Item 13.1.1
Attachment 2

HalifACT Acting on Climate Together

Creation of Flood Hazard Maps
Halifax Regional Municipality

Final Report

CBCL

221111.00 • April 2024

Disclaimer

This Report and associated Flood Hazard Mapping has been prepared by CBCL Limited (the Consultant) for the Halifax Regional Municipality (the Client) for the sole use and benefit of the Client and to support the Client's staff in making decisions around planning, critical infrastructure prioritization, and emergency management in a holistic way. This Report and associated Flood Hazard Mapping shall not be used for purposes other than those stated above and is not to be relied upon by Third Parties. This Report and associated Flood Hazard Mapping is to be used in accordance with, and with full understanding of, the stated Uncertainty and Limitations described within the text of the report. Further, the Consultant is not responsible for the accuracy of any information supplied by or produced by others.

04	Final Report	A. Wilson	05-Apr-2024	G. Waugh
03	Final Draft Report	A. Wilson	1-Dec-2023	S. McClearn
02	Draft Report	A. Wilson	16-Aug-2023	S. McClearn
01	Internal Draft Review	A. Wilson	20-Jul-2023	S. McClearn
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Project No. 221111.00

Maritime Centre, 1505 Barrington Street, Suite 901, Box 606, Halifax, NS,
B3J 2R7 | 902-421-7241 | CBCL.ca | info@CBCL.ca

April 05, 2024

Victoria Fernandez
Stormwater and Coastal Resilience Engineer
Engineering and Building Standards / Planning and Development
Halifax Regional Municipality
PO Box 1749
Halifax, NS B3J 3A5

Ms. Fernandez:

RE: Creation of Coastal, Pluvial, Fluvial Flood Hazard Maps for Halifax Regional Municipality

CBCL was retained in Fall 2022 by HRM to create Coastal, Pluvial, and Fluvial Flood Hazard Maps for the entirety of the Halifax Regional Municipality (HRM), and CBCL is pleased to submit this final report and the associated flood hazard maps (both PDF and digital versions) that we have developed in response to the requirements stated by HRM.

The mapping associated with this report results from a high-level study intended to support the HRM to make decisions about Current and Future Planning, Critical Infrastructure Prioritization, and Emergency Management Planning and Operations. It will also serve to help identify areas that may require a more detailed assessment.

Please accept this updated Final report, and do not hesitate to contact the undersigned with any questions or comments you may have with regards to the contents of this report.

Yours very truly,

CBCL Limited

[redacted]

Graham Waugh, P.Eng.,
Water Resources Engineer

[redacted]

[redacted]

Reviewed by:
Alexander Wilson, M.Eng., P.Eng.
Senior Technical Specialist – Water Resources

Project No.: 221111.00

Project Team:

Team Member	Project Role
Sandy McClearn, P.Eng., PMP, LEED AP	Project Manager
Alexander Wilson, M.Eng., P.Eng.	Project Advisor and Reviewer
Graham Waugh, P.Eng.	Water Resources Fluvial Lead
Ying Zhang, M.E.Sc., P.Eng.	Pluvial Modeller
Noushin Mohammadzadeh, M.Sc.,	Additional Resources
Uthra Sreekumar, M.Eng., E.I.T.	Fluvial Modeller
Amaury Camarena, P. Eng., M.Sc.	Coastal Lead
Vincent Leys, P.Eng., M.Sc., PMP	Coastal & Climate Technical Reviewer
Tom Kozlowski, P.Eng., M.A.Sc.	Coastal Modeller
Léa Braschi, M.Sc.	Climate and Water Resources Scientist
Sarah O'Rourke, B.Sc., CTech.	Geographical Information Systems Specialist

Executive Summary

Climate Change is projected to be driving an increase in the frequency and intensity of extreme weather events across the country, and the Halifax Regional Municipality's (HRM) climate action plan (HalifACT: Acting on Climate Together) identifies flood hazard mapping as a key initial product to support municipal staff in making decisions around Current and Future Planning, Critical Infrastructure Prioritization, and Emergency Management Planning and Operations.

CBCL Limited (CBCL) was selected in September 2022 to create and supply the necessary bespoke and extensive pluvial, fluvial, and coastal spatial flood hazard maps for the entire HRM under multiple climate scenarios, recurrence intervals, and time frame combinations. The flood mapping is being provided at a resolution of 1-10 m and will support HRM in making decisions at the Area and Site levels of resolution. This mapping effort is the first step of a larger comprehensive initiative designed to provide HRM with the necessary tools to identify climate-driven flooding hazards. The project scope specifically excludes the possibility that it will be used to inform decisions at the Asset Design level, and the project output is not intended to replace the need for detailed flood modelling specific to assets or locations for design projects, but rather to identify such needs where they may arise.

To produce the report and its accompanying mapping, CBCL used industry standard best practices and methodologies wherever available to carry out a program that included field data collection of water levels for the purpose of calibrating the resulting flood mapping, a precipitation analysis, analysis and modelling of both Pluvial/Fluvial and Coastal flooding events resulting from the modelled precipitation events, and finally the production of the resulting flood hazard mapping for the designated return periods and scenarios. To manage the unprecedented scale and breadth of this study, CBCL also innovated some new approaches to facilitate the processing of the vast amount of data that was generated. This included for example the development and use of customized flood modelling for processing and entering data into the models, as well as flood mapping tools that reduced the need for manual adjustments.

Approach and Methodology

Our approach to this project included several components:

- **Data Collection:** The basis for the assessment was established by collecting data from a variety of sources. This included historical rainfall patterns, river flow dynamics, sea-level records, infrastructure sizing, wave and wind conditions and detailed topo-bathymetric information.

- **Precipitation Analysis:** A thorough analysis of historical rainfall data was conducted to understand the frequency and distribution of rainfall patterns that influence pluvial and fluvial (rain-induced) flooding, a foundational component of our assessment. Climate changes to extreme precipitation were then used to generate future precipitation scenarios.
- **Pluvial-Fluvial Analysis and Modelling:** The interaction of rainfall, flows, and flooding events demanded rigorous modeling. This required calibration of models against historical flood events.
- **Coastal Analysis and Modelling:** Coastal regions, particularly in the face of storm surges, posed unique challenges. We employed advanced numerical models to project fundamental aspects of coastal flood hazards including a combination of present and projected future water levels. These extreme water levels were derived including contributions of tides, storm surge and sea level rise.
- **Flood Mapping:** The stated purpose of the project is the production of flood hazard maps. These comprehensive maps span several flood scenarios including a wide range of annual exceedance probabilities (return periods) and future projections. They are intended to support the initial identification of flood hazards, supporting preliminary decisions related to emergency responses, urban planning, and enhancing of infrastructure resilience.

Navigating Uncertainties

Uncertainties and limitations exist in any undertaking of this magnitude. Where data gaps could not be resolved, experience and judgement were used to develop logical processes underpinning realistic assumptions, such that the results of the modelling can still support the intended purposes of the project. Several uncertainties are explained in more detail in the report:

- **Data Gaps:** Despite exhaustive research efforts, some historical and source data gaps persist, affecting the precision of the model results. For example, the HRM Lidar data is missing/has erroneous elevation data in many lakes, which needed to be addressed through careful estimation and judgment to produce reasonable model results.
- **Model Calibration:** The calibration of numerical models for flooding risks involves the adjustment of model parameters to match observed or historical data. However, even with calibration, the inherent complexity of natural processes (such as seasonal changes, antecedent conditions, etc.) introduces uncertainty. This is an uncertainty that exists with every hydrologic and hydraulic model.
- **Infrastructure Data:** Available information consists of partial and fragmented data for infrastructure (bridges, culverts, dams, etc.), as opposed to detailed field surveys, which introduces uncertainties. Lack of structure geometric information was addressed through Lidar and air photo analysis, combined with typical structure overburden depth information.
- **Excluded Factors:** Secondary flood mechanisms like wave run-up, ice jamming, and debris jamming are not explicitly considered in the flood mapping.
- **Limited Future Scenarios:** Our flood mapping focuses on the 2100 horizon, potentially missing long-term climate impacts beyond this timeframe.

- **Uncertainties associated with climate change projections:** Climate science is highly uncertain and adjustments to future projected changes in temperature and precipitation are constantly being made.

Recommendations to Address Known Uncertainties

For purposes that require more detailed modelling, such as asset protection and design, we recommend that HRM complete the following:

- **Infrastructure Surveys:** A comprehensive survey of bridges, culverts, and dams is essential for enhancing model accuracy.
- **Data Gap Reduction:** Addressing data gaps such as bathymetry or average water levels will increase the representativeness of the calculated water levels.
- **High-Risk Area Prioritization:** Identifying and prioritizing high-risk areas streamlines resilience efforts.
- **Community Engagement:** Regular public consultations aid data collection and map refinement.
- **Urban Drainage:** Investigating the role of urban drainage systems is essential for urban flood mitigation.
- **Secondary Flood Mechanisms:** Secondary mechanisms like snow melt-induced flooding, wave run-up and ice jamming demand further exploration.
- **Long-Term Monitoring:** Establishing a long-term hydrometric monitoring plan for high-risk watercourses.
- **Model Maintenance:** Regular model updates, refinements and calibration, provide improvements to accuracy.

Results and Conclusion

This project has produced an extensive set of flood mapping data, covering approximately 10,000 km of watercourse and waterbody length as well as over 300 km of coastline.

In summary, this analysis provides HRM with a regional delineation of potential flooding risks under existing and projected climate conditions and supports the identification of locations where more detailed analysis may be required. The flood hazard maps attached to this report, spanning different return periods and future horizons, serve as valuable resources to inform decision-making regarding infrastructure, emergency response, and urban planning.

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List of Acronyms

AEP	Annual Exceedance Probability
BIO	Bedford Institute of Oceanography
CHI	Computational Hydraulics International
CMIP5	Fifth Coupled Model Intercomparison Project
DEM	Digital Elevation Model
DFO	Fisheries and Oceans Canada
DHI	Danish Hydraulic Institute
ECCC	Environment and Climate Change Canada
EPA	Environmental Protection Agency
HRM	Halifax Regional Municipality
HyVSEP	Hydrographic Vertical Separation Surfaces
IDF	Intensity-Duration-Frequency
IEA	International Energy Agency
IPCC	Intergovernmental Panel on Climate Change
NRCan	Natural Resources Canada
NRCC	Northeast Regional Climate Centre
NSPW	Nova Scotia Public Works
RCP	Representative Concentration Pathways
RP	Return Periods
SDMM	Servant, Dunbrack, McKenzie and MacDonald Ltd.
SLR	Sea Level Rise
SSP	Shared Socioeconomic Pathways
SWMM	Storm Water Management Model
WSC	Water Survey of Canada

1 Introduction

CBCL Limited (CBCL) was selected by the Halifax Regional Municipality (HRM) to prepare flood mapping for the entire municipality, including coastal areas and areas of stormwater accumulation. The primary objective of this study is to prepare a set of defensible flood extent maps for major rivers and coastline within the study area, considering both existing and projected future climate conditions. Flood mapping provides valuable information on flood hazards which can help inform decision making and planning in communities. The present study represents the first step in a suite of tools HRM is working on to identify climate-driven flooding hazards and to improve resiliency. The project scope specifically excludes the possibility that it will be used to inform decisions at the Asset Design level, and the project output is not intended to replace the need for detailed flood modelling specific to assets or locations for design projects. The present report is intended to be a technical document that describes the engineering methodology that was followed to assemble and analyse the existing data, develop and calibrate hydrologic, hydraulic and coastal models, and prepare high level flood maps for a wide range of flood frequencies and climate scenarios. This project has produced an extensive set of flood mapping covering approximately 10,000 km of watercourse and waterbody length as well as over 300 km of coastline.

1.1 Background

The HRM contains the largest (5,475.57 square kilometres) and most dense urban area of Atlantic Canada, mostly located along an extensive amount of coastline, with multiple rivers and numerous lakes. These remarkable natural assets in the region come with flood hazards that can impact the health, safety, security, and economic well-being of the residents of HRM. High tides combined with storm surges and major rainfall events can all produce extreme water levels within urban areas, inland rivers and lakes, as well as along the coast. With the impact of climate change, these growing risks are expected to impact not only land use, services, and infrastructure, but also emergency response efforts.

In 2019, Halifax declared a climate emergency, joining countries and major cities around the world as well as nearly 500 Canadian municipalities. Climate Change directly impacts emergency management and the ability of associated personnel to support preparedness, response, and recovery efforts. The growing risks will compound the challenges on emergency services personnel as well as nongovernmental organizations. Land use planning is a key tool to prepare and adapt to the growing risks to public safety and the mental well-being of HRM residents. Planning to allow safe and resilient communities to grow and flourish needs to be supported by reliable information such as bespoke flood mapping products. Many parts of the municipality are lacking such flood data. For this reason, HRM has undertaken a jurisdiction-wide initiative of Pluvial, Fluvial, and Coastal Flood Hazard Mapping.

Comprehensive and reliable flood mapping across the entire HRM will support the municipality with various planning and management objectives:

- Land Use Planning: Land use planning and zoning by-laws require sound information on flood hazard areas. Floodplain mapping is necessary information to help not just the Municipality understand the risks, but also developers, landowners, and the public.
- Infrastructure/Asset Management: Instream infrastructure including but not limited to culverts, bridges, and dams, are all vulnerable to flooding as well as themselves sometimes contributing to flooding by limiting flow. Additionally, infrastructure and assets within flood hazard areas can be vulnerable to flooding. Understanding the impacts of infrastructure on flooding, and the infrastructure at risk of flooding, helps municipalities plan and prioritize future infrastructure improvements.
- Emergency Management: Emergency management before and during a natural hazard such as flooding requires prior knowledge of at-risk populations and important access and egress routes. Flood maps provide the necessary information to identify critical and vulnerable access routes through communities and provide first responders with information on potential risks they may encounter.
- Flood Mitigation: Flood mitigation requires a thorough understanding of flood hazards and risks. The most critical aspects of flood mitigation are using flood mapping and flood risk information to identify and prioritize potential options.

1.2 Study Area

The study area, as presented in Figure 1-1, includes the urban surface drainage infrastructure (pluvial), watercourses (fluvial), and the coastline of the HRM. The southern, coastal edge of the study area extends from Hubbards on St. Margaret's Bay in the West across the metropolitan area and along the Eastern Shore past Sheet Harbour to Ecum Secum. The inland boundary is bounded to the north by the communities of Hammond's Plains, Enfield, and the Musquodoboit Valley. All major built-up areas including the metropolitan and rural villages are included. The study area includes several major rivers including the Sackville River, Musquodoboit River, Tangier River, and West River (Sheet Harbour). In addition, it encompasses innumerable streams, creeks, and lakes. Figure 1-2 exemplifies the vast amount of water features in the Municipality.

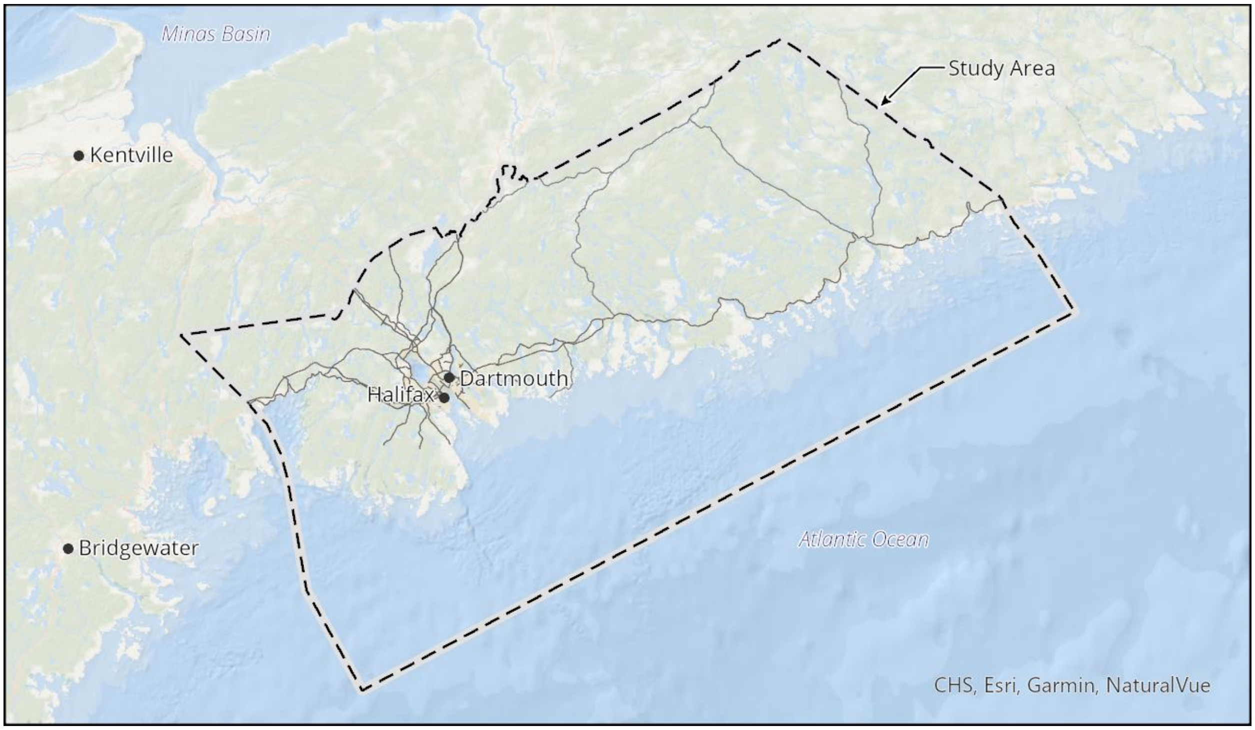

Figure 1-1: Study area.

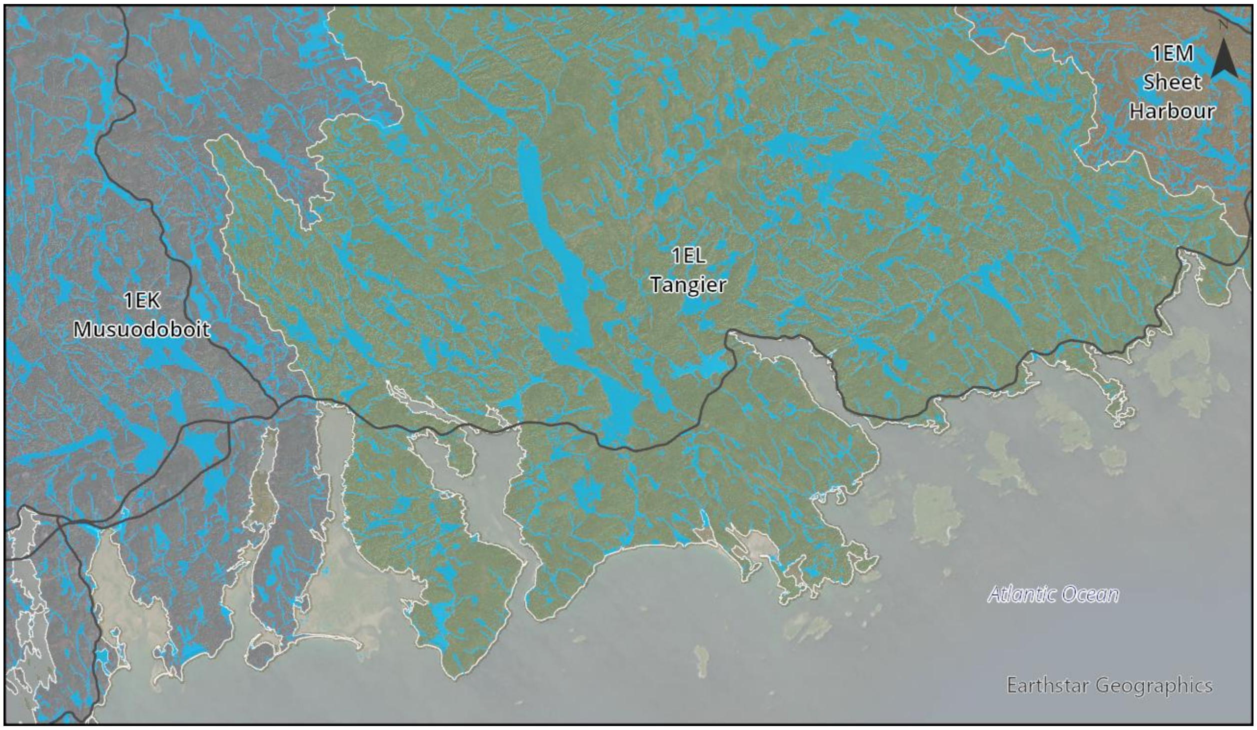

Figure 1-2: Example of extensive watercourse and lake network.

1.3 Scope of Work

The HRM has defined the scope of work for this project to encompass the creation of pluvial, fluvial, and coastal flood mapping for the entirety of the HRM. The following tasks outline the major components of the project:

- Background data review.
- Water level monitoring in selected watercourses and coastal environments.
- Rainfall and sea level rise projections associated with climate change.
- Hydrological analysis and modelling of watersheds.
- Hydraulic modelling of watercourses, including structures and open channels.
- Hydraulic modelling of pluvial runoff (stormwater) via surface drainage (sub-surface storm sewer systems have not been included).
- Coastal extreme water level analysis.
- Hydrodynamic modelling of extreme coastal flood conditions.
- Production of flood maps for a range of return period storms in present and future climate conditions.
- Workshop with HRM to identify vulnerable infrastructure/land/services throughout the study area.

1.4 Flood Mapping Scenarios

The goal of this study is to produce flood maps for the following flood event probabilities outlined in Table 1-1:

Table 1-1: Flood Event Probabilities included in Study

Return Period	Time Horizon	Intended Use
2	2022, 2050, 2100	Emergency Management, Infrastructure
5	2022, 2050, 2100	Emergency Management
10	2022, 2050, 2100	Emergency Management
20	2022, 2050, 2100	Emergency Management and Land Use Planning
50	2022, 2050, 2100	Emergency Management
100	2022, 2050, 2100	Emergency Management and Land Use Planning
200	2022, 2050, 2100	Land Use Planning
500	2100	Infrastructure Prioritization
1000	2100	Infrastructure Prioritization

1.4.1 Fluvial and Pluvial Flooding

Fluvial flooding, also referred to as riverine flooding, is defined as that which occurs along watercourses and waterbodies when flow exceeds the capacity of the channel causing it to spill over the banks into the adjacent lands (e.g. floodplains). Pluvial flooding is a result of extreme precipitation that exceeds the drainage capacity of the landscape, causing overland flow where there might be no natural watercourses. Pluvial flooding is most often associated with urban areas that have high imperviousness and rely on storm drainage infrastructure.

Fluvial flooding from a storm on July 22, 2023, at the Sackville River (Photo taken by Mark Greenwood).

Pluvial flooding on Pleasant Street, August 2023. (Photo taken by HRM).

Figure 1-3: Examples of pluvial and fluvial flooding.

Fluvial and pluvial flood analyses have been carried out on the basis of:

1. A simulated precipitation-driven flood event using IDF data for a duration of 24 hours.
2. Non-frozen ground with typical antecedent soil moisture conditions.

Additional flood mechanisms such as ice or debris jamming, rain with snowmelt, and specific seasonal conditions were not included in the scope of this study. It is recommended that further study be completed to assess the relative importance of these factors.

Flood mechanisms relating to dam operations and hydraulic control structures in the watersheds have not been explicitly incorporated into the models and analysis. Where present, dam and control structures were modelled as passive and without active operation such as storage or release of flows.

1.4.2 Coastal Flooding

Coastal flooding can result from several physical processes that may combine to produce extreme high-water levels. Definitions of these processes can be found in the following itemized list and are displayed graphically in Figure 1-4.

- **Tides:** The rise and fall of the surface of oceans, bays, etc., due principally to the gravitational interactions between the moon, sun, and earth. The characteristics of tides such as amplitude (vertical interval between high and low water levels) and frequency vary depending on a variety of factors including but not limited to geographical position, dimensions, and depth of the body of water.
- **Storm Surge:** Storm surge can be defined as the difference between the observed water level during a storm and the predicted astronomical tide. Storm surges are created by meteorological effects on sea level, such as **wave set-up** (driven by shoreward wind stresses causing an increase in water level) and low atmospheric

pressure. Local storm surge is best estimated with historical tide gauge measurements combined with numerical modelling.

- **Wave Run-up:** This is the additional height that individual waves attain as they dynamically rush up a shoreline, before their wave energy is dissipated due to friction and gravity. it is also referred to as **swash uprush**. Due to the site-specific nature of this parameter, it is excluded from this largescale study.
- **Sea Level Rise (SLR):** SLR is an increase in the ocean's water levels resulting from climate change. Global Mean SLR is caused primarily by two factors: (1) thermal expansion of water from increasing ocean temperatures, and (2) the melting of glaciers and polar ice sheets. Global Mean SLR will accelerate due to climate change, causing increased risks of coastal erosion and flooding. Relative sea level rise (RSLR) represents Global Mean SLR corrected with local factors including but not limited to vertical land motion, or changes in local oceanic circulation. For the future (2050 and 2100) flooding projection, SLR-RCP[1]8.5 (median[2]) is included.

Coastal flood mechanisms from localized wave action such as wave run-up and overtopping were not included in the scope of this study, and therefore estimates are based on static levels. It is noted that site-specific wave run-up may represent a significant contribution to coastal flooding along exposed sites, such as Peggy's Cove. It is recommended that further assessments be carried out to estimate the relative importance of these factors to flooding in the study area.

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[1] Representative Concentration Pathways (RCP).

[2] RCP 8.5 refers to the concentration of carbon that delivers global warming at an average of 8.5 watts per square meter across the planet. The RCP 8.5 pathway delivers a temperature increase of about 4.3˚C by 2100, relative to pre-industrial temperatures. The median represents the 50th percentile projection.

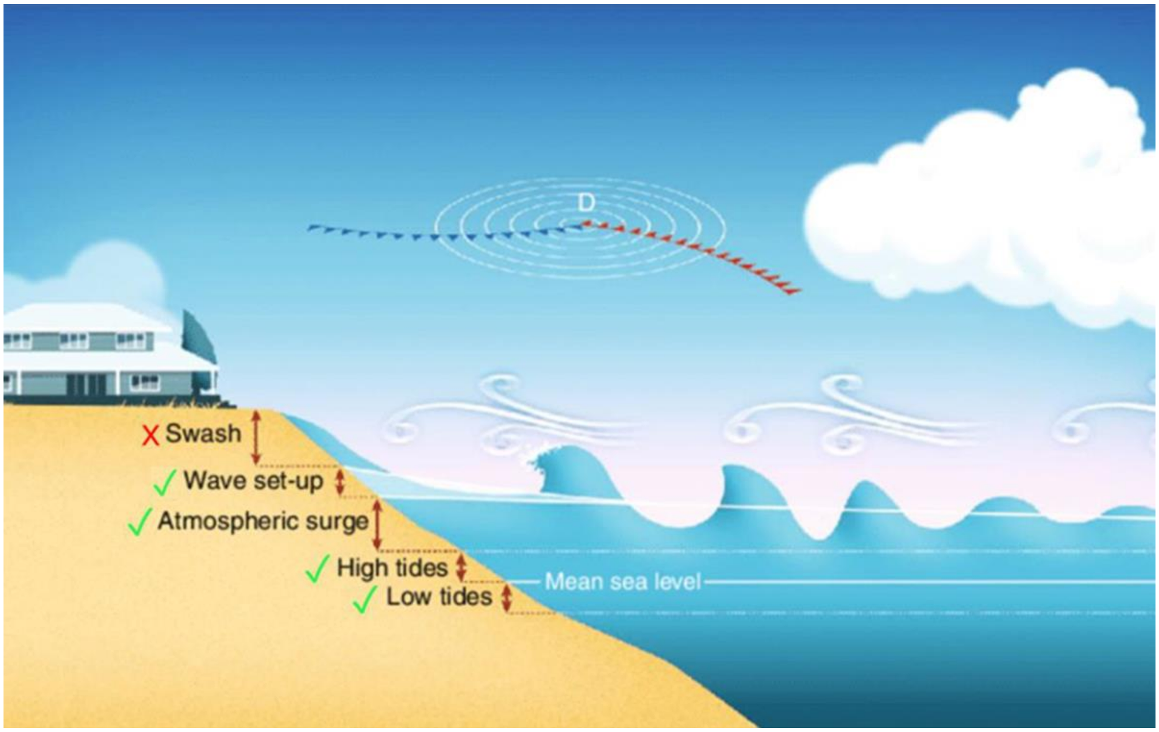

Figure 1-4: Components of extreme total water levels and extreme static water levels (figure adapted from Melet et al., 2018). Green checkmarks indicate components included in extreme static water levels for this study and red Xs indicate extreme water level components excluded from this study.

2 Data Collection

Data required for the project included both publicly available desktop data as well as field data gathered during the project.

2.1 Desktop Data Collection

The project relied primarily on existing desktop data available through public sources, including:

- The 2018 provincial 1-meter LiDAR Digital Elevation Model (DEM) provided by HRM.
- National scale, land cover mapping from Natural Resources Canada.
- Soil drainage class mapping from Agriculture and Agri-Food Canada (2013).
- Nova Scotia Public Works (NSPW) watercourse crossing structures and culvert mapping.
- NS provincial road network mapping.
- NS provincial watercourse and waterbody mapping.
- HRM road network mapping.
- HRM watercourse crossing and culvert mapping.
- Historical river flow data from Water Survey of Canada (WSC).
- Historical climate data from Environment and Climate Change Canada (ECCC).
- Hydrographic Vertical Separation Surfaces (HyVSEPs) (NRCan).
- Canadian tidal elevations (DFO).
- Offshore regional storm surge (Bernier and Thompson, 2006).
- Relative Sea-Level Change tool (NRCan).
- Bathymetric soundings (CHS-NONNA).
- MSC50 wind and wave hindcast (DFO).
- Sackville Rivers Floodplain Study Phase II (CBCL 2017).
- Shubenacadie Lakes Flood Study (CBCL 2020).
- Halifax Stormwater Event Inventory Mapping (Servant, Dunbrack, McKenzie, and MacDonald Ltd. [SDMM 2015] Halifax Harbour Marginal Coastal Study (CBCL 2020a).
- Coastal Engineering Study for Peggy's Cove Master Plan (CBCL 2020b).
- Halifax Regional Municipality Extreme Water Levels (CBCL 2022).
- High water levels in Big Lake, caused by Hurricane Dorian (September 7, 2019) and changes to Long Beach, Nova Scotia; Geological Survey of Canada (Taylor *et.al.* 2021).

The Project Area was first divided into nine sections based on the 1:10,000 Nova Scotia Primary Watersheds layer. HRM provided their 1m resolution LiDAR DEM (2018) as the basis for this project, and CBCL additionally used the 1m resolution LiDAR DEM (2019) available from GeoNOVA to extend the DEM for the primary watersheds that straddled the HRM boundary and were not fully covered within the HRM DEM. CBCL first clipped both DEM sources to the 9 primary watersheds and then adjusted their projections in order to merge them into a single DEM to be used in the model. HRM stated a requirement in the RFP for the use of NAD_1983_CSRS_2010_MTM_5_Nova_Scotia, but the HRM DEM was provided in the WGS 84 coordinate system and the DEM from GeoNOVA was provided in the NAD_1983_CSRS_UTM_Zone_20N projection. CBCL converted both to meet the RFP requirement.

2.2 Water Level Monitoring

Water level monitoring of watercourse and coastal water levels was carried out for the project to gather data for model calibration and validation[3]. A total of six sites were monitored over the course of the project from approximately December 1, 2022, through April 15, 2023. This included three watercourse locations across the HRM representing a range of watershed types and sizes, as well as three coastal sites representing different tidal conditions across the study area (Figure 2-1). Table 2-1 describes the locations, setting, equipment types, and monitoring period.

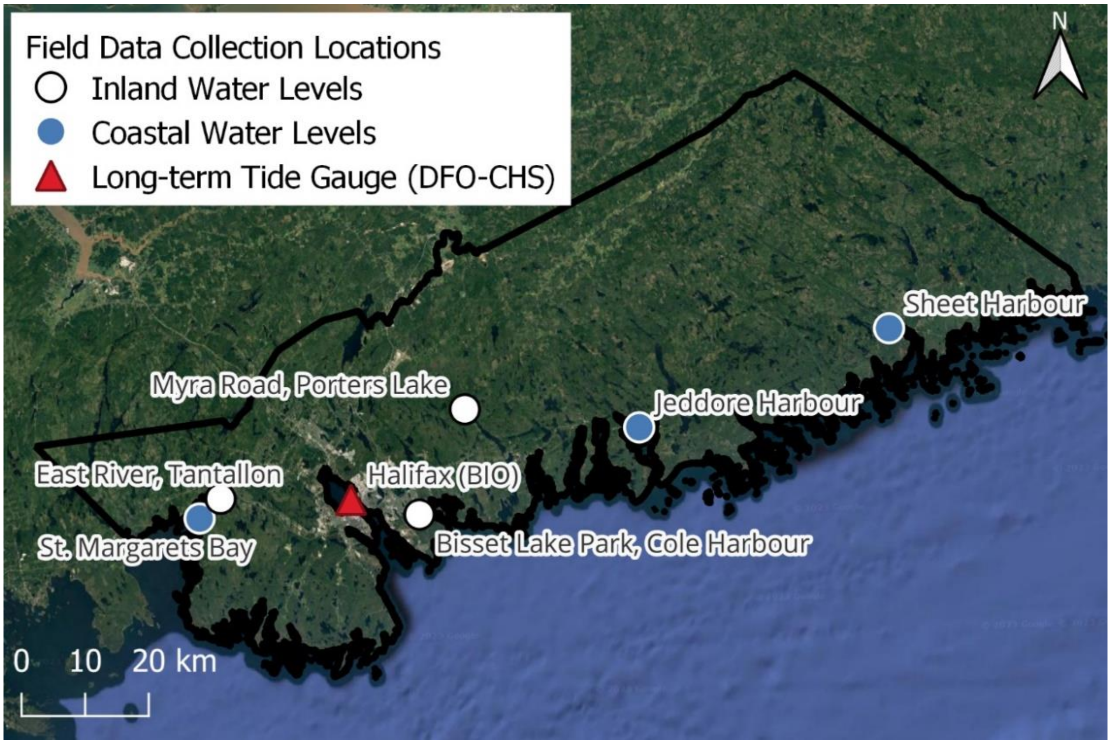

Figure 2-1: Locations of CBCL-deployed instruments and DFO long-term tide gauge.

[3] Calibration in coastal modelling involves adjusting model parameters to match measured data, ensuring the model accurately reflects real-world conditions. Validation assesses the model's performance by comparing its predictions with additional measured data to confirm its reliability.

Table 2-1: Water Level Monitoring Sites

Site	Logger Type	Data Retrieval Method	Monitoring Period	Rationale
East River at St. Margaret's Bay Road (Tantallon) (Inland)	Solinst Levelogger 5	Real Time Data Transfer	Dec. 2022 – Feb. 2023	Mid-sized mixed-use partially rural watershed in western HRM.
Bisset Creek at West Side Road Bridge (Cole Harbour) (Inland)	Solinst Levelogger 5	Real Time Data Transfer	Jan. 2023 – Apr. 2023	Small urban stream and watershed in mid to high density development area.
West Brook at Myra Road (Upstream of Porter's Lake) (Inland)	Solinst Levelogger 5	Manual downloads	Dec. 5, 2022 – Feb. 2023	Mid-sized wilderness watershed in eastern HRM.
St. Margaret's Bay at Shining Waters Marina (Coastal)	ToltHawk Ultrasonic Sensor[4]/ RBR tide gauge	Real Time Data Transfer	Dec. 20, 2022 – Jan. 20, 2023/Feb. 16, 2023 – Apr. 18, 2023	Representative of tides in St. Margaret's Bay and western – Chebucto Head Region.
Jeddore Harbour at Marine Drive Bridge (Coastal)	ToltHawk Ultrasonic Sensor	Real Time Data Transfer	Dec. 5, 2022 – Mar. 21, 2023	Located in eastern HRM within a large enclosed estuary.
Sheet Harbour at West Side Road (Coastal)	ToltHawk Ultrasonic Sensor	Real Time Data Transfer	Dec. 5, 2022 – Apr. 18, 2023	Located in eastern HRM within a funnel shaped estuary.

Equipment selection and installation methods were selected to suit the specific site conditions and in some cases were adjusted for seasonal changes.

[4] After it was discovered that this instrument was malfunctioning and could not be repaired, it was replaced with an RBR Solo[3] wave gauge in tide gauge mode.

The most reliable manner to record water depth is to record water pressure and air pressure separately. The primary equipment for water level monitoring included *Solinst* brand absolute water pressure transducers (Solinst Levelogger 5) with an associated air pressure transducer (Solinst Barologger). The Barologger records atmospheric pressure within the study area, allowing the air pressure effects to be removed from the water depth data. Barometric loggers also record air temperature which is a valuable indicator of winter weather and freeze-thaw cycles and to provide indications of weather pattern effects on the water levels. Where cellular reception was available, a real-time transmission system was installed to transmit data daily (Solinst Levelsender).

ToltHawk brand ultrasonic level sensor devices were deployed at three coastal locations to measure water surface elevation. The ToltHawk devices measure water levels by non-contact ultrasonic signal and wirelessly upload data to a private internet portal for real-time access. This allowed CBCL to monitor water levels in nearly real-time.

A 5-minute recording interval was selected for all Solinst water level loggers and barologger. The recording interval from the ToltHawk sensor was 15 minutes.

Water levels were surveyed at the time of equipment deployment and retrieval to convert the water levels to geodetic elevations. All water levels are in the current Canadian standard CGVD2013 vertical datum.

Measured water levels are depicted for the inland watercourses in Figure 2-2 and for the coastal locations in Figure 2-3. Additional details of the field monitoring program are found in Appendix A.

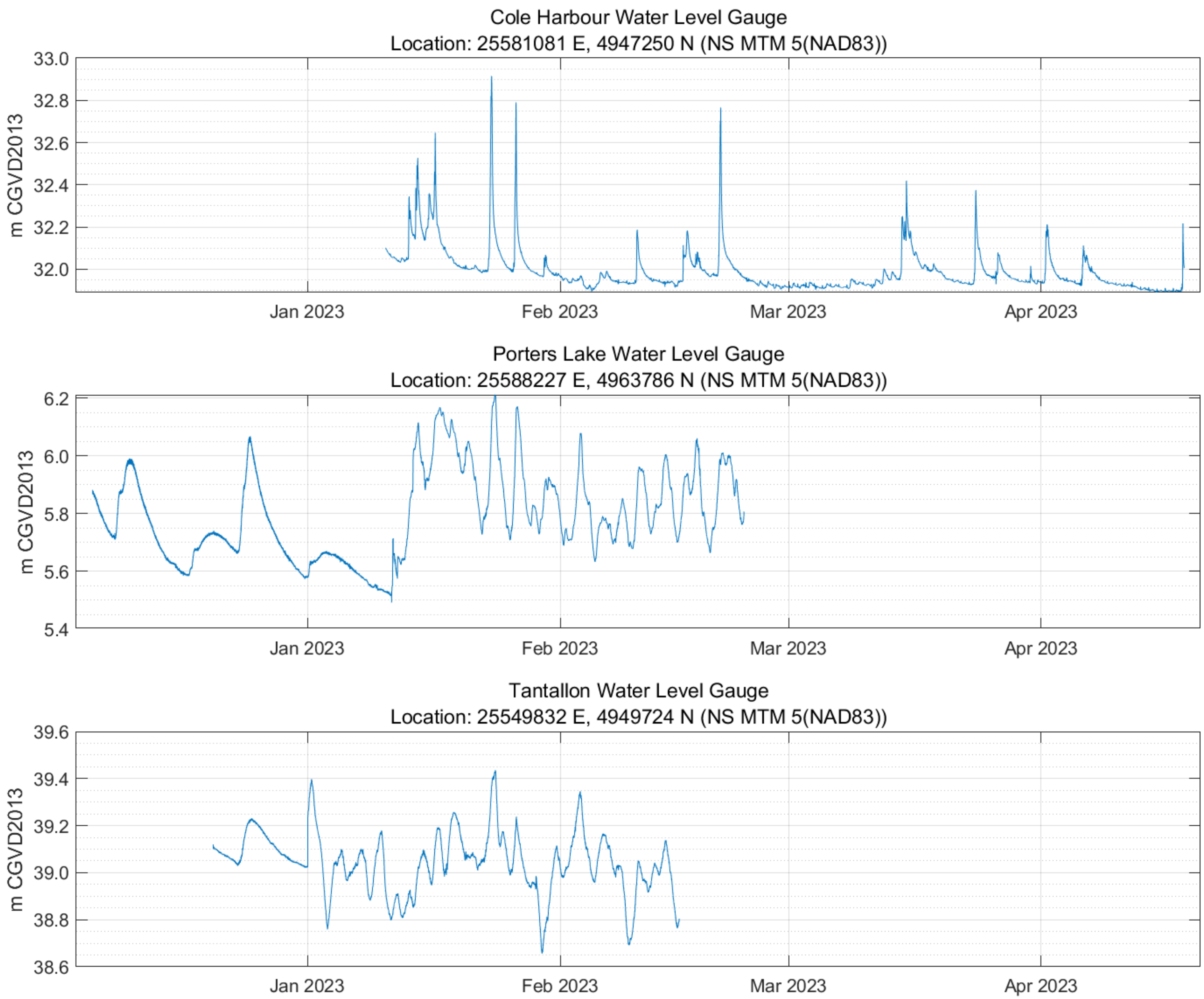

Figure 2-2: Watercourse water level measurements.

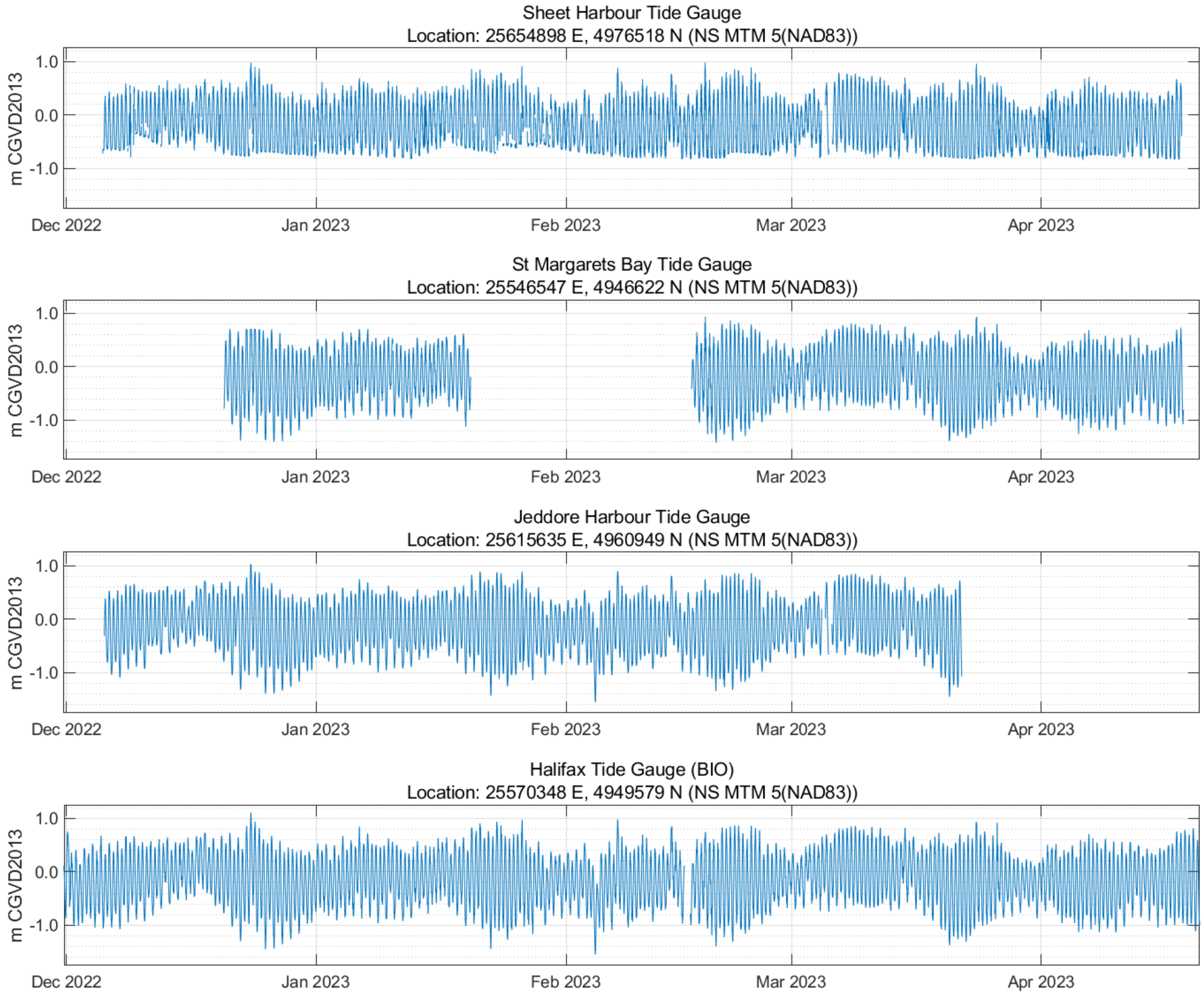

Figure 2-3: Coastal water level measurements.

3 Precipitation Analysis

Extreme rainfall events are defined as instances in which the amount of rain or snow experienced in a location substantially exceeds what is normal. Design storm events are approximations of these events and a central component of hydrological modelling for current and future conditions. Historical climate data is used to derive design storm volumes and intensities.

3.1 Intensity Duration Frequency Data

Intensity-Duration-Frequency (IDF) curves are graphical tools that describe the likelihood of observing a range of rainfall amounts over a range of rainfall durations (from 5 minutes to 24 hours). These are the result of extreme value statistical analyses of historical rainfall intensity and can be used to create synthetic hyetographs (rainfall time-series) to simulate design storm events. IDF curves are important for this project as they enable engineers to derive extreme rainfall events.

Environment and Climate Change Canada (ECCC) is the primary organization that publishes IDF data in Canada. There are two long term climate stations with published IDF data in the study area: Halifax International Airport and Shearwater RCS which replaced the Shearwater Auto climate station. The Halifax Citadel IDF data was considered to be out of date and was not used for this assessment. These two stations are located in the central region of the study area, leaving the eastern and western portions of the study area without published IDF data. Since rainfall patterns can change significantly over a few kilometers, the limited spatial coverage of rainfall data over the study area can create some notable sources of uncertainty.

For this reason, IDF data for each of the study region watersheds was interpolated using the Northeast Regional Climate Centre's (NRCC) Extreme Precipitation in Atlantic Canada Tool (NRCC, 2016). The advantage of using the NRCC Tool is that the interpolation analysis includes both sub-daily rainfall data from 38 ECCC stations (the type used in ECCC's IDF data), as well as daily data from 283 other ECCC stations in the region. This greatly increases the spatial resolution of the IDF data coverage in the Atlantic region. The spatial coverage of climate station data used by the NRCC IDF Tool is shown in Figure 3-1.

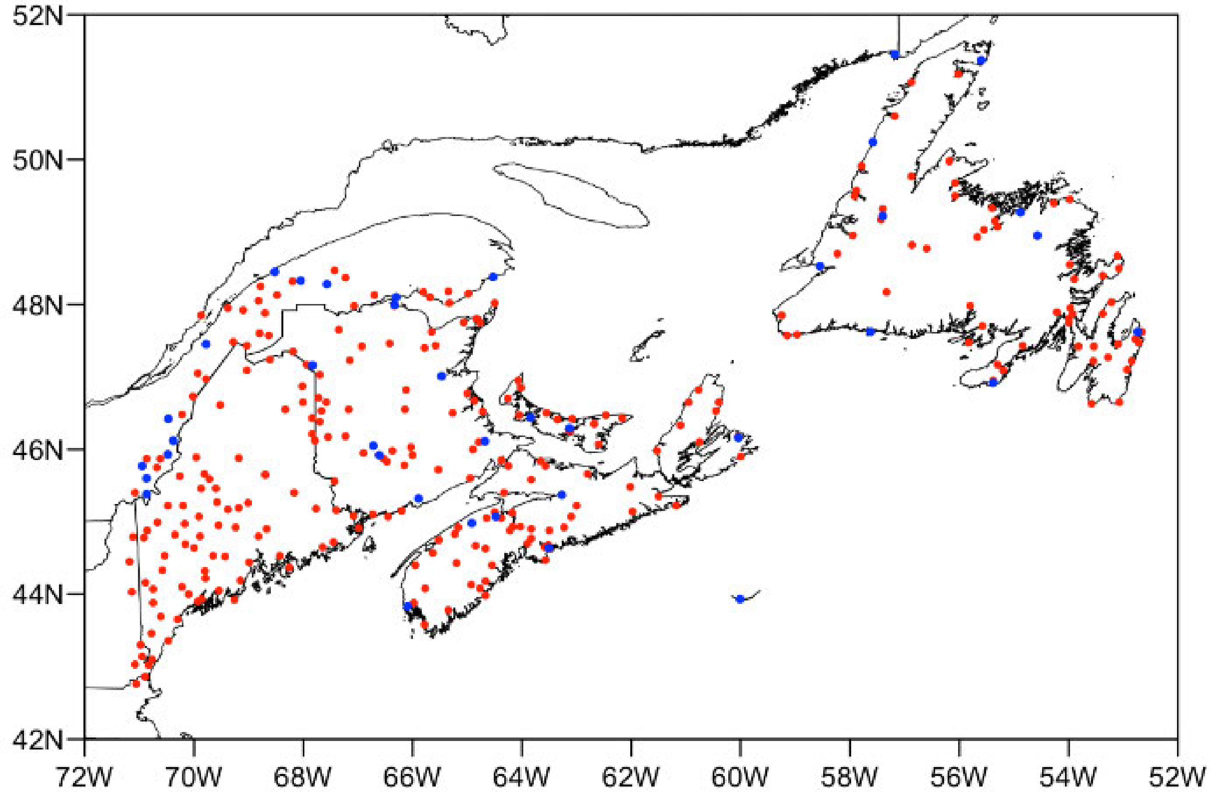

Figure 3-1: Map showing the locations of 38 sub-daily stations (blue) and 283 daily stations (red) used to estimate the gridded precipitation extremes.

A centre point was defined for each of the study area watersheds, as presented in Figure 3-2. This location was then used to interpolate IDF data through the NRCC Tool. For the study region 1DG_1, which is located near the Halifax Airport, the published ECCC IDF data was utilized. For each station, ECCC calculates the rainfall amounts and rates for return periods of 2, 5, 10, 25, 50 and 100 years by fitting a series of annual maximum rainfall rates for the corresponding durations to the Gumbel extreme value distribution using the method of moments. The NRCC tool uses the same methodology for the extreme value fitting with additional stations to obtain the IDF curves for ungauged locations.

In this study, for each selected central point, Precipitation Frequency Duration Tables and Intensity Duration Frequency tables were obtained from the NRCC tool and were used to calculate the statistics (i.e., average and standard deviation) of the original data for a range of different durations in the NRCC tool. The resulting statistics were then used for the interpolation of data to return periods of 20 years and the extrapolation of data to return periods of 200, 500, and 1000 years, by using the same extreme value fitting method. Table 3-1 provides the IDF curve fitting coefficients that were used to define IDF data for each of the study region sub-watersheds. IDF values for the study regions were relatively consistent, showing some variation across the study area. Between the study regions, the 24-hour rainfall depths for the 100-year event ranged from 165 mm to 189 mm. The published ECCC IDF curve for the Halifax Airport yields a value of 24-hour rainfall depth of 169 mm which is consistent with the mid-range of values used.

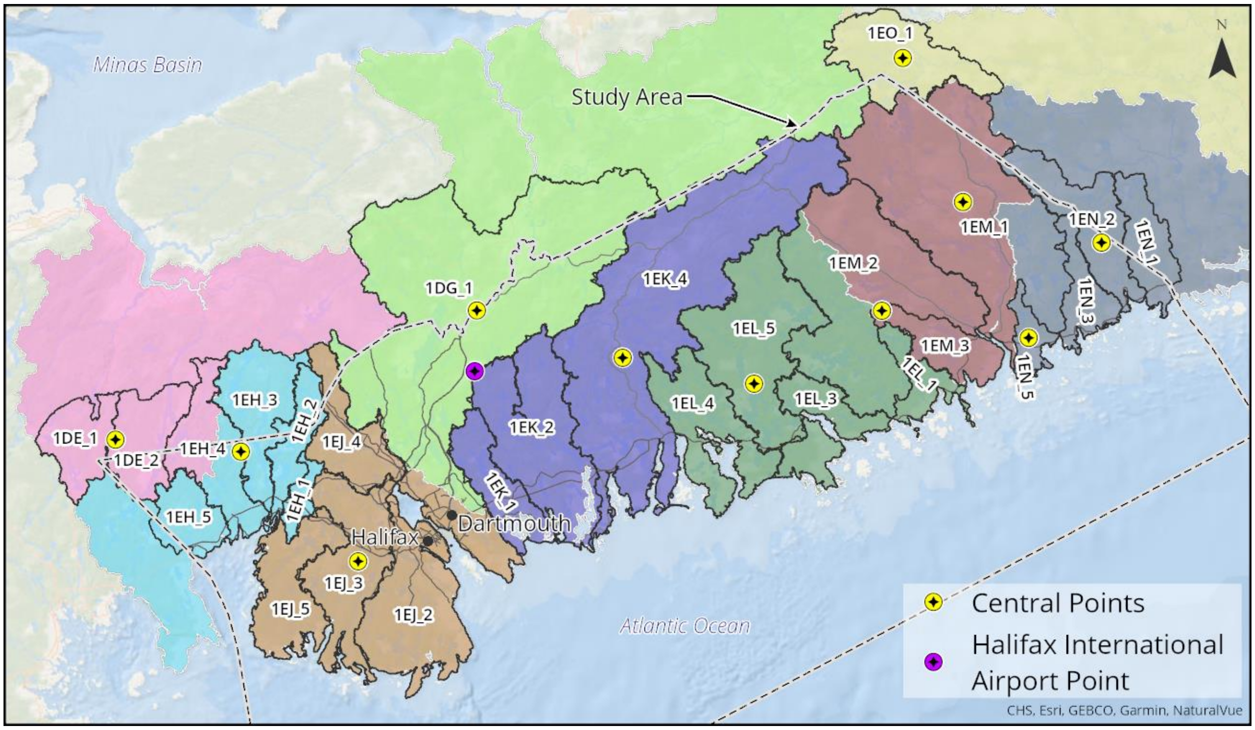

Figure 3-2: Map showing the central points for the study area watersheds.

