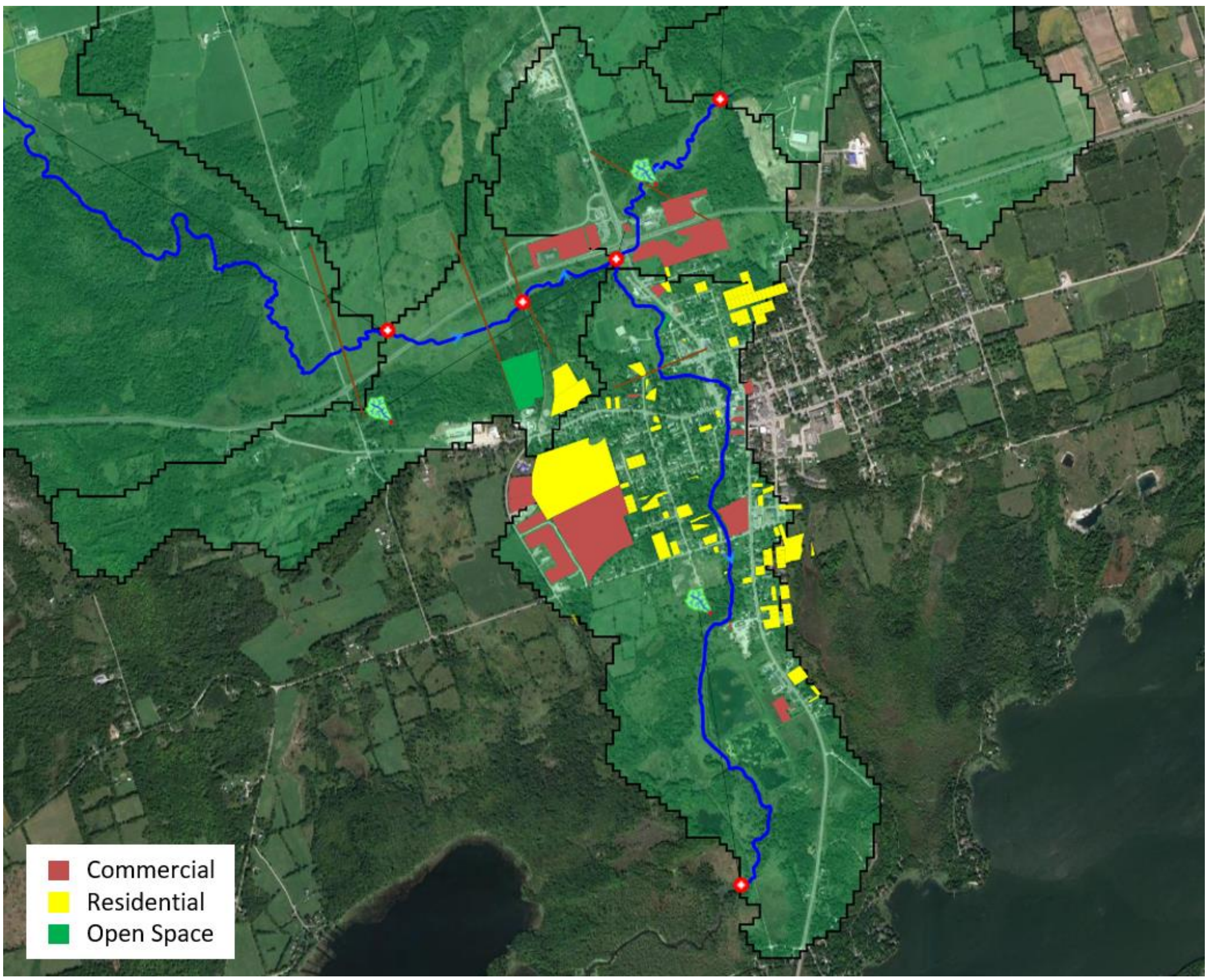

Figure 4-10: Future Land Use Classification

Table 4-11: Sub-basin Properties for the Future Land Use Scenario

Subbasin ID	Initial Abstraction (mm)	Composite CN	Impervious Surface (%)
Sub-1	28.74	63.9	0
Sub-2	23.87	68	0
Sub-3	13.3	79.25	0
Sub-4	25.23	66.8	0
Sub-5	20.39	71.36	0
Sub-6	29.58	63.2	0
Sub-7	14.43	77.88	11.7
Sub-8	22.05	69.7	0

Table 4-12: Comparison Peak Flows for the Current and Future Land Use Scenarios under the 100-year Event

Junction ID	Current Land Use Peak Flow (m^3/s)	Future Land Use Peak Flow (m^3/s)	Peak Flow Change (%)
Jun-1	75.2	75.3	0.133
Jun-2	50	50	0
Jun-2-DS-0	47.8	47.8	0
Jun-3	14.6	14.6	0
Jun-4	75.5	75.6	0.132
Jun-5	31.9	31.9	0

4.10 Climate Change

The impact of climate change on peak flows were evaluated for the Future Land Use scenario under the 100-year event, using the Coupled Model Intercomparison Project Phase 6 (CMIP6) global climate model and assuming a moderate warming scenario (SSP2-4.5). The IDF curve based on historical data and the projected IDF curve under the SSP2-45 climate change scenario were extracted for Madoc using the IDF_CC Tool 6.0 (Western University, 2021). By comparing these IDF curves, it was determined that moderate climate change is expected to increase the 100-year rainfall volume by 9.6%. As such, for modelling climate change, rainfall for the 100-year storm was increased to 107.9 mm. As shown in **Table 4-13**, climate change increased 100-year peak flows by approximately 20% at all junctions.

Table 4-13: Comparison of 100-year Peak Flows under Historical Climate and Climate Change Scenarios, for Future Land Use

Junction ID	Historical Climate Peak Flow (m^3/s)	Climate Change Peak Flow (m^3/s)	Peak Flow Change (%)
Jun-1	75.3	90.2	19.79
Jun-2	50	59.8	19.6
Jun-2-DS-0	47.8	57.3	19.87
Jun-3	14.6	17.6	20.55
Jun-4	75.6	90.7	19.97
Jun-5	31.9	38.2	19.75

5 MODEL EVALUATION

Because the Deer Creek watershed is ungauged, it is not possible to calibrate the hydrological model. This section aims to assess the validity and uncertainty of the model by comparing model results against estimates from peak flow analyses, undertaking a sensitivity analysis, and reviewing HEC-HMS warning messages.

5.1 Peak Flow Frequency Analyses

As a point of comparison, a Regional Flood Frequency Analysis was performed using the Modified Index Flood Method, as described in the MTO Drainage Management Manual (MTO, 1997). This manual provides design charts that can be used to calculate an estimated 25-year Index Flood, Q_{25} [m^3/s], using the equation:

$$Q_{25} = C_{25} \times A^{0.75}$$

where C_{25} is a watershed class coefficient for the 25-year Index Flood and A [km^2] is watershed area.

The Deer Creek watershed has an area of 77.03 km^2 and an average CN value of 67. It is a Southern Ontario type basin because it is dominated by agricultural land use, with medium to sandy loam and little lake coverage. Using the Ontario Flow Assessment Tool, the watershed was determined to have an average slope of 0.05662 m/m and a storage fraction of 11%, which includes both wetlands and open water. From the design charts, it was determined that the base watershed class is 7.2 based on CN value, the slope adjustment is 2.2 (the maximum adjustment value for slope), and the storage adjustment value is -1.3. This yields a net watershed class of 8.1, corresponding to a C_{25} value of 1.90.

Therefore, the 25-year Index Flood value is:

$$Q_{25} = 1.9 \times 77.03^{0.75} = 49.5 \text{ m}^3/\text{s}$$

This estimate is very similar to the modelled 25-year peak flow of 49.2 m^3/s under average moisture conditions, thereby lending confidence to Aquafor's proposed peak flow for the 25-year flood. However, it should be noted that index flows rely only on generalized empirical relationships and do not account for certain factors such as

IDF data, basin shape, reach cross-sectional geometry, and Manning's roughness. As such, Regional Flood Frequency Analysis should only be considered as a rough estimate of peak flow values.

