Port Bruce Riverine and Coastal Floodplain Mapping

Submitted to Catfish Creek Conservation Authority

CATFISH CREEK
CONSERVATION AUTHORITY

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Project No. 2882-021

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1.0 Introduction

The main intent of the project is to update existing Catfish Creek Floodplain Mapping in the community of Port Bruce, Ontario. Port Bruce is located on the natural floodplain of Catfish Creek near its outlet at Lake Erie. Being in close proximity to both Catfish Creek and Lake Erie, Port Bruce is susceptible to riverine and coastal flood hazards. The present study provides an updated technical assessment (using latest data and methods) that quantifies both riverine and coastal flood hazards. The scope of this assignment focuses on technical studies to quantify effects of flooding from river and lake sources.

1.1 Project Background

Original floodplain mapping exercise at Port Bruce was completed in the early 1980's (CCL, 1984), which marked the first time floodlines were established for the community. The original mapping was based on topographic contours derived from orthogrammetry in the 1980's, and are considered outdated by today's standards. Presently, Light Detection And Ranging (LiDAR) data is now available that provides more accurate topographic data of the river and floodplain system compared to topographic data that was available in the 1980's. Given the age of the existing floodplain mapping, the availability of LiDAR topographic data, together with the advancement of hydraulic and coastal modeling provide sufficient justification for updating existing floodplain mapping in Port Bruce.

1.2 Study Area

The study limits in this work include approximately 3.2 km of the Catfish Creek floodplain and about 1.2 km of beach at Port Bruce, Ontario (see Figure 1-1). The study area at Port Bruce is susceptible to flooding from both riverine (Catfish Creek) and coastal (Lake Erie) sources. As such, technical analyses carried out quantify flooding characteristics in terms of extent (how much does flooding extend on a map) and elevations (how high does flooding get).

1.3 Study Scope

The scope of work in this assignment includes technical studies to quantify riverine (Catfish Creek floodplain) and coastal (Port Bruce beach) flood hazards. The study includes delineating the floodplain and establishing floodproofing elevations for a 3.2 km long reach of the Catfish Creek and 1.2 km reach of Lake Erie's beach at Port Bruce. Study requirements include:

- Background review and data collection (historic flooding, previous studies, large scale topographic data, aerial photography, etc.),
- Field investigations (completion of topographic and bathymetric surveys),
- Digital terrain manipulation (merging large scale topographic data with in-river bathymetry),

- Hydrologic assessment (establishing design flows, including taking into account climate change),
- Wave climate assessment (establish design waves propagating from Lake Erie proper),
- Hydraulic assessment (determining flooding inundation limits and flood elevations using numerical modeling),
- Wave uprush assessment (determining wave runup elevations for low lying Port Bruce's beach and inland areas using coastal numerical models),
- Floodplain mapping (developing relevant floodplain maps showing regulatory extent of flooding),
- Floodproofing elevations (identifying top of foundation elevations for developments to be located inside the flood hazard areas), and
- Reporting (summarizing study methodology, findings, recommendations and conclusions).

Note that scope of this report only includes floodplain mapping of the Catfish Creek, and does not include floodplain mapping of the smaller Lake Erie tributaries.

1.4 Horizontal and Vertical Datum

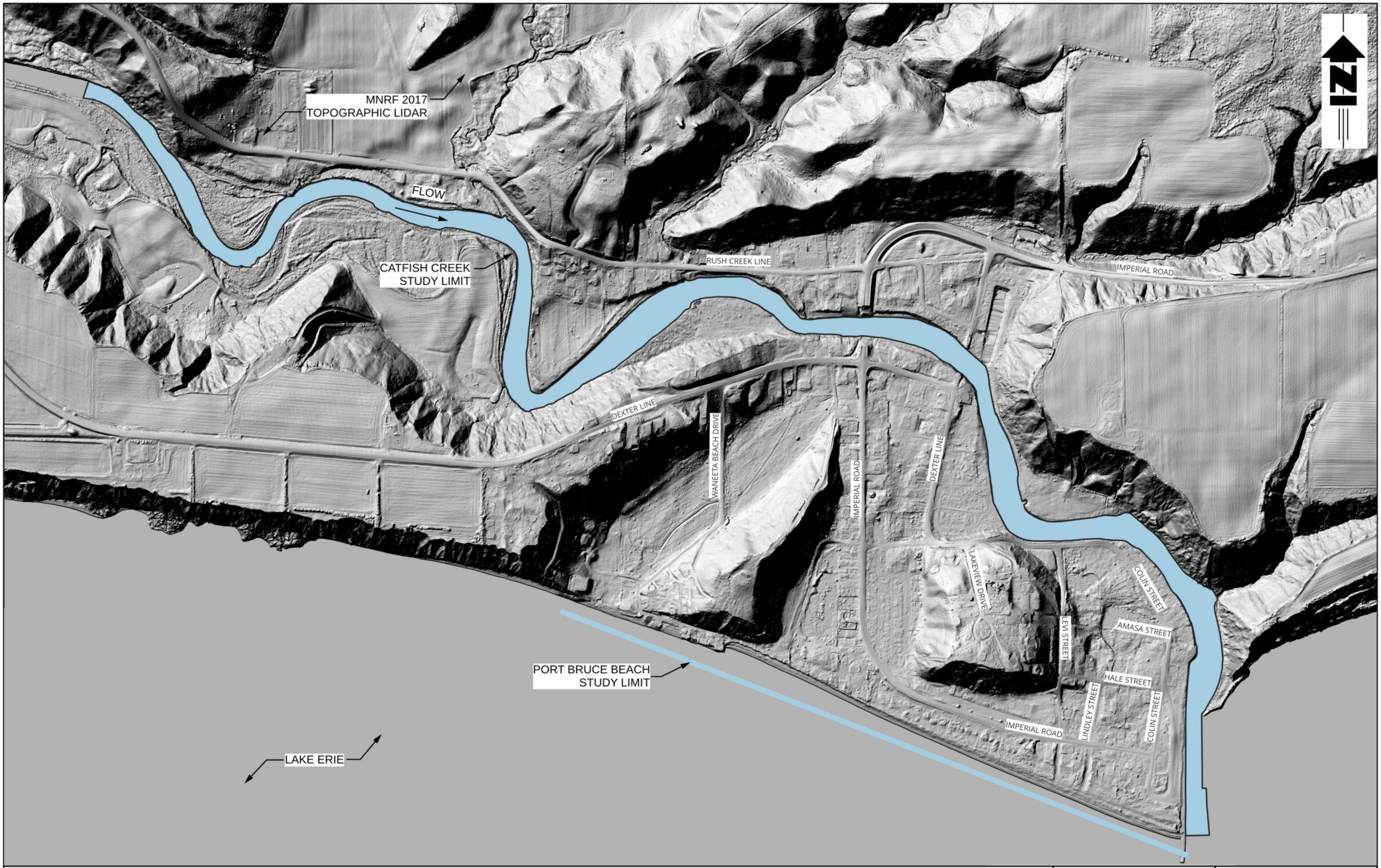
In this assignment the horizontal reference plane used is NAD83(CSRS)/UTM Zone 17N. The vertical datum used is the Canadian Geodetic Vertical Datum 2013 (CGVD2013). All topographic and bathymetric surveys, maps, inundation boundaries, flood elevations and all other references are made to the above noted standard. The project uses SI units, with dimensions reported in meters [m], and discharges reported in meters cubed per second [m^3/s], unless otherwise stated.

Historic riverine flood data has been previously according to Canadian Geodetic Vertical Datum 1928 (CGVD28). For Lake Erie and other Great Lakes water levels are reported according to the International Great Lakes Datum 1985 (IGLD85). For this project, conversions between the three vertical datums is established via a Natural Resources Canada (NRCAN) benchmark 603248, located in Port Bruce (on the west pier). According to the NRCAN benchmark, the conversions are:

IGLD85 = CGVD2013 + 0.468

CGVD28 = CGVD2013 + 0.447

At times in the report references to water surface elevations are made according to the IGLD85 vertical datum. The conversions above provide the reader with means to carry out the conversions to other vertical datums, as required.



STUDY LIMITS

RIVERINE AND COASTAL FLOODPLAIN MAPPING
PORT BRUCE, ONTARIO

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2.0 Background Review

This section documents previous flood studies, historical flood events, and existing flood management infrastructure within the Port Bruce, Ontario. Staff at Catfish Creek Conservation Authority (CCCA) provided much of the historic studies for use in this project. This section summarizes the past studies as they relate to quantification of flood hazards at Port Bruce (riverine and coastal).

2.1 Past Riverine Studies

Several past riverine studies exist at Port Bruce. Each of the past studies are listed below in chronological order, and are briefly summarized for context.

2.1.1 Floodline Mapping Study of the Port Bruce Area (CCL, 1982)

First documented example of floodplain mapping at Port Bruce was completed by CCL (1982). The study provides a historic overview of flooding at Port Bruce, noting that riverine flooding (open water conditions) could occur as a result of inadequate channel capacity, ice jams at the harbour, and generally high lake levels. Hydrologic analysis was carried out using single station frequency analysis methods, and hydrologic modeling. Hydrologic modeling was necessary to quantify the magnitude of the Hurricane Hazel regulatory storm event (referred to as the Regional Storm). Design flows were established for a range of return periods (2-yr to 100-yr) and the Regional Storm. Detailed surveys were completed of the river channel within the study area. Hydraulic analysis using the HEC-2 model (predecessor to the HEC-RAS) was carried out using steady state assumption. HEC-2 was a cross section based hydraulic model, where flow is conveyed unidirectionaly from one cross section to the next. The CCL (1982) study identified a spill area (where the flow exits the main channel and travels overland). Effects of ice jams were considered in the analysis. Floodplain boundaries were mapped, and have been in use ever since by CCCA.

2.1.2 Port Bruce Flood Damage Reduction Study (CCL, 1984)

Building upon their previous work, the CCL (1984) study was initiated to analyze patterns of flooding (open water and ice jams) in more details. The same study also evaluated alternative strategies that could reduce flood damage at Port Bruce. Significant effort was placed on understanding factors that lead to flooding, paying particular attention to causes of ice jams. An economic evaluation of flood hazards was completed, and structural and non-structural flood damage reduction schemes identified and assessed. The main recommendations of the study were to: i) control ice upstream of the harbour, ii) install dyking for low lying areas, and iii) carry out channel dredging to maintain flow capacity. Additional investigations were recommended, and included formulating an ice monitoring program, removing and storing ice, studying impact of offshore ice, and various non-structural alternatives (changes in land use policy, different watershed management, etc).

2.1.3 Port Bruce Flood of February 13-14, 1984 (CCL, 1985)

The CCL (1985) study commented on the ice jam flood that occurred on February of 1984. During that time, high runoff in the watershed combined with ice conditions in the harbour to create widespread flooding at Port Bruce. This study was initiated to examine the factors that contributed to the flooding. The authors examined meteorologic (temperature, precipitation, etc.), hydrologic (flow hydrographs) and hydraulic conditions (ice jams, water levels in lake and river). Recommendation were provided on then current CCCA ice observation and warning/advisory bulletin systems, and findings were presented on effectiveness of tug and land based construction equipment as ice management measures. The main recommendations from the study were to: i) break up the ice on a regular basis, ii) upgrade the observation program, iii) install ice control structure upstream of harbour, and iv) install gauges and monitor low flow conditions.

2.1.4 Port Bruce Harbour Flood Control Project (Crook, 1997)

The Crook (1997) report summarizes a Class Environmental Assessment for Remedial Flood and Erosion Control Projects. The then proposed project included placement of erosion protection on the east side of the creek to avoid further erosion of the clay bluff on east side of the harbour. Said bluff erosion, if allowed to continue, would place eroded bluff materials inside the main channel which would ultimately reduce its flow carrying capacity and cause more flooding. The study documents analyses and evaluation of several erosion protection alternatives. The recommended (and ultimately carried out) alternative included installation of a riprap revetment to address bluff erosion. A comprehensive environmental impact assessment was document in the study, for the purposes of obtaining necessary regulatory approval for the project.

2.1.5 Catfish Creek Watershed Hydrologic Model (Shroeter, 2006)

The purpose of the Shroeter (2006) study was to construct a hydrologic model of the 11 major watershed within CCCA's administrative boundaries. The model included 75 sub-catchments, 53 channel routing reaches, two reservoirs, and one sewage treatment plan and was developed using the GAWSER modeling framework. The modeling was carried out in response to Province's Source Water Protection legislation, required after contamination of drinking water resulted in fatalities in Ontario. A hydrologic model, quantifying flow characteristics in its study watershed, in terms of floods and low flows, is an essential tool for Source Water Protection planning and management. The modeling carried out included water balance quantities (precipitation, evapotranspiration, runoff, baseflow, storage, and total flow), groundwater recharge amounts, flow quantiles (return periods of high and low flows), flow duration information, and water withdrawal rates. To date, the GAWSER model represents the most comprehensive hydrologic modeling carried out in the CCCA's watersheds.

2.2 Past Coastal Studies

Three past coastal studies are available at Port Bruce. Each is listed chronologically, and briefly summarized for context.

2.2.1 Shoreline Management Plan (Philpott, 1991)

Philpott (1991) study summarizes the preparation of a shoreline management plan for Lake Erie shoreline within CCCA's administrative boundaries. The intent of the plan was to balance options of shoreline prevention, protection, environmental impact, monitoring, emergency response, and public education in shoreline management. The main outcome of the shoreline management plan was development of regulatory shoreland zone where development activities would be restricted (due to high risk of coastal flood and erosion hazards). The regulatory shoreland is defined as the farthest landward limit of a) regulatory flood line, b) regulatory erosion line, and c) regulatory dynamic beach limit. Development inside the regulatory shoreline was set as either prohibited, or restricted, as per Provincial regulations. For the beach at Port Bruce, 100-yr active beach width was defined, along with wave uprush elevations for various shoreline types (dunes, sloped structures, vertical walls).

2.2.2 Port Bruce Harbour Hydrologic Study (Triton, 1994)

The Triton (1994) study was completed as a Class Environmental Assessment study for the purpose of assessing hydrologic problems at the Port Bruce harbour. The concerns included flooding from ice jams, streambank erosion, lake shoreline erosion of the east bluff resulting in reduction of channel capacity at the harbour mouth, sediment deposition at the harbour, among others. The study focused on the harbour at Port Bruce and issues at the mouth of Catfish Creek. General problems were identified, and related issues documented. The main intent of this work was to consider problem areas and review inter-relationships between problem areas and their resulting effects on the overall system. The study looked at geology, environmental and social conditions, water levels, waves and coastal climate of Lake Erie, river hydraulics, and ice jams. A comprehensive plan of looking at 21 different options were presented and evaluated in terms of costs and overall benefit to the community. The recommendations were presented in two categories. The main (priority 1) recommendations were to: i) carry out periodic dredging, ii) provide erosion protection at the east bluff, iii) encourage sediment control in the watershed, iv) continue ice jam removal from the harbour. The priority 2 recommendations were to v) install wave baffles on west jetty on a trial basis, vi) replace east jetty with stone revetment, vii) install ice retention upstream of town, viii) install a flood diversion upstream of Highway 73 bridge.

2.2.3 Port Bruce Sedimentation Study (Riggs, 2012)

Riggs (2012) study was initiated to undertake a sediment analysis and assess sedimentation impacts of the mouth of Catfish Creek in Port Bruce. The initial concern was to investigated the potential of Lake Erie's littoral drift material migrating from the lake and depositing at the harbour entrance, which can restrict channel capacity and cause upstream flooding. The patterns of sedimentation from littoral (from lake) and fluvial (from river) sources were considered in the study. The study found strong evidence that littoral drift is by-passing the west pier completely, and is not being deposited at the harbour entrance channel. Majority of the sediment deposited in Port Bruce resulted from upland sources, which were evaluated using measured channel bathymetric surveys from several years and preliminary numerical modeling. Findings of the assessment included that Catfish Creek at Port Bruce is in a state of dynamic morphologic equilibrium, implying that main channel adjusts itself based on incoming sediment, hydrologic flow regime, lake levels, and upstream erosion rates.

3.0 Field Data Collection

This section documents the field data collection activities undertaken as part of this assignment. Callon Dietz Surveyors carried out all topographic and bathymetric surveys in this project. All topographic/bathymetric survey efforts used Global Navigation Satellite System (GNSS) receivers for field measurements. The surveys were performed with a Real Time Kinematic RTK-GNSS unit. Instrument accuracy was 10 mm horizontal and 20 mm vertical or better. Elevation ground proofing was obtained by physically occupying a locally established benchmark. Vertical datum used in the data collection efforts was CGVD2013, and conforms with project requirements and LiDAR data (documented subsequently).

All survey related work was carried out over a period of two days in May of 2023. The survey data collected by Callon Dietz is shown in Figure 3-1.

3.1 Bathymetric Survey of Catfish Creek

For the reach of Catfish Creek influenced by Lake Erie backwater conditions, bathymetric soundings were collected as cross sections of the main channel within the riverine study area. The data collection was accomplished using an eco-sounder mounted on a small boat, and connected to an RTK-GNSS unit for horizontal positions. A total of 40 river cross sections were collected in the 3.2 km study reach of Catfish Creek at Port Bruce. Using such a large number of surveyed river cross sections ensured that geometry of the riverbed (i.e., channel capacity) is appropriately represented for use in hydraulic modeling and floodplain mapping work.

The newly constructed Imperial Road Bridge is the only stream crossing within the study area. The Callon Dietz survey crew visited the bridge, who collected the following data.

- Photograph of bridge opening,
- Top elevation of the bridge deck,
- Measurement from the bridge deck to the underside of the soffit,
- Elevations of the creek at water’s edge (left bank), toe of slope (left bank), a number of points in the main channel, toe of slope (right bank), and water’s edge (right bank),
- Dimensions of structure opening, and
- Number and size of piers.

Limited number of land based topographic points were collected at, and around, the bridge approaches. This was required as the Imperial Road Bridges approaches were constructed slightly higher than the previous bridge. Simply relying on LiDAR topography (which was collected in 2017 and prior to the new bridge being in place) would have misrepresented actual conditions at the site.

3.2 Topo-bathymetric Surveys of Beach and Nearshore

To accurately assess lakeshore flooding and its impacts, detailed bathymetry is required to estimate propagation of waves to the beach and wave uprush (how high will the waves runup the slopes and/or on existing structures).

Callon Dietz survey crew collected a total of eight lake transects, with each transect being approximately 500 m long. As before, survey data collection was accomplished using an eco-sounder mounted on a small boat and linked to an RTK-GNSS unit for horizontal positions. For several of the transects land based topographic survey data of the existing beach, dune system, and upland areas were also collected. The land based topographic data was used to confirm the accuracy of the 2017 MNRF LiDAR data set.

DATA COLLECTION

RIVERINE AND COASTAL FLOODPLAIN MAPPING
PORT BRUCE, ONTARIO

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4.0 Digital Terrain Processing

LiDAR derived digital terrain models are nowadays used for hydraulic modeling as they efficiently capture geometry of the terrain for large areas. However, the LiDAR sensors are not able to penetrate sufficiently through the water's surface, thus resulting in reduced accuracy for the terrain surface below the water line. Geometry of the terrain under the water's surface is thus not captured using typical LiDAR products, but is required for accurate assessments of river hydraulics for floodplain mapping purposes.

This section outlines the methodology that combines LiDAR derived Digital Elevation Model (DEM) with digital terrain models and DEMs derived from topographic and bathymetric surveying. The combining of LiDAR with the survey derived DEMs are used to construct a hydraulic model ready DEM. The end product thus includes a digital surface accurate for both above and below water portions of the river and is used in all subsequent hydraulic modeling.

4.1 LiDAR Digital Elevation Model

The publicly available 2016-18 MNRF Lake Erie LiDAR data set was used in this assignment (MNRF, 2023). For the area around Port Bruce, the LiDAR data was collected in 2017, and thus the LiDAR data in this study area is referred as the 2017 MNRF LiDAR data. LiDAR data includes a Digital Elevation Model (DEM) having a horizontal resolution of 0.5 m x 0.5 m. The vertical datum of the MNRF LiDAR product is CGVD2013, and is consistent with the bathymetric and topographic data gathered during the filed data collection efforts.