Table 3-1: IDF Curve Coefficients for Study Region Watersheds

Study Region Sub-Watersheds	IDF Coefficients	2yr	5yr	10yr	25yr	50yr	100yr	20yr	200yr	500yr	1000yr
		Values Provided by NRCC Tool						Values Estimated by Interpolation or Extrapolation			
1DG_1	A	20.0	25.6	29.3	33.9	37.4	40.8	32.8	44.3	48.8	52.2
	B	-0.535	-0.542	-0.545	-0.549	-0.550	-0.552	-0.548	-0.553	-0.555	-0.555
1EO_1	A	21.0	28.3	33.1	39.3	43.8	48.3	37.8	52.8	58.8	63.2
	B	-0.604	-0.605	-0.604	-0.604	-0.604	-0.605	-0.604	-0.605	-0.605	-0.605
1EN_1, 1EN_2, 1EN_3	A	23.2	30.6	35.5	41.7	46.3	50.8	40.2	55.4	61.3	65.9
	B	-0.605	-0.604	-0.605	-0.605	-0.605	-0.605	-0.605	-0.605	-0.605	-0.605
1EN_5	A	22.4	29.9	34.8	41.1	45.7	50.3	39.6	54.9	61.0	65.5
	B	-0.603	-0.604	-0.603	-0.604	-0.603	-0.603	-0.603	-0.603	-0.603	-0.603
1EM_1	A	21.8	29.3	34.2	40.5	45.1	49.7	39.0	54.3	60.3	64.9
	B	-0.603	-0.604	-0.603	-0.604	-0.603	-0.603	-0.603	-0.603	-0.603	-0.603
1EL_3, 1EL_4, 1EL_5	A	21.8	29.7	34.8	41.4	46.3	51.1	39.8	55.9	62.3	67.1
	B	-0.604	-0.604	-0.603	-0.603	-0.603	-0.603	-0.603	-0.603	-0.603	-0.603
1EL_1, 1EM_2, 1EM_3	A	21.7	29.3	34.3	40.6	45.2	49.9	39.1	54.5	60.6	65.3
	B	-0.604	-0.604	-0.603	-0.603	-0.603	-0.603	-0.603	-0.603	-0.603	-0.603
1EK_1, 1EK_2, 1EK_4	A	22.0	30.6	36.2	43.3	48.6	53.8	41.6	59.0	65.9	71.1
	B	-0.604	-0.604	-0.603	-0.603	-0.603	-0.604	-0.603	-0.603	-0.603	-0.603
1EJ_2, 1EJ_3, 1EJ_4, 1EJ_5	A	22.1	28.7	33.1	38.8	43.0	47.1	37.4	51.2	56.6	60.7
	B	-0.604	-0.603	-0.602	-0.602	-0.602	-0.602	-0.602	-0.602	-0.602	-0.602
1EH_1, 1EH_2, 1EH_3, 1EH_4, 1EH_5	A	21.9	28.7	33.2	39.0	43.2	47.4	37.6	51.6	57.2	61.4
	B	-0.604	-0.604	-0.604	-0.605	-0.604	-0.605	-0.605	-0.605	-0.605	-0.605
1DE_1, 1DE_2	A	21.9	28.6	33.0	38.8	42.9	47.1	37.4	51.2	56.7	60.9
	B	-0.604	-0.605	-0.604	-0.604	-0.605	-0.605	-0.604	-0.604	-0.604	-0.604

For this project, the standard Chicago-type Design Storm Rainfall Distribution (an industry-standard rainfall intensity distribution commonly used in urban stormwater management and drainage system design) (Keifer and Chu, 1957) has been used. The Chicago-type Design Storm, a 24-hour storm distribution that utilizes the full range of intensity rates from the IDF data, provides a reasonable basis for design, displays characteristics that are consistent with the statistics of the IDF curve, and is commonly used for small to medium watersheds (0.25 km^2 to 25 km^2) (Corrugated Steel Pipe Institute, 2007; Ontario Ministry of the Environment, 1998), which is in line with the typical subcatchment sizes in the models for this project. Figure 3-3 shows an example of the rainfall distribution for the Shearwater RCS station under 1 in 100 year event, and Appendix F presents the design event rainfall distributions under the existing climate condition for each set of sub-watersheds, and the corresponding characteristics of the design events.

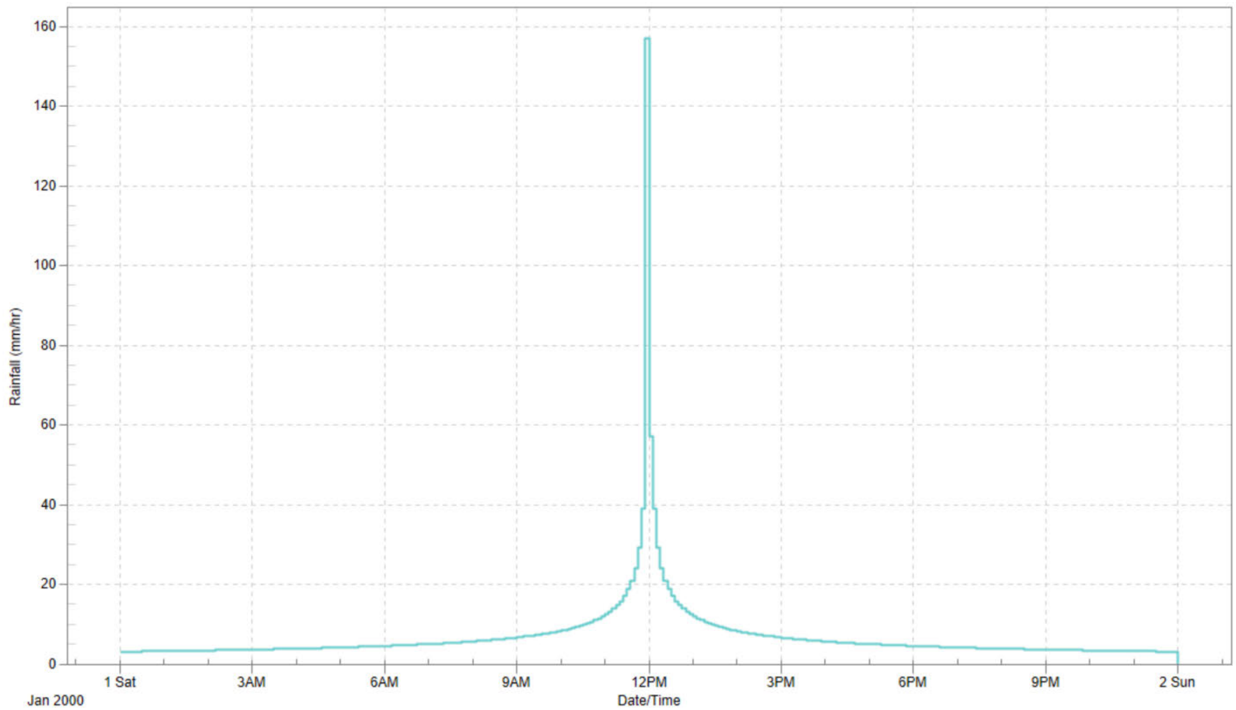

Figure 3-3: An Example 24-h Rainfall Distribution for the Shearwater RCS Station under a 1 in 100 Year Rainfall Event (Existing Conditions).

3.2 Climate Change Rainfall Projections

Climate change is expected to cause an increase in extreme precipitation, primarily due to the ability of warmer air to hold more moisture. Therefore, it is not appropriate to use IDF data based on historical information alone for long-term planning, and estimates of future changes in extreme precipitation must be obtained.

To project future changes in extreme precipitation, climate models and emission scenarios are used. A climate model is a computer representation of atmospheric, oceanic, and other processes. Climate models use greenhouse gas emission scenarios as inputs to project climate into the future. The Intergovernmental Panel on Climate Change (IPCC) has established future emission scenarios, including Representative Concentration Pathways (RCPs) and Shared Socioeconomic Pathways (SSPs).

Climate models use approximations in their mathematical formulations. While all climate models are based on well-established processes, each model uses different approaches (resolutions, assumptions, etc.) which provide different results. This means that a given model can overestimate or underestimate the actual climate impacts. Therefore, climate projections are best obtained from an ensemble of models that cover a range of possibilities (to the extent that they can be modelled). Projections are thus reported as a percentile of the model ensemble (e.g., 10th, 50th, 90th).

3.2.1 Approach and Methodology

Projecting changes in precipitation extremes is challenging, in part because some precipitation processes, such as thunderstorms, happen on spatial scales that are smaller than the resolution of global climate models. Therefore, the recommended method for projecting precipitation extremes is the use of the Clausius-Clapeyron Equation, which is based on projected changes in temperature rather than precipitation.

With this “temperature scaling” approach, each degree of warming is taken to result in an approximately 7% increase in precipitation intensity (Westra et al. 2014). This method is considered scientifically defensible by authoritative sources such as CSA PLUS 4013:19 and Cannon et al. (2020), the ECCC report that will inform climate change updates to the building and bridge codes.

Although the general relationship between warming and extreme precipitation is robust, it is noted that the scaling rate may vary significantly around the approximate value of 7% (Cannon et al. 2020). There is also some evidence to suggest a doubling of the Clausius-Clapeyron scaling rate for shorter (i.e., sub-daily) precipitation event durations; however, there is no clear guidance to apply this at present (CSA 2019) and thus this has not been applied for the main climate scenario (RCP 8.5 50th Percentile) selected for this project. Some discussion is included in the Climate Scenario Sensitivity Analysis Add-On (Appendix D, Section D.2.4).

The application of the Clausius-Clapeyron method is described by CSA (2019). In general, temperature projections were obtained and then converted to a precipitation projection using the simplified Clausius-Clapeyron equation. Key decision points for the methodology include:

- **Climate Scenario:** The industry-standard scenario of RCP 8.5 was selected by HRM (the client) as the climate scenario for this project. Given the use of RCP 8.5, CBCL recommended the use of the 50th percentile of the modelling ensemble, which was confirmed by the client following the receipt of the Nov 01, 2022, Progress Memo

"Recommendation on Climate Scenario for Flood Mapping". As stated in this memo, the primary factor considered for this recommendation is that, when using RCP 8.5, the 50th percentile is the "industry standard" scenario used in the vast majority of climate and coastal assessments across Canada. The RCP 8.5 90th percentile is a less common choice when used on its own. Hence, the "RCP 8.5 50th Percentile" is the climate scenario that has been adopted in this project and is the scenario that has been used for temperature projections here.

- **Generation of Climate Model Ensemble:** Since the RCP 8.5 was selected by the client, Fifth Coupled Model Intercomparison Project (CMIP5) models downscaled with a method called BCCAQv2 that are available on ClimateData.ca were used for temperature projections (i.e., rather than the more recent CMIP6 models, which are still being investigated).
- **Projection Location:** The analysis was conducted on temperature projections both (1) area-averaged across HRM, and (2) for the grid cell showing the greatest future increase in annual average temperature. The difference was found to be negligible for the RCP 8.5 50th percentile. Therefore, area-averaged (across HRM) annual average temperature projections were used.
- **Return Periods.** In accordance with the guidance from CSA (2019), the percentage increases were consistently applied across all return periods. This approach was chosen due to the existing level of uncertainty in the scientific understanding of how projections fluctuate among different return periods. While it could be valuable for future research to compare the projections from this assessment with methodologies for evaluating future probable maximum precipitation, such a comparison was beyond the scope of the current project.
- **Baseline:** The temperature projections were compared to a baseline of 1981-2010 (the standard baseline for CMIP5 projections).
- **Projection Horizons:** Projection horizons are defined based on the planning needs of the project. In this case, the client requested 2050 and 2100. The best practice for projection horizons is to use 30-year time periods, to account for natural variability in the earth's system. Thus, the year ranges that were used were 2036-2065 (to represent 2050) and 2071-2100 (to represent 2100). Note that CMIP5 climate model outputs are not readily available past the year 2100. Thus, the 2100 projection is actually centered around the 2080s and may be an underestimate of projections for 2100.

3.2.2 Projected Changes in Precipitation Intensity

The projected change in precipitation intensity is approximately 20% and 40% for the 30-year periods representing 2050 and 2100, respectively (Table 3-2). This is consistent with expected values for changes in extreme precipitation in Atlantic Canada. This percentage increase can be applied to the historical hyetograph used for hydrological/hydraulic modelling to approximate the effects of climate change on future flows (see example shown in Figure 3-2). It is important to consider the uncertainty on this estimate when interpreting the resulting flood lines.

Table 3-2: Projected Changes in Extreme Precipitation Intensity for HRM Using RCP8.5

	2050	2100
Lower 10th Percentile	11%	27%
Median 50th Percentile	19%	39%
Upper 90th Percentile	30%	55%

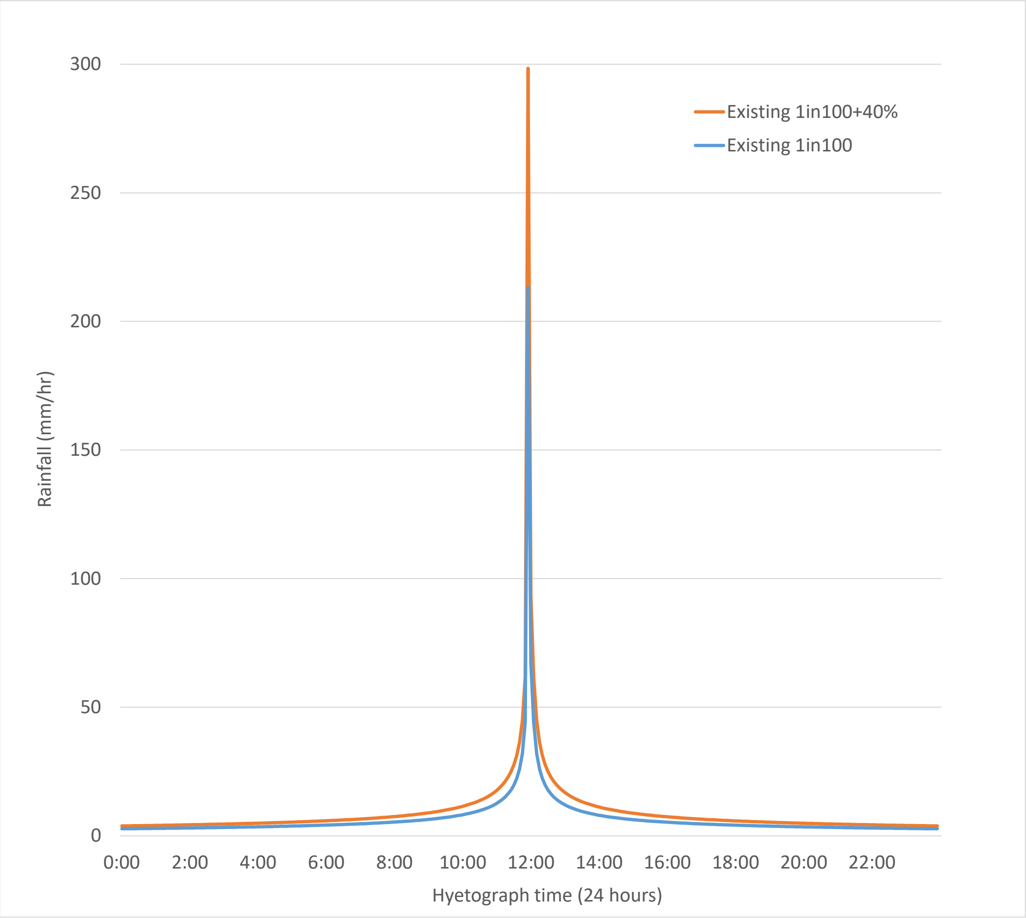

Figure 3-2 : Example Climate Change Percent Change Applied to Historical Hyetograph (1 in 100 year storm, model 1EH_1).

4 Pluvial-Fluvial Analysis and Modelling

The inland portion of the study for pluvial – fluvial analysis has been divided into study regions based on the provincial Primary Watershed designations and are listed below and shown in Figure 4-1:

- 1EH (East/Indian River).
- 1EJ (Sackville River, includes most of the metropolitan area).
- 1DG (Shubenacadie/Stewiacke River).
- 1EK (Musquodoboit River).
- 1EL (Tangier River).
- 1EM (East/West River, Sheet Harbour).
- 1EN (Liscomb River).
- 1DE (St. Croix River).
- 1EO (St. Mary's River).

The nine study regions were further split into smaller sub-watersheds to facilitate modelling, resulting in a total of 27 separate sub-watershed models, as presented in Figure 4-2. These sub-watersheds are primarily defined from the provincial Secondary Watershed designation.

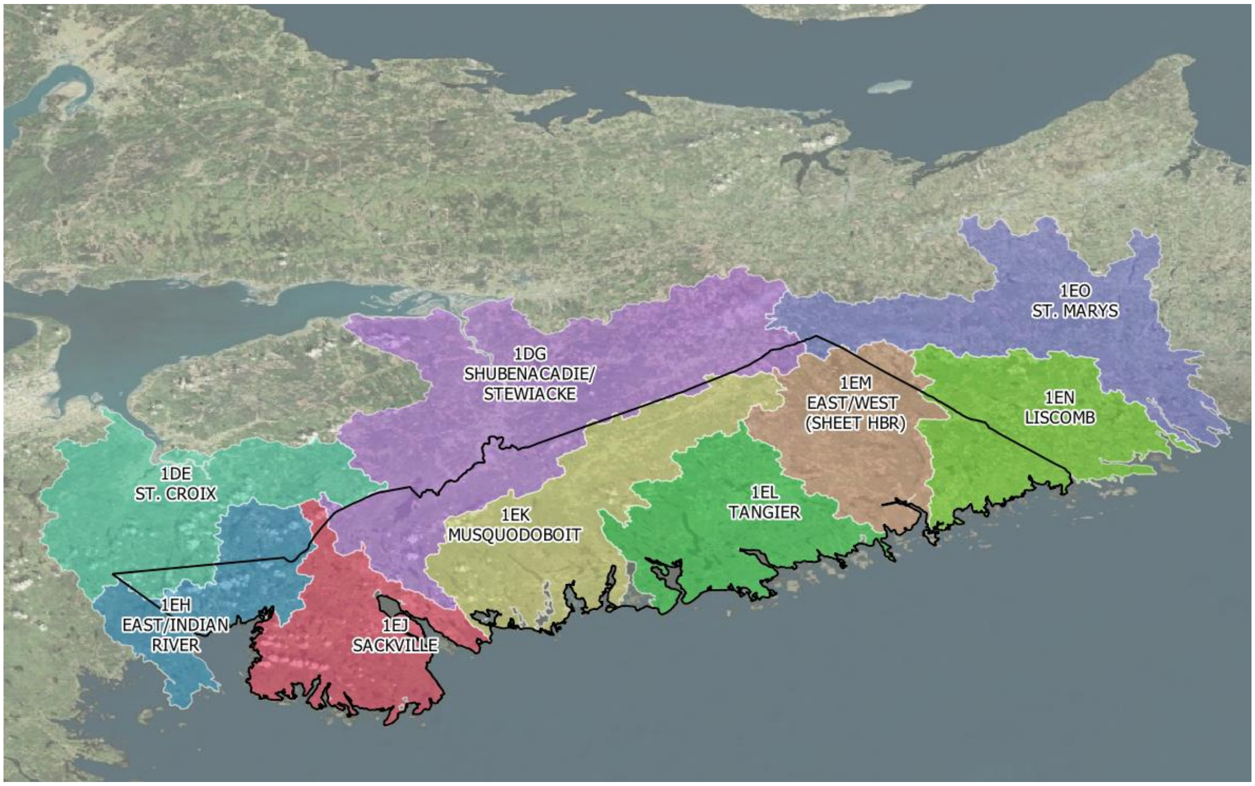

Figure 4-1: Inland (pluvial, fluvial) study regions.

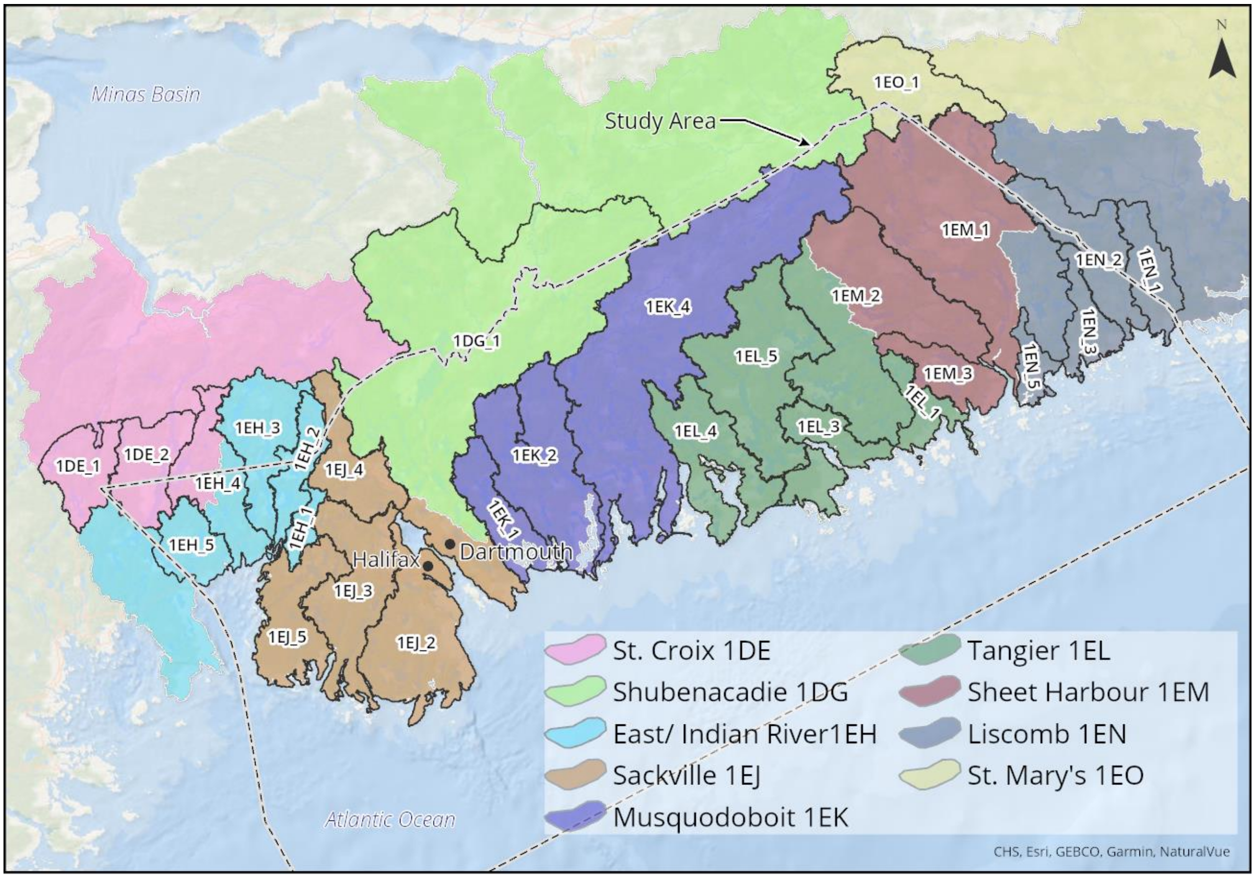

Figure 4-2: Sub-watersheds for the inland (pluvial, fluvial) study regions.

4.1 Assessment Methodology

The following process was followed in this assessment to analyse existing data, prepare and run hydrologic and hydraulic models, and ultimately produce flood maps for the study area. In general terms, the hydrologic modelling simulates the water cycle as it receives rainfall as input, estimates water infiltration, sheet flow, and total runoff into the watercourses. The hydraulic model, on the other hand, receives the total runoff as input, and calculates the accumulation of flows and water volumes in the watercourses, as well as the water levels throughout the system. More detail on the software and its capabilities is presented further down.

1. Collection and Analysis of the following data:
    a. Lidar Data.
    b. Existing information on bridge and culvert structures.
    c. Land Cover data.
    d. Soil mapping data.
    e. Extreme rainfall event data from climate stations within the study area.
    f. Intensity-Duration-Frequency curve data.
    g. Flow data from flow gauging stations within the study area.
2. Flow Monitoring at key locations within the study area.
3. Modelling of each watershed and watercourse in the study area, using the following steps:
    a. Delineation of watershed and extraction of hydrologic parameters (topography, shape, surface roughness, land cover, soil infiltration).
    b. Running the hydrologic models to produce peak runoff rates for each sub-watershed and each main watershed.
    c. Preparing the hydraulic models by entering the geometric data for all the channels and main structures (bridges, culverts, dams) in the study area.
    d. Assigning representative coefficients for channel roughness, channel vs floodplain boundaries, and hydraulic loss coefficients for structures.
    e. Setting up downstream water level boundaries to represent the tidal water level variations.
4. Calibration of the model and running the various requested rainfall events and climate scenarios:
    a. Preparing representative datasets of measured rainfall and flow information.
    b. Running the models with rainfall as input and adjusting hydrologic parameters to produce representative model results.
    c. Comparing model results with measured flows and water levels in the study area.
5. Preparation of flood maps for each area, for each rainfall event and for each climate scenario. These include extreme coastal water level flood maps for nine return periods and three sea level rise scenarios:
    a. Return periods: 2-, 5-, 10-, 20-, 50-, 100-, 200-, 500-, and 1000-year (the latter two for the 2100 time horizon only).
    b. Existing Climate.
    c. Future Climate (including sea level rise) for the 2050 time horizon.
    d. Future Climate (including sea level rise) for the 2100 time horizon.

4.2 Modelling Software

An integrated hydrological-hydraulic model was set up for each of the 27 sub-watersheds using the PCSWMM software produced by Computational Hydraulics International (CHI). PCSWMM is a platform based on top of the US Environmental Protection Agency (EPA) Storm Water Management Model (SWMM) version 5. US EPA SWMM is a hydrologic and hydraulic model with origins dating to 1971, with regular updates. It is an industry standard modelling system used to study urban and rural watersheds and it can perform unsteady flow calculations to simulate channel hydraulics by dynamically solving the continuity and momentum equations with a finite difference scheme.

The following simulation methods were used:

- Rainfall – runoff loss method – Green-Ampt Soil Infiltration Equation.
- Runoff routing method – Non-linear Reservoir Routing Equation.
- Channel routing – 1D Dynamic Wave with Full Momentum Equations.

4.3 Model Parameterization

Automatic watershed delineation and channel terrain extraction tools in PCSWMM were employed. The created model subcatchments and channels were reviewed by the CBCL team to ensure they represented the topography and natural drainage features. The approach used to develop and parameterize the models' key components is described below.

4.3.1 Watershed Delineation and Hydrologic Parameters

The Project Area was first divided into nine sections based on the 1:10,000 Nova Scotia Primary Watersheds layer. HRM provided CBCL with a 1m resolution LiDAR DEM (2018) as the basis for this project, CBCL additionally used the 1m resolution LiDAR DEM (2019) available from GeoNOVA to extend the DEM for the primary watersheds that straddled the HRM boundary and were not fully covered within the HRM DEM. CBCL first clipped both DEM sources to the 9 primary watersheds and then adjusted their projections in order to merge them into a single DEM to be used in the model. Both the DEM from GeoNOVA in the NAD_1983_CSRS_UTM_Zone_20N projection, and the HRM DEM in WGS84 were reprojected to NAD_1983_CSRS_2010_MTM_5_Nova_Scotia as per the RFP requirement.

Watershed delineation of each of the sub-watersheds associated with the 27 main watersheds was therefore performed in PCSWMM using the 2018 HRM LiDAR, supplemented (beyond the HRM border) by the 2019 provincial 1-meter LiDAR Digital Elevation Model (DEM) available through GeoNOVA. Physical subcatchment parameters, including slope and flow length, were also derived in PCSWMM from the DEM.

Subcatchments were reviewed manually and further refined where necessary to best represent flow paths. The target subcatchment size for watershed delineation across the entire study area was 100 hectares. Where specific areas of flood vulnerability were included in the models, the watershed delineation was reduced and refined to include some additional detail in the models. Subcatchment slopes were derived from the LiDAR DEM. The average subcatchment slopes for each of 27 main watersheds range from 0.7% to 6.5%, with minimum slope of 0.02% and maximum slope at of 15.1%.

Land cover areas for existing development conditions were delineated within the study area based on the Natural Resource Canada National Land Cover Mapping. The watersheds are predominantly rural and forested with some cleared land for agriculture, primarily along the river valleys of the Musquodoboit River. Subcatchment roughness was assigned based on McCuen et al. (1996), following the standard practice for the SWMM software, as presented in Table 4-1.

Table 4-1: Subcatchment Roughness Based on Different Land Cover Types

Land Cover Types	Manning's Roughness Coefficient n
Temperate or Sub-polar Needleleaf Forest	0.8
Temperate or Sub-polar Broadleaf Deciduous Forest	0.4
Mixed Forest	0.4
Temperate or Sub-polar Shrubland	0.4
Temperate or Sub-polar Grassland	0.24
Wetland	0.24
Cropland	0.24
Barren Land	0.05
Urban (Roads / Built Up Area)	0.013
Water	0.011

Soil survey data obtained from Agriculture and Agri-Food Canada (2013) was used for determining soil infiltration (hydraulic conductivity) of the subcatchments. Hydraulic conductivity and Saturation Head values from Rawls et al (1993) were assigned to each soil drainage class: Rock, Sandy Loam, Loam, Sandy Clay Loam, and Clay Loam, as presented in Table 4-2 on the following page. Hydraulic conductivity values estimated for each soil group were assigned to subcatchments as a weighted average.

Table 4-2: Infiltration Parameters based on Different Soil Types

Soil Types	Conductivity (mm/h)	Suction Head (mm)
Clay	0.254	320.04
Clay Loam	1.016	210.058
Loam	3.302	88.9
Loamy Sand	29.972	60.96
Mine	0.1	400
Rock	1	210
Sand	120.396	49.022
Sandy Clay Loam	1.524	219.964
Sandy Loam	10.922	109.982
Silt Loam	6.604	169.926
Silty Clay Loam	1.016	270.002

Reservoirs, lakes, and wetlands were considered to be impervious, having no soil infiltration losses during the simulated storm events.

4.3.2 Modelling of Watercourse Channels

Once the hydrologic part of the model is assembled, and the model is able to produce estimates of soil water infiltration and accumulation of surface runoff, the next step in the modelling is to create the geometric descriptions of the channels and hydraulic structures influencing the flows and water levels. This is called the hydraulic part of the modelling. The modelling is focused on representing channels and floodplains as linear elements (1-dimensional), and the model is able to represent rising and falling flows, hence, the approach to modelling is called 1D dynamic modelling.

Hydraulic routing used 1D dynamic, full momentum equations in PCSWMM. The study area includes a complex dendritic network of watercourses and lakes, as displayed in an example presented in Figure 4-3. Representing 1D flow paths with transects crossing the channel and floodplain requires careful layout and review to ensure they represent the large range of watercourse size and the varying configuration of confluences, lakes, and urban channel modifications. Placement and selection of transect lines were carefully considered to ensure that hydraulics were properly modelled and that flood mapping results would be correctly portrayed within the confines of the transect lines.

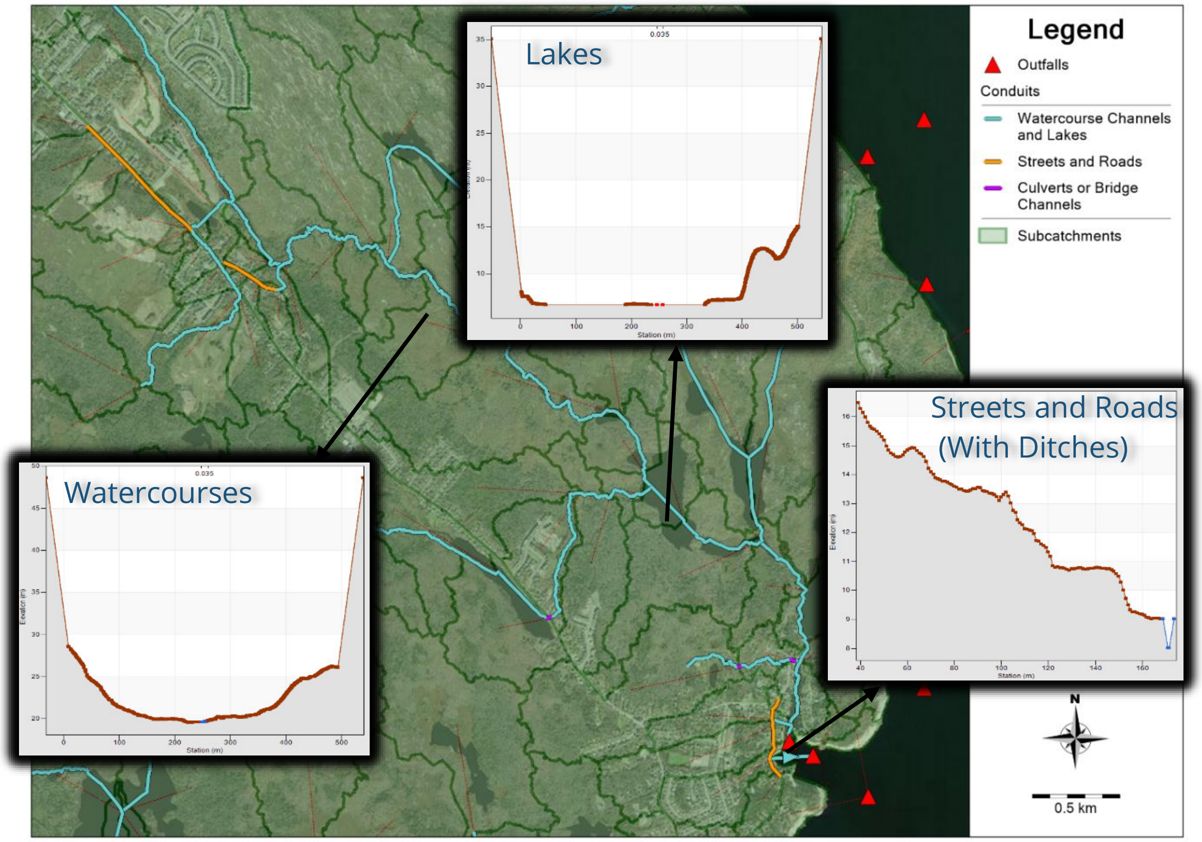

Figure 4-3: Example map Showing the Configuration of Watercourses, Lakes, and Urban Channel Modifications.

Due to the extensive study area, automated transect delineation tools were needed. Extensive testing was carried out using both the PCSWMM Transect Creator Tool as well as ArcGIS tools to identify optimal methods for generating transects. The primary goal was to identify the best method to produce the most representative configuration of transects. The secondary goal was to test a range of transect widths (across the channel and floodplain) and transect spacing (distance between transect lines along the river) to identify the optimal configuration in terms of computational time and quality of results. Both the width of transects and spacing must vary to account for the size and topography of the watercourses. Small watercourses with narrower valleys and tighter meander patterns require a shorter transect width and a closer spacing. It is especially important for small watercourses that the transect width be contained to the primary watercourse valley and not cross over into another low-lying area. When this happens, the transect can over represent the capacity of the watercourse, leading to incorrect water levels. To overcome this, CBCL manually adjusted the transects where this was needed.

The most preferable method for generating representative cross-sections was found to be the PCSWMM Transect Creator Tool. This tool was used to automatically delineate transects along watercourses and waterbodies at a 10 m spacing (parallel to flow) and a width of 500 m (perpendicular to flow). However, automated tools cannot fully represent channels and floodplains in all cases and require a careful review. With approximately 100,000 transects created across the study area, careful review of the automated transect lines was essential. Manual edits were most often required to correct transects at lakes, the confluence of watercourses, and in the estuarine areas.

4.3.3 Hydraulic Parameters

The key parameters used in hydraulic modelling of watercourses are focused on quantifying energy losses from the system that ultimately impact water levels. These include:

- Channel, floodplain, and pipe frictional losses represented by Manning's roughness values.
- Flow contraction and expansion turbulence losses represented by head loss coefficients.

Manning's roughness values for the channel and floodplain were interpreted from satellite imagery and site photographs. Roughness values were taken from the industry standard guide originally written by Chow (1959), as presented in Table 4-3. Channel roughness values generally ranged from 0.035 and 0.045 with exceptions in some areas where the channel was visibly smoother or rockier. Floodplain roughness was assigned an average value based on the mix of land cover types in the cross sections.

Table 4-3: Roughness for Open Channels, Floodplains, and Closed Conduits

Type	Descriptions	Manning's Roughness Coefficient n*
Open Channels	Clean, straight, full stage, no rifts or deep pools	0.025 – 0.033
	Clean, winding, some pools and shoals	0.033 – 0.045
	Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stages with bottom: gravels, cobbles, and few boulders	0.03 – 0.05
	Sluggish reaches, weedy, deep pool	0.05 – 0.08
Floodplains	Temperate or sub-polar needleleaf forest	0.16
	Temperate or sub-polar broadleaf deciduous forest	0.1
	Mixed forest	0.12
	Temperate or sub-polar shrubland	0.06
	Temperate or sub-polar grassland	0.035
	Wetland	0.05
	Cropland	0.04
	Barren land	0.03
	Urban (Roads / Built Up Area)	0.016
	Water	0.011
Closed Conduits	Culvert pipes and boxes	0.02

* Based on Chow, 1959.

4.3.4 Hydraulic Structures

The study area features an extensive number of hydraulic structures including dams, weirs, bridges, and most notably culverts. The maps in Appendix B include a map of culvert and bridge structures including the amount of available information on each. Hydraulic structures, such as bridges and culverts, can have a strong influence on upstream flood water levels and downstream flows, and were be included where possible. The best option is to use surveyed or measured structure details. However, field survey was not included in the scope of this project, and no structure was surveyed. Where available, HRM or provincial information on culvert or bridge structures was used, and where available, surveyed data from the Sackville Floodplain Study and the Shubenacadie Lakes Floodplain Study was incorporated, and where not available, estimates based on a set process (see next section) were made. As described in the following section, best efforts were made to produce estimates that were generally representative of the capacity of the various drainage structures in the study area. This is estimated to produce a general representation of the impact of the drainage structures on upstream flooding and on downstream flows. The results are likely to be quite representative for larger bridge structures, where the Lidar geometry represents the hydraulic opening generally well. For smaller structures, there is a higher amount of uncertainty, but this is associated with smaller flows, and associated smaller impacts. It is noted that smaller structures, even when sizes are known, may not be represented well in hydraulic models in general, since they are subject to sedimentation, blockage by vegetation or debris, settling, scour, bending and vandalism, which may severely affect their capacity.

4.3.4.1 Watercourse Crossings

Bridge structure decks have typically been removed from the DEM surface, meaning that an approximate structure opening area can be estimated from the DEM. For culverts, however, the road embankment is still typically present in the DEM surface, meaning that their dimensions cannot be easily extracted from the DEM. As such the following workflow was used to estimate and assume structure details:

Identification of Structures

Structure location data was compiled from the following sources:

- HRM watercourse crossing structure layer.
- NSPW structure layer.
- Dams and weirs identified through satellite imagery, local area knowledge, and in the hydrographic network shapefile from the province.

Evaluation of Necessity to Include Structure

- Structures are not deemed necessary to include:
  - If the roadway low elevation (sag) is less than 1.0 m higher than the watercourse invert (or DEM invert at the crossing), then the resulting additional risk of flooding

  upstream related to the presence of the structure is deemed minimal during an extreme event due to the release of flows through road overtopping.
  - If the roadway low elevation (sag) equals to or is larger than 1.0 m but less than 4.0 m higher than the watercourse invert (or DEM invert at the crossing) and there is no development upstream.
- Structures are deemed necessary to include:
  - If the roadway low elevation (sag) is equal to or is larger than 1.0 m above the watercourse invert (or DEM invert at the crossing) and there is development upstream.
  - If the roadway low elevation (sag) is equal to or is larger than 4.0 m above the watercourse invert (or DEM invert at the crossing).

Estimation of Structure Details

- To estimate a culvert diameter or width, the visible watercourse wetted width was estimated from satellite imagery.
  - For watercourse widths smaller than 6.0 m, the closest culvert size from the following list was selected:
    - 600 mm (round pipe).
    - 1050 mm (round pipe).
    - 3000 mm (rectangular box with height measured as height between watercourse invert and the top of the road minus overburden height).
  - For watercourse widths equal or larger than 6.0 m:
    - Structure width taken as distance from mid-bank on either side of watercourse and structure height to be taken as the height inferred from watercourse invert to the road surface minus fixed overburden height. For long culvert crossings beneath multiple roads, the lowest overflow elevation over the road is used.
- To estimate a bridge structure opening:
  - A default rectangular opening is to be assumed with the structure width measured as the distance between the mid-banks on either side of watercourse.
  - For structure height:
    - Where the bridge width is less than 6.0 m, the height is to be measured between the watercourse invert and the top of the road (minimum elevation measured within 50 m of the crossing) minus fixed overburden height.
    - Where the bridge width is larger than 6.0 m, the height is to be measured between the watercourse invert and the top of the road (minimum elevation measured within 50 m of the crossing) minus fixed overburden height.

4.3.4.2 Dams and Reservoirs

The study area contains dams, control structures, and reservoirs operated for drinking water supply, hydroelectric power generation, and lake level management, as well as natural unregulated lakes, as displayed in an example presented in Figure 4-4. The natural topography of the reservoirs and lakes combined with either controlled outlets or natural outlets provide a hydraulic routing storage effect that can attenuate peak flows.

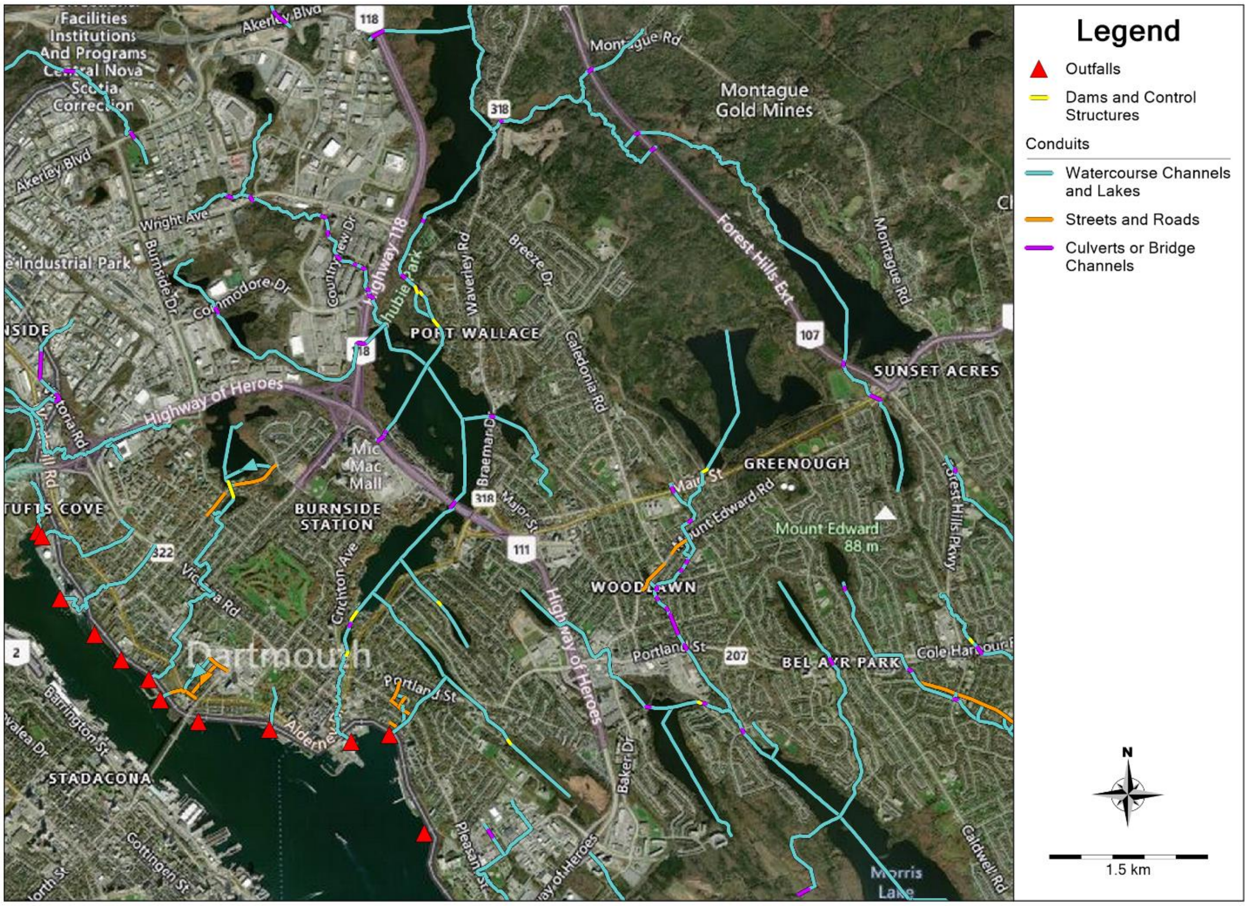

Figure 4-4: An Example Showing the Configuration of Watercourses, Lakes, and Urban Channel Modifications.

Lake and reservoir routing and storage was accounted in the model via 1D hydraulic conveyance represented by transects at equal intervals along the waterbody. Bathymetric data was not available for lakes and reservoirs, so the DEM surface elevation was assumed as the antecedent (i.e. starting) water levels in the waterbody at the time of simulation.

Dams and control structures within the model domain were included with simplified outlet and spillway conditions. No information was available regarding operational release rates or outlet pipes. Outflow through dams and control structures was simulated via a sharp crested weir structure with a width measured as the visible spillway crest width in the satellite imagery. The weir crest elevation was taken as the upstream water level surface in the DEM.

4.3.5 Previously Identified Urban Flood Risk Areas

In 2015, HRM commissioned Servant, Dunbrack, McKenzie, and MacDonald Ltd. (SDMM) to review flood risk locations along stormwater management infrastructure within Halifax and

Dartmouth. The *Halifax Stormwater Event Inventory Mapping* produced by SDMM identified locations where stormwater surcharge or flooding could potentially pose a risk to people, property, or traffic flow. Traffic hazards were the most commonly flagged risk by SDMM and these sites cover both urban and suburban areas.

CBCL had proposed to increase the level of detail of the modelling where needed to support the study objectives, while keeping a lower general level of detail outside of vulnerable areas, to stay within the project timeline. During initial model testing, it was identified that a very coarse level of detail in some areas would produce unrealistic results in geographic areas with varying slopes. It was decided that an overall high level of detail, with further refinement in vulnerable urban areas, would produce more representative results in all areas.

In order to identify areas for further refinement, CBCL reviewed the mapping by SDMM and selected a subset of 30 areas that had a high density of flagged flood risks. In these areas additional detail was added for surface drainage from the DEM and satellite imagery. Model subcatchments, conduits, and nodes were refined to be able to depict flooding at the same scale as identified in the *Halifax Stormwater Event Inventory Mapping*. Streets and roads within these areas were represented by standard assumed street, gutter, and curb dimension data based on HRM Engineering Guidelines. Streets in these areas without curb and gutter (e.g. with ditches) were represented by manually drawn transect lines that sampled the DEM.

Modelling details are limited to a desktop level spatial data and do not include field surveyed terrain or infrastructure. In addition, no subsurface stormwater system (pipe) information was available at the time of study. Therefore, as discussed with HRM, the subsurface stormwater system drainage capacity has been assumed to remove the amount of surface runoff equivalent to a 2-year rainfall event. This amount of runoff has been subtracted from the surface flow in these portions of the model.

4.3.6 Initial Conditions

The SWMM models were simulated with a standard design storm event, allowing rainfall to build-up and lead to increasing runoff rates through the peak of the storm. This is a standard approach that is appropriate for the size of watersheds within the study area.

The study area has an extensive number of lakes, which hold significant storage capacity. However, since lakes and reservoirs are naturally maintained by outlet conditions, they can be represented as having a stable, base level of water equivalent to their outlet elevation. Typically this base level is represented by the LiDAR surface used for modelling. An additional challenge was encountered with the provided LiDAR, whereby some lakes and waterbodies had “holes” in the DEM with no elevation data. This meant that post-processing was required to interpolate elevations within the lakes to provide topographical support for the model tools (watershed delineation, delineating watercourses, setting watercourse elevations) to run. While the DEM had suitable elevations within certain lake areas, there were inconsistencies in many areas that created issues for the model tools. CBCL initially used an automated tool to fill the “holes” in the DEM, but this was not always successful and provided unexpected results, and we found it necessary to manually check the work done by the tool. Since the data had to be manually corrected in many cases to allow the processing tools to run, the assumed base water level in the lake may not necessarily be representative of the natural or reservoir operating levels. This adds an element of uncertainty to the projected flood levels in lakes where the data suffers from this defect. CBCL has used best practices to account and adjust for this problem. Additional information would be needed to generate floodline delineations for uses and decisions where lower uncertainty levels are required.

4.3.7 Boundary Conditions

The model did not require any internal inflows or inter-watershed boundary conditions (i.e. direct flow from adjacent watercourses).

In order to avoid underestimating the tidal levels in the river estuaries, the average maximum annual level is selected as the peak coastal level (as a varying tidal level, not as a fixed value) and applied as a coastal boundary condition. This corresponds to the 1 in 2 year extreme coastal water level, and was applied to all pluvial-fluvial flood event simulations. This is an industry standard method the modelling the combined coastal-riverine interactions within estuaries that takes into account a reasonably conservative scenario without significantly compounding the return periods modelled.

Bridge structures across coastal inlet can have a significant effect on upstream water levels. Although coastal inlets are generally included within the coastal model, the bridge structures have not been included within that model. As such, the SWMM model has been extended downstream in these cases to include bridge. The SWMM model can evaluates the energy losses through the structure and the volume of water accumulated upstream, to calculate the water level upstream of the bridge, and its subsequent impact on the upstream water levels in the river system. The downstream boundary condition is taken from the coastal modelling downstream (seaward side) of the bridge.

4.4 Calibration – Validation

Model calibration and validation is the process of adjusting hydrologic and hydraulic model parameters within their normal ranges to evaluate the model’s ability to reproduce recorded flows from a past storm event. Antecedent moisture conditions for the calibration events were accounted for by adjusting the depression storage and initial soil moisture deficit. In addition, running a prior rainfall event was used to set up initial water levels and flows throughout the system, to match the initial measured flows prior to the measured extreme flow event. Once set up, to maintain consistent results, the same initial conditions were used by models in each main watershed for the various design rainfall events to be flood mapped.

Some water level gauging was conducted as part of this study, which is presented in Section 2.2. Although water level monitoring data can be very helpful in supporting model calibration, there were no suitable events that reached close to a 1 in 2 year event during the monitoring period. The maximum increase in water levels for any of the 3 gauges reached 0.6 m to 0.8 m only (not of flooding significance), and therefore, the ECCC gauged flow data was the sole source of flow gauging data used for model calibration.

A review of the various flow gauging stations operated by the Water Survey of Canada, part of ECCC, was conducted to identify potential recorded flow events in the study area that were generally in the 50-year and 100-year range of return periods, and devoid of snowmelt, to ensure the runoff and flow mechanisms were similar to the design events modelled. Through this process, the following stations and events were identified, as shown on Table 4-4:

Table 4-4: Peak Flows of Historical Flood Events Recorded in the Study Area

Station Name	Date of Peak	Peak Flow (m^3/s)
Shubenacadie River at Enfield-01DG006	11/20/1990 12:00:00 PM	83
East River at St Margaret's Bay-01EH003	11/12/1991 12:00:00 PM	9.9
Liscomb River-01EN002	08/16/1971 01:41:00 PM	317
Sackville River at Bedford-01EJ001	12/11/2014 12:00:00 PM	85
Musquodoboit River-01EK001	08/16/1971 01:41:00 PM	360

Other stations had data but were either in areas that did not have sufficiently representative hydrologic characteristics (e.g. Saint Mary's River and Beaver Bank River have data, but are in different main watersheds), or did not have available data at the times of the largest floods (e.g. Little Sackville River).

To allow the models to estimate flows for the events identified above, rainfall data was needed for the same time period, as close as possible to the target watershed, in a recording interval that is as short as possible.

Table 4-5 on the following page shows which rain gauge was assigned to each watershed area, and to which flow gauging station with the locations presented in Figure 4-7.

