# GOVERNMENT OF NEWFOUNDLAND AND LABRADOR

# FLOOD-RISK MAPPING PROJECT
# PORTUGAL COVE – ST. PHILIP'S

# FINAL REPORT

Submitted to:

**Water Resources Management Division**
**Department of Environment and Conservation**
**Government of Newfoundland and Labrador**
St. John's, NL

Submitted by:
**Amec Foster Wheeler Environment & Infrastructure,**
**a Division of Amec Foster Wheeler Americas Limited**
St. John's, NL

May 2015

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May 25, 2015

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Dr. Amir Ali Khan, P.Eng., Manager
Hydrologic Modelling Section
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Department of Environment and Conservation
Government of Newfoundland and Labrador
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St. John's, NL A1B 4J6

Dear Sir:

**Re: Final Report**
**Flood-Risk Mapping Study Project**
**Portugal Cove – St. Philip's, NL**

Amec Foster Wheeler Environment & Infrastructure (Amec Foster Wheeler) is pleased to provide the above-noted Final Report.

We would like to thank you for the opportunity to provide our services to the Government of Newfoundland and Labrador

Yours truly,

**Amec Foster Wheeler Environment & Infrastructure,**
**a Division of Amec Foster Wheeler Americas Limited**

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HX/cjy

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# LIMITATIONS

1. This report was prepared exclusively for the Government of Newfoundland and Labrador, Department of Environment and Conservation, Water Resources Management Division (WRMD) by Amec Foster Wheeler Environment & Infrastructure, a Division of Amec Foster Wheeler Americas Limited (Amec Foster Wheeler).

2. The work performed in this report was carried out in accordance with the Standard Terms of Conditions made part of our contract. The conclusions presented herein are based solely upon the scope of services and time and budgetary limitations described in our contract.

3. The report was prepared in accordance with generally accepted environmental study and/or engineering practices for the exclusive use of WRMD. No other warranties, either expressed or implied, are made as to the professional services provided under the terms of our contract and included in this report.

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6. The contents of this report are based on the information collected during a review of available background information, interviews, site inspection and investigation activities, our understanding of the actual site conditions, and our professional opinion according to the information available at the time of preparation of this report. This report gives a professional opinion and, by consequence, no guarantee is attached to the conclusions or expert advice depicted in this report. This report does not provide a legal opinion in regards to Regulations and applicable Laws.

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## LIST OF ACRONYMS

| | |
|---|---|
| AEP | Annual Exceedence Probability |
| AHCCD | Adjusted and Homogenized Canadian Climate Data |
| Amec Foster Wheeler | Amec Foster Wheeler Environment & Infrastructure, a Division of Amec Foster Wheeler Americas Limited |
| ARC | Antecedent Runoff Condition |
| BC | Beachy Cove Brook |
| BR | Broad Cove River |
| BUI | Bouline Soil Group |
| CanSIS | Canadian Soil Information Service |
| CDED | Canadian Digital Elevation Data |
| CN | Curve Number |
| COH | Cochrane Soil Group |
| COL | Colinet Soil Group |
| DEM | Digital Elevation Model |
| FDRP | Flood Damage Reduction Program |
| FHA | Federal Highway Administration |
| FXH | Fox Harbour Soil Group |
| GC | Goat Cove Brook |
| GIS | Geographic Information System |
| GNL | Government of Newfoundland and Labrador |
| HEC-GeoHMS | Hydrologic Engineering Center-Geospatial Hydrological Modelling Extension |
| HEC-GeoRAS | Hydrologic Engineering Center-Geospatial River Analysis System |
| HEC-HMS | Hydrologic Engineering Center-Hydrologic Modelling System |
| HEC-RAS | Hydrologic Engineering Center-River Analysis System |
| HMS | Hydrologic Modelling System |
| IDF | Intensity, Duration, and Frequency |
| IMA&SN | Inundation Mapping Area and Stream Network |
| $km^2$ | Square kilometres |
| LEGEO | Leading Edge Geomatics |
| MLS® | Multiple Listing Service® |
| MR | Main River |
| MTA | Multiple-Time-Around |
| NL | Newfoundland and Labrador |
| NLCEL | Newfoundland and Labrador Consulting Engineers Ltd. |
| PCSP | Portugal Cove – St. Philip's |
| PUV | Pouch Cove Soil Group |
| RDSA | Residential Development Scheme Area |
| RFFA | Regional Flood Frequency Analysis |
| RFP | Request for Proposal |
| RLD | Residential Low Density |
| RMD | Residential Medium Density |
| RoW | Right-of-Way |
| SAL | Salmonier Soil Group |
| SCS | Soil Conservation Service |

| | |
|---|---|
| TBY | Torbay Soil Group |
| UN | Unnamed Stream |
| USACE | U.S. Army Corps of Engineers |
| WRMD | Water Resources Management Division |
| WSC | Water Survey Canada |
| WV2 | WorldView 2 |

## EXECUTIVE SUMMARY

The objectives of this study were to produce 1:20-year and 1:100-year inundation and flood-risk maps by taking into consideration the expected changes in development conditions and climatic conditions in the study areas. Conceptually, a flood-risk mapping study consists of three major components: hydrology, hydraulics, and topographic mapping. The hydrologic component involves the determination of the response of a watershed to major meteorological events such as rainstorms, rapid snowmelt, or a combination of both. The output from the hydrologic component, in the form of flood flows for specified probabilities, serves as the major input in the hydraulic analysis. The hydraulic analysis defines the response of the selected river reaches to the hydrologic input. The output from the hydraulic analysis, in the form of water surface profiles is applied to detailed topographic maps to delineate the extent of flood water levels on the flood plain.

The Town of Portugal Cove – St. Philip's (Town of PCSP) is located on the Avalon Peninsula west of the City of St. John's, spreading over 56.4 km$^2$. The study area of 68.7 km$^2$ encompasses five rivers/streams and their tributaries listed below:

- Main River (18.0 km$^2$);
- Unnamed Stream (0.4 km$^2$);
- Beachy Cove Brook (8.6 km$^2$);
- Goat Cove Brook(2.6 km$^2$); and
- Broad Cove River (39.1 km$^2$).

The primary study tasks for this project can be summarized as follows:

- Conduct a review of existing information relevant to the flood-risk mapping study.
- Identify the areas and waterbodies (streams and lakes) for which inundation mapping will be required. This task was necessary since a portion of the study area is rural and protected for which developments are limited or restricted. The municipal and watershed boundaries adopted for this study also do not overlap.
- Conduct a field program to collect LiDAR and other geometric data required for establishing the hydrologic and hydraulic models, and to collect other information required for calibration and verification of these models.
- Conduct a remote sensing analysis to characterize the land cover conditions in the study watersheds, and to combine the land cover conditions with the distribution of soil types to characterize the precipitation loss in response to a storm event.
- Determine 1:20-year and 1:100-year flood flows at locations of interest using a statistical approach and deterministic models.
- Establish hydraulic models to determine water surface profiles and inundation areas in response to 1:20-year and 1:100-year storm flows.
- Evaluate the effect of climatic change on the peak flows and inundation areas.

- Prepare inundation and flood-risk maps for specified precipitation (rainfall and/or snowmelt), development, and climatic change scenarios.
- Evaluate the hydraulic capacities of the existing stream-crossing structures.

For the purpose of this study, the perennial stream network as shown on the 1:50,000 topographic maps was adopted. The inundation modelling covers the area within the municipal boundary with the following exclusions:

- Areas inside the municipal boundary but outside the overall watershed boundary (e.g., Brock's Pond, Ocean Pond); and
- Areas delineated in the future municipal plan as Rural and Protected Watershed where developments are sparse or will be restricted in the future.

The waterbodies identified for inundation modelling include numerous streams as well as lakes. The modelling approach taken is designed to consider the different requirements for lakes and streams. In this approach, the peak water level in the lakes in response to the hydrological events is simulated using Hydrologic Engineering Center-Hydrologic Modelling System (HEC-HMS), which considers the entire inflow hydrograph. A lake is simulated as a level pool, and variation of water level through the lake is considered negligible. The lakes in the study area are generally small and this approximation is acceptable. The peak water levels in the streams in response to the peak flows are simulated using Hydrologic Engineering Center-River Analysis System (HEC-RAS), and the peak water levels (and the corresponding flows) for the lakes are used as boundary conditions in the simulation.

A field survey program was conducted to collect various data required for the establishment and calibration of the hydrological and hydraulic models. A critical requirement in the production of the inundation mapping is the availability of highly accurate topographic data. To meet this requirement, a LiDAR survey was conducted covering the areas within the municipal boundary. Additionally, field surveys were conducted to collect information relating to stream channel geometry (stream-crossing structures and cross-sections), water levels and flows.

A remote sensing analysis was conducted to estimate the distribution of US Soil Conservation Service (SCS) Curve Number (CN) for the five study watersheds to be used as input in the subsequent hydrological modelling. SCS CN is an index of the runoff generation potential of a basin and is a function of soil drainage characteristics and land use/cover conditions. A geo-referenced land classification map covering the five study watersheds was developed through this analysis. This land classification map was then combined with the available soil classification map to generate a map delineating the distribution of CN. The CN distribution for future development conditions was also estimated.

A hydrologic analysis was conducted with the following objectives:

- To determine the 1:20-year and 1:100-year Annual Exceedence Probability (AEP) flow estimates at points of interest. These flows were subsequently used as input to the hydraulic model to estimate flood levels for the Inundation Mapping Area and Stream Network (IMA&SN).
- To determine the peak water levels for the lakes under 1:20-year and 1:100-year AEP flow conditions.

The Water Resources Management Division (WRMD) requires that the 1:20-year and 1:100-year AEP flows be determined using statistical analysis as well as deterministic modelling approach. There are advantages and disadvantages for each of the two approaches. The statistical approach is simple to use and the required flows can be determined quickly and conveniently. The deterministic modelling approach considers the significant variables contributing to the hydrological processes in a watershed.

An update of the Regional Flood Frequency Analysis (RFFA) for Newfoundland and Labrador (NL) was completed by Amec Foster Wheeler in September 2014. The study watersheds are located within the South East Hydrological Region defined by the RFFA study. The 1:20-year and 1:100-year flows were estimated for each of the locations where five pressure transducers were installed as part of the field programs, and for the location of the hydrometric station 02ZM006 located within the study watersheds.

The 1:20-year and 1:100-year AEP flow estimates were simulated using a deterministic numerical model. The WRMD requires that the non-proprietary US Army Corps of Engineers (USACE) Hydrologic Center's Hydrological Modelling System (HEC-HMS) be used for simulating the hydrological behaviour within the study area. Additionally, the WRMD requires that the Geospatial Hydrological Modelling Extension of the HEC-HMS model, the HEC-GeoHMS, be used for preparation of geometric data in Esri ArcGIS and for generation of the hydrological inputs for import into the HEC-HMS.

The HEC-HMS model included a basin model and meteorological model. The basin models were prepared within HEC-GeoHMS. Input into the basin model includes topographic parameters, storage and discharge characteristics for lakes and ponds, and baseflow. The meteorological input includes updated Intensity, Duration, and Frequency (IDF) data. The established hydrological models were calibrated by comparing results with RFFA, and with measured data for selected storm events. The established hydrological models were used for generating flows used in the subsequent hydraulic modelling. Additionally, water levels in the lakes and ponds within the IMA&SN in response to the 1:20-year and 1:100-year rainfall events were simulated using the hydrological models.

The hydraulic modelling was conducted to simulate water surface profiles resulting from the 1:20-year and 1:100-year AEP flows. For a given location, the water level elevation in response to the peak flood flow is determined by the geometric characteristics (dimensions and frictions to flow) of the channel and flood plain as well as by the magnitude of the flows. The WRMD requires that for simulating the hydraulic behaviour of the streams, the latest version of the non-proprietary HEC-RAS and the Geo-RAS extension be used. The simulation of water surface profiles using HEC-RAS and HEC-GeoRAS involved the following steps:

- Preparation of geometric input using HEC-GeoRAS and high accuracy LiDAR and field survey data and import the geometric data into HEC-RAS.
- Preparation of flows and boundary conditions for the HEC-RAS model. The flows were obtained in the hydrological modelling. The boundary conditions included the sea level and outflow from the lakes and ponds.
- Model calibration, simulation and export modelling results to HEC-GeoRAS.
- Delineation of inundation zones using HEC-GeoRAS and Geographic Information System (GIS) tools.

The 1:20-year and 1:100-year water level profiles were simulated with HEC-RAS, which were then integrated with the water levels for the lakes and ponds. The computed water surface elevations were then used in conjunction with the LiDAR topographic data to delineate inundation zones within the IMA&SN.

The established hydrological and hydraulic models were used to simulate climatic change conditions in a similar fashion as for the current conditions. The climatic change projections were based on the Finnis (2012) report. Using output from the hydrological and hydraulic models, inundation and flood hazard mapping was generated for 1:20-year and 1:100-year flow for current and future climatic conditions.

The hydraulic capacities of the stream-crossing structures within the IMA&SN were evaluated based on the following two criteria:

- the surface of the road at the crossing site should not be overtopped during the design storm event; and
- the backwater from the stream-crossing structure should not cause flooding of upstream residences.

The 1:20-year and 1:100-year water level elevations at the stream-crossing sites for current development and climate conditions were determined from the hydraulic model and compared with the above criteria.

Key recommendations are as follows:

1. It is recommended that the GNL implement a program to survey the peak flood water levels following significant flood events. Proper documentation of these data will provide valuable information for future inundation studies, and for updating existing flood studies. A significant constraint for this inundation study is the lack of suitable calibration data for the hydraulic model. The most ideal calibration data for this purpose should consist of peak flood water levels in various areas of the watersheds. These data are best collected shortly after significant flood events.

2. It is recommended that standards be developed regarding the identification of inundation modelling areas and the associated stream network, especially when numerous small streams are involved. In this study, the perennial stream network shown on 1:50,000 topographic map was adopted as the modelling stream network. The WRMD may wish to adopt this as the standards, or other standards at its discretion.

3. It is recommended that the LiDAR survey should be required to cover the entire watershed areas to avoid the difficulties associated with working with different DEMs.

4. It is recommended that the CN for a particular land classification and soil type be given as a range, rather than a discrete value, to enable calibration of the hydrological model. Mechanisms could be developed to enable consideration of the effect of a shallow soil profile and the permeability of the underlying bedrock on the selection of the CN.

## TABLE OF CONTENTS

PAGE

## TABLE OF CONTENTS (cont)

PAGE

## LIST OF TABLES



## LIST OF FIGURES

## LIST OF APPENDICES

# 1.0 INTRODUCTION

In Canada, flood plain management falls under the jurisdiction of the provinces, as they are primarily responsible for water resources and land use matters. One of the roles of the federal government is to reduce major disruptions to regional economies and to reduce disaster assistance payments. Traditionally, this had been achieved by building structural measures to control flooding. In the 1950s, 1960s, 1970s, and to a lesser extent in the 1980s, the federal government allocated millions of dollars, in conjunction with the provinces, to build dams and dykes. Extensive flood damages across Canada in the early 1970s clearly demonstrated that a new approach to reducing flood damage was needed. These flood events were the catalyst for the federal government to initiate the national Flood Damage Reduction Program (FDRP) in 1975 under the *Canada Water Act*. The FDRP had been carried out under cost-shared federal-provincial agreements.

In 2011, a new 3-year Climatic Change Adaptation Initiative was announced by the Province of Newfoundland and Labrador (NL) to update and undertake new flood-risk mapping studies. The flood studies are managed through the Water Resources Management Division (WRMD), Department of Environment and Conservation, Government of Newfoundland and Labrador (GNL). The flood-risk mapping study for the Town of PCSP was proposed under this initiative. The previous flood-risk mapping study for the Town of PCSP was conducted in 1996.

Flood-risk maps have been incorporated into a wide range of applications including public safety, infrastructure design, water resources management, environmental assessment, land use development, municipal and development planning, setting of structural design criteria, and response to floods. As the global climate changes in the coming decades, sustainable management of water resources will be critical. One potential result of climate change is an increase in flooding. Floods have the potential to cause significant personal injury, loss of property, loss of life and disruption of transportation systems. To assist with planning in and around potential flood-risk areas and to minimize damages associated with flooding, information on the projected spatial extent and expected frequency and severity of floods is crucial.

## 1.1 Objectives

The objectives of this study were to produce 1:20-year and 1:100-year Annual Exceedence Probability (AEP) inundation and flood-risk maps by taking into consideration the expected changes in development and climatic conditions in the study areas. Conceptually, a flood-risk mapping study consists of three major components: hydrology, hydraulics, and topographic mapping. The hydrologic component involves the determination of the response of a watershed to major meteorological events such as rainstorms, rapid snowmelt, or a combination of both. The output from the hydrologic component, in the form of flood flows for specified probabilities, serves as the major input in the hydraulic analysis. The hydraulic analysis will define the response of the selected river reaches to the hydrologic input, out of basin water diversions, and any other pertinent factors. The output from the hydraulic analysis, in the form of water surface profiles is applied to detailed topographic maps to delineate the extent of flood water levels.

WRMD developed a Technical Document for Flood-Risk Mapping Studies, which serves as the technical guidelines for this project. Additional guidelines were provided in the document "Hydrologic and Hydraulic Procedures for Flood Plain Delineation" (Environment Canada, 1976).

