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FLOOD HAZARD ASSESSMENT REPORT

AT

ELEPHANT LAKE COTTAGES
HARCOURT, DYSART ET AL., ONTARIO

PREPARED FOR:

95 DEVELOPMENTS

December 19, 2024

Table of Contents

1. Site Background

King EPCM was retained by the Client (95 Developments) to conduct engineering investigations and services for the property located along the shoreline of Elephant Lake and Benoir Lake in the Harcourt Township, County of Haliburton, Municipality of Dysart et al. (the Site), as part of cottage permit submissions. The following document is a report of flood hazard analyses used to design the property. The property is located along the shoreline of Elephant Lake and Benoir Lake within parts or whole of Lots 32 and 33, Concession 12, Lots 27-31, Concession 11, Lots 27-31, Concession 10, Lots 27-33, Concession 9, and Lots 27-31, Concession 8, Harcourt Township, County of Haliburton, Municipality of Dysart et al. (Site).

The property has a wide variety of landscapes, including dense forests, wetlands, rocky cliff escarpments, sandy valley lands, clay slopes, and sandy beaches. The Site property is approximately 2000 Acres in size, divided into two different Blocks (phases), Northern phase with approximate area of 405,584.6 m^2, and Southern phase with approximate area of 852,307.2 m^2. While Northern phase is completely located east of Benoir Lake, behind Benoir Lake Rd., the Southern phase is extended from the west-south boundary (east of Elephant Lake) to the south boundary of the Site which is located on the north of the Elephant Lake, near the shoreline.

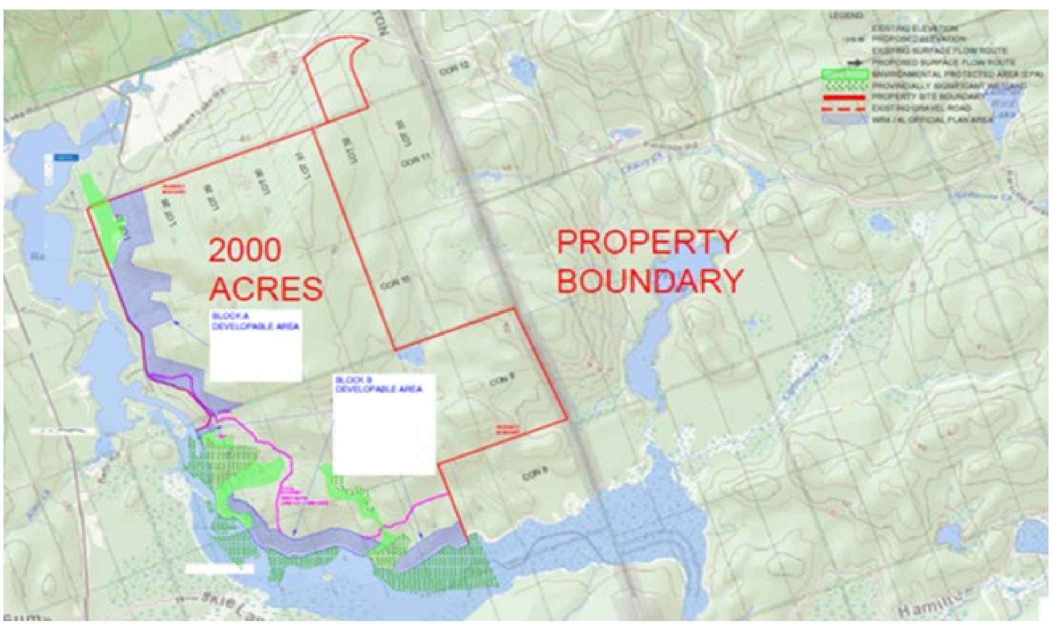


Figure 1- Topographic map of 0th Benoir Lake Rd., Dysart et al., ON

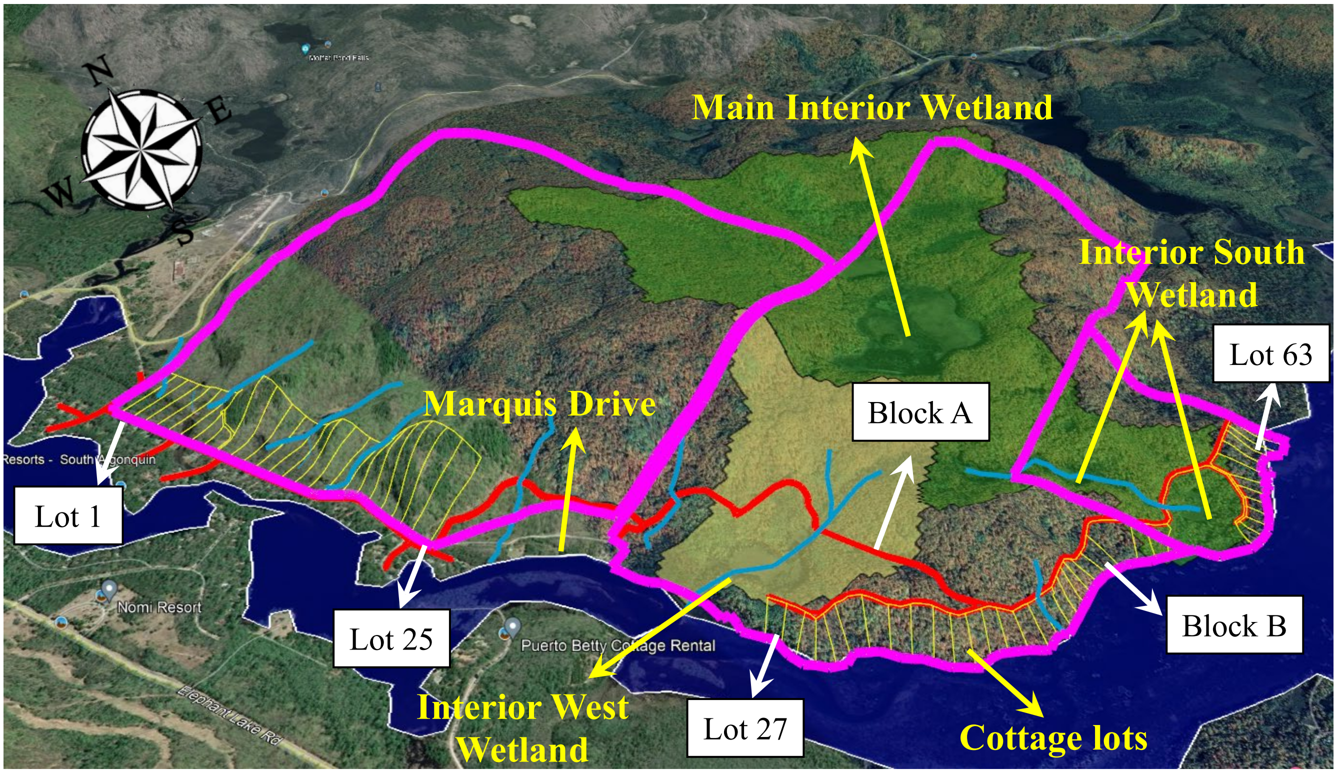


Figure 2 –Overview of subwatersheds, wetlands, roadways, and cottage lots

2. Madawaska (main) Watershed & Baptiste Lake Dam

The Madawaska River flows 270 km from its headwaters in Algonquin Provincial Park to the Ottawa River at Arnprior (Appendix I). This river is organized by tributary and further divided into a series of reaches or sections. The York River (120 km) is one of the main tributaries located at the south part of main watershed. It spans out into Elephant, Benoir and Baptiste Lakes and passes through Bancroft. This river is in the Saint Lawrence River drainage basin, and flows from the southern extension of Algonquin Provincial Park to the Madawaska River. There are 41 dams on the Madawaska River Watershed, 9 of which are built on the York River. These serve as static or operated flood and level control structures (Appendix II).

Baptiste Lake is a large, beautiful lake located in Herschel Township, Bancroft. This lake is approx.. 2226 hectares and part of a three-lake chain with Benoir and Elephant Lakes. The Baptiste Lake Dam is located approximately 6km north of Bancroft (Fig. 3). In 1931-1933 a dam was built at the east end of the Baptiste Lake along the York River by the Ontario Department of Public Works and the Ontario Hydro Electric Power Commission. The purpose of the dam was to regulate flows to hydroelectric generating facilities on the Madawaska River system. The current dam was built in 1966 and is located 10 m downstream of the 1931 dam. Baptiste Lake Dam is 6.9 m high by 112m long with four log sluices, one gate and a weir. Each of the log sluices are 4.3 m long with a sill elevation of 26.8 m (88 ft) Local Datum. The gate is 1.1 m long by 1.1 m high with a sill elevation of 26.8 m (88 ft) LD. The weir is 61 m long with a crest elevation of 29.9 m (98 ft). An imperial water level gauge is installed at the dam (Appendix III).

Figure 3- Baptiste Lake Dam

The consequences of failure of the Baptiste Lake Dam would result in potential property damage and incremental loss of life downstream (OMNR, 2006), however, the development site of the current project on the shores of Elephant Lake will not be a concern. But the concern of the cottage construction in this project is the rising water level of the York River on the north shore of the Elephant Lake, which is controlled by Baptiste Lake Dam. The dam is operated according to the operation manual developed by MNR in 1988 (Appendix IV).

The operating zone represents mid-range water levels both above and below the normal operating zone in which inconvenience and minor damage can occur. Usually, water levels in these ranges are of short duration, except during prolonged wet or dry periods. The high-water level zone/low water level zone represents extremes of the water level range. These are above and below the operating zone, respectively. Water levels in the high-water level zone usually occur only at the peak of an abnormally high spring run-off or following an extremely large summer rainfall period and are of short duration. As it can be seen, the level of Baptiste Lake is usually maintained between 6.5- and 8.8-feet LD. The operating range in summer is 351.49 m to 351.94 m with the normal water level at 351.76 m which provides an operating range of about 0.47 m (about 18 inches). During the winter the reservoir is lowered to 351.33 m. The typical annual mode of operation of Baptiste Lake is summarized in Table 1.

Table 1- Baptiste Lake Operating Regime (OMNR, 2018)

Season	Operation
Spring & Summer	March 15 drawdown (6.50 - 7.00), dependent on snow level, moisture content and general weather forecast. Restrict drawdown for walleye/muskie from April 15 to mid July no less than one foot, then maintain to September 1. May 1 high water level zone: 8.50 feet July 15 summer optimum operating level: 7.80 feet.
Fall & Winter	Fall drawdown for lake trout to 7.30 feet and will remain constant until the end of October. The level will go no lower than 6.50 until March 15. January 1 freeze up at 7.50 feet.

3. Model Introduction

The numerical model chosen to undertake the hydraulic modelling tasks within this consultancy is HEC-RAS 6.0 Beta 3. This model is an open-source software with the same capabilities as other (commercial) software, with a proven track-record of having been implemented in a wide range of scenarios worldwide, is one of the main reasons for this model selection. HEC-RAS can be implemented in 1D, 1D-2D or 2D mode, and the mathematical approach to each of the dimensional modules is based on the Navier-Stokes equations. We used the 1D mode later for culvert design modeling. In this report, due to the 2D nature of flooding, HEC-RAS 2D model has been used to provide the flood inundation maps. HEC-RAS 2D flood model was developed by US Army Corp of Engineer's Hydrological Engineering Center (HEC). The model can solve 1D and 2D flood hazard problems through 1D, 2D and 1D/2D coupling approaches.

4. Geometric Data

HEC-RAS 2D requires following geometric datasets:

- Digital Elevation Model (DEM)
- Manning's n layer
- 2D flow area & mesh generation
- Boundary condition lines

4.1. Digital Elevation Model

The Digital Elevation Model (DEM) represents floodplain terrain in flood model. Globally freely available DEMs (SRTM, ALOS World3D and ASTER) and locally available high resolution can be used for the terrain representation in flood model. In this project, we used two different sources of elevation fields for dem creation (Figure 4), the low-resolution public contours provided by MNRF and high-resolution points through LiDAR data by Rock Robotic were used to model topography.