Table 4-5: Summary of Rain Gauge, Watershed, and Flow Gauging Station Assignments

Model Name	Station ID	Station Name	Date of Peak	Peak Flow (m^3/s)	Rain Gauge	Drainage Area Flow Gauge (km^2)	Distance between the Rainfall Gauge and Flow Station (km)
1DE_1	01DG006	SHUBENACADIE RIVER AT ENFIELD-01DG006	11/20/1990 12:00:00 PM	83	Shearwater	389	33.6
1DE_2	01DG006	SHUBENACADIE RIVER AT ENFIELD-01DG006	11/20/1990 12:00:00 PM	83	Shearwater	389	33.6
1EH_1	01EH003	East River at St Margaret's Bay-01EH003	11/12/1991 12:00:00 PM	9.9	Bedford Range	26.9	18
1EH_2	01EH003	East River at St Margaret's Bay-01EH003	11/12/1991 12:00:00 PM	9.9	Bedford Range	26.9	18
1EH_3	01EH003	East River at St Margaret's Bay-01EH003	11/12/1991 12:00:00 PM	9.9	Bedford Range	26.9	18
1EH_4	01EH003	East River at St Margaret's Bay-01EH003	11/12/1991 12:00:00 PM	9.9	Bedford Range	26.9	18
1EH_5	01EH003	East River at St Margaret's Bay-01EH003	11/12/1991 12:00:00 PM	9.9	Bedford Range	26.9	18
1EN_1	01EN002	Liscomb River-01EN002	08/16/1971 01:41:00 PM	317	ECUM SECUM	389	7.7
1EN_2	01EN002	Liscomb River-01EN002	08/16/1971 01:41:00 PM	317	ECUM SECUM	389	7.7
1EN_3	01EN002	Liscomb River-01EN002	08/16/1971 01:41:00 PM	317	ECUM SECUM	389	7.7
1EN_4	01EN002	Liscomb River-01EN002	08/16/1971 01:41:00 PM	317	ECUM SECUM	389	7.7
1EN_5	01EN002	Liscomb River-01EN002	08/16/1971 01:41:00 PM	317	ECUM SECUM	389	7.7
1EJ_1	01EJ001	Sackville River at Bedford-01EJ001	12/11/2014 12:00:00 PM	85	Bedford Range	146	1.6
1EJ_2	01EJ001	Sackville River at Bedford-01EJ001	12/11/2014 12:00:00 PM	85	Bedford Range	146	1.6
1EJ_3	01EJ001	Sackville River at Bedford-01EJ001	12/11/2014 12:00:00 PM	85	Bedford Range	146	1.6
1EJ_4	01EJ001	Sackville River at Bedford-01EJ001	12/11/2014 12:00:00 PM	85	Bedford Range	146	1.6

Model Name	Station ID	Station Name	Date of Peak	Peak Flow (m^3/s)	Rain Gauge	Drainage Area Flow Gauge (km^2)	Distance between the Rainfall Gauge and Flow Station (km)
1EJ_5	01EJ001	Sackville River at Bedford-01EJ001	12/11/2014 12:00:00 PM	85	Bedford Range	146	1.6
1EL_1	01EN002	Liscomb River-01EN002	08/16/1971 01:41:00 PM	317	ECUM SECUM	389	7.7
1EL_2	01EN002	Liscomb River-01EN002	08/16/1971 01:41:00 PM	317	ECUM SECUM	389	7.7
1EL_3	01EN002	Liscomb River-01EN002	08/16/1971 01:41:00 PM	317	ECUM SECUM	389	7.7
1EL_4	01EN002	Liscomb River-01EN002	08/16/1971 01:41:00 PM	317	ECUM SECUM	389	7.7
1EL_5	01EN002	Liscomb River-01EN002	08/16/1971 01:41:00 PM	317	ECUM SECUM	389	7.7
1EK_1	01EK001	Musquodoboit River-01EK001	08/16/1971 01:41:00 PM	360	Shearwater	650	35.4
1EK_2	01EK001	Musquodoboit River-01EK001	08/16/1971 01:41:00 PM	360	Shearwater	650	35.4
1EK_4	01EK001	Musquodoboit River-01EK001	08/16/1971 01:41:00 PM	360	Shearwater	650	35.4
1DG_1	01EK001	SHUBENACADIE RIVER AT ENFIELD-01DG006	11/20/1990 12:00:00 PM	83	Shearwater	650	33.6
1EO_1	01EK001	Liscomb River-01EN002	08/16/1971 01:41:00 PM	317	ECUM SECUM	650	7.7
1EM_1	01EN002	Liscomb River-01EN002	08/16/1971 01:41:00 PM	317	ECUM SECUM	389	7.7
1EM_2	01EN002	Liscomb River-01EN002	08/16/1971 01:41:00 PM	317	ECUM SECUM	389	7.7
1EM_3	01EN002	Liscomb River-01EN002	08/16/1971 01:41:00 PM	317	ECUM SECUM	389	7.7

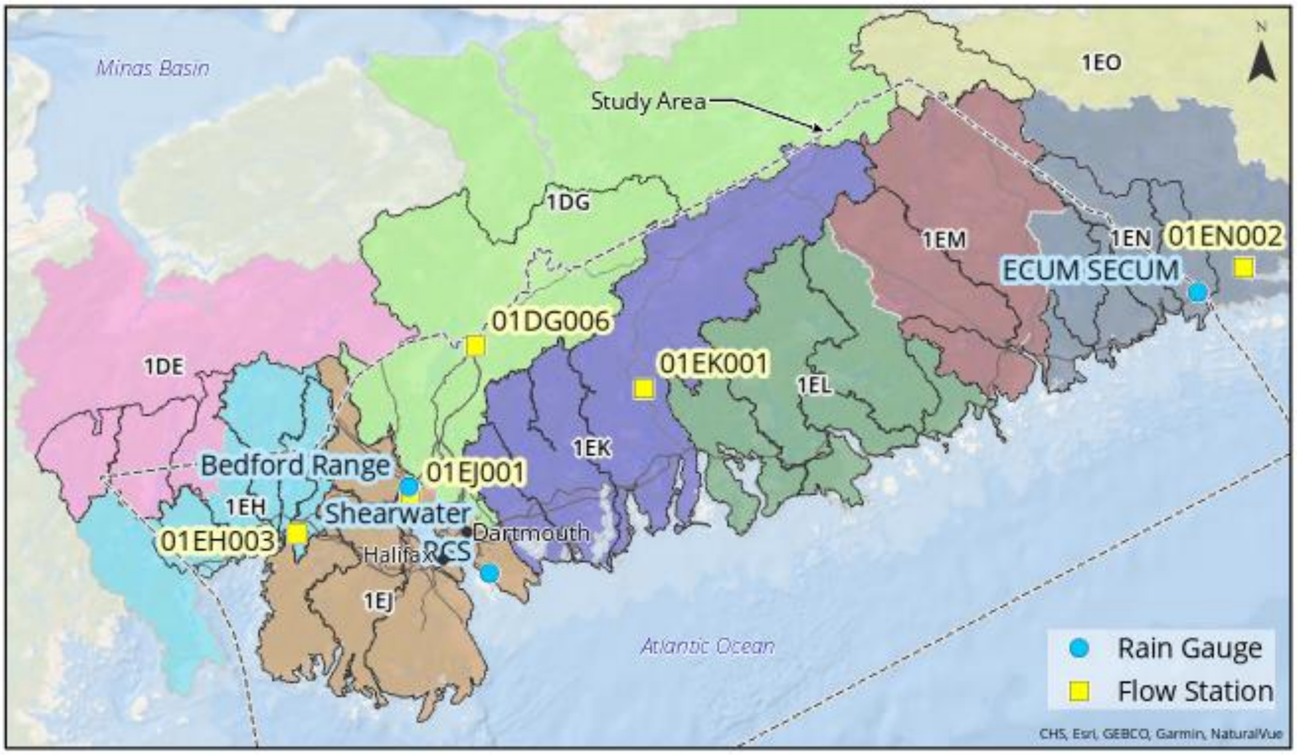

Figure 4-5: Locations of the rain gauge and flow gauging stations.

Calibration results are presented in the figures below, from Figure 4-6 to Figure 4-12 and show how the models are able to reproduce the general flow hydrograph of the extreme flow events, with particular attention to the peak flow. In the figure, the dashed green line shows the measured data, and the continuous red line shows the model results at the same location (at the corresponding model node). The blue columns at the top of the graph represent the measured rainfall data.

A summary table of the parameter adjustments is presented in Table 4-6.

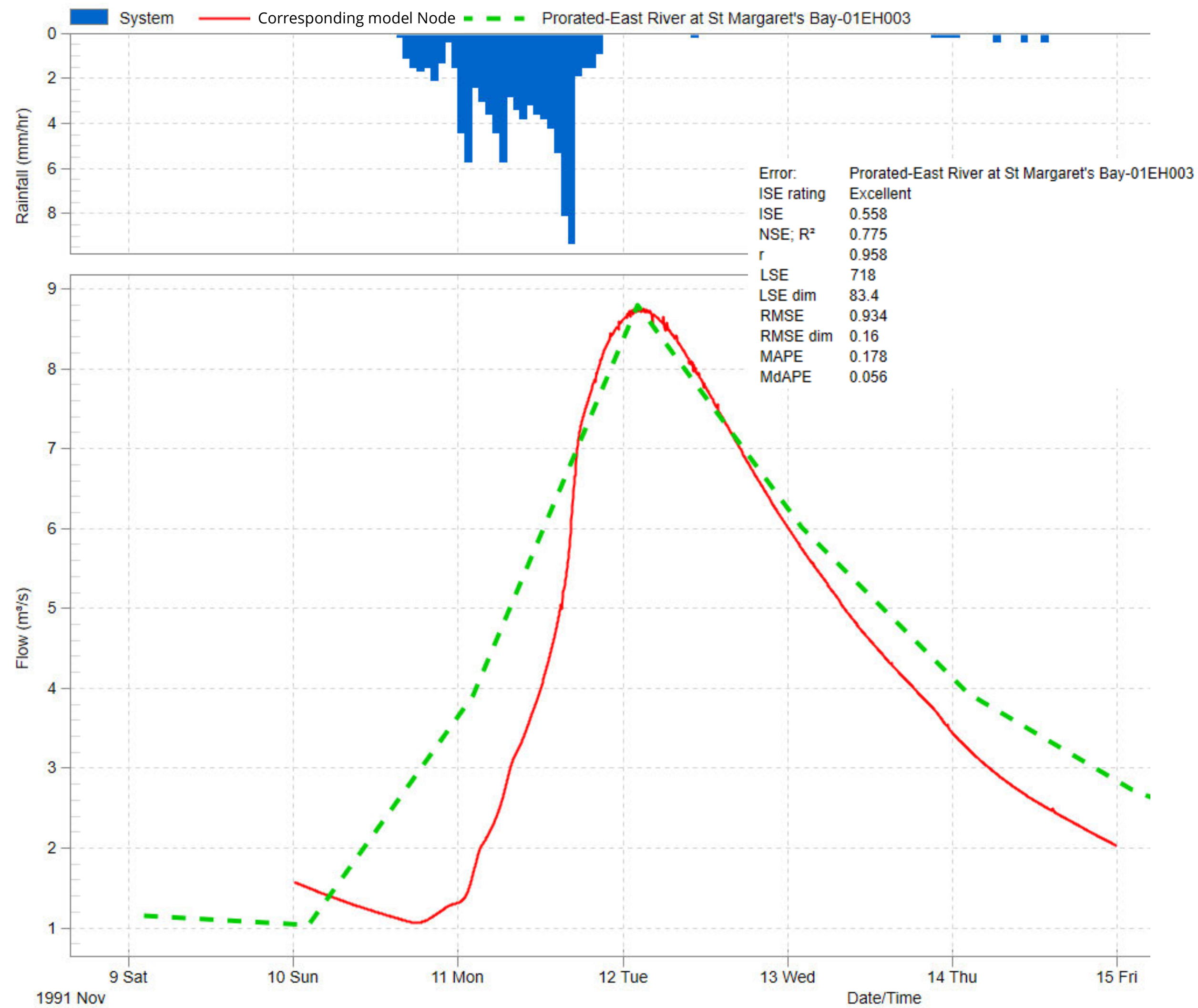

Figure 4-6: Calibration result for the 1EH_1 model.

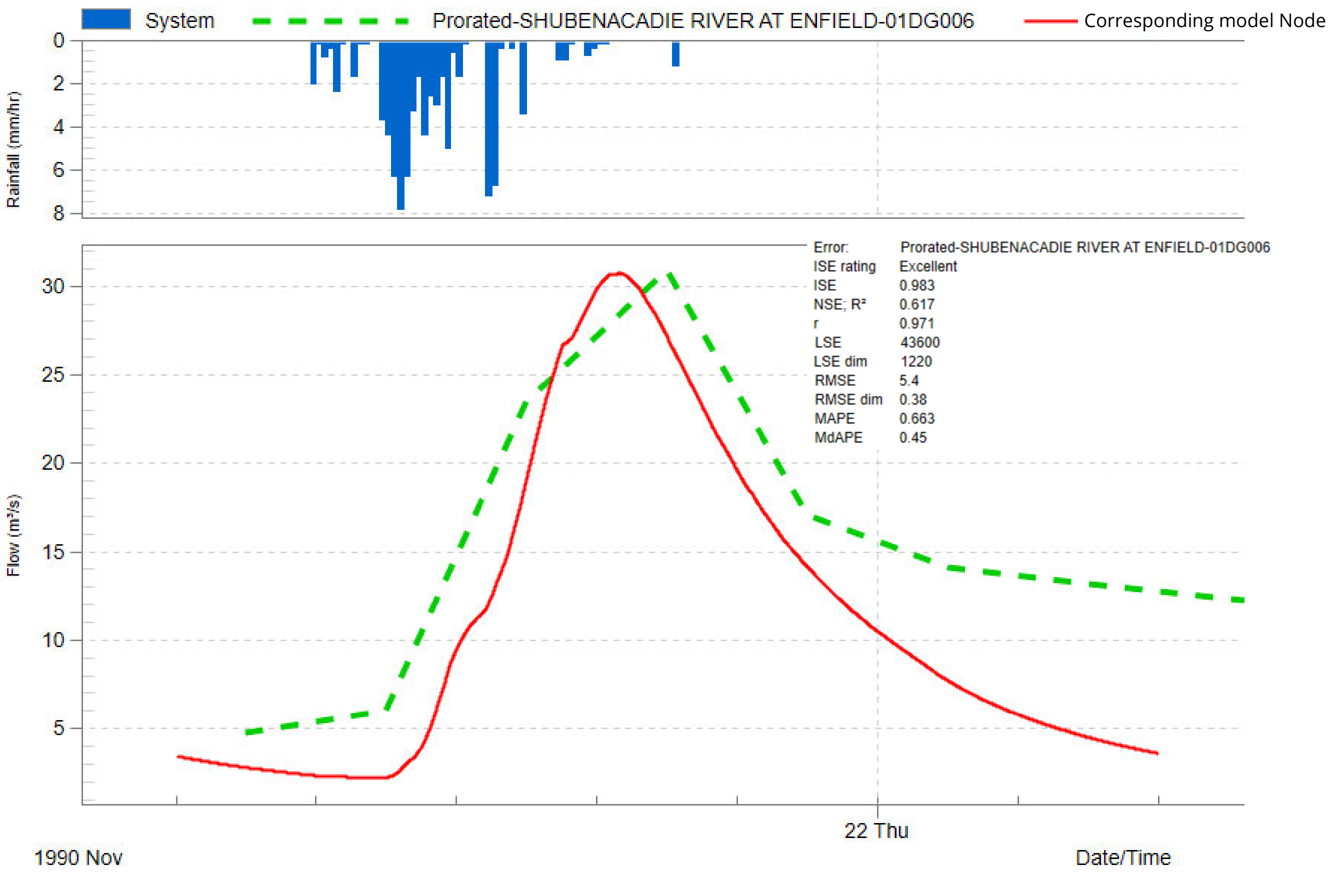

Figure 4-7: Calibration result for the 1DE_2 model.

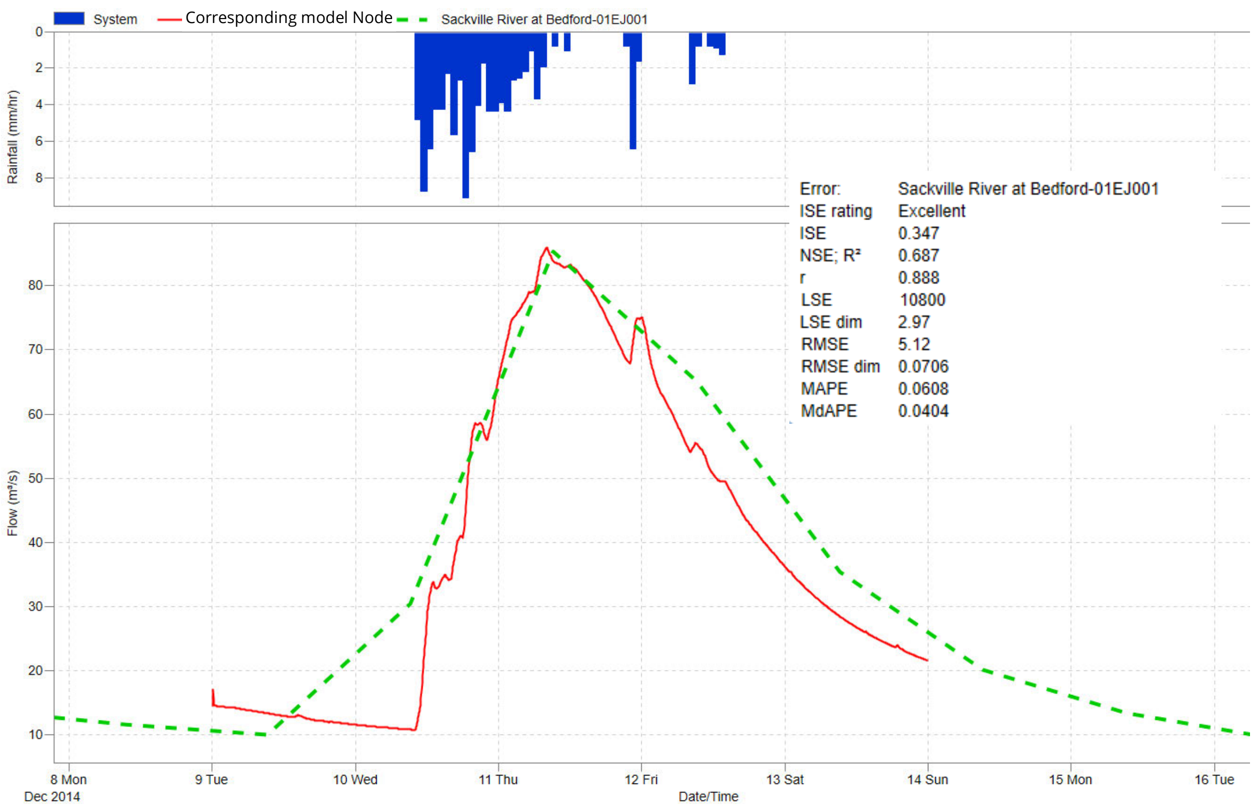

Figure 4-8: Calibration result for the 1EJ_4 model.

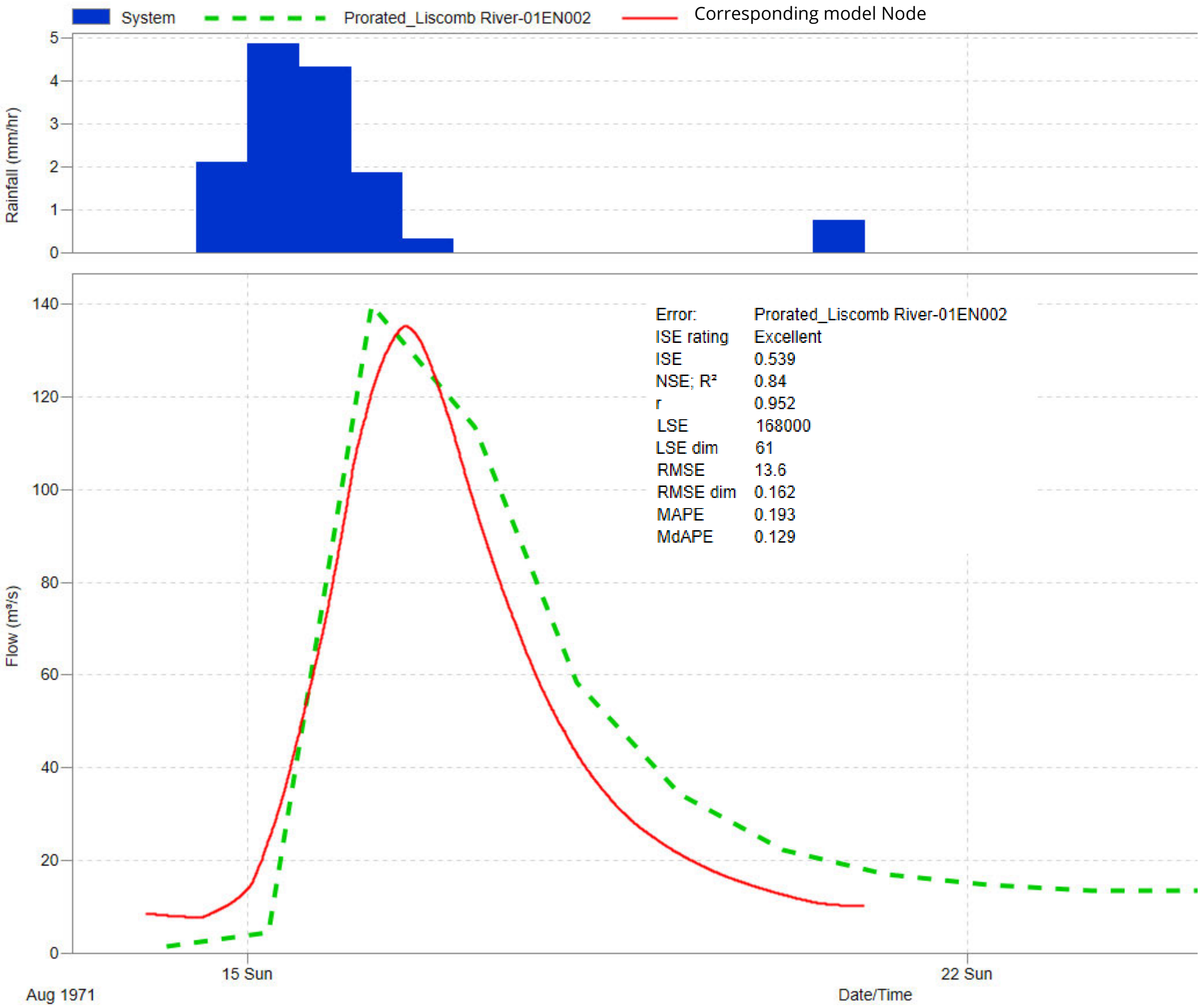

Figure 4-9: Calibration result for the 1EN_2 model.

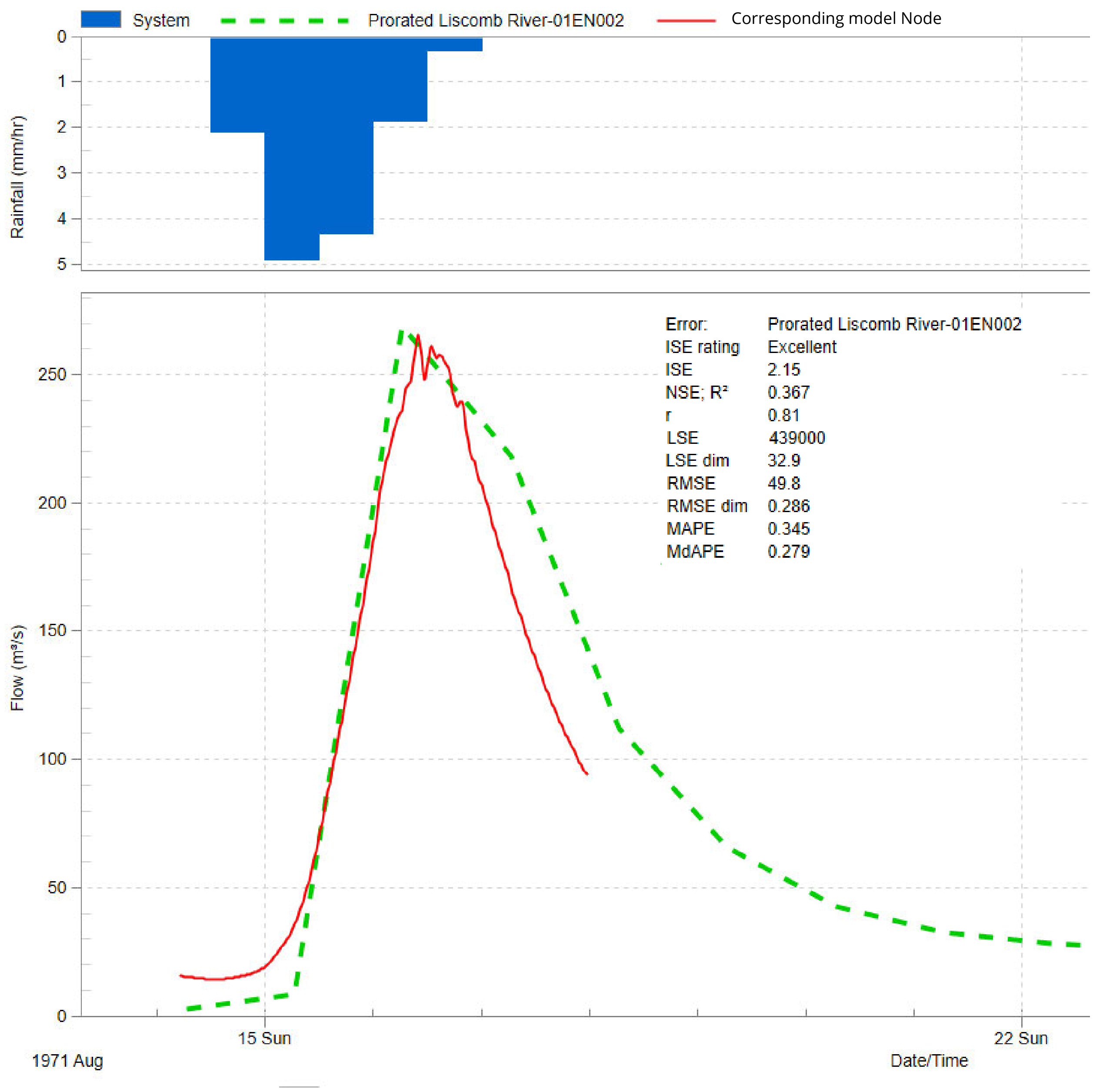

Figure 4-10: Calibration result for the 1EM_2 model.

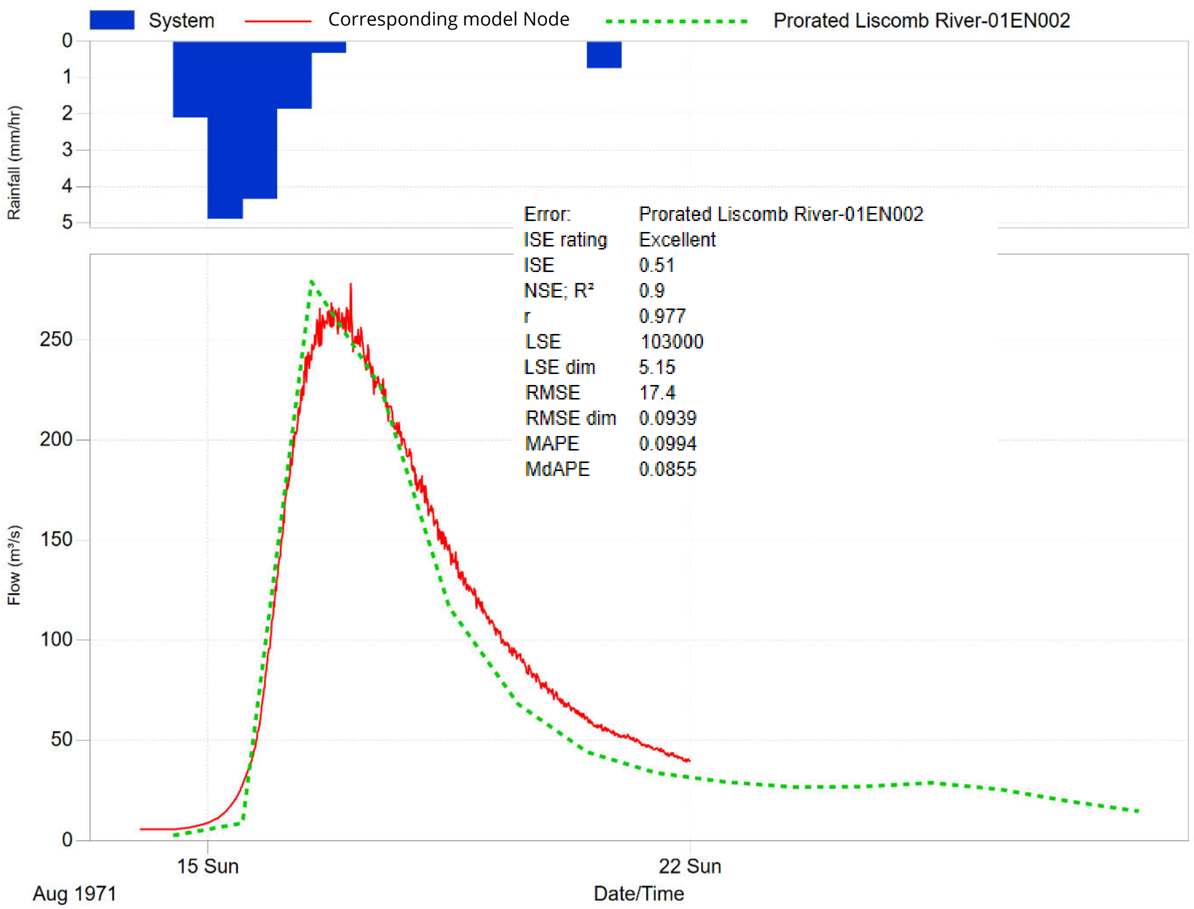

Figure 4-11: Calibration result for the 1EL_5 model.

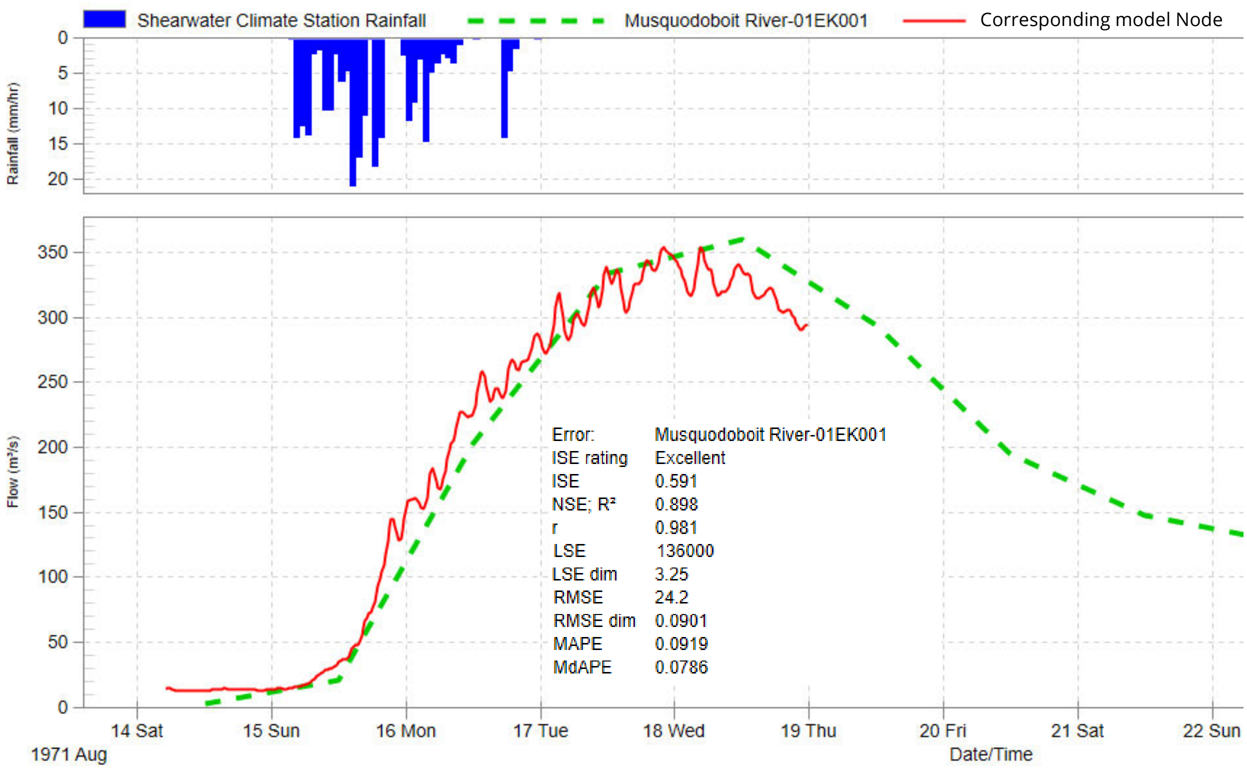

Figure 4-12: Calibration result for the 1EK_4 model.

Table 4-6: Summary Table of Parameter Adjustments to Support Model Calibration

Model Name	Calibration Station ID	Drainage Area Flow Gauge (km^2)	Calibration Source / Methodology	Ksat* Adjustment factor	MOFL** Adjustment factor
1DE_1	01DG006	389	Followed 1DE_2 calibration factors	0.04	4.5
1DE_2	01DG006	389	Used ECCC Flow Gauge in Area	0.04	4.5
1EH_1	01EH003	26.9	Used ECCC Flow Gauge in Area	0.01	9.5
1EH_2	01EH003	26.9	Followed 1EH_1 calibration factors	0.01	9.5
1EH_3	01EH003	26.9	Followed 1EH_1 calibration factors	0.01	9.5
1EH_4	01EH003	26.9	Followed 1EH_1 calibration factors	0.01	9.5
1EH_5	01EH003	26.9	Followed 1EH_1 calibration factors	0.01	9.5
1EN_1	01EN002	389	Followed 1EN_2 calibration factors	0.005	0.5
1EN_2	01EN002	389	Used ECCC Flow Gauge in Area	0.005	0.5
1EN_3	01EN002	389	Followed 1EN_2 calibration factors	0.005	0.5
1EN_5	01EN002	389	Followed 1EN_2 calibration factors	0.005	0.5
1EJ_2	01EJ001	146	Followed 1EJ_4 calibration factors	0.01	0.25
1EJ_3	01EJ001	146	Followed 1EJ_4 calibration factors	0.01	0.25
1EJ_4	01EJ001	146	Used ECCC Flow Gauge in Area	0.01	0.25
1EJ_5	01EJ001	146	Followed 1EJ_4 calibration factors	0.01	0.25
1EL_1	01EN002	389	Followed 1EL_5 calibration factors	0.005	0.1
1EL_3	01EN002	389	Followed 1EL_5 calibration factors	0.005	0.1

Model Name	Calibration Station ID	Drainage Area Flow Gauge (km^2)	Calibration Source / Methodology	Ksat* Adjustment factor	MOFL** Adjustment factor
1EL_4	01EN002	389	Followed 1EL_5 calibration factors	0.005	0.1
1EL_5	01EN002	389	Used ECCC Flow Gauge in Area	0.01	0.125
1EK_1	01EK001	650	Followed 1EK_4 calibration factors	0.01	0.25
1EK_2	01EK001	650	Followed 1EK_4 calibration factors	0.01	0.25
1EK_4	01EK001	650	Used ECCC Flow Gauge in Area	0.01	0.25
1DG_1	01EK001	650	Followed 1DE_2 calibration factors	0.04	4.5
1EO_1	01EK001	389	Followed 1EM_2 calibration factors	0.01	0.512821
1EM_1	01EN002	389	Followed 1EM_2 calibration factors	0.01	0.512821
1EM_2	01EN002	389	Used ECCC Flow Gauge in Area	0.01	0.512821
1EM_3	01EN002	389	Followed 1EM_2 calibration factors	0.01	0.512821

* Ksat is the soil saturated hydraulic conductivity model parameter.
** MOFL is the maximum overland flow length model parameter.

4.4.1 Simulation Performance

Table 4-7 presents an example of time steps, runoff continuity error, flow routing error, and the overall runoff coefficient of each model for the 1 in 100 year rainfall event under the existing climate conditions.

Table 4-7: Time Steps, Runoff Continuity Error, Flow Routing Error, and the Overall Runoff Coefficient of Each Model for the 1 in 100 Year Rainfall Event (Existing Conditions)

Model Name	Minimum Time Step (s)	Average Time Step (s)	Maximum Time Step (s)	Runoff Volume Continuity Error (%)	Flow Routing Continuity Error (%)	Overall Runoff Coefficient (Surface Runoff / Total Precipitation)
1EN_1	0.50	0.50	0.50	-0.04	-12.48	0.995
1EN_2	0.50	0.50	0.50	-0.03	-2.21	1.000
1EN_3	0.50	0.61	1.84	-0.04	-29.19	0.988
1EN_5	0.17	0.51	1.27	-0.04	-29.04	0.988
1EM_1	0.46	0.50	0.54	-0.04	-0.05	0.961
1EM_2	0.36	0.50	0.54	-0.06	0.49	0.972
1EM_3	0.50	0.91	1.71	-0.05	-2.24	0.975
1EL_1	0.17	0.65	2.19	-0.11	-2.37	0.981
1EL_3	0.31	0.54	1.45	-0.08	-35.72	0.978
1EL_4	0.37	0.70	1.16	-0.09	-1.71	0.937
1EL_5	0.09	0.50	0.71	-0.07	0.82	0.968
1EJ_2	0.50	0.53	0.74	-0.09	-18.91	0.984

Model Name	Minimum Time Step (s)	Average Time Step (s)	Maximum Time Step (s)	Runoff Volume Continuity Error (%)	Flow Routing Continuity Error (%)	Overall Runoff Coefficient (Surface Runoff / Total Precipitation)
1EJ_3	0.18	0.50	0.58	-0.07	-5.23	0.976
1EJ_4	0.50	0.50	0.50	-0.12	0.01	0.982
1EJ_5	0.50	0.50	0.50	-0.06	-81.83*	0.976
1DG_1	0.50	0.50	0.50	-0.01	0.61	0.799
1EO_1	0.01	0.60	1.20	-0.02	-0.43	0.981
1EK_1	0.16	0.55	0.80	-0.06	-217.29**	0.472
1EK_2	0.27	0.50	0.57	-0.08	-0.38	0.984
1EK_4	0.04	0.16	0.48	-0.01	8.67	1.000
1EH_1	0.49	0.58	1.38	-0.02	-6.72	0.904
1EH_2	0.13	0.50	2.05	-0.02	2.39	0.869
1EH_3	0.30	0.72	1.06	-0.01	92.48***	0.900
1EH_4	0.46	0.52	0.99	-0.01	1.17	0.884
1EH_5	0.29	0.50	0.87	-0.01	0.49	0.963
1DE_1	0.41	0.50	0.80	-0.02	-0.57	0.925
1DE_2	0.24	0.51	1.36	-0.02	0.66	0.844

* It is noted that the high flow routing error is related to the fact that SWMM does not take into account the inflows from a tidal boundary in its flow routing continuity calculations. In this case, a large proportion of the model is under tidal influence, causing this high value, which is not actually a routing error.
** Similarly, the Cole Harbour tidal bay allows vast volumes of water to flow into the model from its tidal boundary,
*** Similarly, this model is tidally influenced in a large portion of its downstream reach.

4.4.2 Sensitivity Analysis

Appendix G presents the findings of the sensitivity analysis that was carried out relative to the 1 in 100 year rainfall event under the existing climate condition. Nine models were selected for this assessment. The impacts of the uncertainties associated with the subcatchment roughness, subcatchment soil conductivity, channel roughness, and structure openings were evaluated on the modelled water levels and flows results.

4.5 Summary of Key Findings

- The overall objective of the pluvial and fluvial flood mapping effort is to identify initial general flood risk zones to support the identification of infrastructure at risk, emergency management operations, and long-term decision making, to be confirmed with further model refinement and supporting data.
- The assessment was able to identify general flood risk zones, within the limitations of the supporting data and general modelling detail and assumptions. The calibration

results showed good agreement between measured flow data (ECCC stations) and modelled flows at 7 sites in the HRM. The climate change assessment and definition of scenarios allowed the evaluation of risks associated with the uncertainty of a changing climate.

- The modelling shows that urban areas are especially prone to flash flooding, with a fast response to rainfall, with the Halifax peninsula being vulnerable to very short and intense rainfall events. Other areas in HRM will reach peak flows under longer, more continuous rainfall events, reaching several hours in duration, possibly more than 24 hours in some cases.
- Key technical challenges included the vast size of the study area and the length of watercourse to flood map. This is the largest flood mapping effort conducted to date in Nova Scotia and is vastly larger than any previous flood mapping effort in the HRM (almost 200 times more flood mapping length but with similar timelines and budget to earlier efforts). Consequently, the project scope did not include any survey work, was based on existing data only, and relied heavily on automated methods for hydrologic and hydraulic modelling. To reduce the need for detailed manual adjustments throughout the study area, the level of detail of the modelling was increased throughout the study area (to 50m cross-section spacing), so that the topography was better reflected in the model.
- It is expected that the model geometry should be quite representative of the topography of the HRM. The main sources of remaining uncertainty are associated with the available data to support the modelling. The paucity of information available on bridge, culvert, and dam infrastructure translates into uncertainty of flood risk extents both upstream of structures, where water might back up more or less than hydraulic opening assumptions allow, and downstream of structures, where flows could be greater or lower than opening assumptions allow. Another notable source of uncertainty relates to the HRM LiDAR data that had fragmented and assumed elevation data in many of the lakes, rendering flood modelling and mapping in lakes somewhat uncertain. Finally, natural variability and uncertainty related to limited rainfall and flow gauging data over a vast area limits how representative the available data is of actual rainfall and flow processes in the study area. Further detail is provided in the Uncertainty and Limitations sections, as well as the Recommendations section.
- Another source of uncertainty relates to the fact that this assessment is focused on rainfall as a main flooding mechanism. There are other flood mechanisms, such as snowmelt, ice accretion, ice jams, debris accumulation, and debris jams. It is estimated, and supported by HRM staff experience, that ice and debris are rarely a flood mechanism within HRM. There are also main sources of uncertainty unaccounted for in this assessment which include antecedent conditions prior to a flood event, the likelihood of rain on snow and rain on frozen ground, whether some very large watersheds are more susceptible to rainfall events longer than 24 hours, and the joint probability of storm surge and extreme rainfall, which are all the subject of ongoing research. At this point, the recommendation is to follow ongoing research in the province and implement recommended approaches from the results of the research.

- Technical recommendations include collecting survey data for all hydraulic structures (bridges, culverts, dams), collecting further rainfall, flow and water level data throughout the HRM, and regularly refining and updating the hydrologic and hydraulic models.

5 Coastal Analysis and Modelling

The HRM has an extensive coastline, with more than 300 km of coastal areas including bays, inlets, beaches, cliffs, and various types of coastal infrastructure. Flooding due to extreme coastal water levels is a significant hazard throughout this entire area. In addition, sea levels have been rising globally and are projected to continue to rise due to climate change. Paired with extreme storm events, coastal flooding can result in significant damage to coastal infrastructure and flooding of property. Defining extreme water levels is critical for flood mapping, coastal hazard assessments, coastal planning, policy development, infrastructure design, and ecosystem management.

The objectives of this coastal analysis and modelling were to conduct a desktop review of extreme static water levels for the coastal zones of HRM due to the combined effects of tides, storm surges, and sea level rise. Following this, hydrodynamic storm surge modelling was conducted based on the information determined from the desktop review.

This resulted in the production of a spatially varying model of the water surface during extreme events throughout the HRM's coastal zone. This extreme water level surface was then used for the extraction of flooding extents.

This chapter describes the methodology and results of the analysis and modelling used to estimate coastal water levels and flooding extents.

For the definitions of physical processes contributing to coastal flooding, the reader is referred to Section 1.4.2.

5.1 Summary of Previous Studies

The following studies were used to assemble initial supporting data on extreme water levels:

- Halifax Regional Municipality Extreme Water Levels (CBCL, 2022).
- Halifax Harbour Marginal Coastal Study (CBCL, 2020a).
- Coastal Engineering Study for Peggy's Cove Master Plan Project (CBCL, 2020b).
- High Water Levels in Big Lake, Caused by Hurricane Dorian (September 7, 2019) and Changes to Long Beach, Nova Scotia (Taylor et al., 2021).

Coastal model calibration parameters, primarily the extreme value analysis of the Bedford Institute of Oceanography's long term tide gauge, were sourced from data presented in the 2022 study by CBCL. Model validation data was sourced from two CBCL studies, (CBCL 2020a, and CBCL 2020b) as well as the Taylor et al. Big Lake study (2021).

5.1.1 Halifax Regional Municipality Extreme Water Levels

A desktop review of extreme static water levels in HRM was completed by CBCL in 2022 to produce a summarized extreme water levels report, to be used by decision makers, planners, and consultants to determine potential coastal flooding hazards. Extreme static water levels include the effects of tides, storm surge, and sea level rise. Extreme water levels due to storm surge were calculated using a statistical analysis (extreme value analysis) where long-term tide gauge data was available (Zone 3, corresponding to the Bedford basin region, specifically the Bedford Institute of Oceanography (BIO) (see Figure 5-1). The return periods (RP) for the extreme water levels provided in the analysis include 2-, 5-, 10-, 20-, 50-, and 100-year RP storms for all coastal zones and additionally, 200-, 500-, and 1000-year RP storms for Zone 3.

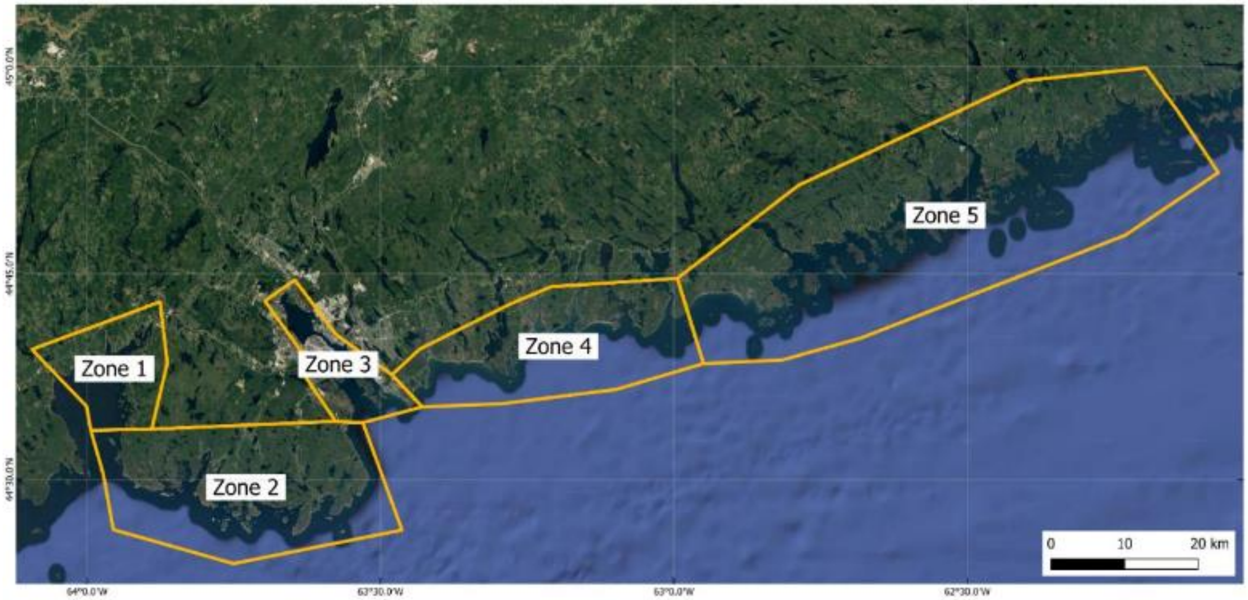

Figure 5-1: CBCL 2022, Extreme Water Levels Study – Zone Definition.

A range of sea level rise projections were derived from the Fifth Assessment Report (AR5, IPCC 2013) and the Sixth Assessment Report (AR6, IPCC 2021).

A key limitation of this study is that the extreme water levels defined are appropriate only for sites on open coasts due to the potential amplification of storm surge that can occur in inlets, bays, etc. and with complex near-shore bathymetry. The extreme static water levels listed in the study only include the impacts of tides, storm surges, and sea level rise. Wave run-up and storm water runoff contributions are not included. Site specific studies are recommended to determine the impacts of these parameters on extreme water levels.

The findings of the 2022 study that are most relevant to the present study are the analysis of extreme water levels at BIO and projections of sea level rise which were both used as modelling inputs.

5.1.2 Halifax Harbour Marginal Coastal Study

This 2020 report (CBCL, 2020a) discusses work done for the Halifax waterfront area to determine flooding extents for future waterfront development projects, considering all relevant processes including:

- Wave run-up.
- Tides.
- Storm surge.
- Sea level rise.

For this project several flood maps for a variety of climate scenarios were developed.

5.1.3 Coastal Engineering Study for Peggy's Cove Master Plan Project

Similar to the 2020 CBCL Halifax Harbour Marginal Coastal Study (CBCL, 2020b), this 2020 report outlines work done to determine coastal flood extents in the Peggy's Cove area and took into account wave run-up, tides, storm surge, and sea level rise. Flood maps for a variety of climate scenarios were developed.

5.1.4 High Water Levels in Big Lake, Caused by Hurricane Dorian (September 7, 2019) and Changes to Long Beach, Nova Scotia

This 2021 report (Taylor et al., 2021, with contribution from CBCL) examines recent storm surge events and morphological changes at Long Beach in Lower East Chezzetcook. The study included tide gauge measurements during Hurricane Dorian (2019), during which the peak water level at Big Lake was 0.5 m higher than at the Halifax (BIO) tide gauge. This illustrates the significant variation in the peak storm surge throughout the region during a given storm event. This work provided valuable model validation data for the present study.

5.2 Study Area

The HRM region has an extensive coastline covering several hundred kilometres, along with many coastal bays and inlets. An overview map of the study area is provided in the following figure (Figure 5-2). The region includes numerous coastal villages and rural communities and the major urban centre surrounding the Bedford Basin. Coastal extreme water levels and SLR pose a risk to existing shoreline properties, working harbours, tourism, properties, seafood processing sites, municipal infrastructure, and cultural and historic sites.

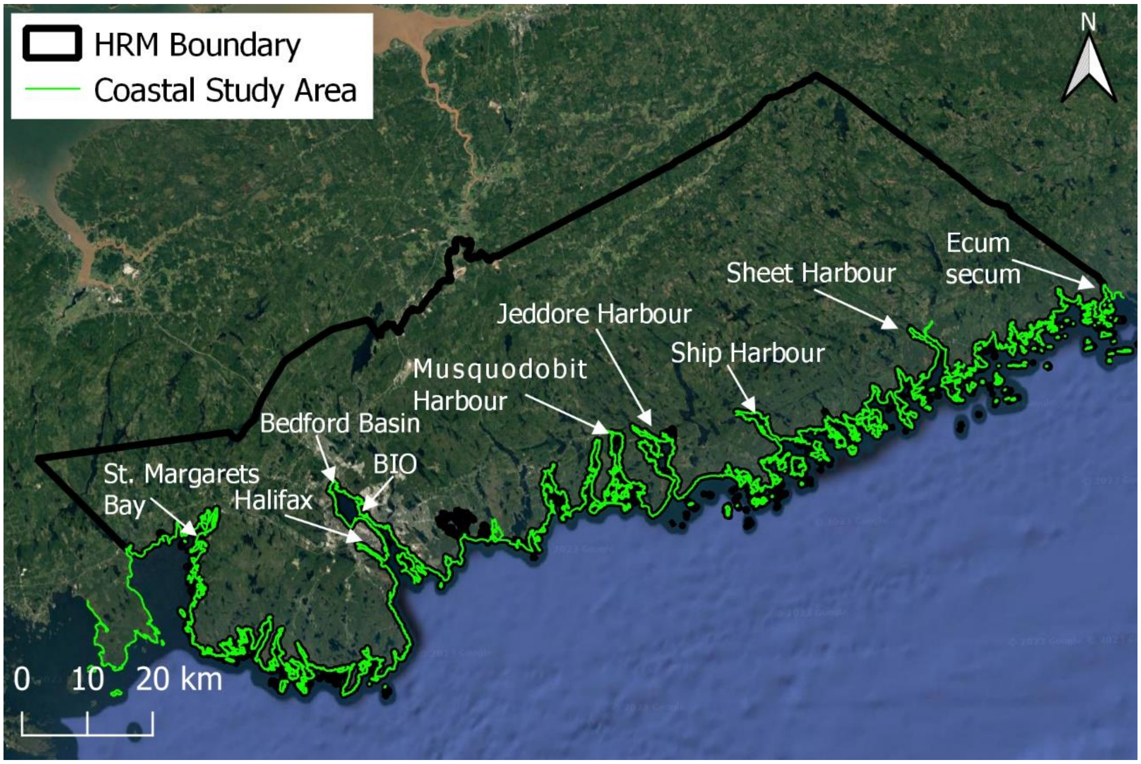

Figure 5-2: Map of project area, showing study extent and locations of places of interest and water level monitoring locations.

5.3 Coastal Methodology

The coastal extreme water level analysis progressed through the following major phases:

1. Analysis of existing information on:
    a. Bathymetry.
    b. Water levels.
    c. Sea level rise.
    d. Offshore wind and wave climate.
2. Tide gauge monitoring at key coastal locations.
3. Hydrodynamic modelling of extreme coastal flood conditions based on the following inputs:
    a. Tides.
    b. Storm surge, including:
        i. Offshore storm surge.

ii. Nearshore surge from wind and wave setup[5].
c. Sea level rise.
d. Extreme water levels measured at Halifax.
4. Calibration of nearshore water levels using the BIO long-term tide measurements
a. Comparison of peak total water levels for all present scenarios considered in the project (measured vs modelled).
b. Evaluation and definition of physical and numerical parameters to improve nearshore water level correlation (measured vs modelled).
5. Production of flood maps for coastal water levels. These include extreme coastal water level flood maps for nine return periods and three sea level rise scenarios:
a. Return periods: 2-, 5-, 10-, 20-, 50-, 100-, 200-, 500-, and 1000-year (the latter two for the 2100 time horizon only).
b. Existing Climate.
c. Future Climate (including sea level rise) for the 2050 time horizon.
d. Future Climate (including sea level rise) for the 2100 time horizon.

5.4 Overview of Relevant Inputs

The methodology of this coastal analysis represents a multifaceted approach that encompasses field monitoring using strategically positioned tide gauges, vertical datum conversions, the analysis of tidal elevations, consideration of sea level rise, examination of storm surge dynamics (offshore and nearshore), and the application of extreme value analysis to long-term tide gauge data. These elements were used as input directly or indirectly for the development of the regional wave and hydrodynamic model that was developed to derive regional hydrodynamic conditions and extreme water levels (See Section 5.5).

In this section of the report, a description of relevant input information is provided.

5.4.1 Field Monitoring Data

5.4.1.1 2023 Tide Gauge Measurements

To better understand the local tidal regime and provide inputs for modelling, further analysis of project-specific and concurrent water level observations was undertaken. The field program successfully provided measured tidal data for 6 sites within the project area (Figure 5-2). Instrument deployment sites and a summary plot of measured water levels is shown on the following figure (Figure 5-3).

[5] Wave setup describes the increase in nearshore still water level (i.e. averaged over a series of individual waves) that is generated from the breaking of waves approaching the shoreline. Wave setup can be a significant contribution of the storm surge for areas exposed to long swell action.

Details on each instrument and individual data plots are presented in Appendix A.

5.4.2 Datum Conversion (Separation Surface)

For purposes of bathymetry conversion throughout the region, the CD-CGVD2013 conversion was performed with the closest HyVSEPs datapoint to the bathymetry point.