A single-station frequency analysis was also performed for the flow gauge on Moira River near Deloro (02HL005). Maximum instantaneous flow values were extracted for 46 years between 1966 and 2019. The data was ranked from highest to lowest flow, then the estimated return period, T_{est}, was calculated as:

$$T_{est} = \frac{n + 0.12}{m - 0.44}$$

where n is the total number of years of record and m is rank. Theoretical return period under the Gumbel distribution was then calculated. Peak flow was plotted against both estimated and theoretical return periods using a log scale on the x-axis (**Figure 5-1**). A logarithmic curve of best fit was fitted to the "straight" portion of the theoretical data, in order to estimated the 100-year peak flow. Using the equation of the curve of best fit, 100-year peak flow at the 02HL005 station was calculated to be:

$$Q_{100,Moira} = 8.4019 \times \ln(100) + 28.591 = 67.28 \ \mathrm{m^3/s}$$

In order to estimate the 100-year peak flow along Deer Creek based on the 100-year expected peak flow at the 02HL005 station on Moira River, the following equation can be applied:

$$Q_{100,Deer} = Q_{100,Moira} \times \frac{C_{25,Deer}}{C_{25,Moira}} \times \left(\frac{A_{Deer}}{A_{Moira}}\right)^{0.75}$$

This requires $C_{25,Moira}$ to be determined. The portion of the Moira River watershed that discharges to the 02HL005 station has an area of 297.16 km^2. Using the Ontario Flow Assessment Tool, the watershed was determined to have a lake storage fraction of 3.6%. It is a Northern Ontario (Canadian Shield) type basin because the primary soil type is rockland, with lakes comprising more than 3% of the total area. From Northern Basin design chart, it was determined that the base watershed class is 5.9, based on storage fraction. No adjustments were deemed to be necessary, yielding a C_{25} value of 0.87 for Moira River near Deloro. Therefore, the peak flow along Deer Creek can be estimated from $Q_{100,Moira}$ as:

$$Q_{100,Deer} = 67.28 \times \frac{1.90}{0.87} \times \left(\frac{77.03}{297.16}\right)^{0.75} = 53.38 \ \mathrm{m^3/s}$$

This prorated estimate is less than the modelled 100-year peak flow of 75.5 m^3/s under average moisture conditions, but is greater than the modelled 100-year peak flow of 20.9 m^3/s under dry moisture conditions. Again, this supports Aquafor's range of 100-year peak flow values, but the determination of the prorated value does not take into account all factors influencing the hydrology of the Moira River and Deer Creek watersheds.

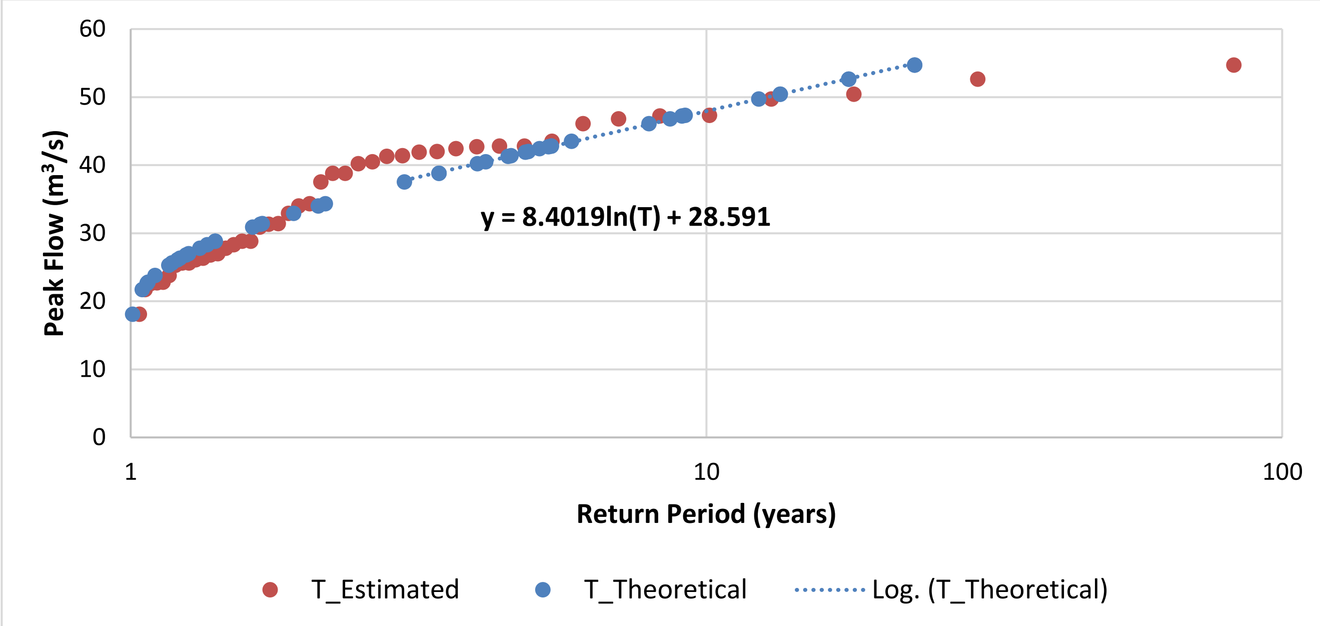

Figure 5-1: Peak Flow as a Function of Estimated and Theoretical Return Period

5.2 Sensitivity Analysis

In addition to the evaluation of the model under various antecedent moisture conditions (**Section 4.8**), the effects of varying impervious fraction and lag time were assessed. Lag time was varied by multiplying the base lag time for each sub-basin by a certain factor; the results for the 100-year flood event are summarized in **Table 5-1** and shown in **Figure 5-2**. Impervious fraction was varied by adding or subtracting a certain amount from each sub-basin instead of multiplying by a factor, since only Sub-7 had a non-zero impervious fraction; the results are summarized in **Table 5-2** and shown in **Figure 5-3**. It should be noted that any negative impervious fractions were set to zero when performing the sensitivity analysis.

The sensitivity analysis revealed that decreasing lag time has a high impact on model results, increasing the downstream 100-year peak flow by 33.1% when sub-basin lag times were multiplied by a factor of 0.5. The impacts of decreasing lag time and increasing imperviousness had more moderate impacts on the results, influencing peak flow by ~20%, while decreasing imperviousness had no effect. From **Section 4.8**, it can be seen that antecedent moisture conditions had the largest impact on peak flows, which were found to range from 20.9 m^3/s (-72.3% compared to average moisture conditions) to 141.6 m^3/s (+87.6%).

Table 5-1: Impact of Varying Lag Time on 100-year Peak Flows at Jun-4 under Average Moisture Conditions

Lag Multiplication factor	Peak Flow (m^3/s)	Peak Flow Increase (%)
0.5	100.5	33.1
0.75	86.8	15
1	75.5	0
1.25	66.3	-12.2
1.5	59.2	-21.6

Table 5-2: Impact of Varying Impervious Fraction on 100-year Peak Flows at Jun-4 under Average Moisture Conditions

Change in Impervious Fraction (%)	Peak Flow (m^3/s)	Peak Flow Increase (%)
-10	75.5	0
-5	75.5	0
0	75.5	0
5	82.8	9.7
10	90.3	19.6

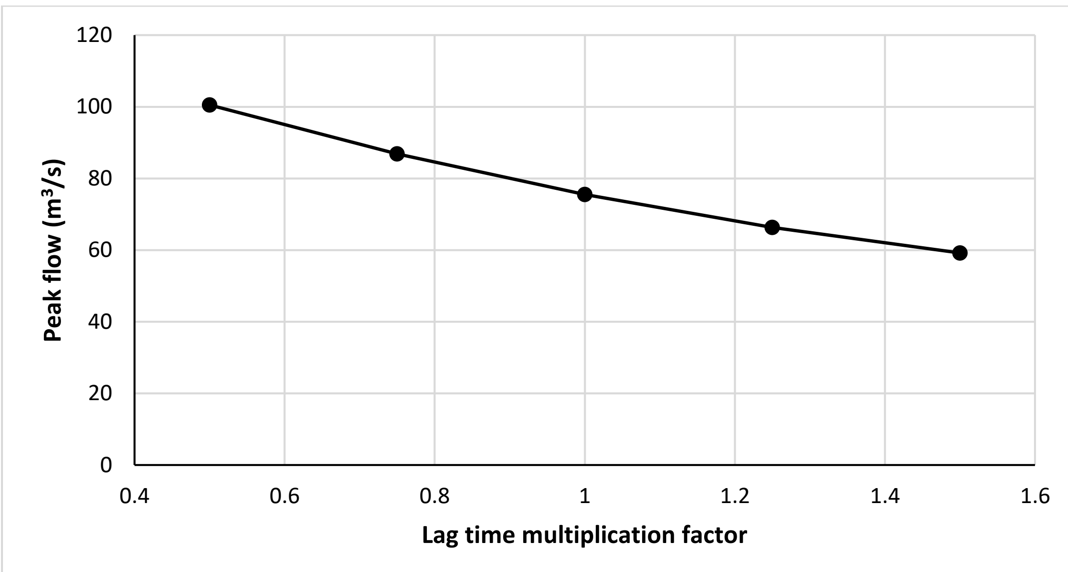

Figure 5-2: Plot of 100-year Peak Flows at Jun-4 as a Function of Lag Time under Average Moisture Conditions