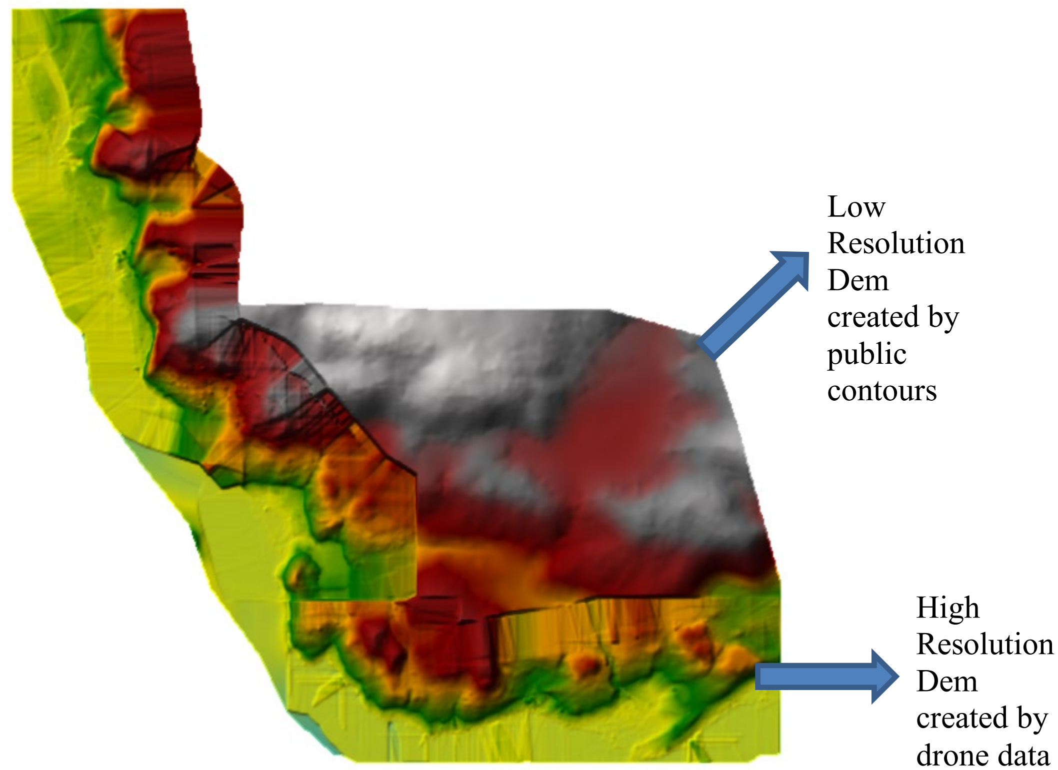


Figure 4- Dem created based on drone data and public contours

4.2. Manning's n layer

Manning's n layer is required to account for infiltration component of water channel, floodplain and its surrounding landcover/land use types in flood model and can be prepared using the existing landcover data and Manning's n Table (Chow 1959). In this project, we used a fixed value of 0.04 for the 2d flow area.

4.3. 2D flow area & mesh generation

2D Flow Area tool is used to determine the flood model computation boundary based on the terrain. Moreover, the 2D Flow Area should follow the terrain and cover the minimum area considered for the simulation run in order to optimize the flood model computation time. This area is shown in Figure 5. Using the 2D Flow Area Generate Points option and provide for DX= 5m and DY= 5m cell values (shows floodplain grid spacing), the computation mesh is generated and used in model numerical simulation.

4.4. Boundary condition lines

Boundary condition lines are the cross-sections upstream and downstream where discharge data in the form of hydrographs (flow or stage) can be provided to the flood model. For this 2D flow area, we have two different inflow boundaries starting from the interior wetland overflow as input hydrograph and the lake shorelines is defined as outlet boundary using stage hydrograph.

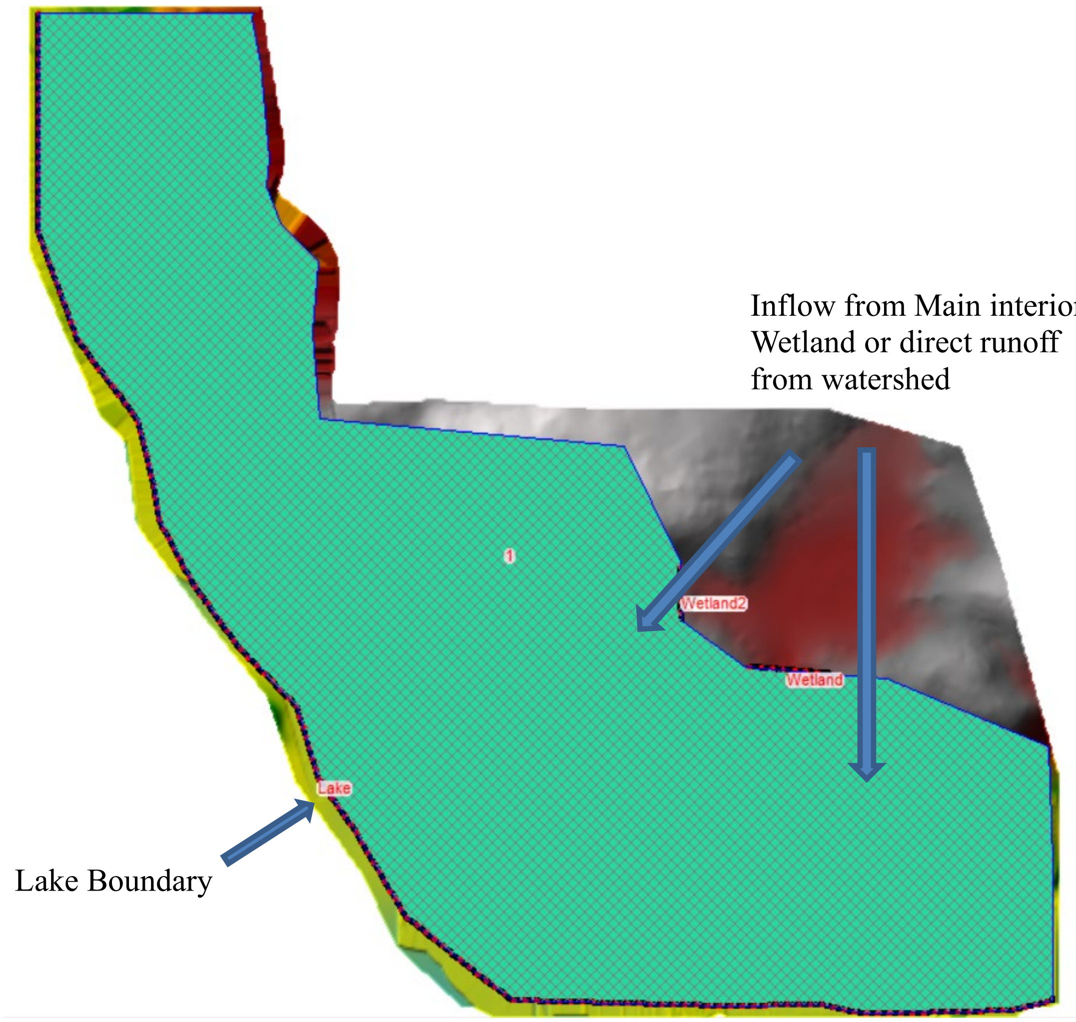


Figure 5- Two-dimensional flow area mesh and boundary conditions.

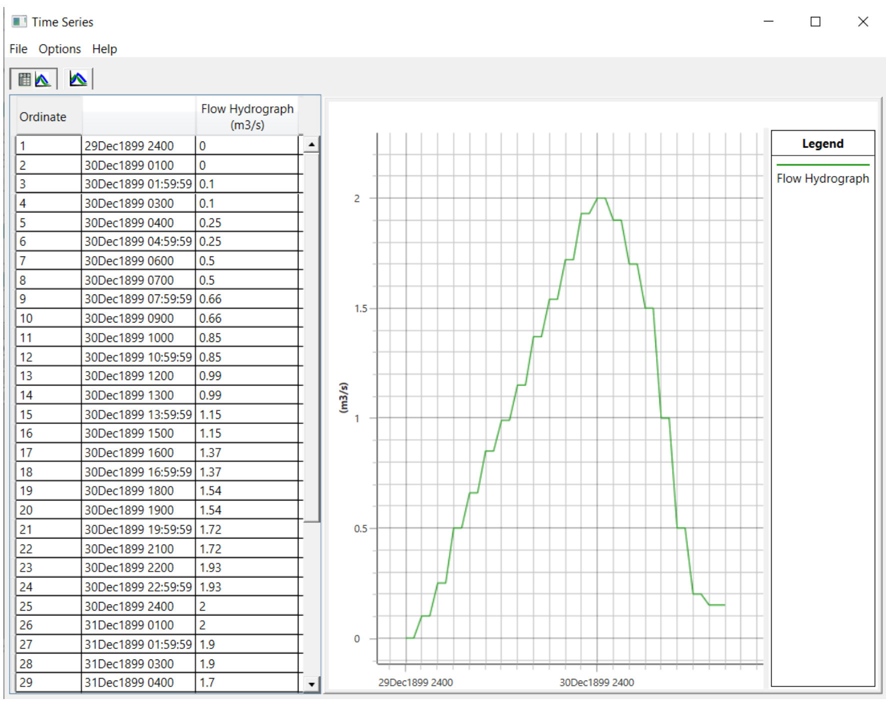
5. Hydrological data

Hydrological data including known event hydrograph and stage height at the upstream and downstream gauge stations or boundaries are required for model setup, calibration and validation. There is no station or flow data on the York River within the project area. The only flow hydrograph data available for the York River near Bancroft and the Baptiste Lake Dam is Water Survey of Canada Station YORK RIVER NEAR BANCROFT (02KD002). This station located downstream of the dam is not useful for our modeling.

OFAT (Ontario Flow Assessment Tool) was used in this study to estimate the flood discharges for different return periods. OFAT contains a series of regional hydrologic models and empirical relationships that generate water flow information. Flow regimes can be determined for a watershed after the watershed has been generated and the required characterizations computed. OFAT currently contains three flow model categories. Each category contains one or more models. In this project, the "Index

Flood Method with Expected Probability Adjustment (EPA)" (Moin and Shaw, 1985) was used to estimate the maximum probable flood discharge of creeks within the site leading to the lake.

This model estimated parameters of: Q1.25, Q2, Q5, Q10, Q20, Q50, Q100, Q200, and Q500. The later value (500 years return period) as a worst-case condition was used to make an artificial hydrograph for 2D hydraulic modeling (Fig. 6). The Figure (inflow boundary condition) presents the flow data assigned at two main creeks overflowing from the main interior wetland. The flow data for open channels is typically unsteady since the discharge through the channel will usually vary with time and the reason for this is the temporal nature of the storm event that produced the flooding event.



Ordinate		Flow Hydrograph (m3/s)
1	29Dec1899 2400	0
2	30Dec1899 0100	0
3	30Dec1899 01:59:59	0.1
4	30Dec1899 0300	0.1
5	30Dec1899 0400	0.25
6	30Dec1899 04:59:59	0.25
7	30Dec1899 0600	0.5
8	30Dec1899 0700	0.5
9	30Dec1899 07:59:59	0.66
10	30Dec1899 0900	0.66
11	30Dec1899 1000	0.85
12	30Dec1899 10:59:59	0.85
13	30Dec1899 1200	0.99
14	30Dec1899 1300	0.99
15	30Dec1899 13:59:59	1.15
16	30Dec1899 1500	1.15
17	30Dec1899 1600	1.37
18	30Dec1899 16:59:59	1.37
19	30Dec1899 1800	1.54
20	30Dec1899 1900	1.54
21	30Dec1899 19:59:59	1.72
22	30Dec1899 2100	1.72
23	30Dec1899 2200	1.93
24	30Dec1899 22:59:59	1.93
25	30Dec1899 2400	2
26	31Dec1899 0100	2
27	31Dec1899 01:59:59	1.9
28	31Dec1899 0300	1.9
29	31Dec1899 0400	1.7

Figure 6- Time series hydrograph for coastal lands overlooking the lake (Inflow boundary condition for main creeks)- Worst case conditions (500yr) assumed for both main creeks

Furthermore, based on Acres International review studies on dam safety of 24 dams and control structures located in the Madawaska River basin (OMNR, 2006), a deterministic rainfall-runoff model entitled SSARR (Streamflow Synthesis and Reservoir Regulation) were developed by OPG as part of the "Dam Safety Assessment Program Probable Maximum Flood Madawaska River". Part of these data is related to Baptiste Dam were presented in Tables 2 and 3 in the following and used as boundary condition for lake water level during flood event simulation. In other words, we used the constant water level related to the highest probable water level of the lake (Summer Rainfall or Spring Rain + Snowmelt) according to the operation curve of Baptiste Dam as the outlet boundary condition (Table 4).