The following Hydrographic Vertical Separation Surfaces for Canadian waters (HyVSEPs) were consulted for datum conversion (Robin et al 2016):

- Conversion from Chart Datum (CD) to Canadian Geodetic Vertical Datum of 2013 (CGVD2013) with values shown in Figure 5-3.

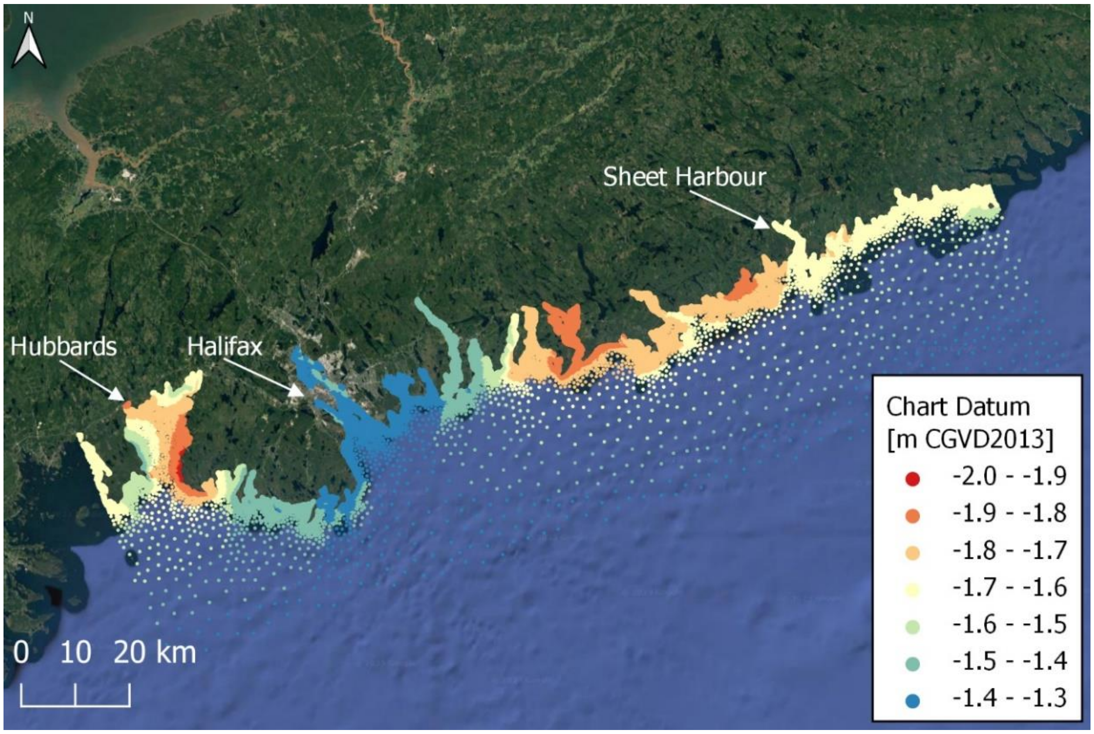

Figure 5-3: The elevation of Chart Datum in the CGVD2013 datum, derived from 2020 CHS HyVSEP surface.

The CGVD2013 elevation of CD was used as a conversion factor to convert elevations between CD and CGVD2013. For example, the Halifax (BIO) chart datum is located at -1.4 m CGVD2013, and so -1.4 m is used as the conversion factor.

$$Elevation_{CGVD2013} = Elevation_{CD} + ConversionFactor$$

5.4.3 Tidal Elevations

Tidal elevations for the project area are sourced from the published DFO elevations for Halifax (at BIO) and are presented in the table on the following page.

Table 5-1: Tidal Elevations at Halifax (BIO)

Tidal Elevations	Metres above Chart Datum (CD)	Metres above CGVD2013
Source : https://tides.gc.ca/en/stations/00491		
Higher High Water Large Tide (HHWLT)	2.17	0.77
Higher High Water Mean Tide (HHWMT)	1.83	0.43
Mean Water Level (MWL)	1.10	-0.30
Lower Low Water Mean Tide (LLWMT)	0.36	-1.04
Lower Low Water Large Tide (LLWLT)	-0.01	-1.41

Tidal elevations are not directly used as inputs for the coastal modelling and are provided only for context and comparison purposes. Additional tidal level information is provided in the CBCL 2022 report titled "Halifax Regional Municipality Extreme Water Levels Final Report".

5.4.4 Sea Level Rise

Climate change-induced sea level rise estimates for 2020, 2050, and 2100 design horizons are derived from the Natural Resources Canada (NRCan) Relative Sea-Level Change tool[6] based on projections from James et al. (2021). The projections for the Bedford Basin, NS, entry is summarized in Table 5-2. These values are taken to be representative of the sea level rise estimates throughout the study area.

The RCP8.5 (median) climate scenario was used for this analysis. Note that the use of the RCP8.5 median values represent an ***intermediate* scenario** and not the more conservative *upper-bound* scenario, which may include the much more conservative RCP8.5 (95th percentile) values over the median (50th percentile) values, along with the collapse of the West Antarctic ice sheet.

For additional information, please see the Halifax Regional Municipality Extreme Water Levels final report (CBCL, 2022).

Table 5-2: Sea Level Rise Estimates at Halifax, NS (Relative to 1986-2005 Conditions)

Sea Leve Rise Scenario	2020	2050	2100
RCP8.5 Median	0.12 m	0.36 m	0.88 m

[6] https://climatedata.ca/explore/variable/slr/?coords=44.69245414103768,-63.63023757934571,12

5.4.5 Storm Surge

Storm surge was estimated for the project area based on the following variables:

- Offshore regional storm surge is based on modelling done of the Atlantic Canada region by Bernier and Thompson (2006). This modelling is on the scale of hundreds of kilometers and accounts for factors such as large-scale pressure changes and wind effects.
- Nearshore local storm surge is based on hydrodynamic modelling done for this project by CBCL. This modelling is on the scale of hundreds of metres (100 m at the shoreline) and accounts for processes such as local wind and wave setup as well as amplification within bays, inlets, and areas with shallow foreshores.
- Extreme value analysis of tide gauge data was used as a calibration factor for the total extreme water levels. This EVA was done on the Bedford Institute of Oceanography (BIO) long-term tide gauge data. The model boundary water levels were adjusted until the modelled total extreme water levels at BIO presented an adequate correlation with the calculated local extreme value.

The inclusion of storm surge in calculated total extreme water levels is outlined in Section 5.4.5.1 and Section 5.5.

5.4.5.1 Offshore Storm Surge Estimate

An existing storm surge hindcast model (Bernier & Thompson, 2006) was used to estimate the regional extreme surge levels. This model predicts extreme storm surge throughout Atlantic Canada for different probabilities and is considered a reliable estimate for offshore storm surge and representative for areas without significant tidal amplification or wind and wave setup (excluding bays, inlets, shallow foreshores, among others). The modelled offshore storm surge values considered for this project are summarized in Table 5-3 and illustrated in Figure 5-4. The area offshore of Halifax was chosen since it is close to the centre of the project area. Offshore storm surge estimates vary ~1-2 cm along the offshore model boundary (~25 km offshore) and so these values are taken as representative estimates of the boundary surge level.

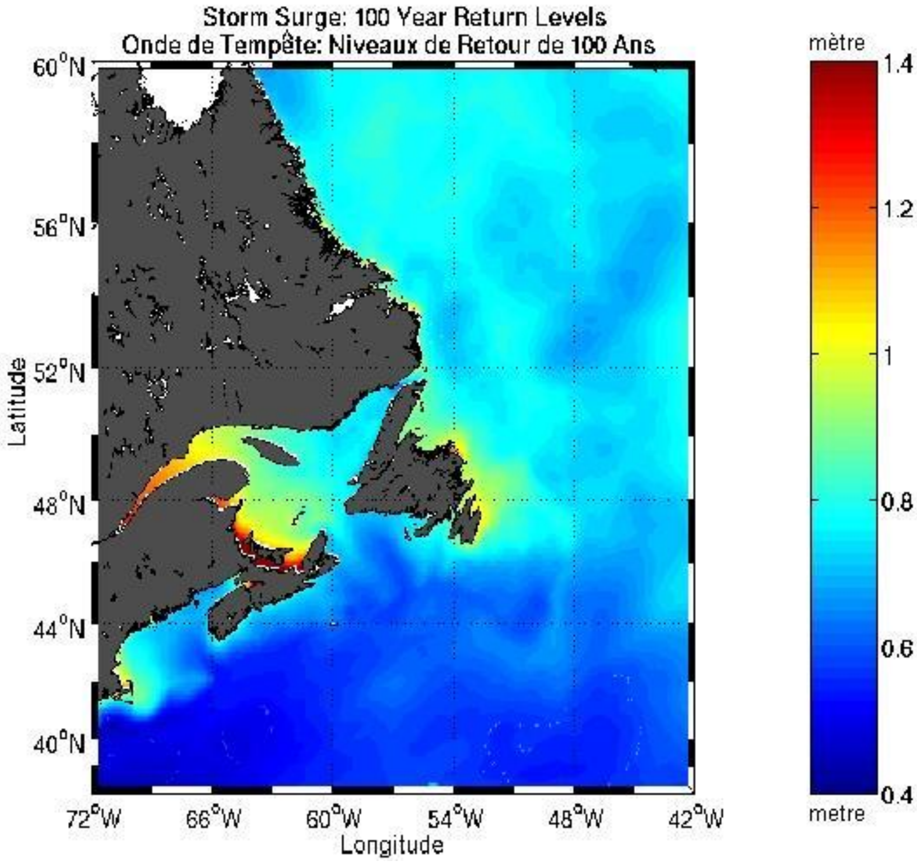

Figure 5-4: Regional Modelled Storm Surge Residual (Bernier & Thompson 2006).

Table 5-3: Offshore Storm Surge Estimate, Offshore of Halifax, NS (Bernier, 2006)

Return Period (RP) / Annual Exceedance Probability (AEP)	Storm Surge Residual (m)
2-year / 50%	0.45
5-year / 20%	0.54
10-year / 10%	0.57
25-year / 4%	0.64
50-year / 2%	0.69
100-year / 1%	0.75

5.4.6 Extreme Value Analysis of Tide Gauge Data in the Bedford Basin

When long-term tide gauge data is available, extreme water levels caused by storm surges can be calculated using a statistical extreme value analysis. For this project area, there is a long-term tide gauge station located in the Bedford Basin at the BIO[7], with approximately

[7] https://www.isdm-gdsi.gc.ca/isdm-gdsi/twl-mne/inventory-inventaire/sd-ds-eng.asp?no=491&user=isdm-gdsi

100 years of data. The dataset was detrended to current mean sea level and used for extreme value analysis of total water levels. Figure 5-5 and Table 5-4 shows the results of the extreme value analysis.

Table 5-4: Extreme Value Analysis of Long-term Tide Gauge at BIO

	Total Water Level [m CGVD2013]		
	2020	2050	2100
1000-year Return Period	1.95	2.20	2.72
500-year Return Period	1.87	2.12	2.64
200-year Return Period	1.76	2.01	2.53
100-year Return Period	1.68	1.93	2.45
50-year Return Period	1.63	1.88	2.40
20-year Return Period	1.49	1.74	2.26
10-year Return Period	1.40	1.65	2.17
5-year Return Period	1.32	1.57	2.09
2-year Return Period	1.21	1.46	1.98

This extreme value analysis of the total water levels at the tide gauge itself is considered representative for the Bedford Basin and area surrounding BIO. Given the extensive coastal coverage of the project area spanning hundreds of kilometers, and the anticipation of varying storm surge levels primarily attributed to wind and wave setup across the region, additional estimates of regional extreme water levels are needed and described in the following sections. A numerical model was developed to estimate the locally variable nearshore storm levels. The extreme water levels calculated for the BIO tide gauge from this extreme value analysis are used for calibration of that hydrodynamic model (for details, see Section 5.5).

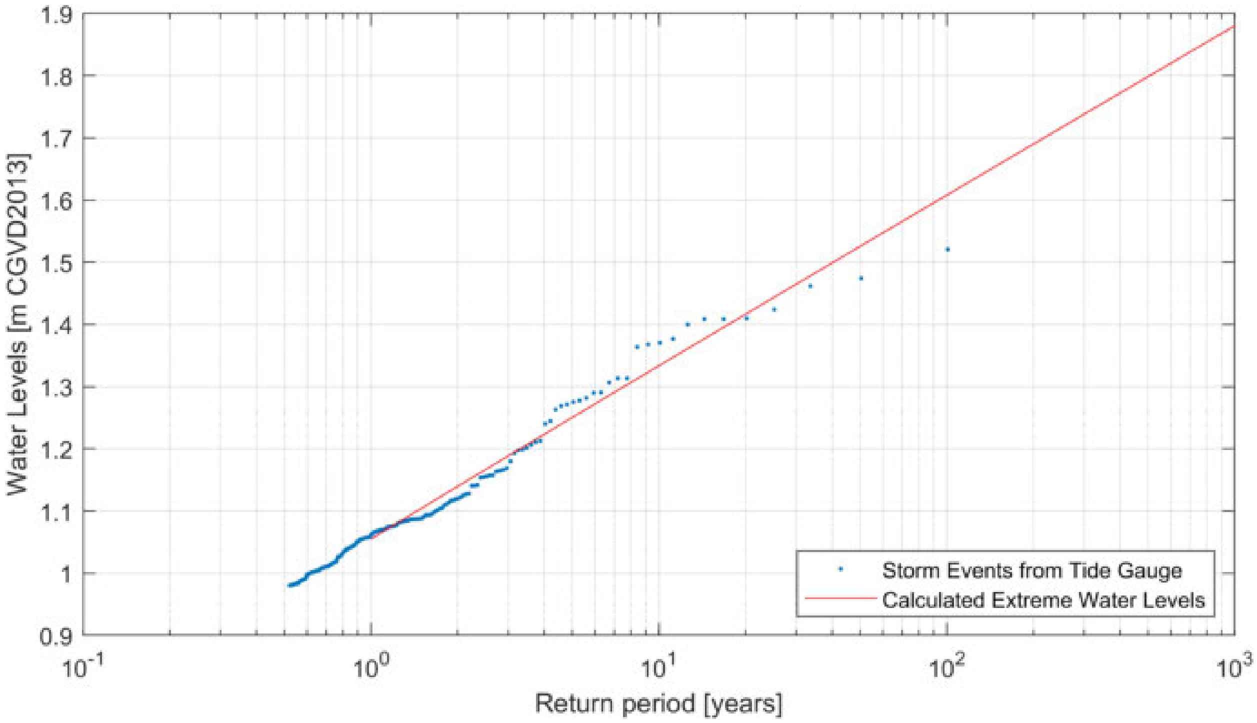

Figure 5-5: Results of the extreme value analysis. This includes the recorded peaks of storm events from the BIO tide gauge and the calculated extreme water levels to the corresponding return period. Note the x-axis has a logarithmic distribution, and therefore the increase in water levels for longer return periods is not linear (CBCL, 2022).

5.5 Hydrodynamic Coastal Modelling

In addition to the regionally estimated offshore storm surge described in the previous section, there will be local increases in storm surge due to wind and wave setup within the many bays, inlets, and shallow areas throughout the study area. This section describes the hydrodynamic modelling performed for this project to estimate this locally variable storm surge.

5.5.1 Modelling Software

Coastal numerical models are a valuable tool to aid in understanding nearshore coastal conditions. The Danish Hydraulic Institute's industry standard two-dimensional MIKE21 suite of models (DHI 2023) was implemented to evaluate extreme water levels throughout the project area. Since wave conditions influence extreme water levels, and vice versa, the spectral wave and hydrodynamic models were run in coupled formulation (HDSW).

Model types and application areas are summarized in Table 5-5 with associated key inputs and outputs.

Table 5-5: Summary of Models Applied in Coastal Assessment

Area of Application	Model	Objective	Main Inputs and calibration	Outputs
Wave Conditions	2D Spectral wave model MIKE21 SW (Coupled)	Wave conditions for input into HD model	Extreme values from offshore wave and wind hindcast	Nearshore wave parameters
Hydrodynamics	2D Hydrodynamic model MIKE21 HD (Coupled)	Extreme water levels and water levels for input into SW model	Extreme values from tide gauge observations	Spatially varying extreme water levels

Coastal modelling was performed to simulate the increase in water levels expected for the 1 in 2-, 5-, 10-, 20-, 50-, 100-, 200-, 500-, and 1000-year return period storm events. The model domain and boundary conditions are described in subsequent sections.

5.5.2 Model Mesh and Bathymetry

Bathymetry for this project was sourced from the Canadian Hydrographic Service's Non-Navigable bathymetry dataset (CHS-NONNA), with extensive bathymetry coverage throughout the project area (Figure 5-6).

Nearshore areas are of sufficient resolution for the purposes of the modelling assignment. However, there are a small number of isolated areas with incomplete coverage which were removed from the modelling domain, the most significant of which being the area of Porters Lake. For these areas, the results of the closest modelled area is taken to be representative of these small unmodelled areas, e.g. the storm surge modelled at the mouth of Porters Lake is taken to be representative of the storm surge within the interior of the lake (see coastal flood mapping limitations section).

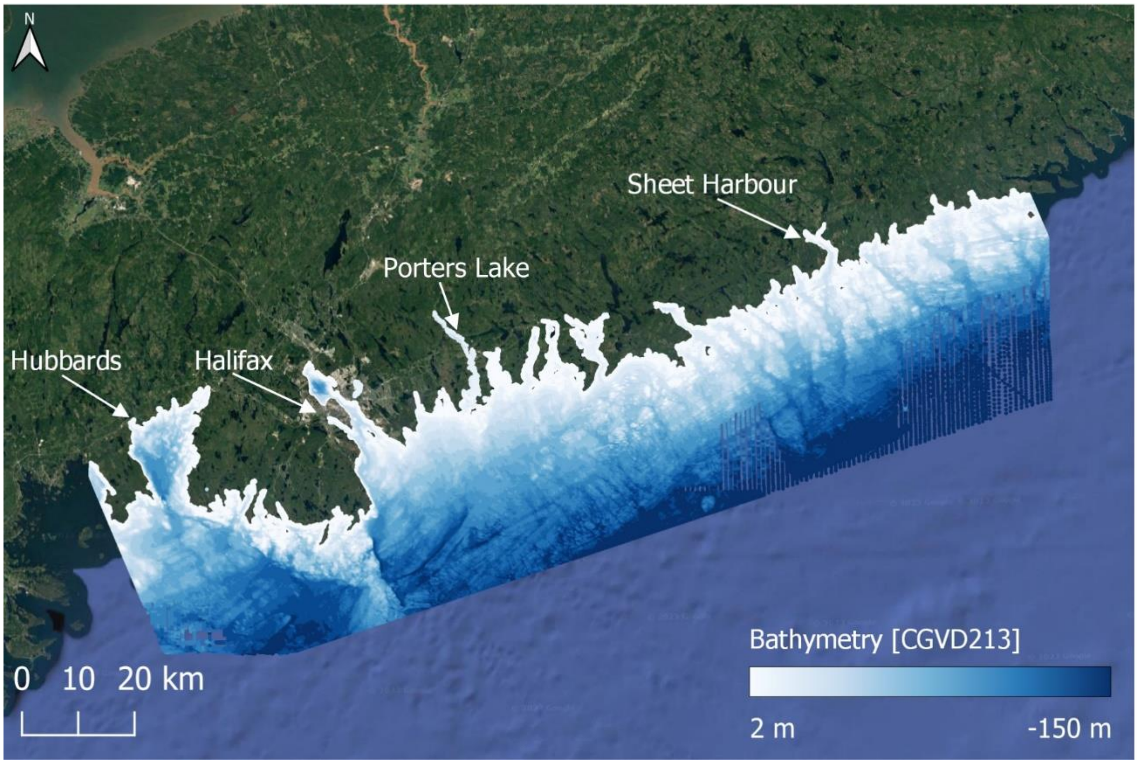

Figure 5-6: Bathymetry used for numerical modelling (CHS-NONNA).

The mesh used for this model is shown in Figure 5-7 showing the full model domain and Figure 5-8 depicting a detail at Halifax. These figures show the mesh resolution of approximately 100 m at the shoreline throughout the model domain and increasing in cell size with increasing water depth.

This mesh encompasses an area between St Margarets Bay in the west, to Ecum Secum in the east, containing the extent of coastal HRM.

This is a large model domain, covering over 300 km of coastline. The minimum model resolution is 100 m, throughout the domain, and therefore areas with complex coastal geometries in the sub-100 m resolution range do not have local effects modelled as precisely. Many of these complex areas would benefit from site-specific modelling where much higher resolutions are possible. These would include specific pieces of infrastructure, such as individual bridges, coastal dunes, or structures.

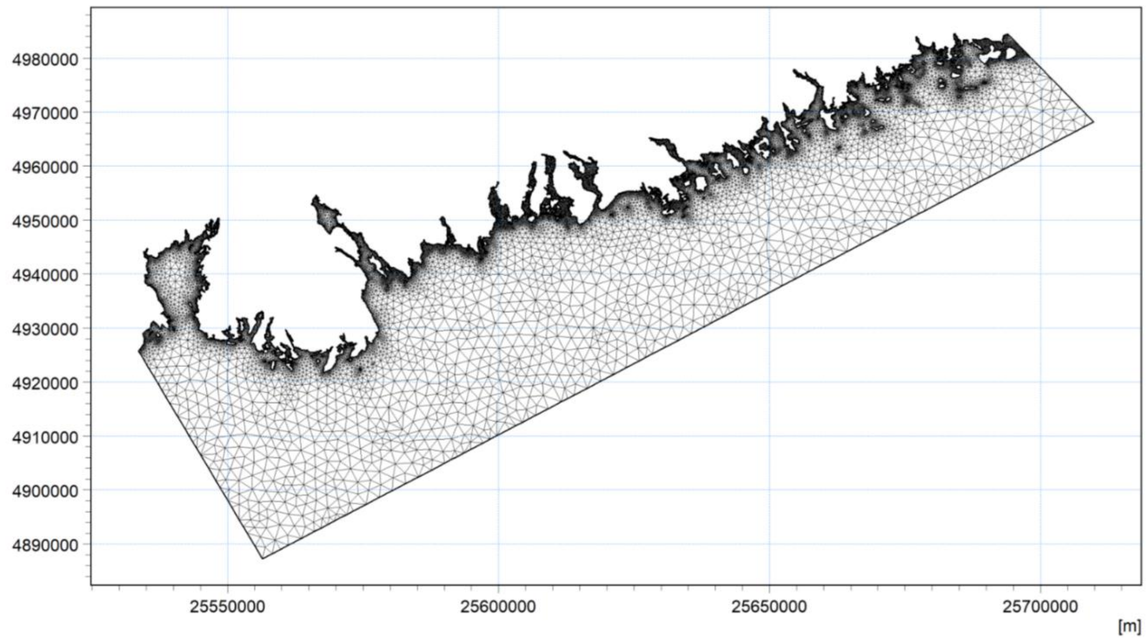

Figure 5-7: MIKE 21 numerical model mesh, full domain (NS MTM5 coordinates).

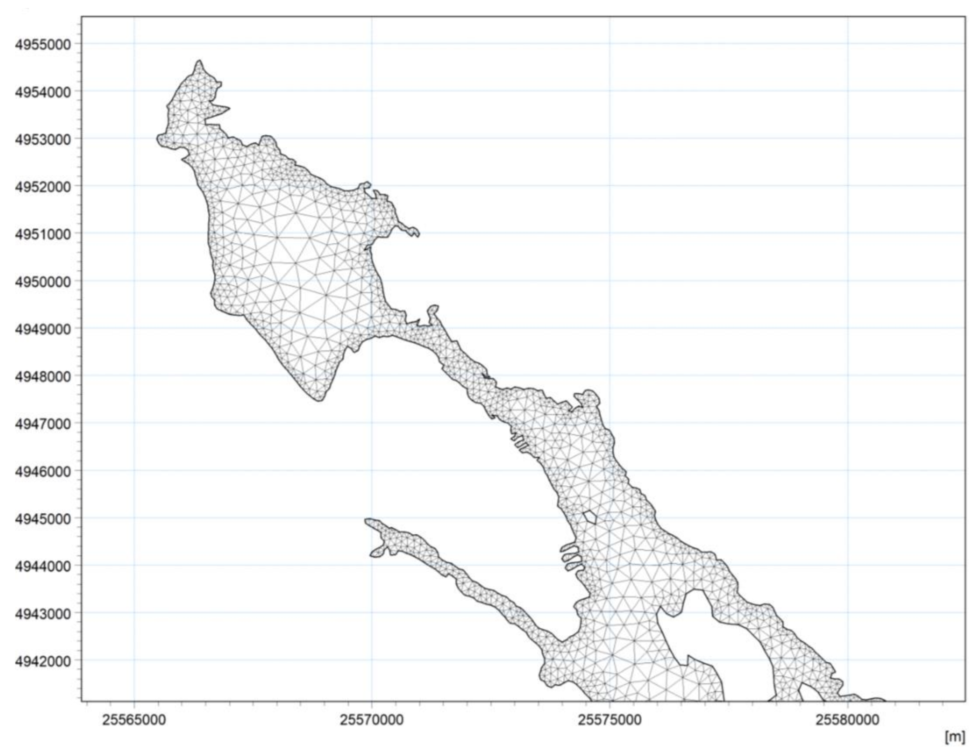

Figure 5-8: MIKE 21 numerical model mesh detail, Halifax Harbour (NS MTM5 coordinates).

The bathymetry interpolated to the hydrodynamic model mesh is presented in Figure 5-9. The bathymetric interpolation was completed by prioritizing the expected quality and reliability of the data, with higher resolution data (i.e. NONNA10) taking precedence over lower resolution data (i.e. NONNA100).

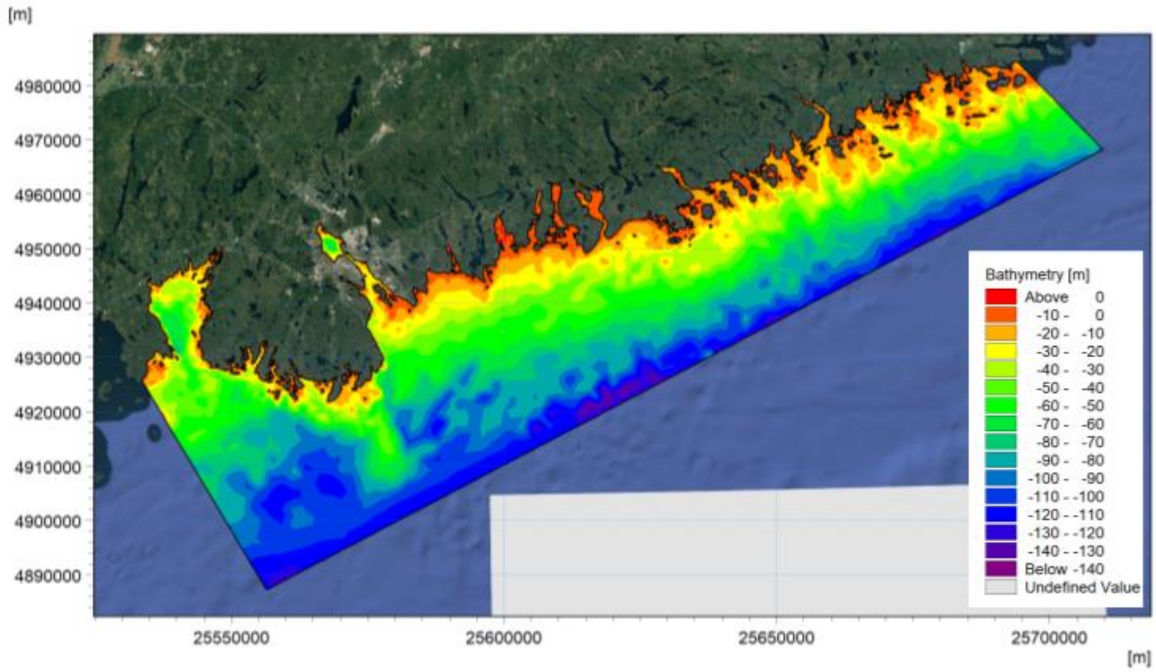

Figure 5-9: Bathymetry interpolated to a model mesh.

5.5.3 Offshore Boundary Conditions

Extreme wind and wave conditions to drive the model boundary were developed from an extreme value analysis of the DFO MSC50 hindcast data, presented in Appendix E. The analysis produced the 1 in 2-, 5-, 10-, 25-[8], 50-, 100-, 200-, 500-, and 1000-year return period wind speeds and wave parameters.

Directional extreme value analyses were completed on the MSC50 offshore winds and waves for all relevant directions of exposure, including east, southeast, south, and southwest. Sensitivity testing confirmed that south and southeast are the directions of exposure most critical (highest contribution) for the generation of wind and wave setup throughout the domain.

Preliminary offshore water levels used as initial input for the model were derived from the offshore storm surge estimates and HHWLT levels. The offshore water levels were then adjusted until the modelled water levels at Halifax (BIO) matched those calculated by the extreme value analysis (see Section 5.6 for more extreme value analysis results, and HRM EWL Report (CBCL, 2022) for additional detail). Different than the simplified approach often used in preliminary level extreme water level studies, this project uses a total water level analysis approach that is further described in the coming sections.

[8] 25-year offshore wind and wave parameters, as well as offshore surge estimates, are considered to be indicative of 20-year conditions.

5.5.4 Model Results

The result of the coastal modelling is a spatially varying extreme water level throughout the HRM's coastal zone that matches (within 1 cm) the BIO tide gauge extreme water levels for the selected return periods as described in the CBCL 2022 Extreme Water Levels report (Zone 3 in that report). For areas outside the Halifax tide gauge location (i.e., Zones 1, 2, 4, and 5, as well as parts of zone 3 that are distant from the Halifax tide gauge), extreme water levels spatially vary as determined from hydrodynamic modelling. This leads to a more accurate storm surge estimation throughout the whole of the coastal HRM.

Nearshore storm surge modelling shows an increase in water levels within many of the inlets and coves throughout the project area, and relatively less surge occurring along stretches of open coastline. For example, in the Portuguese Cove area, little additional surge is expected above the offshore value, with extreme water levels lower than those expected at Halifax. In contrast, the highest modelled water levels within the project area occur along the elongated inlets of the Eastern Shore such as Musquodoboit Harbour, Petpeswick Inlet, and Chezzetcook Inlet, where inlet orientation and shallow bathymetry appear to increase storm surge caused by wind and wave setup. As comparative examples, the modelled 100-year extreme water levels across a few locations are as follows:

- 1.3 m [CGVD2013] at Portuguese Cove.
- 1.7 m at Halifax.
- 2.3 m in the upper reaches of Musquodoboit Harbour, i.e. 1.0 m greater than at Portuguese Cove.

Example water levels resulting from the coastal modelling are depicted in Figure 5-10 and Figure 5-11, showing the 20-year and 100-year events.

The model results from each of the two primary directions of exposure (SE, S) are collected and the highest water level from these directions is taken to be representative of the highest water level at each location[9]. This allows for the presentation of composite high-water levels at each location that are independent of storm direction.

The flood maps are generated using these spatially varying model results throughout the project area. The mapping process itself is described in further detail in Section 6.1.2.

[9] For example, if a model cell calculates a water level of 1.2 m from the Southeast, and 1.4 m from the South, the value for that model cell is taken to be 1.4 m.

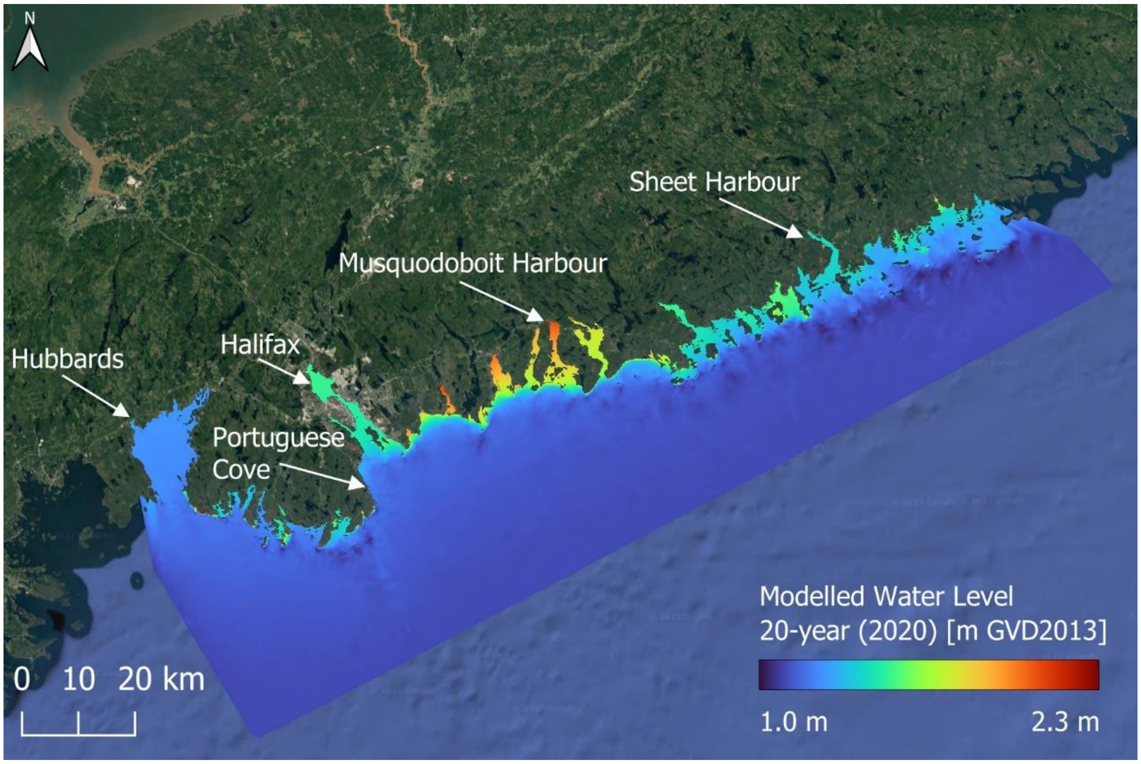

Figure 5-10: Modelled Extreme Water Level: 20-year scenario, present climate conditions, composite of SE and S directions.

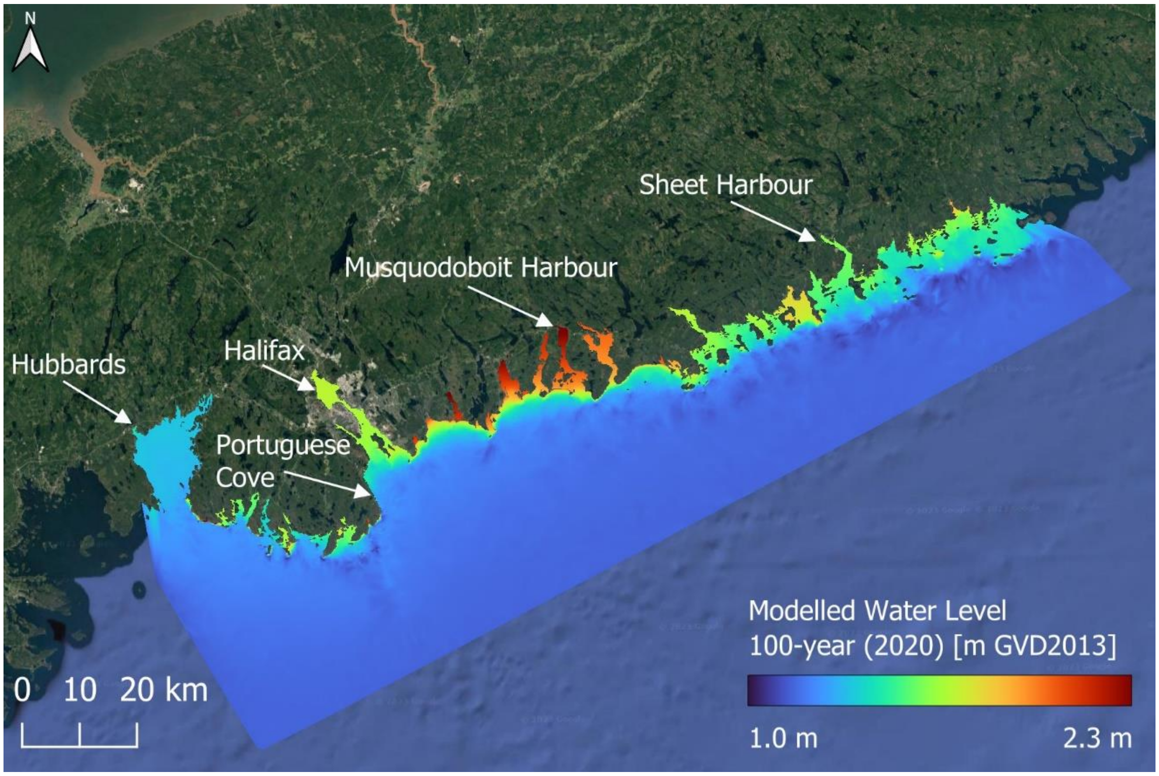

Figure 5-11: Modelled Extreme Water Level: 100-year scenario, present climate conditions, composite of SE and S directions.

5.5.5 Validation Process

The results produced from this analysis were compared with previous studies performed by CBCL. These are: a study of extreme coastal water levels within the HRM (CBCL, 2022), a study of flood extents at the Halifax waterfront (CBCL, 2020a), a study of flood extents at Peggy's Cove (CBCL, 2020b), and a report on high water levels measured at Halifax and Big Lake during Hurricane Dorian (Taylor et al., 2021). Main identified differences arise based on the different processes that are considered for the flood mapping, mainly wave run-up (which is not included in this study due to the large geographical coverage). The resulting maps from this study are still considered a high-resolution representation of the total still water levels for various return periods on future projections, and can be used for preliminary planning, decisions making, and to inform various stages of infrastructure at risk projects.

5.5.5.1 CBCL 2022 Extreme Water Levels Report

CBCL's previous work on coastal extreme water levels (CBCL, 2022) established extreme water level estimates for five zones throughout the HRM. The zones of that assessment are superimposed on the 20-year and 100-year results of the present coastal analysis (Figure 5-12 and Figure 5-13). Since the present analysis is done in much higher resolution, there is a wide range of extreme water level values within the areas covered by the five zones.

Figure 5-12 and Figure 5-13 show the EWL values modelled for the 20-year and 100-year events (2020 conditions). For comparison, Table 5-6 shows the water levels calculated in the previous analysis for these same scenarios.

In general, the EWL values predicted by CBCL's 2022 report fall within the range of the modelled results of the present hydrodynamic modelling, with the exception of Zone 1 where the present analysis shows lower modelled results. Additionally, the confidence of results in Zone 3 is much higher than the other zones due to the presence of long-term water level measurements at BIO, and this allows for the application of the BIO EVA results throughout the HRM at a much higher resolution as well as the computation of more extreme return periods than the previous analysis. Each zone is discussed individually below.

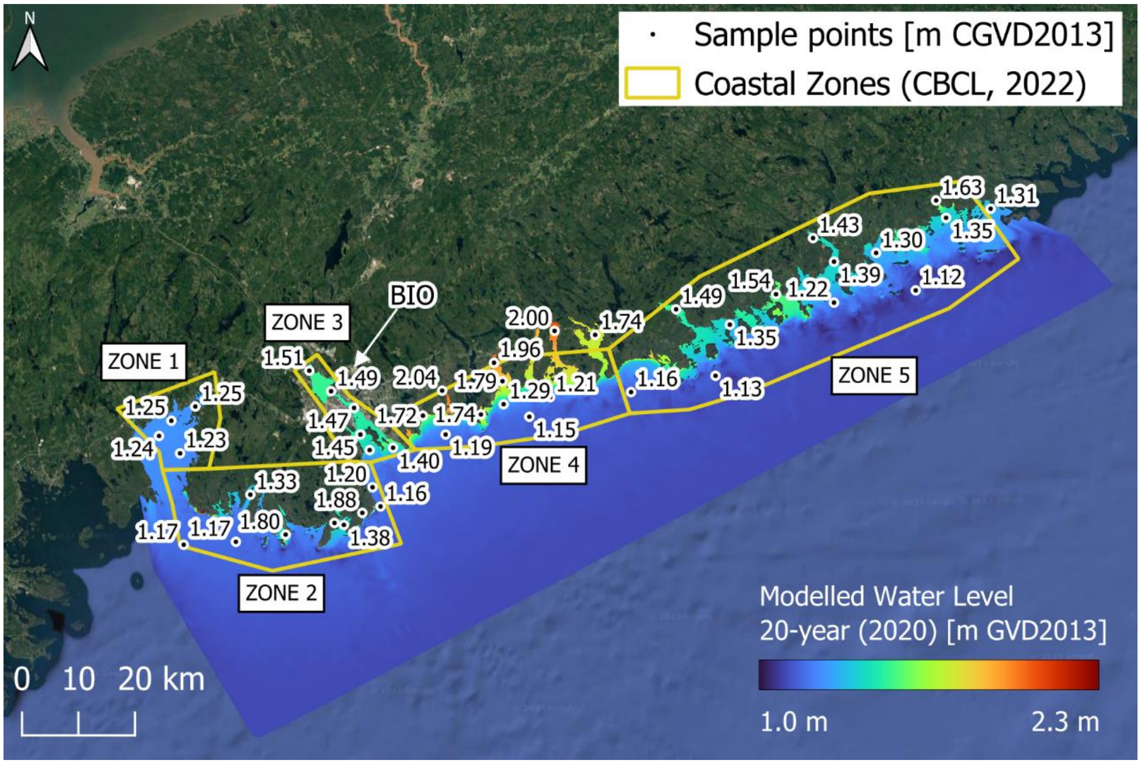

Figure 5-12: Modelled 20-year EWL with sample point values and zones superimposed from CBCL's previous EWL report (CBCL, 2022).

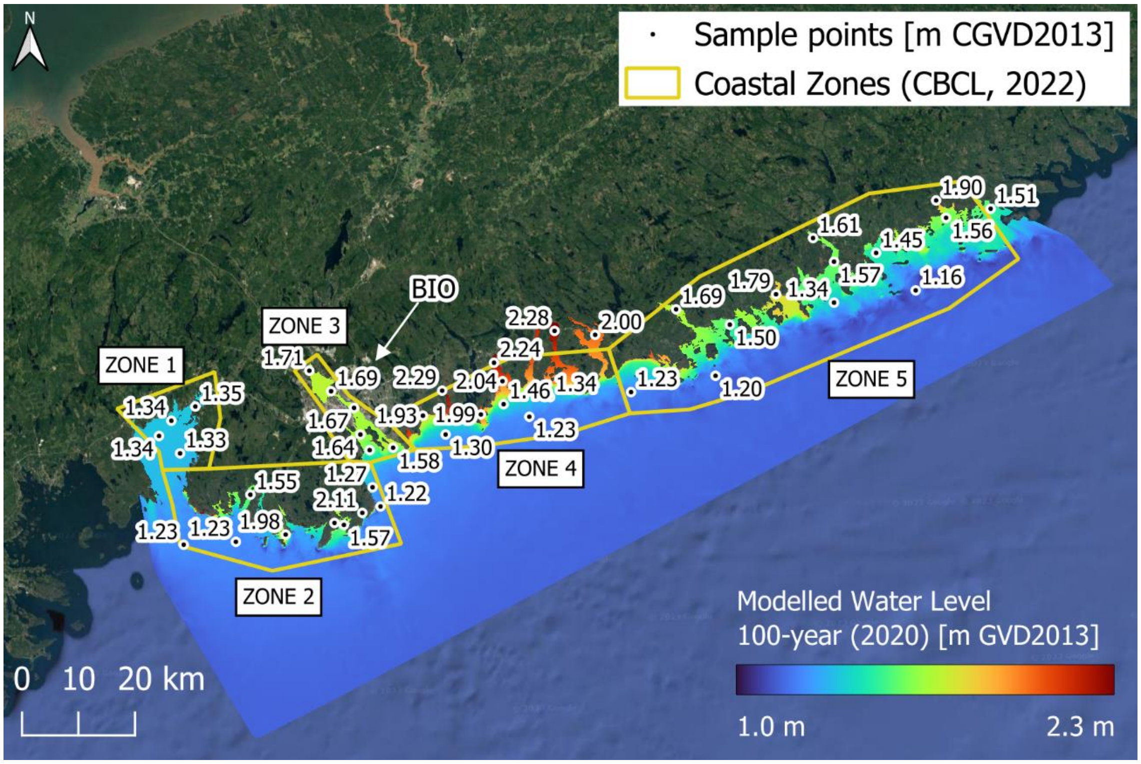

Figure 5-13: Modelled 100-year EWL with sample point values and zones superimposed from CBCL's previous EWL report (CBCL, 2022).

Table 5-6: Comparable EWL Values from CBCL Halifax Extreme Water Levels Report (CBCL, 2022)

Water Level (m CGVD2013)	Zone 1	Zone 2	Zone 3	Zone 4	Zone 5
20-Year RP	1.71	1.56	1.49	1.46	1.42
100-Year RP	1.82	1.66	1.68	1.55	1.50

Zone 1

Zone 1 is the zone with the largest difference between the present methodology and that of the CBCL 2022 analysis. The extreme water level in Zone 1 was assessed at approximately 0.5 m higher in CBCL's 2022 report than in the present analysis. This discrepancy can be accounted for by the methods and data sources used for each analysis. The present analysis was completed using an extreme value analysis of the long-term tide gauge at BIO and by applying the hydrodynamic coastal model, extending those results throughout the HRM. The previous analysis was completed using published tide table values for a representative HHWLT and conservatively adding the regional offshore storm surge modelled by Bernier (2006). The regional surge modelling resolution is coarse and approximate in this area, and due to the complexity of the geometry of St. Margaret's Bay itself, the present model is thought to be a more reliable representation of extreme water levels in St. Margaret's Bay than the previous analysis. The modelled EWLs in Zone 1 vary relatively little, with modelled values between ~1.2 and ~1.3 m CGVD2013 for 20-year conditions, and between ~1.3 and ~1.4 m CGVD2013 for 100-year conditions.

Zone 2

The values assessed by CBCL's 2022 report for Zone 2 fall within the modelled range of values. Depending on the geography of the coastal areas, in particular in shallow inlets like Indian Harbour or Ketch Harbour, the modelled EWL varies. The modelled EWLs in Zone 2 vary between ~1.2 and ~1.9 m CGVD2013 for 20-year conditions, and between ~1.2 and ~2.1 m CGVD2013 for 100-year conditions.

Zone 3

Zone 3 is the zone whose values were used for the definition of the methodology of this project and as calibration of the modelling framework that was developed. The values at BIO where the extreme value analysis was performed for the 2022 report were resolved within 1 cm for the hydrodynamic modelling. The modelled EWLs in Zone 3 vary between ~1.3 and ~1.5 m CGVD2013 for 20-year conditions, and between ~1.3 and ~1.7 m CGVD2013 for 100-year conditions.

Zone 4

Zone 4 shows the largest range of modelled EWL values. This is driven by the fact that narrow inlets with complex geometries in this zone lead to extreme amplification of total EWL. The modelled EWLs in Zone 4 vary considerably, with modelled values between ~1.2

and ~2.0 m CGVD2013 for 20-year conditions, and between ~1.3 and ~2.3 m CGVD2013 for 100-year conditions. The values predicted in CBCL's 2022 report fall within these ranges.

Zone 5

The modelled EWLs in Zone 5 are less extreme than in Zone 4 and vary between ~1.1 and ~1.6 m CGVD2013 for 20-year conditions, and between ~1.2 and ~1.9 m CGVD2013 for 100-year conditions. The values predicted in CBCL's 2022 report fall within these ranges.

5.5.5.2 Halifax Waterfront

A comparison of the results in Figure 5-14 between the present study and the 2020 CBCL study shows that there is generally very good agreement between the predicted water levels and flood extents. There is a slight increase in the flood extent in the 2020 study due to the inclusion of wave run-up in its methodology. The site is also located very close to the Halifax BIO long-term tide gauge and thus there is little expected difference in storm surge between the waterfront and the tide gauge.

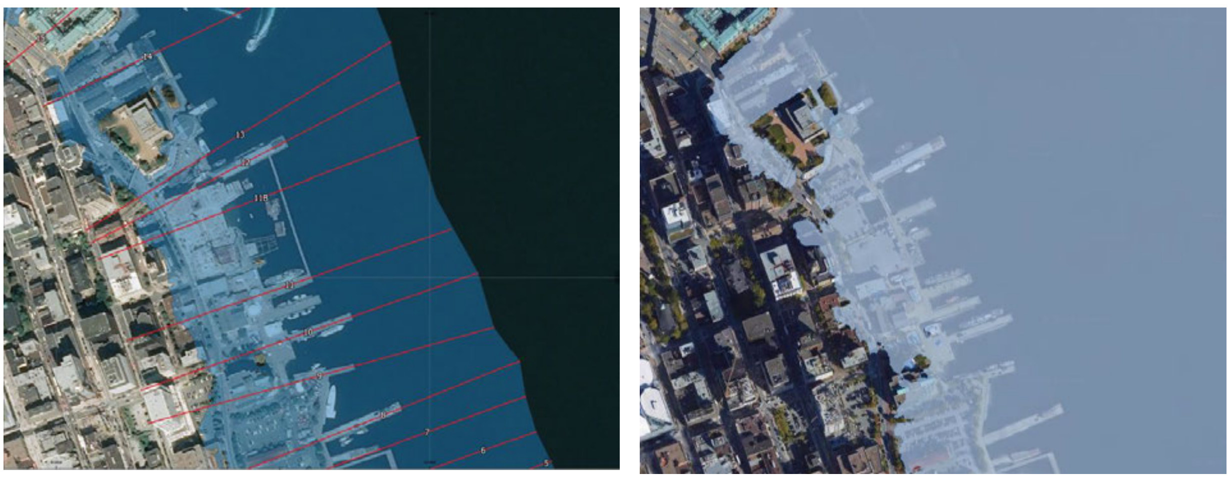

Figure 5-14: Comparison of flood extents developed for the Halifax Waterfront. CBCL, 2020a (left) and present study (right). 100-year event, year 2100 for both studies.

5.5.5.3 Peggy's Cove

The comparison of the predicted flood extents in Figure 5-15 between the present study and the 2020 CBCL study of the Peggy's Cove area show a larger flooded area in the 2020 study than predicted by the present study, as would be expected since the 2020 study includes the effects of wave run-up. Due to the exposure at the site and the magnitude of the waves, a significant increase in flooding extents would be expected when wave run-up is included.

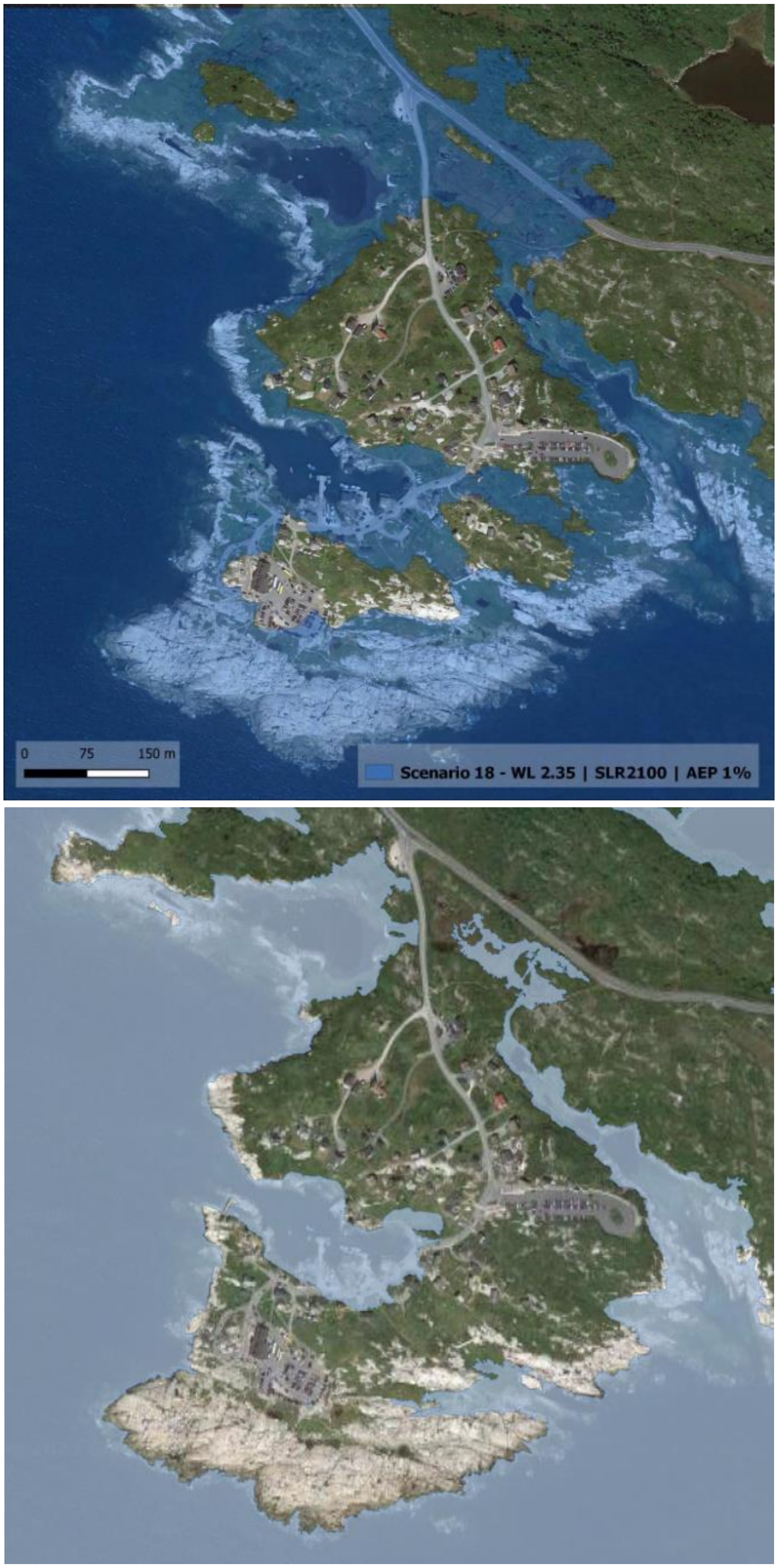

Figure 5-15: Comparison of flood extents developed for Peggy's Cove (100-year return period, current climate conditions). CBCL, 2020b (top, including wave run-up) and present study (bottom, static storm surge excluding wave run-up).