## 1.2 Study Area

The Town of PCSP is located on the Avalon Peninsula west of the City of St. John's. The location and boundary of the Town are shown in Figure 1.1. Since its amalgamation in 1992, the Town of PCSP is one of the largest municipalities in NL, having an area of over 56.4 $km^2$. The study area within the Town identified in Figure 1.1 encompassed five rivers/streams and their tributaries listed below:

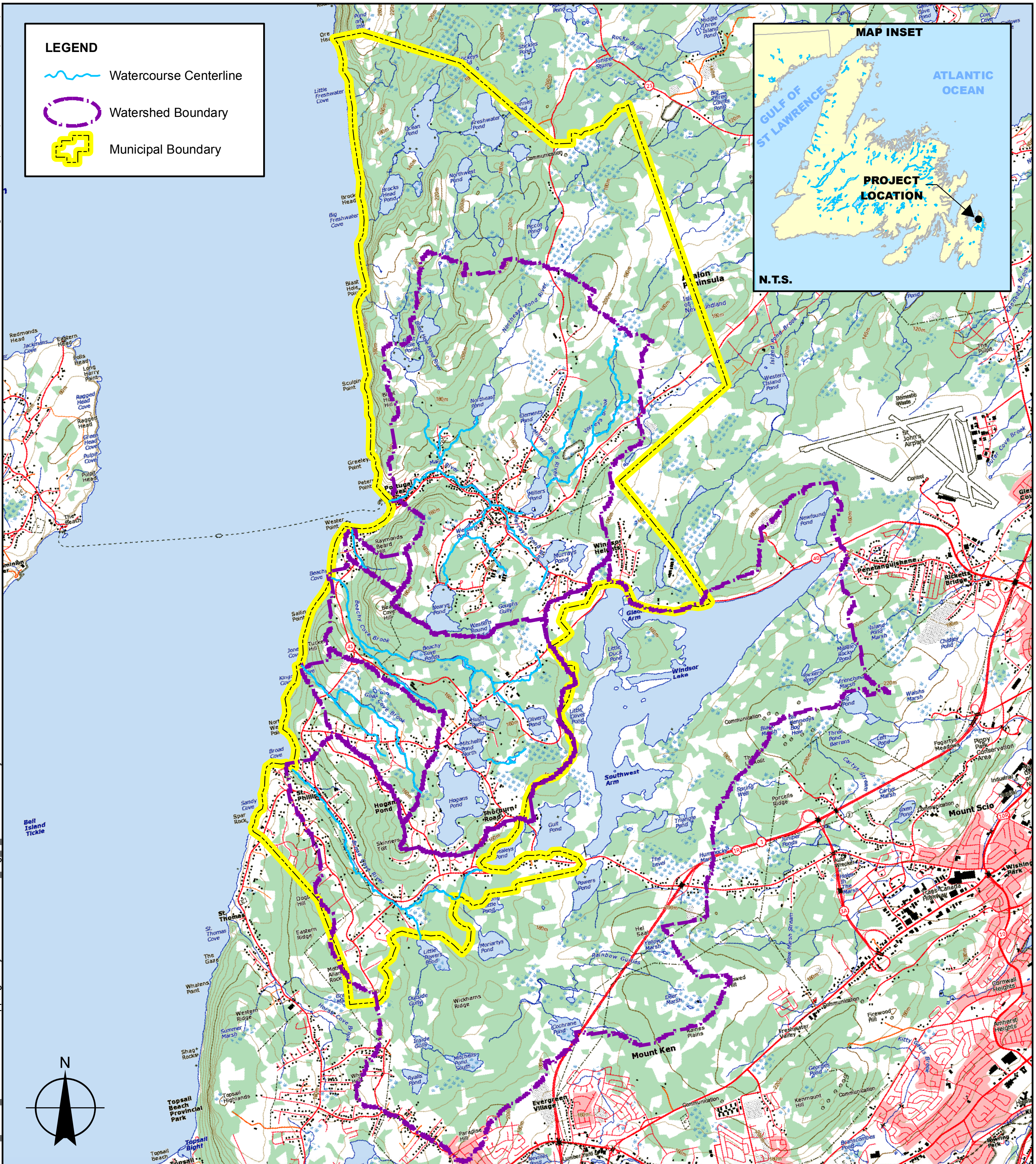
- Main River (18.0 $km^2$);
- Unnamed Stream (0.4 $km^2$);
- Beachy Cove Brook (8.6 $km^2$);
- Goat Cove Brook (2.6 $km^2$); and
- Broad Cove River (39.1 $km^2$).

The total area of the watersheds is 68.7 $km^2$.

## 1.3 Work Scope

The primary study tasks for this project can be summarized as follows:

- Conduct a review of existing information relevant to the flood-risk mapping study (Section 2).
- Identify the areas and waterbodies (streams and lakes) for which inundation mapping will be required (Section 3). This task was necessary since a portion of the study area is rural and protected for which developments are limited or restricted. The municipal and watershed boundaries referenced for this study also do not overlap.
- Conduct a field program to collect LiDAR and other geometric data required for establishing the hydrologic and hydraulic models, and to collect other information required for calibration and verification of these models (Section 4).
- Conduct a remote sensing analysis to characterize the land cover conditions in the study watersheds, and to combine the land cover conditions with the distribution of soil types to characterize the precipitation loss in response to a storm event (Section 5).
- Determine 1:20-year and 1:100-year AEP flood flows at locations of interest using a statistical approach and deterministic models (Section 6).
- Establish hydraulic models to determine water surface profiles and inundation areas in response to 1:20-year and 1:100-year storm flows (Section 7).




SOURCE: Geogratis NTS Topo Map / PCSP Municipal Topo Date: 19/05/2015 User: jonathan.thornton

Path: G:\GIS\PROJECTS\TE144022_GovNL_PCSP_2014FloodRiskMappingStudy\MXD\20150514_Fig_1_1.mxd

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| CLIENT: WATER RESOURCES MANAGEMENT DIVISION DEPARTMENT OF ENVIRONMENT AND CONSERVATION GOVERNMENT OF NEWFOUNDLAND AND LABRADOR | SCALE: 0 800 1,600 3,200 Metres | PROJECT: FLOOD RISK MAPPING STUDY PORTUGAL COVE - ST. PHILIP'S | DWN BY: JT |
| | | | CHK'D BY: HX |
| amec foster wheeler | DATUM: NAD 83 CSRS | TITLE: STUDY AREAS | DATE: MAY, 2015 |
| | PROJECTION: MTM ZONE 1 | | REV. NO: 0 |
| | PROJECT NO: TE144022 | | FIGURE NO: 1.1 |

- Evaluate the effect of climatic change on the peak flows and inundation areas (Section 8);
- Prepare inundation and flood-risk maps for specified precipitation, development, and climatic change scenarios (Section 9);
- Evaluate the capacities of the existing stream-crossing structures (Section 10); and
- Develop recommendations (Section 11).

# 2.0 BACKGROUND INFORMATION

## 2.1 Description of the Study Area

The following sections discuss the elements of the hydrological cycle in the study area that contribute to runoff generation and flooding.

### 2.1.1 Climatic and Hydrological Setting

The Avalon Peninsula (9,220 km²) makes up the southeast portion of the island of Newfoundland. The mean annual precipitation over much of the peninsula is approximately 1,400 mm decreasing to approximately 1,000 mm along the extreme northern coast and extreme western coast. A summary of the monthly rainfall and precipitation distribution and their extremes is provided in Table 2.1 based on meteorological records collected at the St. John's International Airport. The monthly rainfall and precipitation distribution is also shown in Figure 2.1. The heaviest rainfall occurs from August through December. In August and September, the possibility exists for tropical storms to affect southern Newfoundland. If these warm air cyclonic systems track close enough to the island, intense rain can fall over a short period.

**Table 2.1 Monthly Rainfall and Precipitation Distribution and their Extremes**

| Description | Jan | Feb | Mar | Apr | May | Jun | Jul | Aug | Sep | Oct | Nov | Dec | Year |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| **Rainfall (mm)** | 66.0 | 61.6 | 84.8 | 96.1 | 97.9 | 97.5 | 91.6 | 100.0 | 129.6 | 153.7 | 124.8 | 102.9 | 1206.4 |
| **Snowfall (cm)** | 88.7 | 71.0 | 57.3 | 25.3 | 4.4 | 0.0 | 0.0 | 0.0 | 0.0 | 2.4 | 22.4 | 63.4 | 335.0 |
| **Precipitation (mm)** | 149.2 | 129.5 | 142.2 | 122.9 | 102.6 | 97.6 | 91.6 | 100.0 | 129.6 | 156.2 | 148.1 | 164.8 | 1534.2 |
| **Average Snow Depth (cm)** | 18 | 32 | 22 | 6 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 7 | |
| **Extreme Daily Rainfall (mm)** | 84.6 | 67.1 | 71.4 | 91.7 | 81.3 | 75.2 | **121.2** | 80.5 | 99.4 | 100.8 | 97.2 | 85.1 | |
| **Extreme Daily Snowfall (cm)** | 60.1 | 54.9 | 50.0 | **68.4** | 25.4 | 13.5 | 0.0 | 0.0 | 0.3 | 19.8 | 25.3 | 49.3 | |
| **Extreme Daily Precipitation (mm)** | 84.6 | 68.3 | 72.0 | 91.7 | 83.1 | 75.2 | **121.2** | 80.5 | 99.4 | 100.8 | 97.2 | 85.1 | |
| **Extreme Snow Depth (cm)** | 144 | **180** | 133 | 105 | 30 | 3 | 0 | 0 | 0 | 9 | 36 | 64 | |

Source: http://climate.weather.gc.ca/climate_normals/results_1981_2010_e.html?stnID=6720&lang=e&StationName=st.+john%27s&SearchType=Contains&stnNameSubmit=go&dCode=1

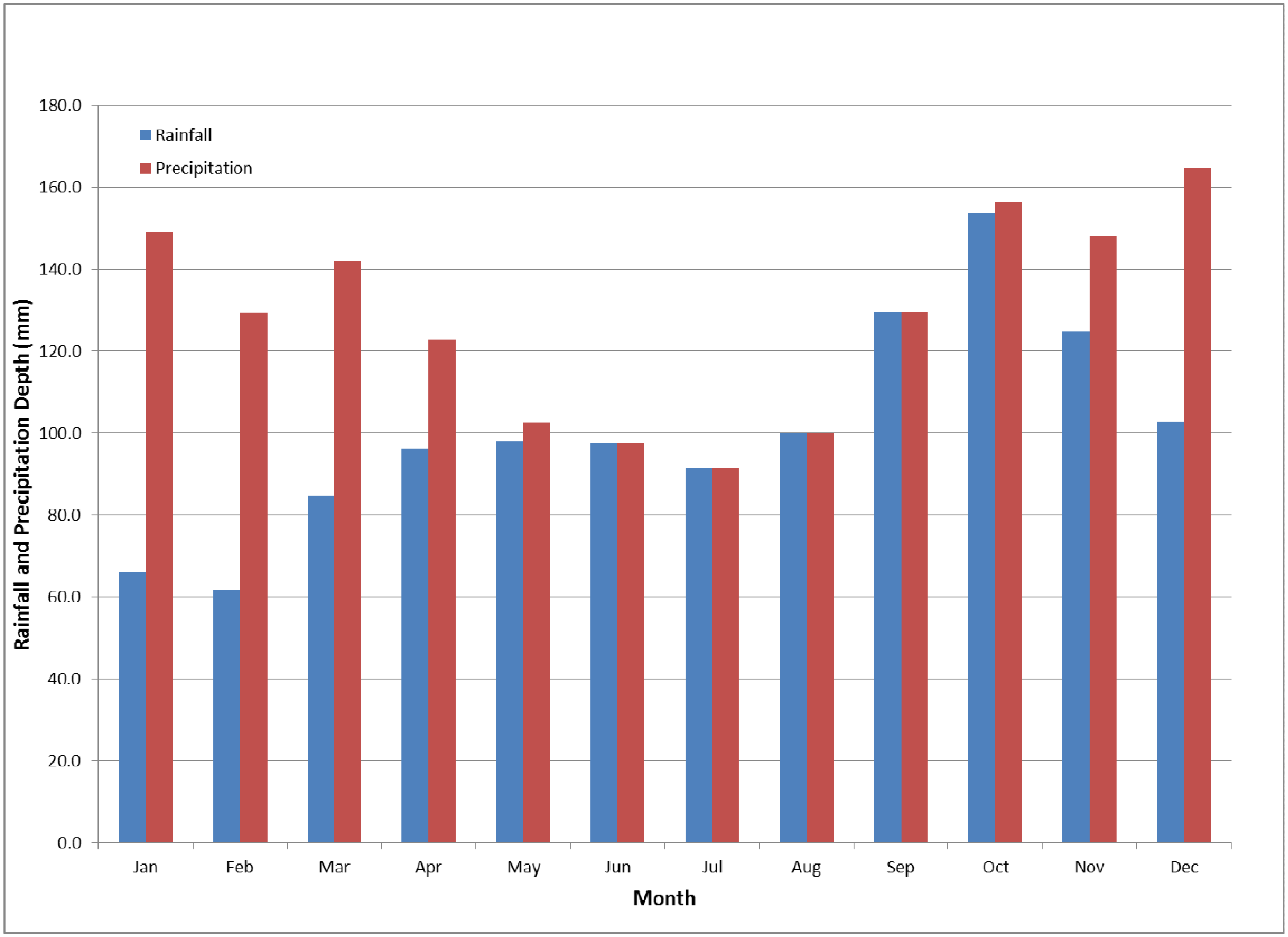



**Figure 2.1 Monthly Rainfall and Precipitation Distribution for St. John's International Airport (Station No: 8403505)**

During October and November, the increasing temperature contrast between the freshly snow-covered northern areas, and the much milder southern coastal districts and offshore waters, results in an intensification of the mid-latitude cyclonic storm systems that approach the Province from the southwest. Consequently, this period is characterized by a marked upswing in the frequency of strong winds and precipitation.

In the spring, the dominant north winds decrease cyclonic activity, which in turn decreases the number of intense rainfall events. The months of March to May are generally not associated with frequent flooding events, except for those involving ice jams at a local level. In the spring, easterly airflow patterns are generally associated with cold air, and a remaining snow pack, which increases the occurrence of rain on snow events. However, the Avalon Peninsula is generally characterized by relatively mild winters with considerable variation in snow cover and snow water equivalent. For most communities in this area, rainfall is the leading cause of flooding.

### 2.1.2 Physiographic Setting

Much of the Avalon Peninsula is characterized by barren, irregular and rough topography, with several peaks over 250 m high (http://www.env.gov.nl.ca/env/waterres/cycle/hydrologic/nl.html). Much of the coastline rises abruptly from the sea and is indented with numerous bays and inlets (http://cfs.nrcan.gc.ca/pubwarehouse/pdfs/32584.pdf). These topographic characteristics result in steep coast lines and rocky/bouldery stream channels, and give rise to the development of waterfalls (Figures 2.2 through 2.4). The steep coast line protects the inland areas from flooding caused by waves and storm surges originating from the Atlantic Ocean.


**Figure 2.2 Steep Coastline at the Discharge of an Unnamed Stream into Conception Bay**


**Figure 2.3 Main River near Churchill's Road**


**Figure 2.4 Waterfall on Main River near its Discharge into Conception Bay**

Nearly 40% of the peninsula is forest covered. The peninsula was glaciated and the overburden deposits consist generally of ground moraine, outwash, and other glaciofluvial deposits.

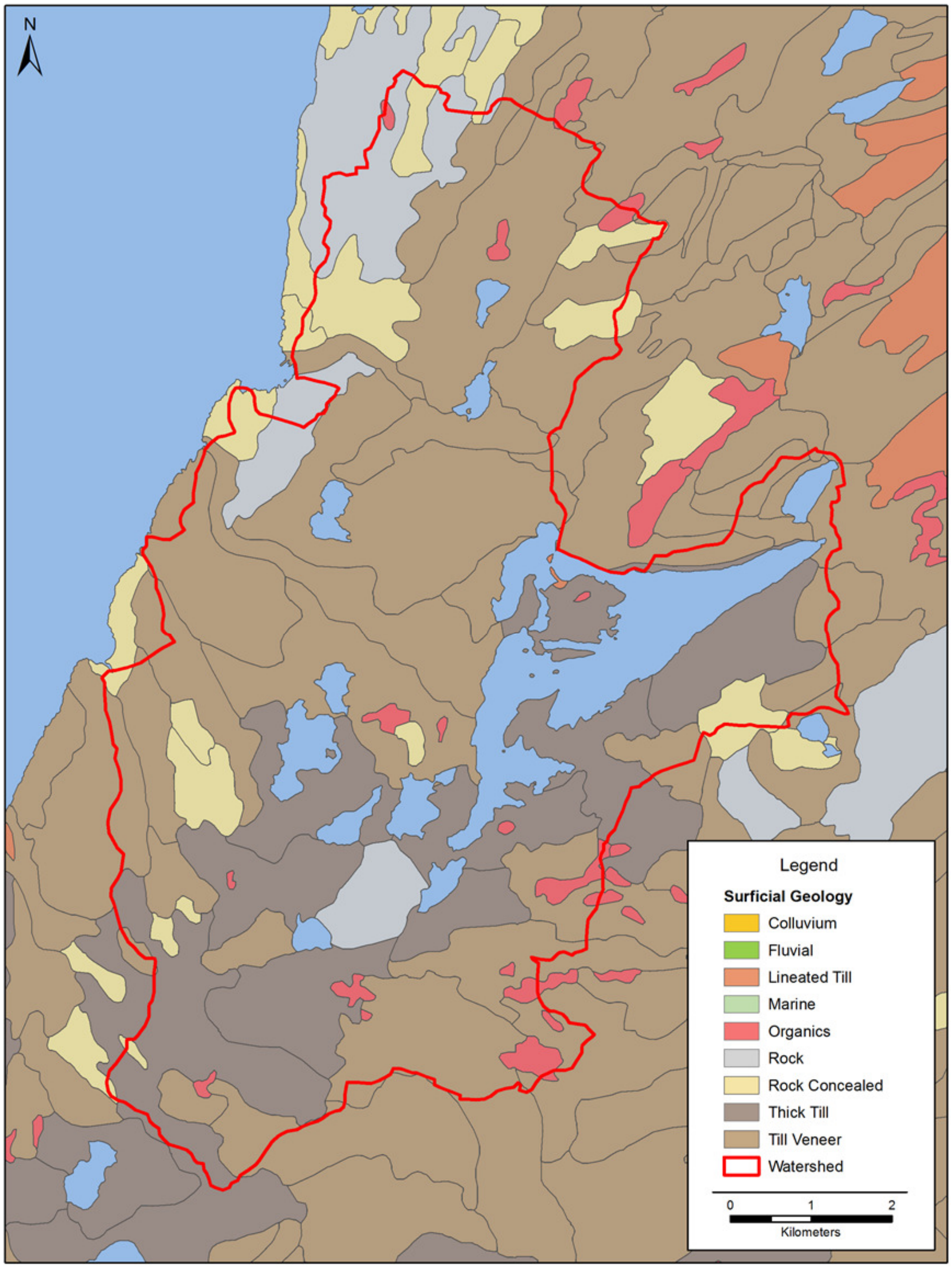
The land area of the Town of PCSP consists of approximately 5,970 hectares (59.7 $km^2$). A significant portion of the land area of the Town of PCSP is rural ((http://pcsp.ca/userfiles/files/TRACT%20-%20PC-SP%20Municipal%20Plan%20(2014-2024),%20September,%202014.pdf). There are a significant number of lakes and ponds within the municipal boundary and immediate surrounding areas, with Windsor Lake being the largest and most dominant.