Table 2- SSARR Results for Summer Rainfall Events (OMNR, 2006)

Dam	Summer Event	Peak Inflow (m^3/s)	Peak Outflow (m^3/s)	Peak Water Level (m)
Baptiste Lake Dam Deck Elevation 353.39	2-yr	99.7	38.5	351.95
	5-yr	153.3	57.1	352.01
	10-yr	192.1	70.2	352.07
	25-yr	244.2	87.5	352.13
	50-yr	285.2	100.9	352.19
	100-yr	327.9	114.6	352.25
	PMP	1465.3	364.1	352.95

Table 3- SSARR Results for Spring Rain + Snowmelt Events (OMNR, 2006)

Dam	Summer Event	Peak Inflow (m^3/s)	Peak Outflow (m^3/s)	Peak Water Level (m)
Baptiste Lake Dam Deck Elevation 353.39	2-yr	154.2	35.1	351.45
	5-yr	204.9	45.6	351.49
	10-yr	237.7	52.3	351.53
	25-yr	278.6	60.6	351.57
	50-yr	308.6	66.6	351.61
	100-yr	338.4	72.5	351.65
	PMP+100y SWE	1248.2	443.0	353.25

Table 4- Water surface elevation (outlet boundary condition for lake)

Scenario	100yr (Summer Rainfall Events)	PMP+100yr SWE[1] (Spring Rain+Snowmelt Events)
	1	2
WSL (m)	352.25	353.25

[1] Snow Water Equivalent

6. Culverts Location & Capacity (Existing vs Proposed)

Based on our initial site visit (2020-2021), the existing culverts on this property were either partially blocked or demolished, rendering them incapable of conveying a 100-year storm event without water overflowing onto the road surface (Figure 7).

There are two main creeks in this area where the proposed culverts (#2 and #6) intersect with the roadway. A hydraulic simulation has also been conducted in the next section to evaluate their capacity and performance during 100-year storm events. Additionally, two small culverts (less than one foot in diameter) were observed on minor watercourses crossing the roadway. See Fig. 7 for more information.

Figure 7- Existing CSP culverts

Based on the existing terrain derived from LiDAR data and the slope of the upstream catchment area draining into the lake, the engineer has identified nine critical intersections along the proposed roadway where culverts are required (see Table 4 for culvert sizes). These are further supported by roadside runoff collection ditches to manage surface water effectively. Among these, culverts No. 2 and No. 6 stand out as critical components, with proposed diameters of 1500 mm (59 in.) and 750 mm (30 in.), respectively. These culverts are specifically designed to handle the 100-year flood event, ensuring sufficient capacity to manage the flow effectively. The next section evaluates their performance in facilitating the safe passage of floodwaters under various scenarios.

Table 4- Proposed Culvert Size & Material

Culvert Name	Recommended Dia. (mm)	Recommended Dia. (in)
Culvert #1	457	18
Culvert #2	1500	59
Culvert #3	457	18
Culvert #4	457	18
Culvert #5	457	18
Culvert #6	750	30
Culvert #7	457	18
Culvert #8	457	18
Culvert #9	457	18

All culverts are recommended to be double-walled HDPE Culverts, corrugated outside and smooth inside
Culverts may be substituted with corrugated steel pipe where stock is not available
All culverts has a recommended length of 12.0m.
Culverts must be joined using appropriate manufacturer recommendations

The upstream and downstream cross-sections of the main creek flowing east of the site show maximum average water depths of 40 cm and 24 cm, respectively, during a 100-year flood (Figs. 8-9). The hydraulic modeling results in the next section confirm that the proposed culverts are adequately designed to handle these flows efficiently.

Upstream of the roadway, the eastern creek will split into two separate branches based on the topography/grading. The main branch (Eastern Creek A) is expected to have a maximum water flow depth of 48 cm and a flow rate of 1.52 cms (Figs. 10-11). For this section, a 59-inch (1500 mm) diameter culvert (Culvert No. 2) has been proposed. The secondary branch (Eastern Creek B), which diverges from the main eastern creek, will have a maximum flow depth of 16 cm and a flow rate of 0.6 cms (Figs. 10-11). An 18-inch diameter culvert (Culvert No. 3) is proposed for this branch.

Additionally, another main creek flows to the west of the site. Based on its upstream and downstream cross-sections, shown in Figure 12, the maximum water depths are 13 cm and 21 cm, respectively, with a flow rate of 2.05 cubic meters per second. The engineer has chosen a relatively large culvert with a diameter of 750 mm (30 in.) to ensure the safe passage of 100-yr floodwaters along this branch. Its performance has been evaluated using a hydraulic model, as detailed in the next section.

Figures 13 to 15 illustrate the locations of the proposed culverts at the intersections of the proposed roadway with the valleys/potential waterways in the upstream catchment. As shown, the main culverts, No. 2 and No. 6, are positioned along the mainstream paths of runoff generated within the upstream catchment area.

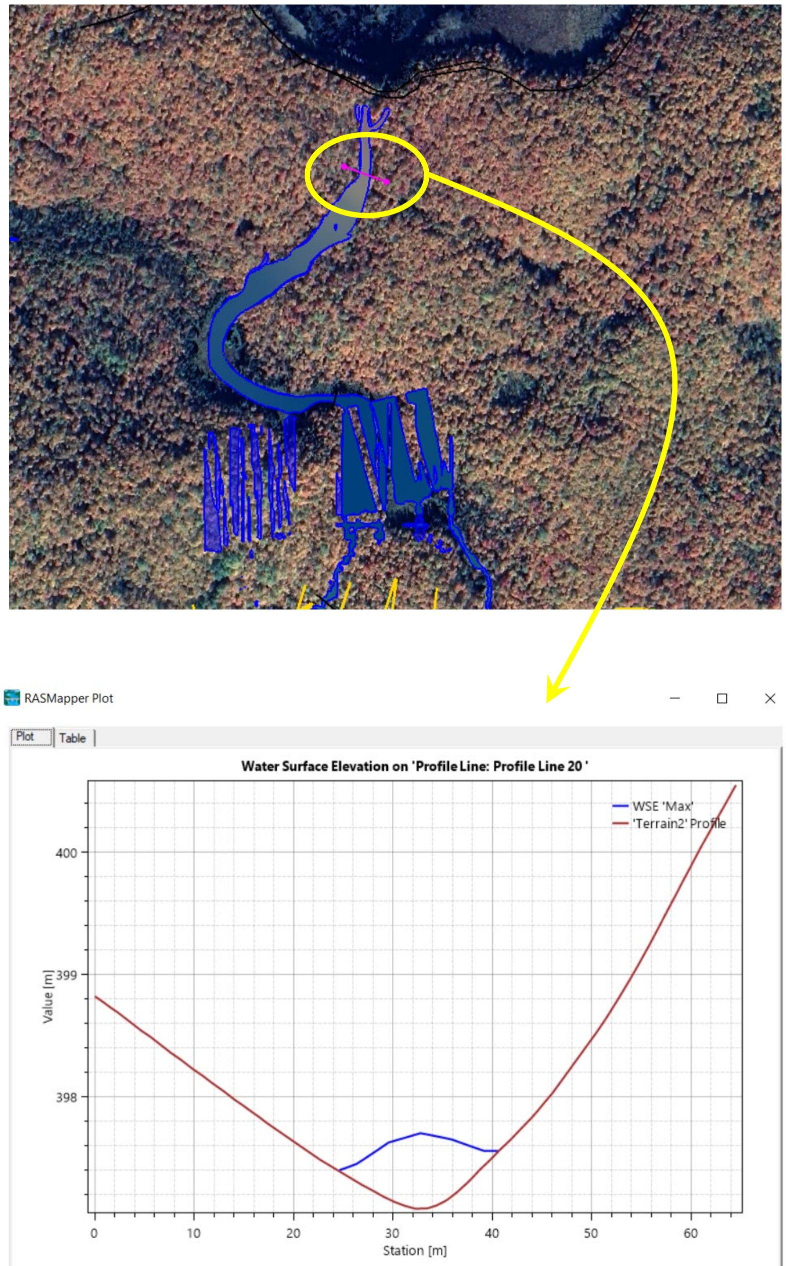


Figure 8- Cross-section of Main Eastern Creek, Upstream.

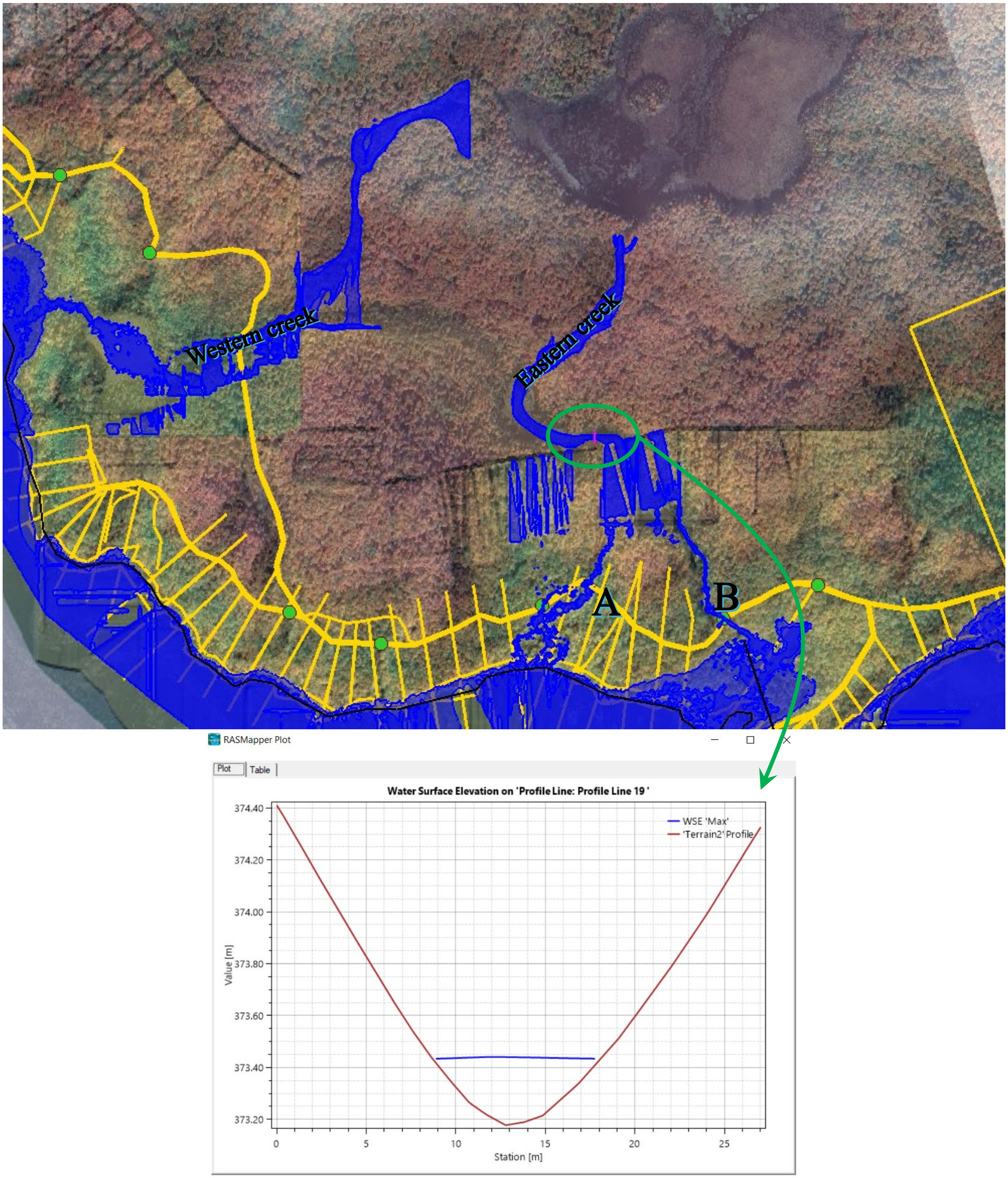


Figure 9- Cross-section of the Main Eastern Creek, Downstream, Before Dividing into Two Separate Branches