The topographic survey within the study area was used to compare elevations between data collected using survey grade instrumentation and the LiDAR DEM product. In areas where the two sources of data overlapped, comparisons showed that on the ground measurements of elevations are consistent with the LiDAR DEM product, thus providing confidence in use of the LiDAR DEM elevations.

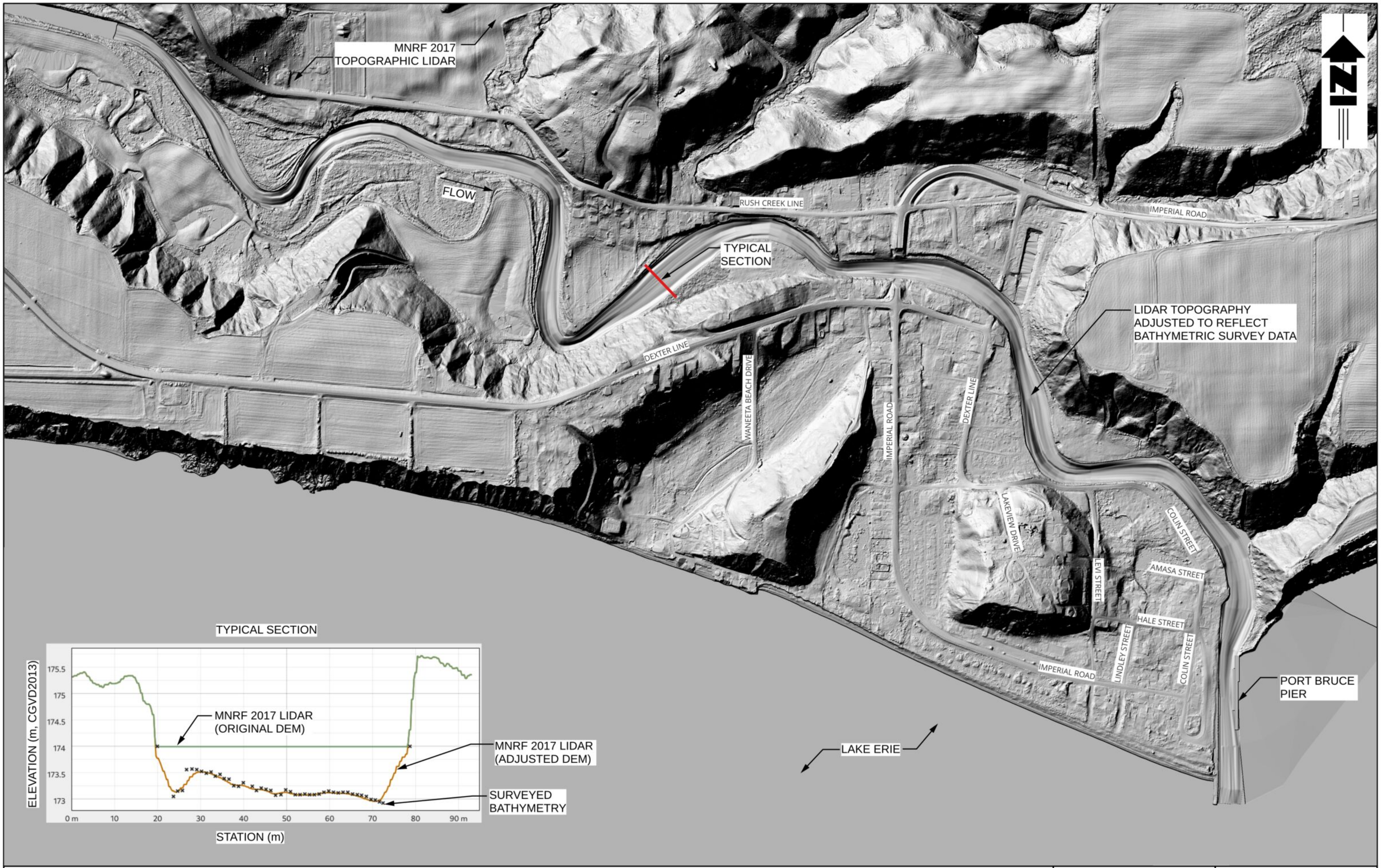
4.2 Merging Topographic and Bathymetric Surveys with LiDAR

For the Catfish Creek the bathymetric soundings collected during the field campaign was used to create a Triangulated Irregular Network (TIN) model, and then convert it to a 0.5 m in-stream DEM. A customized procedure, similar to one provided by Merwade et. al. (2005), was used to transform the river alignment and the bathymetric survey from a Cartesian to a curvilinear orthogonal system. The reason for the coordinate transformation is that construction of a TIN surface using cross section based river bathymetry is much simpler in the curvilinear orthogonal system than in the Cartesian system. After construction of the TIN surface in the curvilinear orthogonal system was completed, the surface was converted back to the Cartesian system, and used to construct an in-stream only 0.5 m DEM. The in-stream only DEM uses only the surveyed bathymetry data, which was used to define the underwater geometry of the main channel.

Manual adjustments in some parts of Catfish Creek (i.e., under heavy vegetation, and at Port Bruce Pier) was required to ensure consistency in the overall digital surface to actual conditions.

The 0.5 m DEMs representing in-stream bathymetry, and manual adjustments were “burned into” (or merged with) the large scale 0.5 m LiDAR DEM ultimately producing a hydraulic model ready product that accurately captures all above and below water terrain of the river and banks (required for accurate floodplain modeling). The merged digital surface (consisting of LiDAR derived ground surface, surveyed topography and bathymetry, as well as manual adjustments) include the best available geometric data for the study area.

Figure 4-1 shows the final hydraulic model ready DEM. A typical cross section is shown in an inset of Figure 4-1 that shows the original 2017 MNRF DEM surface, the surveyed bathymetry, and the 2017 MNRF DEM surface that was adjusted using the bathymetric survey.



HYDRAULIC MODEL READY DEM

RIVERINE AND COASTAL FLOODPLAIN MAPPING
PORT BRUCE, ONTARIO

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5.0 Riverine Flow Characterization

This section provides a general descriptions of the study watershed, completes a flow characterization assessment (documenting the methodology used to establish design flows), presents a preliminary climate change analysis, and provides a summary of the design flows used for the floodplain mapping update.

Catfish Creek watershed encompasses a catchment area of 395 km^2 (calculated using Ontario Water Information Tool, OWIT), with creeks flowing generally in the southerly direction towards Lake Erie. The catchment area above does not include Lake Erie trubutaries, which are part of the CCCA administrative boundary. Majority of the Catfish Creek watershed is agricultural, with smaller land use fraction consisting of forest cover and urban use areas. Catfish Creek has incised into the surrounding land mass over geologic time, and forms the main channel conveying flow from its upland areas to its outlet at Lake Erie in Port Bruce.

Hydrologic modeling using recent data and methods were not part of the scope of work of this assignment. Instead, this study relies on results from single station frequency analysis, regional flow assessment techniques and previous modeling to establish flows. Previous hydrologic modeling work carried out in CCL (1982) were used to quantify Regional Storm peak flows, and values from that study were adopted (and adjusted) for use in this work. As the CCL (1982) study was commissioned specifically to prepare the original floodplain mapping, its use is believed to be justified in this assignment. More recent hydrologic modeling work by Shroeter (2006) was also used to extracted computed flows, which were compared the original modeling work by CCL (1982).

5.1 Single Station Frequency Analysis

Single station frequency analysis of the Water Survey of Canada streamflow gauge of Catfish Creek at Sparta (id. 02GC018) was carried out. Annual maximum instantaneous flows were extracted from the historic record from 1965-2022 and fit to common statistical distributions used in hydrology (Log Normal 3, Log Pearson 3, and Generalized Extreme Value). The computations were carried out using the Method of Moments and method of L-Moments to estimate parameters of the distributions. The Generalized Extreme Value Distribution, with parameters estimated using L-Moment was ultimately selected in this work, as this distribution is recommended for use in Canada based on recent studies (Zhang et al, 2019). Results of the statistical analysis at the Sparta gauge is shown in column (2) in Table 5-1.

As there is an increase in drainage area from the gauge at Sparta to Port Bruce, the flows require adjustment to take the additional drainage area into consideration. The technique of flow scaling was used in this work. The scaling was used to determine flow at the outlet (with the larger drainage area) using the scaling relationship recommended in the Ministry of Transportation of Ontario Drainage Management Manual (MTO, 1997). The results of the flow scaling at the outlet at Port Bruce are provided in column (3) in Table 5-1.

5.2 Regional Flow Assessment

To further justify flows used at the outlet of Catfish Creek, regional analysis using the Unified Ontario Flow Method (UOFM), summarized in MTO (2016) was used to check the flow scaling results provided in Table 5-1 (note that not all return periods are available from the UOFM method). The UOFM uses watershed drainage area at the location of interest, along with total annual precipitation and areas of wetlands as physical parameters in its regional regression relationships. The regression relationships were used to obtain flow estimates for peak flows ranging from 2-yr to 100-yr using the UOFM. The results of the regional analyses are shown in column (4) in Table 5-1.

Interpretation of the results indicate that flow scaling technique used in this work is appropriate, and compares favourably with scaling from single stations statistical analysis.

Note that the regional flow assessment only provides flows for quantiles up to 100-yr return period, but does not provide peak flows associated with the Regional Storm (Hurricane Hazel). The Regional Storm values can only be obtained via hydrologic modeling.

5.3 Previously Quantified Regional Flows

Previous studies (CCL, 1982; Shroeter, 2006) have completed hydrologic modeling of the Catfish Creek watershed, and have quantified Regional Storm peak flows. Carrying out hydrologic modeling was not part of the scope of work in this study. As a result, previous estimates of Regional Flows were used.

CCL (1982) have carried out hydrologic modeling, and estimated peak flow for the Regional Storm (documented in column (5) in Table 5-1). Similarly, peak flows associated with the Regional Storm was also established by Shroeter (2006), and is shown in column (6) of Table 5-1.

As previous floodplain mapping at Port Bruce used the Regional Flow values in CCL (1982), a decision was made to retain this value as a starting point in this work. One reason for this decision is to allow for comparisons between 1982 and 2024 flows. Further, CCL (1982) Regional Flows were established by a qualified engineering consultant for the specific purpose of delineating floodlines. The Shroeter (2006) study had a much broader scope, and its focus was not solely the determination of the Regional Flows, but also for drought related flows, groundwater, water balance, etc.

The climate change adjustment factor (considered in this work), will apply a factor and assess changes to the Regional Storm floodlines.

5.4 Summary of Flows

Summary of flows is provided in Table 5-1, and includes: a) the gauge statistics (using 1965-2022 data range) in column (2), b) scaling statistics to the outlet at Port Bruce, shown in column (3), c) results from regression analysis using the UOFM in column (4), d) flows quantified from CCL (1982) in column (5), and e) flows quantified in Shroeter (2006) in column (6).

Table 5-1: Flood Frequency Analysis

Return Period [yrs] (1)	Q [m^3/s] @ Sparta (gauge, GEV) (2)	Q [m^3/s] @ Outlet (scaled) (3)	Q [m^3/s] @ Outlet (UOFM) (4)	Q [m^3/s] @ Outlet (CCL, 1982) (5)	Q [m^3/s] @ Outlet (Shroeter, 2006) (6)
2	108.1	134.8	96.0	-	110.0
5	148.4	185.0	-	-	158.0
10	171.6	214.0	169.5	-	191.0
20	191.6	238.9	-	-	223.0
50	214.5	267.5	233.5	-	264.0
100	229.7	285.6	261.3	267.4	298.0
Regional	-	-	-	672.7	764.0

5.5 Design Flows

Inspection of the results from Table 5-1 shows that the flows at Port Bruce for the 100-yr event have changed from 267.4 m^3/s in 1982 to 285.6 m^3/s in 2024. The increase in peak flow is about 7%. As a result, the peak Regional Flow estimated in 1982 is recommended to be increased by the same factor of 7%.

As final design flows in this assignment, flows in column (3) are used as design flows for return periods ranging from 2-yr to 100-yr. The Regional Flow used is a scaled version of the CCL (1982) flow, which works out as 672.7 m^3/s x 1.07 = 719.8 m^3/s. The scaled value closely resembles the value determined in Shroeter (2006). The design flows used in this report are listed in Table 5-2.

Table 5-2: Design Flows at Port Bruce

Return Period [yrs]	Q @ Outlet [m^3/s]
2	134.8
5	185.0
10	214.0
20	238.9
50	267.5
100	285.6
Regional	719.8

5.6 Climate Change Adjustments

Current version of the Provincial Policy Statement notes that Ontario should prepare for impacts resulting from climate change, which may increase the risk associated with natural hazards. Impacts of future climate change on magnitude and frequency of flood flows within the Catfish Creek watershed has not been assessed in detail. Climate change assessment is a more involved exercise that requires generating appropriate hydrometeorological inputs and running hydrologic process models to obtain flow characteristics under future conditions.

Recently published procedure by the Province using temperature scaling allows the water resources practitioner to use outputs from Canadian Global Climate Models and determine how much design rainfall is anticipated to change in a future climate. In essence, Provincially developed procedure is intended to be used to develop climate adjusted rainfall (climate adjusted 2-yr to 100-yr, and climate adjusted Hurricane Hazel Regional Storm). The response of the climate adjusted rainfall must be simulated via an existing hydrologic model to ultimately identify anticipated changes to flow characteristics from climate change.

A hydrologic model is needed to determine changes to peak flows resulting from climate change. The most recent hydrologic model (Shroeter, 2006) is not available for the task, as the source code of the GAWSER model was developed and maintained by the author, who has since retired. None presently exist that could efficiently update and use the GAWSER model for routine tasks as re-running different magnitude storms.

Given that hydrologic modeling is not presently available, this work applies a factor of 15% to peak flows to represent possible influence of climate change within the time horizon representing mid century (2050's). Change factors ranging from 10-20% are commonly used in British Columbia (EGBC, 2018) even when large scale hydrologic modeling has been completed and are available.

It is recognized that temperature scaling techniques for the study area have peak rainfall factors for the mid and end of century time horizons in excess of the above cited change factor. However, until a detailed climate change impact assessment is carried out, a 15% increase in peak flow shall be applied as a preliminary climate change factor for this assignment.

Floodplain mapping in this work is completed using flows with and without consideration of climate change. Note that floodlines estimated using the 15% factor should be considered preliminary (for information purposes only) until such time as more detailed climate change studies are undertaken or become available.

6.0 Hydraulic Modeling

This section focuses on hydraulic modeling and provides details on data and analytical tools used in the assessment. Hydraulic models are analytical tools that evaluate characteristics of movement of water over time and space. The hydraulic models use existing geometry of river/floodplain with specified design flows to determine water surface elevation profiles and inundation depths/extents for a river reach in question.

Hydraulic modeling in this assignment was completed using both 1D and 2D numerical modeling. The 2D analyses allow for accurate assessment of spatial and temporal characteristics of flooding processes, and its resulting overland flow inundation patterns in greater detail than older 1D analyses. Ice jam analyses and its evaluation of impacts are assessed using 1D hydraulic modeling, as ice analyses have not yet been fully implemented in 2D hydraulic models.

6.1 Model Description

The hydraulic analysis carried out in this assessment uses the Hydrologic Modeling Center's River Analysis Systems (HEC-RAS), developed and maintained by the US Army Corps of Engineers. The HEC-RAS model is currently the standard hydraulic model widely used in North America and beyond. HEC-RAS allows its users to carry out 1D and 2D river hydraulic analyses, using steady or unsteady techniques. Depending on the type of analysis required different variants of the models were used. Version 6.4.1 of the HEC-RAS model was used in this work, as it was latest at the time of this writing.

Implicit in 1D hydraulic models are approximations that allow river flow to travel unidirectionally from one cross section to the next, which may not always be accurate in cases of wide and shallow floodplains where overland flow patterns govern flow hydraulics. In such cases use of 2D hydraulic modeling is better suited to capture physics of the flow.

In this work 1D and 2D variant of the HEC-RAS hydraulic model were used to quantify detailed behaviour of the hydraulics within the study area. 1D model variant was used to capture the behaviour of the ice jam dynamics, as this type of analyses is not yet available in 2D modeling. For all other work, 2D model variant was used as it is considered more accurate in capturing physics of the flow, but is also more computationally demanding.

The ability of the 2D model to capture river and floodplain hydrodynamics makes it ideal for the study where 2D effects dominate (such places where flow is suddenly released into relatively flat areas, such as downstream of the Imperial Road Bridge where overland spills occur). HEC-RAS 2D model uses the theory of sub-grid finite volumes to solve the governing flow equations and capture flow dynamics. 2D models uses a large number (in the tens or hundreds of thousand) of discrete elements to represent the geometry (river and floodplain) of the study area. Using a large number of elements allows for capturing geometry of the physical system with high degree of accuracy, especially when the goal of the assignment is to evaluate flow paths, depths, velocities and spill characteristics of flow areas resulting from passage of large flood events. The advantage of 2D modeling is that a range of flood flows (from small to extreme) can be assessed in time and space, while making a minimum number of assumptions.

By definition 2D hydrodynamic models are depth-averaged, implying that computations of flow velocity are averaged along the water column. For relatively shallow flows and wide flooded areas capturing vertical velocity is not necessary to represent the essence of the problem under consideration.

Required data for hydraulic modeling includes:

a) Terrain surface that captures key geometric features within the river and floodplain (i.e., hydraulic model ready DEM),

b) Model grid that discretizes the study area into a large number of computational elements (2D models) or model cross sections (1D models),

c) Hydraulic structures (bridges, culverts, weirs, dikes, etc.),

d) Initial and boundary conditions (flows and levels), and

e) Manning's roughness coefficients for the main channel and the overbank areas.

For the assessment of ice jams and its effects, 1D steady state variant of the HEC-RAS model was used.

6.2 Model Development

HEC-RAS hydraulic modeling was used to develop simulation models for this work. One distinct 2D modeling domain was developed for the Catfish Creek from the outlet at Lake Erie to approximately 3.2 km upstream (as measured along the centerline of main channel). The same extent was used for the 1D (ice jam) model as well.

6.2.1 Digital Surface Data

The hydraulic model ready Digital Elevation Model (DEM) was used as the basic terrain surface data for the hydraulic modeling work. The terrain surface incorporated surveyed river bathymetry, thus accurately capturing geometry of both the river bed and the floodplain areas. Development of the hydraulic model ready DEM surface is presented in detail in Section 4.0, and is depicted graphically in Figure 4-1.

6.2.2 Hydraulic Roughness

Hydraulic roughness in terms of Manning's coefficient was derived using South Western Ontario Orthorectification Project SWOOP2020 aerial photography within the study areas. Values used in the modeling were based on typical roughness values correlated with the surface treatment. Figure 6-1 shows the roughness values used in this work, which are consistent with standard practice for similar land use classes.