### 2.1.3 Overburden and Bedrock Geology

The geology of the study area potentially has a significant effect on the hydrology of the area. Geologically, the study area can be divided into upper and lower portions, where the upper portion retains significant surface water that recharges the groundwater through permeable bedrock and overburden layers, as well as feeding the rivers. The lower portion is located on faulted layers of rock that are very resistant to weathering with very little overburden cover. Approximately 70% of the study watersheds has a surficial geology designation of rock, concealed rock, and till veneer which indicates very little overburden cover, most of which is located in the lower portion of the study area (Figure 2.5).

All the lakes and ponds within the study area (including Windsor Lake, Powers Pond, Gull Pond, Healeys Pond, Hogans Pond, Olivers Pond, Mitchells Pond, Hughs Pond, and Nearys Pond) are located within a geological syncline structure (King, 1990). The Mistaken Point Formation is the top geological layer of the syncline structure and consists of irregular argillaceous siltstones and sandstones that are considered an important bedrock groundwater recharge feature. The harder sandstones of the Drook Formation are located below the Mistaken Point Formation. The syncline structure also acts as a basin that contains thick (between 5-20 m) hummocky till and organic layers that act as localized recharge areas (King, 1990).

In the lower portion of the study area, the topographically steep rock formations include basalts of the Harbour Main Group, and thrusted northeast to southwest faults that uplifted layers of the Drook Formation. The Main River and Broad Cove River, Goat Cove Brook, and Beachy Cove Brook follow localized northwest-southeast trending faults that are perpendicular to the northeast to southwest thrust faults. Overburden thickness and infiltration capacity are limited in the lower portion of the study area, given the steep topography of the underlying bedrock layers. Significant bare rock exists in the lower portion, with till veneer layers reported to be less than 3 m in thickness (King, 1990).




Source: Batterson, M.J. 2000.

**Figure 2.5 Surficial Geology of the Study Area**

### 2.1.4 Existing and Planned Future Development

According to the 2011 Census, the Town's population grew from 6,575 residents in 2006 to 7,366 residents in 2011; a growth rate that is six times greater than the Province as a whole. The geographical diversity of development patterns within the community has resulted in a relatively low population density.

In 2014, the Municipal Plan 2014-2024 for the Town of PCSP was being finalized. The Plan establishes guidelines and policies for managing future growth and development of the Town by providing a land use strategy for the 10-year period from 2014 to 2024. The Municipal Plan outlines Council's policies for overall land use development for provision of community amenities, and for watershed and environmental protection. Maps showing the generalized future land use from 2014 to 2024 are presented in Appendix A.

## 2.2 Historical Flooding and Flood Prone Areas

### 2.2.1 Historical Flooding

The Flood Events Inventory maintained by WRMD was acquired and reviewed, which documents the flood and flood damages experienced by the Province since 1950. This inventory contains information on a large number of floods affecting various communities and areas around the Avalon Peninsula, including a flood event experienced in the Town of PCSP on April 11, 1986. This event was caused by 77 mm rain falling in 11 hours. The severity of the flood was exacerbated by additional snowmelt and frozen ground conditions. The inventory also contains a record of a 1986 ice-jam flood on the Broad Cove River.

### 2.2.2 Flood-Prone Areas

Mr. Chris Milley, the Town Manager, provided locations and a detailed narrative of the flood-prone areas within the Town of PCSP. Maps showing the flood-prone areas and a narrative outlining the flood and flood damages experienced in these areas are provided in Appendix B.

It should be noted that while some of the identified flood-prone areas are associated with the flood water overtopping the banks of the streams, many of the cited flooding issues are associated with localized drainage (e.g. surface runoff causing flooding and erosion while making its way to the stream). Local drainage issues are best addressed through proper drainage investigation, planning, and management. These issues are not within the scope of this study.

## 2.3 Previous Studies

Several previous studies are available that cover or otherwise contain relevant information for this project. A brief summary of the studies and relevant information is provided as follows:

**Flood-risk Mapping Study of the Town of PCSP, and Outer Cove, March 1996:** This study was commissioned by the then Newfoundland Department of Environment and conducted by CORETEC Incorporated in association with Davis Engineering & Associates Limited. This study included modelling to estimate the 1:20-year and 1:100-year flood profiles and delineate the inundation areas. Inundation areas delineated in this study are included in Appendix B.

**Town of PCSP Storm Water Management Plan, Murray's Pond Brook/Main River and Broad Cove River, May 2010:** This study was commissioned by the Town of PCSP and conducted by Newfoundland and Labrador Consulting Engineers Ltd. (NLCEL). This study included a survey of cross-sections and stream-crossing structures for Main River from Murray's Pond to its discharge into Conception Bay, and for Broad Cove River from Little Power's Pond to its discharge into Conception Bay. Hydrological and hydraulic modelling were conducted to delineate the 1:100-year inundation areas and to examine the hydraulic performance of existing stream-crossing structures under current and future development conditions.

**Town of Logy Bay- Middle Cove – Outer Cove, Flood-risk Mapping Study, August 2012:** This study was commissioned by the Department of Environment and Conservation, WRMD and conducted by CBCL Limited. This study included an update of the rainfall Intensity, Duration, and Frequency (IDF) curves prepared by Environment Canada for the meteorological station of the St. John's International Airport (#8403506). This update was conducted using rain gage data collected by the City of St. John's at Windsor Lake for the period from 2001-2010. Suitable data for this purpose was not available for the meteorological station at the St. John's International Airport maintained by Environment Canada.

**Regional Flood Frequency Analysis (RFFA) for Newfoundland and Labrador, 2014 Update:** This study was commissioned by the Department of Environment and Conservation, WRMD and conducted by AMEC Environment & Infrastructure (2014). This study involved the development of sets of statistical equations for estimating flood flows associated with specific return periods in ungauged watersheds. In this study, the island of Newfoundland was divided into five regions (the Town of PCSP is located in the southeast region). Regression equations were developed for estimating peak flood flows with return frequencies ranging from 1:20-year to 1:200-years.

## 2.4 Hydrometric Stations

The Water Survey of Canada maintains a network of hydrometric stations, where stream flow records are obtained, sometimes over a long period. These records document the historical floods experienced in various physiographic regions, and provide valuable data for calibrating and verifying hydrological models used in flood studies. There is currently one active hydrometric station located on Northeast Pond River in the Main River Watershed (02ZM006, drainage area 3.63 $km^2$). The flow at this station is unregulated natural flow, and this hydrometric station has been active since 1953. A discontinued hydrometric station was located on Broad Cove River near St. Philip's (Station No. 02ZM007). Flow records for this station are available from 1967 to 1982. However, flow for this station was regulated by the dam and spillway structure at the outlet of the Windsor Lake.

For planning purposes, Water Survey Canada divides the large geographic areas into divisions, sub-divisions, and sub-sub divisions based in part on the physiographic characteristics of an area. The study area is located in the sub-sub division of 02ZM which includes the City of St. John's. There is a relatively high density of hydrometric stations in this sub-sub division because of the populated status of this area. Although these hydrometric stations are not located within the study area, the flow records for some of these hydrometric stations may be representative of the study area because of their proximity and similar physiographic and climatic conditions. The flow records for these hydrometric stations can be used to augment the flow records available within the study watershed.

# 3.0 INUNDATION MAPPING AREA, STREAM NETWORK AND MODELLING APPROACH

## 3.1 Inundation Mapping Area and Stream Network (IMA&SN)

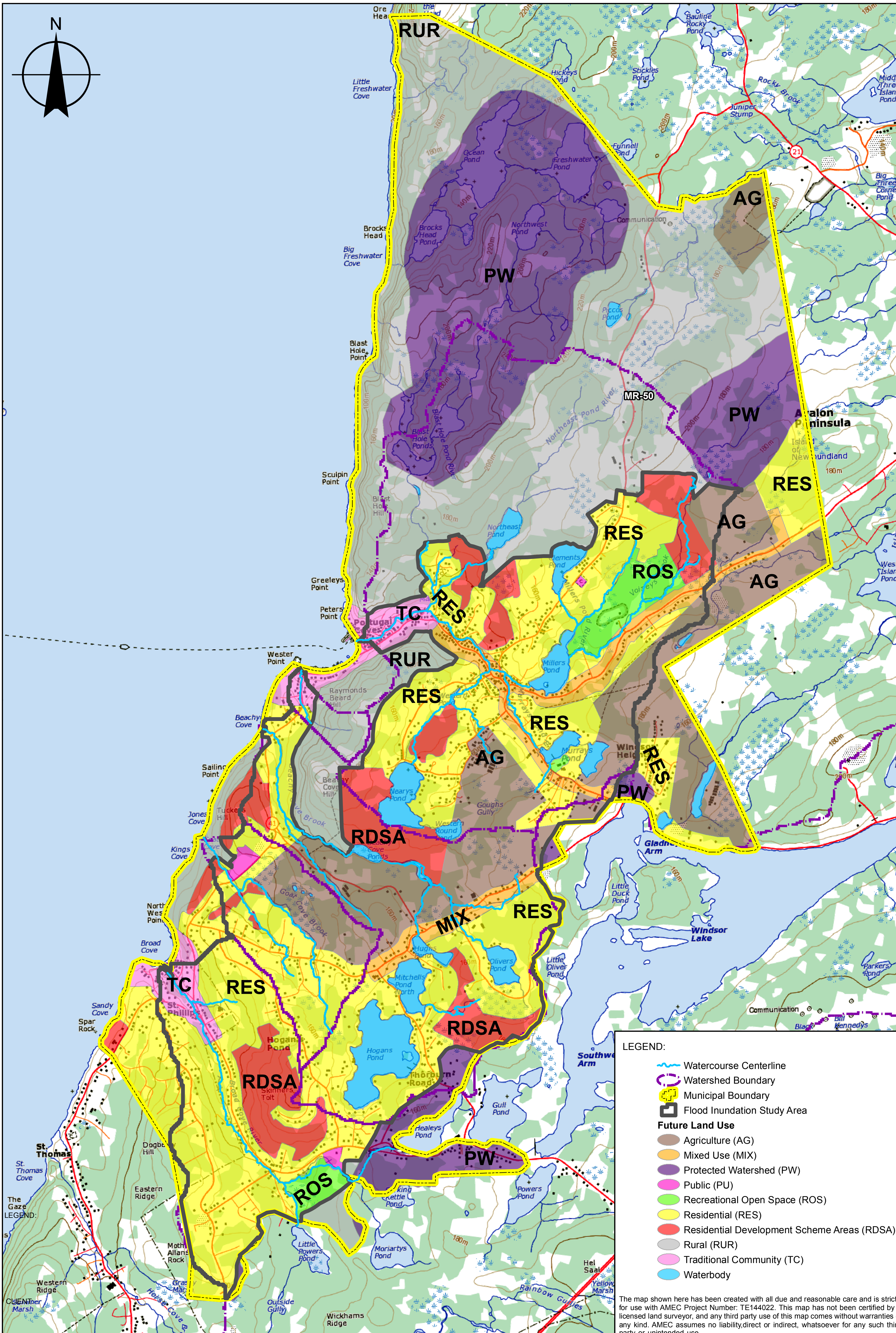
This study considers both the municipal boundary for the Town of PCSP, as well as the overall boundary for the five watersheds. However, significant watershed areas fall outside the municipal boundary (e.g., the Windsor Lake); there are also significant municipal areas that fall outside the overall boundary of the five watersheds. Additionally, it is recognized that for some of the municipal areas, existing developments are sparse and future developments will be restricted based on the municipal planning regulations, and that inundation mapping for these areas will not be required. For this inundation mapping project, it is important to delineate and establish the areas and stream network for which inundation mapping will be conducted. This is referred to as the Inundation Mapping Area and Stream Network (IMA&SN).

The identification of the IMA&SN for the study area is shown in Figure 3.1, which shows the municipal boundary as well as the boundaries for the five watersheds. The generalized future land use zoning adopted in the Municipal Plan 2014-2024 is also shown on the map. For the purpose of this study, the perennial stream network as shown on the 1:50,000 topographic maps were adopted. It should be noted that the stream network is a function of the map scale and resolution. As the map scale and resolution increase, more streams and drainage features will become visible and presented. It was accepted that the stream network as presented on the 1:50,000 topographic maps provides sufficient coverage for the purpose of this study.

The following steps were taken in the development of the IMA&SN:

- Areas inside the overall watershed boundary but outside the municipal boundary were excluded (e.g., Windsor Lake);
- Areas inside the municipal boundary but outside the overall watershed boundary were excluded (e.g., Brock's Pond, Ocean Pond); and
- Areas delineated in the future municipal plan as Rural and Protected Watershed were excluded as development in these areas are sparse or restricted in the future.

The delineated IMA&SN for the study area is shown in Figure 3.2, and inundation mapping will be conducted for the areas and streams shown in Figure 3.2. The IMA&SN includes numerous streams, many of which are small streams. For easy referencing, the IMA&SN is further divided into mapping zones. As shown in Figure 3.2, the Main River watershed consists of five mapping zones (MR1 through to MR5); the Beachy Cove Brook watershed consists of three mapping zones; the other three watersheds consist of one mapping zone each. The watershed ID (MR for Main River, UN for unnamed stream, BC for Beachy Cove Brook, GC for Goat Cove Brook, and BR for Broad Cove River) and the mapping zones are used throughout the study and in the establishment of the hydrological and hydraulic models for identifying various hydrological and hydraulic elements.




CLIENT:
WATER RESOURCES MANAGEMENT DIVISION
DEPARTMENT OF ENVIRONMENT AND CONSERVATION
GOVERNMENT OF NEWFOUNDLAND AND LABRADOR

amec foster wheeler

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| DATUM: NAD 83 CSRS | PROJECT: FLOOD RISK MAPPING STUDY PORTUGAL COVE - ST. PHILIP'S | DWN BY: JT | DATE: MAY, 2015 |
| PROJECTION: MTM ZONE 1 | TITLE: DELINEATION OF INUNDATION MAPPING AREA AND STREAM NETWORK (IMA&SN) | CHK'D BY: HX | REV. NO: 0 |
| PROJECT NO: TE144022 | | SCALE: 1:40,000 | FIGURE NO: 3.1 |

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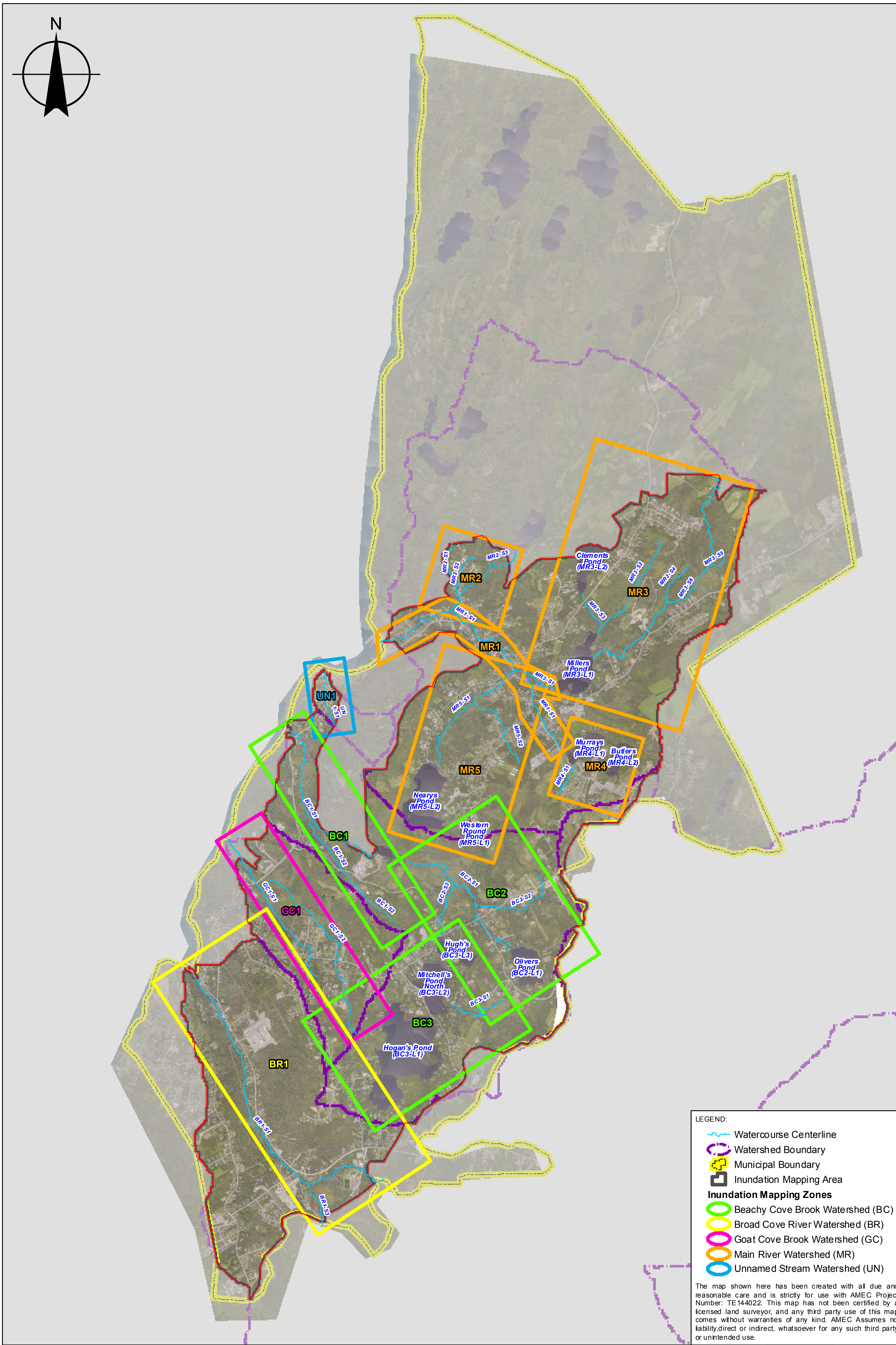
SOURCE: Geogratis NTS Topo Map / PCSP Municipal Topo

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| | DATUM: NAD 83 CSRS | | CHK'D BY: HX | REV. NO: 0 |
| amec foster wheeler | PROJECTION: MTM ZONE 1 | TITLE: IMA & SN AND MAPPING ZONES | SCALE: 1:40,000 | FIGURE NO: 3.2 |
| | PROJECT NO: TE144022 | | | |

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SOURCE: LEGEO LiDAR Imagery 2014, PCSP Municipal Topo

Table 3.1 lists the waterbodies (streams and lakes) in the IMA&SN grouped into each watershed and each mapping zone. Inundation mapping was conducted for each of the waterbodies contained in Table 3.1.