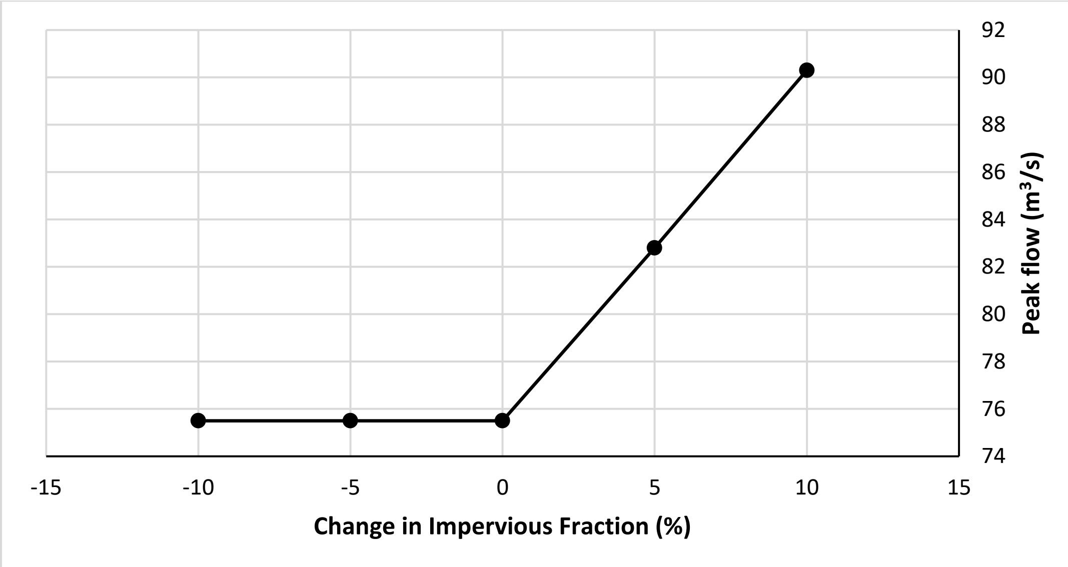

Figure 5-3: Plot of 100-year Peak Flows at Jun-4 as a Function of Impervious Fraction under Average Moisture Conditions

5.3 Warning Messages

Seven warning messages were produced when running the HEC-HMS model: three of these warned that using index flows for determining spatial steps would have increased the number steps, while the remaining four messages warned that small convergence errors (<0.1 m^3/s) existed at all reaches. The uncertainty associated with these warnings is not expected to have a significant impact on model results. Warning messages received for the 100-year storm event under average moisture conditions are listed as an example in Appendix E.

5.4 Model Acceptability

In the absence of a flow gauge, Aquafor undertook a rigorous approach to estimating peak flows along Deer Creek. This included: ensuring that streams were accurately represented when estimating reach length and time of concentration; defining 8-point cross-sections for each reach based on LiDAR and survey data in order to accurately model flow routing; calculating time of concentration as a sum of overland flow, shallow concentrated flow, and stream flow travel times; and undertaking peak flow frequency analyses to help evaluate the validity of the model. Considering these precautions and the absence of significant warning messages, Aquafor is confident that the model can reliably simulate flows to be used in the hydraulic model.

6 REFERENCES

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APPENDIX A

Design Storm Hyetographs

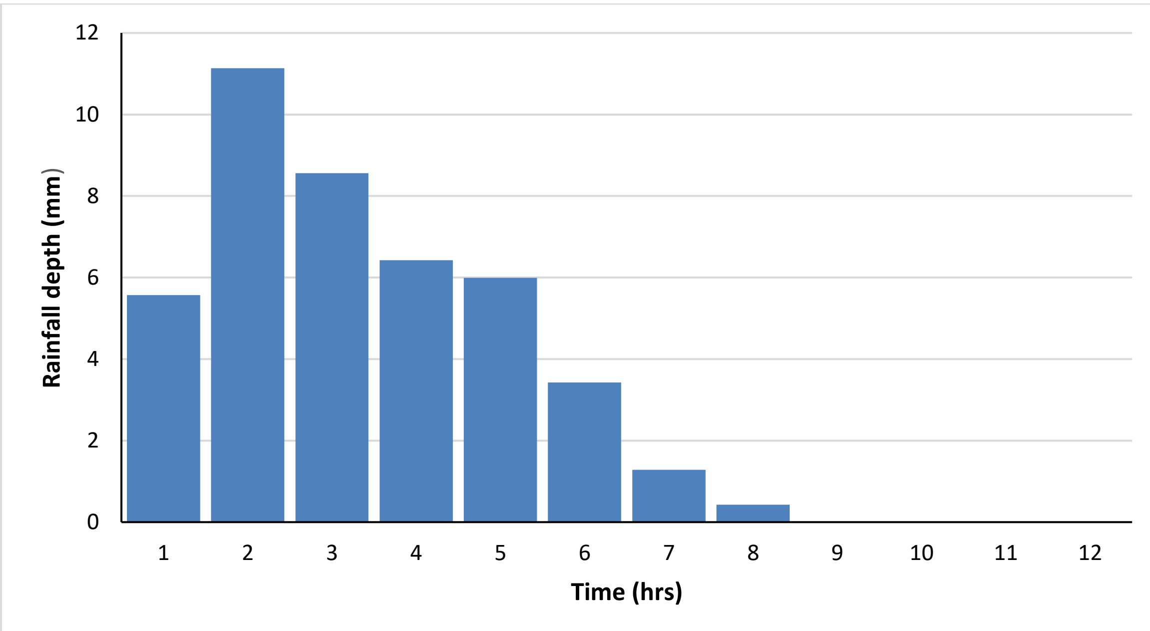

Figure A.1: Hyetograph for the 2-yr, 12-hr AES 30% Design Storm for Southern Ontario

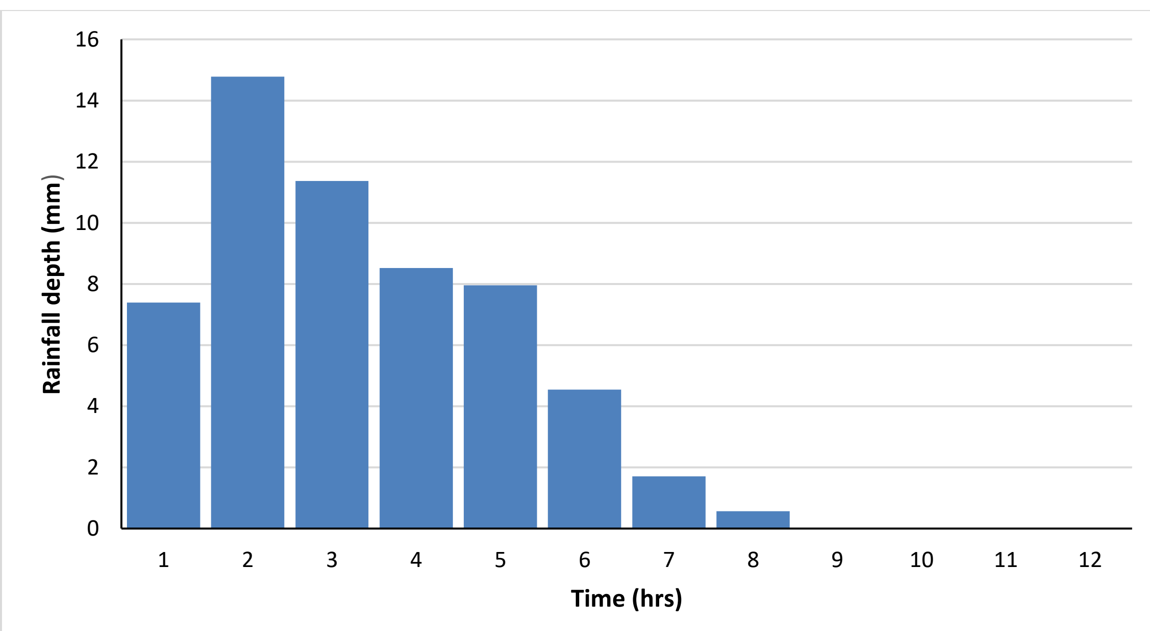

Figure A.2: Hyetograph for the 5-yr, 12-hr AES 30% Design Storm for Southern Ontario