5.5.5.4 Hurricane Dorian at Halifax and Chezzetcook

The water levels during Hurricane Dorian (7 September 2019) were measured at Halifax BIO (Canadian Hydrographic Service, part of Fisheries and Oceans Canada – Tides Program) and in Big Lake, Lower East Chezzetcook (Taylor et al., 2021). Peak water level was 0.5 m higher at Big Lake (Figure 5-16). For comparison, the present study's modelling shows a water level difference of 0.4 m between Big Lake and Halifax BIO for a 20-year event, which is relatively consistent with the Hurricane Dorian observations. The two data points being far from each other gives additional confidence in the model results. This comparison provides a reasonable confirmation of the predictive capacity of the present study's hydrodynamic modelling.

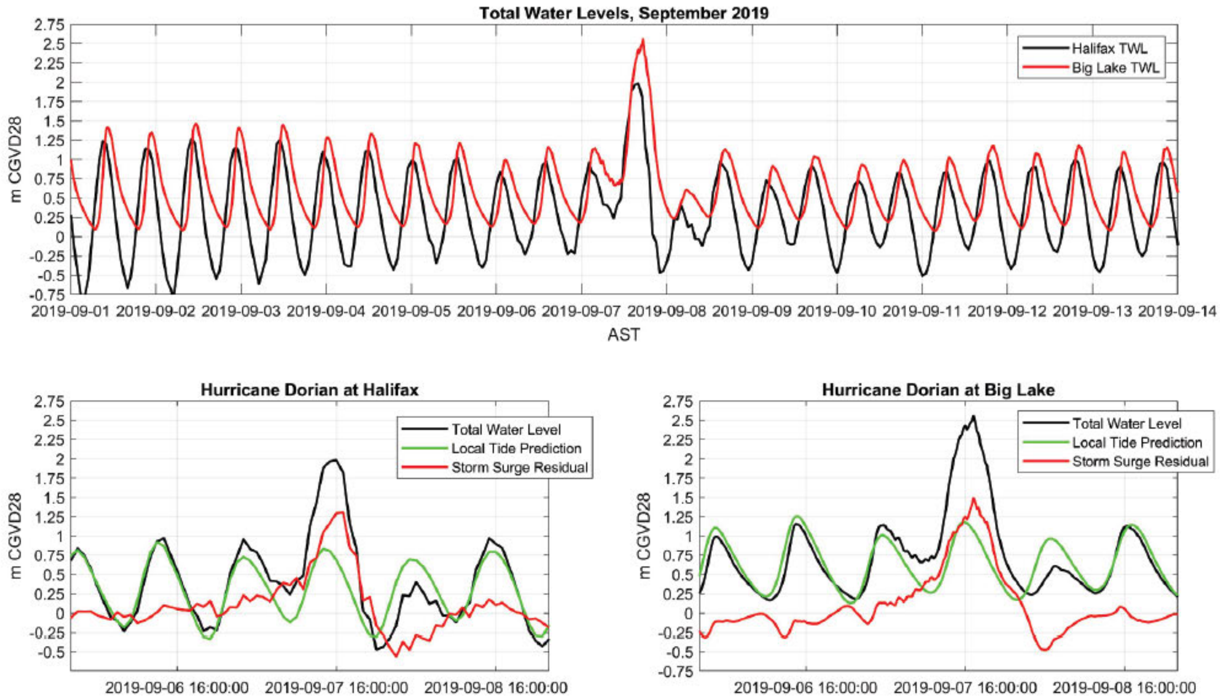

Figure 5-16: Measured water levels at Halifax and Big Lake (Taylor et al., 2021).

5.6 Total Extreme Coastal Water Levels

Total extreme coastal water levels (EWL) were estimated based on numerical modelling of local storm surge and calibrated to an extreme value analysis of water levels at Halifax (BIO long-term tide gauge). This includes the effects of tides, total storm surge and wave setup, and produces a spatially varying water level throughout the study area. Since SLR is expected to be mostly uniform throughout the region, a static or uniform SLR value for each future scenario is added.

The calculation of the extreme water levels is expressed by the following equation:

$$\begin{aligned} &Total\ Extreme\ Coastal\ Water\ Level\ (EWL) \\ &\quad = Modelled\ Extreme\ Water\ Level\ (9\ Return\ Periods) \\ &\quad + Sea\ Level\ Rise\ (2020, 2050, or\ 2100, RCP\ 8.5 median) \end{aligned}$$

Small-scale and/or location-specific effects such as wave run-up, overtopping, impoundments of culverts or bridges, or beach breaches are beyond the scope of this analysis and are therefore excluded. Variations in the wave exposure, topology, and geotechnical properties at each location may contribute significant additional flooding. Flood maps delineated based on the presented methodology can be used to identify areas where detailed assessments would be required based, for example, on levels of vulnerability and infrastructure at risk.

Unique EWL values are calculated in every model cell throughout the domain (with 100 m resolution at the coastline), which are then interpolated (using GIS) to a 10 m resolution grid and intersected with the 1 m resolution DEM for flood mapping purposes.
EWL values at six locations of interest are extracted and summarized in Table 5-7.

Table 5-7: Extreme Water Level Results at Six Locations of Interest

Extreme Water Level [m CGVD2013]						
Return Period [Years]	St. Margarets Bay	Halifax BIO	Musquodoboit Harbour	Jeddore Harbour	Ship Harbour	Sheet Harbour
Present Climate						
2	1.09	1.21	1.59	1.36	1.21	1.17
5	1.16	1.32	1.65	1.49	1.32	1.28
10	1.21	1.4	1.87	1.62	1.41	1.35
20	1.25	1.49	2.02	1.75	1.51	1.43
50	1.34	1.63	2.2	1.92	1.65	1.56
100	1.35	1.69	2.3	2.01	1.71	1.61
200	1.39	1.77	2.43	2.13	1.78	1.7
500	1.51	1.88	2.61	2.29	1.88	1.84
1000	1.58	1.96	2.75	2.4	1.94	1.94
Year 2050						
2	1.33	1.45	1.83	1.6	1.45	1.41
5	1.4	1.56	1.89	1.73	1.56	1.52
10	1.45	1.64	2.11	1.86	1.65	1.59
20	1.49	1.73	2.26	1.99	1.75	1.67
50	1.58	1.87	2.44	2.16	1.89	1.8
100	1.59	1.93	2.54	2.25	1.95	1.85
200	1.63	2.01	2.67	2.37	2.02	1.94
500	1.75	2.12	2.85	2.53	2.12	2.08
1000	1.82	2.2	2.99	2.64	2.18	2.18

Extreme Water Level [m CGVD2013]						
Return Period [Years]	St. Margarets Bay	Halifax BIO	Musquodoboit Harbour	Jeddore Harbour	Ship Harbour	Sheet Harbour
Year 2100						
2	1.85	1.97	2.35	2.12	1.97	1.93
5	1.92	2.08	2.41	2.25	2.08	2.04
10	1.97	2.16	2.63	2.38	2.17	2.11
20	2.01	2.25	2.78	2.51	2.27	2.19
50	2.1	2.39	2.96	2.68	2.41	2.32
100	2.11	2.45	3.06	2.77	2.47	2.37
200	2.15	2.53	3.19	2.89	2.54	2.46
500	2.27	2.64	3.37	3.05	2.64	2.6
1000	2.34	2.72	3.51	3.16	2.7	2.7
Coordinates [NS MTM5]						
Easting	25,546,600	25,570,255	25,609,528	25,615,632	25,628,284	25,655,294
Northing	4,946,900	4,949,533	4,961,020	4,960,801	4,964,878	4,976,958

5.7 Coastal Scope Exclusions

Coastal, meteorological, and oceanographic processes combine with complex local geometries and geologies to produce a complex water-coastline interaction. The present analysis accounts for the effects of tides, atmospheric surge, and wave setup to estimate space-varying extreme water levels throughout the project area (Figure 5-2).

This study is intended as a large-scale flood hazard mapping effort, and not a site-specific flood risk assessment. As such, the provided results allow the identification of areas under potential risk of flooding where the following site-specific processes should be included as they have been excluded from the present analysis.

5.7.1 Wave Run-up/Swash Uprush

The wave run-up (swash uprush) height is defined as the upland limit reached by individual waves during a storm. Wave run-up is a complex process with very high spatial variation. The run-up height depends on incident wave conditions (period, height) as well as shoreline type (rock, beach, marsh) and shoreline slope. Wave run-up can be a significant component of coastal flood risk for communities exposed to heavy wave action, such as Peggy's Cove, NS; or as recently witnessed during Hurricane Fiona, Port-aux-Basques NL. Wave run-up is not included in the flood lines.

5.7.2 Wave Overtopping

The present coastal analysis is centred on determining extreme static coastal water level heights. The complex, dynamic, and very geometry-dependent process of wave overtopping (i.e., volumes of water flowing over a coastal barrier, such as a dune or seawall, during a storm event due to the presence of waves) is excluded from the present study.

5.7.3 Morphological Changes

Coastal morphology within erodible settings such as beaches and dunes will undergo rapid changes in response to large storms, as well as gradual change in response to sea level rise. Breaching of a barrier beach, followed by modification of tidal passage, would be a typical example of such morphological changes which may alter the local extent and magnitude of future storm surges. The present analysis assumes a static coastline based on the DEM and does not consider the effects of morphological changes over time.

5.8 Summary of Key Findings

The HRM encompasses a long stretch of coastline where coastal flooding is a hazard. The key findings of the coastal analysis and modelling include:

- There is variability in the expected extreme water levels throughout the HRM. Long, narrow, shallow shore-perpendicular coastal inlets such as Musquodoboit Harbour or Petpeswick Inlet are expected to have significantly higher extreme water elevations than Inner Halifax Harbour, but areas of deep-water, open coastline, such as at Portuguese Cove, are expected to have somewhat lower extreme water levels than at Halifax. Extreme water levels in relatively deep coastal inlets such as Halifax Harbour, Sheet Harbour, or Ship Harbour are not expected to be as high as in shallower inlets. St. Margaret's Bay is expected to have somewhat lower extreme water levels than Halifax.
- The present study does not include flooding extent due to individual wave run-up. Comparison of the present results with previous studies that include the effect of wave run-up suggests that in areas of relatively low wave exposure (such as the Halifax waterfront) the present study produces generally similar results, however in areas of high wave exposure (such as at Peggy's Cove) there can be a significant underestimate of the flooding extent.
- The results of this study allow one to identify areas where more detailed study may be required to understand the impact of site-specific processes, based on the degree of vulnerability of the community at risk.

6 Flood Mapping

Pluvial, fluvial, and coastal flood lines have been prepared for all flood return periods under existing and future climate horizons as outlined in Table 1-1.

A geodatabase of all final mapping products including flood line maps will be provided as a digital geodatabase file with the final report.

The following sections describe the methodology implemented to delineate flooding risk throughout the municipality.

6.1 Methodology

Flood lines developed for coastlines and rivers using the multiple watershed and coastal models have been drawn separately to represent the conditions of the AEP (Annual Exceedance Probability) scenario for each of the riverine and coastal flood mechanisms.

6.1.1 Pluvial-Fluvial Flood Mapping

Turning modelling results into maps requires an automatic process that matches the calculated water levels for each scenario with the shape of the land. Even though this can be done directly within the modelling software, limitations of file size limit the scale of the flood maps they can produce and require the use of geographic information systems and interpolation tools to process the results efficiently. This section outlines the methodology that the assessment team followed throughout this process.

The PCSWMM software, although well suited for resolving the complex hydrology and hydraulics of the watersheds and capable of producing small scale flood maps, is stretched to the limits of its abilities with flood mapping at the scale encountered on this project. The need to produce model results of water levels that follow the undulating landscape, while respecting the sharp changes in topography, or impacts from flow-restrictive structures, mean that typical model methods cannot be scaled up, and high-resolution modelling needs to be maintained. This resulted in models that were of significant size and required significant computing power and time to produce results and extract results. To help support this challenge, CBCL developed a custom workflow with ArcGIS that used the model results from PCSWMM, assigned them to representative lines, and created a surface based on the lines. Using ArcGIS allowed greater computing capacity to generate flood maps at the scale required for this project. In addition, the PCSWMM tools for flood mapping were found through testing to be less representative of flood extents than the customized ArcGIS method.

Simulated water elevation results at model nodes were assigned to customized transect entities using ArcGIS. A triangulated irregular network (TIN) surface was created from the transect lines. Flood depth maps and flood lines were generated by intersecting the simulated flood water TIN surface onto the 1-m provincial LiDAR DEM to provide the outline of the flood areas.

The level of resolution of the model output used to prepare the flood maps is directly related to the spacing of transects (cross-sections), which are at 50 m intervals along watercourses. This means that flows and water levels are calculated by the model at intervals of at most 50m, and less where structures, confluences, or added detail in urban areas exist. While this is higher than average resolution for any flood mapping product, there will still remain some limitations related to sharp changes in topography.

Minor islands and pools less than 400 m^2 have been removed to eliminate noise from the flood line.

6.1.2 Coastal Flood Mapping

6.1.2.1 Coastal Flood Extent Polygon and Depth Raster Extraction

The coastal flood mapping begins with the extreme water level elevations modelled in MIKE21 (described in Chapter 5). These extreme water levels are interpolated to a TIN surface with 10m resolution using ArcGIS. The TIN surface is then intersected with the 1m resolution HRM LiDAR-derived DEM[10] from 2018. This intersection produces a flood extent polygon of all areas that are lower than the nearest coastal water level. Simply, all coastline that is lower than the nearby extreme water level is shown as flooded.

Within the flood extent polygon, depths are calculated as the difference between the DEM elevation and the modelled extreme water level surface. These values are exported as depth raster files. To reduce the size of these files, seaward areas with an elevation below -5 m CGVD2013 are removed from the depth raster files, since these areas are permanently under water.

6.1.2.2 Coastal Barriers and Culverts

There are many low areas that are separated from the ocean by strips of higher elevation coastal barriers such as coastal highways or dune systems. In these instances, the inland areas lower than the coastal extreme water levels are also potentially flooded. It is

[10] Note that the flood lines are extracted from the 2018 DEM. Changes to the topography since 2018 are not reflected in the analysis.

assumed that the inland area will flood to the same elevation as the coastal waters, i.e., the coastal flood waters will fully pass through culverts or associated infrastructure or natural features. Typically these low-lying areas contain small lakes or drainage ditches.

Generally speaking, the majority of low-lying areas behind coastal barriers are connected to the coast by culvert or channel and can be expected to flood during high water events. The decision was made to conservatively show all such low-lying areas as flooded.

Detailed local assessment may show that there is no possibility of coastal flooding of such low areas behind coastal barriers, due to e.g., a completely impermeable coastal barrier without culverts or channels. In such cases the present analysis will be an overestimation of the flooding extent.

6.1.2.3 Bridges

All bridges in the area will appear as flooded, and caution is required in the interpretation of bridge flooding. This is a consequence of the bridges being removed (or “burned out”) from the HRM DEM. Where a bridge stands, the DEM will generally show a watercourse, and not a high elevation barrier. Thus, when a flooding contour line or polygon is extracted from the DEM in the area of a bridge, a smooth result is obtained showing the water line *beneath* the bridge, and not of the bridge itself. This may cause confusion when inspecting the flood elevations with a satellite overlay, since all[11] bridges will appear flooded. Generally, if the flood extent at a bridge covers the bridge’s abutments and approach, it may be considered flooded.

6.1.2.4 Static and Dynamic Flood Effects

Regarding the coastal flood line mapping, it is important to note that the maps represent static water levels without the effect of wave run-up. During a storm event, other dynamic components will contribute to the extreme water levels observed. For example, wave run-up can significantly enhance the damage caused during the storm. Wave run-up at exposed coastline areas can be much greater than in sheltered areas. However, the results of this study allows HRM to identify areas where more detailed assessment may be required to understand the impact of site specific processes based on the degree of vulnerability of the community at risk.

[11] There are occasional bridges throughout the area that have not been removed from the HRM DEM. These are usually smaller bridges and are treated as culverts, i.e., the coastal flood waters are expected to fully flow through these bridges.

7 Uncertainty and Limitations

The present study uses current best practices for the hydrological and hydraulic modelling and coastal analysis to produce flood maps. As with any study that relies on historical, environmental, and climate data, there are various areas of uncertainty, which are outlined below:

- There is variability and randomness in the timing and magnitude of extreme coastal water levels, and peak river flow rates that could align to produce flood levels. This study uses the approach of combining 1% AEP (1 in 100 year) for one mechanism (e.g. rainfall) with a 50% AEP (1 in 2 year) for another concurrent mechanism (e.g. storm surge), which is a reasonable approach, and is also used in other jurisdictions, such as Ontario.
- There is a high level of uncertainty related to future rainfall and sea-level rise projections under climate change. There is uncertainty in both climate change emission scenarios and in climate change models due to the evolving scientific understanding of Earth's systems and their interactions, natural variability in the climate system, and the limitations of climate models.
- The calibration of the pluvial and fluvial systems is based on reproducing extreme historical flood events by adjusting the various hydrologic and hydraulic parameters. However, flooding from any given storm event is highly dependent on the pre-storm (antecedent) conditions. Antecedent conditions include seasonal changes, amount of vegetation, snow on the ground, snowmelt, wind conditions, and soil saturation and permeability (changes with temperature, wetness, and dryness). Even with robust calibration, simulated peak flow rate uncertainty is generally considered to be ± 35%. The additional factors listed above and below further increase this uncertainty.
- Though the pluvial and fluvial systems are calibrated on the best available data for rainfall and flow measurements, this data is sparse and limited. In several instances, the most representative flow event occurred in 1971 (Hurricane Beth), at which time hourly rainfall data was only available at the Shearwater Airport climate station, and daily rainfall data was only available at the Ecum Secum climate station. Similarly, there are only 5 flow monitoring stations in the study area that had representative data that was suitable for calibration. Consequently, many watersheds are calibrated to flow or rainfall data that can be some distance away and may not be representative.
- The potential exists for the design rainfall event to occur on entirely frozen ground, which could occur in late fall or early winter, and is likely to result in higher runoff rates.
- In portions of low gradient and deep slackwater channels (i.e. interconnected wetland – watercourses reaches), the channel bathymetry and its flow capacity below the LiDAR

surface is uncertain. This study assumed a conservative channel bed or baseline water level based on the LiDAR surface. This approach is considered reasonable as the downstream grade control that maintains the normal water level will still be present during flood conditions.
- The operation of several dams and reservoirs could affect storage and release rates during flood events.
- The pluvial and fluvial modelling has been done using industry standard 1D hydraulic simulation. There may be areas with complex flow patterns that would be better represented using a 2D hydraulic model.
- The pluvial and fluvial models are calibrated on rainfall and flow events that are in the 50-year and 100-year range of return periods. Results for return periods far from these values (notably the 1 in 2 year and 1 in 500 year and above) are less representative of hydrologic and hydraulic processes that would exist during such events.
- Culverts and bridge structures have been added based on the best available GIS and desktop level data at the time of the study. Structure geometry (i.e. sizes) and invert elevations have not been surveyed in the field. Geometry and elevations have been inferred from DEM and Satellite imagery to fit within the context of the nearby reach and road embankments. As such, assumptions of culvert and bridge sizes could be different from the actual sizes, and the flood mapping would be correspondingly impacted.
- The HRM DEM did not include representative water level data in most areas with waterbodies. As such, assumptions needed to be made to provide some elevation data in those areas in the models. Other areas included some inconsistent data. The discrepancies included gaps (or holes) in the DEM, inconsistent elevations within water bodies, and landform dips that did not reflect reality. CBCL carried out post-processing where these discrepancies appeared, to interpolate elevations and provide topographical support to allow the model tools (watershed delineation, delineating watercourses, setting watercourse elevations) to run. While the DEM had suitable elevations within certain lake areas, there were inconsistencies in many areas that created issues for the model tools. We initially used an automated tool to address the discrepancies in the DEM, but this was not always successful and sometimes provided unexpected results, and we found it necessary to manually check the work done by the tool. Since the data had to be manually adjusted to allow the model tools to run, the actual water level at the time of the LiDAR survey is not known and this means that the water levels used as model input may not be representative of actual water levels at the time of survey. Best efforts were made to fill the gaps and address the inconsistencies, but since this was not within the original scope of the project, the data was only adjusted to support the modelling. In addition, since in those instances the actual water level was not known, assumptions were made about the water level at the time of the LiDAR survey, which may not reflect the actual field conditions. Therefore, any difference between assumed water levels and actual water levels would have corresponding effects on the flood mapping results. The HRM DEM was not corrected using the provincial DEM data, since the water levels are not necessarily consistent between the two DEMs (where the LiDAR was

flown at different times). Introducing provincial DEM data in lakes would have resulted in additional uncertainties.
- Another source of uncertainty relates to the fact that this assessment is focused on rainfall as a main flooding mechanism. There are other flood mechanisms, such as snowmelt, ice accretion, ice jams, debris accumulation, and debris jams. It is estimated, and supported by HRM staff experience, that ice and debris are rarely a flood mechanism. There are also main sources of uncertainty unaccounted for in this assessment which include antecedent conditions prior to a flood event, the likelihood of rain on snow and rain on frozen ground, whether some very large watersheds are more susceptible to rainfall events longer than 24 hours, and the joint probability of storm surge and extreme rainfall, which are all the subject of ongoing research in Nova Scotia. At this point, the recommendation is to follow ongoing research in the province and implement recommended approaches from the results of the research.
- Coastal flooding from wave run-up is not included in the analysis and could have significant impact in areas exposed to extreme waves.
- Coastal erosion is not included in the scope of this project. In the future, accelerated erosion could lead to flooding of additional areas.

For additional context on the exclusions and limitation of the coastal flood mapping, please see Section 5.7.

This report has been prepared based on a specific scope of work, and it should be read in its entirety. The findings and recommendations are based on information collected to date at the time of writing, and on simplified mathematical formulations of complex dynamic natural processes. While the modeling effort incorporated as much relevant data as possible within the study schedule and budget, uncertainties associated with data gaps and modeling approximations are inherent to this type of study. Results should be interpreted with caution and actual conditions encountered in the future may vary significantly from the estimates presented in this study. We recommend that results be revisited by HRM as new information becomes available.

While the flood line maps represent scenarios of combined pluvial – fluvial and coastal flooding, there are additional flood mechanisms not included here that could worsen flood hazards.

These include:
1. Wave run-up and overtopping.
2. Riverine ice jamming.
3. Debris jamming.
4. Rain with snowmelt.
5. Longer or different types of rainfall events.
6. Specific seasonal conditions that were not included in the present study.

We recommend that further study be carried out to assess the relative importance of these factors to local flooding.

The findings and recommendations of this study are based on available information at the time of completion, with uncertainties associated with data gaps, notably temporal and spatial gaps in rainfall, water level and flow gauge information, LiDAR data, as well as modelling approximations inherent to this type of study. We recommend that the flood maps be reviewed carefully by HRM to ensure that they are consistent with local knowledge and that more detailed analysis is conducted to reduce uncertainty in areas of interest.

8 Recommendations

The intent of the flood mapping prepared in this assessment and described in this report is to provide HRM with a tool to support the identification of infrastructure at risk, emergency management operations, and long-term decision making.

It is noted that the following recommendations apply to improving the quality of the flood mapping results, and how representative they are of actual risks of flood extents. They do not relate to addressing the flood risks identified in the mapping, or the next steps in the plan that HRM is following with respect to overall flood resilience in the municipality. CBCL recommends the following actions and next steps:

- Conduct a comprehensive survey of all structures (bridges, culverts, dams) in HRM to obtain their precise geometry and update the model accordingly.
- Review the LiDAR data and close the data gaps in a manner that provides data in waterbodies that is representative of the water level at the time of survey.
- Identify and prioritize high risk areas for more detailed analysis.
- Conduct regular public and First Nation consultations with regards to collecting data related to flood risk in the HRM, as well as to confirm and/or adjust the flood mapping over time.
- Investigate the role of storm sewer (sub-surface) drainage systems on urban flooding.
- Investigate the role and importance of secondary flood mechanisms including coastal wave run-up and overtopping, ice jamming, debris jamming, dam operations, seasonal conditions, and rain with snowmelt.
- Develop a long term hydrometric (flow and water level) monitoring plan for watercourses with high flood risk.
- Conduct regular model updates, refinements, and calibration for both flows and water levels, especially in places which may be considered in future decisions for improved resilience.

In terms of using the flood mapping prepared in this assessment, it is recommended that caution be exerted and that the general high-level nature of the maps be kept in mind when evaluating decisions to improve resilience. This tool represents a first step in the construction of an overall model and should be used as a general guide to identify areas that will need further refinement and study. They are not to be used to identify risks of flooding for specific properties or singular infrastructure items.

With regards to the release of the flood maps in a publicly accessible context, it is recommended that:

- A disclaimer be presented prior to accessing the maps that note the general nature of the maps, the purpose of the maps, i.e. that they were prepared as a tool to support HRM with the identification of infrastructure at risk, emergency management operations, and long term decision making, and not as a means to identify risks of flooding for specific properties or singular infrastructure items.
- The maps be accessible up to a limited scale, such that individual properties are not identifiable as being at risk of flooding or not.
- A separate document be released to the public prior to the release of the maps, explaining to the public the overall HRM program (of which this project is just one component) for improving long term public safety and infrastructure resiliency in the context of a changing climate. The document should describe the fact that the present assessment represents the first step in a number of steps to improve resilience, noting that the present report generally identifies areas that may benefit from further study and model refinement, that the next step involves the collection of survey data for an extensive amount of infrastructure within HRM, followed by further model improvements. The process is intended to be a long term, continuously improving decision support tool to identify areas which are most in need of intervention, and assist a long term, resilient, planning process.

9 Conclusion

CBCL has completed an assessment of pluvial, fluvial, and coastal flooding within the HRM. Industry standard best practices were followed for the hydrological, hydraulic, and coastal modelling and analysis. The analysis has been based on the available data at the time of study which includes numerous desktop type data sources as well as water level monitoring data from coastal and watercourse sites. This project has produced an extensive set of flood mapping covering approximately 10,000 km of watercourse and waterbody length as well as over 300 km of coastline.

Flood mapping includes defined flood probabilities ranging from the 2-year to the 200-year event for existing climate and the future 2050 and 2100 time horizons, and additionally the 500-year and 1000-year events for the 2100 time horizon. This range of flood scenarios, presented in Table 1-1, provides an extensive dataset to quantify flood hazards across HRM. The flood mapping provides critical inputs for land use planning, infrastructure and asset management, emergency management, and flood mitigation planning.

The assessment was able to identify general flood risk zones, within the limitations of the supporting data and general modelling detail and assumptions. The climate change assessment and definition of scenarios allowed the evaluation of risks associated with the uncertainty of a changing climate.

The modelling shows that urban areas are especially prone to flash flooding, having a fast response to rainfall, with the Halifax peninsula being vulnerable to very short and intense rainfall events. Other areas in HRM will reach peak flows under longer, more continuous rainfall events, reaching several hours in duration, possibly more than 24 hours in some cases.

Key technical challenges included the vast size of the study area and the length of watercourse to flood map. This is the largest flood mapping effort conducted to date in Nova Scotia and is vastly larger than any previous flood mapping effort in the HRM. Consequently, the project scope did not include any survey work, was based on existing data only, and relied heavily on automated methods for hydrologic and hydraulic modelling. To reduce the need for detailed manual adjustments throughout the study area, the level of detail of the modelling was increased throughout the study area (to 50m cross-section spacing), so that the topography was better reflected in the model.

The present study represents the first step in a suite of tools HRM is working on to identify climate-driven flooding hazards and to improve resiliency. Following steps include the preparation of public-oriented information documents to introduce the overall goals and intermediate steps to the public, followed by further survey, and refined modelling of areas identified as being vulnerable. The project scope specifically excludes the possibility that it will be used to inform decisions at the Asset Design level, and the project output is not intended to replace the need for detailed flood modelling specific to assets or locations for design projects. The present report is intended to be a technical document that describes the engineering methodology that was followed to assemble and analyse the existing data, develop and calibrate hydrologic, hydraulic and coastal models, and prepare high level flood maps for a wide range of flood frequencies and climate scenarios.

10 Closure

CBCL wishes to thank the Halifax Regional Municipality for the opportunity to work on the Creation of Flood Hazard Maps for the municipality. Proactive flood mapping is an important tool for the protection of public safety and planning future growth in communities. The present study provides a set of consistent and detailed flood lines across pluvial, fluvial, and coastal areas to aid in planning purposes for the HRM.

Yours very truly,

Prepared by:

[redacted]
Water Resources Engineer

Tom Kozlowski, M.A.Sc., P.Eng.
Coastal Engineer

Léa Braschi, M.Sc.
Climate and Water Resources Scientist

Reviewed by:

Alexander Wilson, M.Eng., P.Eng.
Senior Reviewer & Technical Specialist, Water Resources

Amaury Camarena, P.Eng., M.Sc.
Group Lead – Coastal Engineer

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APPENDIX A

Water Level Monitoring

This section contains additional details of the field instrumentation and water level gauge deployment program initially discussed in Section 2.2. For a map of the deployment locations, see Figure 2-1.

A.1 Bissett Creek (Cole Harbour)

At this location along the inland watercourse of Bisset Creek downstream of Colby Drive, a Solinst Levelogger 5 pressure transducer was installed[12]. The instrument was secured within a length of PVC pipe secured to a length of steel rebar that was driven into the streambed. The instrument and water level were then surveyed to establish a measurement within the CGVD2013 vertical datum. The Levelogger was connected via cable to a Solinst Levelsender telemetry unit which was fastened with plastic tie-straps to a nearby tree (Figure 11-1). The collected data is shown in Figure 11-2.

Figure 11-1: Field installation of the Solinst Levelsender telemetry unit at Bisset Creek, fastened to the tree with plastic tie straps. The Levelogger unit is deployed within the adjacent watercourse.

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[12] The expected accuracy of the water level measurements is +/-8 mm according to manufacturer's specification sheet. An additional 2 cm to 3 cm of error is expected based on the precision of survey equipment.

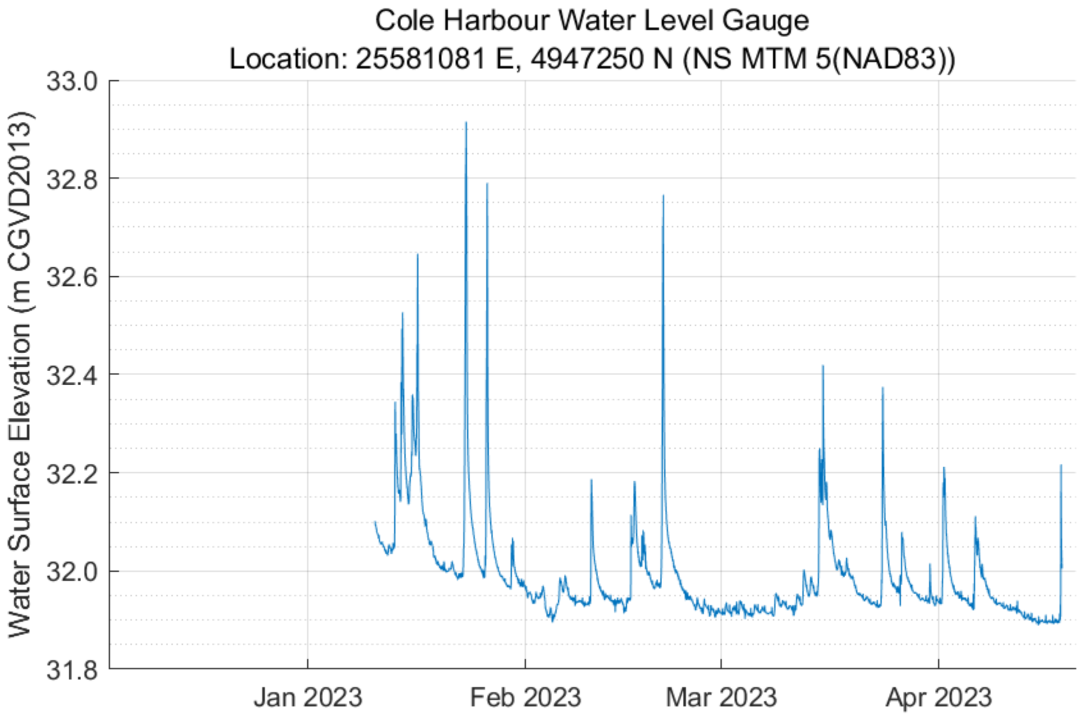


Figure 11-2: Water surface elevation measured at the Bisset Creek (Cole Harbour) location.

A.2 East River (Tantallon)

At this location along the East River in Tantallon downstream of St. Margaret's Bay Road, a Solinst Levelogger 5 pressure transducer was installed[13]. The instrument was secured within a length of PVC pipe and deployed on the streambed using a steel weight since the streambed was too rocky to allow for the driving of a length of steel rebar. The instrument and water level were surveyed to establish a measurement within the CGVD2013 vertical datum. The Levelogger was connected via cable to a Solinst Levelsender telemetry unit which was fastened with plastic tie-straps to a nearby tree (Figure 11-3 and Figure 11-4). The collected data is shown in Figure 11-5.

[13] The expected accuracy of the water level measurements is +/-8 mm according to manufacturer's specification sheet. An additional 2 cm to 3 cm of error is expected based on the precision of survey equipment.


Figure 11-3: Field installation of the Solinst Levelsender telemetry unit at East River (Tantallon), fastened to the tree with plastic tie straps. The Levelogger unit is deployed within the adjacent watercourse.


Figure 11-4: The East River upstream of the instrument, facing St. Margaret's Bay Road.

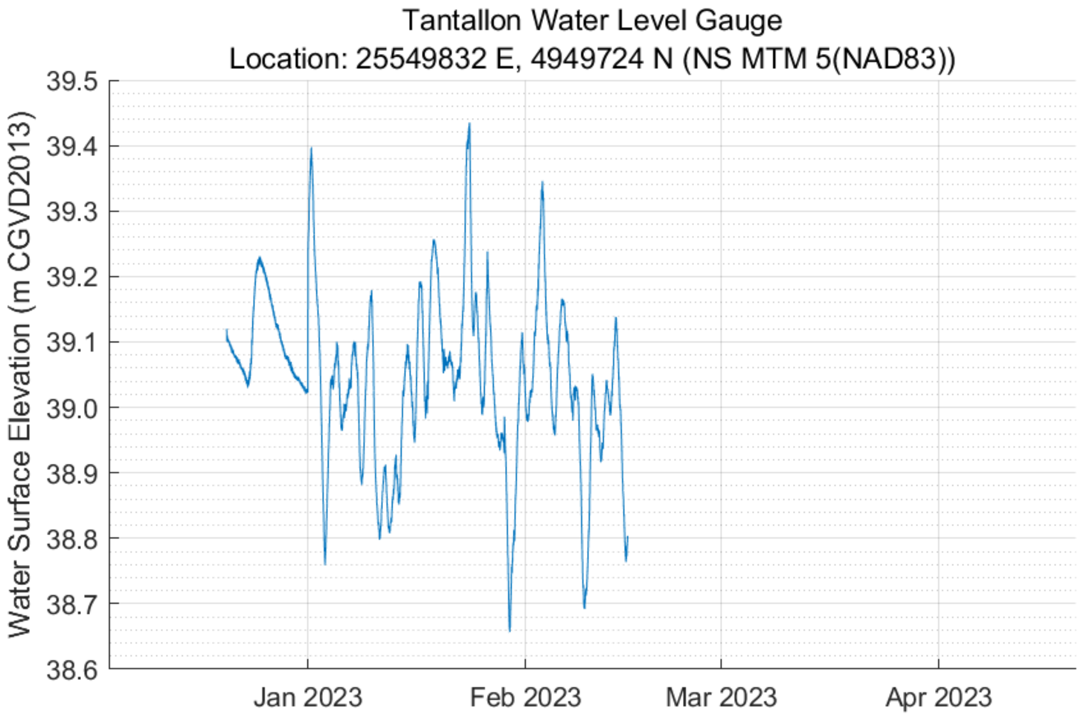


Figure 11-5: Water surface elevation measured at the East River (Tantallon) location.

A.3 West Brook at Myra's Road (Porter's Lake)

At this location along the West Brook watercourse in Porter's Lake at Myra's Road, a Solinst Levelogger 5 pressure transducer was installed[14]. The instrument was secured within a length of PVC pipe and deployed on the streambed using a steel weight since the streambed was too rocky to allow for the driving of a length of steel rebar. Shallow conditions at the site necessitated that the instrument be installed in a shallow pool. The instrument and water level were measured relative to the Myra's Road bridge to establish a measurement within the CGVD2013 vertical datum, which was referenced with the NS 2018 LiDAR survey. A survey using RTK GPS equipment was not possible at this location due to tree cover and the lack of cell network coverage. This lack of cell network coverage also determined the type of instrument which was possible to deploy at this location. At the outset of the project, it was planned for a ToltHawk device to be deployed at this location, but since both the ToltHawk and Levelsender devices require cell network coverage, a Solinst Levelogger 5 was deployed at this location without a telemetry component. Data was manually downloaded during the field program. At the final visit in April 2023, it was

[14] The expected accuracy of the water level measurements is +/-8 mm according to manufacturer's specification sheet. An additional 15 cm of uncertainty in vertical accuracy is expected based on the precision of HRM LiDAR.

discovered that the Levelogger was missing, presumed stolen. Field installation location is shown in Figure 11-6. Collected data is shown in Figure 11-7.

Figure 11-6: West Brook watercourse at Myra's Road. Instrument installed in shallow pool on bottom left of photo.

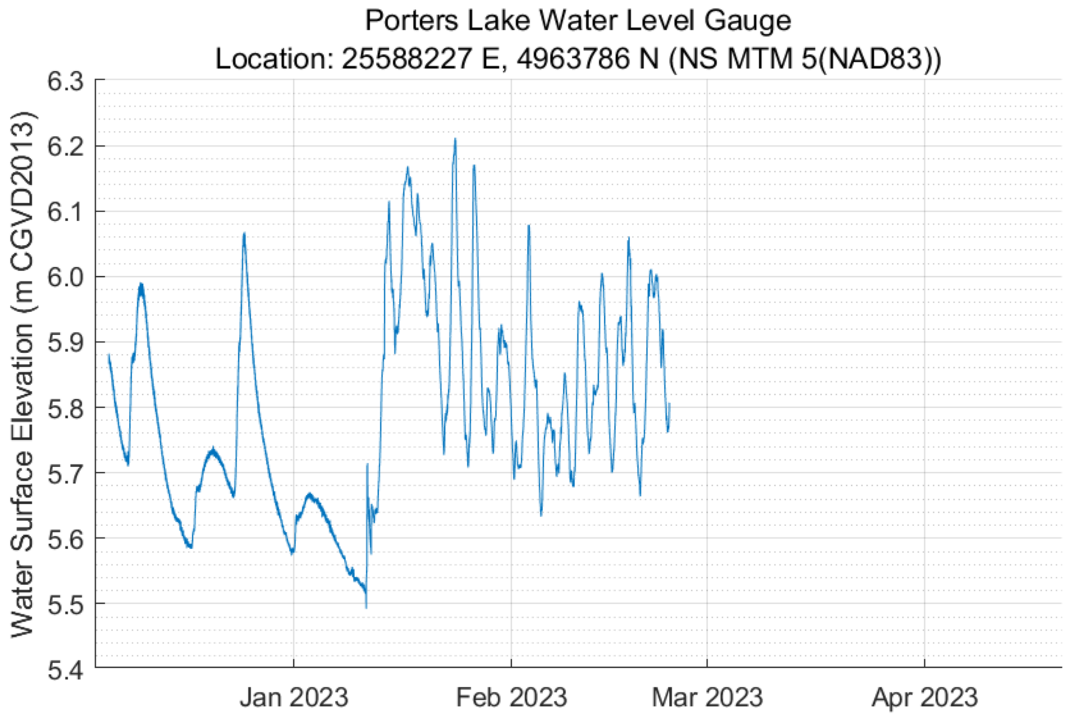


Figure 11-7: Water surface elevation measured at the West Brook (Porters Lake) location.

A.4 Sheet Harbour at West Side Road

At this location within Sheet Harbour at West Side Road, an ultrasonic ToltHawk sensor was deployed on a small bridge at an inlet along the western coast of Sheet Harbour[15]. The instrument was secured to a length of zinc-plated L-profile slotted angle using a rubber fitting. The slotted angle was then secured to the West Side Road bridge non-destructively using two ratchet straps. For security, a padlock was installed in the ratchet straps and they were covered with dark-coloured tape. The selected deployment location is above a set of small rapids at the end of a small river where the water level is controlled from the sea during everything but low tide, when the water level is controlled from the river (Figure 11-8 and Figure 11-9). The instrument's sensor was surveyed into the CGVD2013 datum for correction of the measured water levels. The unit was connected via cell network and delivered data upload in near real-time. The collected data is shown in Figure 11-10.

[15] The expected accuracy of the instrumentation is +/- 1 cm according to manufacturer's specification sheet (ToltHawk). An additional 2 cm to 3 cm of uncertainty is expected based on the precision of survey equipment.

Figure 11-8: Deployment location at West Side Road bridge, facing towards Sheet Harbour. The instrument can be seen attached to the central guardrail post.

Figure 11-9: Deployment location at West Side Road bridge. The ToltHawk instrument itself can be seen at the bottom of the guardrail post.

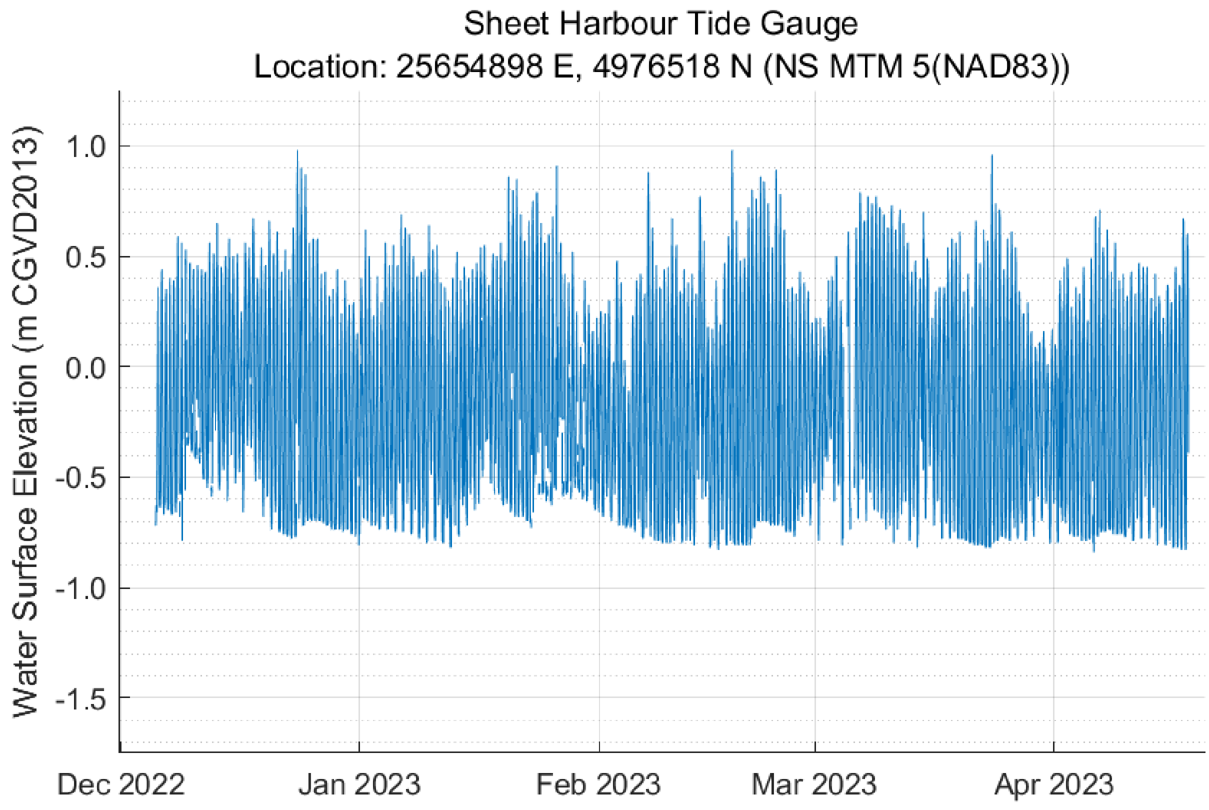


Figure 11-10: Water surface elevation measured at the Sheet Harbour location.

A.5 Jeddore Harbour at Marine Drive

At this location within Jeddore Harbour at Marine Drive, an ultrasonic ToltHawk sensor was deployed on the Salmon River Bridge at an inlet along the northern reach of Jeddore Harbour[16]. The instrument was secured to a length of zinc-plated L-profile slotted angle using a rubber fitting. The slotted angle was then secured to the Salmon River Bridge non-destructively using two ratchet straps. For security, a padlock was installed in the ratchet straps, and they were covered with dark-coloured tape. This instrument was deployed as originally planned. The deployment location is above a gap in a causeway crossing a northern arm of Jeddore Harbour. The water depth at this location at the time of deployment was in excess of 2 m. See the deployment location in Figure 11-11 and Figure 11-12. The instrument's sensor was surveyed into the CGVD2013 datum for correction of the measured water levels. The unit was connected via cell network and delivered water level data upload in near real-time. The collected data is shown in Figure 11-13.

[16] The expected accuracy of the instrumentation is +/- 1 cm according to manufacturer's specification sheet (ToltHawk). An additional 2 cm to 3 cm of uncertainty is expected based on the precision of survey equipment.

Figure 11-11: The Salmon River Bridge deployment site.

Figure 11-12: The deployed instrument, seen hanging below a guardrail post above Jeddore Harbour.

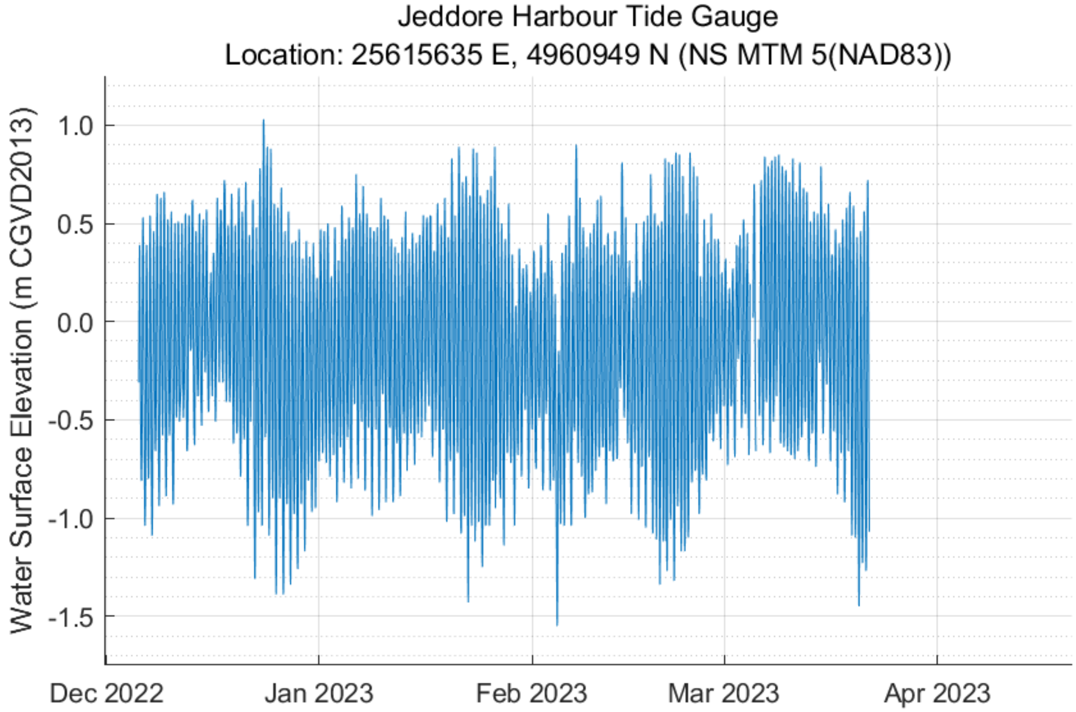


Figure 11-13: Water surface elevation measured at the Jeddore Harbour location.

A.6 St. Margaret's Bay (Shining Waters Marina)

An ultrasonic ToltHawk sensor was deployed at the end of the pier from an elevated post[17]. The sensor height was raised following the December 23, 2022 storm when it was discovered that water levels rose too close to the sensor to be recorded. The sensor requires a minimum distance to properly send and receive ultrasonic signals.

At this location within Saint Margaret's Bay at Shining Waters Marina, an ultrasonic ToltHawk sensor was deployed on a post at the end of a wharf. The instrument was secured to a length of zinc-plated L-profile slotted angle using a rubber fitting. Since the attachment location required the offsetting of the instrument from the wharf to ensure a clear line-of-site to the water surface, a square frame was made from the slotted angle. The slotted angle was then secured to the post using wood screws.

This instrument was deployed as originally planned. The water depth at this location at the time of deployments was approximately 2 m. See the deployment location in Figure 11-14.

[17] The expected accuracy of the instrumentation is +/- 1 cm according to manufacturer's specification sheet (ToltHawk). An additional 2 cm to 3 cm of uncertainty is expected based on the precision of survey equipment. The manufacturer's stated pressure resolution for the RBR device is +/- 0.05%.

The ToltHawk instrument's sensor was surveyed into the CGVD2013 datum for correction of the measured water levels. The unit was connected via cell network and delivered water level data upload in near real-time. After functioning properly for 30 days (Dec 20, 2022 to Jan 20, 2023), the device began to malfunction (unexpected loss of instrument connectivity). Several attempts at repair and reinstallation were attempted during the next several weeks (during which time data was lost). The instrument was ultimately replaced on Feb 16, 2023 with an RBR Solo[3] wave gauge functioning in tide gauge mode which captured data throughout the remainder of the deployment period. This instrument was anchored to the seabed with concrete blocks and water surface elevations were surveyed in the CGVD2013 vertical datum. The collected data is shown in Figure 11-15.

Figure 11-14: St. Margaret's Bay ToltHawk and RBR instrument deployment site.

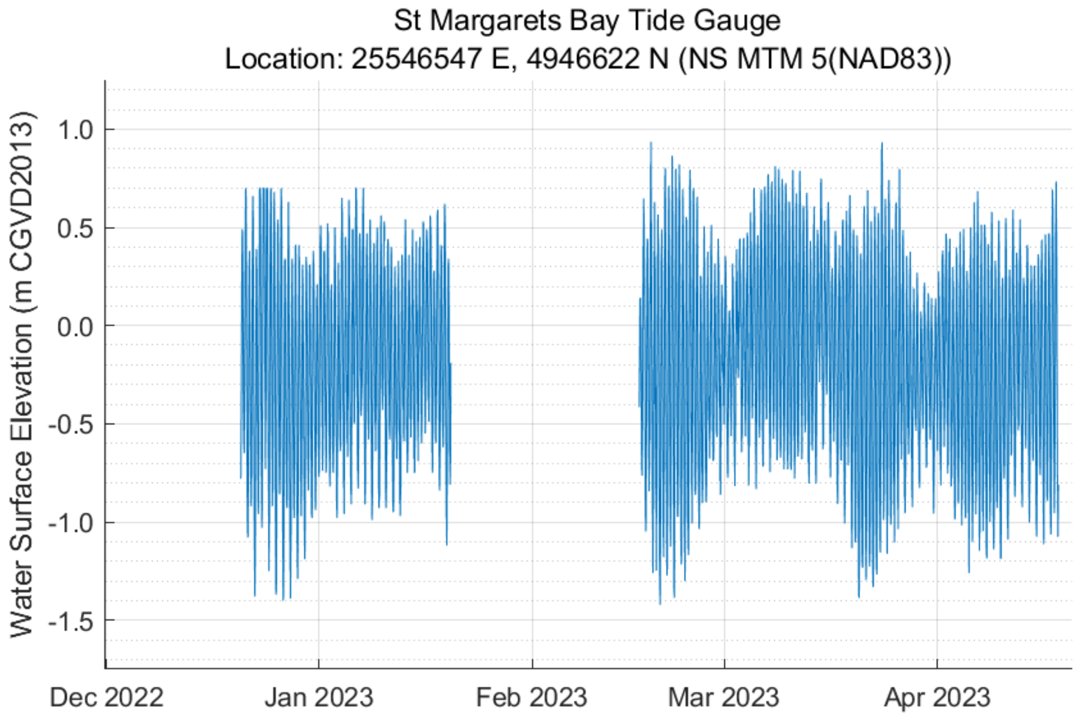
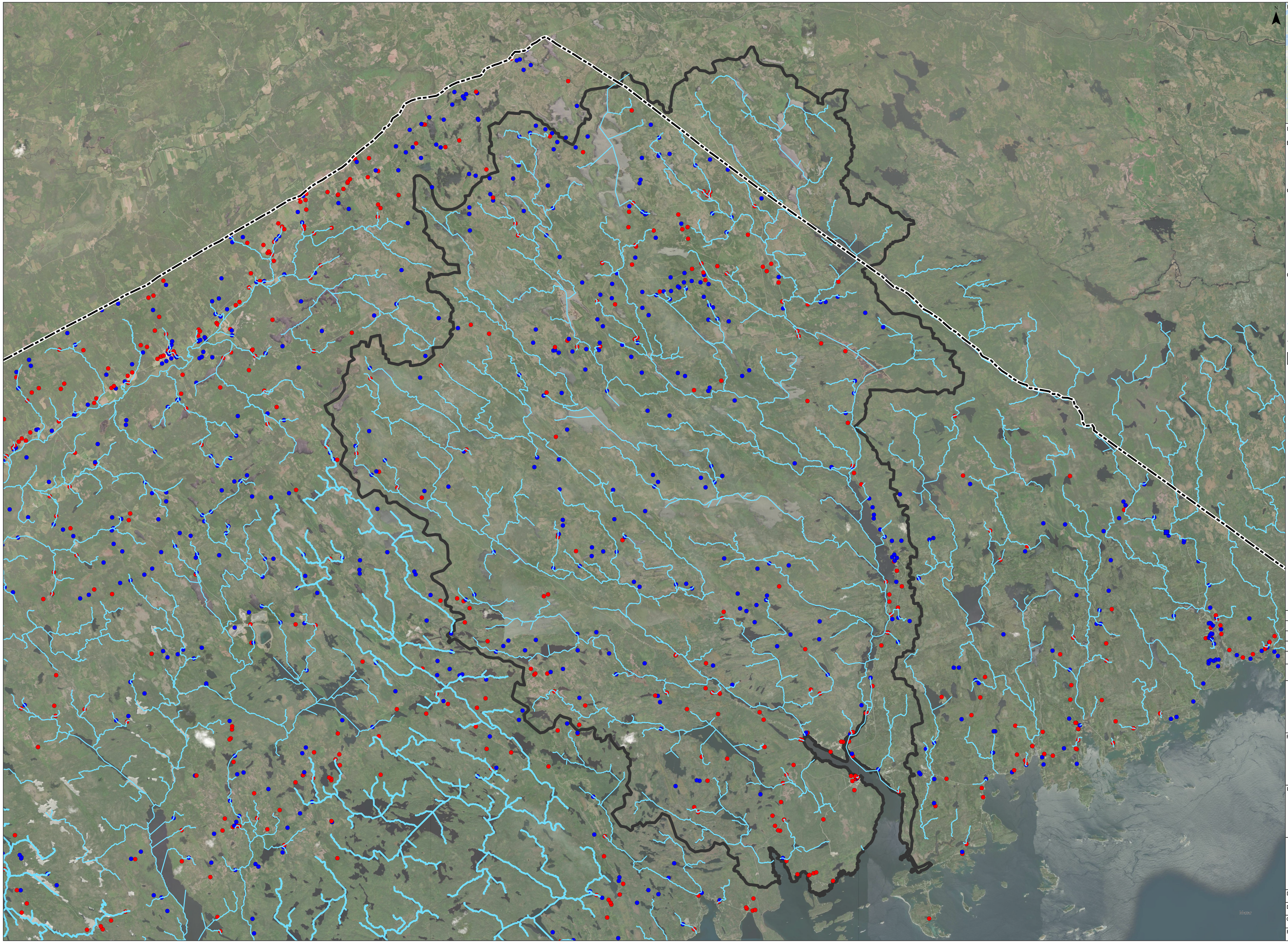


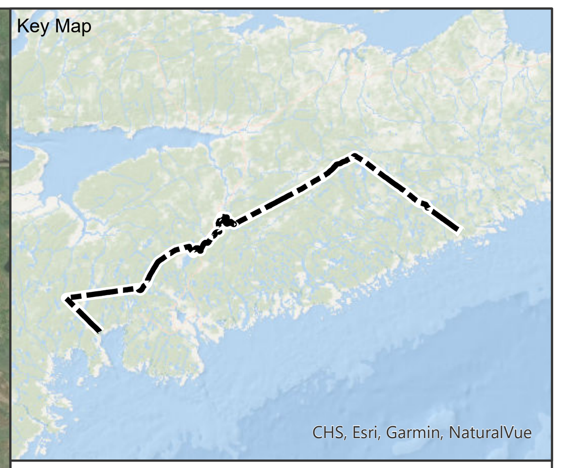
Figure 11-15: Water surface elevation measured at the Saint Margaret's Bay location. ToltHawk measurements are shown from Dec 20, 2022 to Jan 20, 2022. RBR tide gauge measurements are shown from Feb 16, 2023 to April 18, 2023.