***Table 3.1 Stream and Lakes included in the IMA&SN***

| Watershed | Mapping Zones | Stream/Lake/Pond ID | Note |
|---|---|---|---|
| Main River (MR) | MR1 | MR1-S1 | Main River from Murray's Pond to discharge |
| | MR2 | MR2-S1 | |
| | | MR2-S2 | |
| | | MR2-S3 | Northeast Pond River |
| | MR3 | MR3-S1 | |
| | | MR3-S2 | |
| | | MR3-S3 | Millers Pond River |
| | | MR3-S4 | |
| | | MR3-S5 | |
| | | MR3-S6 | Voiseys Brook |
| | | MR3-L1 | Millers Pond |
| | | MR3-L2 | Clements Pond |
| | MR4 | MR4-S1 | |
| | | MR4-L1 | Murrays Pond |
| | | MR4-L2 | Butlers Pond |
| | MR5 | MR5-S1 | |
| | | MR5-S2 | |
| | | MR5-L1 | Western Round Pond |
| | | MR5-L2 | Nearys Pond |
| Unnamed Stream (UN) | UN1 | UN1-S1 | Unnamed Stream |
| Beachy Cove Brook (BC) | BC1 | BC1-S1 | Beachy Cove Brook |
| | | BC1-S2 | |
| | BC2 | BC2-S1 | |
| | | BC2-S2 | |
| | | BC2-L1 | Olivers Pond |
| | BC3 | BC3-S1 | |
| | | BC3-L1 | Hogans Pond |
| | | BC3-L2 | Mitchells Pond North |
| | | BC3-L3 | Hughs Pond |
| Goat Cove Brook (GC) | GC1 | GC1-S1 | |
| | | GC1-S2 | Goat Cove Brook |
| Broad Cove River (BR) | BR1 | BR1-S1 | Broad Cove River |
| | | BR1-S3 | |
| A total of 23 streams and 10 lakes | | | |

## 3.2 Overall Modelling Approach

The waterbodies identified for inundation modelling include numerous streams as well as lakes. For this study, the inundation mapping will be produced using the Hydrologic Engineering Center-River Analysis System (HEC-RAS) model under peak flow conditions (steady state). However, this approach is not suitable for simulating the peak water levels in the lakes and ponds, as the water levels in lakes and ponds are determined not only by the peak inflow, but also by the volume of the inflow. Therefore, simulation of water levels in the lakes and ponds must consider the inflow hydrograph under unsteady state flow conditions (flow varies with time).

The modelling approach adopted was designed to consider the different modelling requirements for lakes and streams. In this approach, the peak water level in the lakes in response to the hydrological events was simulated using Hydrological Engineering Center-Hydrologic Modelling System (HEC-HMS), which considers the entire inflow hydrograph. A lake is simulated as a level pool, and variation of water level through the lake is considered negligible. This approximation is sufficiently accurate as the lakes in the study area are generally small. The peak water levels in the streams in response to the peak flows were simulated using HEC-RAS, and the peak water levels (and the corresponding flows) for the lakes were used as boundary conditions in the simulation.

The establishment and simulation using the HEC-HMS model for determining stream flows and lake water levels are discussed in Section 6. The establishment and simulation using the HEC-RAS model for determining stream water level profiles are discussed in Section 7.

# 4.0 FIELD PROGRAMS

The objective of the field survey program was to collect various data required for the establishment and calibration of the hydrological and hydraulic models. A critical requirement in the production of the inundation mapping is the availability of highly accurate topographic data. To meet this requirement, a LiDAR survey was conducted covering the areas within the municipal boundary. Additionally, field surveys were conducted to collect information related to stream channel geometry, water levels and flows. A summary of the LiDAR survey and field surveys completed for this study is provided in this section.

## 4.1 LiDAR Survey

The LiDAR survey provided high resolution 3D geospatial imaging data and ortho-photography. The LiDAR data was required for the development of an accurate Digital Elevation Model (DEM) for inundation mapping. The images produced by the aerial photography serve as the basis for the inundation mapping.

The LiDAR data and aerial photography were collected and processed by Leading Edge Geomatics (LEGEO). LEGEO used a Riegl LMS-680ii Airborne Scanner that made use of a powerful laser source with Multiple-Time-Around (MTA) processing and digital full waveform analysis. This combination allowed for the operation at varying flight altitudes and was ideally suited for an aerial survey of the complex terrain.

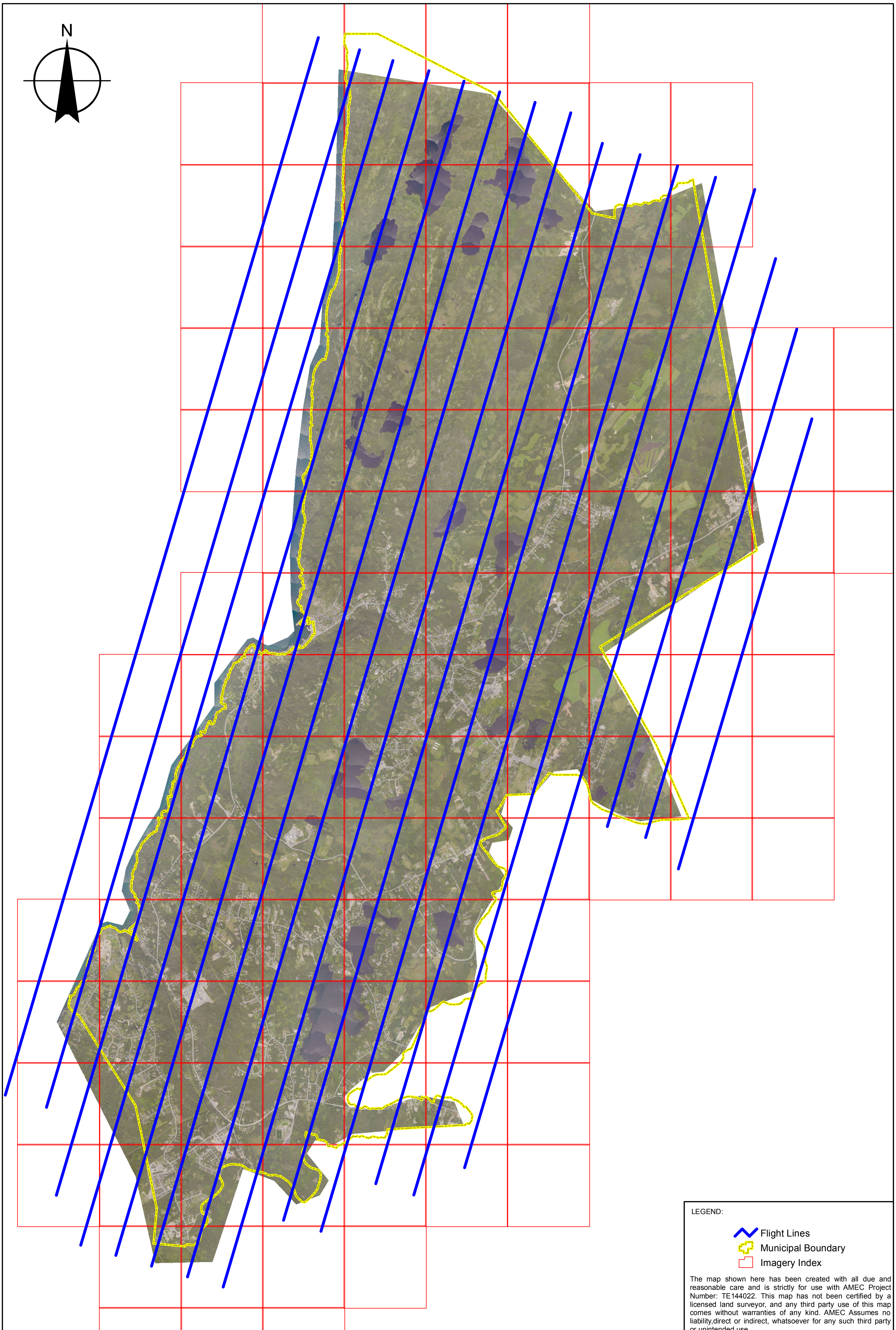
Figure 4.1 shows the coverage area and flight lines used for the acquisition of LiDAR and aerial photography data. The LiDAR survey was conducted on August 25, and the aerial photography survey was conducted on August 30, 2014. Only data located within the municipal boundary was utilized for post-processing. Flights were conducted during cloud-free conditions with no unusual obstacles or impediments encountered during the airborne and ground control surveys.

Details on the LiDAR survey, data processing, quality control, and data accuracy verification are documented in a LiDAR survey report contained in Appendix C.

## 4.2 Field Survey

The field survey program completed for this study consisted of the following five components:

- **Stream-crossing Structures:** The survey of bridges and culverts on the stream systems was required for establishing the HEC-RAS models used for generating the flood-risk mapping.
- **Survey of Stream Cross-sections:** Surveys of cross-sections were performed to establish the geometry below the water level and within the main channel of the streams. The survey data, in combination with the LiDAR survey data, was required for establishing the HEC-RAS models.

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| CLIENT: **WATER RESOURCES MANAGEMENT DIVISION DEPARTMENT OF ENVIRONMENT AND CONSERVATION GOVERNMENT OF NEWFOUNDLAND AND LABRADOR** | 400 200 0 400 800 Metres<br>DATUM: NAD 83 CSRS | PROJECT: **FLOOD RISK MAPPING STUDY PORTUGAL COVE - ST. PHILIP'S** | DWN BY: JT<br>CHK'D BY: HX | DATE: MAY, 2015<br>REV. NO: 0 |
| amec foster wheeler | PROJECTION: MTM ZONE 1<br>PROJECT NO: TE144022 | TITLE: **LiDAR SURVEY COVERAGE AREA AND FLIGHT LINES** | SCALE: 1:42,000 | FIGURE NO: 4.1 |

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SOURCE: LEGEO LiDAR Imagery 2014,PCSP Municipal Topo

- **Survey of Flow Control Structures and Dams:** These data were required for establishing the HEC-HMS models used for simulating flows for the flood-risk mapping.
- **Water Level and Flow Monitoring:** Hydrometric data was required for calibrating and verifying the hydrologic and hydraulic models.
- **Ground-truthing for Flow Diversions:** Ground-truthing was performed to identify potential flow diversions within the five study watersheds.

Details of the field survey program are summarized in a field report contained in Appendix D. A brief discussion of each of the field survey components is provided in the following sections.

### 4.2.1 Survey of Stream-Crossing Structures

The stream-crossing structures were identified based on the intersections of the stream network with the road network. These structures typically include culverts and bridges. The stream network and the road network were dependent on the map scale. As the map scale and resolution increase, more streams and roads are visible and presented. For the purpose of this project, the stream network was defined as those shown on the 1:50,000 topographic map. The road network was based on 1:2,500 scale municipal maps obtained from the Town of PCSP.

Figure 3.2 shows the IMA&SN developed for the flood mapping study. The IMA&SN defines the study area and stream network for which inundation modelling was conducted and mapping prepared. Therefore, stream-crossing structures identified within the IMA&SN had to be surveyed so they could be represented in the HEC-RAS models.

A stormwater management study was completed in 2010 by NLCEL for the Town of PCSP. This study included survey and modelling of the following sections of the Main River and Broad Cove River:

- Main River from Murray's Pond to its final discharge into the Conception Bay; and
- Broad Cove River from Little Power's Pond to its final discharge into the Conception Bay.

Stream-crossing structures for the above two streams have been surveyed as part of the NLCEL study, and the survey data as incorporated into the established HEC-RAS models was obtained by Amec Foster Wheeler from the Town of PCSP and from NLCEL. The survey of some of the crossing structures on these two streams was not repeated.

The dominant portion of the stream-crossing structures consisted of culverts. During the survey, the conditions of the stream-crossing structures were recorded. A summary of the stream-crossing structures surveyed and their conditions is provided in Table 4.1. The locations of the stream-crossing structures are shown in Figures 4.2 through 4.6.

**Table 4.1 Summary of Stream Crossing Structures**

| Stream Crossing ID | Structure Type | Structure Dimensions | Structure Condition | Comment |
|---|---|---|---|---|
| **Main River** | | | | |
| MR-1 | Bridge | | | |
| MR-2 | CSP arch with open bottom | Span: 6.2 m Rise: 1.8 m | Good condition, no blockages or damage noted. | |
| MR-3 | Bridge | Span: 5.0 m Height: 1.4 m | | |
| MR-4 | Bridge | | | |
| MR-5 | CSP arch | Span: 1.6 m Rise: 1.1 m | Good condition, but has some notable blockages by boulders. | |
| MR-6 | Bridge | | | |
| MR-7 | CSP circular | Diameter: 0.8 m | Culvert is new and is in good condition. | |
| MR-8 | CSP arch | Span: 1.65 m Rise 0.95 m | Good condition, no blockages or damage noted. | |
| MR-9 | CSP arch | Span: 1.4 m Rise: 1.0 m | Good condition, no blockages or damage noted. | |
| MR-10 | | | | Not within inundation mapping area |
| MR-11 | | | | Not within inundation mapping area |
| MR-12 | | | | Not within inundation mapping area |
| MR-13 | Bridge | | | |
| MR-14 | CSP Arch Culvert | Span: 5.0 m Height: 2.4 m | | |
| MR-15 | Bridge | | | |
| MR-16 | Box Culvert | Span: 4.3 m Height: 1.7 m | | |
| MR-17 | Bridge | | | |
| MR-18 | Bridge | | | |
| MR-19 | CSP arch with open bottom | Span: 6.0 m Rise: 1.4 m | Good condition, no blockages or damage noted. | |
| MR-20 | Bridge | Span: 3.9 m Height: 2.2 m | | |
| MR-21 | CSP arch with open bottom | Span: 5.7 m Rise: 1.5 m | Good condition, no blockages or damage noted. | |
| MR-22 | Bridge | | | |
| MR-23 | CSP arch | Span: 1.8 m Rise: 1.2 m | Some rusting, mostly on bottom. | |
| MR-24 | CSP arch | Span: 1.8 m Rise: 1.2 m | Some rusting, mostly on bottom. | |
| MR-25 | CSP circular | Diameter: 0.60 m | Bottom of culvert is rusting. | |
| MR-26 | Corrugated HDPE circular | Diameter: 0.70 m | Culvert is in good condition but there is a buildup of silt in the culvert. | |
| MR-27 | CSP circular | - | | Catch basin and pipe installation |
| MR-28 | CSP arch with open bottom | Span: 4.0 m Rise: 2.0 m | Good condition, no blockages or damage noted. | |
| MR-29 | Bridge | Span: 2.0 m Height: 0.9 m | | |
| MR-30 | Bridge | Span: 2.3 m Height: 0.6 m | | |
| MR-31 | CSP arch | Span: 1.4 m Rise: 1.0 m | Good condition, no blockages or damage noted. | |
| MR-32 | C.S.P. Culvert | Diameter: 0.6 m | | |
| MR-33 | C.S.P. Culvert | Diameter: 0.6 m | | |
| MR-34 | C.S.P. Culvert | Diameter: 0.45 mm (3) | | |
| MR-35 | CSP circular | Diameter: 1.2 m | Fair condition, large indent on top of culvert on downstream side. | |
| MR-36 | Bridge | | | |
| MR-37 | CSP open bottom arch | Span: 2.8 m Rise: 2.0 m | Good condition, no blockages or damage noted. | |
| MR-38 | C.S.P. Culvert | Diameter: 0.45 m | | |
| MR-39 | Culvert | Diameter: 3 @ 0.6 mm | | |
| MR-40 | CSP arch | Span: 1.2 m Rise: 0.8 m | Fair condition, minor rusting. Infilled with gravel on bottom. | |

**Table 4.1 Summary of Stream Crossing Structures**

| Stream Crossing ID | Structure Type | Structure Dimensions | Structure Condition | Comment |
|---|---|---|---|---|
| MR-41 | C.S.P. Culvert | Diameter: 0.6 m | | |
| MR-42 | Concrete Culvert | Diameter: 0.6 m | | |
| MR-43 | | | | Driveway |
| MR-44 | CSP circular | Two culverts with 0.4 m and 0.5 m diameter. | Culverts appear to be partially blocked with mud/detritus at the upstream end. | |
| MR-45 | Corrugated HDPE circular | Diameter: 0.60 m | Road over top of culvert is disintegrating. | |
| MR-46 | CSP circular | Diameter: 0.60 m | | |
| MR-47 | | | | No structure present |
| MR-48 | CSP circular | Diameter: 1.0 m | Good condition, no blockages or damage noted. | |
| MR-49 | CSP circular | Diameter: 1.5 m | New culvert in good condition, no blockages or damage noted. | |
| MR-50 | | | | Not within inundation mapping area |
| **Unnamed Stream** | | | | |
| UN-1 | CSP circular | Diameter: 1.45 m | Bottom of culvert rusting away. | |
| UN-2 | CSP circular | Diameter: 1.0 m | | |
| UN-3 | | | | Not present |
| UN-4 | CSP Circular | Diameter: 0.80 m | Culvert is in good condition but there are signs of rusting along the bottom. There is a steel mesh trash rack located immediately upstream of the inlet. | |
| UN-5 | CSP/HDPE circular | Diameter: 0.60 m | Culvert in good condition | |
| UN-6 | CSP circular | Diameter: 0.90 m | Bottom of culvert badly rusted (portions are rusted away). The inlet of the culvert is blocked by grass clippings dumped by nearby residents. At the time of the survey there was no water flowing through the culvert. | |
| UN-7 | CSP circular | Diameter: 0.45 m | Culvert is in good condition. | |
| UN-8 | CSP circular | Diameter: 0.60 m | Culvert is in good condition. | |
| UN-9 | CSP arch | Span: 1.2 m Rise: 0.80 m | Generally culvert is in good condition with some rust visible on the inside (primarily along the bottom). | |
| **Beachy Cove Brook** | | | | |
| BE-1 | CSP circular | Diameter: 2.0 m | Good condition, no blockages or damage noted. | |
| | CSP arch | Span: 1.8 m Rise: 1.4 m | Good condition, no blockages or damage noted. | |
| BE-2 | CSP circular | Diameter: 0.50 m, 0.20 m | Edge of the downstream side of the the corrugated culvert is bent and shows signs of rusting. There are no obvious signs (e.g. holes) of deterioration. | |
| BE-3 | CSP circular | Diameter: 0.60 m | Bottom of culvert is rusted. | |
| BE-4 | | | | No Structure Present |
| BE-5 | CSP circular | Diameter: 1.5 m | Bottom of culvert rusted. | |
| BE-6 | CSP circular | Diameter: 2 @ 1.35 m | Good condition, slight rusting along bottom but no signs of deterioration. | |
| BE-7 | Concrete box | Height: 0.9 Width: 3.05 m | Good condition with no signs of deterioration. | |
| BE-8 | CSP arch | Span: 1.9m Rise: 1.2 m | Bottom of culvert is rusty but there were no perforations visible. | |
| BE-9 | CSP Circular | Diameter: 2 @ 0.60 m | Culverts rusted along bottom but no sign of perforation. | |