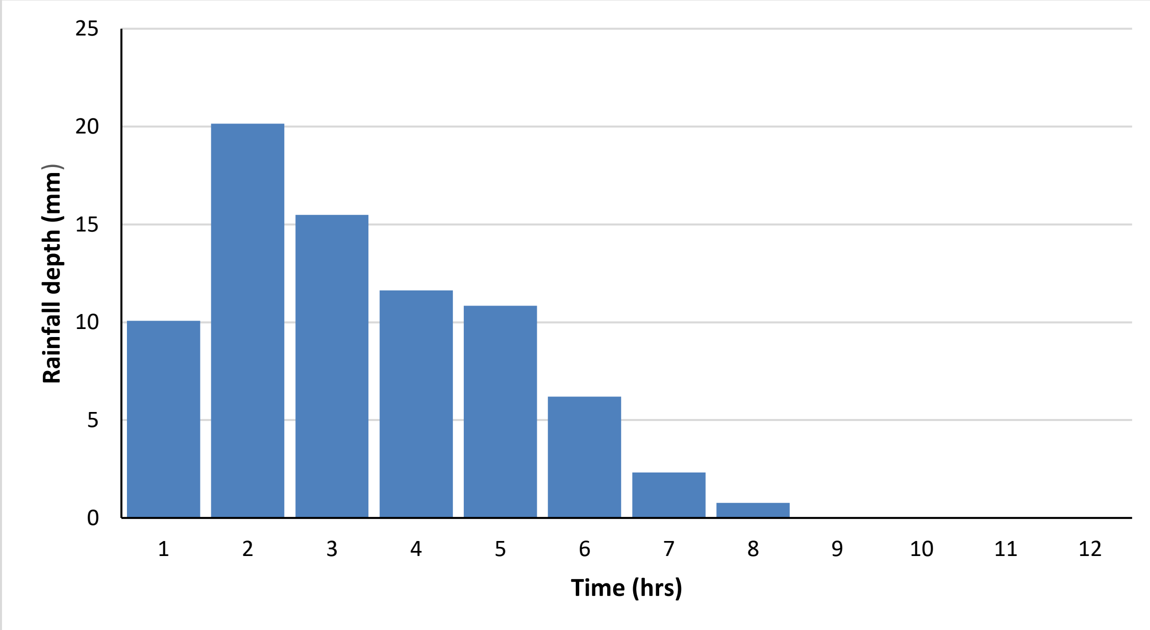

Figure A.3: Hyetograph for the 25-yr, 12-hr AES 30% Design Storm for Southern Ontario

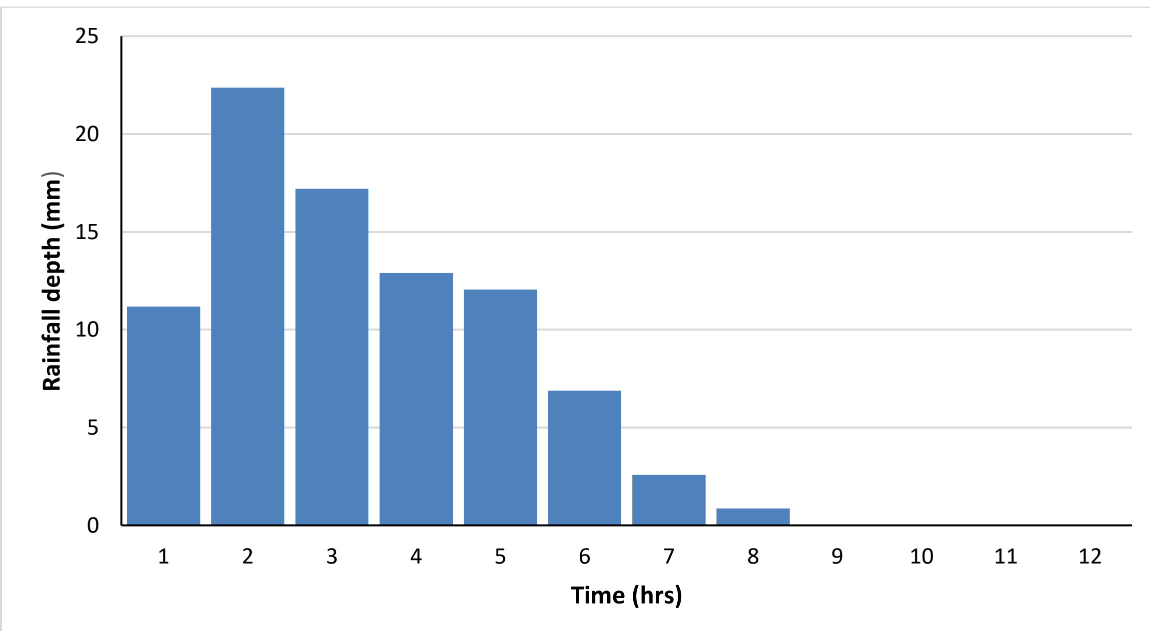

Figure A.4: Hyetograph for the 50-yr, 12-hr AES 30% Design Storm for Southern Ontario

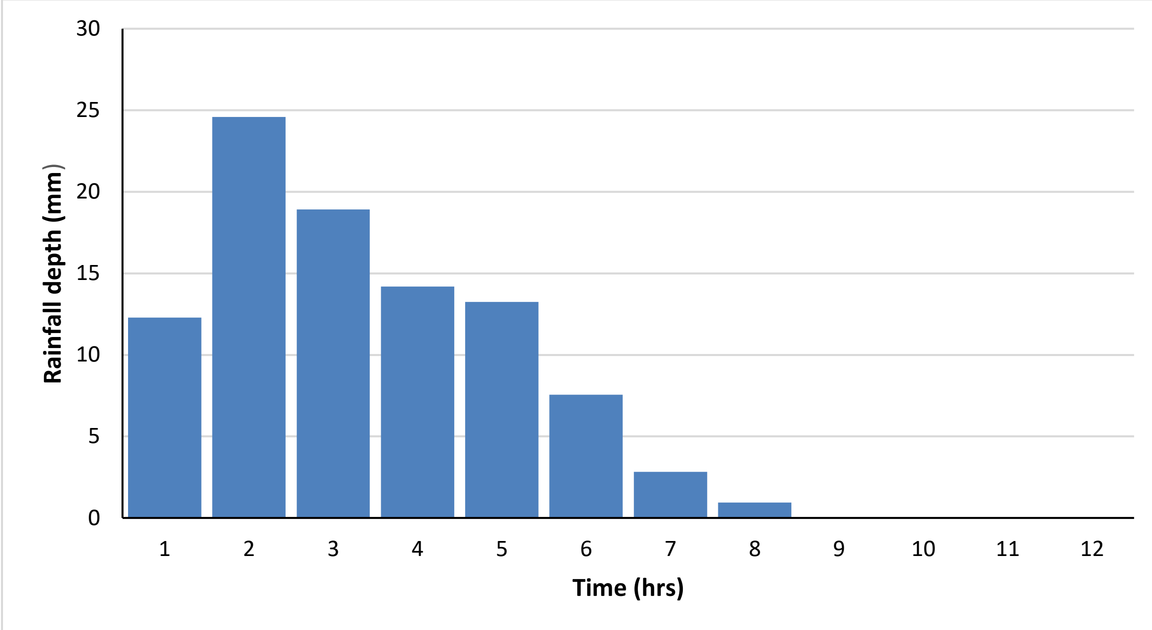

Figure A.5: Hyetograph for the 100-yr, 12-hr AES 30% Design Storm for Southern Ontario

APPENDIX B

SCS Curve Number Variation Based on Antecedent Moisture Conditions

Table B.1: Variation in SCS Curve Number based on Antecedent Moisture Conditions (from Environmental Water Resources Group Ltd., 2017)