HYDRAULIC ROUGHNESS ZONES

RIVERINE AND COASTAL FLOODPLAIN MAPPING
PORT BRUCE, ONTARIO

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PROJECT NO. 2882-021

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FIG. NO: 6-1

REV.: 0

6.2.3 2D Model Mesh and Breaklines

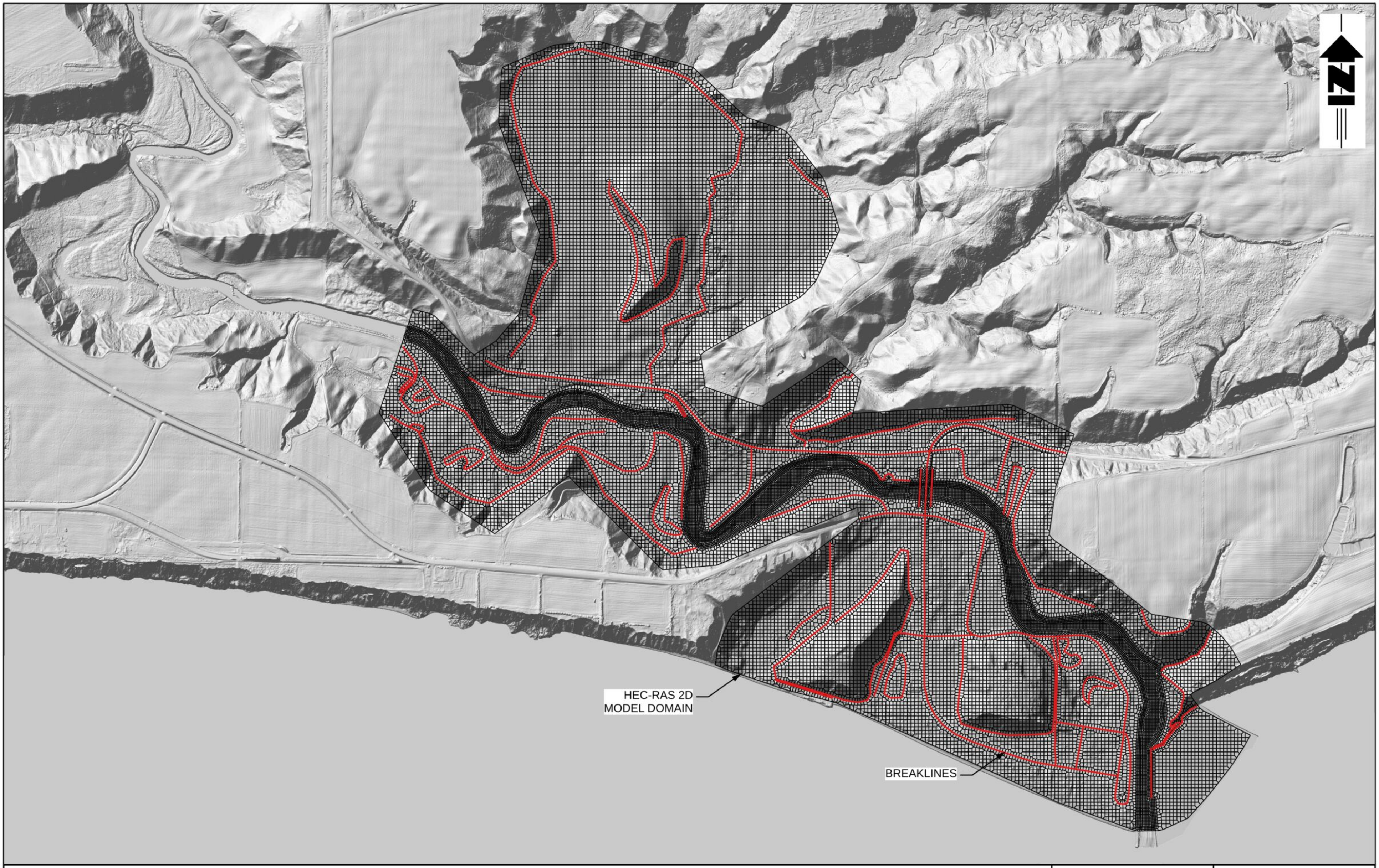
Model grid for the study area was constructed using unstructured elements of varying geometric proportions. To adequately represent river and floodplain geometry within study area the modeling domain was discretized using elements of various sizes. Fine resolution mesh was used in areas that were deemed to control flow characteristics, like main channels, bridges approaches, dikes, roadways, top and bottom of slopes, etc. Coarser resolution mesh was used elsewhere in the model domain in areas that are not anticipated to control flow propagation but could still be inundated. Care was taken to include appropriate grid resolution in the model to capture relevant features, and still keep computation times to a minimum.

A HEC-RAS 2D model schematic is presented in Figure 6-2, where the numerical model grid is shown, along with breaklines and hydraulic structures. A close up of the model mesh, better showing variation of grid sizes, is shown in Figure 6-3. Generally, areas within the 2D model domain that are anticipated to carry bulk of the flow were discretized with finer elements (such as main channels and at hydraulic structures). Areas further away were assigned larger grid cells, as these areas will likely not govern in determining flow behaviour (such as open fields for example). Model breaklines were placed at locations where geometry changes slope (like top of channel banks, tops and bottoms of slopes, road center-lines, etc). When used properly, breaklines allow the model to limit the number of grid cells (and thus reduce computational time), while capturing relevant flow hydraulics.

6.2.4 1D Model Cross Sections

For the HEC-RAS 1D model cross sections were extracted from the hydraulic model ready DEM. A schematic of the 1D model is shown in Figure 6-4. The extent of the cross sections was chosen to fully encompass flood lines from Regional Storm conditions. The Imperial Road Bridge was added to the model by entering the surveyed geometry (deck, soffit, piers, etc.). Ineffective flow areas, obstruction areas and levee nodes (dyke features at existing roadway embankment sections) were included in the modeling, as appropriate.

A total of 31 hydraulic cross sections are represented in the 1D model. Geometry of the floodplain at Port Bruce downstream of the Imperial Road Bridge suggests that under high flows several spill areas may be possible. 1D hydraulic models can not be used to accurately capture such spills. Given that 2D hydraulic modeling is included in this study, 1D cross sections were set up to include spill areas as backwater conditions of the main channel. Such an approximation is believed to be appropriate in the context of proving an assessment of ice jams and their impact.



HYDRAULIC MODEL GRID

RIVERINE AND COASTAL FLOODPLAIN MAPPING
PORT BRUCE, ONTARIO

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FIG. NO: 6-2

REV.: 0

HYDRAULIC MODEL GRID ZOOMED

RIVERINE AND COASTAL FLOODPLAIN MAPPING
PORT BRUCE, ONTARIO

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SCALE: 1:5,000

FIG. NO: 6-3

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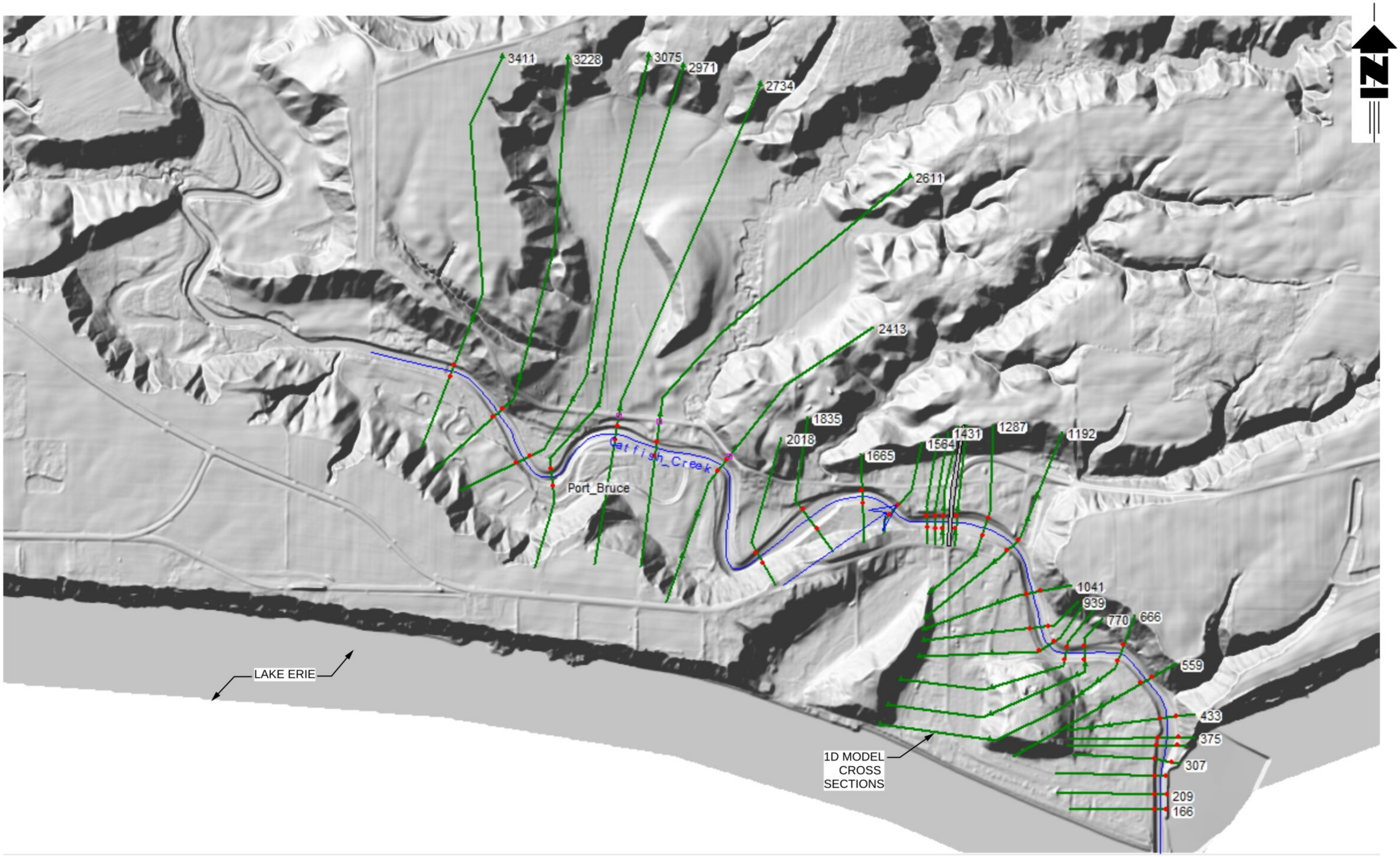

HYDRAULIC MODEL SECTIONS (1D MODEL)

RIVERINE AND COASTAL FLOODPLAIN MAPPING
PORT BRUCE, ONTARIO

TRUE

PROJECT NO. 2882-021

DATE: FEB 2024

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SCALE: N/A

FIG. NO: 6-4

REV.: 0

6.2.5 Hydraulic Structures

There is only one hydraulic structure (Imperial Road Bridge) within the modeling study area that was included in the HEC-RAS model. Bridge structure was coded into the HEC-RAS model (same inputs for 1D and 2D model variants). Latest version of the HEC-RAS 2D model allows bridge information to be included in similar manner as the 1D model variant, thus providing consistency in input geometry among model variants. Care was taken to develop internal upstream and downstream breaklines according to best modeling practice to properly represent the geometry at bridges in 2D (which can control water levels upstream).

6.2.6 Initial and Boundary Conditions

Initial conditions in the HEC-RAS 1D model was set as is standard practice for steady state models (where downstream water level is specified, and upstream peak flow). The numerical solution used was standard step method, with adjustments made for ice jams.

Initial conditions in the HEC-RAS 2D model domain were set as dry during initial conditions, meaning no water was initially in the modeling domain. The finite volume flow solver used in HEC-RAS was flexible enough to allow dry bed starting conditions. Design flows were gradually added at the upstream model boundary to simulate peak flow conditions while achieving model stability. As the present analyses involves riverine floodplain mapping only, flow were ramped up to design conditions. Once flows reached design conditions, they were applied sufficiently long to achieve steady state conditions in the system and thus obtain maximum water levels during the desired event.

The downstream boundary conditions in Lake Erie were set as the 20-yr return period using the monthly average lake level statistic (calculated as 174.55 m CGVD2013). Hourly Lake Erie water level observations from the Port Stanley and Port Dover gauges (for years 1961-2022) were downloaded and subsequently analyzed. Hourly data was converted to monthly data, and used to extract annual maximum monthly water level. The annual extremes were fit to several statistical distributions to estimate corresponding return period values. Statistical fits using method of moments and L-moments have yielded similar results. For the purposes of this work, Generalized Extreme Value distribution with parameters estimated using L-moments, was selected and used to establish downstream boundary condition for the modeling work.

Since the flood levels in Port Port Bruce (upstream of the Imperial Road Bridge) are influenced more by the hydraulic gradient of the river than the backwater conditions of the lake, the starting value of the downstream boundary condition does not significantly impact flood levels and flood lines. For the area downstream of the Imperial Road Bridge lake levels are the governing mechanism of flooding, that are summarized in Section 7 of this report.

6.3 Simulations Scenarios

In this work, simulation scenarios were considered that included clear water and ice jam simulations. Pertinent details are provided below.

6.3.1 Clear Water Simulations

For clear water simulations, HEC-RAS 2D hydraulic model was used. Individual scenarios were set up to simulate 2, 5, 10, 20, 50, 100-yr and Regional Storm conditions (summarized in Section 5.5). Climate adjusted flows included a 15% increase for 100-yr and Regional Storm conditions, which can be considered preliminary. In all cases, the downstream water level was kept at 20-yr mean monthly water level of Lake Erie, and is consistent with previous studies in the area.

6.3.2 Ice Jam Simulations

For ice jam simulations, HEC-RAS 1D hydraulic model was used. The flow scenarios were set up for 2, 5, 10, and 20-yr return period flow conditions. As above, the downstream water level was kept at 20-yr mean monthly water level of Lake Erie.

For the assessment of ice jams a limited scope hydraulic modeling was carried out. Ice jams result from a combination of factors including i) ice conditions, ii) river flows, iii) lake levels, and iv) river geometry. A set of parameters were defined using a reasonable combination of ice jam factors, and 1D steady state HEC-RAS ice jam analyses. A conversation with CCCA staff revealed that ice thickness is monitored during the winter/spring months. Based on the past measurements, ice thickness of 0.3 m are possible in the harbour, and areas upstream.

For the hydraulic modeling in this work, ice jam thickness of 0.3 m was assumed to occur between the southerly tip of Port Bruce pier and approximately 700 m upstream. The HEC-RAS 1D steady state model was used to determine water surface profiles within the study area resulting from 2, 5, 10, and 20-yr flows in combination with a 0.3 m thick ice sheet that is about 700 m long (starting at the pier and extending upstream). The lake level in the simulations of ice jams was set at 174.55 m CGVD2013, same as in the clear water (non-ice) conditions.

It is not customary to carry out ice jam assessments using combination of scenarios, as it is unlikely that all flood mechanisms would occur at the same time. For example, combinations of 100-yr lake level with 100-yr flood with an ice jam would be extremely rare. As a reasonable scenario this work considers 20-yr lake level in combination with an ice jam, with floods ranging from 2-yr to 20-yr.

6.4 Model Limitations

The modeling effort used in the development of the HEC-RAS 1D and 2D hydraulic modeling in the study area was consistent with generally accepted engineering practice at the present time. However, all models and methodologies have inherent limitations and should be clearly acknowledged and understood. Some of the limitations include:

a) The modeling assumes rigid bed conditions and neglects possible effects of channel migration and river bed scouring during extreme events,

b) Channel and floodplain are assumed to flow under clear water (and ice jam) conditions, with potential influence of debris neglected from the simulations,

c) The processed LiDAR data used to derive hydraulic model ready DEM neglects presence of buildings within the floodplain (i.e., the buildings are digitally removed from the DEM). As such, flow between individual buildings will not be well captured with the present modeling.

d) Calibration data for the study area was not available (measurements of water surface elevation during peak flooding), meaning that Manning's roughness coefficients were not adjusted (or tuned) from their assumed default values. Verification check of the flows were likewise not possible, as observations from a second large event were not available.

e) The hydraulic model developed follow standard practice for floodplain mapping assignments in the Province. Further refinement to the modeling shall be required for localized and/or site specific hydraulic assessments and design. Consultation with a Qualified Professional Engineer is recommended for such cases.

7.0 Coastal Water Levels and Wave Assessments

This sections presents technical analyses related to quantification of Lake Erie water levels and waves at the beach at Port Bruce. The aim of this section is to update the floodproofing standard at Port Bruce, given current Provincial definitions and updated technical analyses (this work).

7.1 Background

The original assessment of coastal hazards is summarized in the CCCA Shoreline Management Plan (Philpott, 1991). The Shoreline Management Plan report provides a high level description of the Lake Erie shoreline within CCCA watershed boundary and documents the coastal hazard assessment that was completed at the time. Of most relevance to this work is the assessment of the wave uprush elevation for the main beach at Port Bruce that was historically used as a floodproofing elevation for development activities along the main beach.

The design wave uprush elevations were computed by Philpott (1991) by using a 100-yr nearshore wave conditions, on top of 100-yr instantaneous lake level. Beach profiles were surveyed, and used to determine the wave uprush height above the 100-yr instantaneous water level using several empirical relationships (the best available method at the time). The value of wave uprush height fronting the beach dunes was estimated at 1.5 m above 100-yr instantaneous water level. The authors recognized that much of the dune crest elevation lies below the calculated (empirical) wave uprush level, meaning that during design water level conditions, a wave will overtop the dunes and run as an overland bore instead of running up and down a beach face (as assumed in the empirical calculations). Philpott (1991) states that the exact uprush elevation would be difficult to determine as the land behind the crest of the dunes is relatively flat, and the empirical relationships used do not account for flat land inshore of dune crests. Recognizing the limitations of the wave uprush height estimates, Philpott (1991) reduced the wave uprush height to 0.9 m, and recommended it for use at Port Bruce's main beach on top of the 100-yr instantaneous water level.

Philpott (1991) compared mechanism of beach sediment transport at Port Stanley and Port Bruce. A significant difference found was that Port Stanley's breakwater retains littoral materials, whereas the Port Bruce's western pier by-passes these materials. This implies that given a storm, a greater volume of material can be transported offshore at Port Bruce (and lost to downdrift) compared to what would be happening at Port Stanley for the same storm. Such conditions imply that beach recovery times at Port Bruce would take longer than recovery times at Port Stanley.

Similarly, site specific coastal analysis at Port Stanley carried out by Philpott (1989) identified an active beach width of 40 m, measured inland from the 100-yr instantaneous water level. At Port Bruce, Philpott (1991) identified active beach width in the order of 50 m, as the beach profiles were higher at Port Bruce compared to Port Stanley. Higher beach profiles imply higher wave energy dissipation, and potentially greater cross shore transport. This is the reason why greater active beach width was assigned at Port Bruce compared to Port Stanley.

7.2 Definitions

The Great Lakes – St. Lawrence Technical Guide (MNR, 2001) provides most recent official definitions of flood hazard limits and floodproofing elevations. According to the MNR (2001) Technical Guide, the flood hazard limit on the shores of the Great Lakes are to be defined as a sum of a 100-yr instantaneous water level and a flood allowance for wave uprush and other water related hazards (as completed in historic studies). For the determination of the flood hazard limit, the wave uprush is to be calculated using a 10-yr to 20-yr return period wave heights in conjunction with the 100-yr instantaneous water level (MNR, 2001, Part 3, pg. 3-39).

The MNR (2001) Technical Guide made a distinction between the elevation of the flood hazard limit, and the elevation to be used in the floodproofing standard. For the calculation of the floodproofing standard, the Technical Guide (MNR, 2001, Chapter 3, pg. 3-40) recommends that 50-yr to 100-yr return period wave height be used in conjunction with the 100-yr monthly mean lake level plus the 100-yr storm surge height to determine the floodproofing standard. The stricter definition of the floodproofing standard did not exist when original Shoreline Management Plans for Port Stanley (Philpott, 1989) and Port Bruce (Philpott, 1991) were completed.