**Table 4.1 Summary of Stream Crossing Structures**

| Stream Crossing ID | Structure Type | Structure Dimensions | Structure Condition | Comment |
|---|---|---|---|---|
| BE-10 | CSP circular | Diameter: 0.80 m | Culvert is in good condition with minimal signs of rusting. | |
| BE-11 | CSP circular | Diameter: 0.75 m | Culvert is in good condition with minimal signs of rusting. | |
| BE-12 | CSP circular | Diameter: 0.80 m | Culvert is in good condition. | |
| BE-13 | CSP arch | Diameter: 1.5 m | Culvert is in generally good condition but there is some surface rust. | |
| BE-14 | CSP circular | Diameter: 0.76 m | Good condition | |
| BE-15 | CSP circular | Diameter: 0.50 m | The upper half of the culvert appears to be in good condition. However, the bottom half of the culvert is rusted and on the upstream end the bottom of the culvert seems to be rusted away (was not able to 'feel' the bottom of the culvert with the survey rod). | |
| BE-16 | CSP circular | | Culvert is in good condition. | |
| BE-17 | CSP circular | Diameter: 0.60 m | Culvert is in good condition. | |
| **Goat Cove Brook** | | | | |
| GC-1 | CSP arch | Span: 1.8 m Rise: 1.25 m | | |
| GC-2 | CSP circular | Diameter: 1.2 m | Culvert is in good condition, but is bent on top. | |
| GC-3 | CSP circular | Inlet Diameter: 1.05 m<br>Outlet Diameter: 0.90 m | Bottom of corrugated section rusted through near outlet. Appears that a portion of the culvert under the roadway has been partially crushed and loose fill has fallen into the culvert. | |
| GC-4 | CSP circular | Diameter: 0.60 m | Rock at outlet. | |
| GC-5 | CSP arch | Span: 1.22 m Rise: 0.95 m | Culvert is rusty, especially bottom. Culvert appears to be partially crushed with fill visible through the sides of culvert. | |
| GC-6 | CSP circular | Diameter: 0.90 m | Inlet end of culvert is visibly rusty. | |
| **Broad Cove River** | | | | |
| BR-0 | Bridge | | | |
| BR-1 | | | | No structure located |
| BR-2 | Bridge | | | |
| BR-3 | Bridge | | | |
| BR-4 | | | | Structure not present |
| BR-5 | CSP open bottom arch | | Newly constructed structure (Nov. 2014) | |
| BR-6 | Bridge | | | |
| BR-7 | CSP circular | Diameter: 2 @ 1.5 m | Culvert is in good condition. | |
| BR-8 | CSP arch | Span: 4.0 m Rise: 2.0 m | | Not located in inundation mapping area |
| BR-9 | Concrete and CSP open bottom arch | Span: 6.5 m Rise: 2.1 m | Good condition, no blockages or damage noted. Also has rock walls (gabian baskets). | Not located in inundation mapping area |
| BR-10 | CSP circular | Diameter: 0.50 m | Culvert outlet not visible/appears to be completely submerged. | Not located in inundation mapping area |
| BR-11 | CSP arch | Span: 2.15 m Rise: 1.5 m | Good condition, no blockages or damage noted. | Not located in inundation mapping area |

Note: CSP referrs to corrugated steel pipe
HDPE referrs to high density polyethylene.


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| | DATUM: NAD 83 CSRS | | CHK'D BY: HX | REV. NO: 0 |
| amec foster wheeler | PROJECTION: MTM ZONE 1 | TITLE: LOCATION OF STREAM CROSSING STRUCTURES - MAIN RIVER WATERSHED | SCALE: 1:17,000 | FIGURE NO: 4.2 |
| | PROJECT NO: TE144022 | | | |

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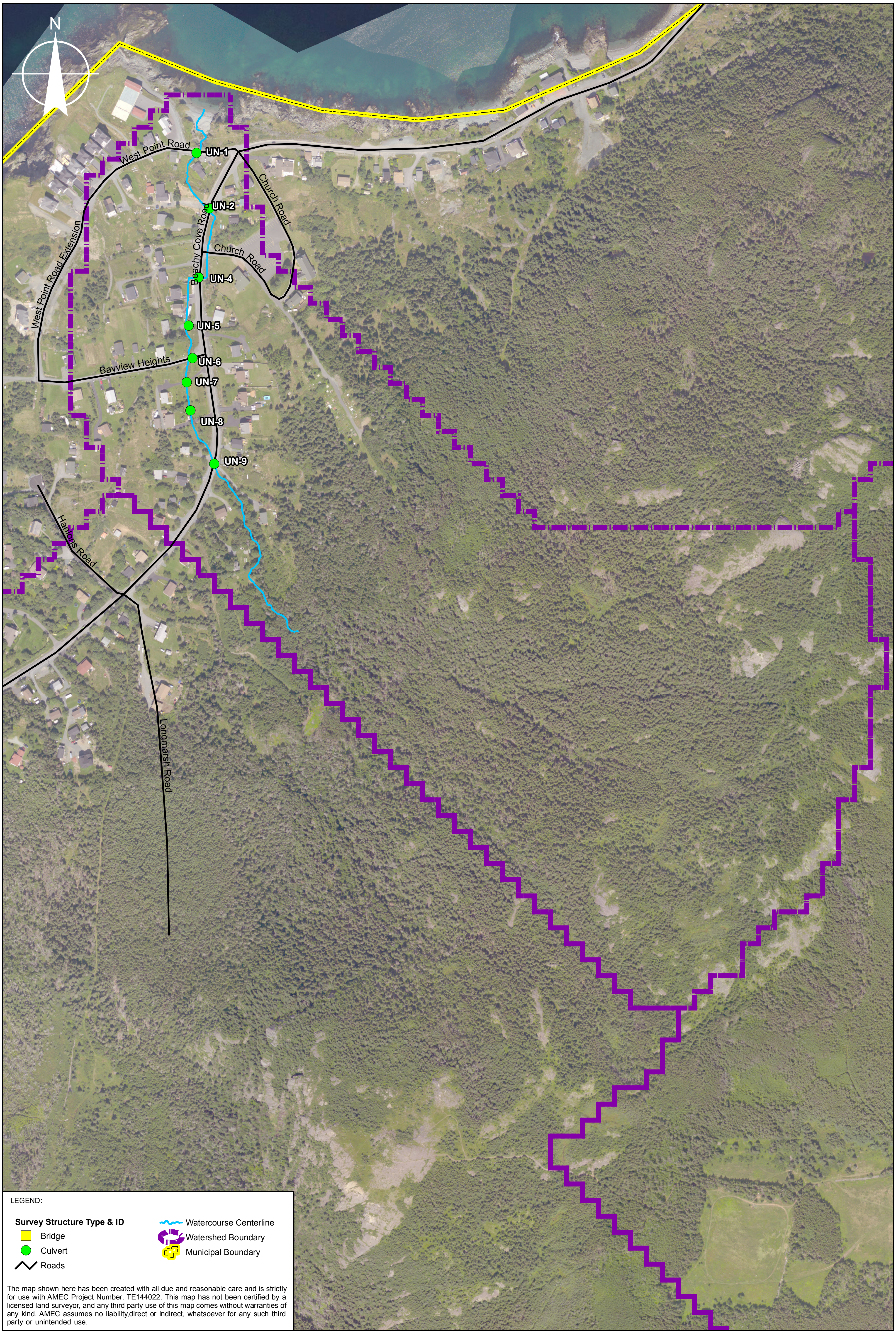


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| | DATUM: NAD 83 CSRS | | CHK'D BY: HX | REV. NO: 0 |
| amec foster wheeler | PROJECTION: MTM ZONE 1 | TITLE:<br>LOCATION OF STREAM CROSSING STRUCTURES<br>- UNNAMED STREAM WATERSHED | SCALE: 1:4,000 | FIGURE NO: 4.3 |
| | PROJECT NO: TE144022 | | | |

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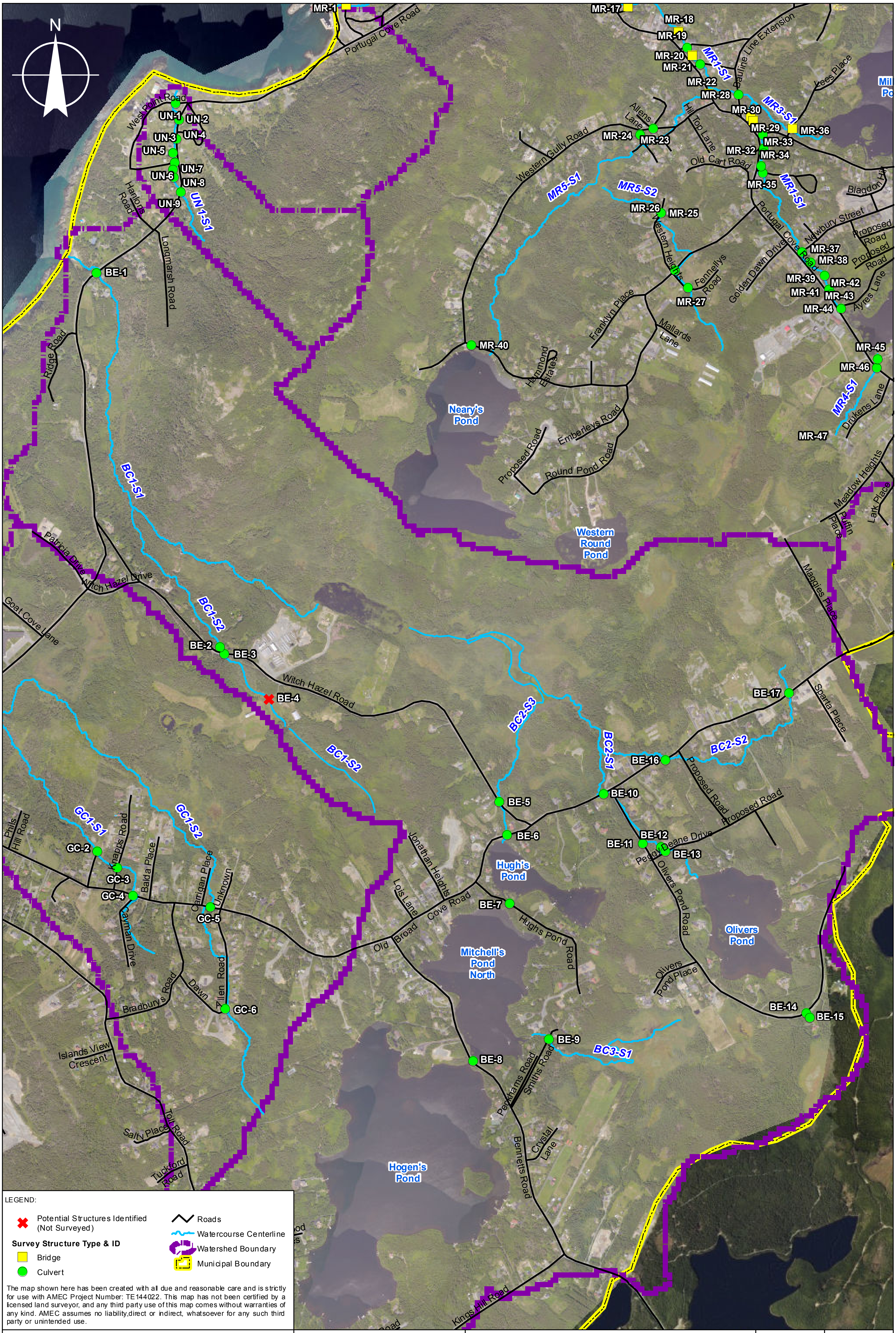
SOURCE: LEGEO LiDAR Imagery 2014,PCSP Municipal Topo

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| amec foster wheeler | PROJECTION: MTM ZONE 1; PROJECT NO: TE144022 | TITLE: LOCATION OF STREAM CROSSING STRUCTURES - BEACHY COVE BROOK WATERSHED | CHK'D BY: HX; REV. NO: 0; SCALE: 1:14,000; FIGURE NO: 4.4 |

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| PROJECTION: MTM ZONE1 | | | CHK'D BY: HX | REV. NO: 0 |
| PROJECT NO: TE144022 | TITLE: LOCATION OF STREAM CROSSING STRUCTURES - GOAT COVE BROOK WATERSHED | | SCALE: 1:10,000 | FIGURE NO: 4.5 |

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| DATUM: NAD 83 CSRS | PROJECT: **FLOOD RISK MAPPING STUDY PORTUGAL COVE - ST. PHILIP'S** | DWN BY: JT | DATE: MAY, 2015 |
| PROJECTION: MTM ZONE 1 | TITLE: **LOCATION OF STREAM CROSSING STRUCTURES - BROAD COVE RIVER WATERSHED** | CHK'D BY: HX | REV. NO: 0 |
| PROJECT NO: TE144022 | | SCALE: 1:12,000 | FIGURE NO: 4.6 |

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SOURCE: LEGEO LiDAR Imagery 2014, PCSP Municipal Topo

A survey of stream-crossing structures was required for establishing the HEC-RAS models. The input data for stream-crossing structures for the HEC-RAS model consisted of the geometry data for the structure itself, as well as four cross-sections. Two of the cross-sections were located upstream and two downstream of the structure to define the flow contraction and expansion caused by the structure. The HEC-RAS manual contains guidelines regarding the location of the cross-sections relative to structures.

Field survey forms were prepared prior to the field survey. These survey forms were designed to capture the input data required for the HEC-RAS models for the bridge/culvert structure and associated cross-sections.

For each identified stream-crossing structure, the survey crew conducted a survey for the structure and completed the survey form. The cross-sections associated with the structure were located based on the guidelines in the HEC-RAS manual. The cross-sections were then surveyed with the survey forms completed. Pictures were taken for each structure in a consistent and methodical manner. For example, the upstream and downstream view of the structure was taken as perpendicular to the structure as possible, in order to facilitate input and verification of the survey data in the HEC-RAS modelling.

#### 4.2.2 Survey of Stream Cross-Sections

This survey included the stream cross-sections not associated with the stream-crossings. The selection of the cross-section survey locations was based on the elevation profiles for the streams located within the IMA&SN. At the beginning of this process, the elevation profile for each of the streams within the IMA&SN was generated using data from the LiDAR survey. Cross-section locations were identified based on the stream profiles wherever there was deemed to be a change of slope. A preliminary set of cross-sections was developed based on the stream profiles.

A rationalization process was then applied to this preliminary set of cross-sections, and a number of cross-sections were excluded, including the following:

- cross-sections located on the two streams surveyed and modelled by NLCEL;
- cross-sections located in swampy areas;
- cross-sections associated with very small streams;
- cross-sections located near the confluence with the Atlantic Ocean where the topography is very steep; and
- cross-sections located in undeveloped forested areas near the headwaters.

After the rationalization process, a final set of cross-sections was identified for the field survey. The WRMD was consulted regarding the identified cross-section locations prior to the field survey.

A cross-section survey form was prepared prior to the field survey. The survey form was designed to capture the input data required by the HEC-RAS model for cross-sections, and included information for evaluating channel roughness.

For surveying each cross-section, the survey form was completed. The survey cross-section was selected perpendicular to the stream. Adequate points were surveyed to capture the geometry detail of the cross-section, including the underwater portion of the channel. Where practical, the cross-sections surveyed extended a minimum of 5 m from the tops of the banks to ensure adequate overlap with the LiDAR data. In addition, the survey was extended back 100 m from the tops of the banks for five of these cross-sections (one on each primary stream) in order to confirm the suitability of using the LiDAR survey for obtaining the overbank portions of the cross-sections. Pictures were taken in a consistent manner to document conditions at cross-sections.

The survey cross-sections were compared with the LiDAR and a summary of comparisons are included in Appendix E.

#### 4.2.3 Survey of Flow Control Structures and Dams

Windsor Lake is used as the water supply source for the City of St. John's, and a dam was constructed at the outlet in the Southwest Arm of the lake near Thorburn Road. An uncontrolled concrete spillway is incorporated into the dam, which releases excess flow into the Broad Cove River. The elevation of the dam, as well as the elevation and dimensions of the spillway relevant to its discharge capacity were surveyed.

Discharge from Murray's Pond in the Main River watershed was controlled by a weir structure. The elevations and dimensions of this structure were surveyed.