Dry Conditions	Average Conditions	Wet Conditions
100	100	100
97	99	100
94	98	99
91	97	99
89	96	99
87	95	98
85	94	98
83	93	98
81	92	97
80	91	97
78	90	96
76	89	96
75	88	95
73	87	95
72	86	94
70	85	94
68	84	93
67	83	93
66	82	92
64	81	92
63	80	91
62	79	91
60	78	90
59	77	89
58	76	89
57	75	88
55	74	88
54	73	87
53	72	86
52	71	86
51	70	85
51	69	85

Dry Conditions	Average Conditions	Wet Conditions
48	68	84
47	67	83
46	66	82
45	65	82
44	64	81
43	63	80
42	62	79
41	61	78
40	60	78
39	59	77
38	58	76
37	57	75
36	56	75
35	55	74
34	54	73
33	53	72
32	52	71
31	51	70
31	50	70
30	49	69
29	48	68
28	47	67
27	46	66
26	45	65
25	44	64
25	43	63
24	42	62
23	41	61
22	40	60
21	39	59
21	38	58
20	37	57
19	36	56
18	35	55
18	34	54
17	33	53
16	32	52
16	31	51
15	30	50
12	25	43
9	20	37
6	15	30

Dry Conditions	Average Conditions	Wet Conditions
4	10	22
2	5	13
0	0	0

APPENDIX C

Cross-Section Plots

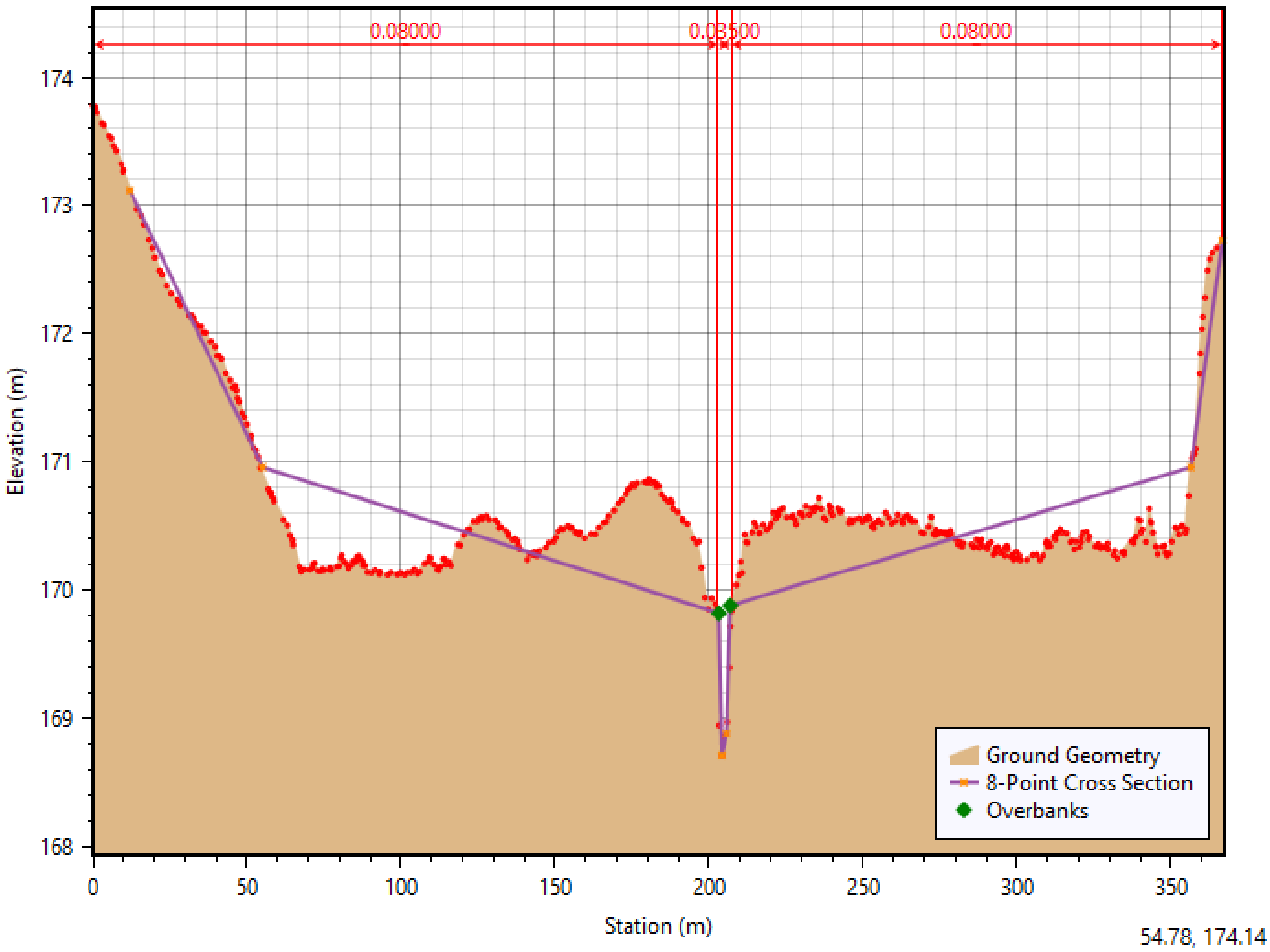

Figure C.1: Plot of Ground elevation and Simplified 8-point Cross-Section for XS-5 (Reach-5)

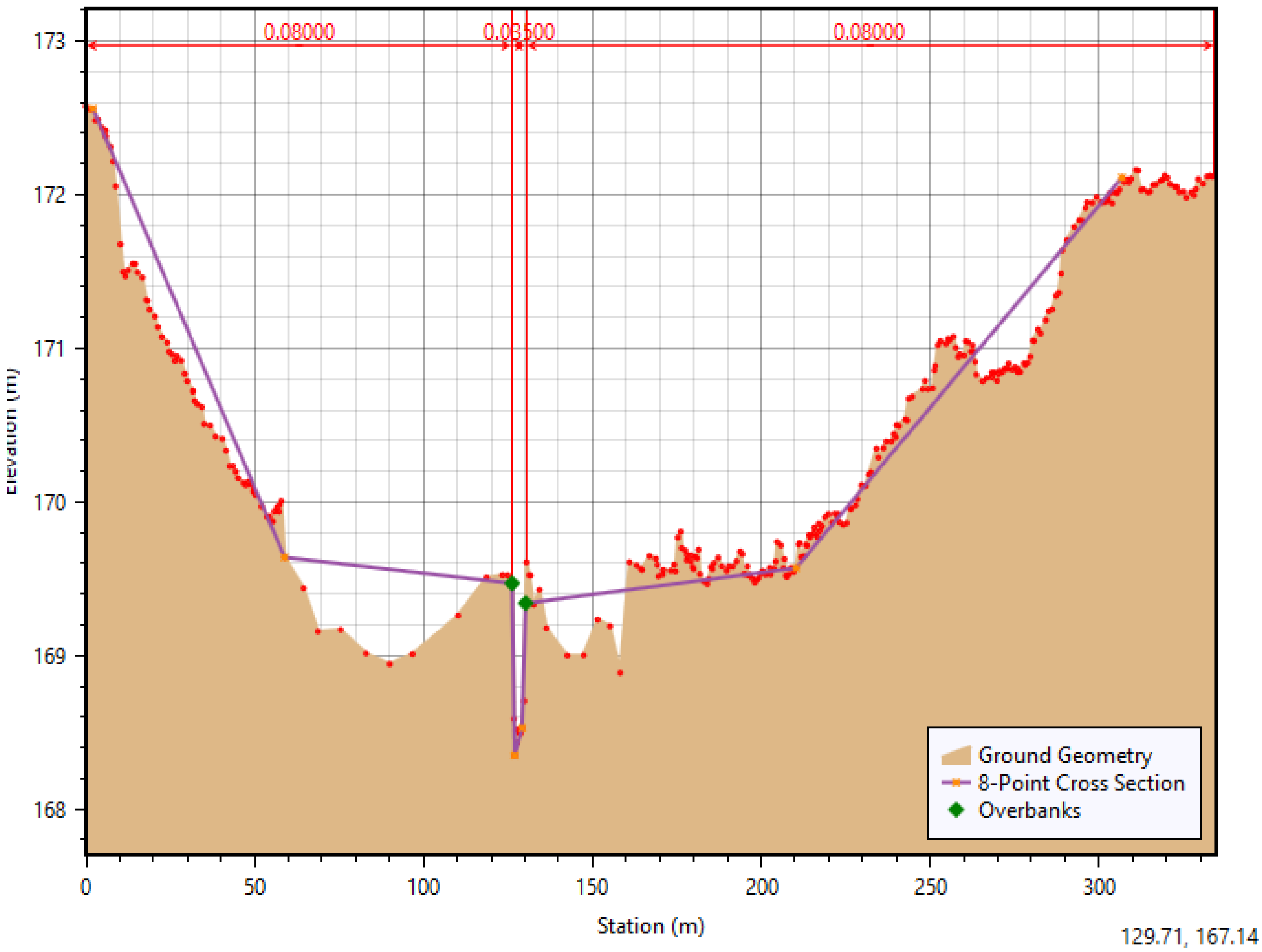

Figure C.2: Plot of Ground elevation and Simplified 8-point Cross-Section for XS-3 (Reach-3)