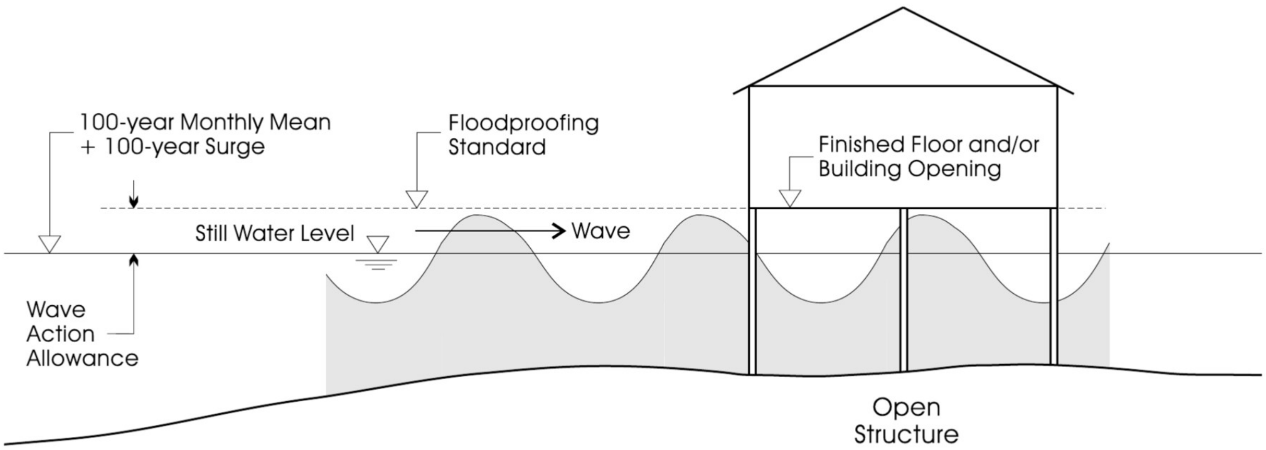
The floodproofing standard is defined in MNR (2001) as (pg. 7-54):

On lakes Superior, Huron, St. Clair, Erie or Ontario, development and site alteration is to be protected from flooding, as a minimum, to an elevation equal to the sum of the 100 year monthly mean lake level plus the 100 year wind setup plus a flood allowance for wave uprush and other water related hazards.

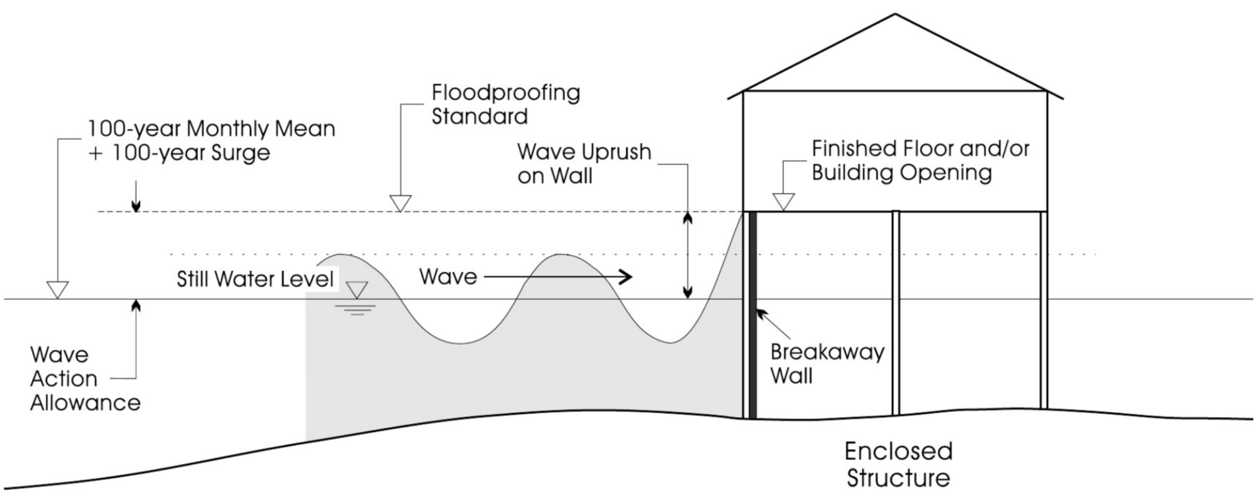
A schematic accompanying the floodproofing definition is shown in Figure 7-1 (taken directly from MNR, 2001).

For the purposes of this document, the floodproofing standard is defined as the top of the foundation. Using this definitions means that structural elements above the foundation (beams, trusses, connections) would be located outside of the wave related hazards.

a) Waves Pass Building



b) Wave Uprush on Building



NOTE: ABOVE IS A REPRODUCTION OF FIGURE 7-30, TITLED FLOODPROOFING STANDARD, TAKEN FROM THE MNR (2001) TECHNICAL GUIDELINES, PART 7, PG. 7-55.

FLOODPROOFING STANDARD

RIVERINE AND COASTAL FLOODPLAIN MAPPING, PORT BRUCE, ONTARIO

PROJECT NO: 2882-021

DATE: FEB 2024

DRAWN BY: PP

SCALE: N/A

FIG. NO: **7-1**

REV.: **0**

7.3 Analysis of Water Levels

Hourly historic water levels at the Port Stanley (ID 12400) and Port Dover (ID 12710) were obtained from the Canadian Hydrographic Service database for years 1962-2022. Historic data was used to establish instantaneous water levels, storm surge heights, and monthly average lake level statistics for Port Stanley and Port Dover.

Storm surges at Lake Erie increase along its eastern basin (i.e., Port Dover's surges are higher than Port Stanley's, and Port Colborne's surges are higher than Port Dover's). As there is no water level gauge at Port Bruce, this work assumes that storm surge characteristics at Port Bruce can be linearly interpolated for sites located between the gauges. In other words, storm surge characteristics at Port Stanley and Port Dover shall be established using gauged data. Then, the surges at Port Bruce (located between Port Stanley and Port Dover) shall be linearly interpolated based on Port Bruce's relative distance alongshore from each gauge.

For the instantaneous water level statistics, annual maximum water levels were extracted from the historic record and used to fit to several common statistical distributions. Comparing the answers among the distributions tested, results from the Generalized Extreme Value (GEV) statistical distribution with parameters estimated using the method of L-moments were selected for use in this work due to best fit, its general robustness and common acceptability in the literature.

Analyses of surge heights at individual gauges was completed by isolating the surge events from a weekly average base water levels. After developing a historic signal of surge heights, statistical analysis was carried out using the same distribution and method as noted above. Port Stanley, being located approximately mid lake, will tend to experience far lesser storm surges than locations at either ends of Lake Erie (such as at Bar Point on the west or Port Colborne on the east).

Lastly, statistical analyses were completed on monthly average data (required for the floodproofing calculations). Hourly data was averaged over each month for each year, and used in the analyses. Results from the statistical analyses are reported in Table 7-1 for Port Stanley and Table 7-2 for Port Dover. The water level data available from Environment Canada was provided in IGLD85 vertical datum, with output tables provided in that datum.

For monthly average lake levels values at Port Stanley and Port Dover are nearly identical, with the difference being attributed to minor errors in measurements. For Port Bruce, the 100-yr mean monthly water level of 175.17 m IGLD85 shall be adopted in this study.

For storm surge heights at Port Bruce, the linear interpolation is carried out as follows: Distance from Port Stanely to Port Dover is 94.9 km, where storm surges are known. As Port Bruce is located 17.3 km away from Port Stanley, it will experience storm surge values closer to Port Stanley's than Port Dover's. Linear interpolation for the 100-yr case produces 0.95 m storm surge height at Port Bruce.

Note that storm surge values in MNR (2001) document report for reach E-9 at Port Bruce are believed to be inappropriate, and shall not be used. The same document reports higher surges at Port Stanley (for the same return period) than at Port Bruce or Port Burwell, which is physically impossible. MNR (2001) document provides an explanation that their reported surge values for Port Stanley were based on measurements, while the Port Bruce and Port Burwell surge values were based on a numerical model.

The storm surge values at Port Bruce are determined through linear interpolation using gauged data between Port Stanley and Port Dover and are used in subsequent analyses.

For determining floodproofing elevations and completing foundation design, design still water level is established as 100-yr mean monthly lake level plus 100-yr storm surge at Port Bruce, which is 175.17 m IGLD85 + 0.95 m = 176.12 m IGLD85 for Port Bruce. Using the applicable vertical datum conversion, the design still water level converts to 175.65 m CGVD2013. Wave uprush is to be applied on top of the design still water level. The text that follows describes the steps undertaken to estimate wave uprush.

Table 7-1: Port Stanley Water Level Statistics (gauge ID 12400)

Return Period [yrs]	Instant. Water Level [m, IGLD85]	Storm Surge Height [m]	Mean Monthly Water Level [m, IGLD85]
2	174.74	0.36	174.53
5	175.02	0.45	174.79
10	175.16	0.52	174.91
20	175.27	0.59	175.01
50	175.38	0.69	175.11
100	175.45	0.77	175.17
200	175.51	0.85	175.21

Table 7-2: Port Dover Water Level Statistics (gauge ID 12710)

Return Period [yrs]	Instant. Water Level [m, IGLD85]	Storm Surge Height [m]	Mean Monthly Water Level [m, IGLD85]
2	175.27	0.95	174.53
5	175.58	1.18	174.78
10	175.73	1.33	174.91
20	175.84	1.46	175.00
50	175.95	1.64	175.10
100	176.01	1.77	175.16
200	176.06	1.89	175.21

7.4 Analysis of Lake Erie Offshore Wave Climate

To estimate the site-specific wave climate at the project site, the latest available wave hindcast data was used. For characterization of offshore wave climate the US Army Corps of Engineers Wave Information Study (WIS) hindcast database for Lake Erie was used. The wave hindcast output node at the location closest to the project site (WIS node ST92176, 6 km offshore, 15 m depth contour, years 1979-2022) was extracted from the database, and used in analyses of the directional statistics for winds and waves. Figure 7-2 shows a relative location of the WIS hindcast node relative to Port Bruce, along with Lake Erie bathymetry.

Lake Erie's dominant wind direction is from the SW, which runs along the main axis of the lake for the maximum fetch. SW winds on Lake Erie generate the highest surges on the lake.

To complete the statistical analyses, wave time series data from the WIS database was divided into bins corresponding to 16 cardinal direction on a compass. Statistical frequency analyses was then carried out for each directional bin. The Generalized Logistic (GLO) statistical distribution, with parameters estimated using the method of L-Moments, was used to fit annual maximum waves and extract quantiles corresponding to return periods ranging from 2-yr to 100-yr. The GLO distribution had a better fit to the extreme wave data than other distributions tested, hence its adoption. The results of the statistical analysis for the SW waves are shown in Table 7-3, as SW waves are most dominant and govern the design.

Table 7-3: Offshore SW Wave Characteristics at WIS Hindcast Node ST 92176

Return Period [yr]	Significant Wave Height [m]	Peak Wave Period [s]
2	3.27	7.0
5	3.73	7.5
10	4.08	8.0
20	4.46	9.0
50	5.06	9.5
100	5.61	10.0

The 100-yr offshore wave conditions shown in Table 7-3 is used for the computation of wave runup for the purposes of establishing Lake Erie floodproofing elevations at Port Bruce. Wave propagation analysis is needed to propagate and transform offshore waves closer to the beach. This is discussed next.

7.5 Wave Propagation and Transformation Modeling

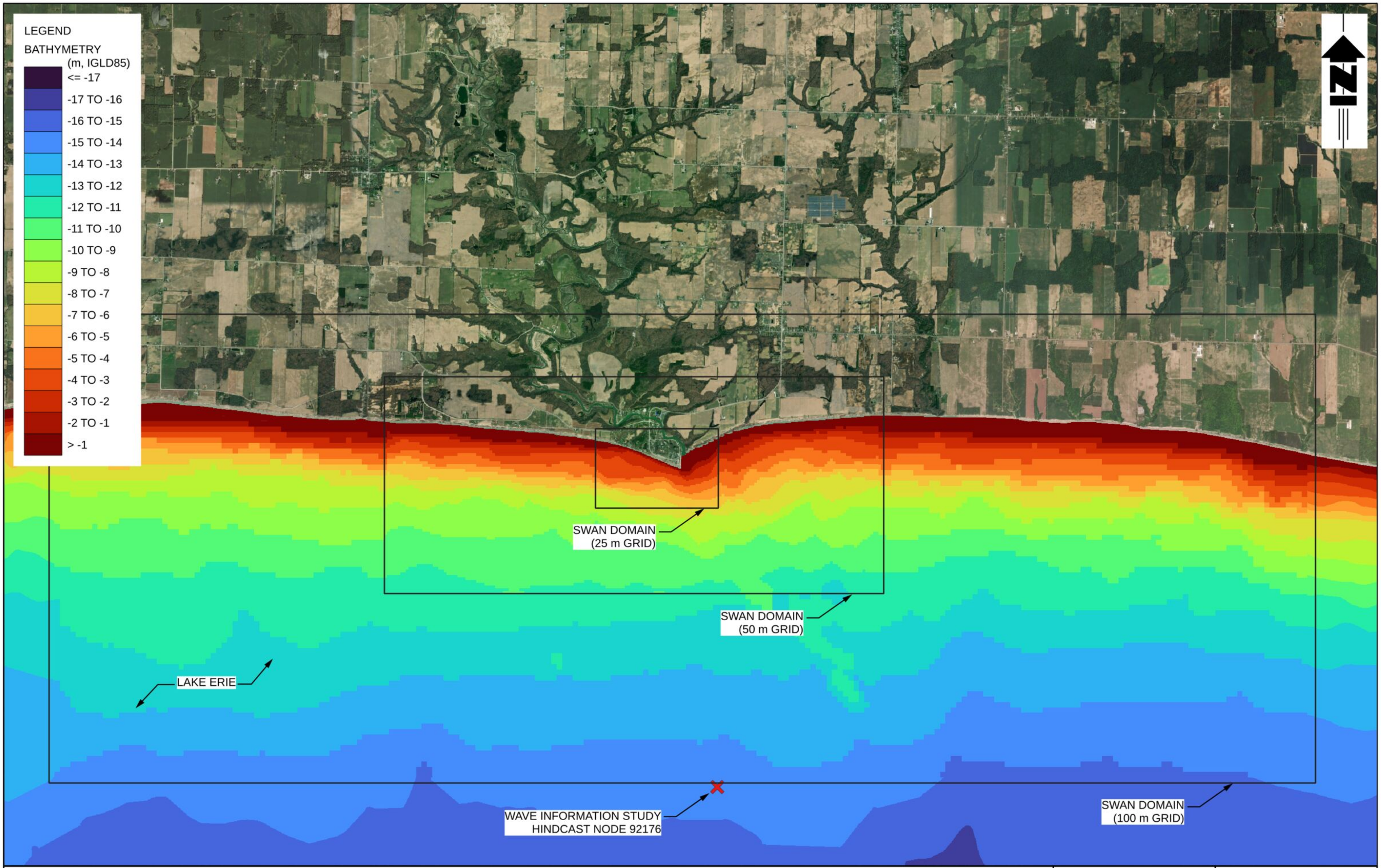
A SWAN wave model was set up and used to propagate waves from the WIS hindcast node to the shoreline. The results from the modeling will be extracted at approximately the 4 m depth contour, which coincides with the offshore limit of the beach profile transects set up for this project. The SWAN model solves the spectral action balance equation and captures the effects of spatial wave propagation, refraction, shoaling, generation, dissipation and nonlinear wave-wave interactions. Processes of wave breaking, bottom friction and (simplified) diffraction effects have been included in this work. The most important feature of SWAN relating to the current project is its ability to estimate the growth and propagation of wind generated waves from offshore to the nearshore area of the project site.

Figure 7-2 shows the SWAN model domains, which have been set up as a series of nested numerical model domains (using 100 m, 50 m and 25 m grids), developed for the purposes of propagating waves from the WIS hindcast node (offshore) to the project site (nearshore). Publicly available lake contours were used from which a Triangulated Irregular Network (TIN) model of the underwater area of the lake was created and used to develop the SWAN model domains. The 100-yr offshore wave characteristics were inputted into the SWAN model, which allowed waves to be transformed to the nearshore portion of the domain (to about the 4 m depth contour). From the 4 m depth contour to the limit of wave uprush, a set of transects have been set up for use in more detailed SWASH (Simulating WAves until SHore) wave modeling that propagate nearshore waves upland. The transects used in SWASH modeling at Port Bruce are shown in Figure 7-3. Design wave conditions obtained from SWAN's 25 m grid (finest resolution nested model) are shown in Table 7-4, and correspond to the offshore limit of each SWASH transect.

Table 7-4: Design Wave Conditions at Port Bruce (from 100-yr SW Winds)

Transect ID	Sig. Wave Height Hm0 [m]	Peak Wave Period Tp [sec]	Wave Direction Dir [Az, deg]
1	2.85	9.85	214.2
2	2.77	9.85	213.7
3	2.86	9.85	212.9
4	2.74	9.85	210.4
5	2.93	9.85	210.3
6	3.01	9.85	209.2
7	3.13	9.85	211.1
8	2.91	9.85	212.5

Having wave conditions summarized in Table 7-4, the next step is to carry out simulations using the SWASH model and propagate the waves inland. This is described next.



WAVE MODELING DOMAINS

RIVERINE AND COASTAL FLOODPLAIN MAPPING PORT BRUCE, ONTARIO

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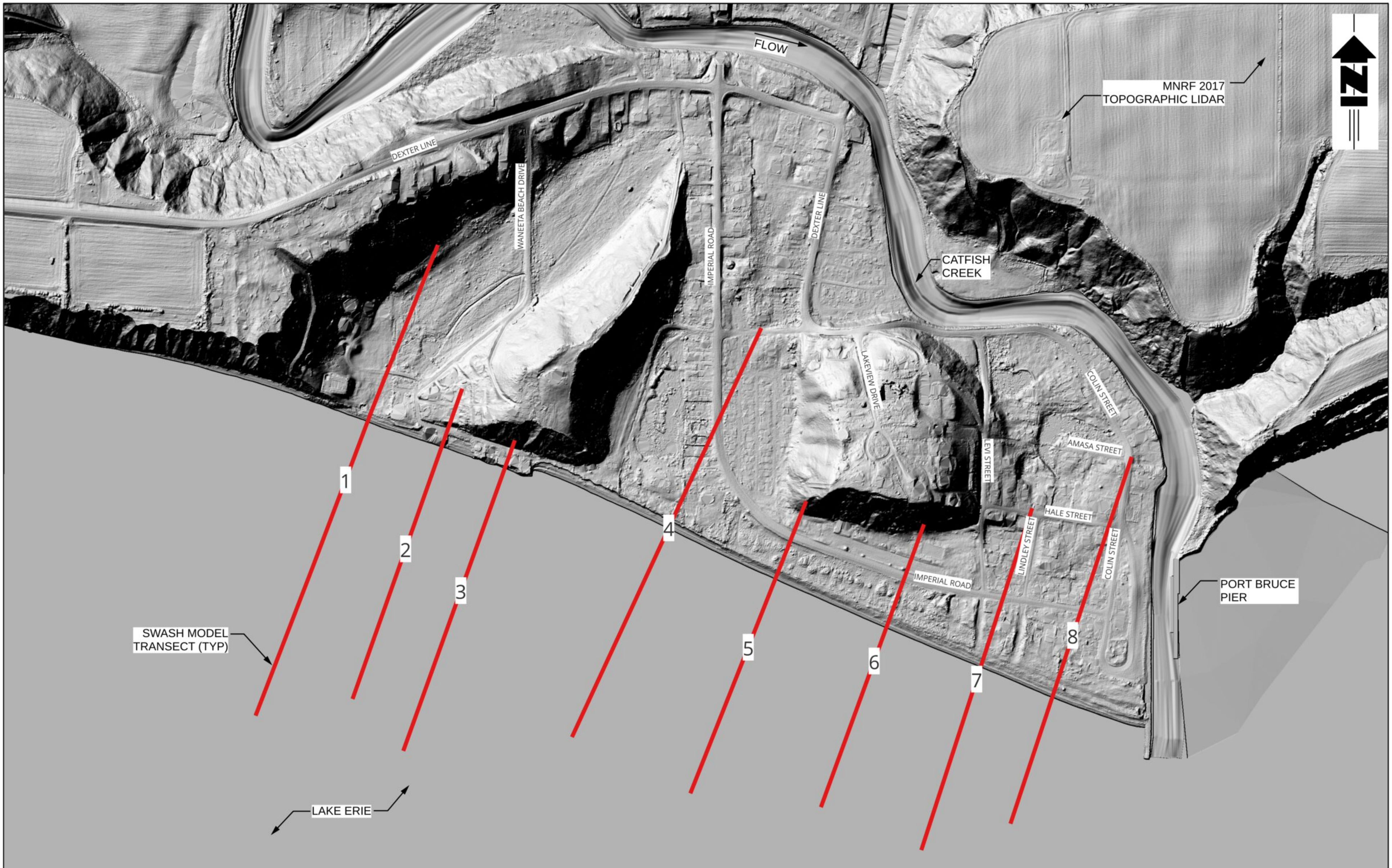
PROJECT NO. 2882-021	
DATE: FEB 2024	



WAVE MODEL TRANSECTS

RIVERINE AND COASTAL FLOODPLAIN MAPPING
PORT BRUCE, ONTARIO

PROJECT NO. 2882-021

DATE: FEB 2024

DRAWN BY: PP

SCALE: 1:7,500

FIG. NO: 7-3

REV.: 0

7.6 Beach Wave Runup Modeling

From the offshore end of each transect to either the toe of the bluff or within the main beach, a 1D variant of SWASH numerical model is used. SWASH model is a sister program to SWAN, and captures nearshore processes (such as wave setup, wave transformation and breaking, wave uprush and overtopping) relevant to this work. The SWASH model is able to compute both wave uprush, and inland propagation of the wave bore (which occurs in conditions when extreme high water level causes incoming waves to overtop the dunes, thus allowing waves to propagate inland as bores).