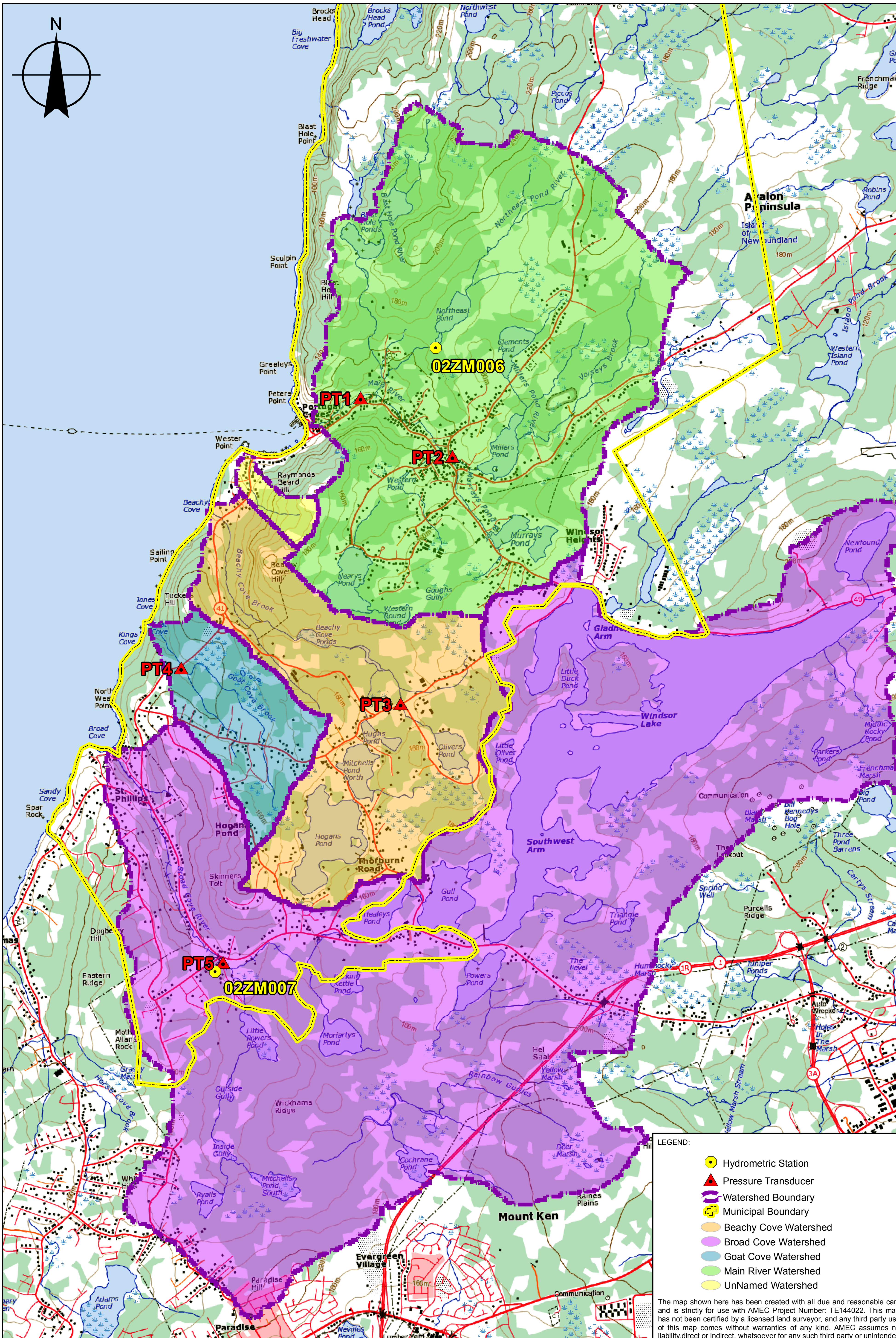
#### 4.2.4 Water Level and Flow Monitoring

Five pressure transducers were installed between September 19 and 22, 2014, and they functioned normally until they were removed on November 14, 2014. A cross-section survey was conducted at the locations of the installed pressure transducers. The locations of the pressure transducers as well as hydrometric stations within the study watersheds are shown in Figure 4.7.

#### 4.2.5 Ground-truthing for Flow Diversions

The WRMD required that any diversions be identified. If diversions were located, their effect on the watersheds and flooding was to be determined. The following efforts were made to identify potential diversions in the study area:

- Review of the 1:2,500 municipal maps acquired from the Town of PCSP, which showed a few small containment structures. These structures do not result in flow diversion between watersheds.

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| amec foster wheeler | PROJECTION: MTM ZONE 1 | TITLE: PRESSURE TRANSDUCER LOCATIONS | SCALE: 1:40,000 | FIGURE NO: 4.7 |
| | PROJECT NO: TE144022 | | | |

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SOURCE: Geogratis NTS Topo Map / PCSP Municipal Topo

- Review of the aerial photography obtained in the LiDAR survey in an attempt to identify anomalies around the lakes near the watershed boundaries. No anomalies were identified during this exercise.
- The field crew walked the trails in accessible areas around the lakes near the watershed boundaries in an effort to identify potential diversion structures. No diversion structures were identified during this exercise.
- The Town of PCSP and City of St. John's were consulted regarding the presence of potential diversion structures. No diversion structures were identified during this exercise.

Based on the level-of-effort outlined above, and the relatively populated state of the study watersheds, it was deemed unlikely that there were any diversion structures of consequence in these watersheds.

# 5.0 REMOTE SENSING ANALYSIS AND ESTIMATE OF SCS CURVE NUMBER (CN)

The objective of this task is to estimate the distribution of US Soil Conservation Service (SCS) CN for the five study watersheds to be used as input in the hydrological modelling. A SCS CN is an index of a basin's runoff generation potential and is a function of soil drainage characteristics and land use/cover conditions. Theoretically, the value of CN can range from 0 to 100, the higher the CN value, the lower the permeability and higher runoff generation potential.

For this project, the land use/cover conditions for the five study watersheds were estimated through a remote sensing analysis. A geo-referenced land classification map covering the five study watersheds was developed through this analysis. This land classification map was then combined with the available soil classification map to generate a map delineating the distribution of CN. The approach adopted in this project for developing the CN distribution mapping is outlined in the following sections.

## 5.1 Remote Sensing Analysis

A report summarizing the remote sensing analysis is provided in Appendix F. A brief summary of the remote sensing completed is provided in this section.

The WRMD requires that the areas covering the five study watersheds be classified into one of the eight categories summarized in Table 5.1.

***Table 5.1 WRMD Land Classification Categories***

| *WRMD Land Cover* | *Examples* |
|---|---|
| *Forest* | *Forests.* |
| *Residential* | *Small homes and subdivisions.* |
| *Commercial* | *Large building and parking lots, schools, shopping malls, industries, plants, etc.* |
| *Deforested areas* | *Patches of treed and un-treed areas adjacent to forest roads, areas with open green fields in forested zones.* |
| *Barren land* | *Non-vegetated areas.* |
| *Fields/pastures/open spaces* | *Agricultural areas, farmer fields; parks, cemeteries, golf courses, etc. within urban area, low-lying grass areas near airport, vegetated areas.* |
| *Swamps/wetlands/waterbodies* | *Swamps; wetlands; lakes, ponds, and rivers.* |
| *Unclassified* | *No data, cloud, shadow, snow/ice.* |

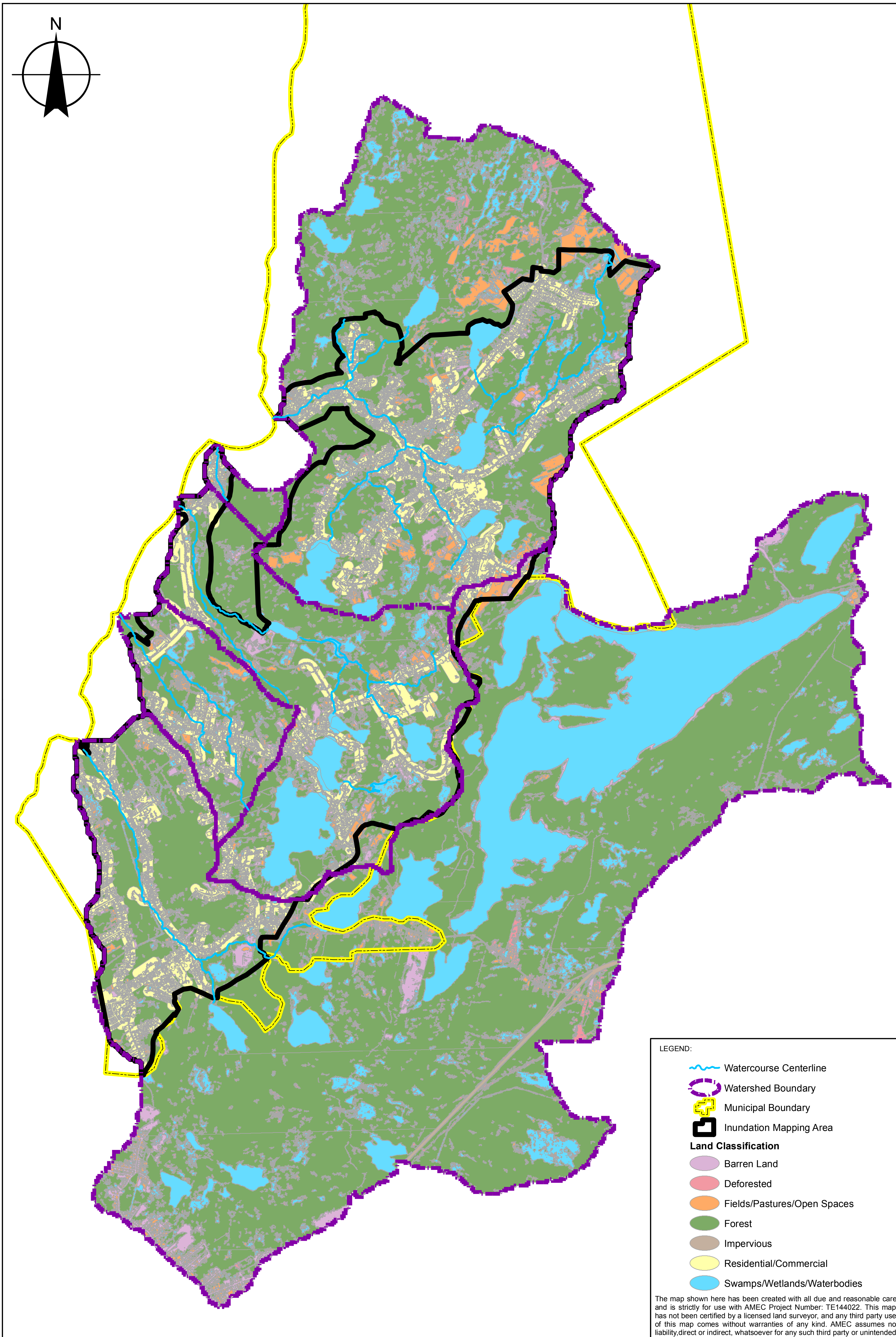
The data used in the remote sensing analysis consists of 0.5 m resolution, 8-band multispectral WorldView 2 (WV2) imagery captured in September 2012, and provided to Amec Foster Wheeler by WRMD. The WV2 images were delivered as previously ortho-rectified datasets and with a combination of clipped and/or full scenes that included fused (panchromatic sharpened) multispectral, natural colour, 8-band multispectral, and false colour infrared images. The remote sensing analysis involved the classification and grouping of each of the image pixels through automated and manual processes. A map showing the land classification obtained through the remote sensing analysis is provided in Figure 5.1.

During the remote sensing analysis, it was found to be impractical to separate residential from commercial land use, as indicated in Table 5.1, and the two land classifications were combined. For the residential land cover class, road centerlines were utilized to define residential properties. Centerlines were buffered by 60 m on each side, resulting in the creation of a residential/commercial zone.

A summary of the land cover percentage by each of the land classifications is provided in Table 5.2. The overall area of the five watersheds is nearly 60% classified as forested. The residential/commercial developments accounts for approximately 18% of the overall watershed area.

***Table 5.2 Land Classification Summary***

| Land Cover Description | Main River | | Unnamed Stream | | Beachy Cove Brook | | Goat Cove Brook | | Broad Cove River | |
|---|---|---|---|---|---|---|---|---|---|---|
| | Area ($km^2$) | Percentage (%) | Area ($km^2$) | Percentage (%) | Area ($km^2$) | Percentage (%) | Area ($km^2$) | Percentage (%) | Area ($km^2$) | Percentage (%) |
| Forest | 10.2 | 57 | 0.3 | 70 | 4.5 | 52 | 1.4 | 53 | 24.6 | 63 |
| Fields/Pastures/Open Spaces | 1.5 | 8 | 0.0 | 3 | 0.5 | 6 | 0.2 | 8 | 1.2 | 3 |
| Deforested | 0.4 | 2 | 0.0 | 3 | 0.1 | 2 | 0.0 | 1 | 0.9 | 2 |
| Barren Land | 0.3 | 2 | 0.0 | 0 | 0.3 | 3 | 0.1 | 4 | 1.2 | 3 |
| Impervious Surface | 0.1 | 1 | 0.0 | 0 | 0.1 | 1 | 0.0 | 1 | 0.3 | 1 |
| Residential | 3.3 | 18 | 0.1 | 21 | 1.6 | 19 | 0.7 | 27 | 2.4 | 6 |
| Swamps/Wetlands/Waterbodies | *2.2* | 12 | 0.0 | 3 | 1.4 | 17 | 0.2 | 6 | 8.4 | 22 |
| Total | 18.0 | 100 | 0.4 | 100 | 8.6 | 100 | 2.6 | 100 | 39.1 | 100 |

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| | DATUM: NAD 83 CSRS | | CHK'D BY: HX | REV. NO: 0 |
| amec foster wheeler | PROJECTION: MTM ZONE 1 | TITLE: LAND CLASSIFICATION FOR CURRENT CONDITION | SCALE: 1:40,000 | FIGURE NO: 5.1 |
| | PROJECT NO: TE144022 | | | |

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SOURCE: PCSP Municipal Topo

## 5.2 Soil Classifications

The SCS classifies the soil into A, B, C, and D soil types for the purpose of determining the CN. The descriptions of these four soil types are shown in Table 5.3.

***Table 5.3 SCS Soil Types***

| SCS Soil Type | Description |
|---|---|
| A | These soils have low runoff potential and high infiltration rates even when thoroughly wetted. They consist chiefly of deep, well to excessively drained sand or gravel and have a high rate of water transmission. |
| B | These soils have moderate infiltration rates when thoroughly wetted and consist chiefly of moderately deep to deep, moderately well to well drained soils with moderately fine to moderately coarse textures. These soils have a moderate rate of water transmission. |
| C | These soils have low infiltration rates when thoroughly wetted and consist chiefly of soils with a layer that impedes downward movement of water and soils with moderately fine to fine texture. |
| D | These soils have high runoff potential. They have very low infiltration rates when thoroughly wetted and consist chiefly of clay soils with a high swelling potential, soils with a permanent high water table, soils with a claypan or clay layer at/or near the surface, and shallow soils over nearly impervious material. |

The WRMD recommended the determination of the SCS soil types based on the drainage class of a soil group as summarized in Table 5.4. The classification of the agriculture soil groups into the SCS soil groups for the study watersheds is summarized in Table 5.5. The agriculture soil distribution in the study watersheds shown in Figure 5.2 can also be converted into the SCS soil distribution as shown in Figure 5.3.

***Table 5.4 SCS Soil Types and Drainage Class***

| Soil Type | Drainage Class | SCS Soil Type |
|---|---|---|
| Very rapidly drained | VR | A |
| Rapidly drained | R | A |
| Well drained | W | A |
| Moderately well | MW | B |
| Imperfectly drained | I | B |
| Poorly drained | P | C |
| Very poorly drained | VP | D |

Soil information for the study area was available through Canadian Soil Information Service (CanSIS) of Agriculture Canada as shown in Figure 5.2. It is seen that the study area is dominantly covered by the Cochrane group of soil, and to a much less extent by the Bauline, Colinet, and Salmonier groups of soil. The drainage characteristics of these groups of soils are summarized in Table 5.5.

***Table 5.5 Summary of the Drainage Characteristics of the Soil Groups***

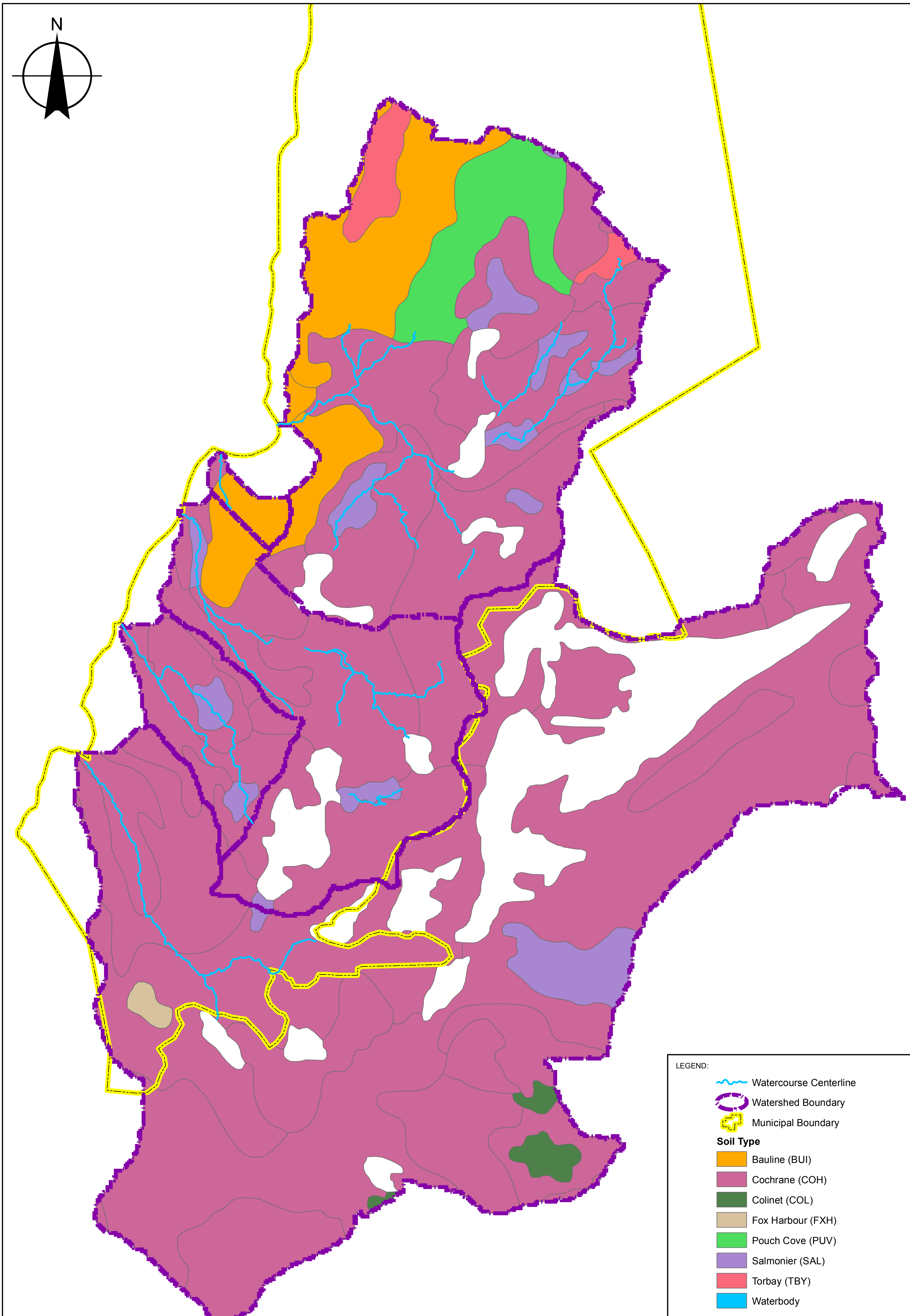
| Soil Type | Drainage Characteristics | Drainage Class | SCS Soil Type |
|---|---|---|---|
| Bauline Soil Group (BUI) | Rapidly drained | R | A |
| Cochrane Soil Group (COH) | Well drained | W | A |
| Colinet Soil Group (COL) | Very poorly drained | VP | D |
| Fox Harbour Soil Group (FXH) | Very poorly drained | VP | D |
| Pouch Cove Soil Group (PUV) | Moderately well drained | MW | B |
| Salmonier Soil Group (SAL) | Very poorly drained | VP | D |
| Torbay Soil Group (TBY) | Poorly drained | P | C |

In order to determine the CN, it was necessary to classify the soil groups encountered in the study watersheds into the soil types indicated by the SCS. It is seen that the dominant portion of soil in the study watersheds belongs in the SCS Type A soil. These soils have low runoff potential and high infiltration rates even when thoroughly wetted. They consist chiefly of deep, well to excessively drained sand or gravel and have a high rate of water transmission.