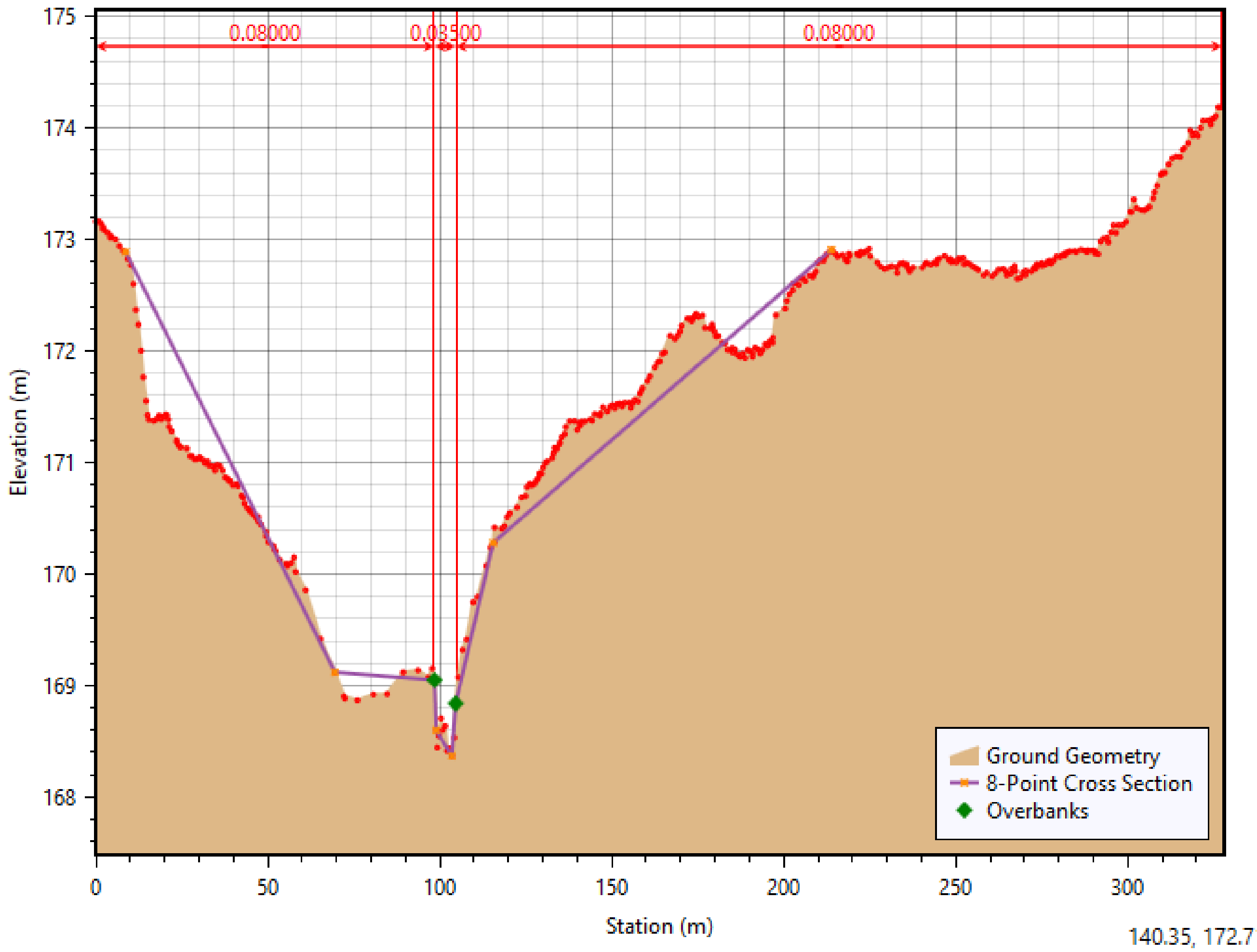

Figure C.3: Plot of Ground elevation and Simplified 8-point Cross-Section for XS-4 (Reach-4)

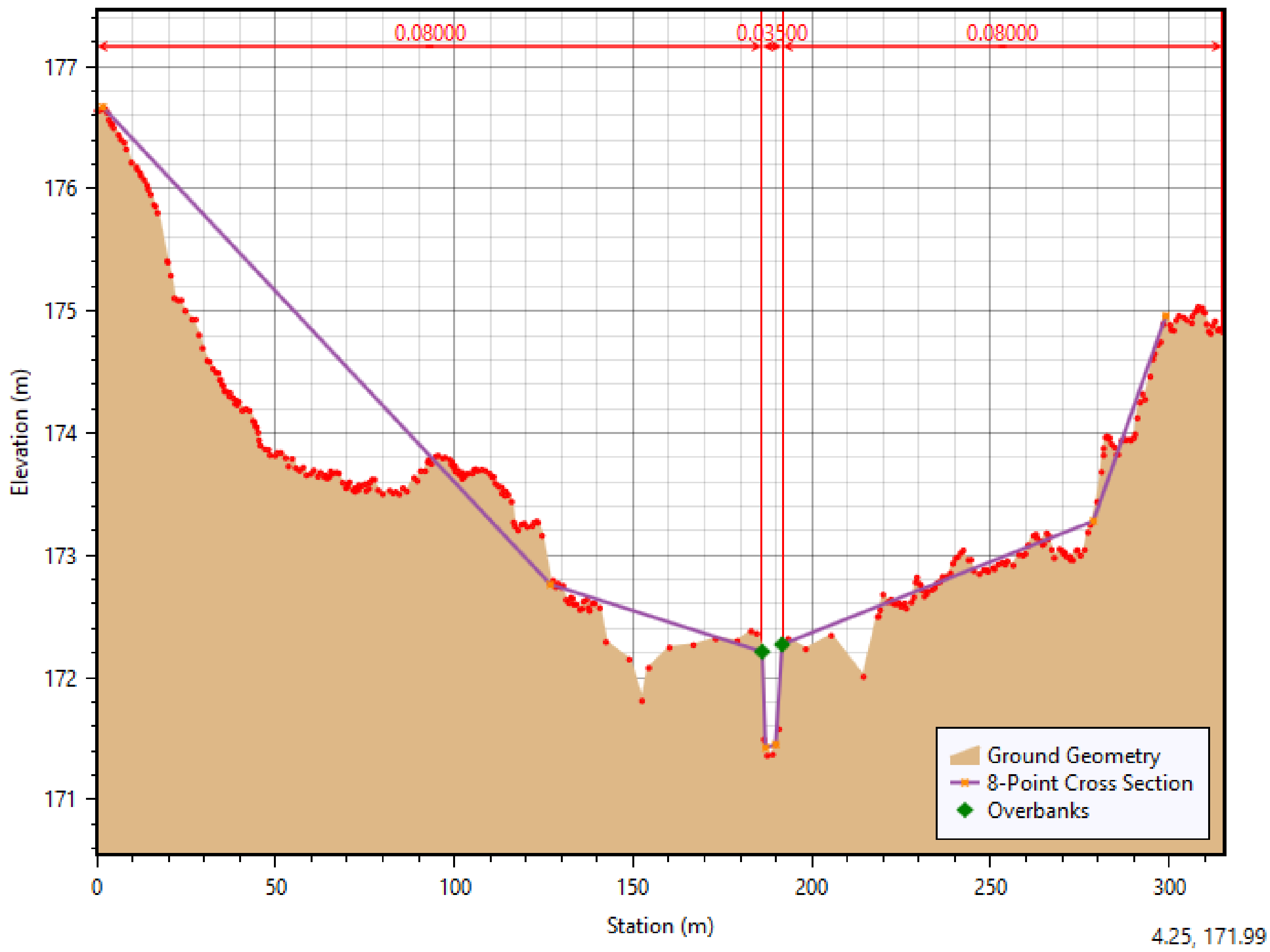

Figure C.4: Plot of Ground elevation and Simplified 8-point Cross-Section for XS-2 (Reach-2)