A total of eight transects were used in the beach wave modeling, using 1 m horizontal resolution. Each transect was extracted using the surveyed bathymetry (for its below water portion) and MNRF 2017 LiDAR data (for its above water topography). Design still water level for use in floodproofing calculations was applied to the SWASH model, as per MNR (2001) definitions. The locations of the transects correspond to locations where bathymetry was collected (see Figure 3-1 for surveyed locations, and Figure 7-3 for transects).

Since the main beach dunes are relatively low compared to the design water level some amount of wave energy propagates inland during design water level conditions. Classical tools can not accurately estimate characteristics of the inland propagation of a wave bore, nor estimate wave uprush characteristics that far inland. Adjustments based on knowledge of coastal processes and professional judgement were used in the Philpott (1991) to describe the governing behaviour and estimate the wave uprush elevation. The state of the art knowledge in numerical modeling of coastal processes in the early 1990's was not yet able to numerically represent the overland wave bore process. The SWASH numerical model overcomes these limitations, and is able to capture behaviour of the overland wave bore and the general wave uprush on a beach at each individual transect.

To quantify wave uprush at each transect a time series of water level (at several output nodes) were extracted, and analyzed to estimate the 2% wave uprush (R2%). R2% is defined as the average elevation of the highest 2% of waves during design conditions. Included in the SWASH analyses is wave setup, defined as the increase in water level at the shoreline due to wave breaking in the surf zone. It is unknown if previous analyses used wave setup in their calculations, or if necessary adjustments were made.

Results of the wave uprush modeling are presented in Section 8.

8.0 Floodplain Mapping

Results from the hydraulic and wave modeling carried out in this assignment are presented in this section, as are procedures used to develop flood inundation limits, access and egress hazards, ice jam results, and identification of coastal floodproofing elevations.

8.1 Riverine Floodplain Mapping

8.1.1 Inundation Limits

The natural floodplain of the Catfish Creek in the upstream portion of the study area is generally bounded by the landmass (bluff) features on either side of the river. After the retreat of the glaciers the river carved its path over geologic time through remaining landmass leaving what is presently the natural floodplain. Under 100-yr flow conditions, the floodplain is generally bounded by the higher landmass to the south, and generally the Rush Creek Line roadway embankment. Under Regional Flow conditions, the Rush Creek line access road is generally overtopped, with backwater extending significant distance upstream. Two smaller tributary streams empty into Catfish Creek upstream of the Imperial Bridge. The floods on the Catfish create backwater to both of the tributaries.

A spill area floodplain feature at Port Bruce is located downstream of the Imperial Road Bridge. During 100-yr flood conditions the inundation downstream of the bridge spills the right bank, travels overland at angles perpendicular to the main channel, and eventually finds its way to Lake Erie. Under Regional Flow conditions, the spill downstream of the Imperial Road Bridge is three pronged, with flow going overtopping a) Waneeta Beach Drive, b) Dexter Line/Imperial Road, and c) rest going through the main channel/harbour.

Figure 8-1 shows the inundation limits from 100-yr and Regional Flow conditions on the same plot. Plot on Figure 8-2 shows the 100-yr inundation floodline, along with climate change adjusted floodline (based on the preliminary 15% increased in peak flow). As expected, higher flows under climate change conditions incrementally widen the floodplain. Similarly, Figure 8-3 shows the Regional Flow inundation extent, along with the climate change adjusted floodline (based on the same 15% increase in peak flows). Under higher flow the floodplain is wider, but the general pattern of behaviour (three pronged flow split downstream of the Imperial Road Bridge) remains.

INUNDATION LIMITS

RIVERINE AND COASTAL FLOODPLAIN MAPPING
PORT BRUCE, ONTARIO

TRUE

PROJECT NO. 2882-021

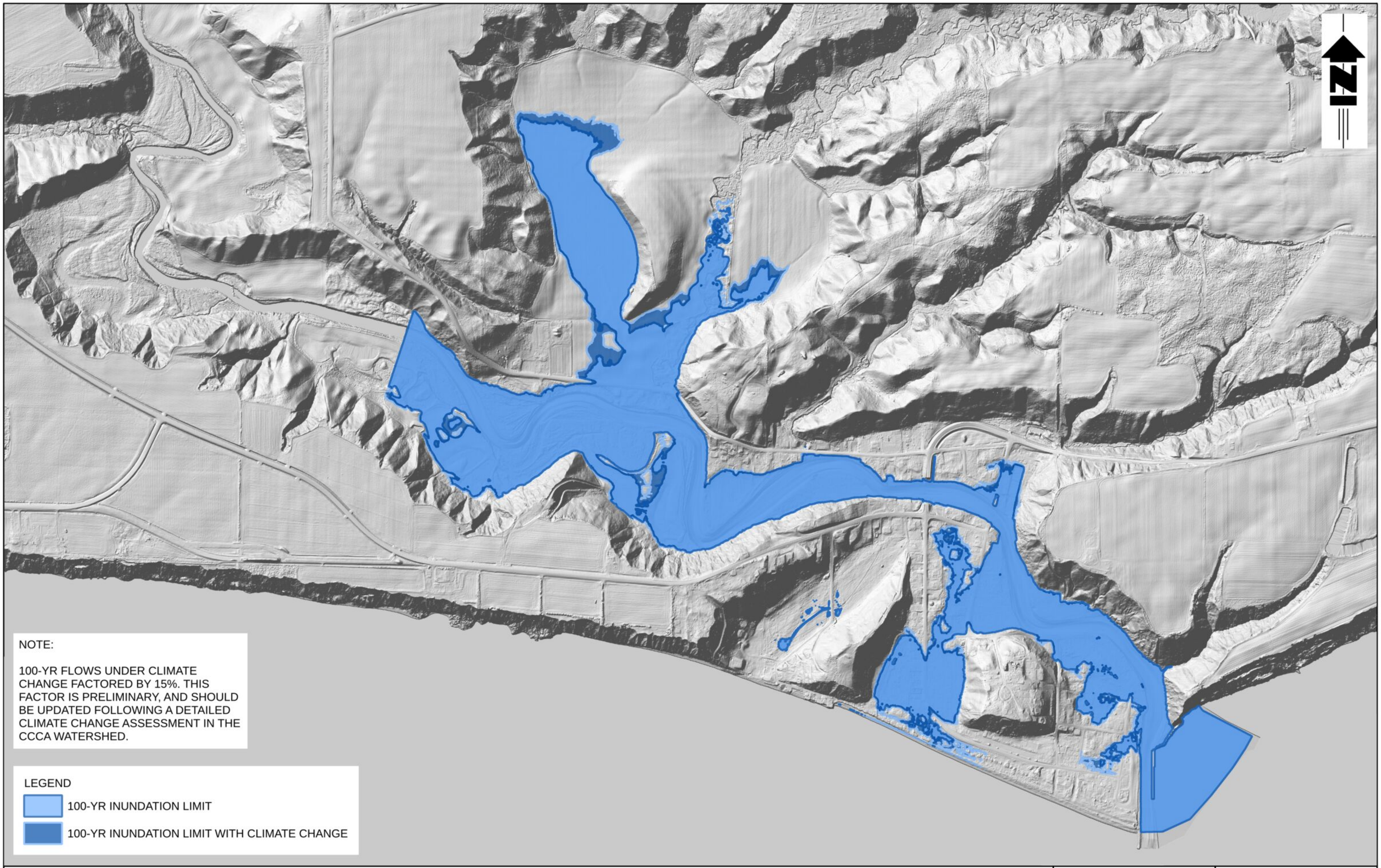
DATE: FEB 2024

DRAWN BY: PP

SCALE: 1:15,000

FIG. NO: 8-1

REV.: 0



CLIMATE CHANGE INUNDATION LIMITS 100-YR FLOWS

RIVERINE AND COASTAL FLOODPLAIN MAPPING
PORT BRUCE, ONTARIO

DRAWN BY: PP
SCALE: 1:15,000
FIG. NO: 8-2
REV.: 0
PROJECT NO. 2882-021
DATE: FEB 2024

CLIMATE CHANGE INUNDATION LIMITS REGIONAL FLOWS

RIVERINE AND COASTAL FLOODPLAIN MAPPING
PORT BRUCE, ONTARIO

TRUE

PROJECT NO. 2882-021

DATE: FEB 2024

DRAWN BY: PP

SCALE: 1:15,000

FIG. NO: 8-3

REV.: 0

8.1.2 Access/Egress Hazards

For the purposes of evaluating access and egress (to and from Port Bruce) during times of flooding the 2D hydraulic model was used to estimate depths and velocities along traveled road surfaces. Access/egress is evaluated using Regional flow conditions. Traveled road surfaces are defined as access roads used during times of flood hazards. Vehicles traveling on access roads can include cars, trucks, and emergency vehicles (firetrucks, and ambulances). Hydraulic modeling carried out produced spatially varied depths and velocities that are used to evaluate whether the traveled surfaces meet existing Provincial access/egress standards.

The Provincial standard for access/egress is evaluated based on depths, velocities, and a product of depth and velocity. MNR (2002) states that reasonably low risk conditions for pedestrian access during times of flooding are reached when depth does not exceed 0.8 m, velocity does not exceed 1.7 m/s and a product of depth and velocity does not exceed 0.4 m^2/s (MNR 2002, Appendix p.27). Thus, if any one of three quoted criteria are exceeded, the Provincial standard is considered not met.

Access/egress road profiles evaluated are those surface that are anticipated to be inundated during Regional Storm conditions. Table 8-1 lists the locations of the road profiles considered, while Figure 8-4 shows their locations on a plan area map (along with direction arrows for each profile). Figures 8-5 to 8-12 show profile plots for ground surface vs water level (top), velocity (middle), and depth (bottom). Having profile data in Figures 8-5 to 8-12 allows for evaluation of the Provincial access/egress standard to be evaluated. Summary of depths, velocities and their product is shown in Table 8-2.

The results in this work indicate that all access/egress roads within the study area do not meet the required Provincial standard under Regional Storm conditions.

Table 8-1: Access/Egress Profile Names

Profile ID	Descriptions
1	Rush Creek Line West
2	Rush Creek Line East
3	Imperial Road North
4	Dexter Line North
5	Dexter Line East
6	Colin Street North
7	Colin Street East
8	Imperial Road South

Table 8-2: Access/Egress Summary at Port Bruce

Profile ID	Max Depth [m]	Max Velocity [m/s]	Max Depth x Velocity [m^2/s]	Is Provincial Standard Met [Yes or No]
1	3.2	0.9	1.5	No
2	1.5	1.9	1.9	No
3	0.3	2.0	0.6	No
4	0.6	2.5	1.5	No
5	1.5	0.5	0.6	No
6	1.5	2.5	3.75	No
7	1.5	1.5	2.25	No
8	1.5	1.0	1.0	No

Notes:
Max Depth x Velocity may occur at different point than either Max Depth or Max Velocity.