APPENDIX B

Maps Showing the Data Available on Structures

(Mapping is Provided Separately)





Legend

Bridge Points

- Span Length Only
- No Span Length or Soffit Elevation

Culvert Points

- Length and Width
- Width Only
- Length Only
- No Length, Width or Height

Culvert Lines

- With Z value
- Without Z value
- Model Conduits
- HRM Boundary
- Major Watershed

No	Date	Description
2	APR. 5, 2024	Issued for Final Report
1	NOV. 24, 2023	Issued for Final Draft Report
A	MAR 29, 2023	Issued for Information

Revision or Issue

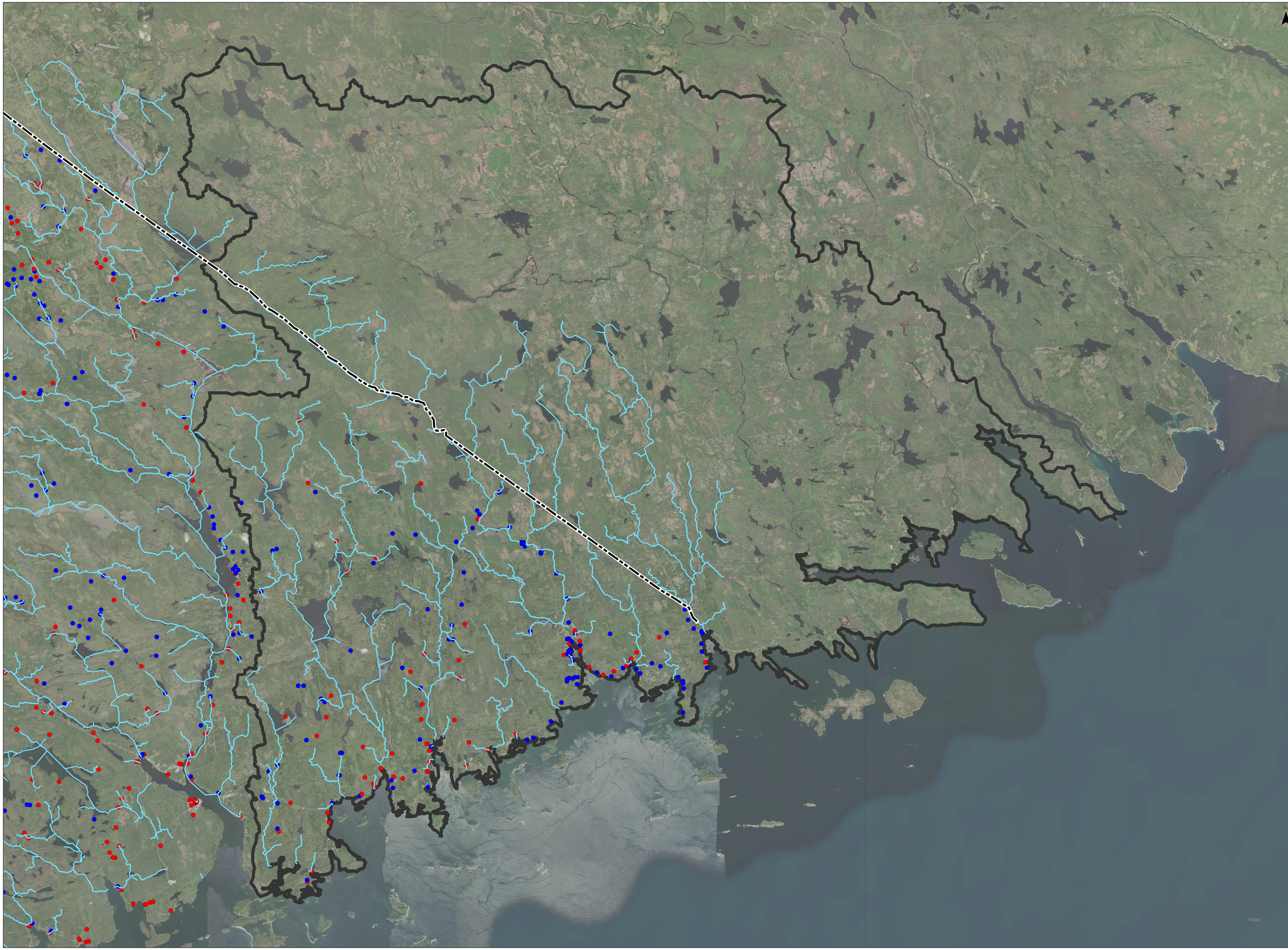
Project:

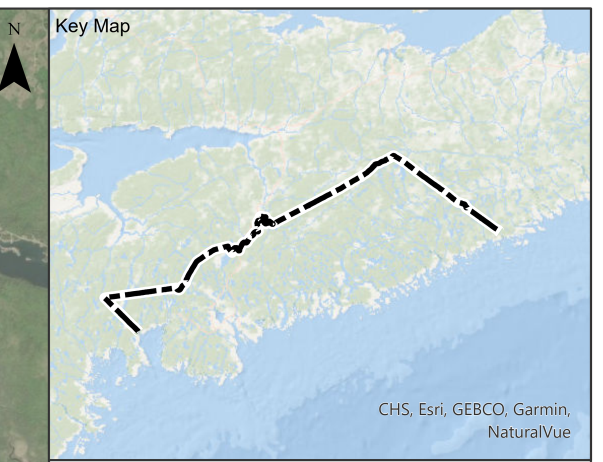
Creation of Pluvial, Fluvial and Coastal Flood Hazard Maps

Drawing Title:

Maps of available information on culvert and bridge structures. Watershed: EAST/WEST (SHEET HBR)

Date:	APR. 2024	Scale:	1:89,000
Drawn:	CM	Designed:	ATW
Checked:	---	Approved:	---
Project No:	221111.00	Drawing No:	1.0





Legend

Bridge Points

- Span Length Only
- No Span Length or Soffit Elevation

Culvert Points

- Length and Width
- Width Only
- Length Only
- No Length, Width or Height

Culvert Lines

- With Z value
- Without Z value
- Model Conduits
- HRM Boundary
- Major Watershed

No	Date	Description
2	APR. 5, 2024	Issued for Final Report
1	NOV. 24, 2023	Issued for Final Draft Report
A	MAR 29, 2023	Issued for Information

Revision or Issue

HALIFAX

Project:

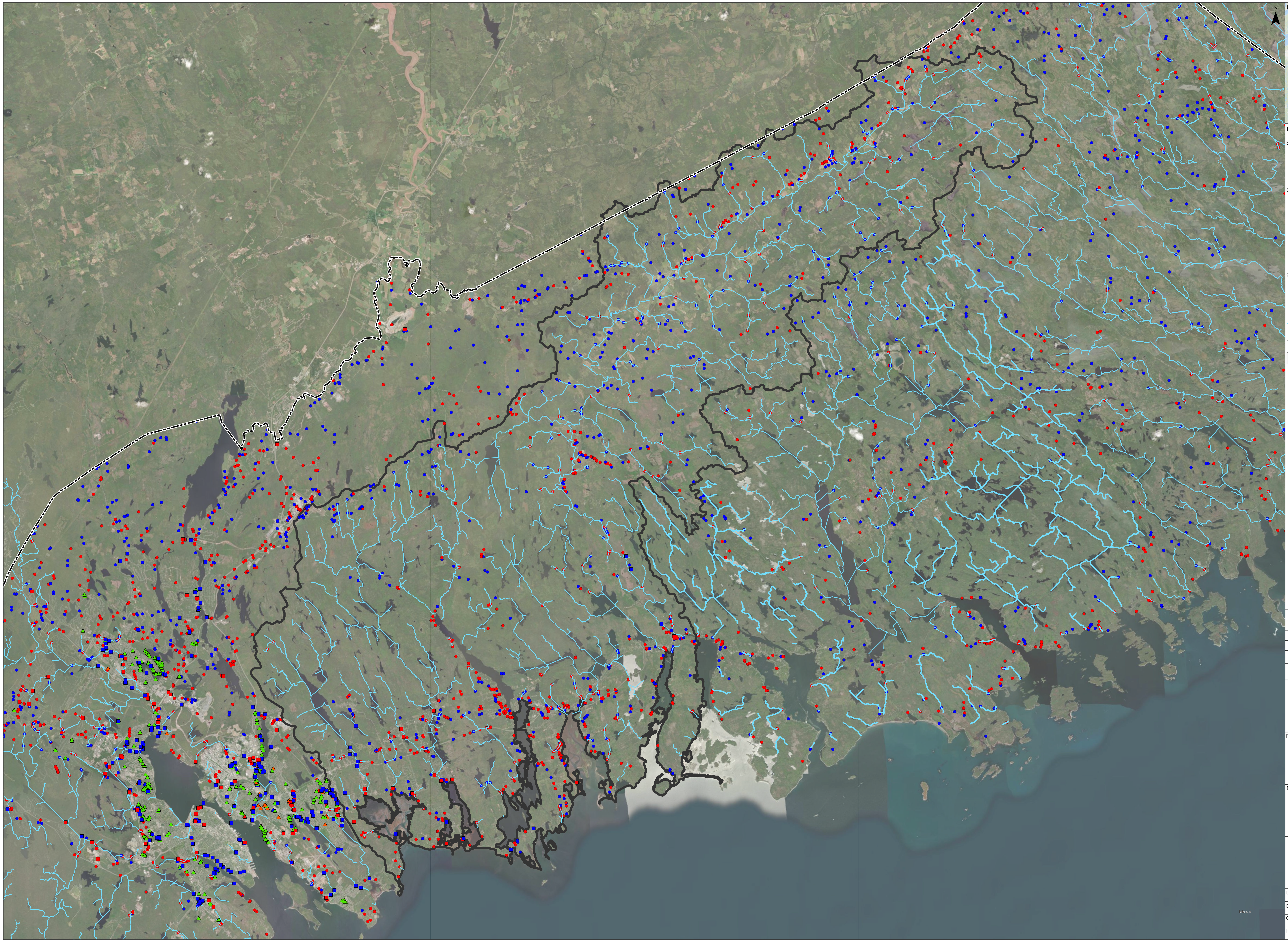
Creation of Pluvial, Fluvial and Coastal Flood Hazard Maps

Drawing Title:

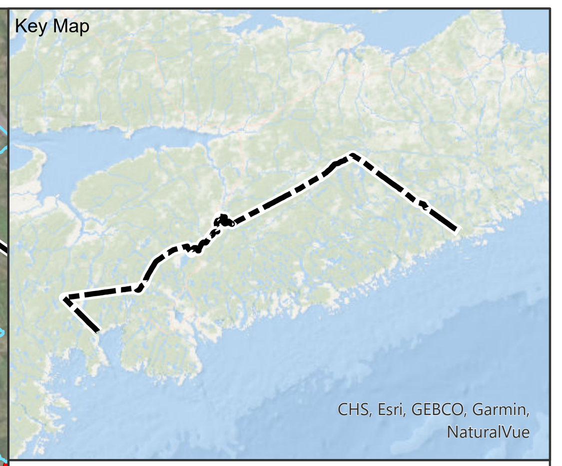
Maps of available information on culvert and bridge structures.
Watershed:
LISCOMB

CBCL

Date:	APR. 2024	Scale:	1:85,000
Drawn:	CM	Designed:	ATW
Checked:	---	Approved:	---
Project No:	221111.00	Drawing No:	1.0



Key Map



Legend

Bridge Points

- Span Length Only
- No Span Length or Soffit Elevation

Culvert Points

- Length and Width
- Width Only
- Length Only
- No Length, Width or Height

Culvert Lines

- With Z value
- Without Z value
- Model Conduits
- HRM Boundary
- Major Watershed

No	Date	Description
2	APR. 5, 2024	Issued for Final Report
1	NOV. 24, 2023	Issued for Final Draft Report
A	MAR 29, 2023	Issued for Information

Revision or Issue

HALIFAX

Project:

Creation of Pluvial, Fluvial and Coastal Flood Hazard Maps

Drawing Title:

Maps of available information on culvert and bridge structures.
Watershed:
MUSQUODOBOIT

CBCL

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Checked:	---	Approved:	---
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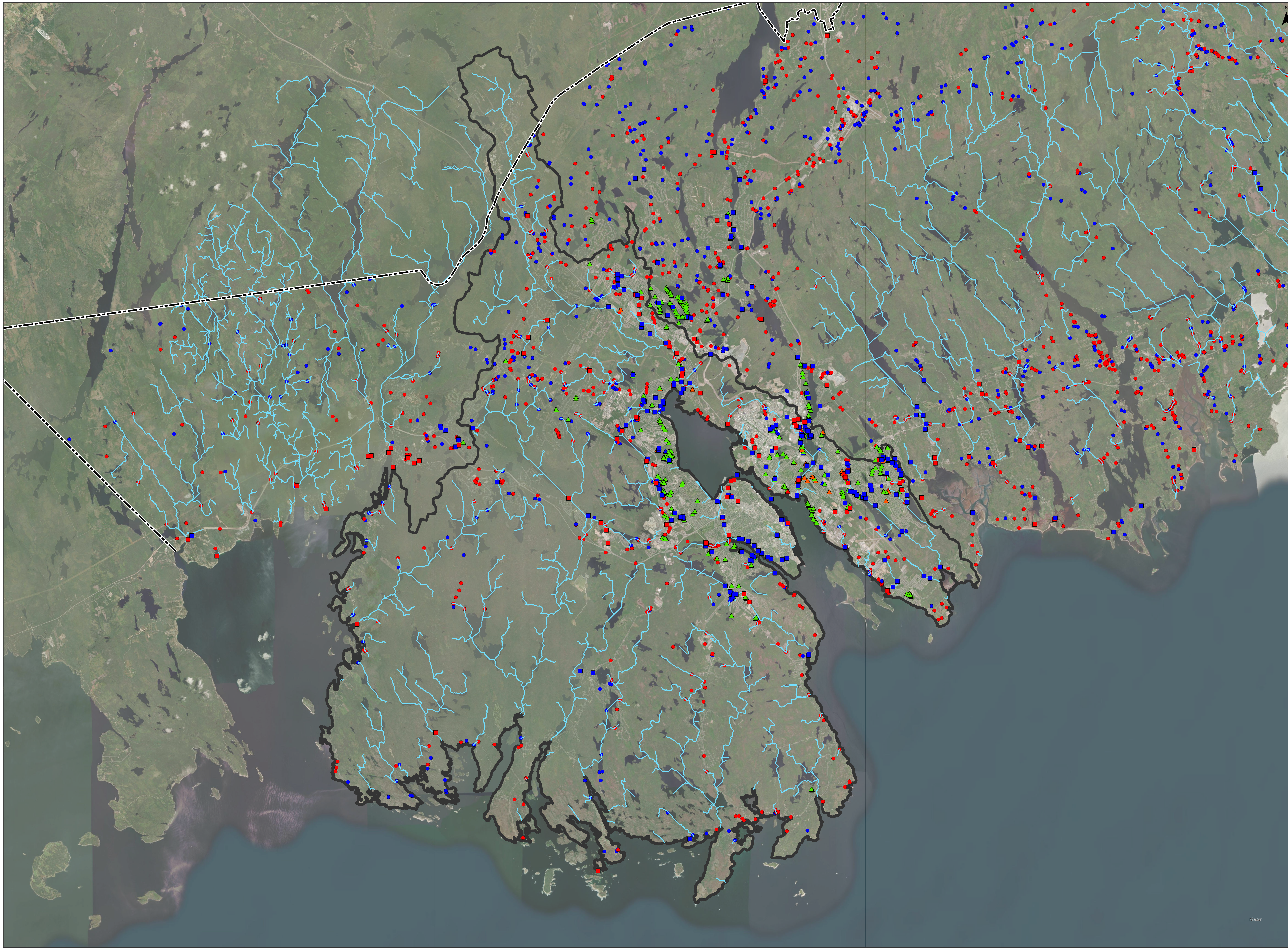

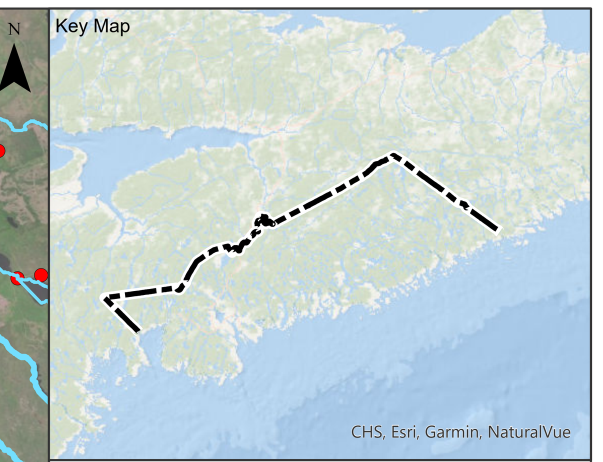

Legend

Bridge Points
- Span Length Only
- No Span Length or Soffit Elevation

Culvert Points
- Length and Width
- Width Only
- Length Only
- No Length, Width or Height

Culvert Lines
- With Z value
- Without Z value
- Model Conduits
- HRM Boundary
- Major Watershed

No	Date	Description
2	APR. 5, 2024	Issued for Final Report
1	NOV. 24, 2023	Issued for Final Draft Report
A	MAR 29, 2023	Issued for Information

Revision or Issue

HALIFAX

Project:

Creation of Pluvial, Fluvial and Coastal Flood Hazard Maps

Drawing Title:

Maps of available information on culvert and bridge structures.
Watershed:
SACKVILLE

CBCL

Date:	APR. 2024	Scale:	1:103,000
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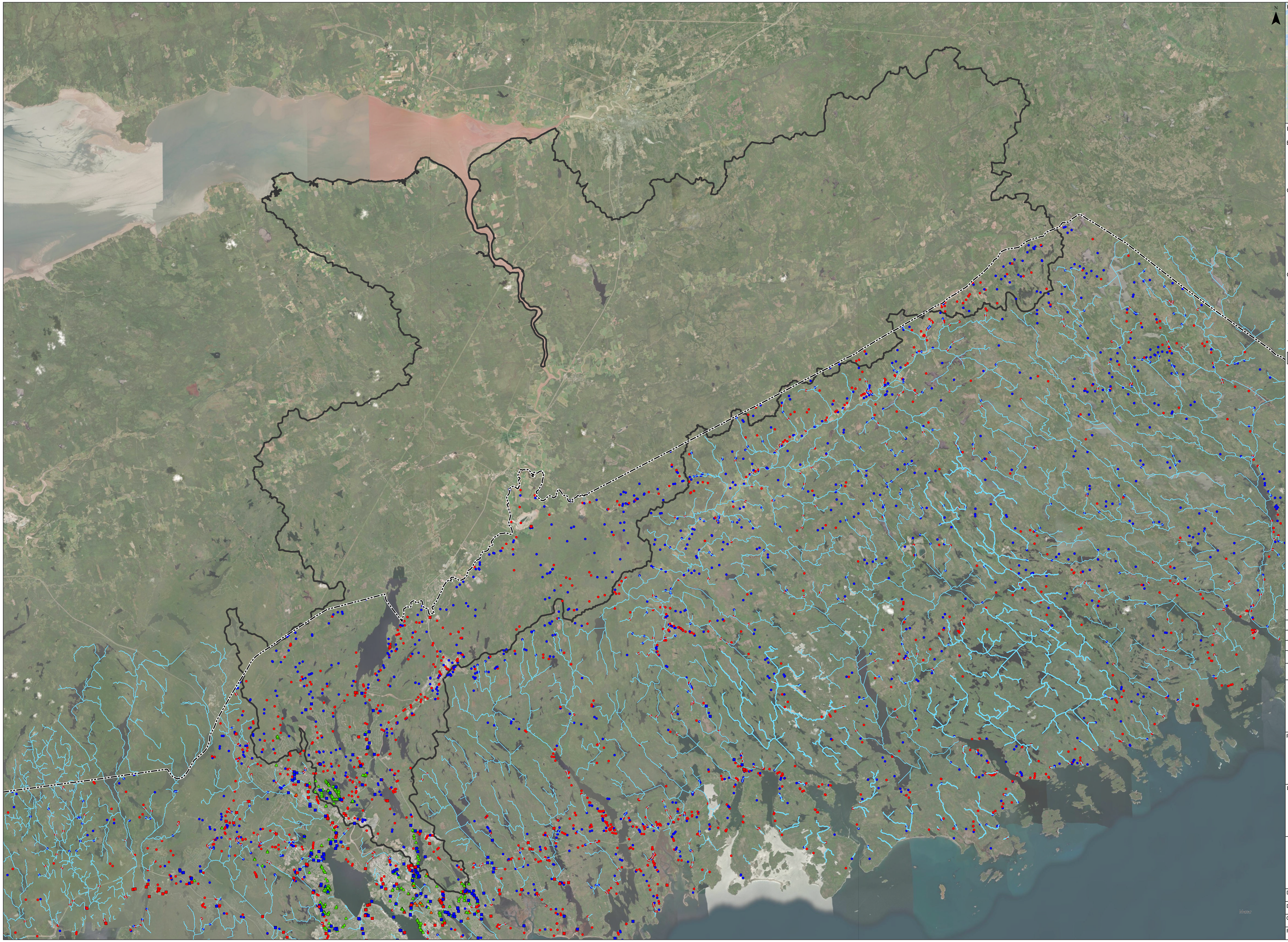
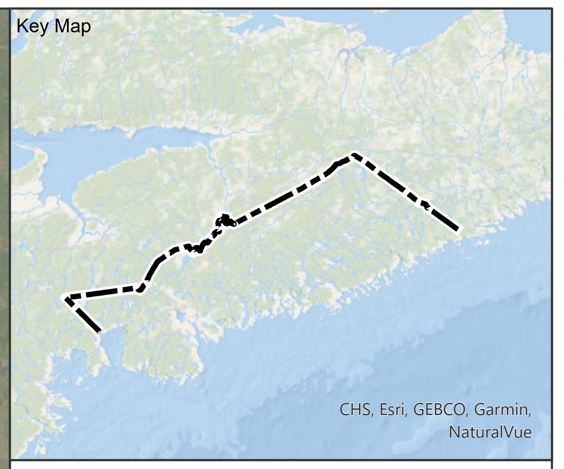
Checked:	---	Approved:	---
Project No:	221111.00	Drawing No:	1.0





Legend

Bridge Points

- Span Length Only
- No Span Length or Soffit Elevation

Culvert Points

- Length and Width
- Width Only
- Length Only
- No Length, Width or Height

Culvert Lines

- With Z value
- Without Z value
- Model Conduits
- HRM Boundary
- Major Watershed

No	Date	Description
2	APR. 5, 2024	Issued for Final Report
1	NOV. 24, 2023	Issued for Final Draft Report
A	MAR 29, 2023	Issued for Information

Revision or Issue

HALIFAX

Project:

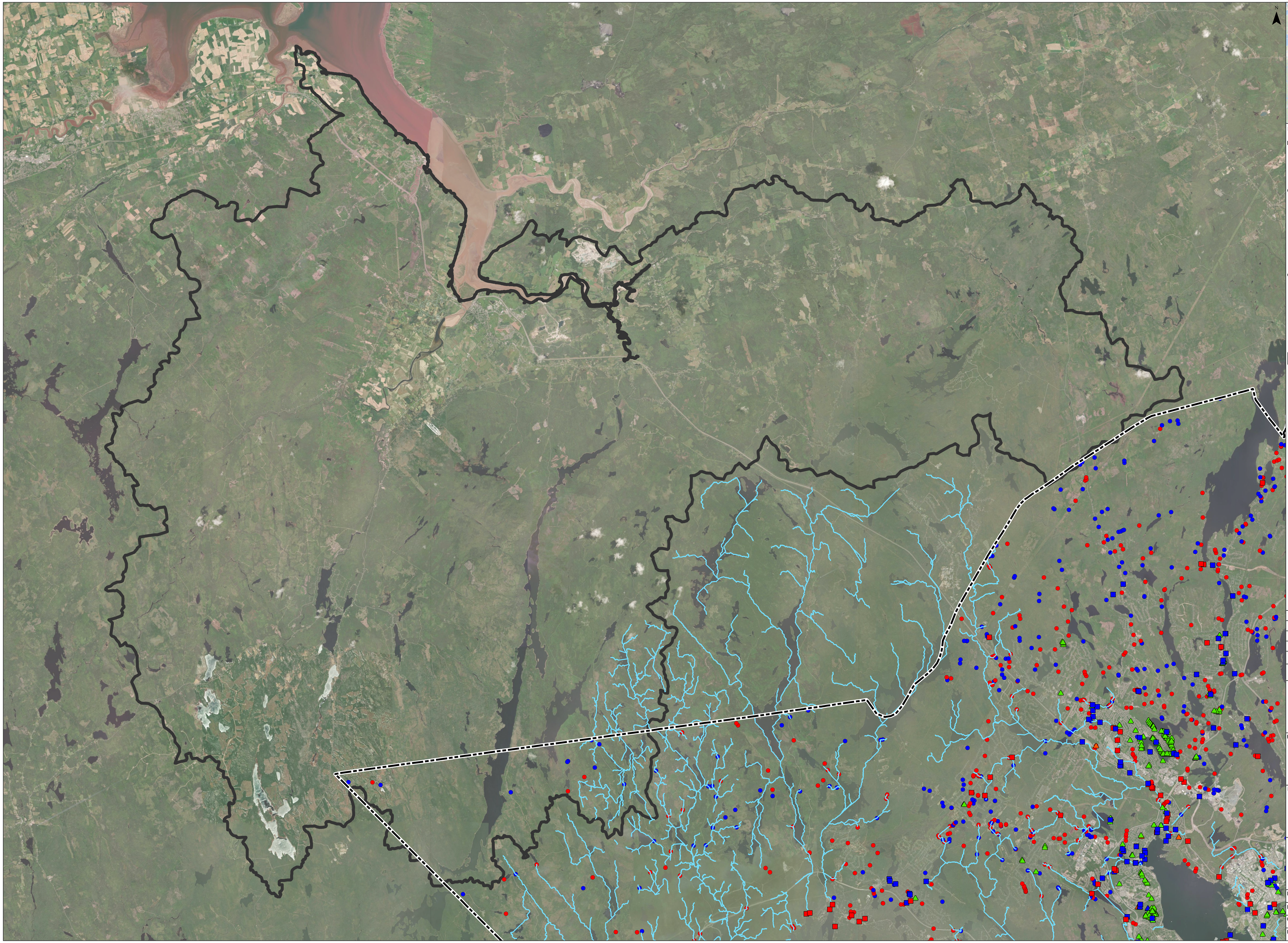
Creation of Pluvial, Fluvial and Coastal Flood Hazard Maps

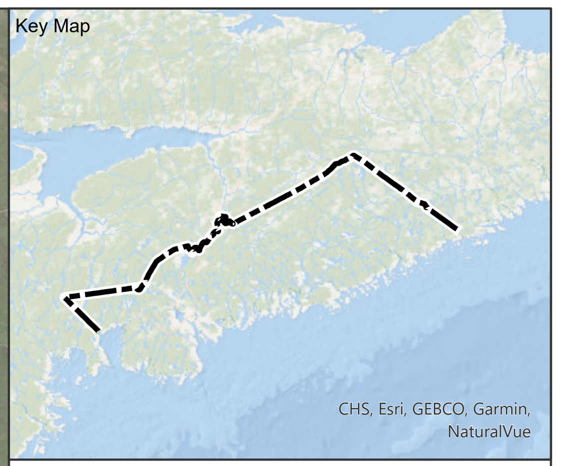
Drawing Title:

Maps of available information on culvert and bridge structures. Watershed: SHUBENACADIE/STEWIACKE

CBCL

Date: APR. 2024	Scale: 1:158,000
Drawn: CM	Designed: ATW
Checked: ---	Approved: ---
Project No: 221111.00	Drawing No: 1.0





Legend

Bridge Points
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- No Span Length or Soffit Elevation

Culvert Points
- Length and Width
- Width Only
- Length Only
- No Length, Width or Height

Culvert Lines
- With Z value
- Without Z value
- Model Conduits
- HRM Boundary
- Major Watershed

No	Date	Description
2	APR. 5, 2024	Issued for Final Report
1	NOV. 24, 2023	Issued for Final Draft Report
A	MAR 29, 2023	Issued for Information

Revision or Issue

HALIFAX

Project:

Creation of Pluvial, Fluvial and Coastal Flood Hazard Maps

Drawing Title:

Maps of available information on culvert and bridge structures.
Watershed:
ST. CROIX

CBCL

Date:	APR. 2024	Scale:	1:89,000
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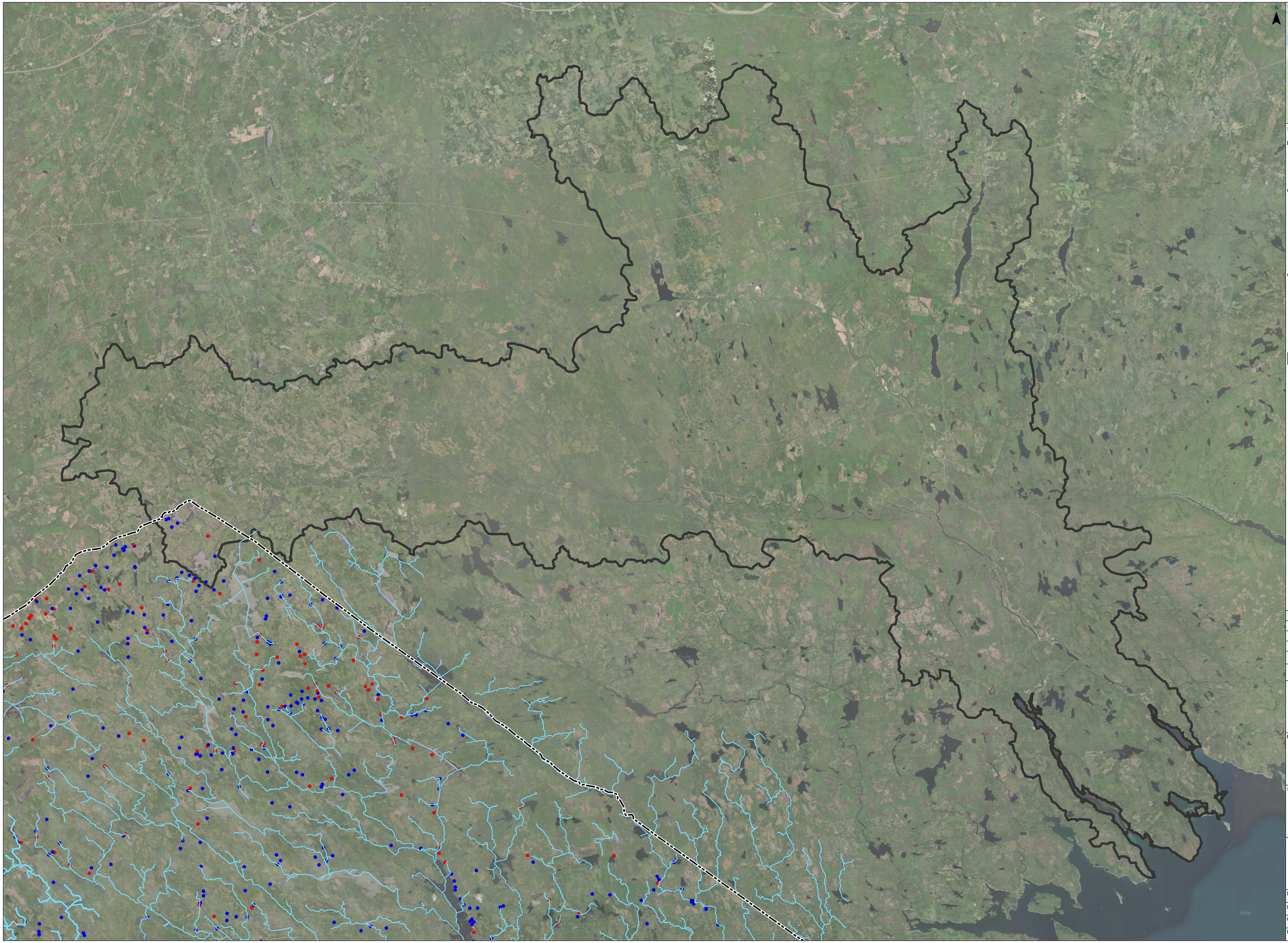
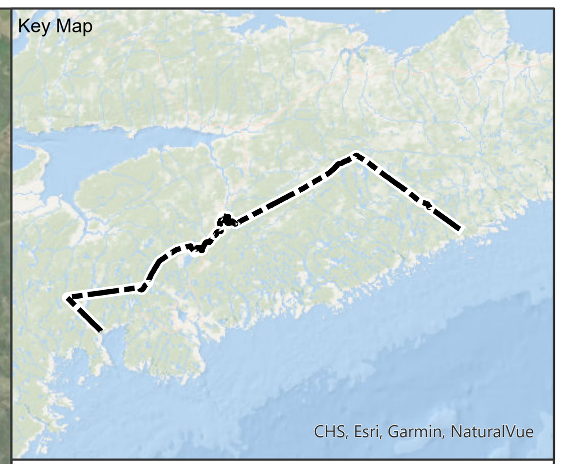
Checked:	---	Approved:	---
Project No:	221111.00	Drawing No:	1.0



Key Map



Legend

Bridge Points

- Span Length Only
- No Span Length or Soffit Elevation

Culvert Points

- Length and Width
- Width Only
- Length Only
- No Length, Width or Height

Culvert Lines

- With Z value
- Without Z value
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- HRM Boundary
- Major Watershed

No	Date	Description
2	APR. 5, 2024	Issued for Final Report
1	NOV. 24, 2023	Issued for Final Draft Report
A	MAR 29, 2023	Issued for Information

Revision or Issue

HALIFAX

Project:

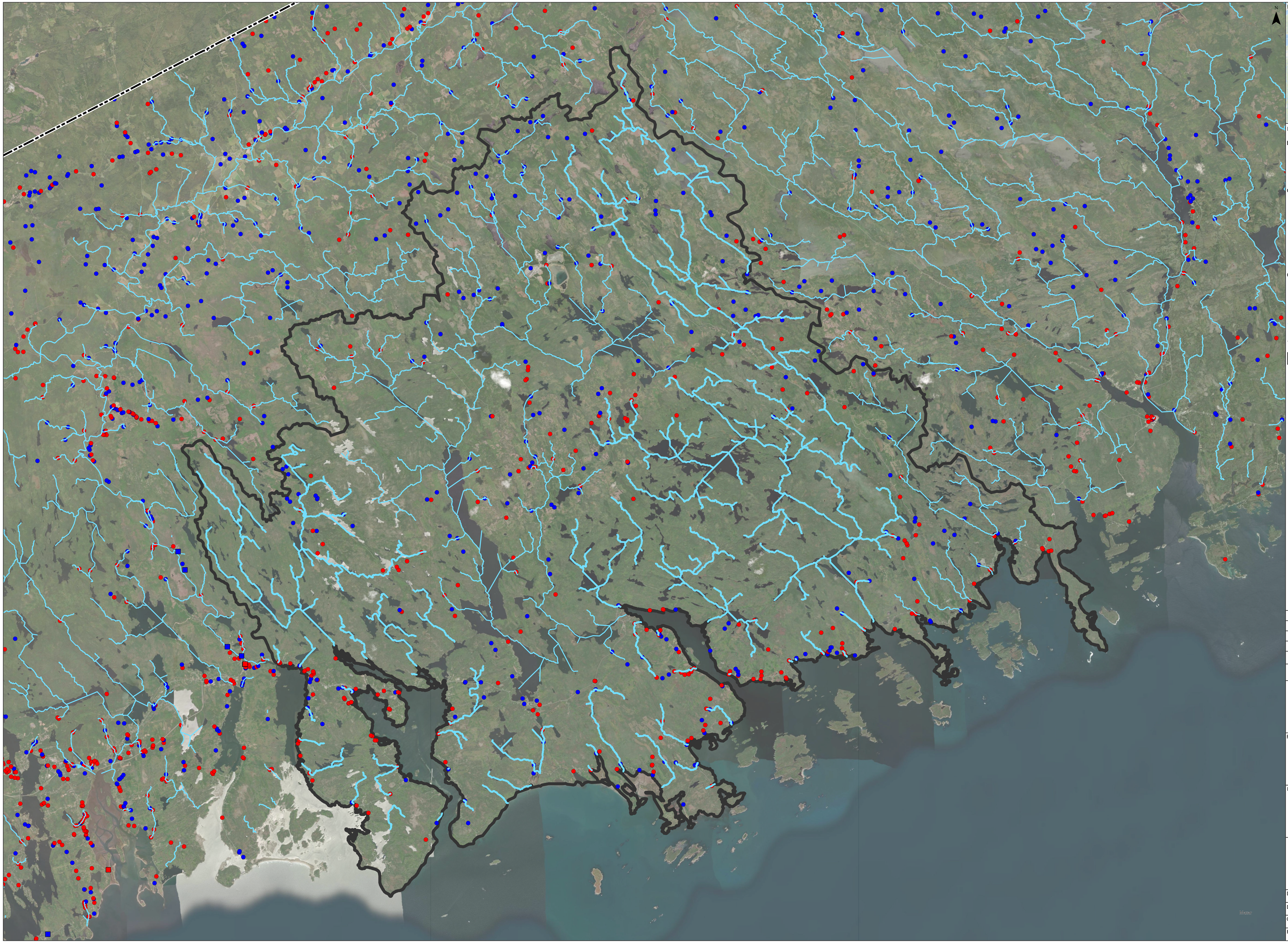
Creation of Pluvial, Fluvial and Coastal Flood Hazard Maps

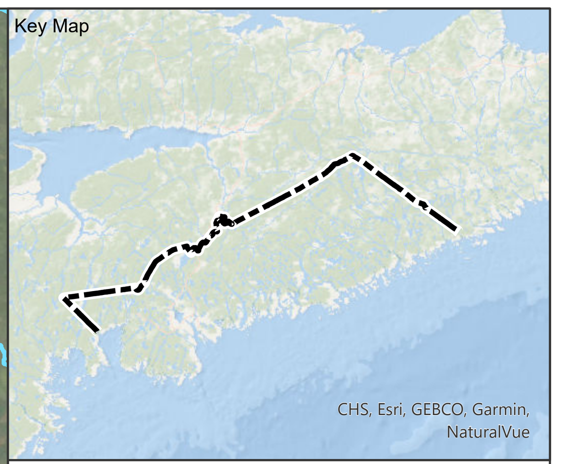
Drawing Title:

Maps of available information on culvert and bridge structures.
Watershed:
ST. MARYS

CBCL

Date:	APR. 2024	Scale:	1:110,000
Drawn:	CM	Designed:	ATW
Checked:	---	Approved:	---
Project No:	221111.00	Drawing No:	1.0





Legend

Bridge Points

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- No Span Length or Soffit Elevation

Culvert Points

- Length and Width
- Width Only
- Length Only
- No Length, Width or Height

Culvert Lines

- With Z value
- Without Z value
- Model Conduits
- HRM Boundary
- Major Watershed

No	Date	Description
2	APR. 5, 2024	Issued for Final Report
1	NOV. 24, 2023	Issued for Final Draft Report
A	MAR 29, 2023	Issued for Information

Revision or Issue

HALIFAX

Project:

Creation of Pluvial, Fluvial and Coastal Flood Hazard Maps

Drawing Title:

Maps of available information on culvert and bridge structures.
Watershed:
TANGIER

CBCL

Date:	APR. 2024	Scale:	1:89,000
Drawn:	CM	Designed:	ATW
Checked:	---	Approved:	---
Project No:	221111.00	Drawing No:	1.0

APPENDIX C

Flood Maps for Key Areas

(Mapping is Provided Separately)

APPENDIX D

Climate Change Sensitivity Assessment

D.1 Introduction

For this project, HRM specified the use of one scenario, the Representative Concentration Pathway (RCP) 8.5, which is a higher-end pathway adopted by the Intergovernmental Panel on Climate Change (IPCC) Fifth Assessment Report (AR5). Given this selection, CBCL further recommended the use of the 50th percentile (rather than the 90th percentile, a less common choice when used on its own with RCP 8.5). Thus, the flood mapping conducted in this project used the RCP 8.5 median scenario for extreme precipitation.

These industry-standard practices were selected because these are easily justifiable and enabled the project to proceed in a timely manner. Nonetheless, best practice is to take a risk-based approach by considering climate change uncertainty, and where possible assessing several scenarios. Therefore, CBCL devised a climate change scenario sensitivity analysis add-on, to be completed in parallel with the flood mapping. The outcome of this task is to provide a wider perspective of climate change uncertainty for the flood mapping results generated with the RCP 8.5 50th percentile scenario.

This climate change sensitivity assessment is divided into two parts:

1. **Definition of climate scenarios (Section D.2).** The first part of the work considered emerging guidance and research on climate scenarios and methodologies, explored the range of projections based on different climate scenarios, and selected two scenarios for the sensitivity assessment.
2. **Flood line sensitivity testing (Section D.3).** The second part of the work consisted of propagating the selected precipitation intensity scenarios through the flood models to assess how climate uncertainty translates into flood line uncertainty.

Note that the purpose of this assessment was limited to assessing the sensitivity of flood lines to climate change impacts on precipitation intensity; therefore, climate-related coastal scenarios (i.e., sea-level rise) were kept constant.

D.2 Definition of Climate Scenarios

D.2.1 Best Practices for Climate Uncertainty

This sub-section provides additional background on risk-based planning. Ultimately, a risk-based planning process provides better information to support decision making. This involves choosing climate change events that are of equal or greater magnitude than the design event and then creating flood maps based on these events. This allows us to visualize the potential risk to vulnerable areas if the design event has underestimated the true impact of climate change. This process allows the client to modify the floodplain boundary at those locations, modify the Land Use By-Laws, implement mitigation measures (which could include retreat), or make no modifications. Regardless of the decision, the assessment facilitates a more informed choice.

This risk-based approach is considered best-practice by several authoritative sources:

- Federal Hydrologic and Hydraulic Procedures for Flood Hazard Delineation (2019).
- Nova Scotia Flood Mapping Guidelines and Specifications (2020).
- Municipal Flood Line Mapping: Planning Horizons and Considerations (2019).
- CSA PLUS 4013: Technical Guide: Development, Interpretation and Use of Rainfall Intensity-Duration-Frequency (IDF) Information (2019).
- Environment and Climate Change Canada – guidance available on ClimateData.ca.

This climate scenario sensitivity assessment allows HRM to assess how climate change uncertainty may influence the outcomes of the overall mapping. This could be the basis of a future risk-based approach (i.e., systematic assessment of climate change and coastal scenario uncertainty across the domain, and selection of different flood lines in different areas depending on risk tolerance).

D.2.2 Emission Scenarios

This subsection provides background on Representative Concentration Pathways (RCPs), Shared Socioeconomic Pathways (SSPs), recent debate on emission scenario selection based on likelihood, and ultimately the emission scenarios that were chosen for this climate change sensitivity assessment. Note that the term “emission scenarios” is used broadly to also refer to forcing scenarios (i.e., those not explicitly defined based on emission levels).

Figure D1 provides a historical overview of the generations of emission scenarios that have been used to drive climate models, including RCPs (adopted for the IPCCC2013?) and SSPs (adopted in the IPCCC 2020?). These are based on numerous assumptions related to policy and societal developments.

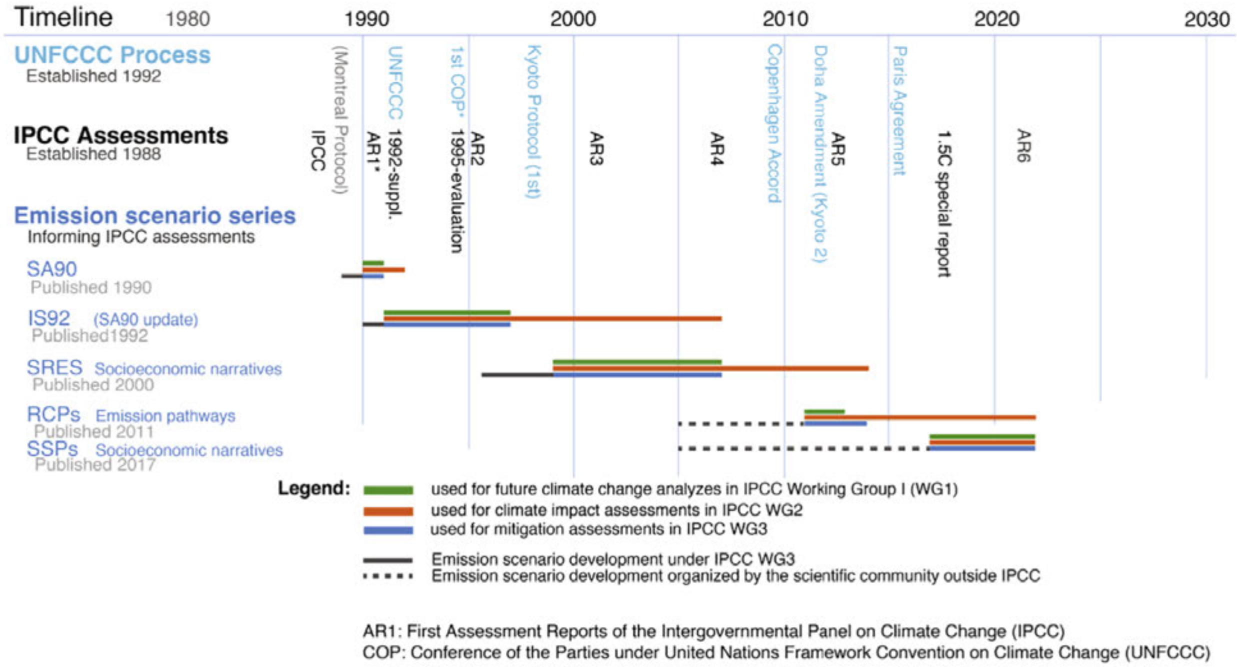

Figure D1: Timeline of intergovernmental processes and emission scenarios.

Representative Concentration Pathways (RCPs)

The Representative Concentration Pathways (RCPs), originally published in 2011 and adopted by the IPCC Fifth Assessment Report, have been widely used for the past decade (IPCC 2013).

- **RCP 4.5** is one of the intermediate stabilization pathways in which radiative forcing is stabilized at approximately 4.5 W/m^2 after 2100. Radiative forcing refers to the difference between the incoming energy from the Sun absorbed by the Earth's atmosphere and the energy radiated back to space.
- **RCP 8.5** is a high-end pathway in which radiative forcing reaches greater than 8.5 W/m^2 by 2100 and continues to rise for some time afterwards.

Shared Socioeconomic Pathways (SSPs)

The IPCC Sixth Assessment Report has adopted Shared Socioeconomic Pathways (SSPs), based on five narratives describing alternative socio-economic developments. The SSPs were developed to standardize the societal factors that result in given levels of climate change. Within each SSP "family" there can be multiple emissions scenarios that lead to different levels of radiative forcing. Therefore, the SSPs are labelled both with the family and the resulting radiative forcing. For example, SSP5-8.5 has radiative forcing that corresponds with RCP8.5 (O'Neill et al. 2014).

- **SSP1 and SSP5** assume relatively optimistic human development, including substantial investments in education and health, rapid economic growth, and well-functioning institutions. However, while SSP5 assumes this will be driven by an energy-intensive, fossil fuel-based economy, SSP1 envisions an increasing shift toward sustainable practices.

- **SSP3 and SSP4** are more pessimistic in terms of future social and economic development, with a fast-growing population, increasing inequalities, and little investment in education or health in poorer countries.
- **SSP2** represents a "middle of the road" scenario, where development continues along historical patterns throughout the 21st century.

Emission Scenario Selection
For decades there has been debate about whether emission scenarios choice should be based on likelihood. For a long time, experts felt that these likelihood assumptions fall into the category of "unknowable knowledge" and would thus be subjective (Strandsbjerg Pederson et al. 2022). In a recent shift, the AR6 recognized that the likelihood of high emissions scenarios such as RCP8.5 or SSP5-8.5 is considered low (IPCC 2021).

Hausfather and Peters (2020) suggested that RCP 8.5, which was originally designed as an unlikely high-risk future, becomes more implausible with every passing year. They note that emission pathways to reach RCP 8.5 generally require an unprecedented fivefold increase in coal use by the end of the century, which is larger than some estimates of coal reserves, and inconsistent with many energy forecasts. Based on this, they suggest that assessments should move forward with the most likely (best-guess) scenarios. Pielke et al. (2022) argued for an additional metric for assessing best-guess scenarios: consistency with observed CO_2 emission growth rates and with the International Energy Agency Stated Policies Scenario (STEPS) near-term projections. According to Pielke et al. (2022), these are approximately in line with RCP 4.5, RCP 6.0 and SSP 2-4.5 (Figure D2).

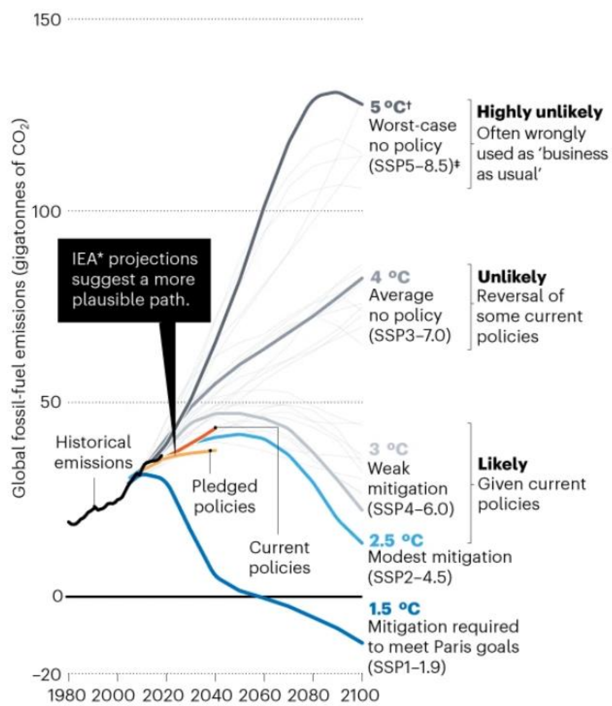

Figure D2: Emissions scenarios compared to observations and International Energy Agency (IEA) forecasts (Hausfather and Peters 2020; Pielke et al. 2022).

This discussion might lead the reader to exclude RCP 8.5 and SSP 5-8.5 from assessments because it is potentially less likely to occur. However, ultimately, robust adaptation policy solutions must be based on a wide range of plausible scenarios rather than a chosen best-guess (Strandsbjerg Pederson et al. 2022), as remarked in Section D.2.1 on risk-based approaches. Despite stating that SSP 5-8.5 has low likelihood, the IPCC (2021) also cautions that important feedback effects could be much larger than estimated by current climate models, and that it is therefore important to consider a full range of scenarios as possible outcomes (IPCC 2021).

Hence, there is value in continuing to use RCP 8.5 and SSP 5-8.5, if these are not being communicated as the most-likely outcome, but rather as an unlikely high-risk future (Hausfather and Peters 2020). This is also the approach taken by the Canadian Center for Climate Services, which provides both CMIP6 models based on both SSP2-4.5 and SSP5-8.5 on ClimateData.ca.

Hence, for this climate change sensitivity assessment, we have chosen to use RCP 8.5 and SSP 5-8.5, not based on a likelihood assessment, but as test of a potential high-risk future.