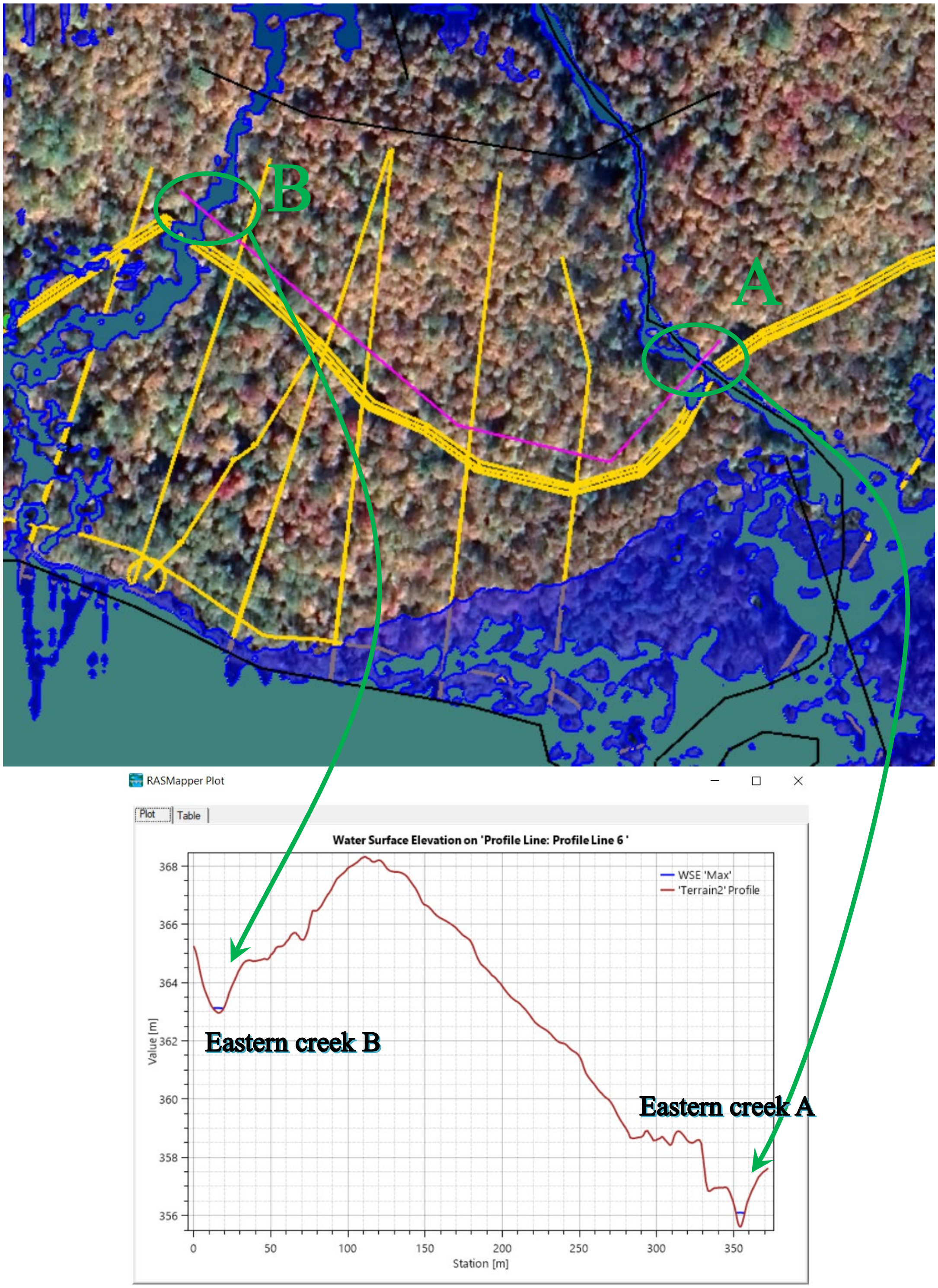
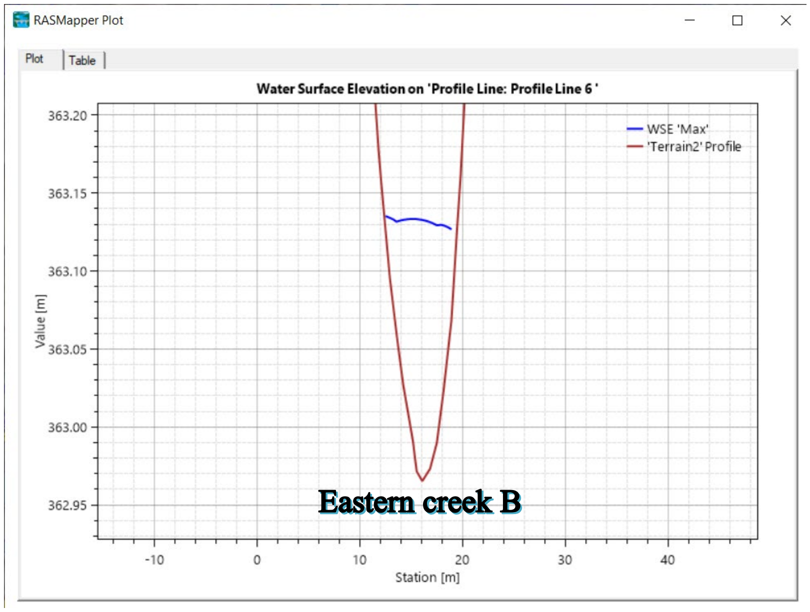


Figure 10- Cross-section of the Main Eastern Creek, Downstream, After Dividing into Two Separate Branches



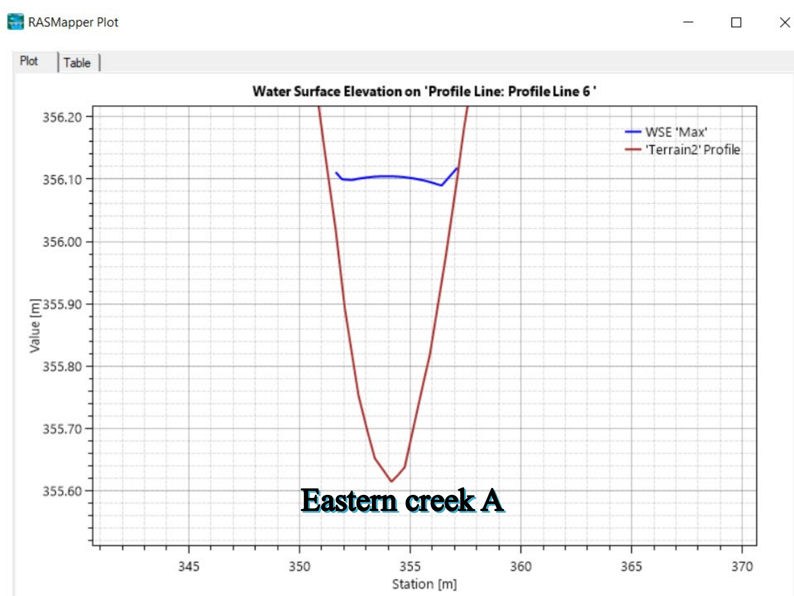


Figure 11- Cross-sections of the Eastern Creek Tributaries

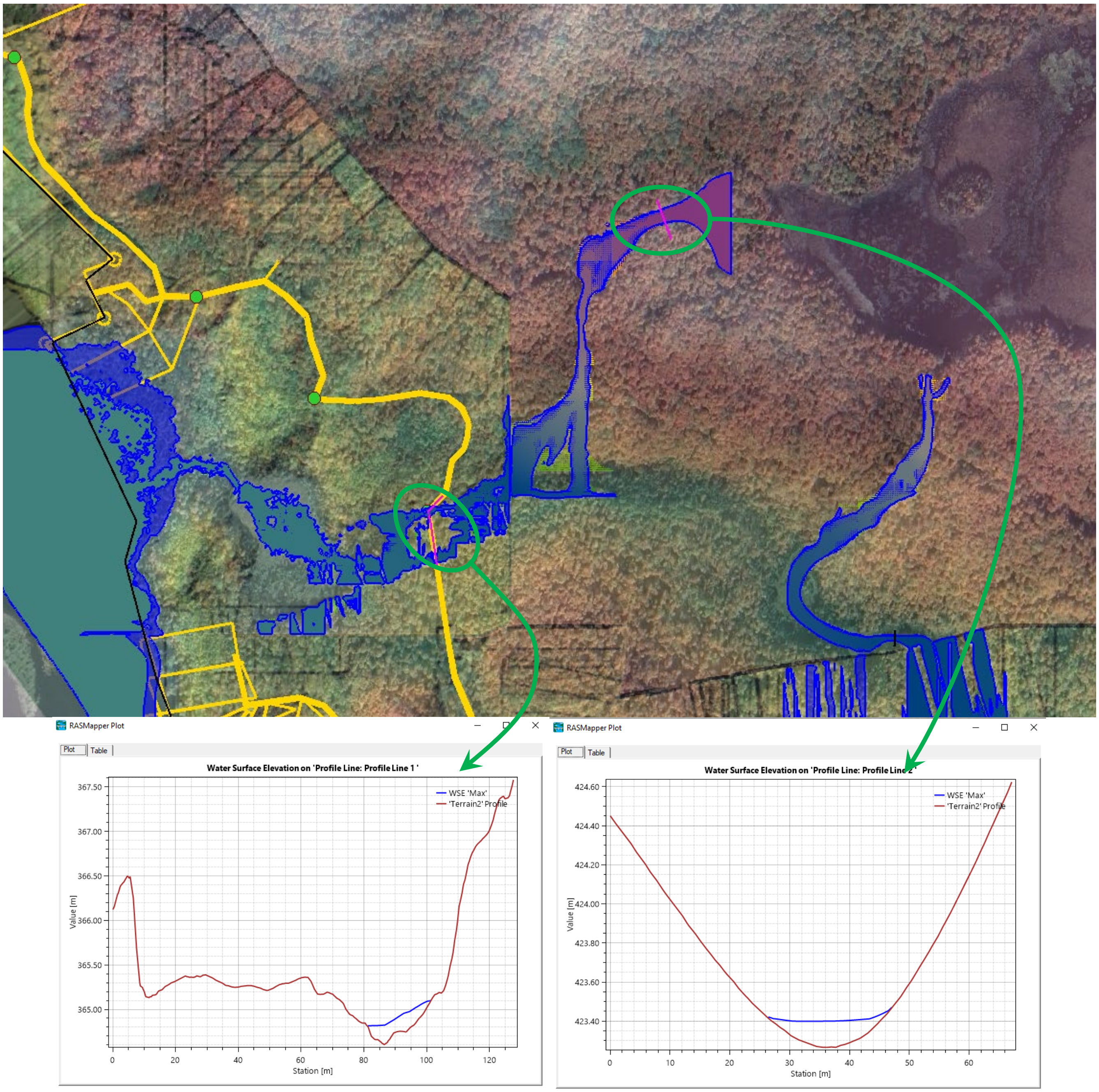


Figure 12- Cross-section of Main Western Creek, Upstream and Downstream.