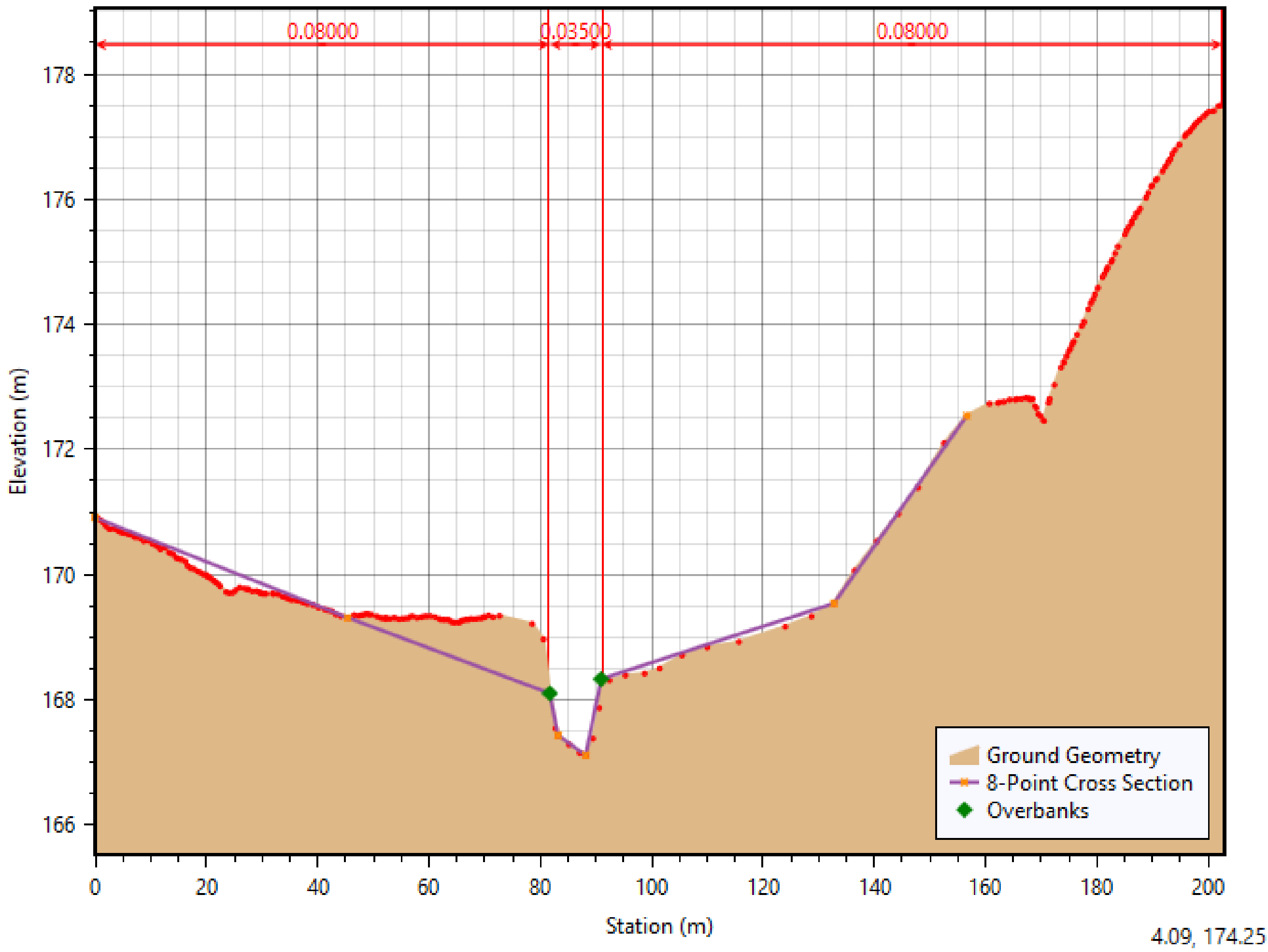

Figure C.5: Plot of Ground elevation and Simplified 8-point Cross-Section for XS-1 (Reach-1)