The classification of the agriculture soil groups into the SCS soil groups as summarized in Table 5.5 is based entirely on soil drainage class without consideration of soil depth, whereas the SCS soil classification outlined in Table 5.2 for soil types of "A" and "B" requires the soil profile to be deep in order to be valid. The overburden coverage in the study watersheds in many areas is thin and discontinuous, while the underlying bedrock generally has low permeability. This may imply that the runoff potential in the study watersheds may be higher than predicted based on the overburden soil conditions.

## 5.3 Estimate of CN for Current Development Condition

The empirical CN values are subject to variability resulting from rainfall intensity and duration, total rainfall, soil moisture conditions, cover density, stage of growth and temperature. These causes of variability are collectively called the Antecedent Runoff Condition (ARC). ARC is divided into three classes: I for dry conditions, II for average conditions, and III for wetter conditions. The WRMD requires that the CN be determined assuming ARC III. Table 5.6 summarizes the CN values recommended by the WRMD based on the land cover and SCS soil types and ARC III.

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PROJECT NO: TE144022

PROJECT: FLOOD RISK MAPPING STUDY
PORTUGAL COVE - ST. PHILLIP'S

TITLE: AGRICULTURE SOIL DISTRIBUTION

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SOURCE: PCSP Municipal Topo

***Table 5.6 CN in Relation to SCS Soil Types and Land Cover***

| WRMD Land Cover | A | B | C | D |
|---|---|---|---|---|
| Forest | 50 | 74 | 85 | 89 |
| Residential | 78 | 88 | 94 | 96 |
| Commercial | 96 | 97 | 98 | 98 |
| Deforested areas | 75 | 87 | 92 | 94 |
| Barren land | 89 | 94 | 97 | 98 |
| Fields/pastures/open spaces | 59 | 78 | 88 | 91 |
| Swamps/wetlands/waterbodies | 100 | 100 | 100 | 100 |
| Unclassified | NA | NA | NA | NA |

Using GIS tools, the land use classification as summarized in Figure 5.1 and SCS soil distribution as summarized in Figure 5.3 can be combined to produce the CN distribution using the CN values presented in Table 5.6. The CN distribution for the study watersheds is summarized in Figure 5.4.

In Table 5.6 the CN is provided as discrete values. For example, under forest land cover, the CN in Table 5.6 is considered to be one of four discrete values, depending on the SCS soil type. In reality, the CN for a particular soil type will likely vary within a range. Calibration of the hydrological modelling (to be discussed in Section 6.0) would be justified if the CN value for a particular soil type is considered to be a range of values, rather than a discrete value.

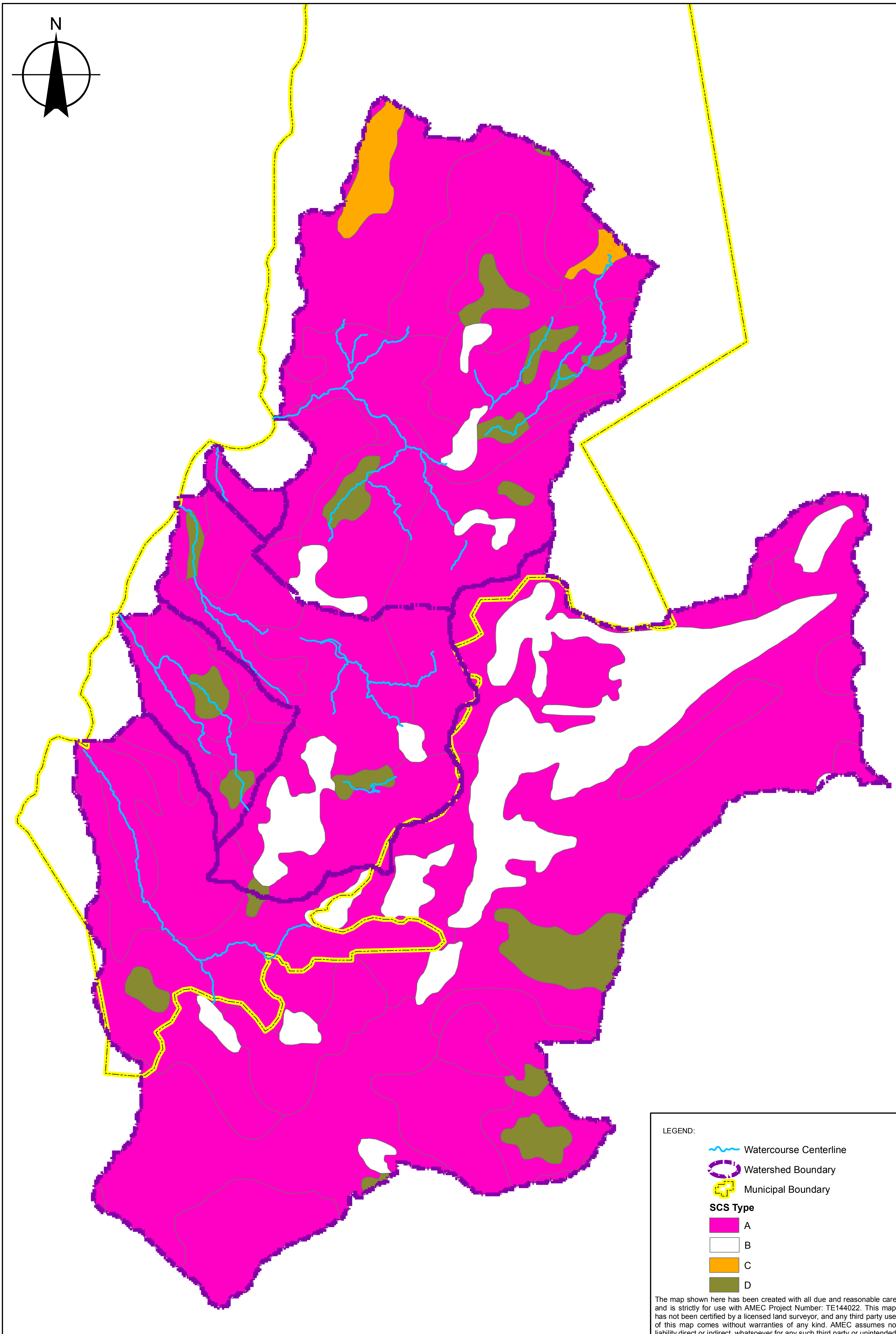
## 5.4 Estimate of CN for Future Developed Condition

### 5.4.1 Evaluation of Expected Future Development

Housing starts per annum, from 2008 to 2014, for the Town of PCSP, were provided to Amec Foster Wheeler. The data indicate 77 houses on average per year were constructed over the last five years. To estimate future development conditions, it was assumed that the average trend of 77 housing starts per annum will continue over the next 10 years. The following development assumptions were applied to determine the spatial extent of residential development over this future time frame:

- each building lot is 23 m by 30 m;
- the street Right-of-Way (RoW) is 20 m;
- lots are on both sides of the street; and
- the area to be covered by new residential development will include 20% area for ancillary infrastructure (intersections, etc.).

The resultant forecasted development is approximately 85,000 m$^2$ per annum, or 850,000 m$^2$ (0.85 km$^2$) over the next 10 years.

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MTM ZONE 1

PROJECT NO: TE144022

PROJECT: **FLOOD RISK MAPPING STUDY PORTUGAL COVE - ST PHILLIPS**

TITLE: **SCS SOIL DISTRIBUTION**

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| SCALE: 1:40,000 | FIGURE NO: 5.3 |

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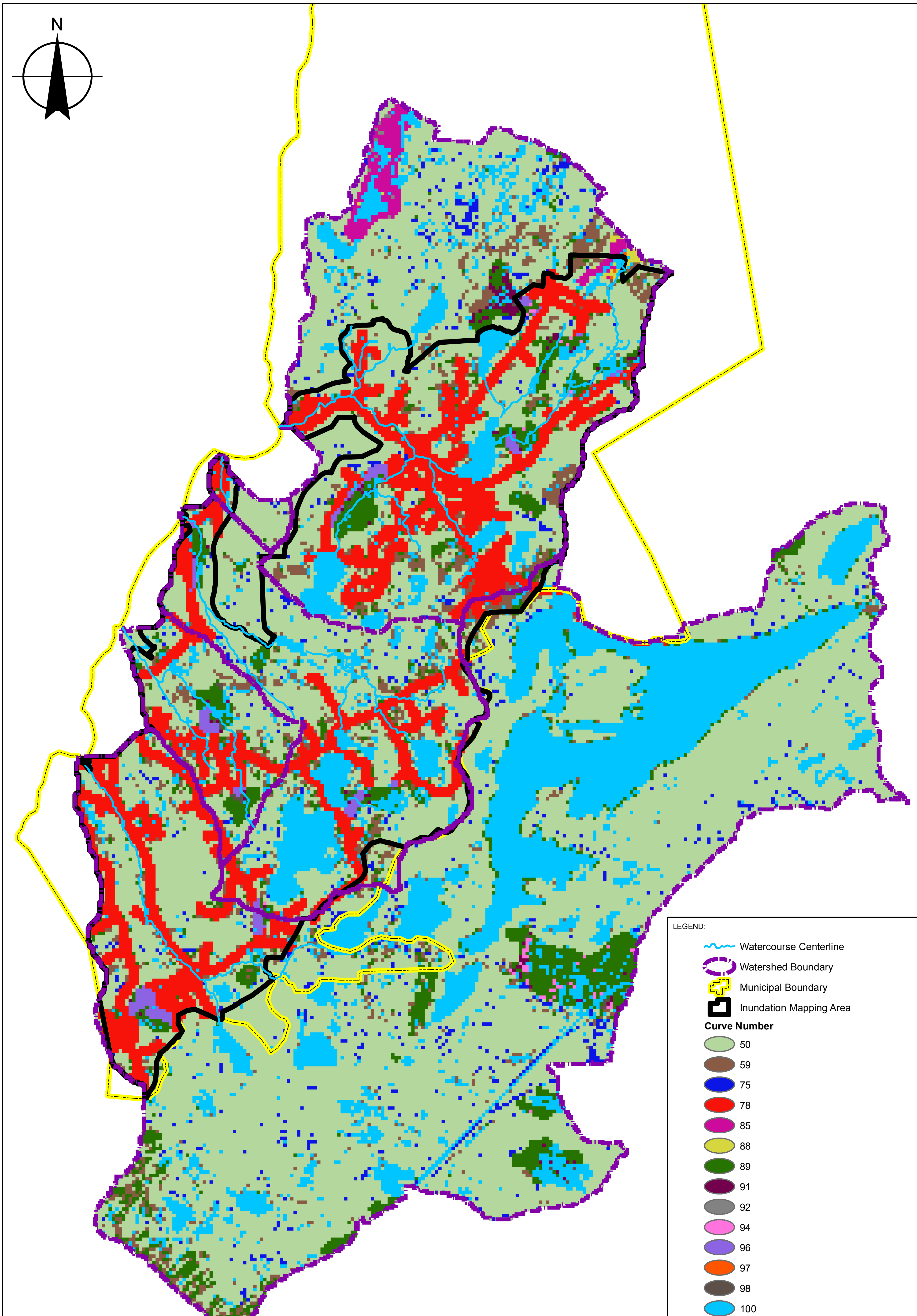
SOURCE: PCSP Municipal Topo

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| CLIENT: WATER RESOURCES MANAGEMENT DIVISION DEPARTMENT OF ENVIRONMENT AND CONSERVATION GOVERNMENT OF NEWFOUNDLAND AND LABRADOR | 1,000 500 0 Meters | PROJECT: FLOOD RISK MAPPING STUDY PORTUGAL COVE - ST. PHILLIP'S | DWN BY: TM | DATE: MAY, 2015 |
| | DATUM: NAD 83 CSRS | | CHK'D BY: HX | REV. NO: 0 |
| amec foster wheeler | PROJECTION: MTM ZONE 1 | TITLE: CN DISTRIBUTION FOR CURRENT DEVELOPMENT CONDITION | SCALE: 1:40,000 | FIGURE NO: 5.4 |
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Based on the forecasted area, a theoretical development scheme was plotted, which represents the 'Fully Developed Condition' (Figure 5.5). The Fully Developed Condition is the current development within the study area watershed boundaries, plus 850,000 $m^2$ of residential development, placed throughout the Town of PCSP, in areas anticipated to be potentially developed by 2024. In order to evaluate the area(s) most likely to be developed as residential over the next 10 years, four parameters were considered:

1. Current proposed roads;

2. Current land use zoning;

3. Existing features (slopes, wetlands, location of existing roads, etc.); and

4. Current Development / Market Demand.

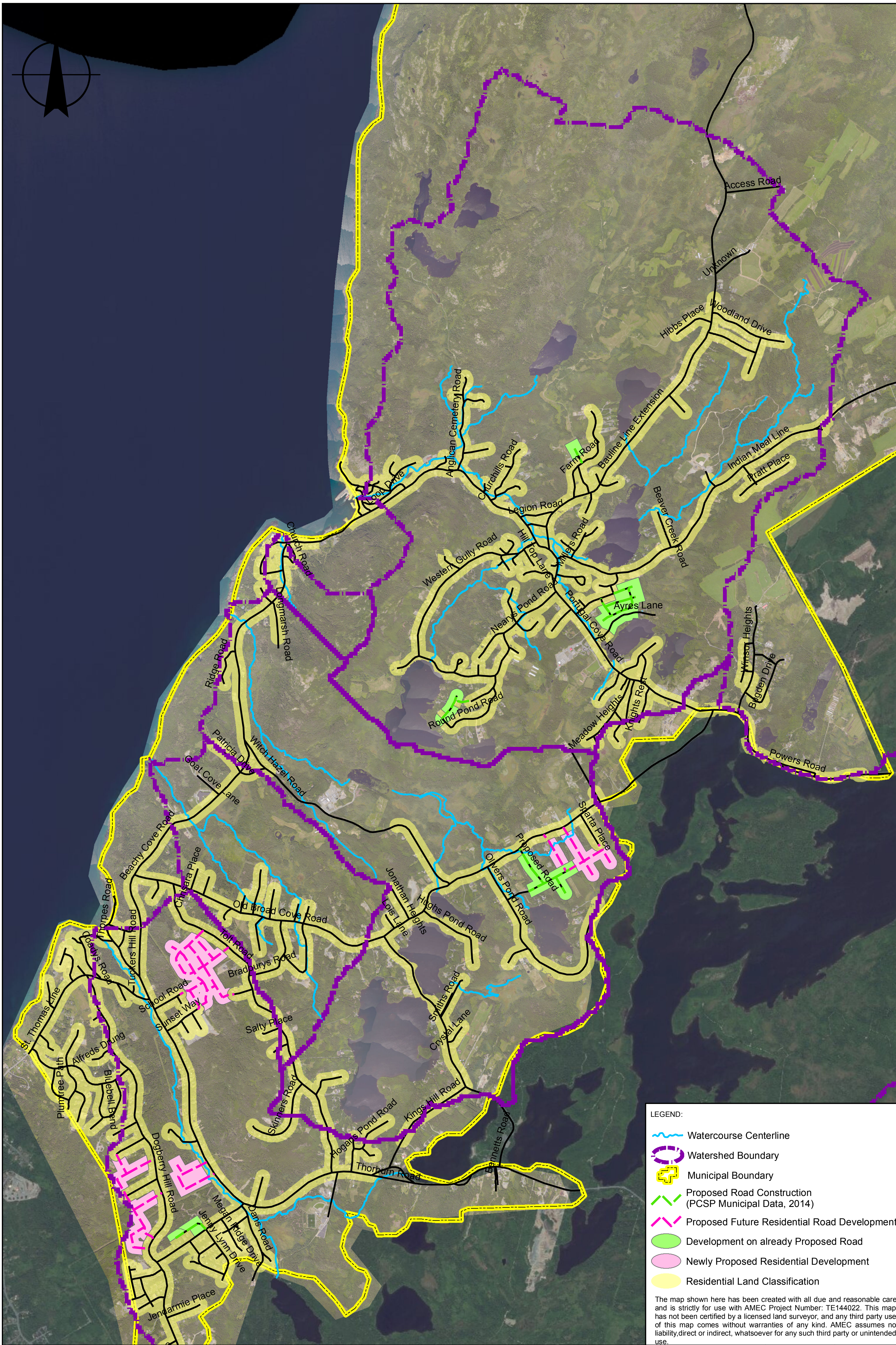
***Current Proposed Roads***

A number of new roads have been already proposed by the Town. Based on GIS information obtained from the Town, there are five areas within the Town of PCSP where new roads or new subdivisions off existing roads, have been proposed, including the following:

- North, off of Dogberry Road;
- South, off of Old Broad Cove Road, near Olivers Pond;
- Extension off the end of Round Pond Road;
- East and South of Newbury Street; and
- North, off of Farm Road.

***Current Land Use Zoning***

The Town of PCSP indicated that the estimated residential lot size (690 $m^2$) is best suited to the existing Residential Medium Density (RMD) zone or Residential Low Density (RLD) zone off existing roads. It is likely that only a small to negligible proportion of the new housing starts will occur within rural residential zones and it would be difficult to make an assumption of what proportion of residential development might be undertaken within the Residential Development Scheme Area (RDSA) over the next 10 years. Therefore, for ease of evaluation, this assessment assumes that all new residential development over the next 10 years will occur within either the RMD or the RLD zones.