ACCESS ROAD OVERTOPPING LOCATIONS

RIVERINE AND COASTAL FLOODPLAIN MAPPING
PORT BRUCE, ONTARIO

TRUE

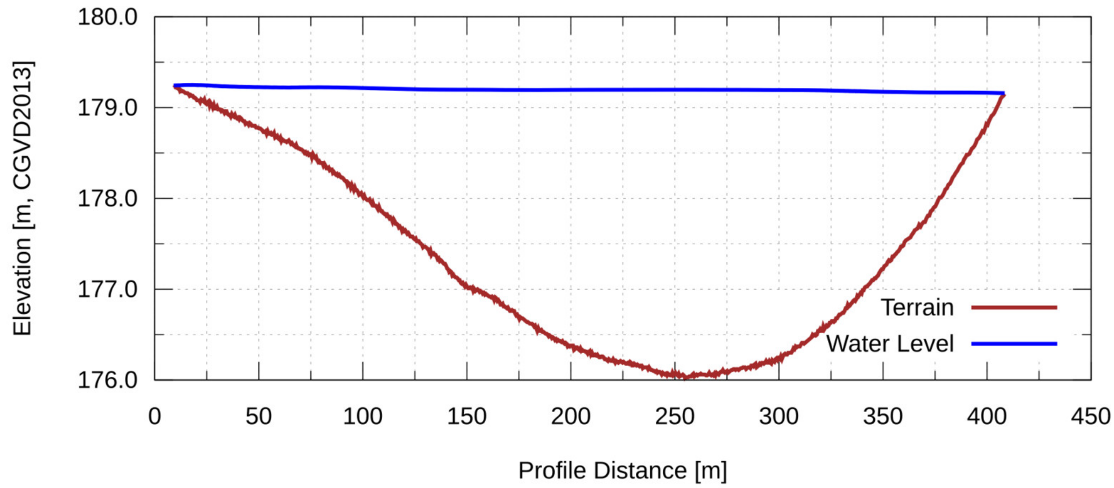
PROJECT NO. 2882-021

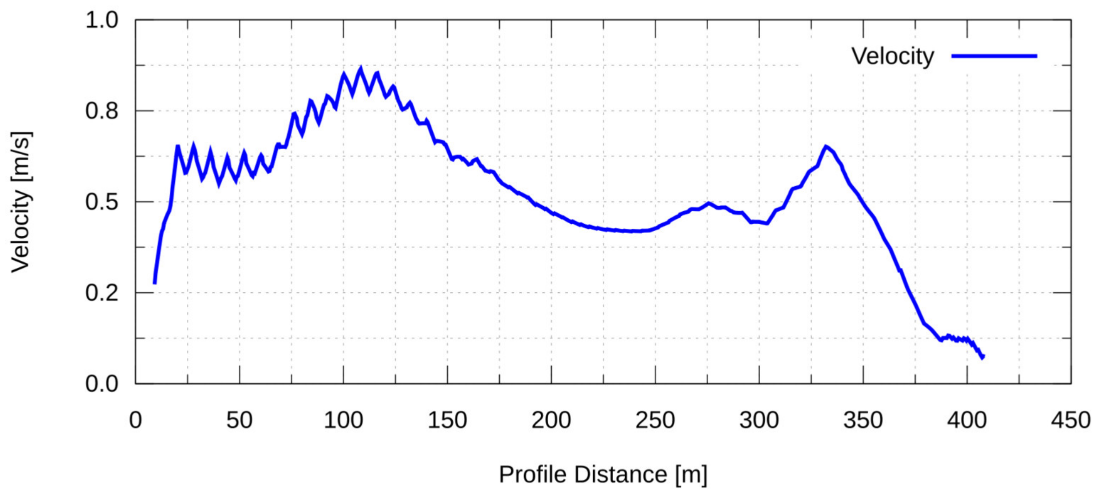
DATE: FEB 2024

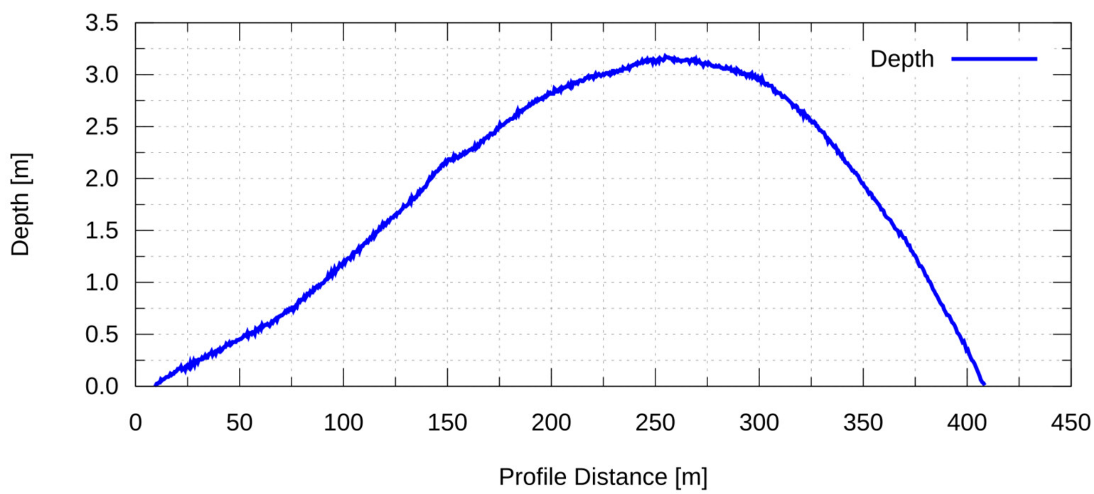
DRAWN BY: PP

SCALE: 1:10,000

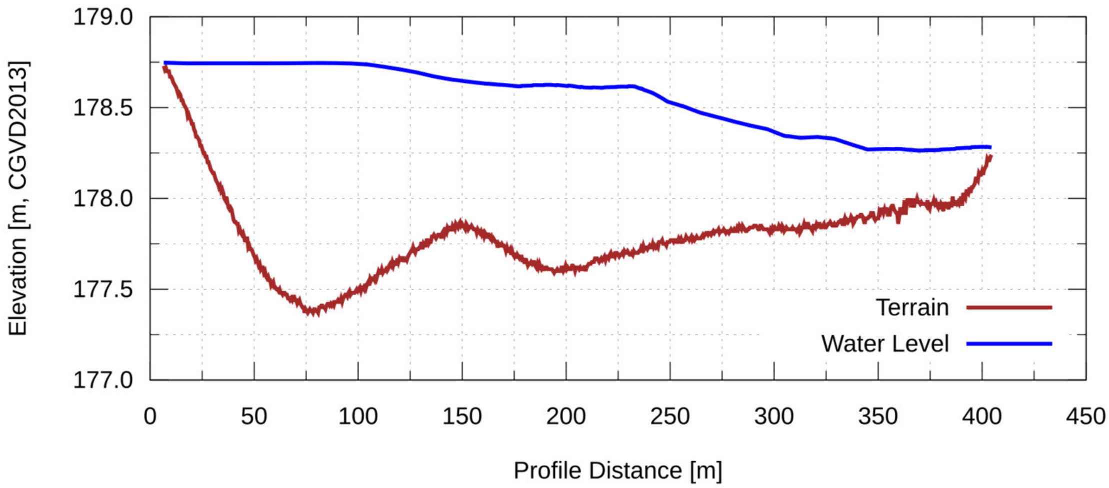
FIG. NO:	REV.:
8-4	0

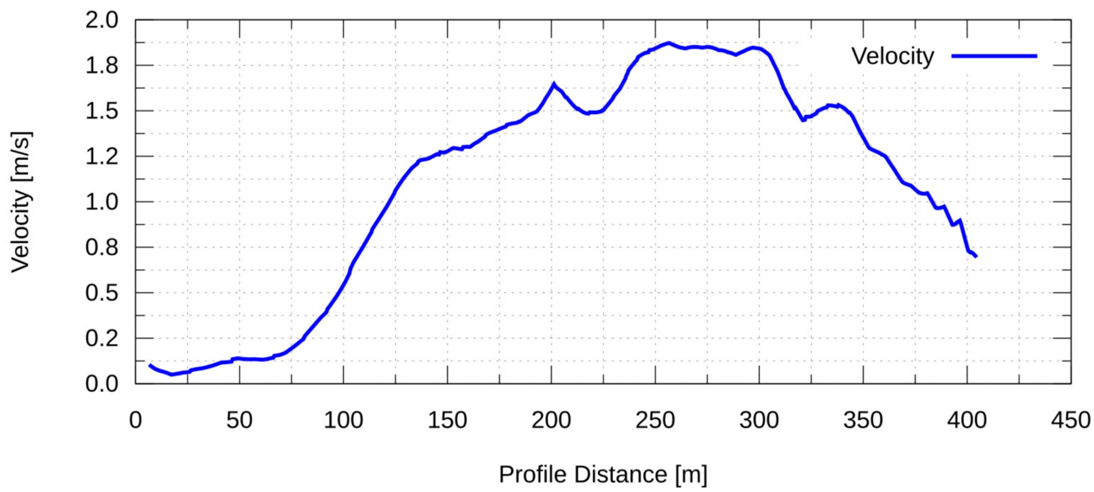


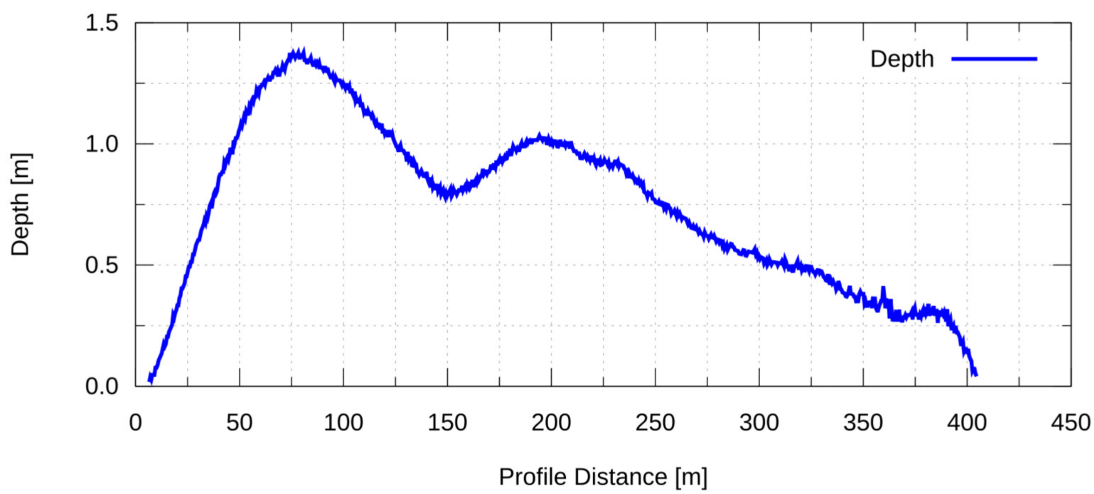




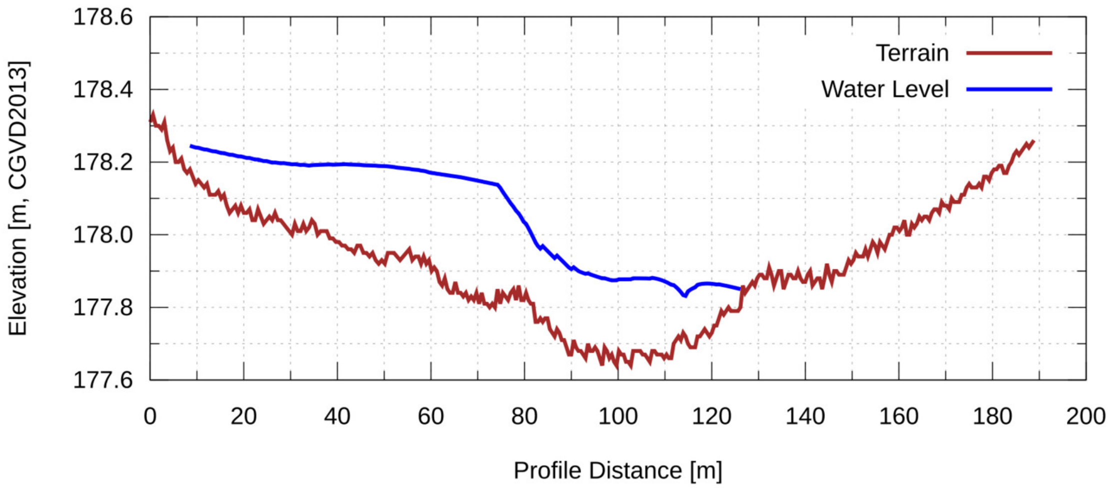
LOC 1: RUSH CREEK LINE WEST	TRUE	DRAWN BY: PP	
		SCALE: AS NOTED	
RIVERINE AND COASTAL FLOODPLAIN MAPPING, PORT BRUCE, ONTARIO	PROJECT NO: 2882-021	FIG. NO: **8-5**	REV.: **0**
	DATE: FEB 2024		

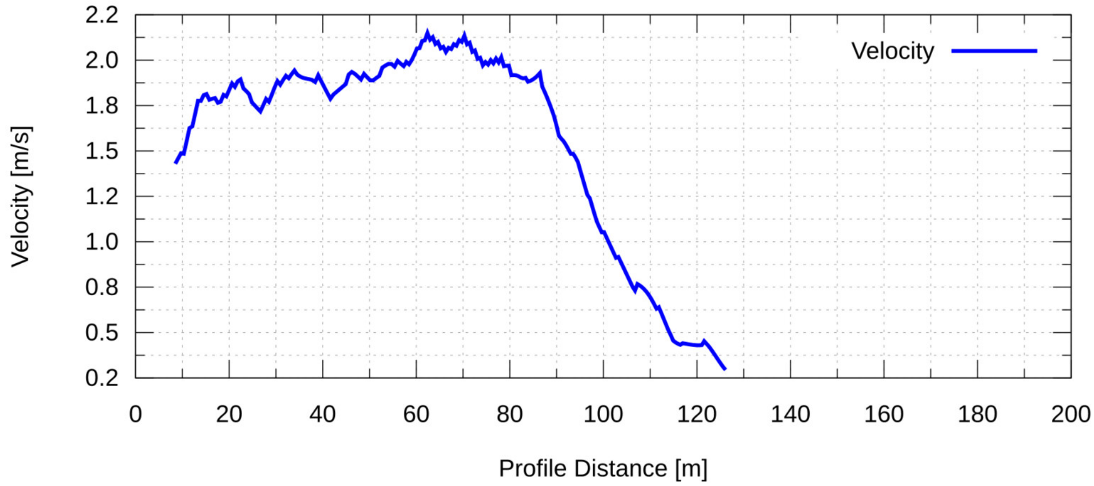


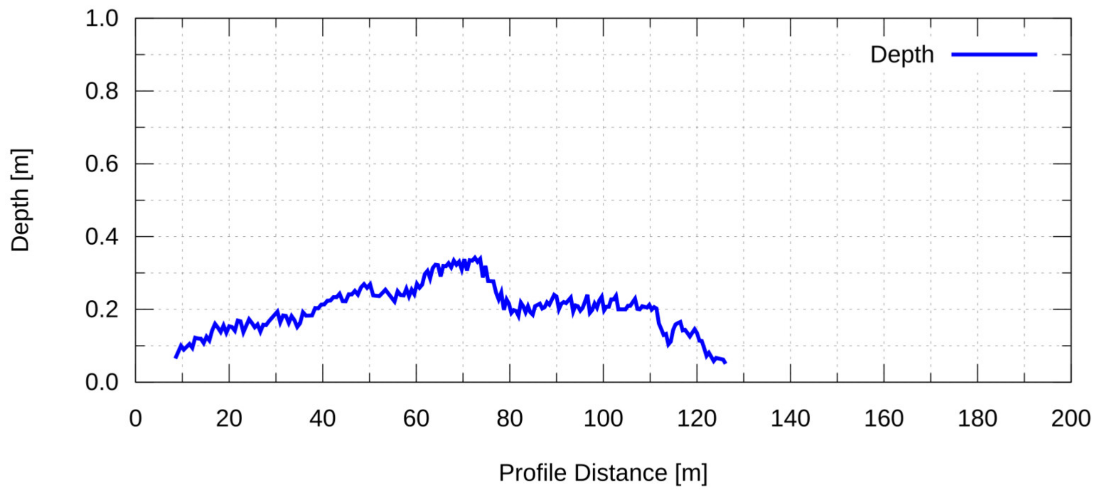




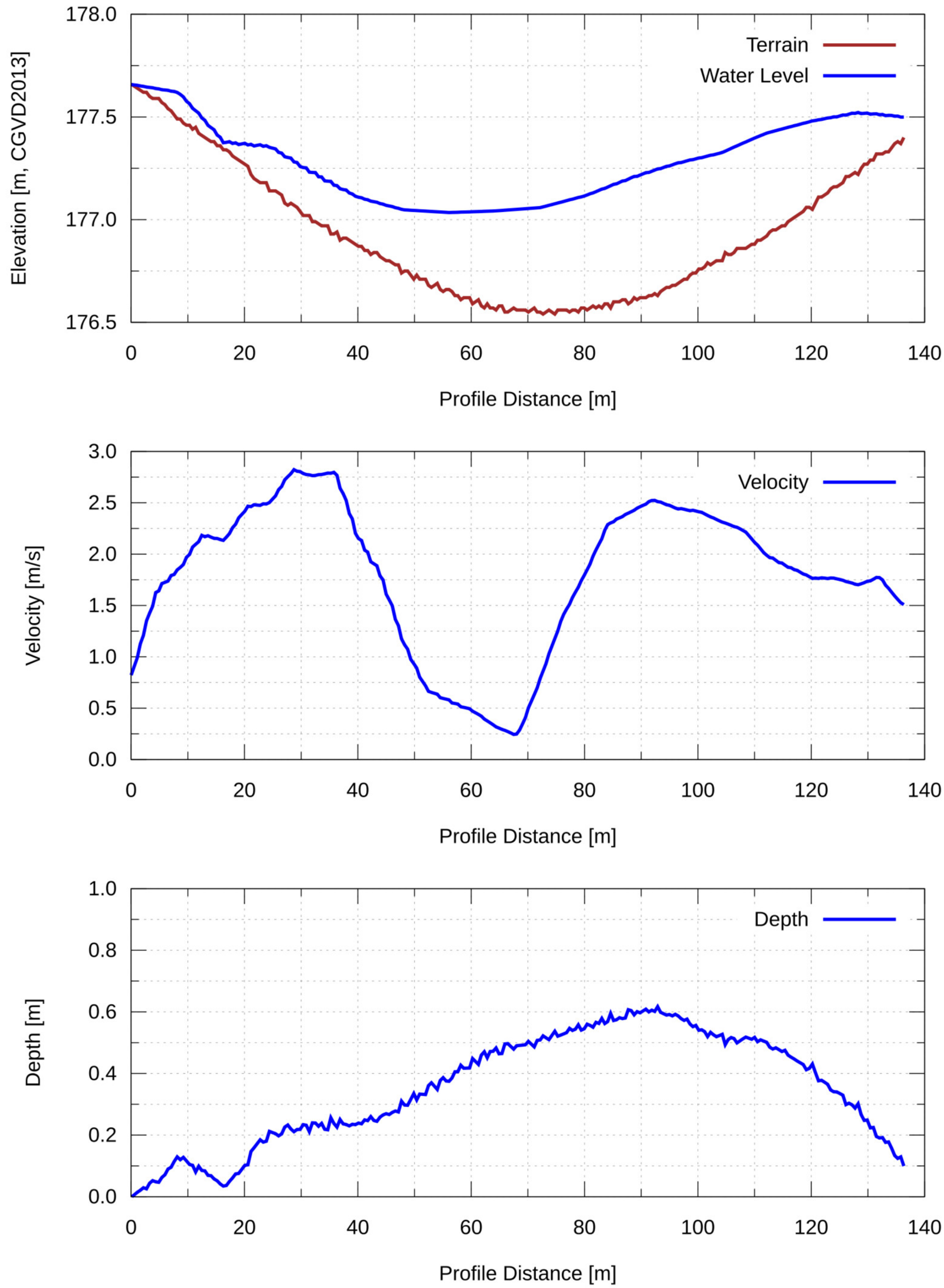
LOC 2: RUSH CREEK LINE EAST	TRUE	DRAWN BY: PP	
		SCALE: AS NOTED	
RIVERINE AND COASTAL FLOODPLAIN MAPPING, PORT BRUCE, ONTARIO	PROJECT NO: 2882-021	FIG. NO: **8-6**	REV.: **0**
	DATE: FEB 2024		







LOC 3: IMPERIAL ROAD NORTH	TRUE	DRAWN BY: PP	
		SCALE: AS NOTED	
RIVERINE AND COASTAL FLOODPLAIN MAPPING, PORT BRUCE, ONTARIO	PROJECT NO: 2882-021	FIG. NO: **8-7**	REV.: **0**
	DATE: FEB 2024		