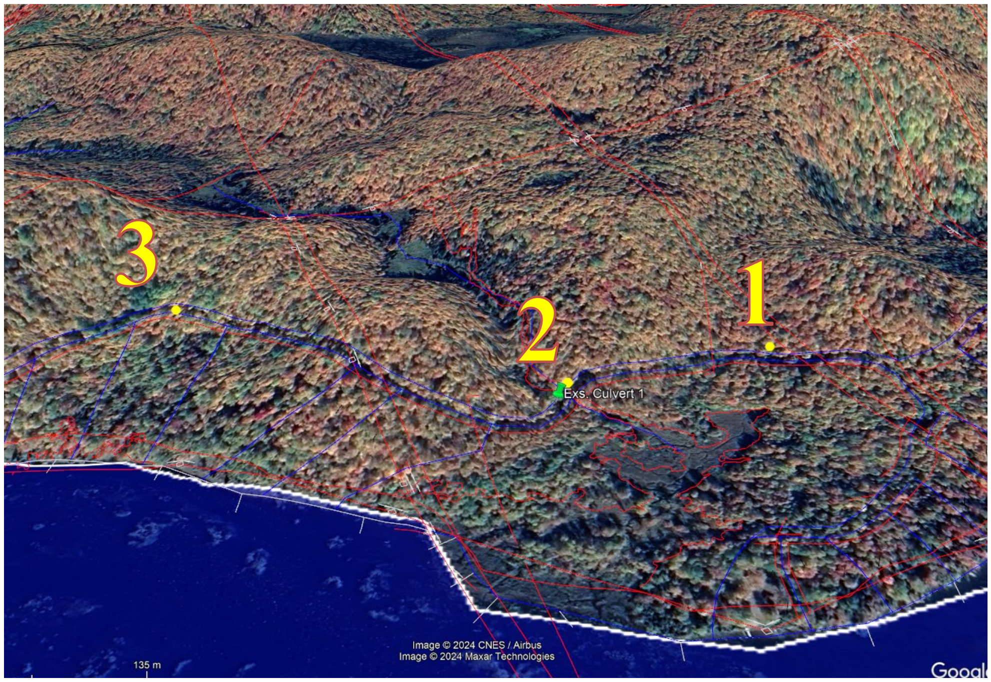


Figure 13- Existing vs Proposed Culvert locations, East of the site

Figure 14- Existing vs Proposed Culvert locations, Middle of the site

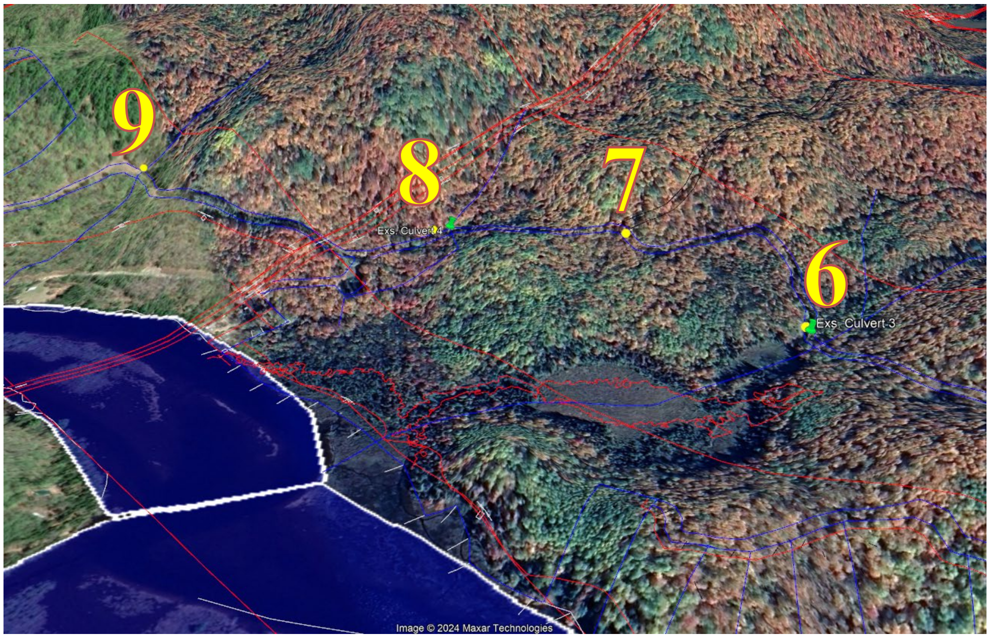


Figure 15- Existing vs Proposed Culvert locations, West of the site

7. Results & Discussion

In this section, an evaluation of the results for the 2D model will be undertaken. We will review the simulation results for a lake elevation of 352.25 m, corresponding to a 100-year summer precipitation event. For this purpose, three cross-sectionals configurations for main creeks overflowing from the wetland within the property site and lake shoreline are evaluated by determining the WSEL. So, we produced the maximum flood inundation map (Fig. 17), longitudinal profile of water level along main creeks within the site property (Figs. 19-20) and lake shoreline (Fig. 21).

As it can be seen in Fig. 18, the longitudinal section of the interior wetland shoreline overlooking the property shows that the ground level (Thalweg) of the eastern creek is 4 m lower than the western creek and as a result a larger flood is likely to occur in the eastern part of site if the wetland overflows downstream.

As expected, the creeks inside the property do not have much effect on the flooding of cottage lots near the lake shoreline, and only in the lots located at their confluence with the lake (delta region), the effects of water level rising and flooding are observed. The severity of flooding in the delta region (Deltas are wetlands that form as rivers empty their water and sediment into another body of water, such as an ocean, lake, or another river) is higher in scenario 2, where the lake level is higher.

Figures 22-23 shows the flooding maps of coastal lands in the worst-case conditions due to the increase of the lake water level with the occurrence of summer rainfall or spring rainfall with a return period of 100 years along with snowmelt. As can be seen, only in some lots there is a problem of flooding on the shoreline, which can be relieved by retreating the site of the cottage to an upper area or by increasing the height of new dwellings.

According to detailed Fig. 23, the lots that will be exposed to generally minimal amount of flooding by increasing the maximum level of the lake (353.25m) and according to the existing topographic map are:

Phase 1 – not affected.

Phase 2 – all waterfront properties are expected to have minor highwater elevation changes, but not significantly

In summary, all of the proposed cottages for Phase 1 and in Phase 2 are generally in close proximity to local creeks or Elephant Lake waterfront. Interior highland and interior stream flooding due to significant storms do not contribute to any flooding hazards for the proposed structures. The technical 1:100-year flood limit is recommended as 352.25m, and all structures should have a 10m horizontal offset from this delineation limit. This horizontal offset is generally managed through the existing 30m shoreline offset.

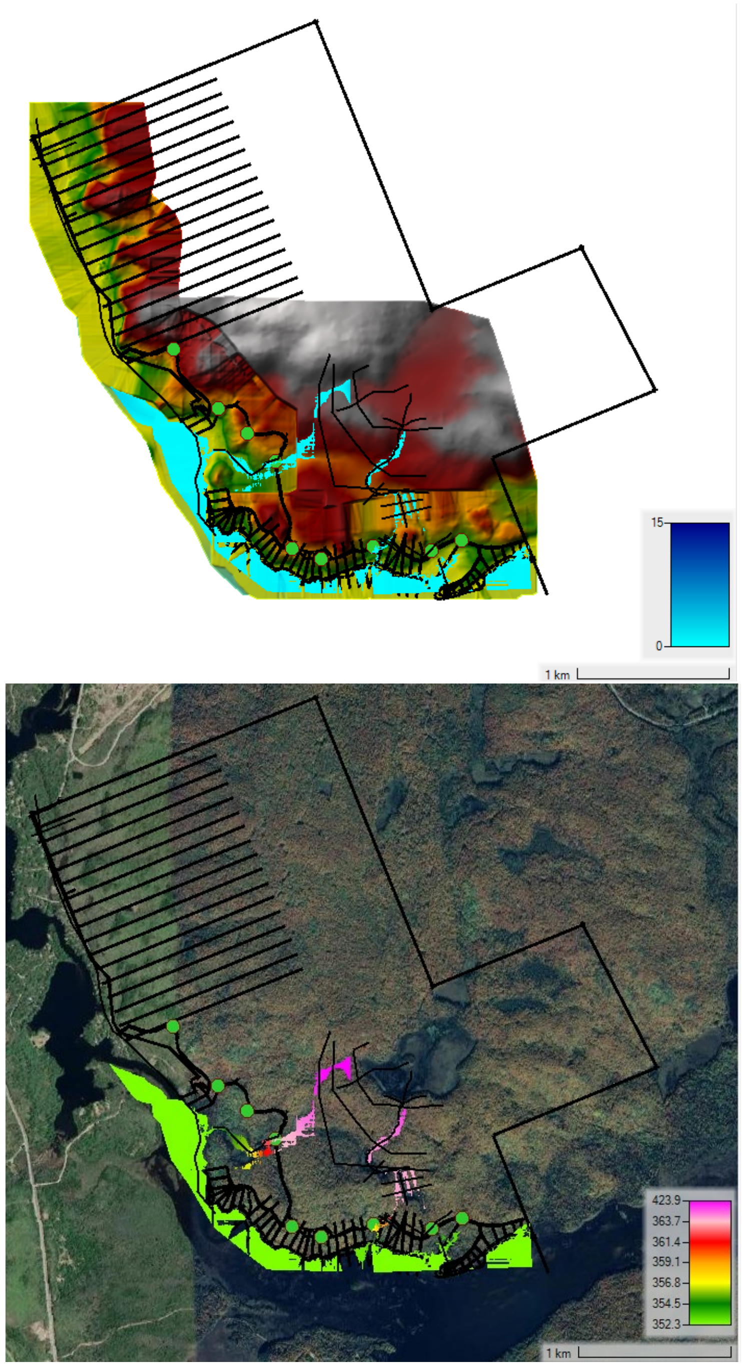


Figure 17- Maximum flood inundation map- 100 Years Event
(Top: water depth, Bottom: water surface elevation, Green circle: culvert locations)