D.2.3 Model Uncertainty and Ensemble Percentiles

In the past, model ensembles were generally constructed by giving equal weighting to each model ("model democracy"). For example, the AR5 ensemble was constructed by choosing one run per model per scenario. The 5-95th % ensemble range was then interpreted as the 17–83% uncertainty range (IPCC 2013).

Although the CMIP6 models have improved compared with CMIP5, with better representation of physical processes and higher resolution, several models also have high "Equilibrium Climate Sensitivity". **Equilibrium climate sensitivity** (ECS) is a measure of the change in temperature caused by a doubling of atmospheric CO_2 from pre-industrial conditions. Based on multiple lines of evidence, the IPCC estimates that Earth's ECS is very likely between 2°C and 5°C, with a likely range of 2.5°C to 4°C (Figure D3). However, several CMIP6 models fall outside of this very likely range (i.e., many of the models are "too hot"), which brings into question the reliability of projections from these models (IPCC 2021).

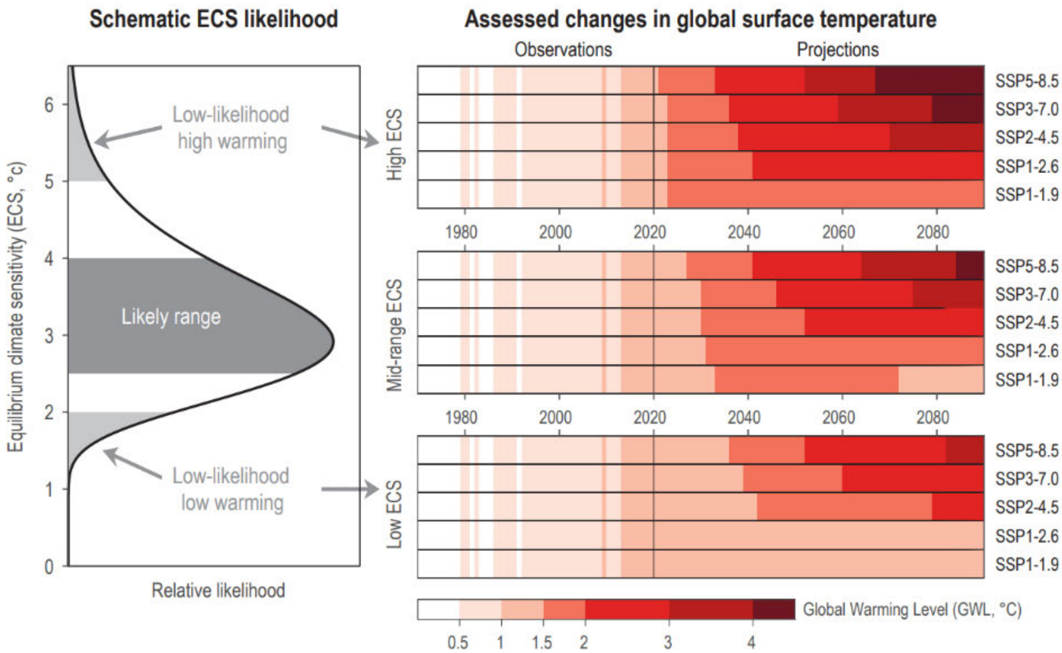

Figure D3: Representation of how assessed changes in global surface temperature vary with the Equilibrium Climate Sensitivity (ECS). The likely range of ECS under AR6 is shown in the y-axis on the left (IPCC 2021).

As a result, there have been calls to move beyond model democracy towards weighted ensembles that account for both model performance and model independence (for further information, refer to the IPCC 2021 for a review of model selection and weighting methods). However, at present there is "no universal, robust method for weighting a multi-model projection ensemble" (IPCC 2021). In Canada, this is complicated by the fact that the Canadian model (CanESM5) has a very high ECS compared to other CMIP6 models. Therefore, for the AR6, the IPCC chose to follow a similar approach as AR5, using the 5-95% range of all models as the likely uncertainty range for projections, with no further weighting of selection (with the exception of global surface air temperature). The IPCC concludes that **expert judgement must be included, as it did for AR5, in the assessment of the projections**. Similarly, the Canadian Center for Climate Services (CCCS) has chosen not to apply model selection to the CMIP6 model ensemble presented on ClimateData.ca.

In addition, the IPCC recommends that upper-bound climate models (with high ECS) be used to develop low-likelihood, high-impact scenarios to explore risks. For regional assessments, it has been recognized that the median of the model ensemble fails to address physically plausible, but low-likelihood, high-impact scenarios, and in some cases is a statistical construct not representative of true conditions (IPCC 2021).

Hence, in line with the IPCC and CCCS, we have chosen for the Climate Change Sensitivity Assessment presented in Appendix D to use the full ensemble of CMIP5 and CMIP6 models, and to consider upper bound models (e.g., 90^{th} or 95^{th} percentile) as low-likelihood, high-impact scenarios, in addition to the median (50th percentile) that was used for the main portion of the project.

D.2.4 Extreme Precipitation Scaling

As described in Section 3.2, extreme precipitation projections for this project were obtained using a temperature scaling method based on the Clausius-Clapeyron equation. Although daily rainfall extremes based on analyses of observed time series from different weather stations seem to follow the Clausius-Clapeyron rate (Westra et al. 2014), there is some indication that hourly rainfall extremes exceed this rate (Westra et al. 2014), which is known as a “super Clausius-Clapeyron rate”. Studies using convection permitting models also found that super Clausius-Clapeyron rates were found in certain cases for the largest extremes (Fosseret al. 2020; Hodnebrog et al. 2019; Knist et al. 2020; Lenderinket al. 2021; Mantegna et al. 2017), in seasons other than summer (Ban et al. 2020), and for the shorter-duration extremes (Cannon and Innocenti 2019).

Hence, in future work, it would be worth considering the implications of a higher scaling rate. However, given the scope of this climate change sensitivity assessment (i.e., to test two additional scenarios), priority was given to a test of the emission scenarios (Section D.2.2) and model ensemble percentiles (Section D.2.3), and an increased scaling rate was not investigated.

D.2.5 Projections Analysis & Comparison

As discussed in the above sections, for this climate change sensitivity assessment, alternative scenarios (percent changes in precipitation intensity) were obtained by varying emission scenarios (Section D.2.2) and model percentiles (Section D.2.3), compared to the base scenario (RCP 8.5, 50th percentile).

The results are shown in Figure D4, indicating a spread in the projected changes in precipitation intensity depending on the emissions scenarios and climate models used. In particular, the following findings are noted for 2071-2100:

- Results obtained with CMIP6 SSP 5-8.5 are similar (within a 5% change in precipitation intensity) to the RCP 8.5 base case used in this project (40% for 2071-2100).
- The upper percentiles of the model ensemble using CMIP5 RCP 8.5 are up to approximately a 60% increase in precipitation intensity for 2071-2100.
- The upper percentiles of the model ensemble using CMIP6 SSP 5-8.5 are up to approximately an 80% increase in precipitation intensity for 2071-2100.

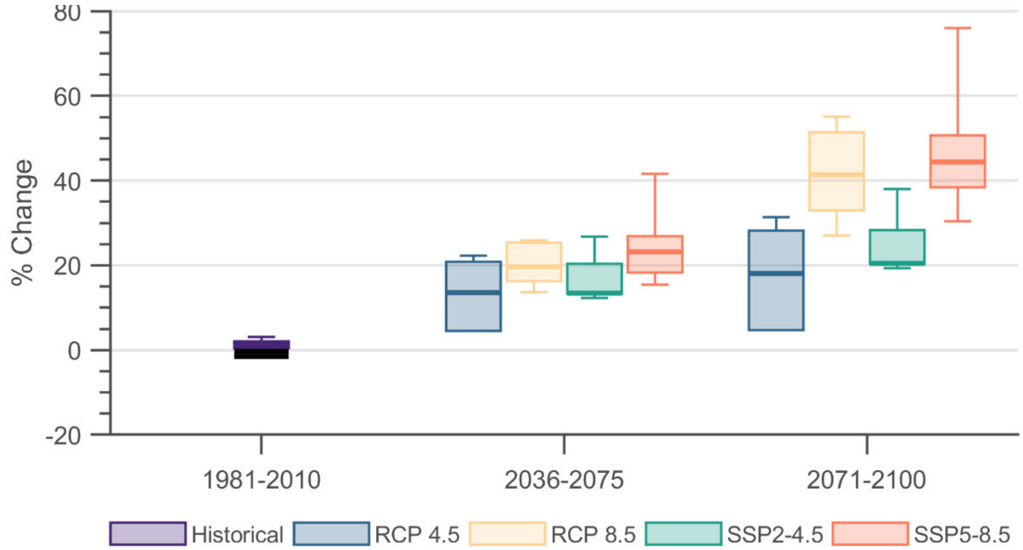

Figure D4: Projected changes in precipitation intensity for HRM based on different emission scenarios and model ensembles. The black line represents observations in the historical period (Shearwater Station). The box plots show the median of the model ensemble (central line), the 25th and 75th percentiles (top and bottom of the boxes), and the 10th and 90th percentiles (top and bottom of the whiskers).

D.2.6 Selected Scenarios

Based on the above findings, the scenarios selected for the climate change sensitivity assessment are 60% and 80% increases in precipitation intensity (Table D1). Note that these have been purposefully defined based on the increases in precipitation intensity themselves, rather than emission scenario-model combinations, as is best practice in sensitivity assessments. Therefore, these should be referred to as the "**60% increase scenario**" and "**80% increase scenario**". These alternative climate scenarios can be used to test for climate change sensitivity when compared to the base case climate scenario (40% increase).

Table D1: Scenarios for increase in precipitation intensity for use in the Climate Change Sensitivity Assessment

	CMIP5 RCP 8.5	CMIP6 SSP 5-8.5
Median	Approximated by base case used in project (40% increase)	
Upper Percentile	Tested with 60% increase scenario	Tested with 80% increase scenario

D.3 Flood Line Sensitivity Testing

In this section, the Upper Percentile values from the Climate Change Sensitivity Assessment were tested with the hydrodynamic models and mapped.

These values include:

- 60% increase in rainfall (above the 1 in 100 year existing climate conditions).
- 80% increase in rainfall (above the 1 in 100 year existing climate conditions).

This allows the visualization of the effects of higher-than-expected rainfall increases on the extents of flooding in various areas.

Four general areas were selected for visualization, to see how the various types of areas might be affected by the additional increases in rainfall:

- A rural area with steeper slopes (Duck Lake).
- A semi-urban area with residential development (Highway 103 in St. Margaret's Bay).
- A semi-urban area with commercial development (Highway 103 interchange with Hammonds Plains Rd in St Margaret's Bay).
- A rural area with shallow wetland slopes on the East River in St. Margaret's Bay.

Those are presented in Figures D5 to D8 on the following pages.

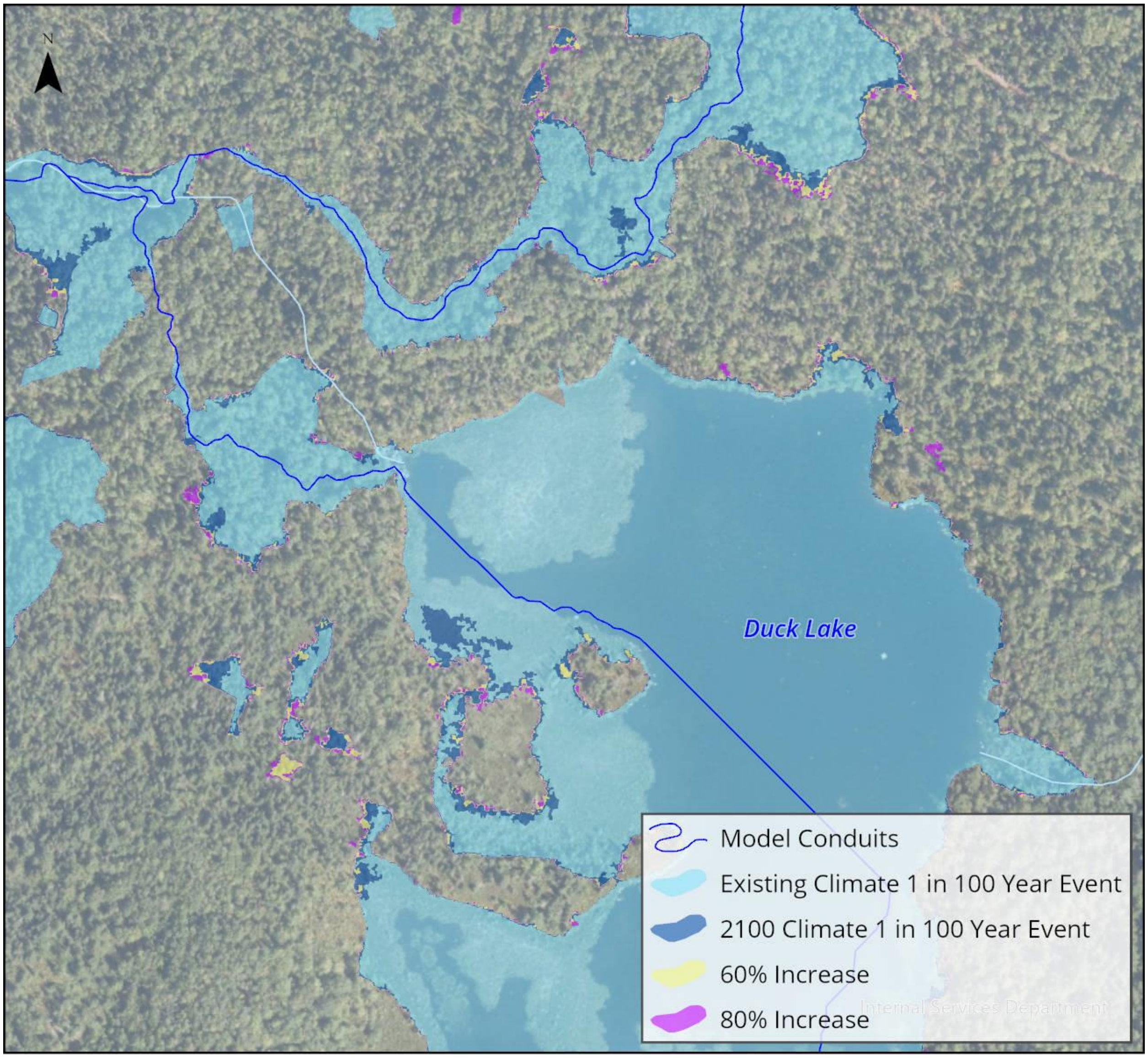

Figure D5: Climate Change Floodline Sensitivity around Duck Lake.

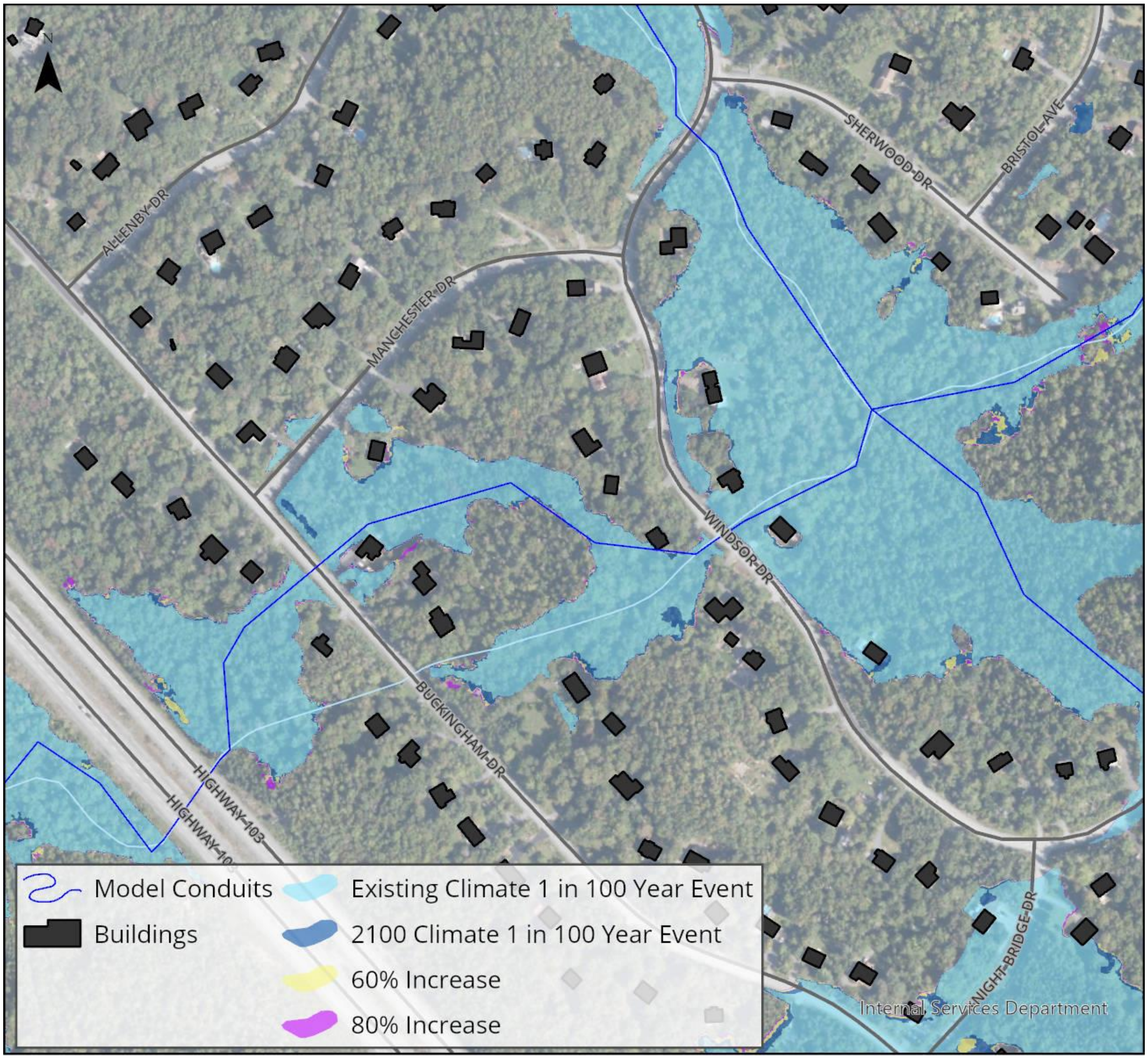

Figure D6: Climate Change Floodline Sensitivity around St Margaret's Bay HWY 103 Residential Areas.

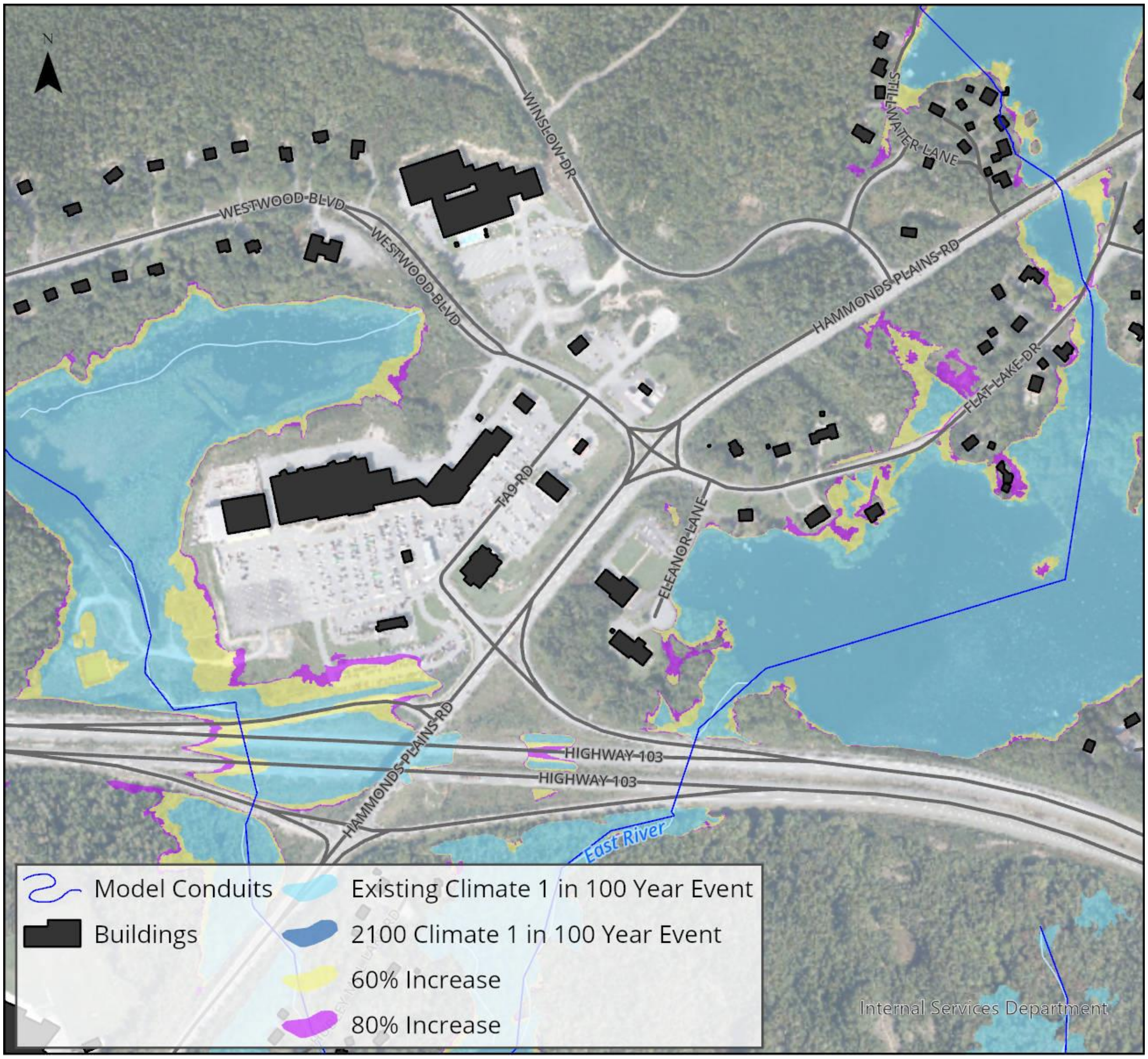

Figure D7: Climate Change Floodline Sensitivity around St Margaret's Bay HWY 103 Interchange.

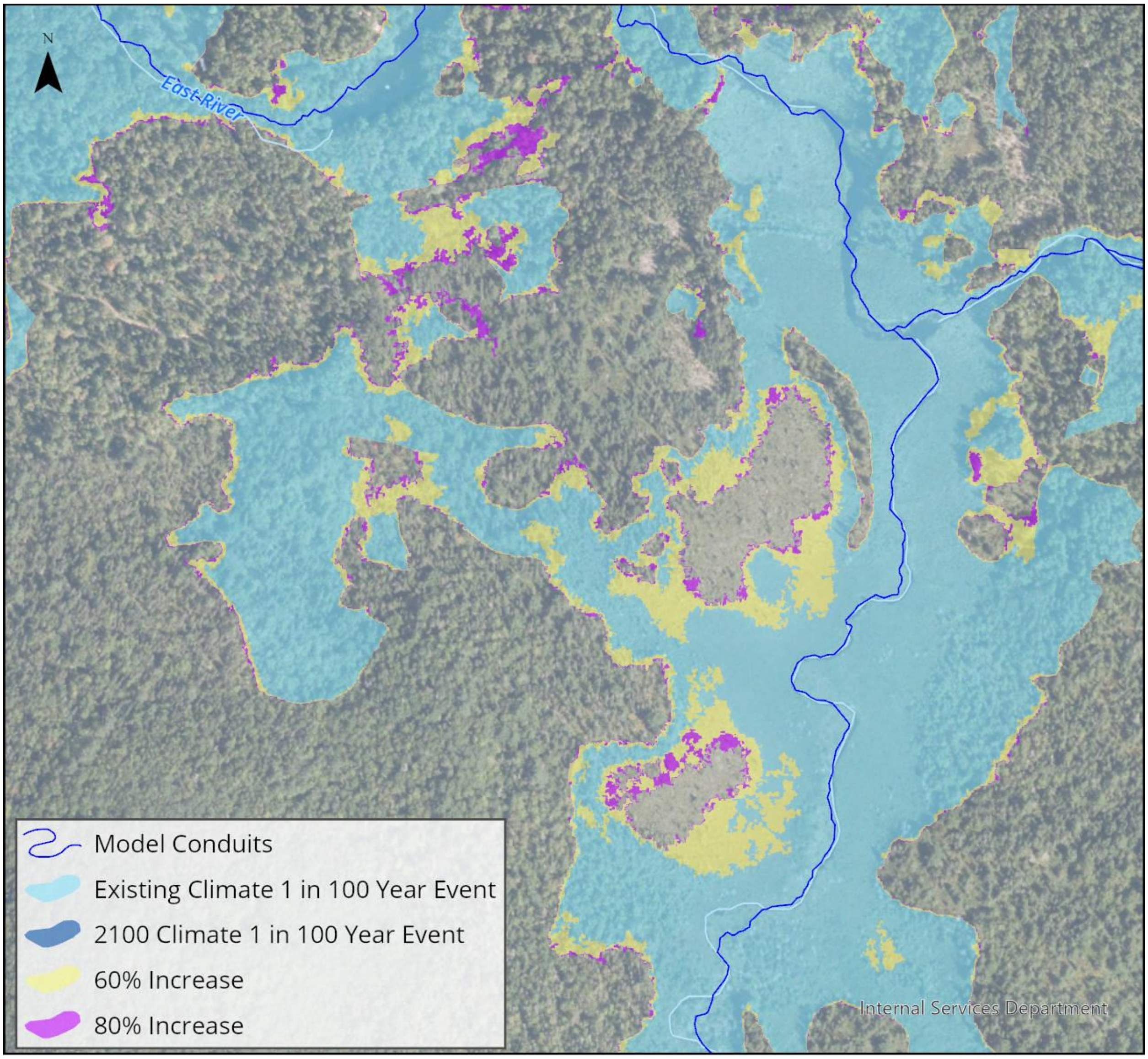

Figure D8: Climate Change Floodline Sensitivity around East River Rural Area.

The results show that when the landscape slopes are generally steep (Figures D5 and D6), then the increase in flooding is minimal, with no discernible impacts, even at the individual residential lot scale. However, when the topography of the area has low slope or flatter ground conditions; (Figures D7 and D8), the increase in the flooding is more pronounced. This emphasizes the importance of floodplain topography or slope on the overall flood extent at any location.

It is noted that even in the instances where slopes are shallower, the increase in flood extent is not sufficient to have a notable impact on the amount of infrastructure at risk. In the case of the Highway 103 interchange with Hammonds Plains Road (Figure D7), there are only a few additional metres of access road and parking lot that would be flooded. Similarly, on Highway 103 itself, the model shows some flooding for the 1 in 100 year event in existing climate conditions, and with the +80% rainfall scenario, the increase of flooding on the highway is marginal. This comparison does not include field surveyed bridge and culvert measurements, thus the flood line comparison should be treated as a relative difference.

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APPENDIX E

Offshore Wind and Wave Hindcast Analyses

Appendix E - Offshore Wind and Wave Hindcast Analyses

List of Figures

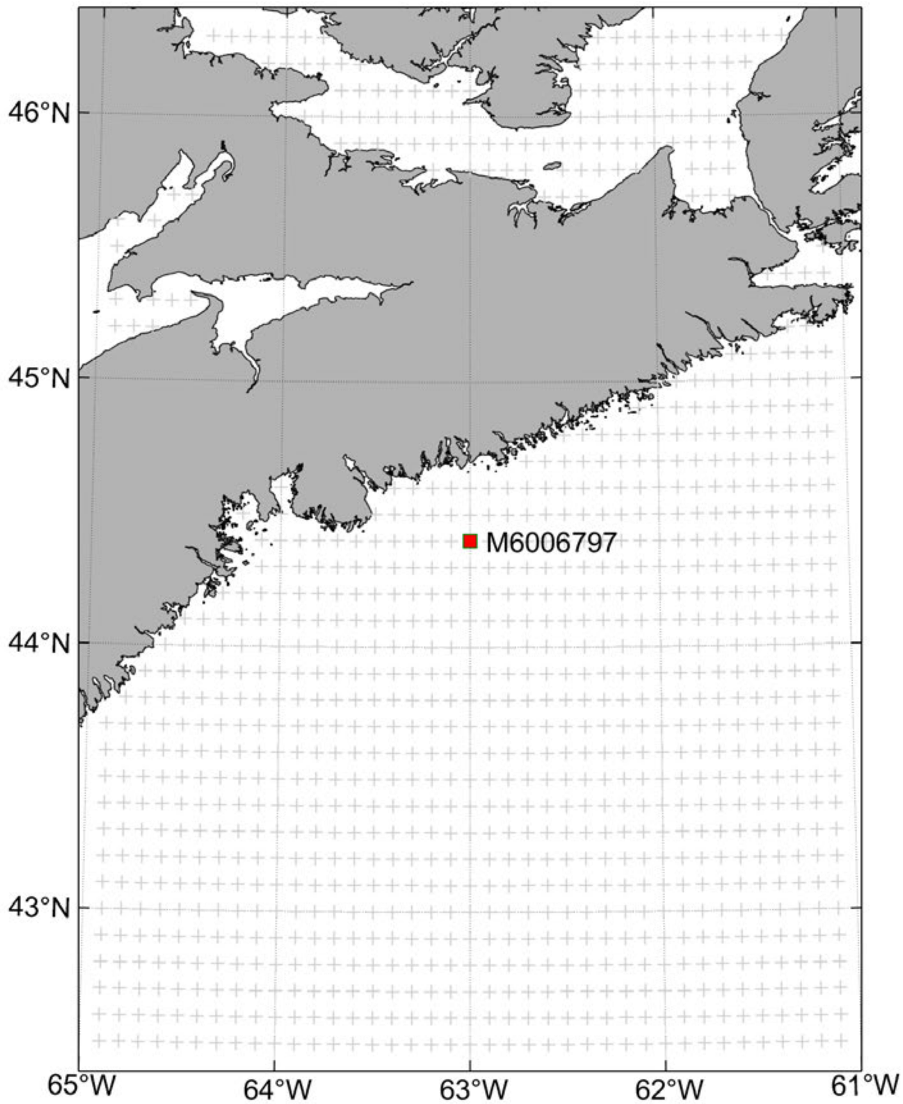


Figure 1: Study Area and Selected MSC50 Grid Point.

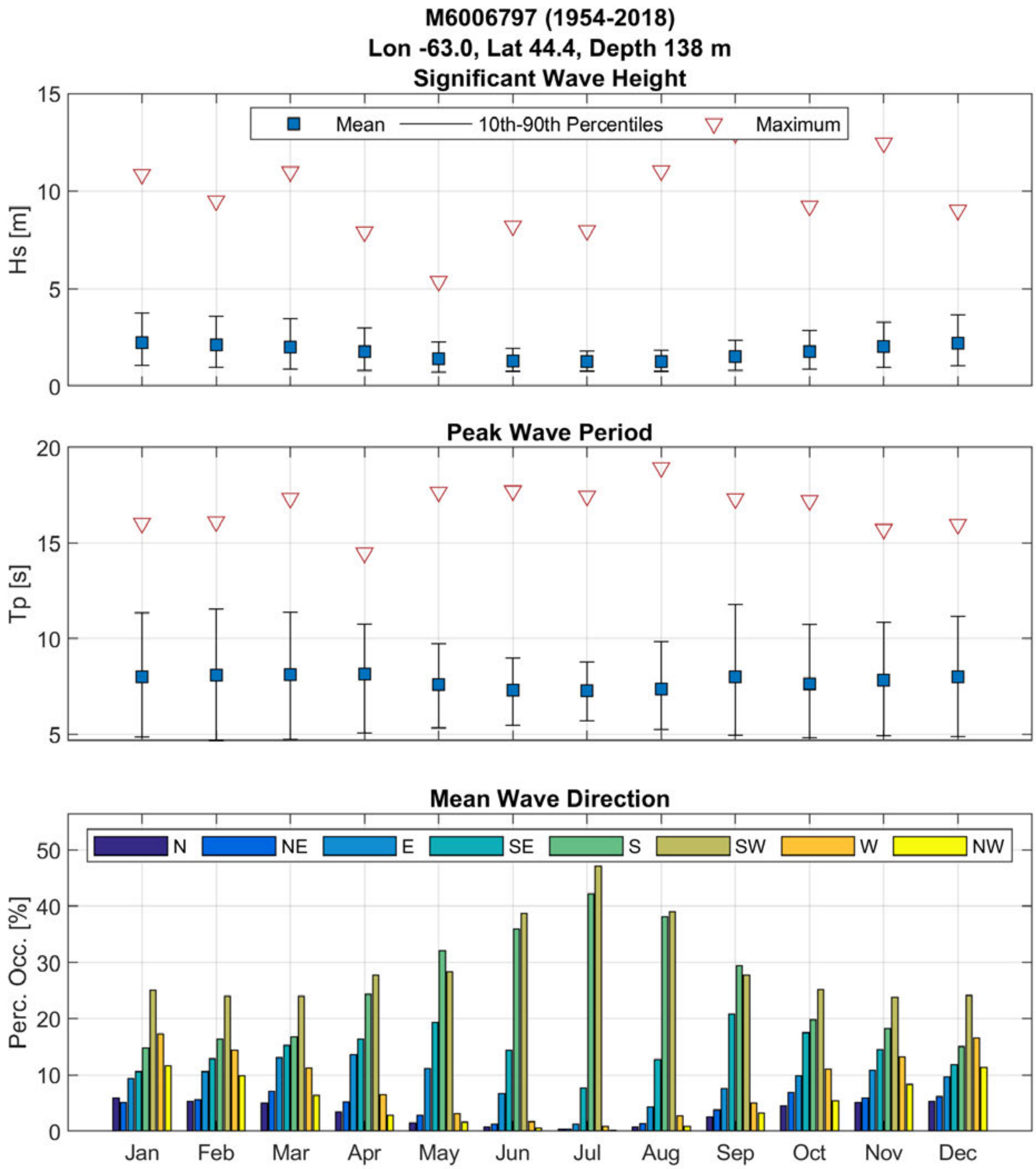
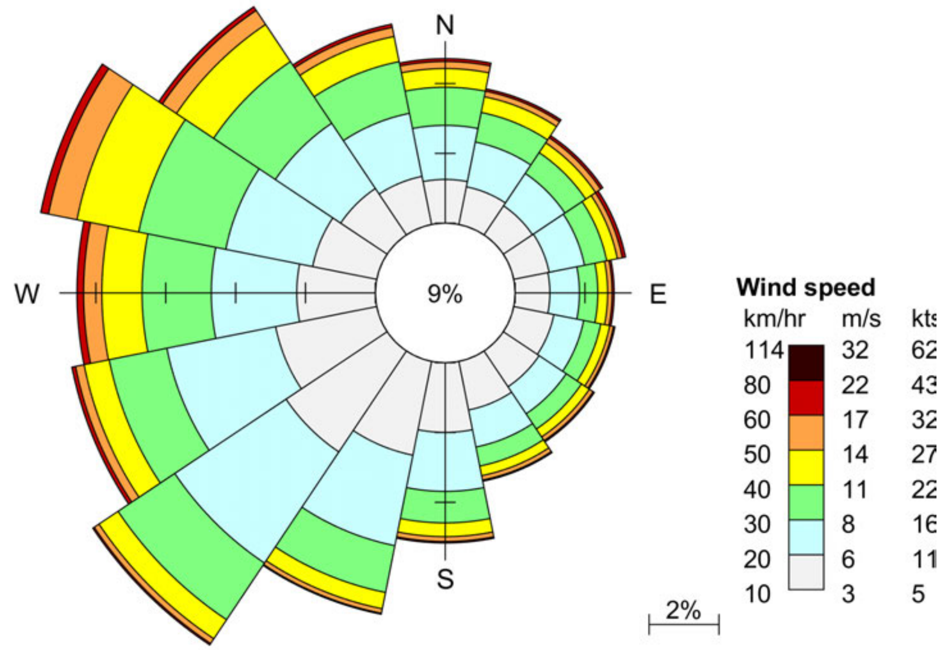
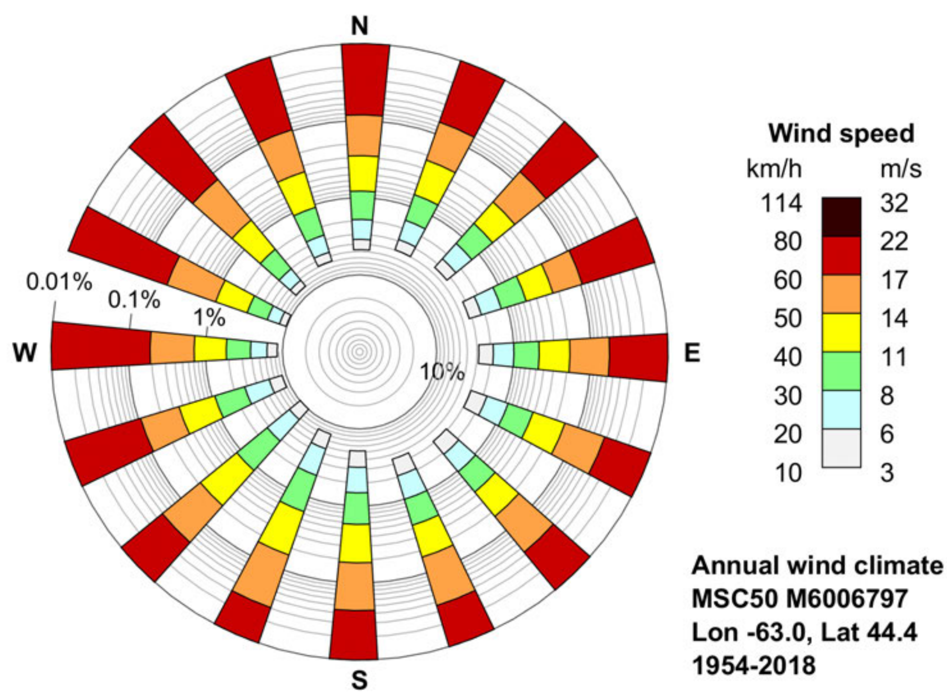


Figure 2: Monthly Variability of Wave Conditions.

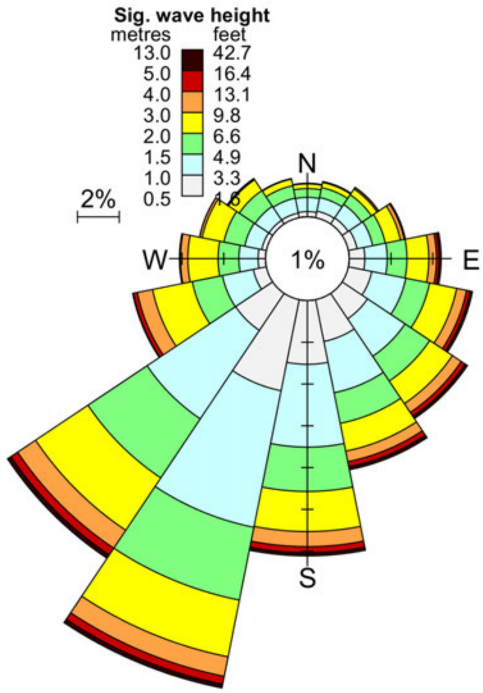


(a) Linear Scale

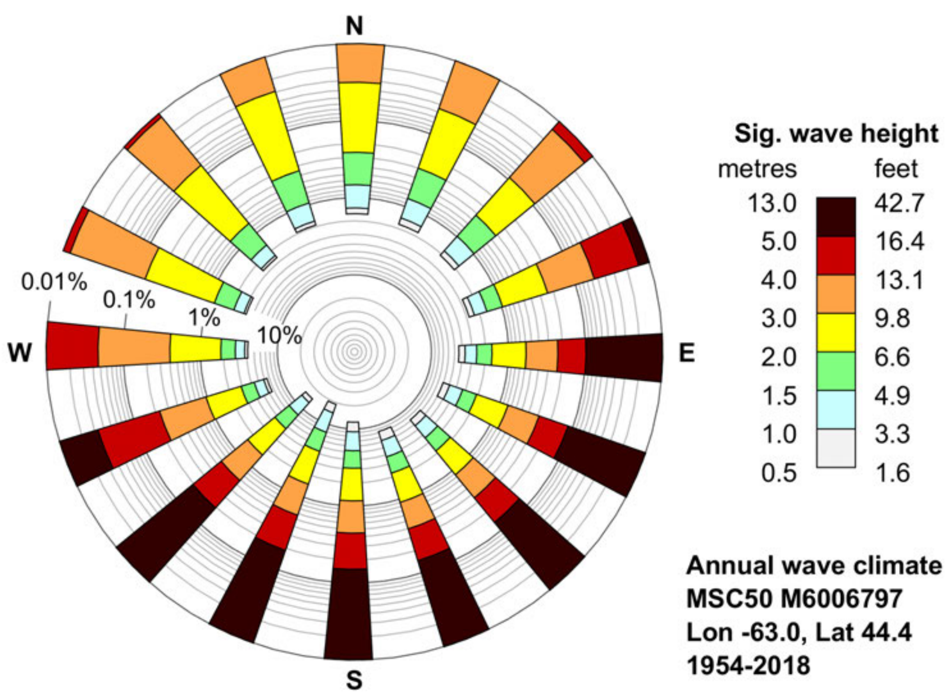


(b) Logarithmic Scale

Figure 3: Wind Rose - Joint Occurrence of Wind Speed and Wind Direction.



(a) Linear Scale



(b) Logarithmic Scale

Figure 4: Wave Rose - Joint Occurrence of Significant Wave Height (Hs) and Mean Wave Direction.

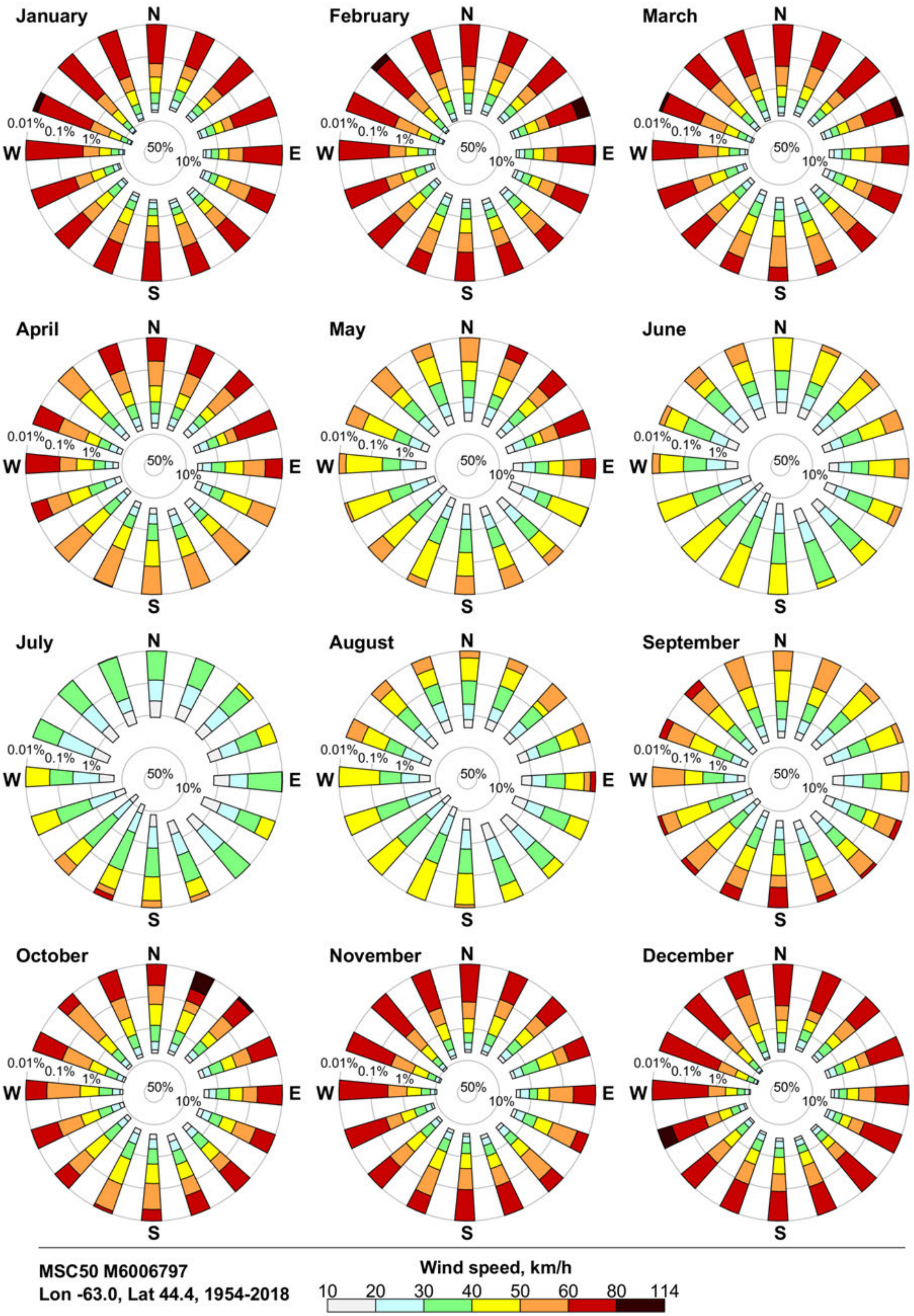


Figure 5: Wind Rose - Monthly Logarithmic Joint Occurrence of Wind Speed and Wind Direction.

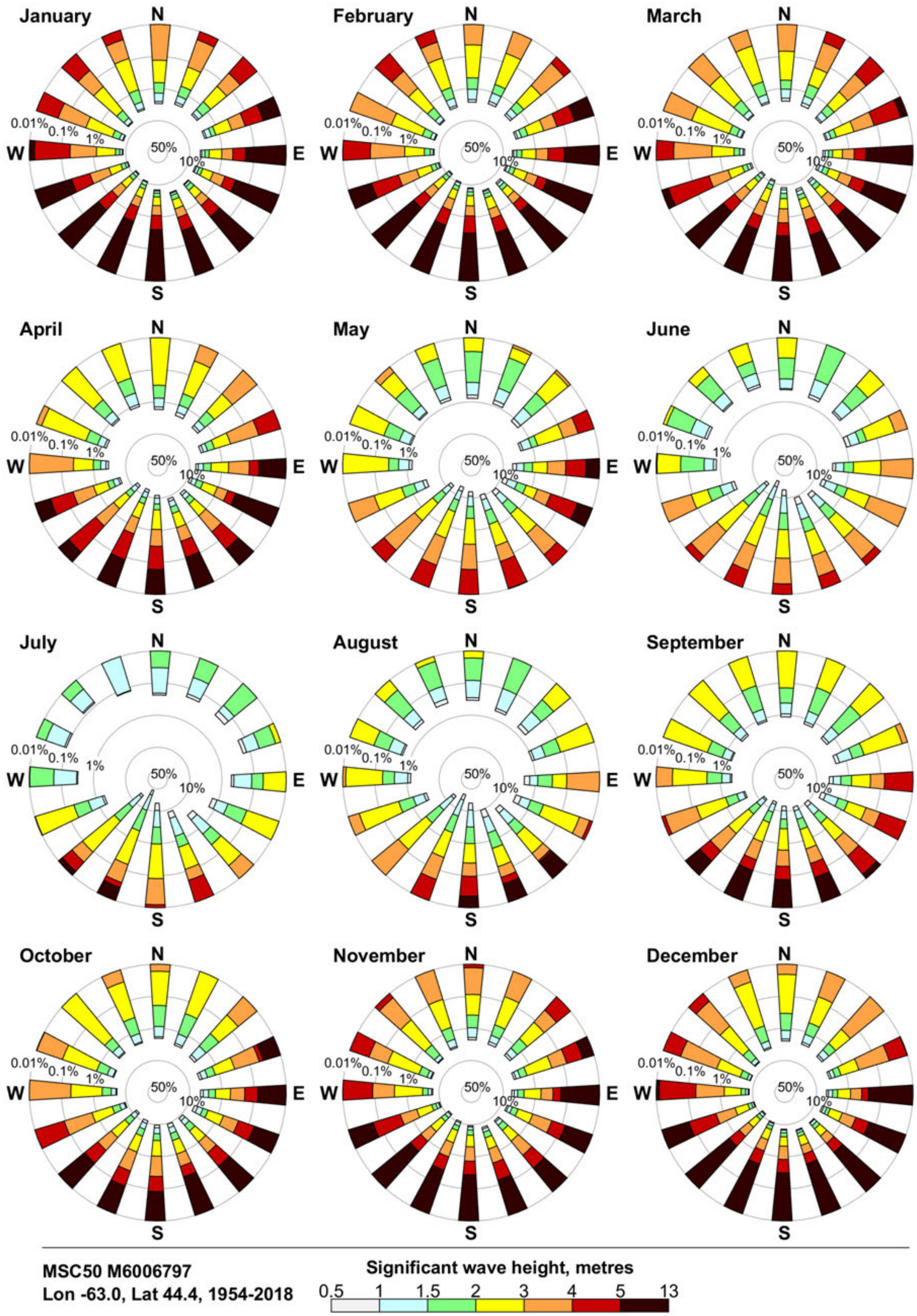


Figure 6: Wave Rose - Monthly Joint Occurrence of Significant Wave Height (Hs) and Mean Wave Direction.

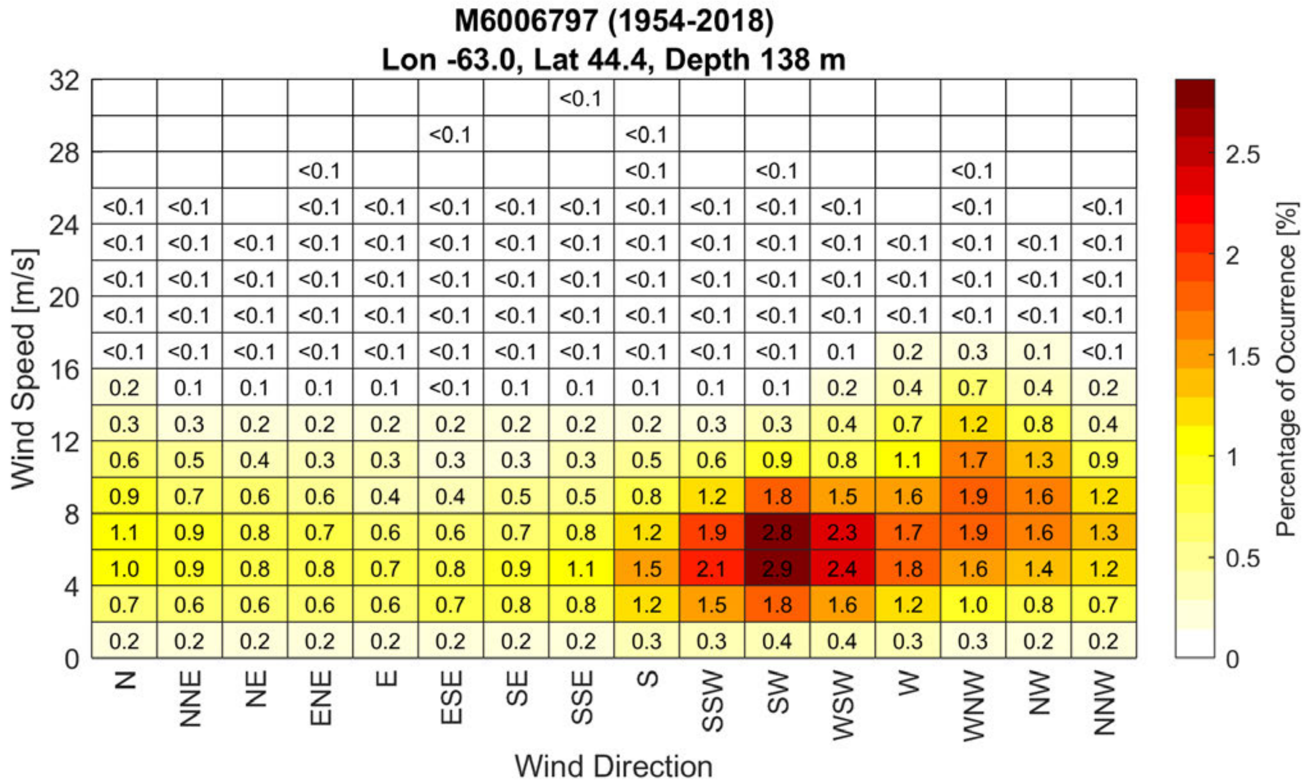


Figure 7: Joint Occurrence of Wind Speed and Direction.