LOC 4: DEXTER LINE NORTH

RIVERINE AND COASTAL FLOODPLAIN MAPPING, PORT BRUCE, ONTARIO

PROJECT NO: 2882-021

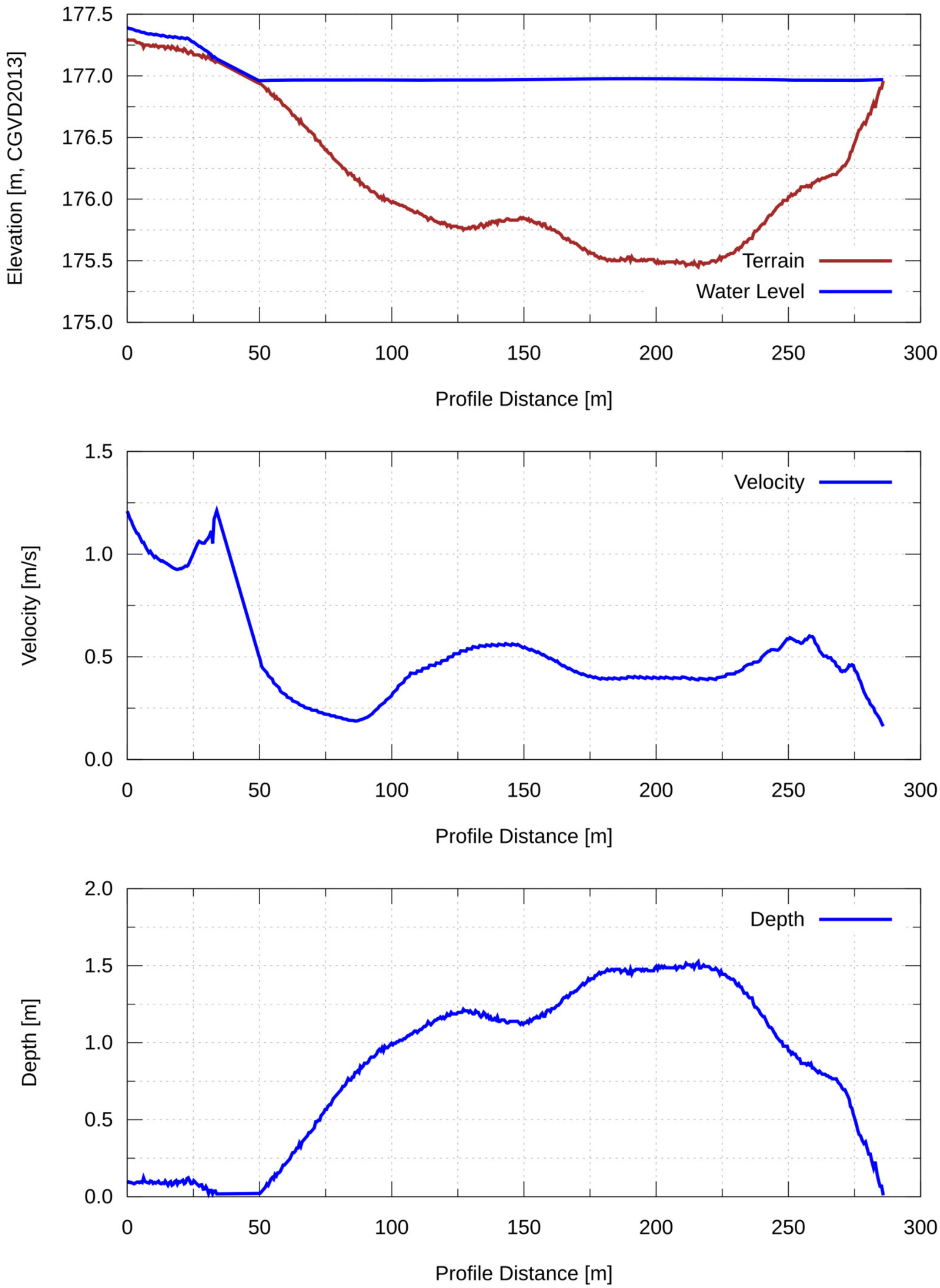
DATE: FEB 2024

DRAWN BY: PP

SCALE: AS NOTED

FIG. NO: 8-8

REV.: 0



LOC 5: DEXTER LINE EAST

RIVERINE AND COASTAL FLOODPLAIN MAPPING, PORT BRUCE, ONTARIO

PROJECT NO: 2882-021

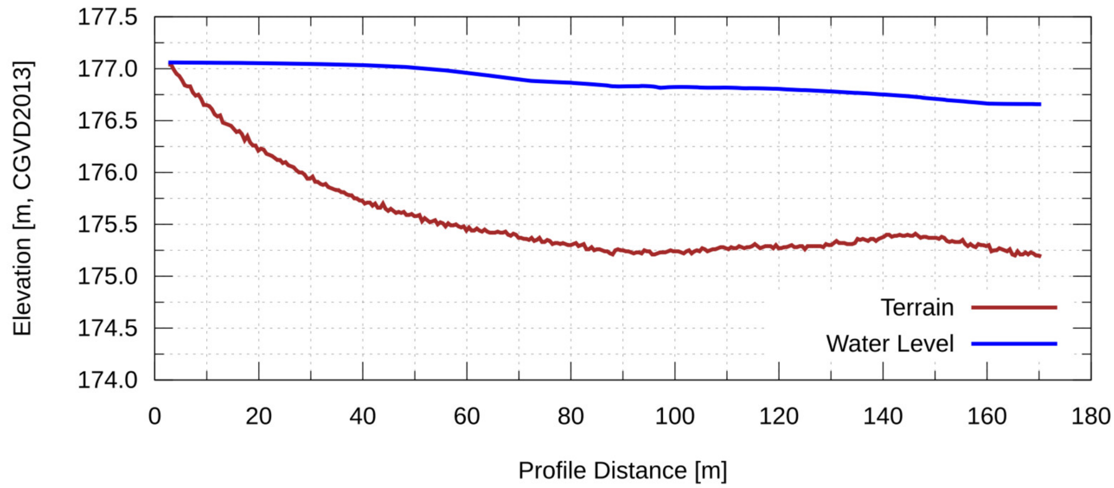
DATE: FEB 2024

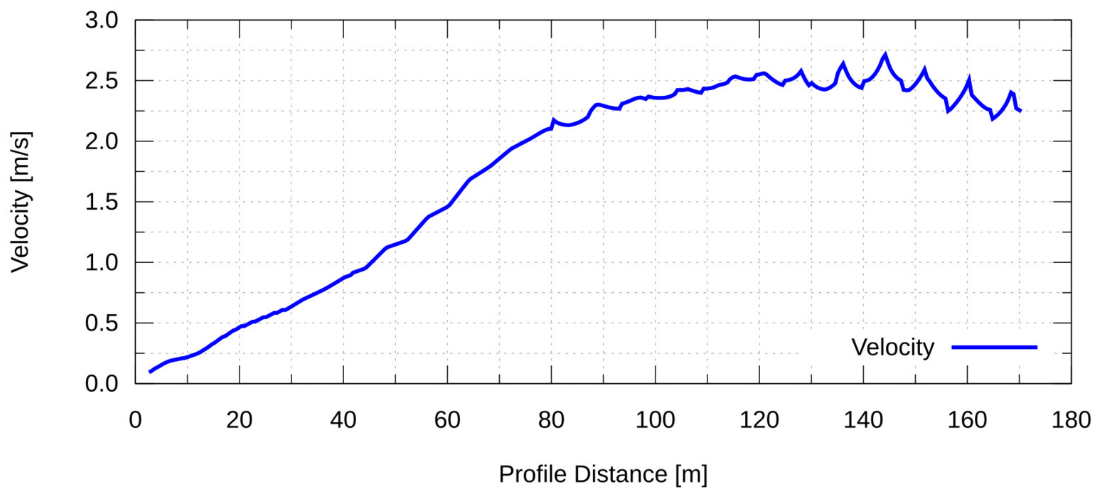
DRAWN BY: PP

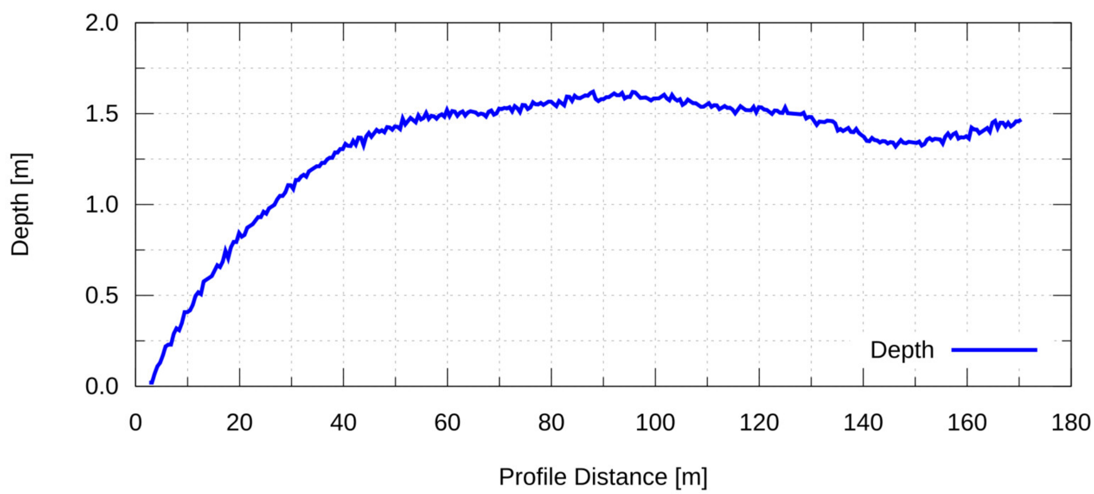
SCALE: AS NOTED

FIG. NO: 8-9

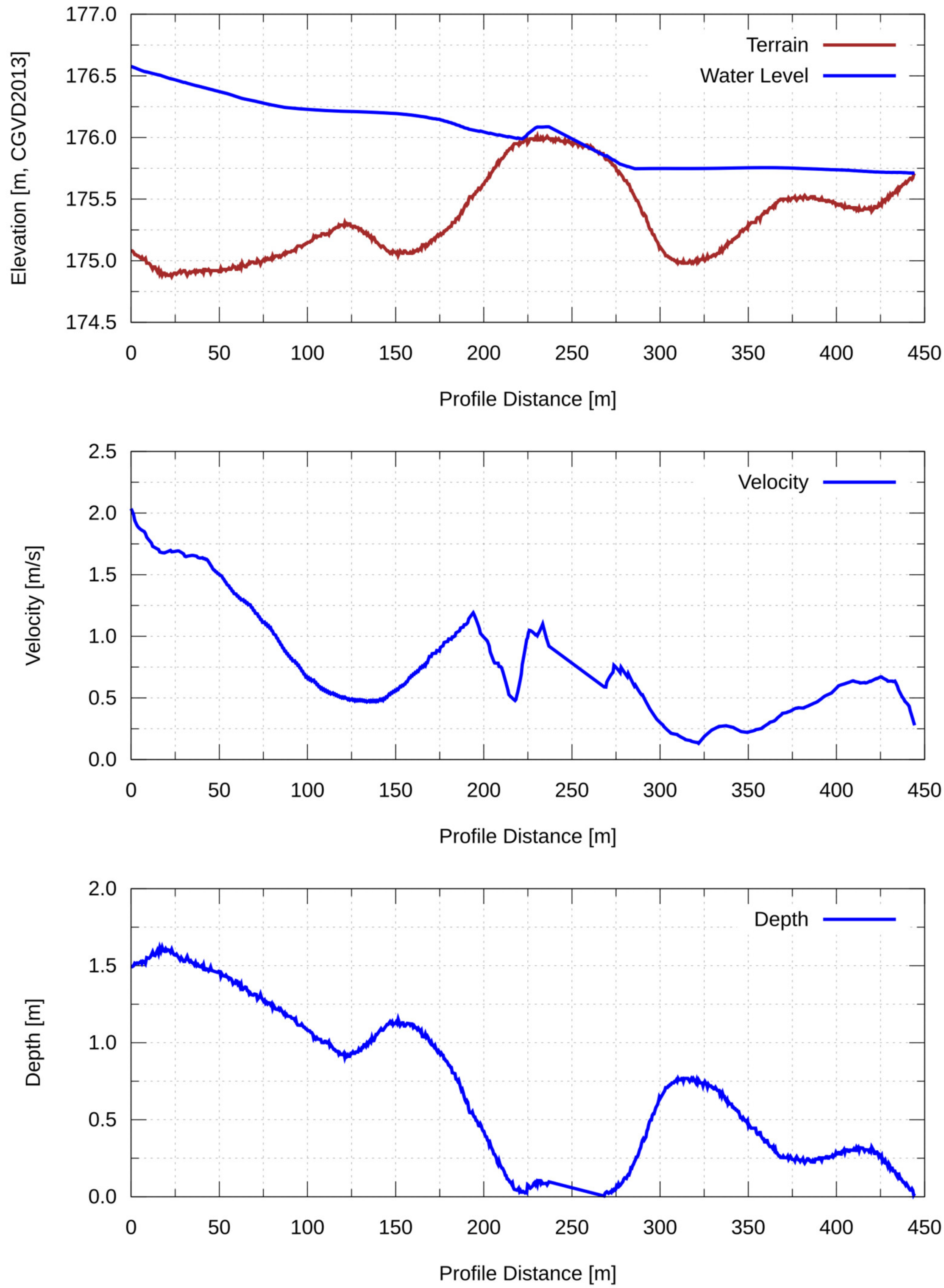
REV.: 0



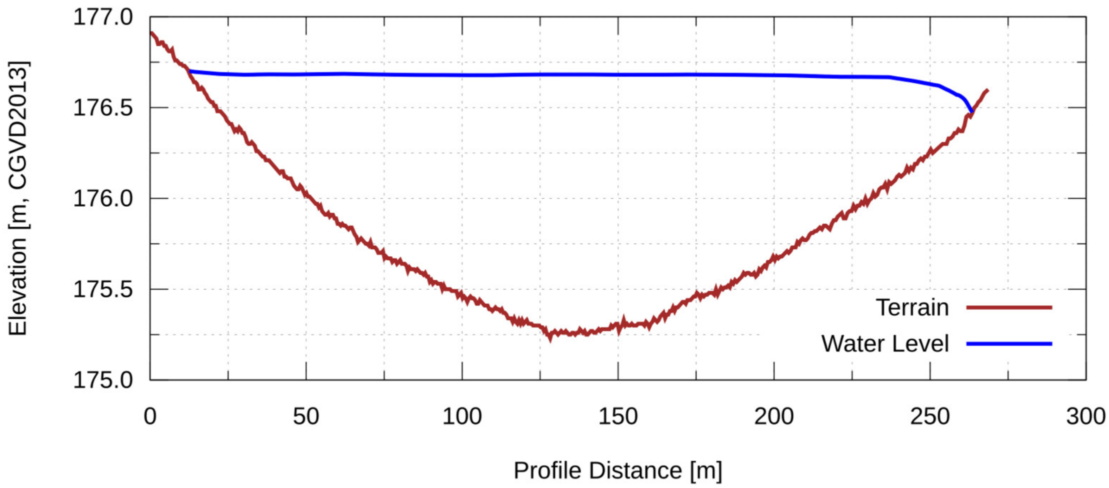


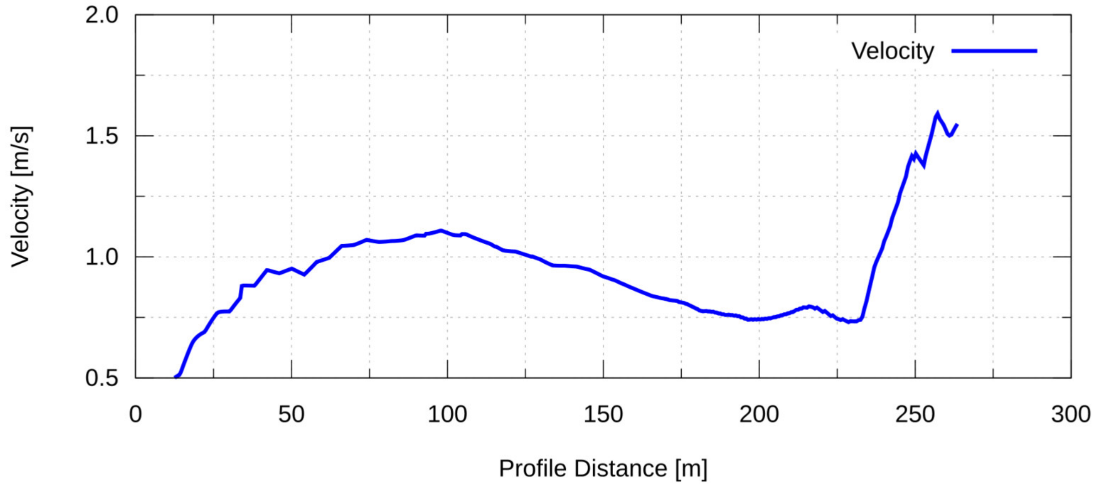


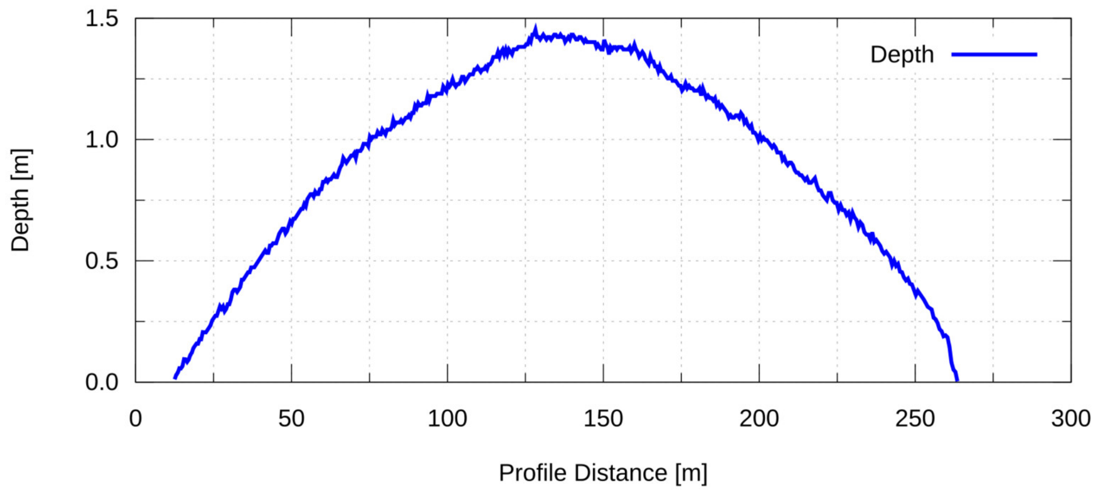
LOC 6: COLIN STREET NORTH		DRAWN BY: PP
		SCALE: AS NOTED
RIVERINE AND COASTAL FLOODPLAIN MAPPING, PORT BRUCE, ONTARIO	PROJECT NO: 2886-021	FIG. NO: **8-10**
	DATE: FEB 2024	REV.: **0**



LOC 7: COLIN STREET EAST	TRUE	DRAWN BY: PP
		SCALE: AS NOTED
RIVERINE AND COASTAL FLOODPLAIN MAPPING, PORT BRUCE, ONTARIO	PROJECT NO: 2882-021	FIG. NO: **8-11**
	DATE: FEB 2024	REV.: **0**







LOC 8: IMPERIAL ROAD SOUTH

RIVERINE AND COASTAL FLOODPLAIN MAPPING, PORT BRUCE, ONTARIO

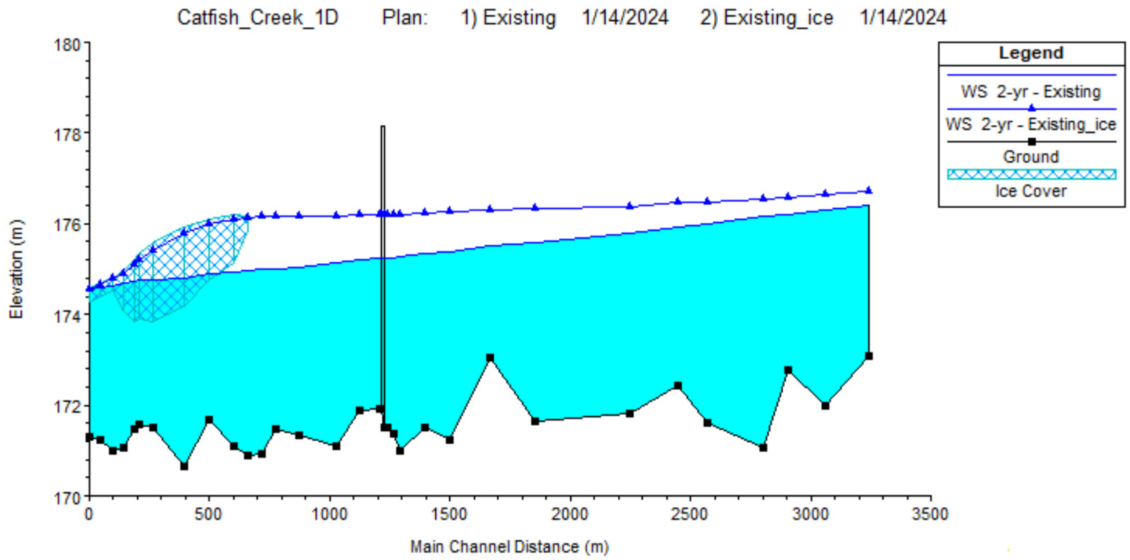
DRAWN BY: PP	
SCALE: AS NOTED	
FIG. NO: 8-12	REV.: 0
PROJECT NO: 2882-021	
DATE: FEB 2024	

8.1.3 Ice Jams

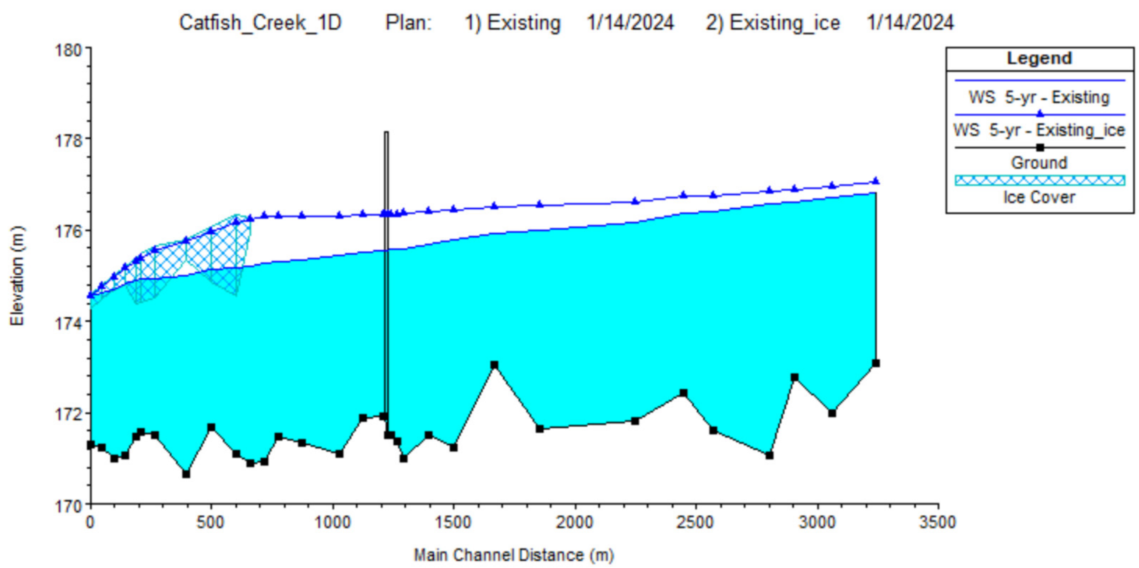
Simulations of ice jams are extremely sensitive to the thickness and extent of ice in the harbour and upstream areas. For this assignment an ice thickness of 0.3 m is selected in the area between the harbour and approximately 700 m upstream. The water level in Lake Erie was set as in the clear water flood conditions (documented above). Simulations of water surface profiles were carried out using the HEC-RAS 1D hydraulic model developed in this work, with the ice jam option turned on. Flood events ranging from 2-yr to 20-yr were considered in the simulations.

Results of the ice jam analyses are shown in Figures 8-13 (for 2-yr and 5-yr floods) and Figure 8-14 (for 10-yr and 20-yr floods). By inspecting the output it is readily observed that ice jams significantly increase the upstream water surface elevations compared to clear water flood conditions. In comparing the 20-yr flow under clear water conditions (no ice) with the 20-yr flow under ice jam conditions, the water levels around the Imperial Road Bridge could rise as much as 1 m. This means that area around the Imperial Road Bridge could experience flooding with a 0.3 m thick ice jam with a 20-yr flow that is comparable to flood profiles that are somewhere between the 100-yr and Regional (clear water flow) conditions.

Even though limited in scope, ice jam analyses performed have revealed extreme effects of harbour ice as the mechanism that could significantly exacerbate riverine flooding in Port Bruce (a known consequence). The thickness and extent of the ice will determine the severity of the flooding that could ensue. A more comprehensive assessment is required before any more conclusions could be drawn. In the meantime, it is recommended to keep monitoring ice thickness and its upstream extent on regular basis, as it can have significant impact on riverine flooding in Port Bruce.