APPENDIX D

Reach Profile Plots

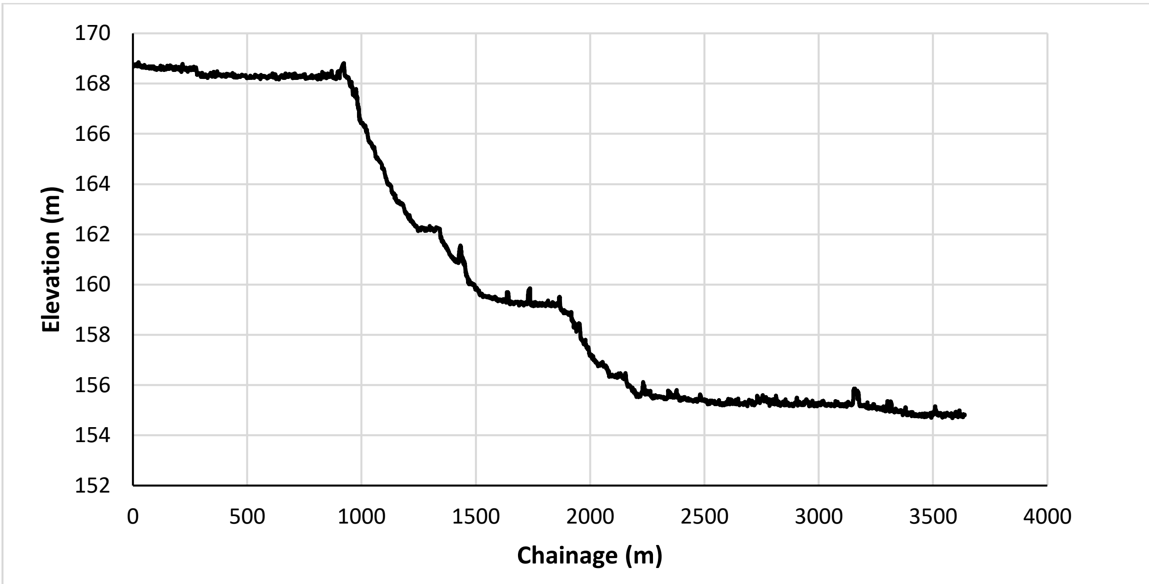

Figure D.1: Profile Plot of Reach-1

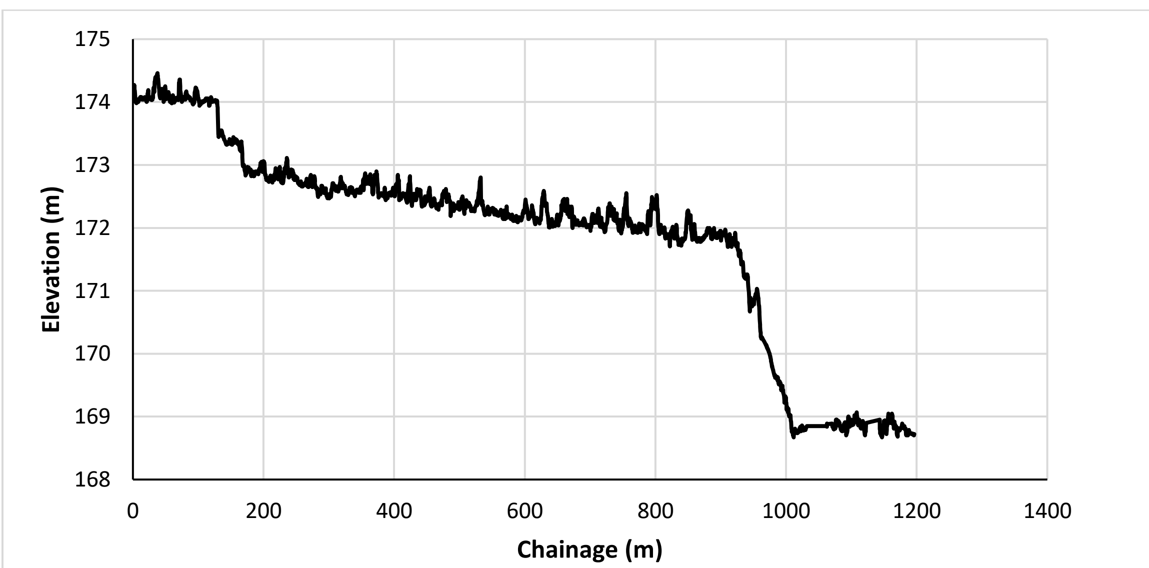

Figure D.2: Profile Plot of Reach-2

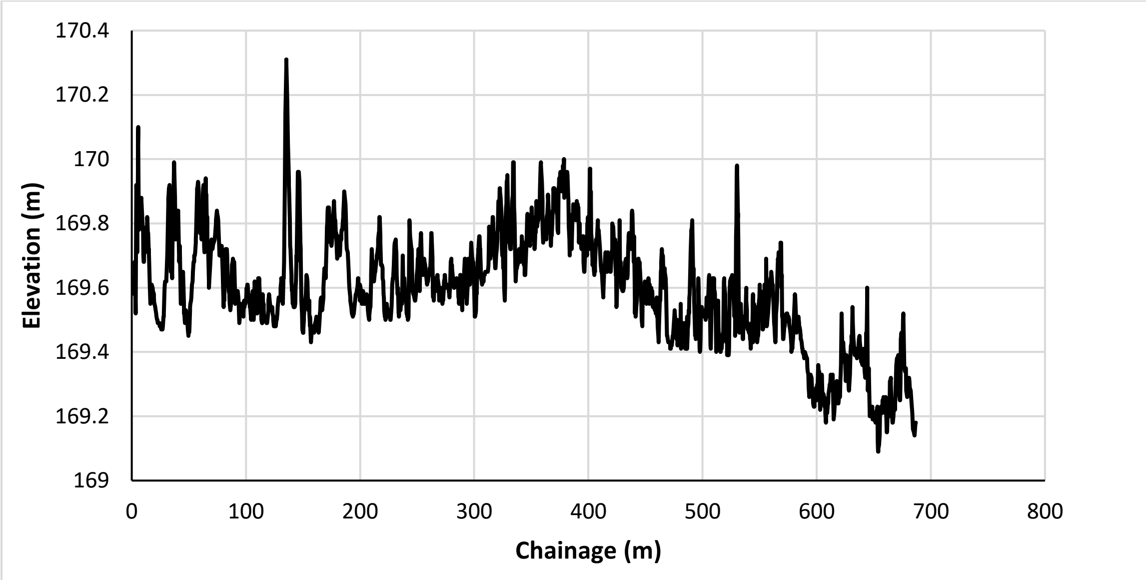

Figure D.3: Profile Plot of Reach-3

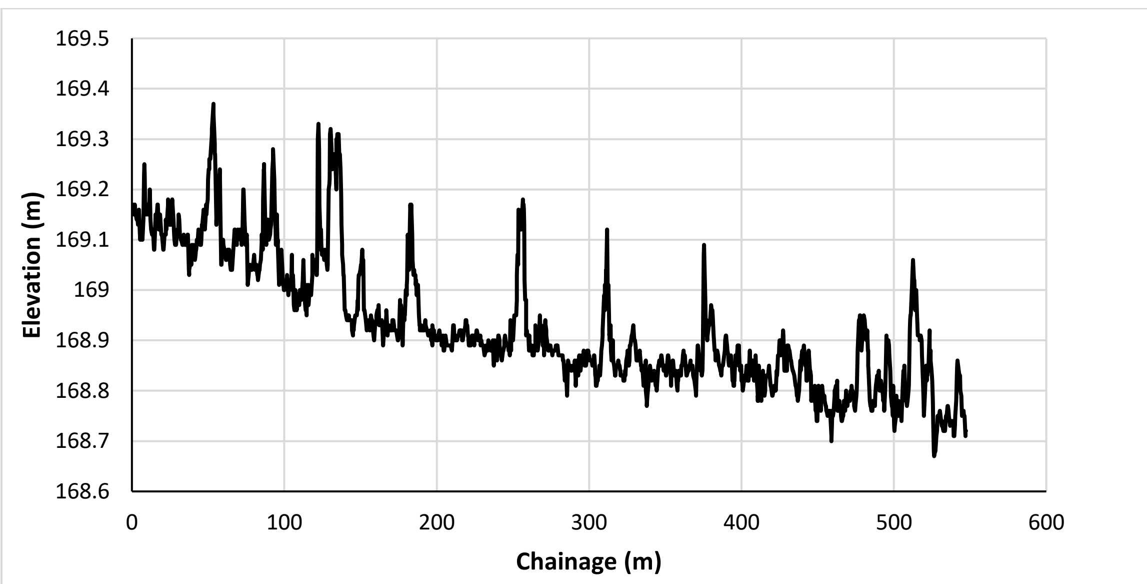

Figure D.4: Profile Plot of Reach-4

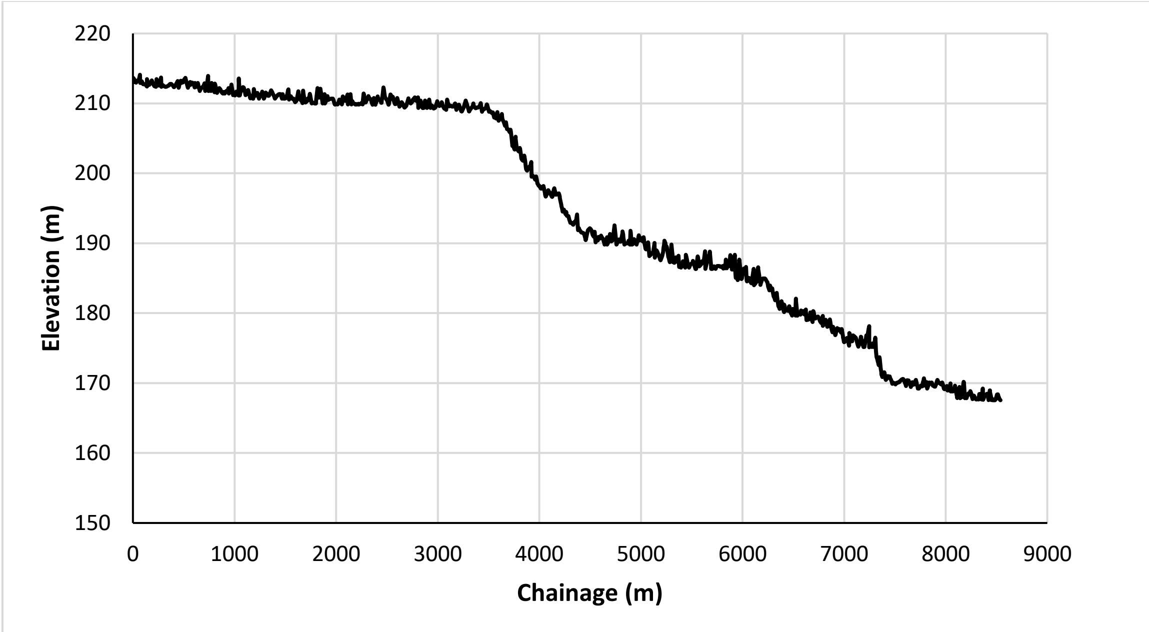

Figure D.5: Profile Plot of Reach-5

APPENDIX E

Example List of HEC-HMS Warning Messages

The warning messages received for the 100-year flood event under average antecedent moisture conditions is as follows:

WARNING 41084: Applying the diffusivity criteria using index flow 16 m3/s at reach "Reach-2" would have increased spatial steps from 6 to 9.
WARNING 41071: Flow depth convergence failed 26 time steps at reach "Reach-1"; maximum convergence error of 0.0527 M.
WARNING 41084: Applying the diffusivity criteria using index flow 38 m3/s at reach "Reach-1" would have increased spatial steps from 10 to 13.
WARNING 41071: Flow depth convergence failed 92 time steps at reach "Reach-5"; maximum convergence error of 0.0187 M.
WARNING 41084: Applying the diffusivity criteria using index flow 7 m3/s at reach "Reach-5" would have increased spatial steps from 55 to 81.
WARNING 41071: Flow depth convergence failed 20 time steps at reach "Reach-3"; maximum convergence error of 0.0481 M.
WARNING 41071: Flow depth convergence failed 20 time steps at reach "Reach-4"; maximum convergence error of 0.0453 M.