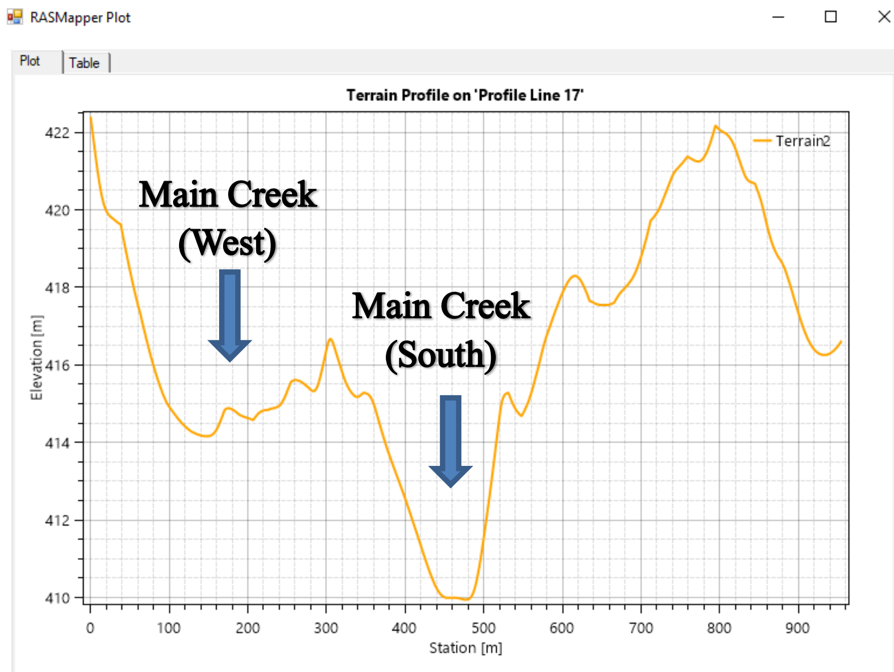


Figure 18- Wetland shoreline longitudinal section overlooking study area

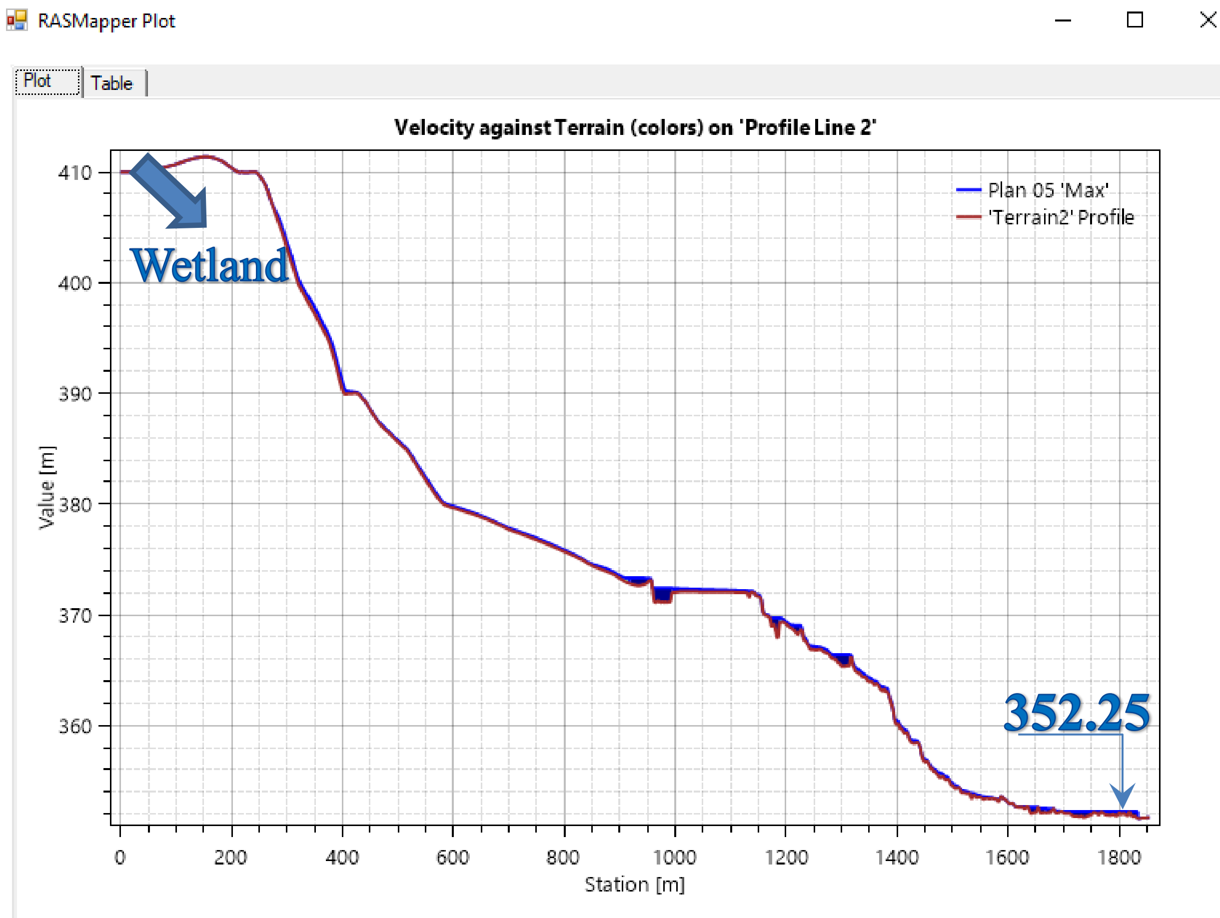


Figure 19- Longitudinal section of Main Creek (South) between wetland and lake with maximum water surface elevation (Lake level= 352.25 m)

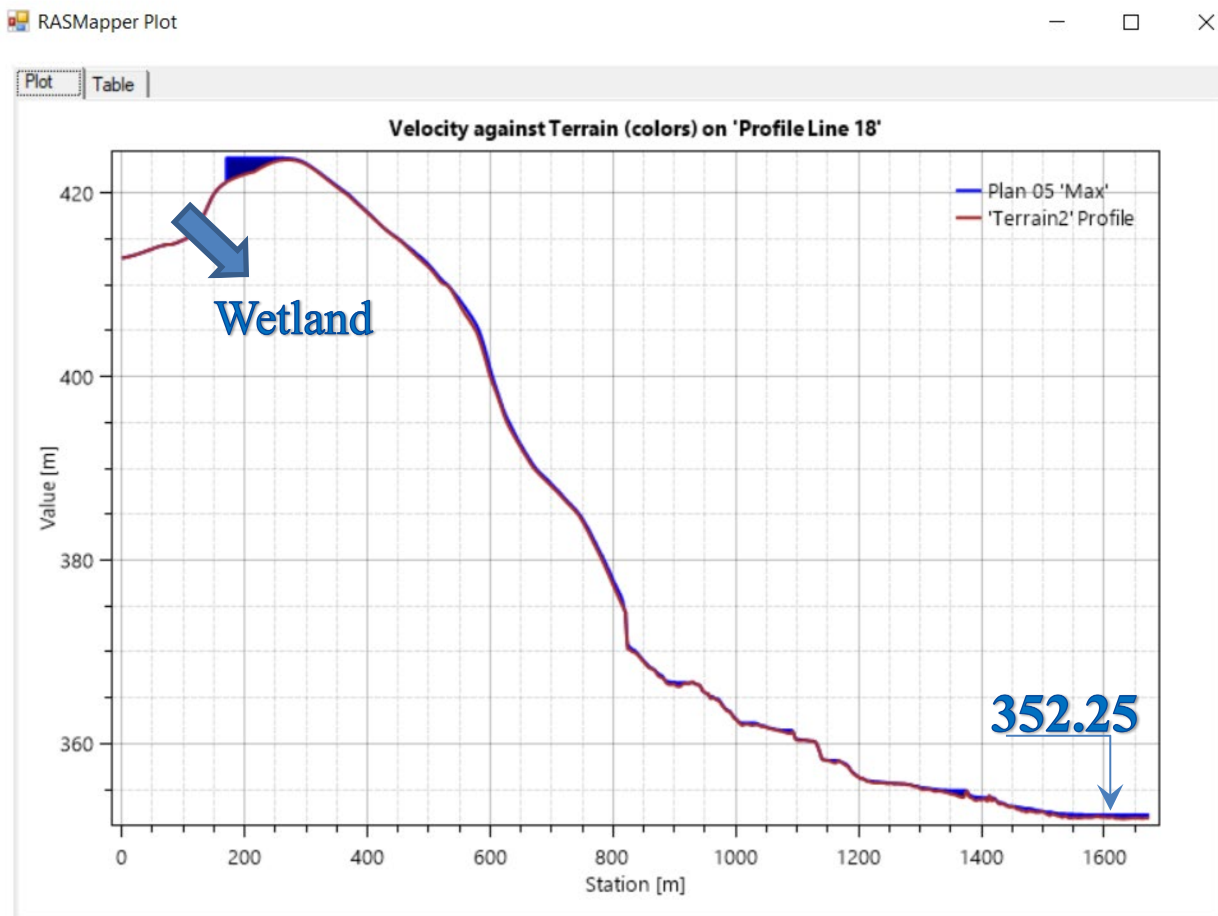


Figure 20- Longitudinal section of West Creek between wetland and lake with maximum water surface elevation (Lake level= 352.25 m)

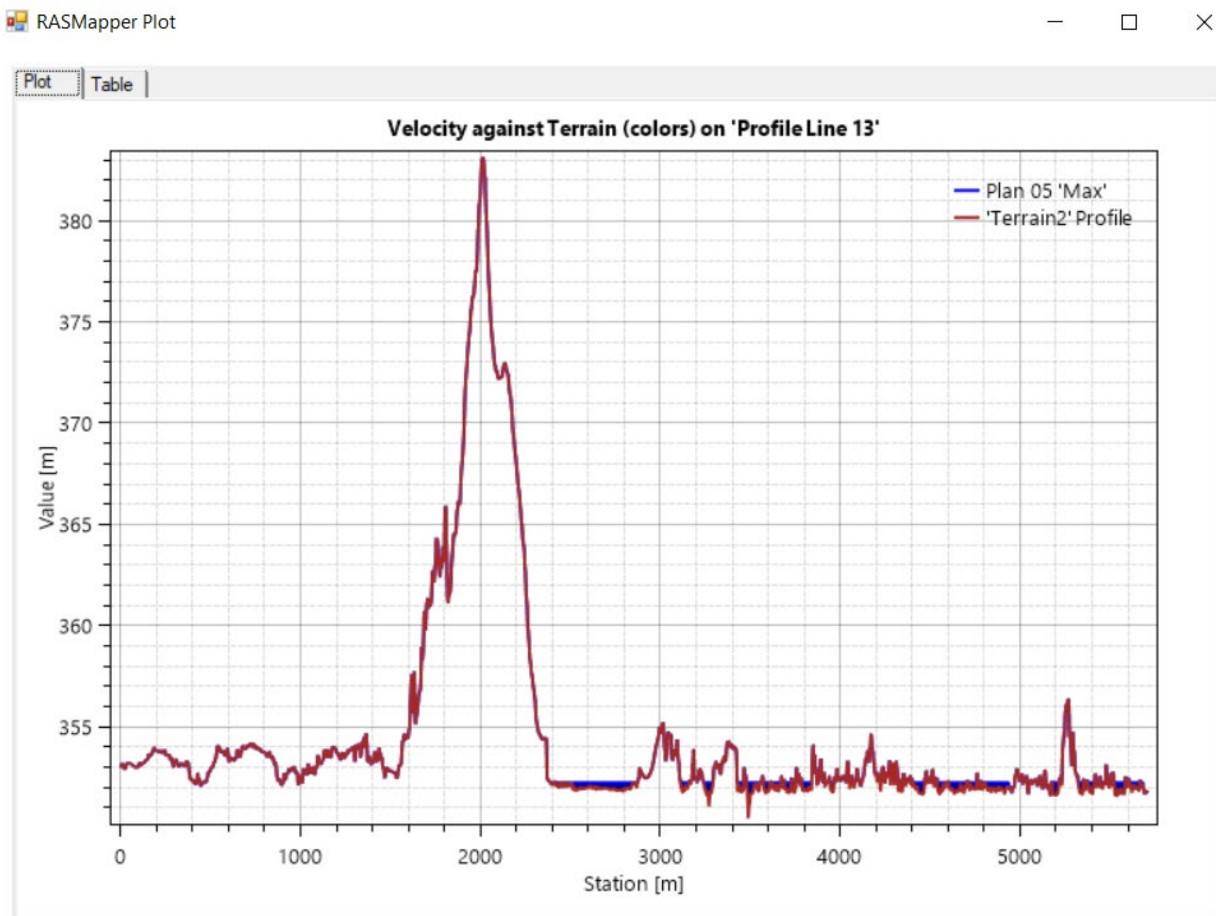


Figure 21- Longitudinal section of shoreline (Lake level= 352.25 m)

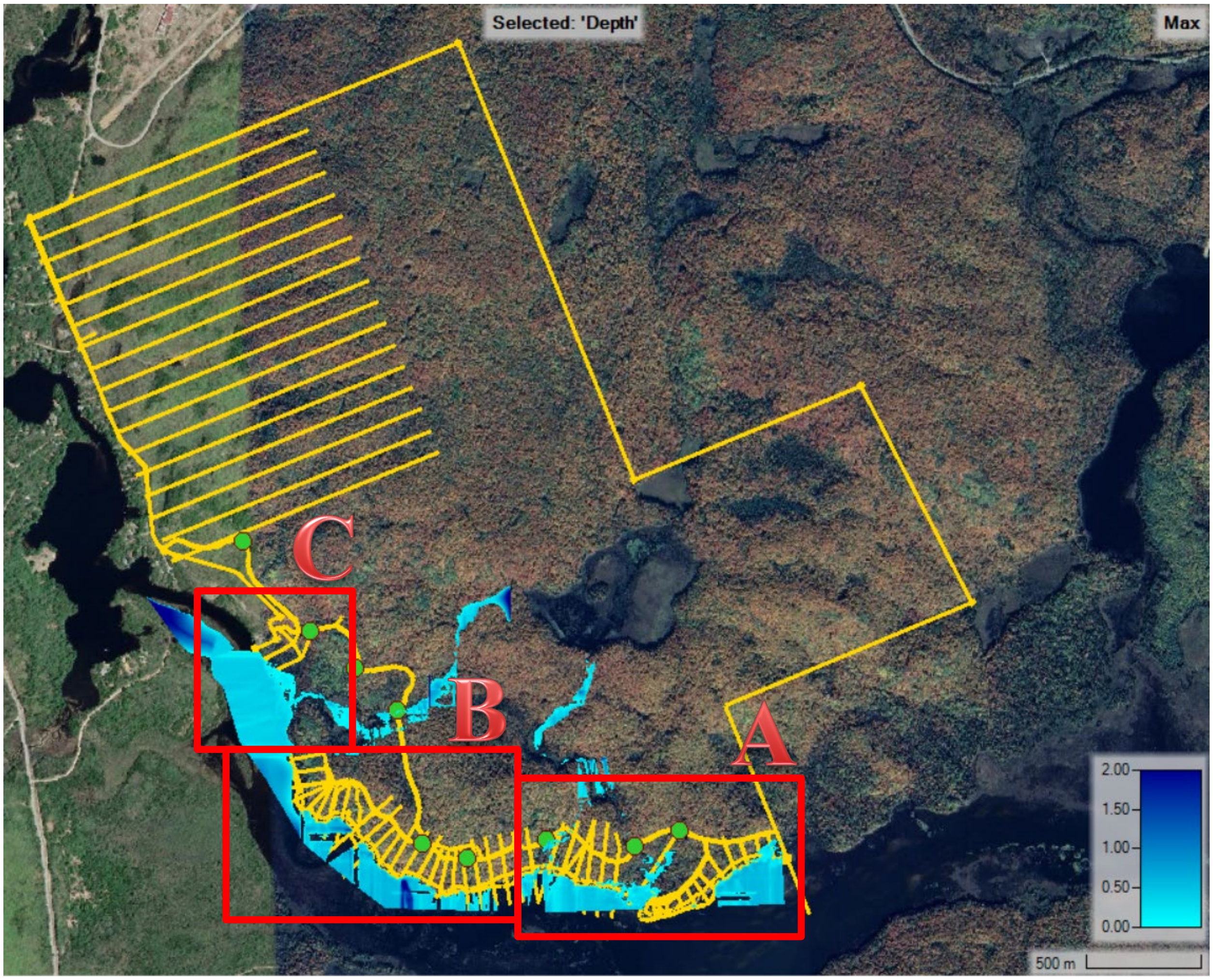


Figure 22- Flooded Area/Lots (100 years event) based on Water Depth (0-2 meter)