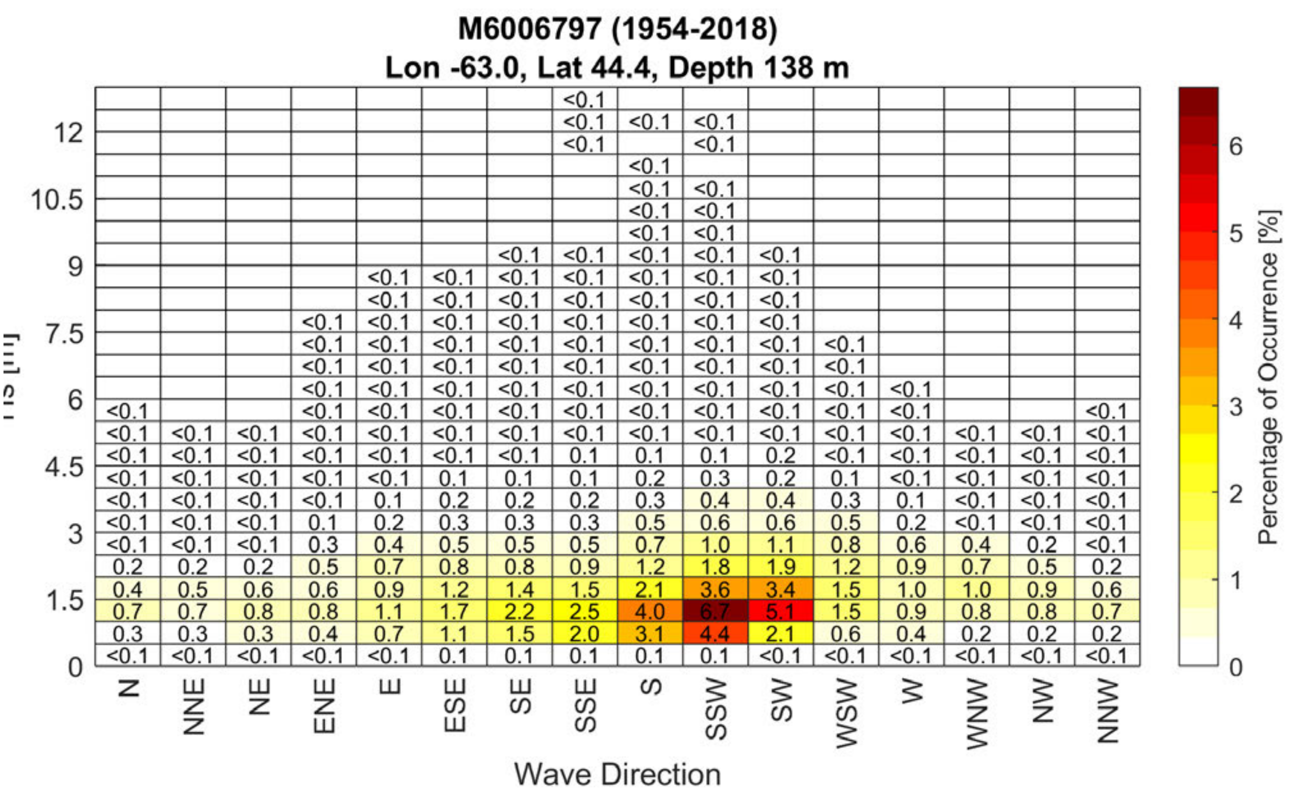
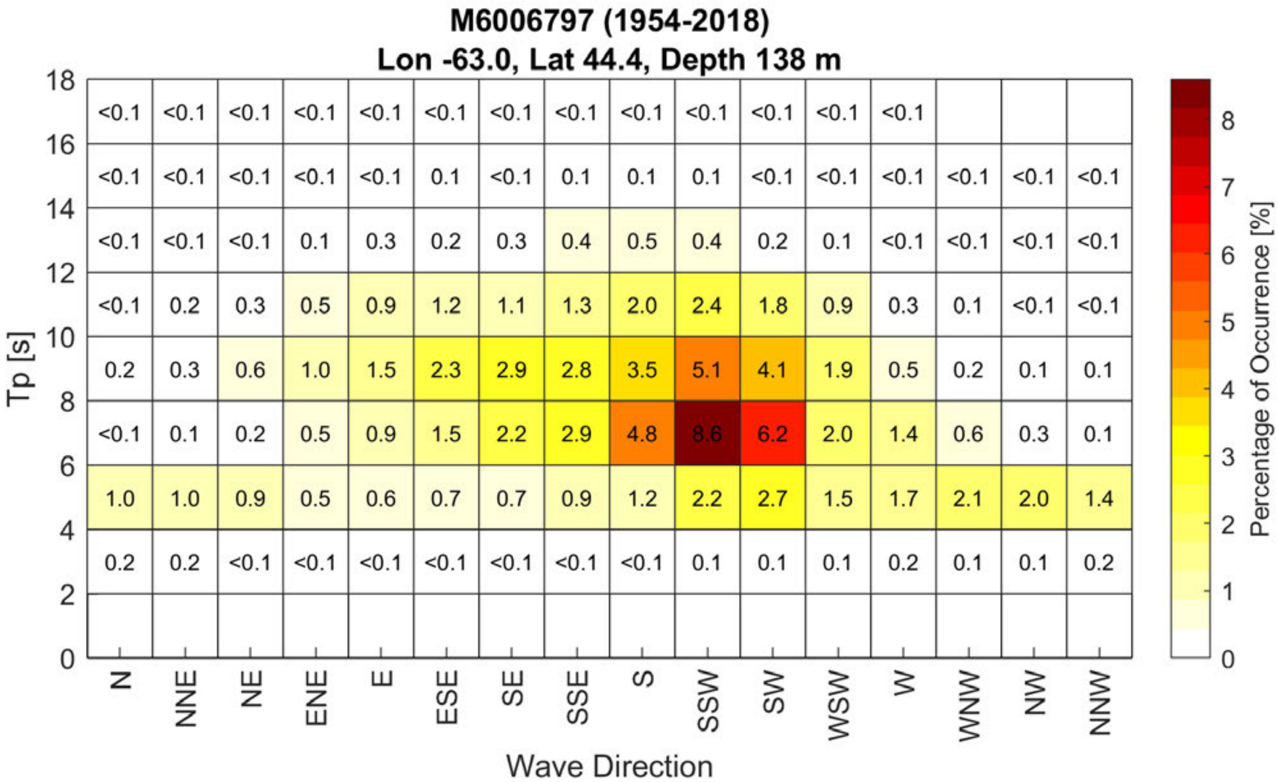


Figure 8: Joint Occurrence of Significant Wave Height (Hs) and Mean Wave Direction.



Tp [s]	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW
16–18	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1			
14–16	<0.1	<0.1	<0.1	<0.1	<0.1	0.1	<0.1	0.1	0.1	0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
12–14	<0.1	<0.1	<0.1	0.1	0.3	0.2	0.3	0.4	0.5	0.4	0.2	0.1	<0.1	<0.1	<0.1	<0.1
10–12	<0.1	0.2	0.3	0.5	0.9	1.2	1.1	1.3	2.0	2.4	1.8	0.9	0.3	0.1	<0.1	<0.1
8–10	0.2	0.3	0.6	1.0	1.5	2.3	2.9	2.8	3.5	5.1	4.1	1.9	0.5	0.2	0.1	0.1
6–8	<0.1	0.1	0.2	0.5	0.9	1.5	2.2	2.9	4.8	8.6	6.2	2.0	1.4	0.6	0.3	0.1
4–6	1.0	1.0	0.9	0.5	0.6	0.7	0.7	0.9	1.2	2.2	2.7	1.5	1.7	2.1	2.0	1.4
2–4	0.2	0.2	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	0.1	0.1	0.1	0.2	0.1	0.1	0.2
0–2																

Figure 9: Joint Occurrence of Peak Wave Period (Tp) and Mean Wave Direction.

APPENDIX F

Rainfall Distributions for Design Events under the Existing Climate Condition

Appendix F

Rainfall Distributions for Design Events Under the Existing Climate Condition

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1 Appendix F – Rainfall Distributions for Design Events Under the Existing Climate Condition

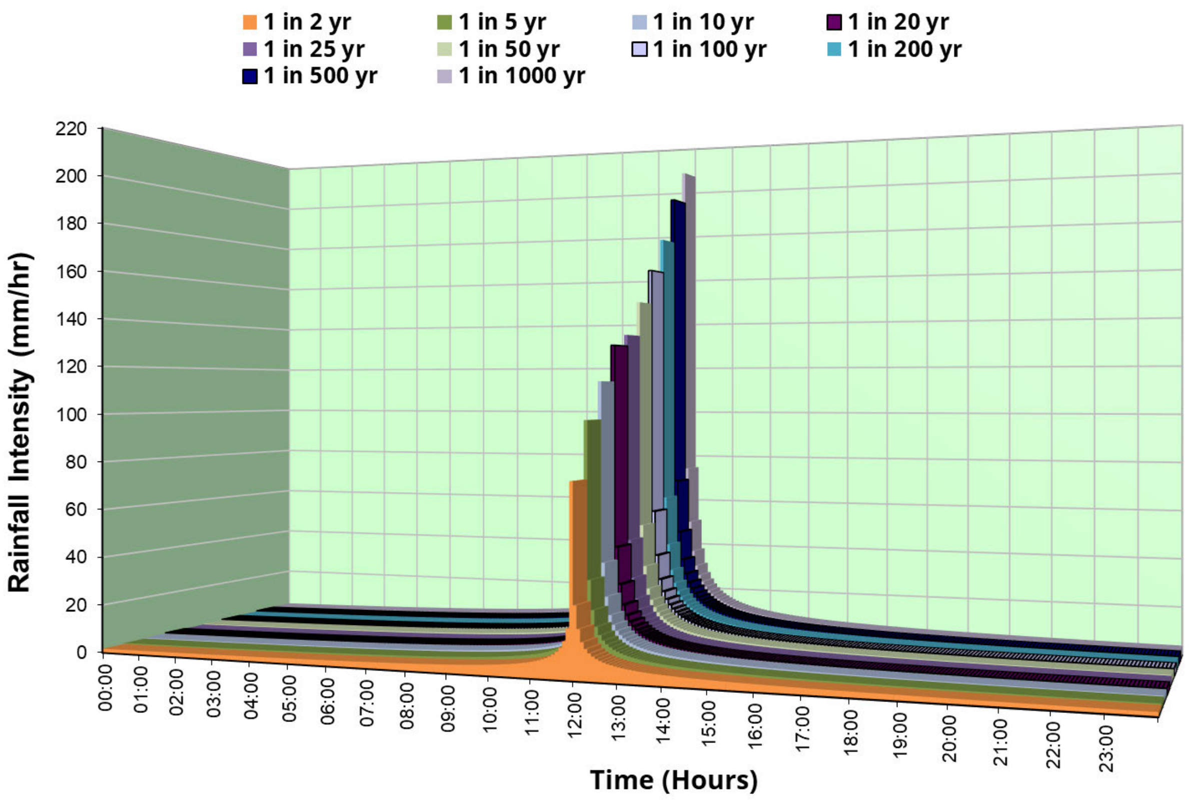

Figure 1: Design Event Rainfall Distributions Under the Existing Condition for the Model 1DG_1

Table 1: Maximum Rainfall Intensity and Total Rainfall of the Design Event Rainfall Distributions Under the Existing Condition for the Model 1DG_1

Return Period	Maximum Rainfall Intensity (mm/h)	Total Rainfall (m)
1 in 2 yr	75.577	87.666
1 in 5 yr	98.437	109.743
1 in 10 yr	113.507	124.411
1 in 20 yr	128.016	137.952
1 in 25 yr	132.639	142.127
1 in 50 yr	146.697	156.302
1 in 100 yr	160.83	169.430
1 in 200 yr	175.062	183.383
1 in 500 yr	193.805	200.731
1 in 1000 yr	207.308	214.715

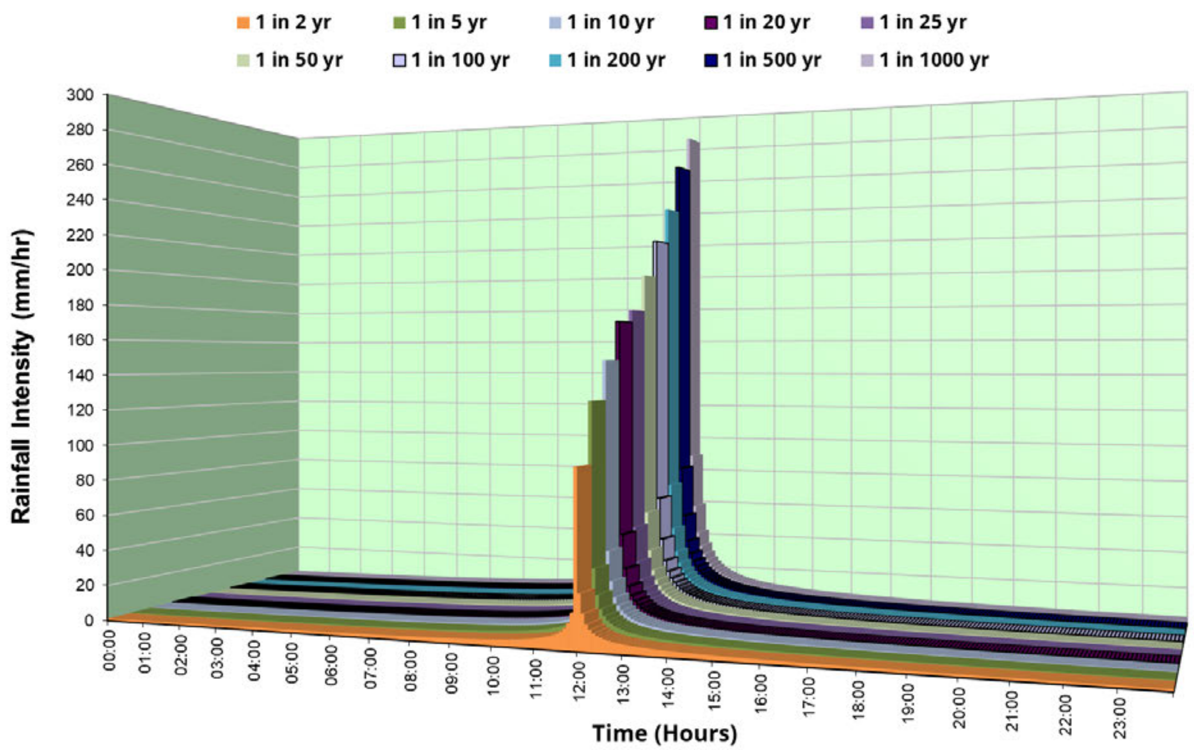

Figure 2: Design Event Rainfall Distributions Under the Existing Condition for the Model 1EO_1

Table 2: Maximum Rainfall Intensity and Total Rainfall of the Design Event Rainfall Distributions Under the Existing Condition for the Model 1EO_1

Return Period	Maximum Rainfall Intensity (mm/h)	Total Rainfall (m)
1 in 2 yr	94.199	73.924
1 in 5 yr	127.26	99.305
1 in 10 yr	148.475	116.518
1 in 20 yr	169.558	133.063
1 in 25 yr	176.286	138.343
1 in 50 yr	196.472	154.184
1 in 100 yr	217.196	169.485
1 in 200 yr	237.432	185.275
1 in 500 yr	264.412	206.329
1 in 1000 yr	284.198	221.769

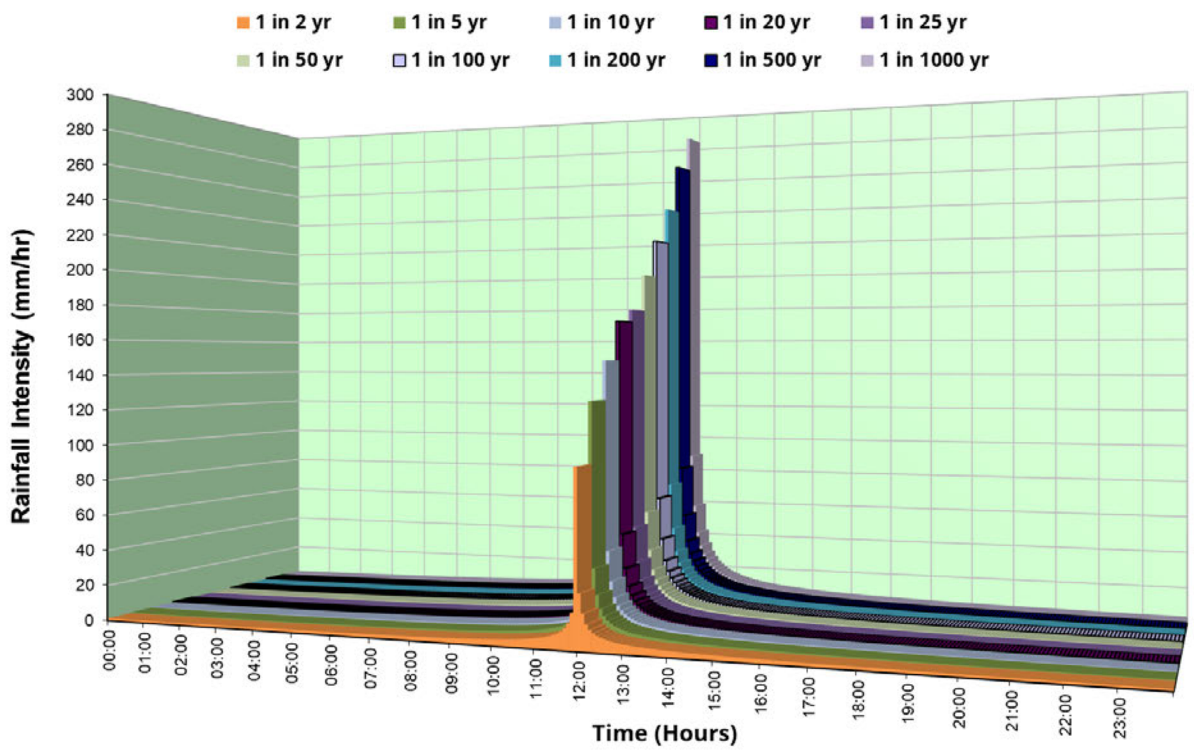

Figure 3: Design Event Rainfall Distributions Under the Existing Condition for the Models 1EN_1, 1EN_2 and 1EN_3

Table 3: Maximum Rainfall Intensity and Total Rainfall of the Design Event Rainfall Distributions Under the Existing Condition for the Models 1EN_1, 1EN_2 and 1EN_3

Return Period	Maximum Rainfall Intensity (mm/h)	Total Rainfall (m)
1 in 2 yr	104.326	81.409
1 in 5 yr	137.261	107.717
1 in 10 yr	159.637	124.570
1 in 20 yr	180.772	141.062
1 in 25 yr	187.517	146.325
1 in 50 yr	208.202	162.467
1 in 100 yr	228.438	178.258
1 in 200 yr	249.123	194.398
1 in 500 yr	275.655	215.102
1 in 1000 yr	296.34	231.244

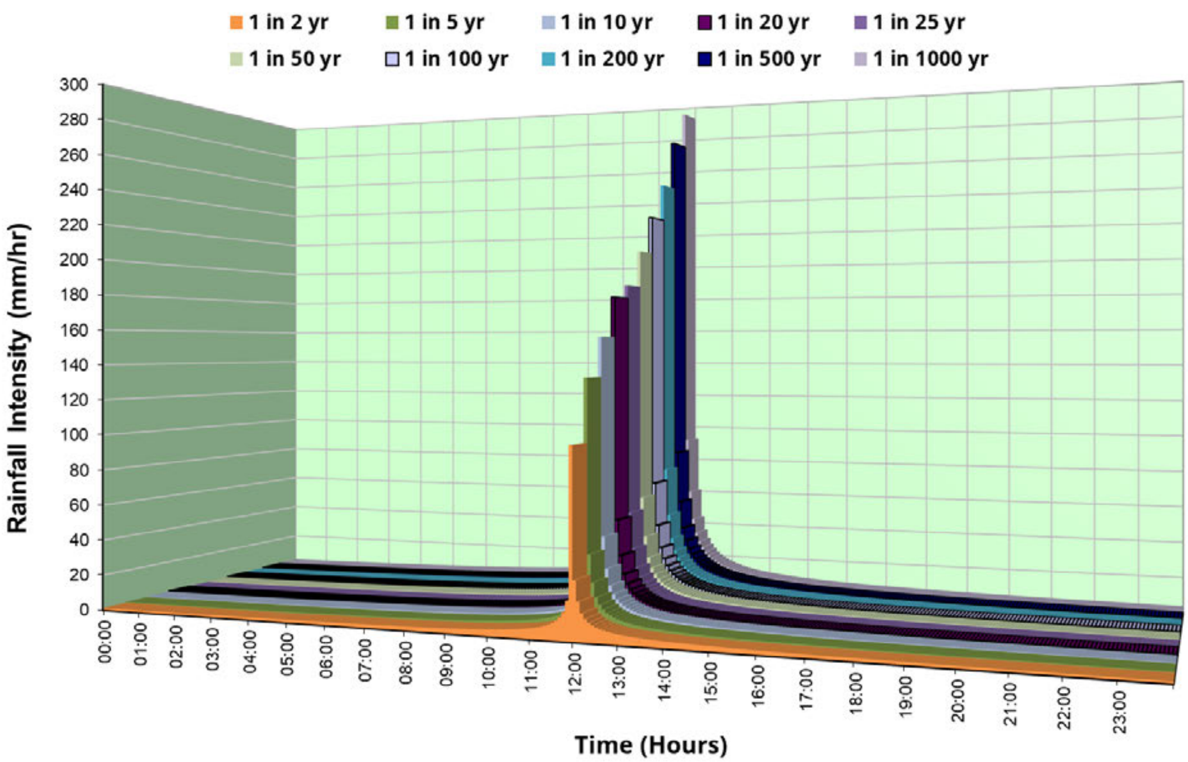

Figure 4: Design Event Rainfall Distributions Under the Existing Condition for the Model 1EN_5

Table 4: Maximum Rainfall Intensity and Total Rainfall of the Design Event Rainfall Distributions Under the Existing Condition for the Model 1EN_5

Return Period	Maximum Rainfall Intensity (mm/h)	Total Rainfall (m)
1 in 2 yr	100.229	79.102
1 in 5 yr	134.121	105.253
1 in 10 yr	155.713	122.892
1 in 20 yr	177.191	139.843
1 in 25 yr	184.36	144.678
1 in 50 yr	204.485	161.384
1 in 100 yr	225.068	177.628
1 in 200 yr	245.651	193.873
1 in 500 yr	272.946	215.413
1 in 1000 yr	293.081	231.306

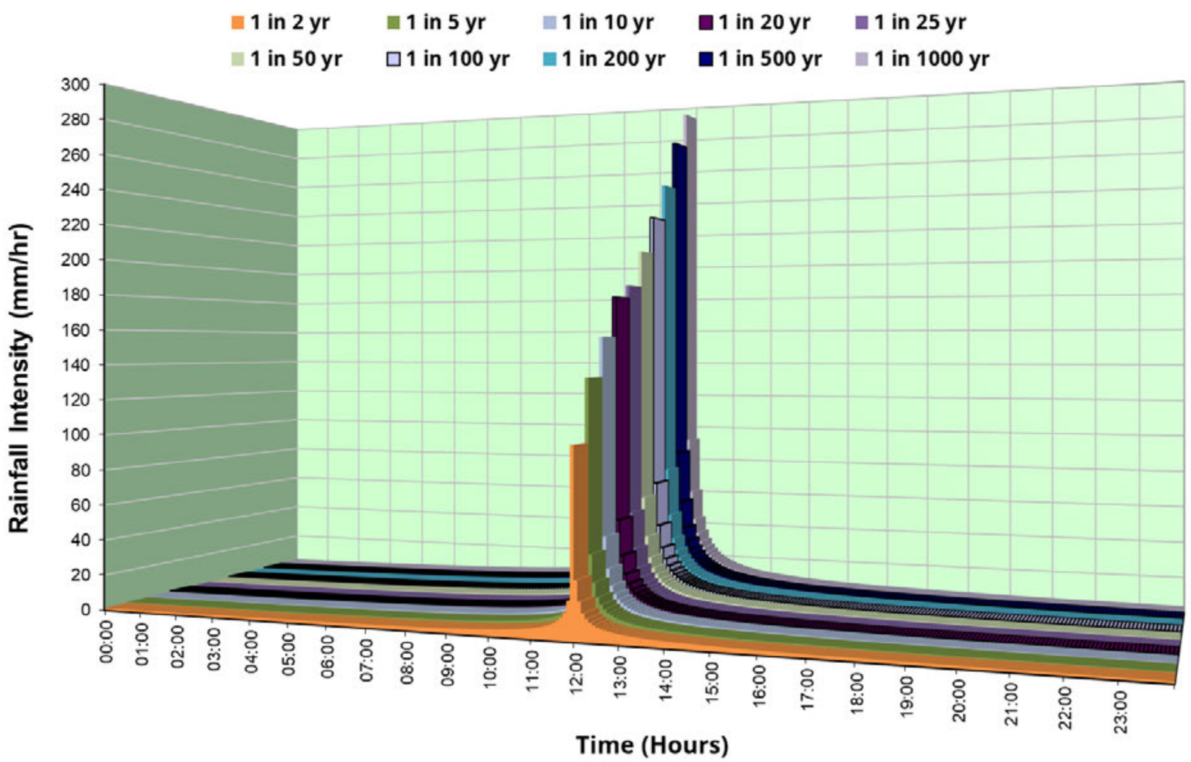

Figure 5: Design Event Rainfall Distributions Under the Existing Condition for the Model 1EM_1

Table 5: Maximum Rainfall Intensity and Total Rainfall of the Design Event Rainfall Distributions Under the Existing Condition for the Model 1EM_1

Return Period	Maximum Rainfall Intensity (mm/h)	Total Rainfall (m)
1 in 2 yr	97.544	76.984
1 in 5 yr	131.43	103.141
1 in 10 yr	153.029	120.773
1 in 20 yr	174.506	137.724
1 in 25 yr	181.669	142.568
1 in 50 yr	201.801	159.264
1 in 100 yr	222.384	175.509
1 in 200 yr	242.966	191.753
1 in 500 yr	269.813	212.942
1 in 1000 yr	290.396	229.186

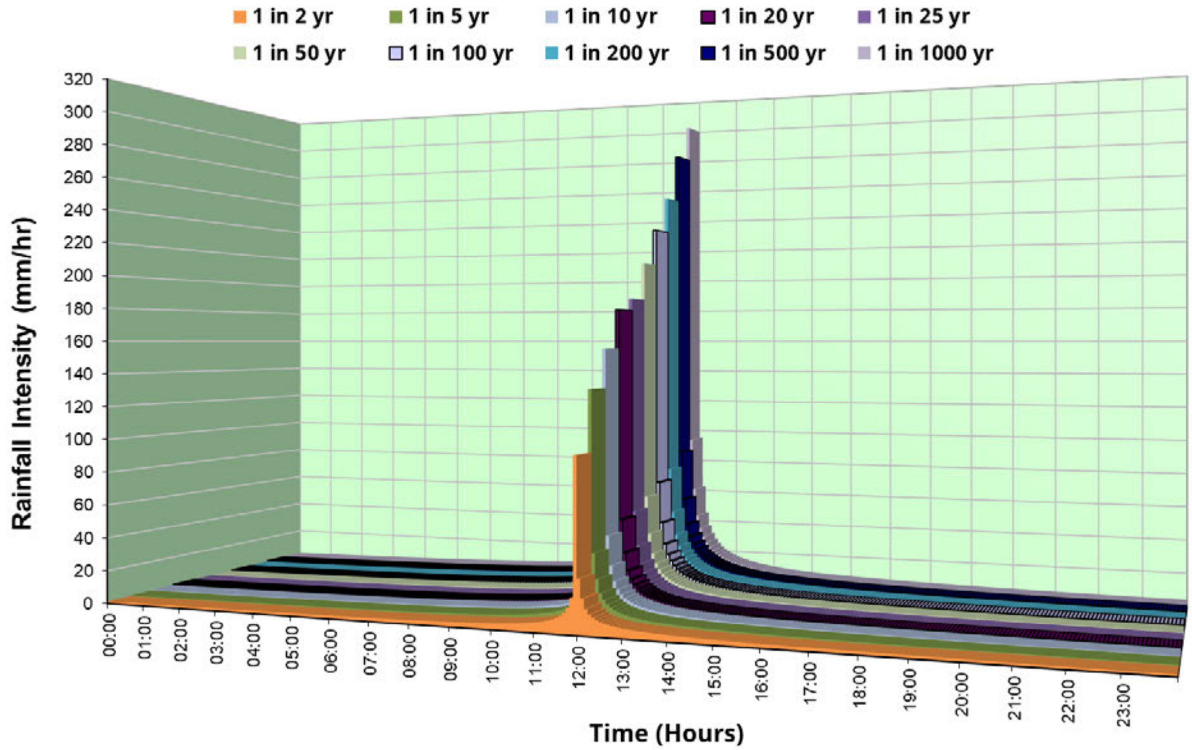

Figure 6: Design Event Rainfall Distributions Under the Existing Condition for the Models 1EL_3, 1EL_4 and 1EL_5

Table 6: Maximum Rainfall Intensity and Total Rainfall of the Design Event Rainfall Distributions Under the Existing Condition for the Models 1EL_3, 1EL_4 and 1EL_5

Return Period	Maximum Rainfall Intensity (mm/h)	Total Rainfall (m)
1 in 2 yr	97.787	76.740
1 in 5 yr	133.224	104.549
1 in 10 yr	155.713	122.892
1 in 20 yr	178.086	140.549
1 in 25 yr	185.245	146.199
1 in 50 yr	207.17	163.504
1 in 100 yr	228.648	180.454
1 in 200 yr	250.126	197.404
1 in 500 yr	278.762	220.005
1 in 1000 yr	300.24	236.956

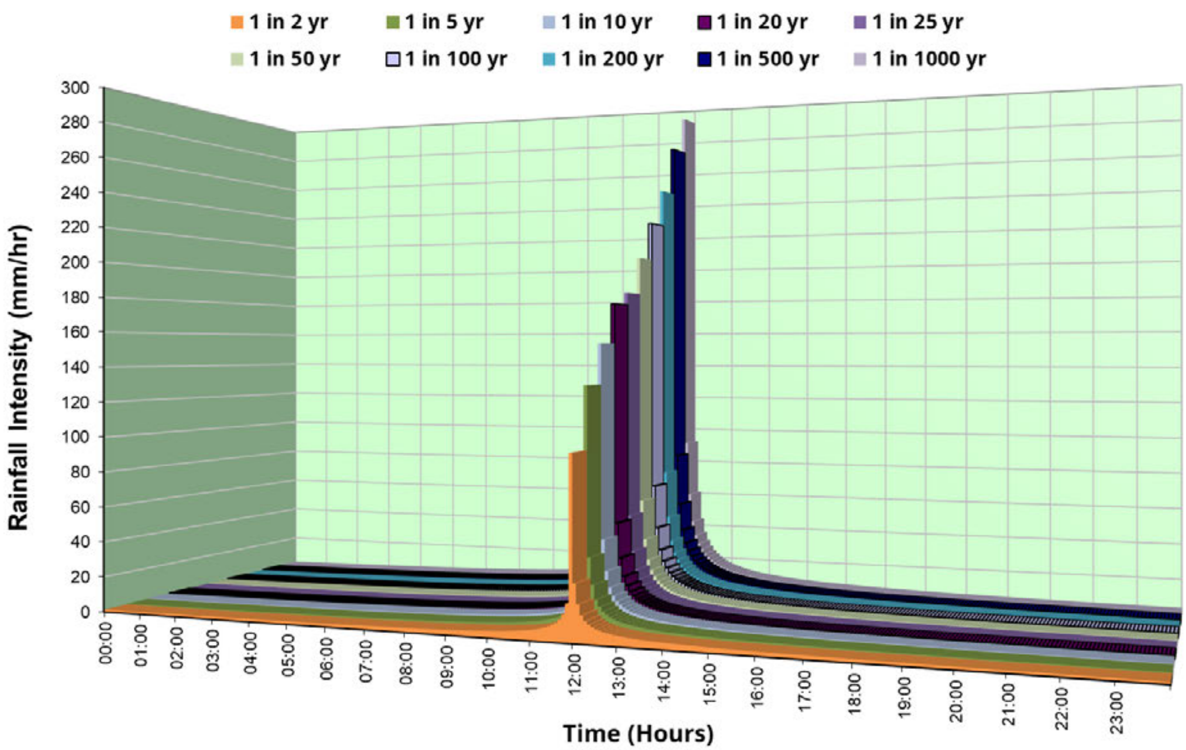

Figure 7: Design Event Rainfall Distributions Under the Existing Condition for the Models 1EL_1, 1EM_2 and 1EM_3

Table 7: Maximum Rainfall Intensity and Total Rainfall of the Design Event Rainfall Distributions Under the Existing Condition for the Models 1EL_3, 1EL_4 and 1EL_5

Return Period	Maximum Rainfall Intensity (mm/h)	Total Rainfall (m)
1 in 2 yr	97.339	76.388
1 in 5 yr	131.43	103.141
1 in 10 yr	153.476	121.127
1 in 20 yr	174.954	138.076
1 in 25 yr	181.665	143.373
1 in 50 yr	202.248	159.619
1 in 100 yr	223.278	176.216
1 in 200 yr	243.861	192.460
1 in 500 yr	271.156	214.001
1 in 1000 yr	292.186	230.599

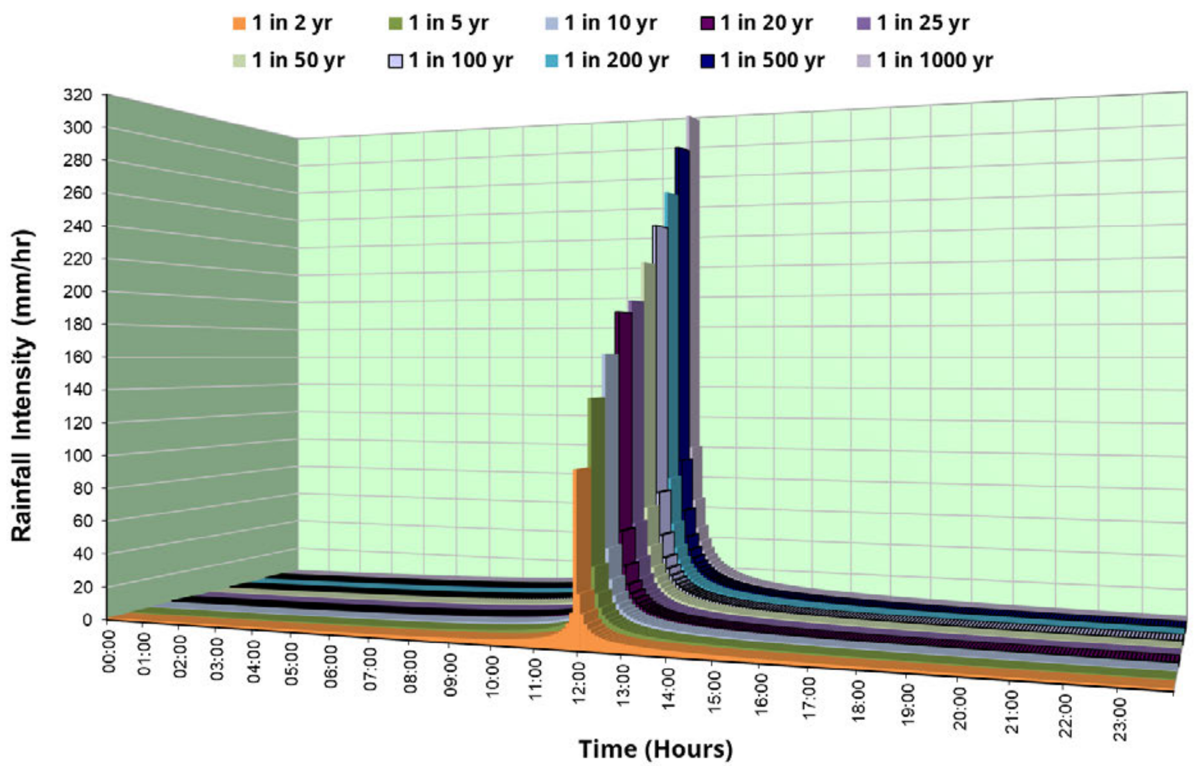

Figure 8: Design Event Rainfall Distributions Under the Existing Condition for the Models 1EK_1, 1EK_2 and 1EK_4

Table 8: Maximum Rainfall Intensity and Total Rainfall of the Design Event Rainfall Distributions Under the Existing Condition for the Models 1EK_1, 1EK_2 and 1EK_4

Return Period	Maximum Rainfall Intensity (mm/h)	Total Rainfall (m)
1 in 2 yr	98.684	77.443
1 in 5 yr	137.261	107.717
1 in 10 yr	161.978	127.835
1 in 20 yr	186.14	146.905
1 in 25 yr	193.747	152.909
1 in 50 yr	217.462	171.625
1 in 100 yr	241.328	189.384
1 in 200 yr	263.997	208.351
1 in 500 yr	294.871	232.717
1 in 1000 yr	318.138	251.080

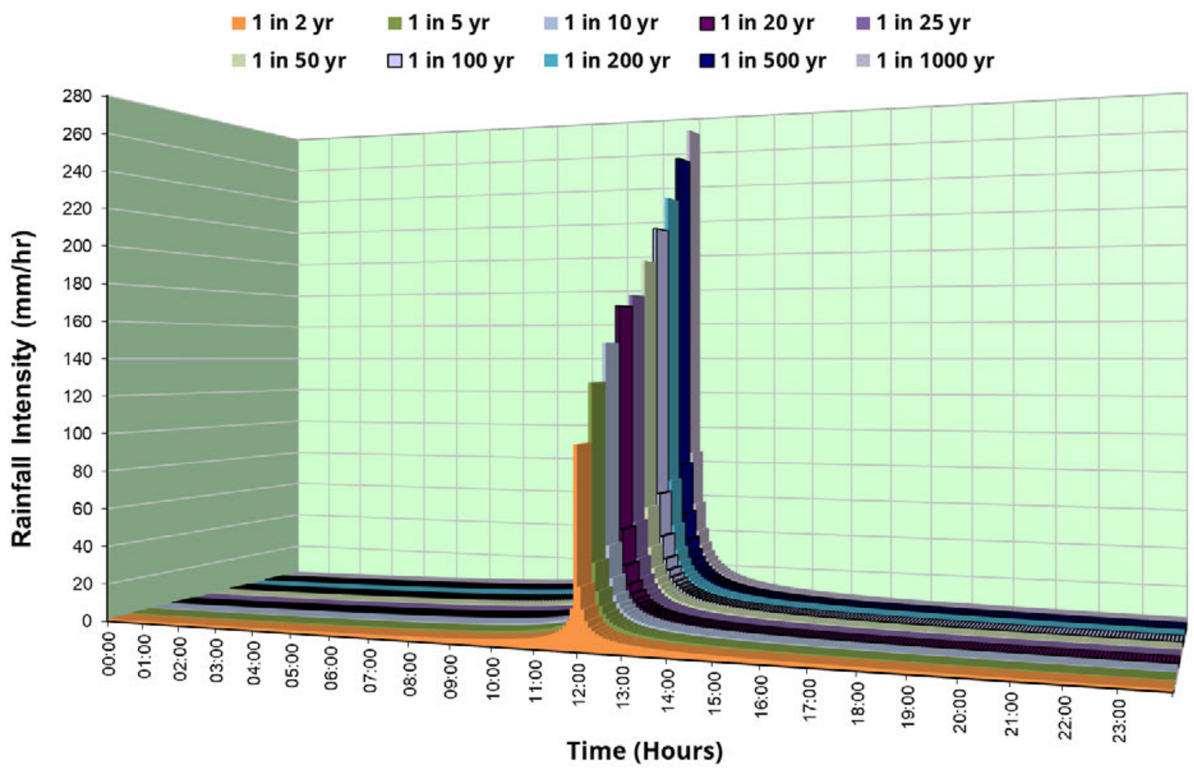

Figure 9: Design Event Rainfall Distributions Under the Existing Condition for the Models 1EJ_2, 1EJ_3, 1EJ_4 and 1EJ_5

Table 9: Maximum Rainfall Intensity and Total Rainfall of the Design Event Rainfall Distributions Under the Existing Condition for the Models 1EJ_2, 1EJ_3, 1EJ_4 and 1EJ_5

Return Period	Maximum Rainfall Intensity (mm/h)	Total Rainfall (m)
1 in 2 yr	99.133	77.796
1 in 5 yr	128.419	101.351
1 in 10 yr	147.739	117.260
1 in 20 yr	166.932	132.493
1 in 25 yr	173.18	137.453
1 in 50 yr	191.927	152.332
1 in 100 yr	210.227	166.857
1 in 200 yr	228.527	181.382
1 in 500 yr	252.629	200.511
1 in 1000 yr	270.929	215.036

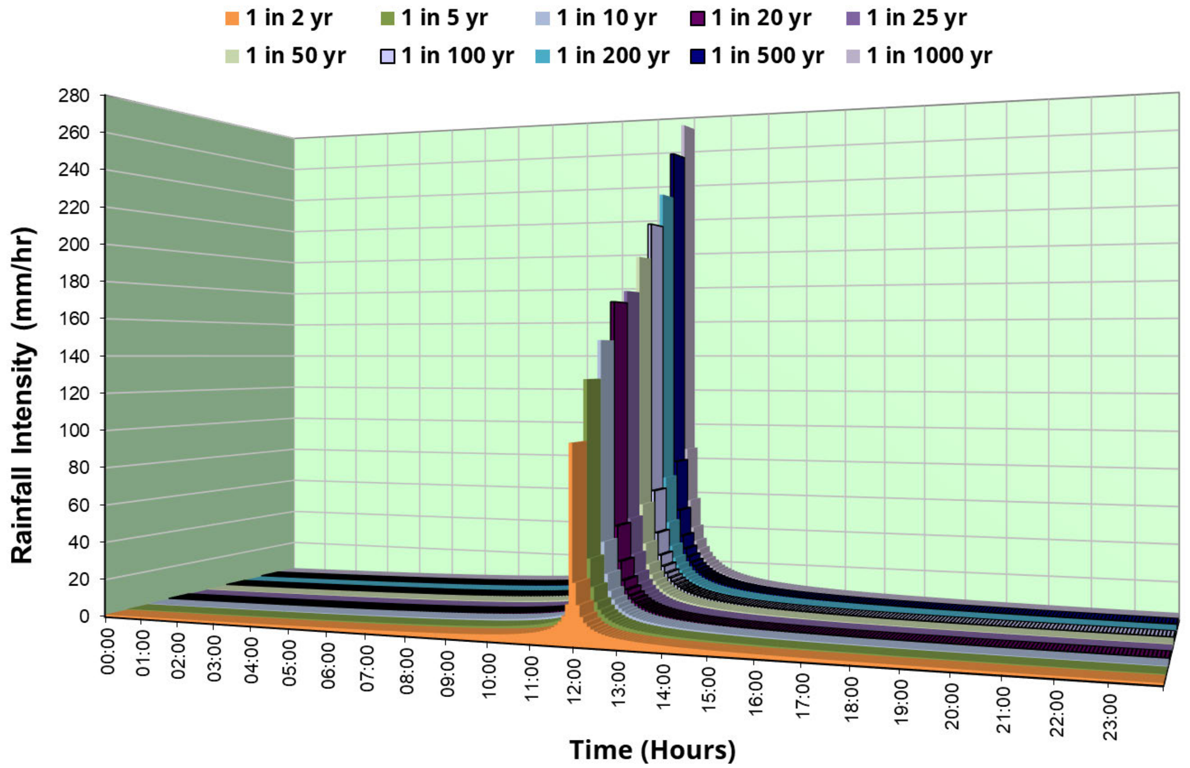

Figure 10: Design Event Rainfall Distributions Under the Existing Condition for the Models 1EH_1, 1EH_2, 1EH_3, 1EH_4 and 1EH_5

Table 10: Maximum Rainfall Intensity and Total Rainfall of the Design Event Rainfall Distributions Under the Existing Condition for the Models 1EH_1, 1EH_2, 1EH_3, 1EH_4 and 1EH_5

Return Period	Maximum Rainfall Intensity (mm/h)	Total Rainfall (m)
1 in 2 yr	98.236	77.092
1 in 5 yr	128.738	101.029
1 in 10 yr	148.924	116.870
1 in 20 yr	169.08	131.938
1 in 25 yr	175.376	136.851
1 in 50 yr	193.78	152.071
1 in 100 yr	213.149	166.327
1 in 200 yr	232.035	181.066
1 in 500 yr	257.218	200.715
1 in 1000 yr	276.104	215.453

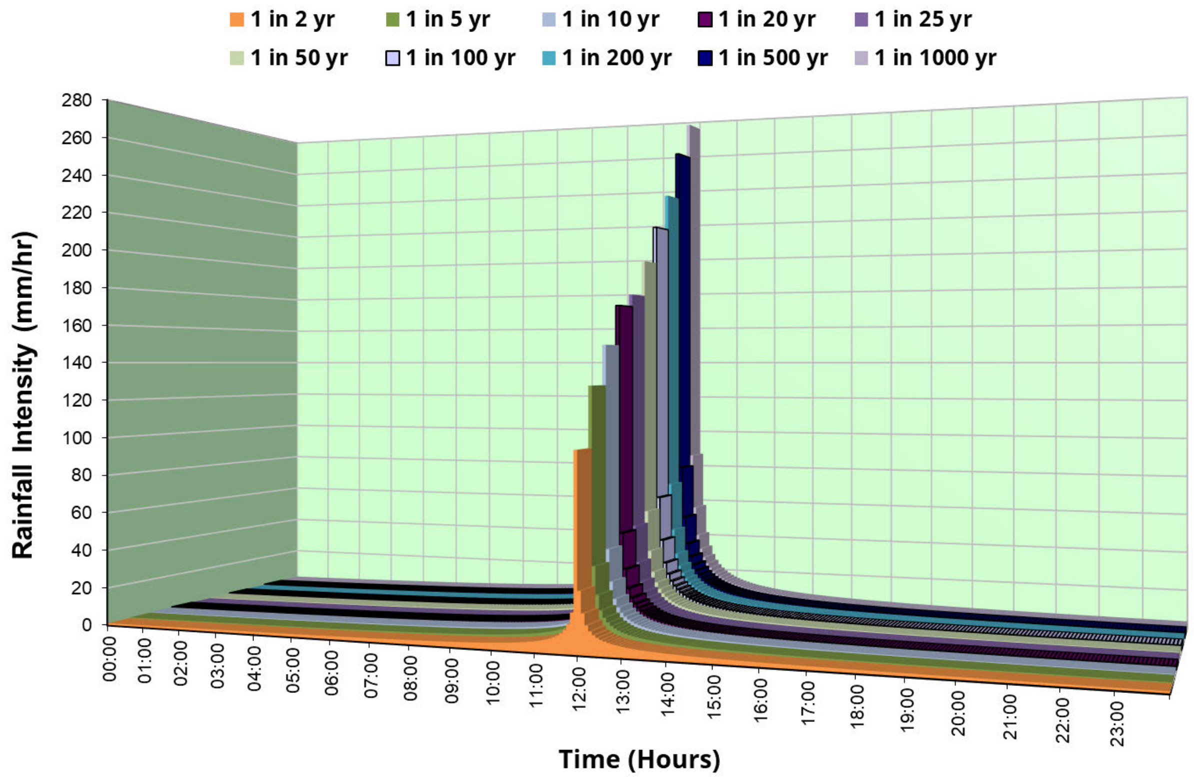

Figure 11: Design Event Rainfall Distributions Under the Existing Condition for the Models 1ED_1 and 1ED_2

Table 11: Maximum Rainfall Intensity and Total Rainfall of the Design Event Rainfall Distributions Under the Existing Condition for the Models 1ED_1 and 1ED_2

Return Period	Maximum Rainfall Intensity (mm/h)	Total Rainfall (m)
1 in 2 yr	98.236	77.092
1 in 5 yr	128.609	100.357
1 in 10 yr	148.026	116.166
1 in 20 yr	167.763	131.653
1 in 25 yr	174.043	136.583
1 in 50 yr	192.913	150.537
1 in 100 yr	211.8	165.274
1 in 200 yr	229.665	180.232
1 in 500 yr	254.336	199.593
1 in 1000 yr	273.176	214.379

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APPENDIX G

Pluvial-Fluvial Model Sensitivity Analysis

Appendix G

Pluvial-Fluvial Model Sensitivity Analysis

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1 Appendix G – Pluvial-Fluvial Model Sensitivity Analysis

1.1 Introduction

Sensitivity analysis is the process of adjusting various model parameters to understand their influence on the modelled results. In addition, since assumptions were made for the structures that do not have available diameter or width information, this assessment includes the evaluation of the influence of structure openings to the modelled results.

This assessment was carried out relative to the 1 in 100 year rainfall event under the existing climate condition. Each primary watershed has one calibrated model selected for this assessment. The outcome of this analysis is to provide a wider perspective of parameter and structure opening uncertainty for the modelled water levels and flows.

1.2 Scenarios

A total of 9 scenarios were considered for this assessment, as listed below.

1. Adjusted subcatchment roughness to be ±50% of the calibrated values.
2. Adjusted subcatchment soil conductivity to be ±50% of the calibrated values.
3. Adjusted subcatchment soil conductivity to be 0.00001 to simulate frozen ground.
4. Adjusted channel roughness to be ±50% of the calibrated values.
5. Adjusted structure openings to be ±30% of the estimated values.

1.3 Assessment Results

The resulting changes in the maximum total inflows and maximum water elevations were averaged for all junctions in each selected model, as presented in Table 1 and Table 2, respectively. Table 3 and Table 4 present the maximum increase and decrease in the maximum water elevation for all junctions in each selected model. The average changes in maximum total inflows for all selected models range from -20.55% to 152.67%, and the average changes in maximum water elevations range from -0.2 m to 0.22 m. The maximum increase in the maximum water elevation is 2.73 m, and the maximum decrease in the maximum water elevation is – 4.11 m.

It was found that subcatchment roughness and channel roughness have larger impacts on the modelled results for all selected models than the soil conductivity and structure openings, except for the model 1EK_1, which was significantly impacted by the soil conductivity. It was also noticed that the changes in the parameters led to instabilities of models on some junctions (parameters were unrealistic), which resulted in a rapid rise of flows that is unreasonable and skewed the assessment results.

Table 1: Average Changes in Maximum Total Inflow for All Junctions in Each Model

Variable	Variable Range	Average Changes in Maximum Total Inflows for all Junctions in each Model (%)								
		1DE_2	1DG_1	1EH_1	1EJ_5	1EK_1	1EL_4	1EM_3	1EN_1	1EO_1
Subcatchment Roughness	50%	-18.13%	-20.55%	-15.78%	-9.72%	-12.33%	-9.92%	-10.99%	-12.13%	-5.08%
	-50%	37.99%	48.64%	0.56%	18.50%	29.59%	13.72%	21.89%	20.64%	26.28%
Subcatchment Soil Conductivity	50%	-2.38%	0.00%	-0.73%	-0.17%	-12.90%	-2.29%	0.11%	-0.11%	-0.68%
	-50%	3.10%	0.23%	0.56%	1.02%	46.21%	-1.09%	-0.09%	0.47%	0.68%
	0.00001	5.86%	0.12%	1.14%	1.11%	152.67%	0.00%	0.85%	0.73%	2.06%
Channel Roughness	50%	-3.09%	-2.07%	-6.36%	-3.17%	-4.59%	-5.82%	-3.45%	-5.44%	-5.08%
	-50%	33.68%	16.99%	26.70%	14.26%	18.47%	8.31%	14.28%	17.55%	15.54%
Structure Openings	30%	0.27%	2.77%	2.25%	1.41%	2.78%	-0.85%	0.15%	0.34%	0.62%
	-30%	0.15%	1.63%	-0.18%	-0.12%	1.97%	-2.39%	-0.93%	0.60%	0.04%

Table 2: Average Changes in Maximum Water Elevations for all Junctions in each Model

Variable	Variable Range	Average Changes in Maximum Water Elevations for all Junctions in each Model (m)								
		1DE_2	1DG_1	1EH_1	1EJ_5	1EK_1	1EL_4	1EM_3	1EN_1	1EO_1
Subcatchment Roughness	50%	-0.04	-0.10	-0.04	-0.03	-0.03	-0.03	-0.03	-0.04	0.10
	-50%	0.07	0.17	0.00	0.04	0.05	0.02	0.06	0.06	0.15
Subcatchment Soil Conductivity	50%	-0.01	0.00	0.00	0.00	-0.04	-0.02	0.00	0.00	-0.01
	-50%	0.01	0.00	0.00	0.00	0.08	-0.01	0.00	0.00	0.01
	0.00001	0.02	0.00	0.00	0.00	0.22	0.00	0.01	0.00	0.01
Channel Roughness	50%	0.03	0.04	0.03	0.03	0.02	0.02	0.05	0.05	0.10
	-50%	-0.05	-0.05	-0.03	-0.05	-0.01	-0.07	-0.05	-0.10	-0.20
Structure Openings	30%	0.00	0.00	-0.01	-0.01	-0.01	-0.02	-0.01	0.00	0.00
	-30%	0.00	0.01	0.02	0.00	0.01	-0.01	0.03	0.01	0.00

Table 3: Maximum Increase in Maximum Water Elevation for all Junctions in each Model

Variable	Variable Range	Maximum Increase in Maximum Water Elevation for all Junctions in each Model (m)								
		1DE_2	1DG_1	1EH_1	1EJ_5	1EK_1	1EL_4	1EM_3	1EN_1	1EO_1
Subcatchment Roughness	50%	0.06	0.09	0.00	0.01	0.08	0.08	0.01	0.00	0.48
	-50%	0.43	1.84	0.01	0.33	0.91	0.49	0.51	0.97	0.36
Subcatchment Soil Conductivity	50%	0.04	0.04	0.01	0.01	0.01	0.03	0.03	0.14	0.01
	-50%	0.37	0.08	0.01	0.08	1.09	0.04	0.03	0.12	0.04
	0. 00001	0.43	0.05	0.03	0.08	2.11	0.00	0.07	0.12	0.05
Channel Roughness	50%	0.47	0.80	0.47	0.39	0.48	0.58	0.32	0.50	0.48
	-50%	0.51	0.95	0.36	0.37	0.47	0.41	0.73	0.63	0.29
Structure Openings	30%	0.09	0.91	0.07	0.18	0.15	0.16	0.10	0.04	0.03
	-30%	0.51	2.73	0.66	0.68	1.72	1.49	0.74	0.38	0.05

Table 4: Maximum Decrease in Maximum Water Elevation for all Junctions in each Model

Variable	Variable Range	Maximum Decrease in Maximum Water Elevation for all Junctions in each Model (m)								
		1DE_2	1DG_1	1EH_1	1EJ_5	1EK_1	1EL_4	1EM_3	1EN_1	1EO_1
Subcatchment Roughness	50%	-0.46	-1.21	-0.63	-0.64	-0.60	-0.50	-0.27	-0.27	-0.14
	-50%	-0.03	-0.24	-0.01	-0.05	-0.11	-0.05	-0.01	-0.01	-0.04
Subcatchment Soil Conductivity	50%	-0.25	-0.20	-0.07	-0.03	-0.78	-0.24	-0.04	-0.09	-0.03
	-50%	-0.03	-0.06	-0.01	-0.03	-0.15	-0.24	-0.01	-0.11	-0.01
	0. 00001	-0.02	-0.05	-0.02	-0.64	-0.13	0.00	-0.01	-0.14	-0.01
Channel Roughness	50%	-0.55	-0.42	-1.26	-1.18	-0.22	-0.29	-0.18	-0.25	-0.14
	-50%	-0.65	-0.80	-0.65	-0.64	-0.51	-0.63	-0.49	-0.56	-0.80
Structure Openings	30%	-0.09	-1.67	-0.96	-4.11	-1.27	-0.80	-0.36	-0.32	-0.05
	-30%	-0.03	-0.46	-0.09	-0.44	-0.27	-0.24	-0.19	-0.12	-0.01

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