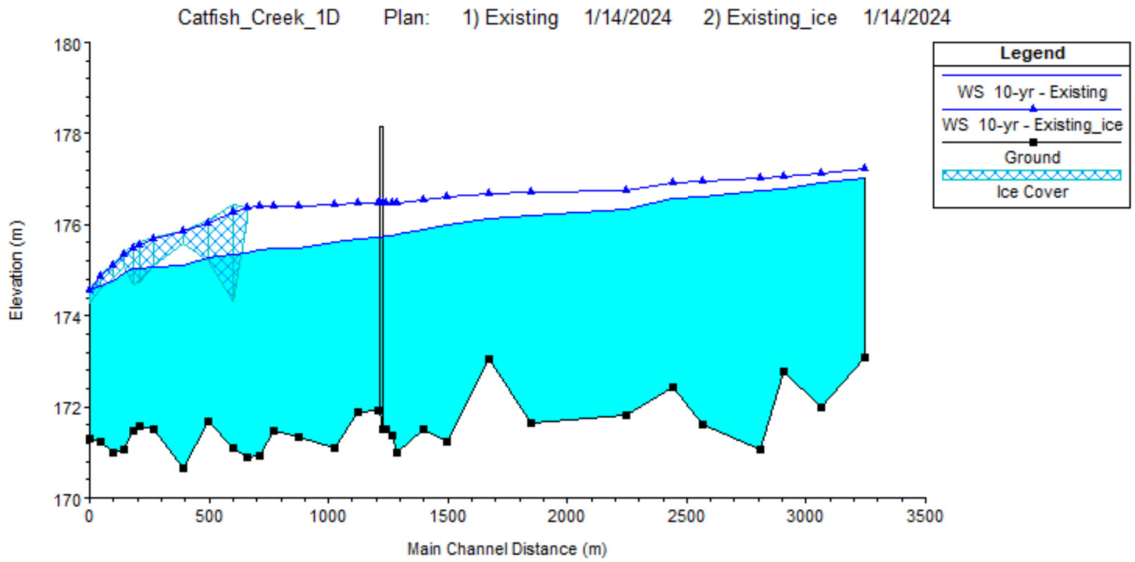


A) 2-YR CLEAR WATER AND ICE JAM WATER SURFACE PROFILES

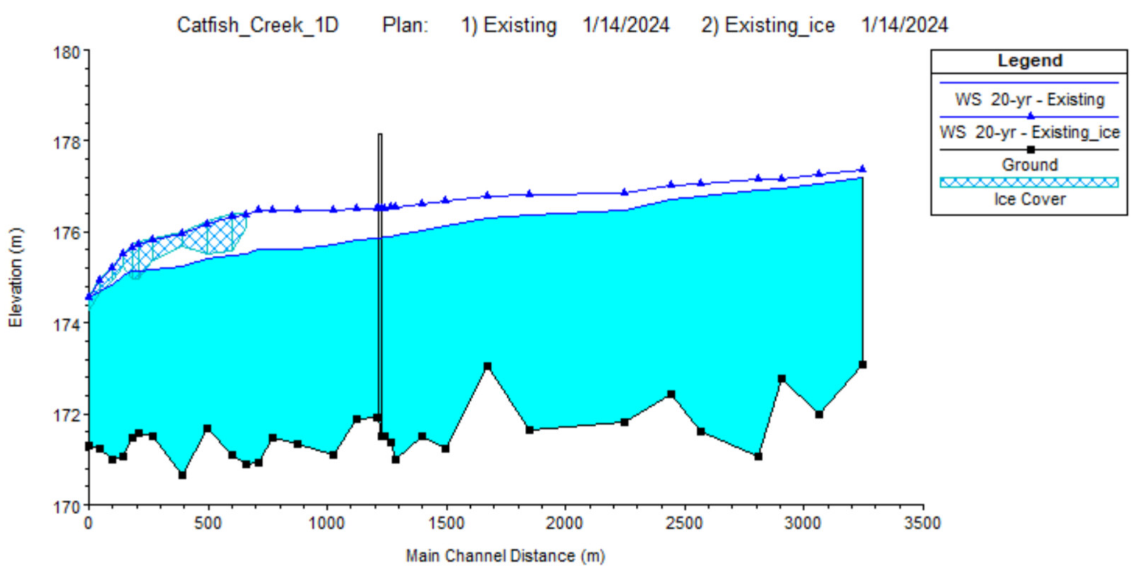


B) 5-YR CLEAR WATER AND ICE JAM WATER SURFACE PROFILES

ICE JAM PROFILES, 2- AND 5-YR FLOWS	TRUE	DRAWN BY: PP	
		SCALE: AS NOTED	
RIVERINE AND COASTAL FLOODPLAIN MAPPING, PORT BRUCE, ONTARIO	PROJECT NO: 2882-021	FIG. NO: **8-13**	REV.: **0**
	DATE: FEB 2024		



A) 10-YR CLEAR WATER AND ICE JAM WATER SURFACE PROFILES



B) 20-YR CLEAR WATER AND ICE JAM WATER SURFACE PROFILES

ICE JAM PROFILES, 10- AND 20-YR FLOWS

RIVERINE AND COASTAL FLOODPLAIN MAPPING, PORT BRUCE, ONTARIO

TRUE	DRAWN BY: PP		
	SCALE: AS NOTED		
PROJECT NO: 2882-021	FIG. NO: 8-14	REV.: 0	
DATE: FEB 2024			

8.2 Coastal Floodproofing Elevations and Guidelines

8.2.1 Coastal Floodproofing Elevations

Coastal floodproofing elevation are established through analysis of outputs from numerical models, along with interpretation of results and professional judgment. Given the updated MNR (2001) definition of the floodproofing standard, the starting water level in the calculations is higher than previously used. For comparison, in Philpott (1991) the analysis was done with the starting water surface elevation that corresponded to the 100-yr instantaneous water level, or 175.6 CGVD28, which converts to 175.15 m CGVD2013. In the present work the starting water level is the 100-yr mean monthly water level plus the 100-yr storm surge, which was established as 175.65 m CGVD2013.

The starting water level for the computations of wave uprush is 0.5 m higher than was previously used in Philpott (1991).

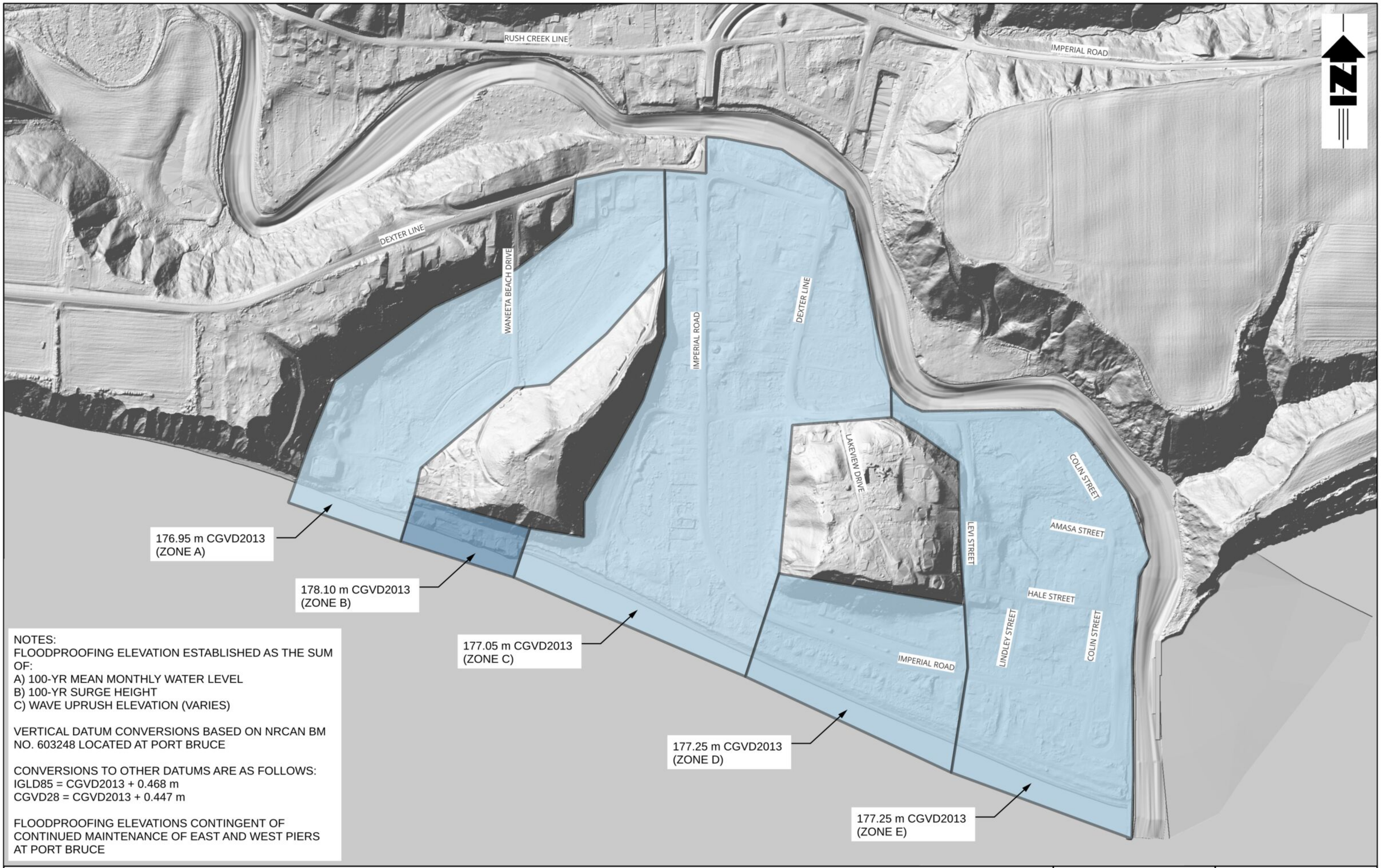
The starting design still water level of 175.65 m CGVD2013 was used in the SWASH model simulations. The results at each of the eight transects were analyzed at several nodes along the beach and inland areas. For the purposes of mapping the spatial extent of the wave uprush, several transects exhibited similar characteristics and were aggregated (or lumped) together.

Wave runup R2% values were computed at eight transects along Port Bruce beach, and are shown in Table 8-3. The reported values represent wave uprush heights assuming waves are freely able to pass through the structure that is on stills. For structures considering perimeter foundations an additional factor is required to the values from Table 8-3. Application of the additional factor for perimeter foundations is discussed in Appendix A.

Table 8-3: Port Bruce Beach Wave Runup Heights and Floodproofing Elevations

Zone	Wave Uprush Height R2% [m]	Floodproofing Elevation FPE [m, CGVD2013]
A	1.1	176.95
B	2.45	178.10
C	1.4	177.05
D	1.6	177.25
E	1.6	177.25

The floodproofing elevation is established by adding the wave uprush height to the design still water level of 175.65 m CGVD2013. The floodproofing elevations are shown graphically in Figure 8-15.



NOTES:
FLOODPROOFING ELEVATION ESTABLISHED AS THE SUM OF:
A) 100-YR MEAN MONTHLY WATER LEVEL
B) 100-YR SURGE HEIGHT
C) WAVE UPRUSH ELEVATION (VARIES)

VERTICAL DATUM CONVERSIONS BASED ON NRCAN BM NO. 603248 LOCATED AT PORT BRUCE

CONVERSIONS TO OTHER DATUMS ARE AS FOLLOWS:
IGLD85 = CGVD2013 + 0.468 m
CGVD28 = CGVD2013 + 0.447 m

FLOODPROOFING ELEVATIONS CONTINGENT OF CONTINUED MAINTENANCE OF EAST AND WEST PIERS AT PORT BRUCE

COASTAL FLOODPROOFING ELEVATIONS

RIVERINE AND COASTAL FLOODPLAIN MAPPING
PORT BRUCE, ONTARIO

TRUE

PROJECT NO. 2882-021

DATE: FEB 2024

DRAWN BY: PP

SCALE: 1:7,500

FIG. NO:	REV.:
8-15	0

8.2.2 Coastal Development Guidelines

It is recommended that CCCA adopts the floodproofing elevation established for Port Bruce, as they conform to Provincial standards.

Appendix A of this document includes a set of development guidelines that were originally formulated for Port Stanley (TRUE, 2022), but have been revised and updated for Port Bruce. The development guides are consistent with the MNR (2001) Technical Guideline, and adopt a similar approach to neighbouring Conservation Authorities. The approach adopted recognizes existing development and provides strategies to eliminate the risk to human life and property damage over time from coastal hazards. The regulatory setbacks established in the original CCCA Shoreline Management Plan (Philpott, 1991) are assumed to still apply.

8.3 Generalized Flood Elevation

Given that riverine and coastal flood elevations have been assessed, a definition of the Generalized Flood Elevation can be set. In areas such as Port Bruce, where flooding can come from riverine (Catfish Creek) or coastal (Lake Erie) sources, the governing flood elevation is defined as the greater among the two.

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Appendix A – Port Bruce Coastal Development Guides

Port Bruce Beach Development Guides
February 1, 2024

DEVELOPMENT ACTIVITY[1]	DYNAMIC BEACH[2]	FLOOD HAZARD[3]
Repairs and/or Maintenance (no intensification of use)	N/A	Permitted without restrictions. Advise of flood risk and potential damage.
Interior Alterations (no intensification of use)	N/A	Permitted without restrictions. Advise of flood risk and potential damage.
Relocation of existing buildings and structures inland	Not Permitted.	Permitted, it meets Lot Redevelopment requirements.
Minor and Major Additions. Additions beyond 100% are considered a Lot Redevelopment.	Not Permitted	Permitted, with restrictions. Additions to be located on the least exposed portion of the lot, and no closer to the lake than the existing structure. Dry[6] passive[5] floodproofing standard applies. The floodproofing elevation are specified on Figure 8-15. Top of foundation to be at, or above, the floodproofing elevation. All services and utilities to be above the floodproofing elevation. Structures encouraged to be supported on piles/piers when floodproofing elevation is 0.8 m or greater than existing grade, and structure is directly exposed to the lake. Perimeter foundations may be considered. However, foundations having floodproofing elevation 0.8 m or greater above existing grade, and directly exposed to the lake, to have i) top of foundation increased by 0.3 m (no opening below top of foundation), and ii) face of the foundation wall exposed to the lake protected with riprap and/or armour stone (top of rock to coincide with top of foundation). All crawl spaces must be non-habitable, and used for

DEVELOPMENT ACTIVITY[1]	DYNAMIC BEACH[2]	FLOOD HAZARD[3]
		non-permanent storage only. Openings below the floodproofing elevation are not permitted. All services and utilities to be above the floodproofing elevation. Water level to be used in foundation design is specified as 0.4 m above the design still water level. Foundation design and site grading details to be provided by a qualified civil/structural engineer.
Minor Structures (non-habitable accessory structures, tool-sheds, movable structures such gazebos and covered decks, pavilions, etc) without utilities and maximum size of 14 m^2	Not Permitted	Permitted, with restrictions. Advise of flood risks and potential damage. Safety concerns due to flooding hazards are to be addressed considering site conditions and nature and use of structure. Design must ensure there is no opportunity for conversion into habitable space in the future.
Major Structures (non-habitable accessory structures such as garages and car-ports) with utilities and maximum size of 50 m^2	Not Permitted	Permitted, with restrictions. To be located on the least exposed portion of the lot, and no closer to the lake than the existing habitable structure. Wet[7] passive[5] floodproofing standard applies. The floodproofing elevation are specified on 8-15. All services are required to be above the floodproofing elevation. The elevation for ingress and egress route to meet or exceed that of the existing habitable structure. Water level to be used in foundation design is specified as 0.4 m above the design still water level. Foundation design and site grading details to be provided by a qualified civil/structural engineer.
Habitable Space above Major Structures (dwelling unit above garage/car port)	Not Permitted	Permitted, as long floodproofing requirements of Major Structures is met.

DEVELOPMENT ACTIVITY[1]	DYNAMIC BEACH[2]	FLOOD HAZARD[3]
		Habitable space must be on the second level.
Lot Redevelopment (reconstruction of buildings or structures, other than those destroyed by flooding or erosion)	Not Permitted.	Permitted, with restrictions. Buildings and structures to be located on the least exposed portion of the lot. The number of dwelling units must remain unchanged if Provincial floodproofing standards for safe access/egress cannot be satisfied. The elevation for ingress and egress route to meet or exceed that of the existing structure on site prior to re-development. Dry passive floodproofing standard applies. The floodproofing elevation are specified on Figure 8-15. Top of foundation to be at, or above, the floodproofing elevation. All services and utilities to be above the floodproofing elevation. Structures encouraged to be supported on piles/piers when floodproofing elevation is 0.8 m or greater than existing grade, and structure is directly exposed to the lake. Perimeter foundations may be considered. However, foundations having floodproofing elevation 0.8 m or greater above existing grade, and directly exposed to the lake, to have i) top of foundation increased by 0.3 m (no opening below top of foundation), and ii) face of the foundation wall exposed to the lake protected with riprap and/or armour stone (top of rock to coincide with top of foundation). All crawl spaces must be non-habitable, and used for non-permanent storage only. Openings below the floodproofing elevation are not permitted. All services

DEVELOPMENT ACTIVITY[1]	DYNAMIC BEACH[2]	FLOOD HAZARD[3]
		and utilities to be above the floodproofing elevation. Water level to be used in foundation design is specified as 0.4 m above the design still water level. Foundation design and site grading details to be provided by a qualified civil/structural engineer.
New Dwellings on existing vacant lots	Not Permitted.	Permitted, with restrictions. Buildings and structures to be located on the least exposed portion of the lot. The elevation for ingress and egress route to meet provincial standards. Dry[6] passive[5] floodproofing standard applies. The floodproofing elevations are specified on 8-15. Top of foundation to be at, or above, the floodproofing elevation. All services and utilities to be above the floodproofing elevation. Structures encouraged to be supported on piles/piers when floodproofing elevation is 0.8 m or greater than existing grade, and structure is directly exposed to the lake. Perimeter foundations may be considered. However, foundations having floodproofing elevation 0.8 m or greater above existing grade, and directly exposed to the lake, to have i) top of foundation increased by 0.3 m (no opening below top of foundation), and ii) face of the foundation wall exposed to the lake protected with riprap and/or armour stone (top of rock to coincide with top of foundation). All crawl spaces must be non-habitable, and used for non-permanent storage only. Openings below the floodproofing elevation are not permitted. All services and utilities to be above the floodproofing elevation.

DEVELOPMENT ACTIVITY[1]	DYNAMIC BEACH[2]	FLOOD HAZARD[3]
		Water level to be used in foundation design is specified as 0.4 m above the design still water level. Foundation design and site grading details to be provided by a qualified civil/structural engineer.
Swimming Pools (above or below ground)	Not Permitted.	Permitted (if not directly exposed to the lake), with restrictions. To be located on the least exposed portion of the lot. Lake level to be used in swimming pool design is specified as 0.4 m above the design still water level (specified in the main body of this document)). Swimming pool design details to be provided by a qualified civil/structural engineer. Servicing and utilities to be located above the floodproofing elevation.
Decks, Boardwalks, and Fixed Walkways	Permitted, provided design has no adverse impacts on ongoing coastal processes. May require a site specific assessment from a qualified coastal engineer or a coastal geomorphologist.	Permitted, provided safety concerns due to flood hazards are addressed considering site conditions and nature and use of development.

Notes:

1 Development Activity means the same as the definition of development under the Conservation Authorities Act.

2 Dynamic Beach limits are delineated in CCCA Shoreline Management Plan (Philpott, 1991).

3 Flood Hazard limits are defined in MNR (2001) as the elevation contour coinciding with wave uprush (using 20-yr design wave conditions) on top of 100-yr instantaneous water levels.

4 Floodproofing standard is defined as a combination of structural changes and/or adjustments incorporated into the basic design and/or construction or alteration of individual buildings, structures or properties subject to flooding hazards so as to reduce the risk of flood damages, including wave uprush and other water related hazards along the shorelines of the Great Lakes (MNR, 2001).

5 Passive floodproofing are techniques which are permanently in place and do not require advance warning and action in order to make the floodproofing and/or flood protection measure effective (MNR, 2001).

6 Dry floodproofing means the use of fill, columns, or design modifications to elevate openings in buildings or structures above the floodproofing standard (MNR, 2001).

7 Wet floodproofing is defined as protection to maintain structural integrity by avoiding external unbalanced forces from acting on buildings during and after a flood, to reduce flood damage to contents, and to reduce the cost of post flood clean up. As such, wet floodproofing requires that the space below the level of the flood standard remain unfinished, be non-habitable, and be free of service units and panels, thereby ensuring minimal damage. Also this space must not be used for storage of immovable or hazardous materials that are buoyant, flammable, explosive or toxic. Furthermore, access ways into and from a wet floodproofed building must allow for safe pedestrian movement (MNR, 2002).

8 Activities proposed other than those outlined in the above development guides may require services from a qualified coastal and/or civil/structural engineers. Such services may include site specific assessments, site reviews and/or designs. Scope of work for such services are to be established during consultations with CCCA staff.