45
46
47
50
51
52
53
61
62
63
352.25
354.00
353.64
353.27
352.91
352.54
352.18
351.81
200 m

A


352.25
354.00
353.64
353.27
352.91
352.54
352.18
351.81
200 m

B

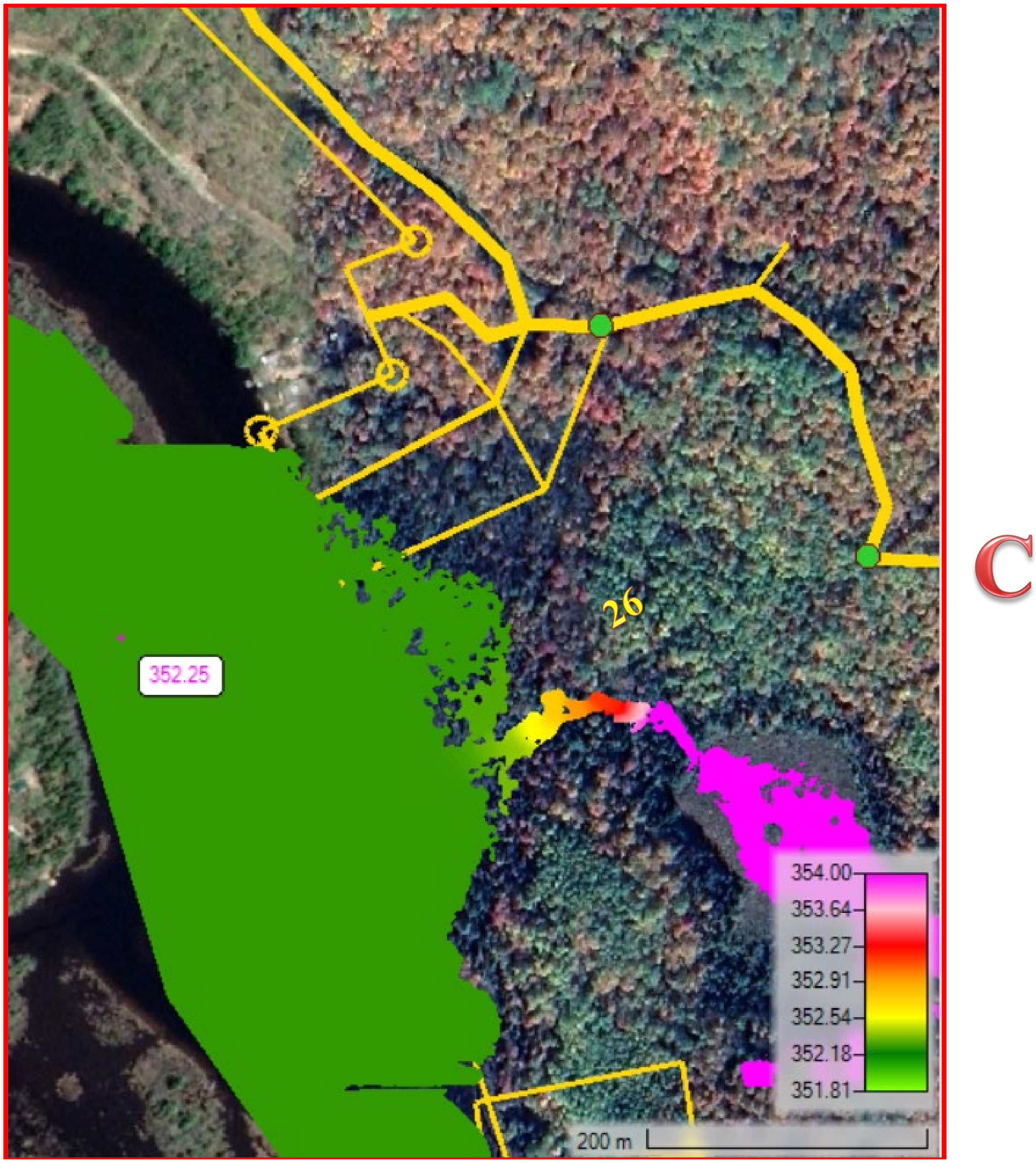


Figure 23- Flooded lots Number (100 years event) based on WSL

8. Reliance & Signature

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Respectfully,

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Senior Engineer – Water Resources
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Yu Tao (Tony) Wang, P. Eng
Principal Engineer
King EPCM

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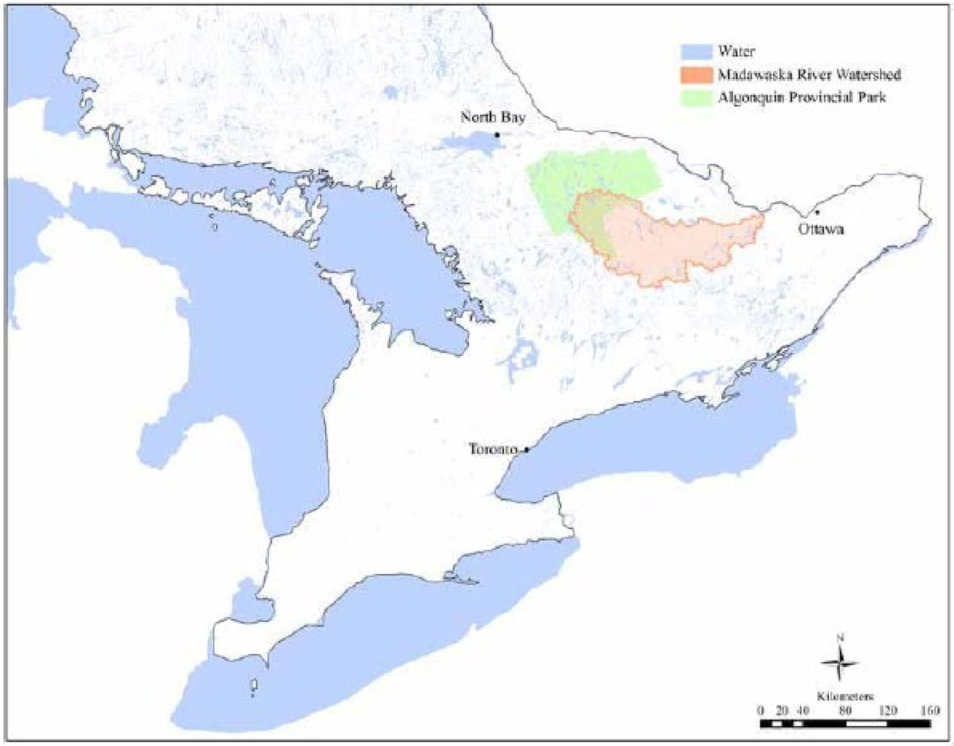
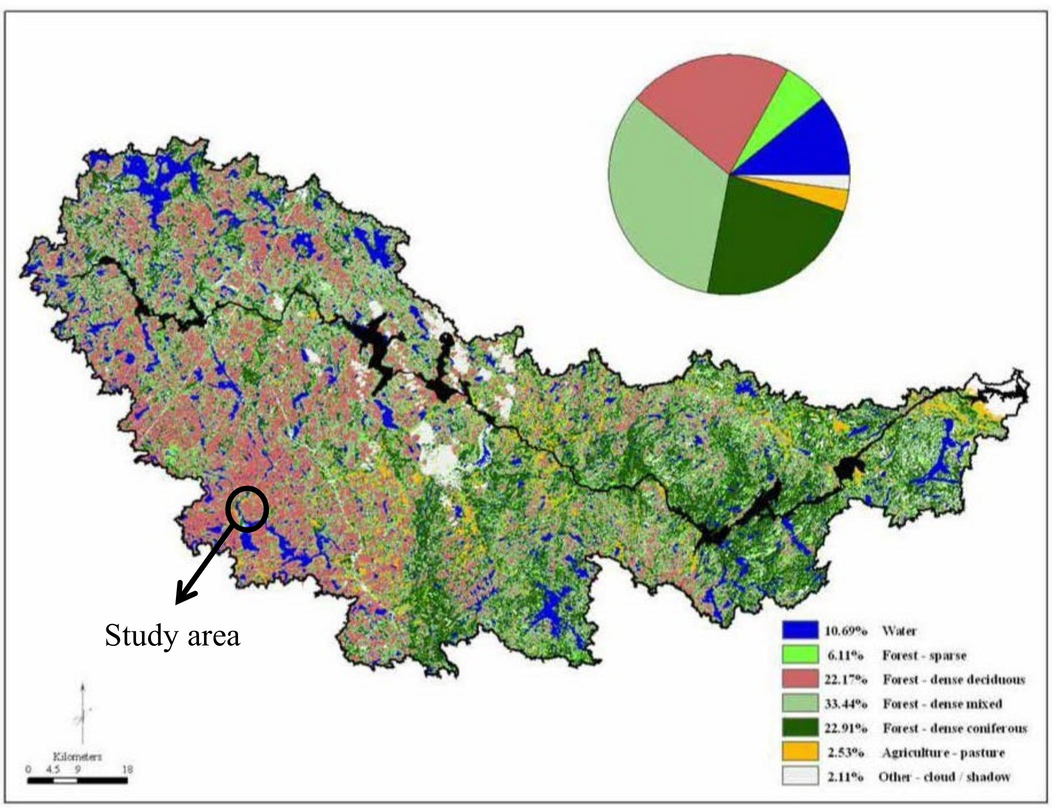
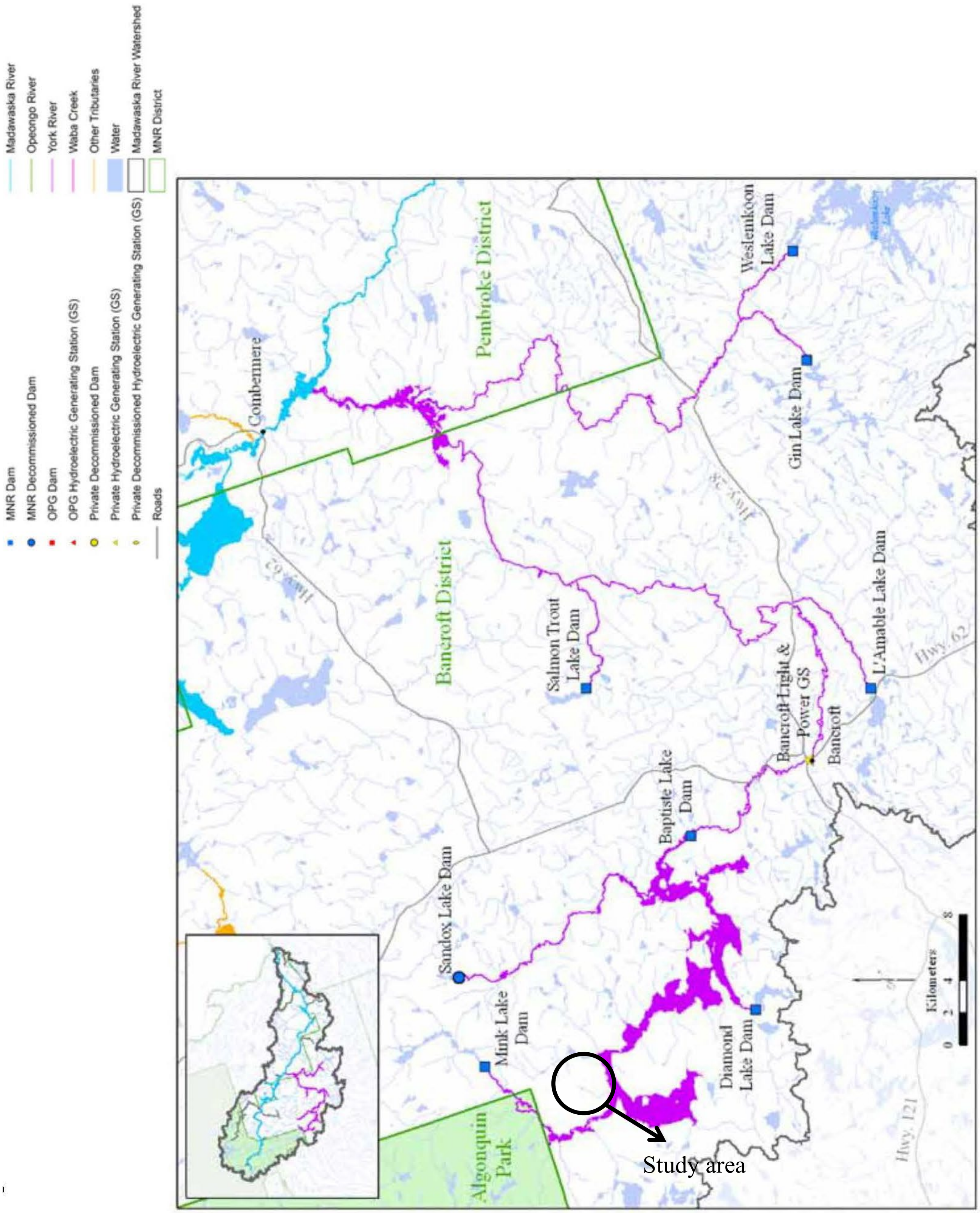
APPENDIX I –Madawaska Watershed & Land cover

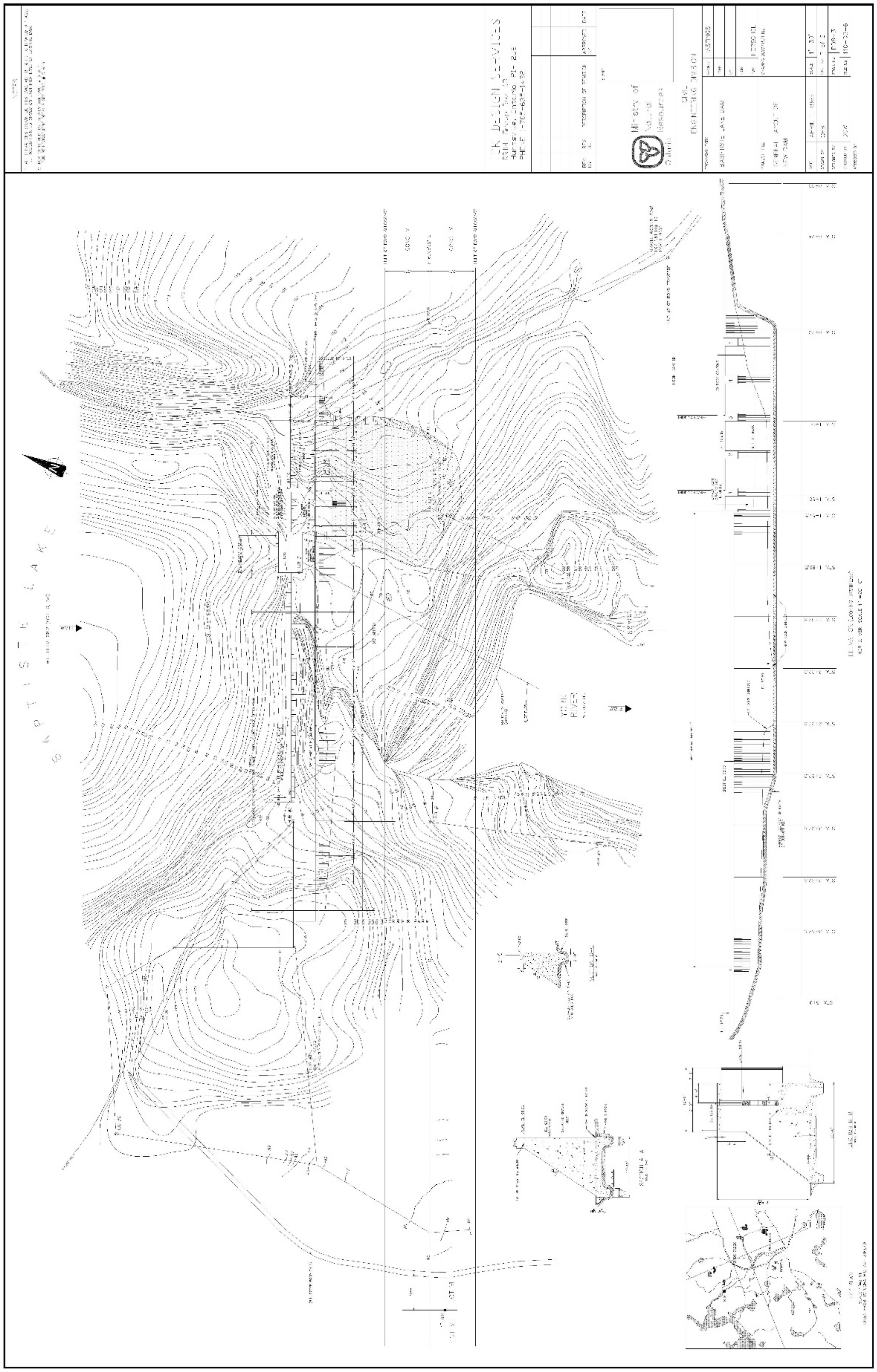

Water
Madawaska River Watershed
Algonquin Provincial Park
North Bay
Ottawa
Toronto
N
Kilometers
0 20 40 80 120 160


Study area
10.69% Water
6.11% Forest - sparse
22.17% Forest - dense deciduous
33.44% Forest - dense mixed
22.91% Forest - dense coniferous
2.53% Agriculture - pasture
2.11% Other - cloud / shadow
Kilometers
0 4.5 9 18

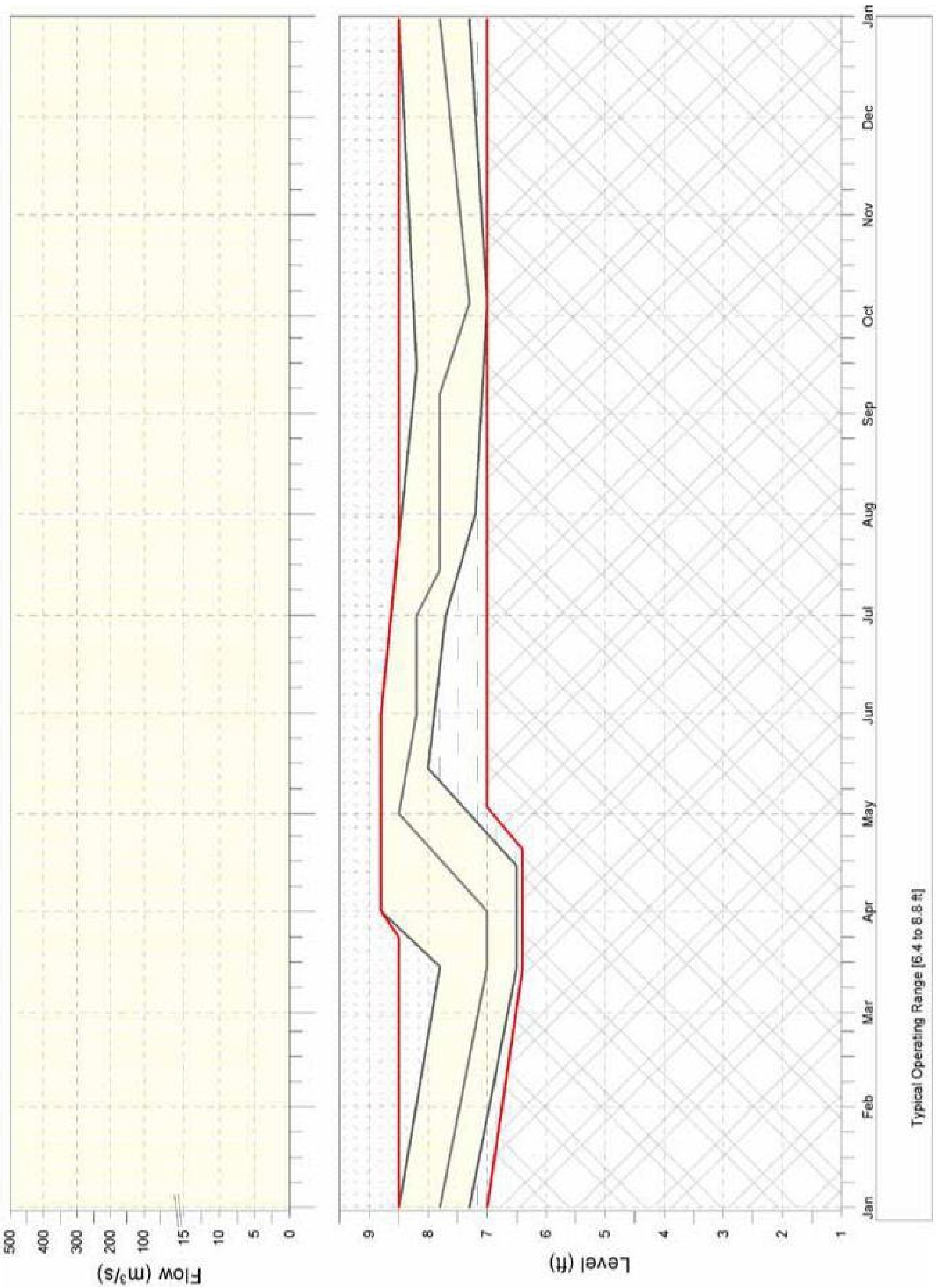
APPENDIX II –Dams on the York River


Madawaska River
Opeongo River
York River
Waba Creek
Other Tributaries
Water
Madawaska River Watershed
MNR District
MNR Dam
MNR Decommissioned Dam
OPG Dam
OPG Hydroelectric Generating Station (GS)
Private Decommissioned Dam
Private Hydroelectric Generating Station (GS)
Private Decommissioned Hydroelectric Generating Station (GS)
Roads
Combermere
Pembroke District
Bancroft District
Weslemkoon Lake Dam
Gin Lake Dam
Hwy. 28
Hwy. 62
Salmon Trout Lake Dam
L'Amable Lake Dam
Bancroft Light & Power GS
Bancroft
Baptiste Lake Dam
Sandox Lake Dam
Mink Lake Dam
Diamond Lake Dam
Algonquin Park
Hwy. 121
Kilometers
0 2 4 8
Study area

APPENDIX III – General Layout of Existing (Baptiste Lake) Dam



APPENDIX IV –Baptiste Lake Operating Band


Flow (m³/s)
500
400
300
200
100
15
10
5
0
Level (ft)
9
8
7
6
5
4
3
2
1
Jan
Feb
Mar
Apr
May
Jun
Jul
Aug
Sep
Oct
Nov
Dec
Jan
Typical Operating Range [6.4 to 8.8 ft]