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| amec foster wheeler | PROJECTION: MTM ZONE 1 | TITLE: ASSUMED FUTURE DEVELOPMENT CONDITIONS | SCALE: 1:30,000 | FIGURE NO: 5.5 |
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***Existing Features***

To evaluate existing features, the Environmental Protection Map (Map 3 of the Municipal Plan and Development Regulations 2014-2024) and aerial photography of the Town of PCSP were reviewed. When assigning areas that might be developed for residential purposes over the next 10 years, areas containing slopes greater than 25%, flood zones, flood-risk areas, and buffer zones were avoided. The location of utility corridors was also considered when plotting potential developable areas. Where new roads were proposed, efforts were made to tie theoretical future roads into existing road intersections and not to encroach on existing residential properties.

***Current Development / Market Demand***

The final evaluation parameter relates to the current market demand for housing in the Town of PCSP, or where development is currently taking place. This was assessed through a search of the Multiple Listing Service® (MLS®) at www.realtor.ca. A map of the Town of PCSP was searched to identify where property listings occur and what areas were listing new or recently constructed homes. This gives an indication of what areas have more recently been developed, which in turn provides an indication of where people are currently choosing to live in the Town of PCSP. As an additional measure, recent aerial photography of the Town of PCSP was reviewed to identify areas that are currently under development.

It was assumed that development over the next 10 years will begin along roads that have already been proposed by the Town but have not yet been constructed. It was assumed that residential development will occur along these roads, to the extent possible, until they have reached capacity. Based on these assumptions, this will account for approximately 296,000 $m^2$ of residential development.

The location(s) of the remaining 554,000 $m^2$ of residential development were estimated based on evaluation parameters 2 to 4 noted above. Based on the evaluation of the current development, the School Road / Sunset Way area was noted as a high market demand area, as evidenced by the current development viewable on aerial photographs and by the high number of new home listings on MLS® (10 listings as of February 25, 2015). It was assumed that residential development will extend within this area over the next 10 years to the extent possible, avoiding environmental features (predominantly steep slopes) and remaining in the RMD zone. Based on the presence of new developments, it was assumed that additional residential development will occur in the southwest end of the Town of PCSP off Dogberry Hill Road, while avoiding significant environmental features. Finally, based on the proposed roads near Olivers Pond and the availability of suitable residential land in that area, the remaining development area was projected to occur within this area with an extension of the proposed roads eastward to Sparta Place.

It should be noted that the analysis of future residential development over the next 10 years is based on what were considered "reasonable assumptions". It is not meant to be an accurate prediction of the future development conditions.

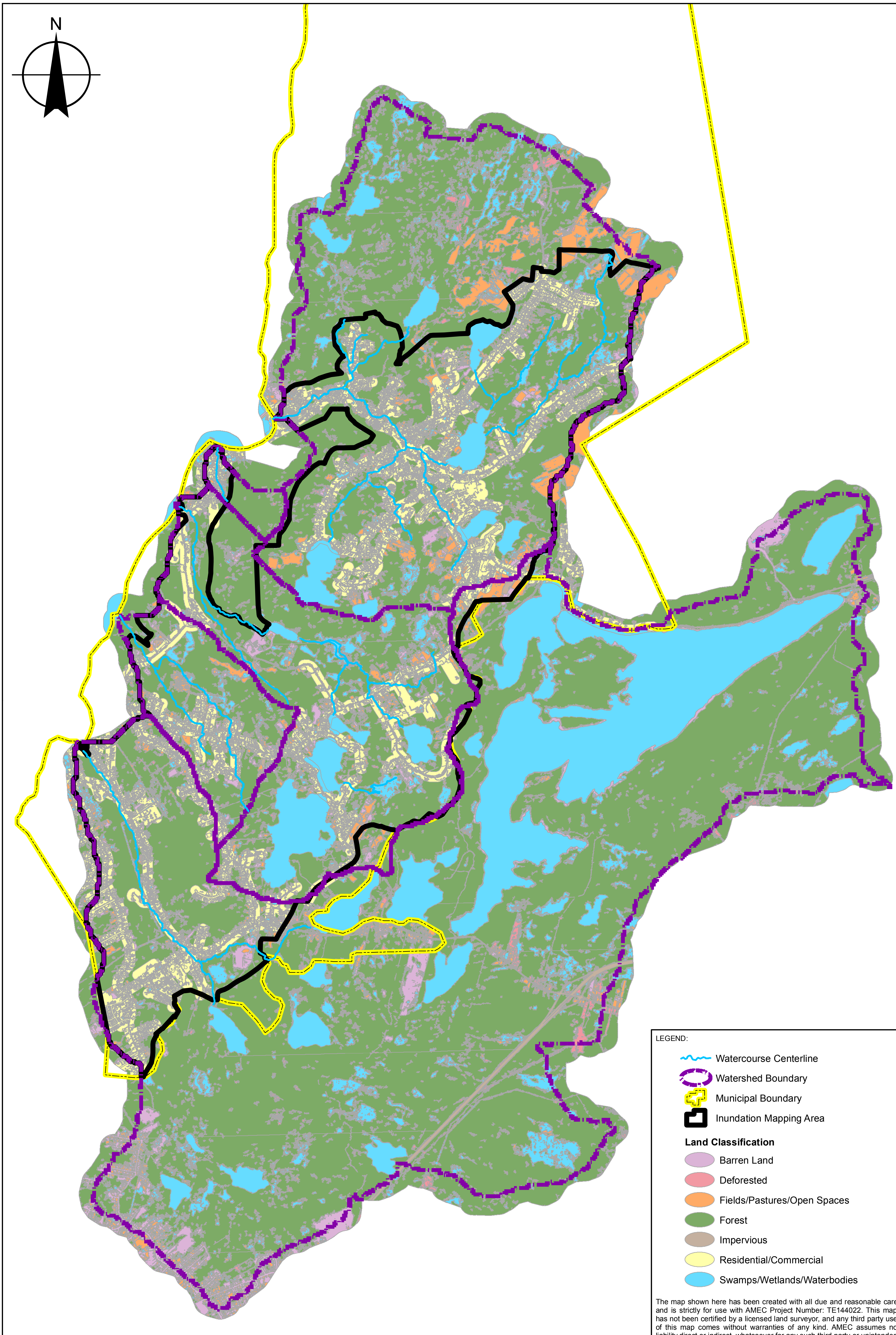
### 5.4.2 Estimate of CN Distribution for Future Development Condition

Based on the assumed future development condition, the land classification for future conditions can be estimated (Figure 5.6). The CN distribution can also be estimated in the same manner as for the existing development conditions (Figure 5.7).

A summary of land classification for expected future conditions in comparison with the current development conditions is provided in Table 5.7. It is apparent that in the foreseeable future, the effect of expected development in the study watersheds on the CN distribution (and thus on the runoff generation potential) is minimal. Therefore, further consideration and modelling to determine the effect of future development on potential flooding risk is not considered necessary. Although local studies related to storm water management may be required.

***Table 5.7 Comparison of Land Classification between Current Development and Future Development Conditions***

| Land Cover Description | Main River | | Unnamed Stream | | Beachy Cove Brook | | Goat Cove Brook | | Broad Cove River | |
|---|---|---|---|---|---|---|---|---|---|---|
| | Area ($km^2$) | Percentage (%) | Area ($km^2$) | Percentage (%) | Area ($km^2$) | Percentage (%) | Area ($km^2$) | Percentage (%) | Area ($km^2$) | Percentage (%) |
| **Current Development Condition** | | | | | | | | | | |
| Forest | 10.2 | 57 | 0.3 | 70 | 4.5 | 52 | 1.4 | 53 | 24.6 | 63 |
| Fields/Pastures/Open Spaces | 1.5 | 8 | 0.0 | 3 | 0.5 | 6 | 0.2 | 8 | 1.2 | 3 |
| Deforested | 0.4 | 2 | 0.0 | 3 | 0.1 | 2 | 0.0 | 1 | 0.9 | 2 |
| Barren Land | 0.3 | 2 | 0.0 | 0 | 0.3 | 3 | 0.1 | 4 | 1.2 | 3 |
| Impervious Surface | 0.1 | 1 | 0.0 | 0 | 0.1 | 1 | 0.0 | 1 | 0.3 | 1 |
| Residential | 3.3 | 18 | 0.1 | 21 | 1.6 | 19 | 0.7 | 27 | 2.4 | 6 |
| Swamps/Wetlands/Waterbodies | *2.2* | 12 | 0.0 | 3 | 1.4 | 17 | 0.2 | 6 | 8.4 | 22 |
| Total | 18.0 | 100 | 0.4 | 100 | 8.6 | 100 | 2.6 | 100 | 39.1 | 100 |
| **Future Development Condition** | | | | | | | | | | |
| Forest | 10.2 | 57 | 0.3 | 70 | 4.4 | 51 | 1.4 | 53 | 24.2 | 62 |
| Fields/Pastures/Open Spaces | 1.5 | 8 | 0.0 | 3 | 0.5 | 6 | 0.2 | 8 | 1.2 | 3 |
| Deforested | 0.4 | 2 | 0.0 | 3 | 0.1 | 2 | 0.0 | 1 | 0.9 | 2 |
| Barren Land | 0.3 | 2 | 0.0 | 0 | 0.3 | 3 | 0.1 | 4 | 1.2 | 3 |
| Impervious Surface | 0.1 | 1 | 0.0 | 0 | 0.1 | 1 | 0.0 | 1 | 0.3 | 1 |
| Residential | 3.3 | 18 | 0.1 | 21 | 1.8 | 20 | 0.7 | 27 | 2.8 | 7 |
| Swamps/Wetlands/Waterbodies | 2.2 | 12 | 0.0 | 3 | 1.4 | 17 | 0.2 | 6 | 8.4 | 22 |
| Total | 18.0 | 100 | 0.4 | 100 | 8.6 | 100 | 2.6 | 100 | 39.0 | 100 |




LEGEND:

- Watercourse Centerline
- Watershed Boundary
- Municipal Boundary
- Inundation Mapping Area

**Land Classification**

- Barren Land
- Deforested
- Fields/Pastures/Open Spaces
- Forest
- Impervious
- Residential/Commercial
- Swamps/Wetlands/Waterbodies



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| | | | CHK'D BY: HX | REV. NO: 0 |
| amec foster wheeler | PROJECTION: MTM ZONE 1 | TITLE: **LAND CLASSIFICATION FOR FUTURE DEVELOPMENT CONDITION** | SCALE: 1:40,000 | FIGURE NO: 5.6 |
| | PROJECT NO: TE144022 | | | |

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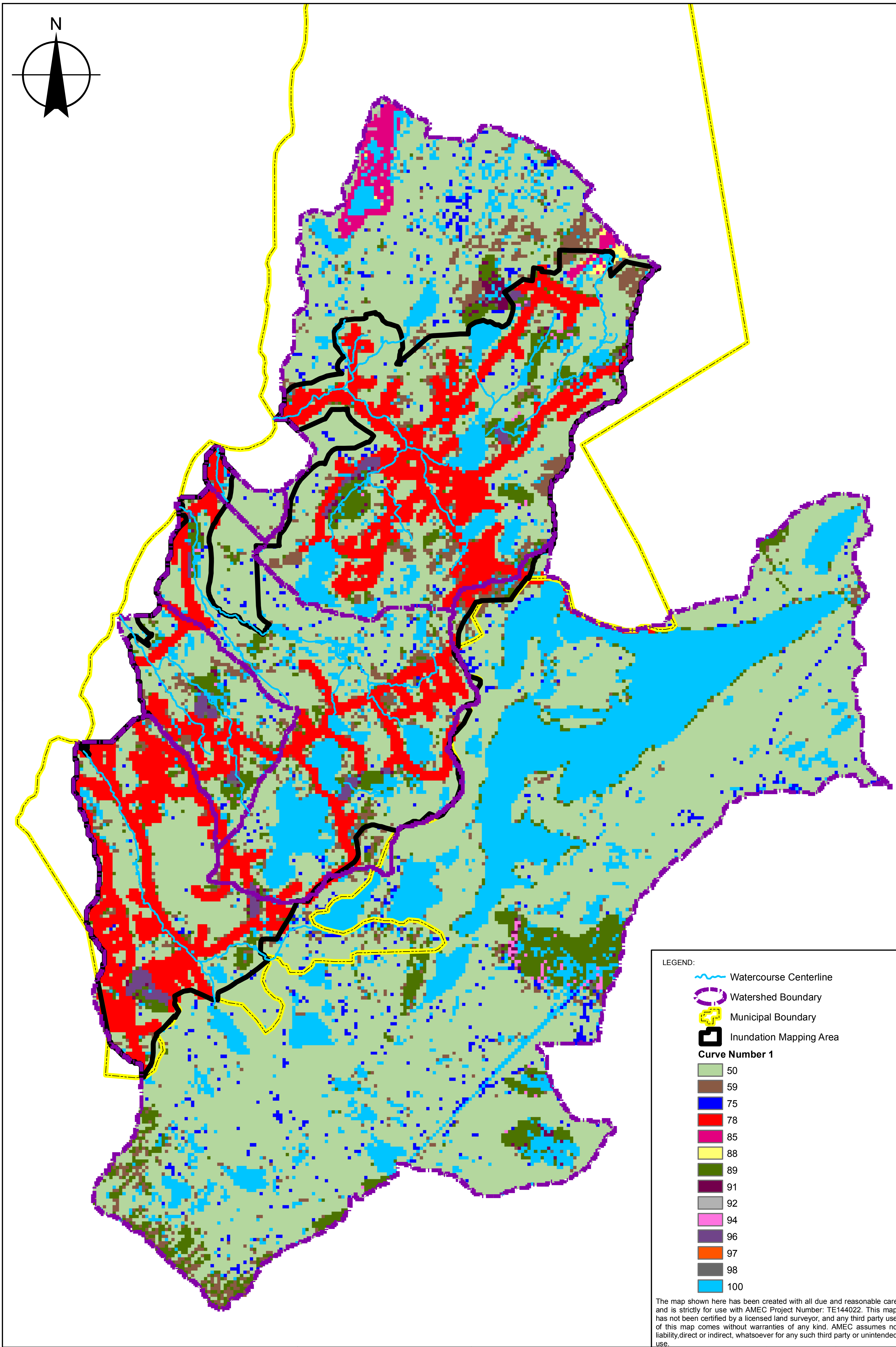


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| amec foster wheeler | PROJECTION: MTM ZONE1 | TITLE: CN DISTRIBUTION FOR FUTURE DEVELOPMENT CONDITION | SCALE: 1:40,000 | FIGURE NO: 5.7 |
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SOURCE: PCSP Municipal Topo

# 6.0 HYDROLOGIC ANALYSIS

The objectives of the hydrologic analysis include the following:

- to determine 1:20-year and 1:100-year AEP flow estimates at points of interest. These flows were subsequently used as input to the hydraulic model to estimate flood levels for the IMA&SN; and
- to determine the peak water levels for the lakes under 1:20-year and 1:100-year AEP flow conditions.

The WRMD requires that the 1:20-year and 1:100-year AEP flows be determined using statistical analysis as well as a deterministic modelling approach. There are advantages and disadvantages for each of the two approaches. The statistical approach is simple to use and the required flows can be determined quickly and conveniently. However, this approach does not consider localized hydrological characteristics of a watershed. For example, the statistical approach is typically based on the watershed area, and does not adequately allow for consideration of the attenuation effect to peak flow provided by the presence of significant storage features (large lakes). The deterministic modelling approach considers the significant variables contributing to the hydrological processes in a watershed; however, this approach requires detailed input of these variables which generally fluctuate with time and location. In addition, there is appreciable uncertainty regarding the determination of these variables. By comparing results from the two alternative approaches, the required flows can be estimated with greater confidence.

## 6.1 Statistical Analysis

An update of the RFFA for NL was completed by AMEC in September 2014. An RFFA is a method by which sets of equations for estimating return period flood flows in ungauged watersheds are developed. A RFFA was originally completed for the island of Newfoundland in 1971. Three updates of the RFFA were subsequently completed in 1984, 1990 and 1999. The AMEC 2014 RFFA update divided the Province of NL into five regions. Single-station statistical analysis was conducted for the hydrometric stations located in each region, and subsequently, regionalized equations were developed for estimating statistical flows with predetermined annual exceedance probabilities.

The AMEC 2014 RFFA update developed one set of single parameter equations and one set of two parameter equations. The single parameter equations adopted the drainage area for estimating the statistical flows. The two parameter equations added a Lake and Swamp Factor to the predictions. However, the statistical analysis indicated that the two parameter equations do not improve the prediction significantly from the one parameter equations. The RFFA study indicates that drainage area is by far the most significant variable contributing to flow in a similar physiographic regions and under similar climatic conditions. However, as indicated previously, the statistical equations cannot consider abnormal hydrological characteristics of a watershed (e.g., large lakes such as the Windsor Lake).

The study watersheds are located within the South East Hydrological Region defined by the RFFA study, therefore regression relationships developed for this region have been used to estimate the 1:20-year and 1:100-year flows as follows:

$$Q_{20} = 2.604 \times (\text{Drainage Area})^{0.775}$$

$$Q_{100} = 3.306 \times (\text{Drainage Area})^{0.780}$$

Where Drainage Area = $km^2$
Q = Instantaneous peak flows ($m^3/s$)

The 1:20-year and 1:100-year flows were estimated for the locations where the five pressure transducers were installed in the field programs, and for the location of the hydrometric station 02ZM006 located in the Main River watershed. These flows are summarized in Table 6.1. The locations of these sites are presented in Figure 4.7.

The hydrometric station 02ZM006 was included in the single-station analysis in the AMEC 2014 RFFA update. The 1:20-year and 1:100-year flows determined for this station in the single station analysis are also included in Table 6.1.

***Table 6.1 Comparison of Instantaneous Flows from Statistical and Modelling Analysis***

| Location | Drainage Area | Return Frequency | From Statistical | | From HEC-HMS Modelling | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | | RFFA Equation | Single Sta. | HEC-HMS Element | Flow without CN Adjustment | Difference with RFFA | Flow with 10% CN Increase | Difference with RFFA |
| | ($km^2$) | (yrs) | ($m^3/s$) | ($m^3/s$) | | ($m^3/s$) | (%) | ($m^3/s$) | (%) |
| **PT1** | 17.00 | 1:20 | 23.4 | | MR1-J6A | 17.7 | -32.2 | 25.2 | 7.1 |
| | | 1:100 | 30.1 | | | 29.0 | -3.9 | 39.1 | 22.9 |
| **PT2** | 6.50 | 1:20 | 11.1 | | MR1-J3A | 7.1 | -56.4 | 10.6 | -4.8 |
| | | 1:100 | 14.2 | | | 12.2 | -16.7 | 17.2 | 17.3 |
| **02ZM006** | 3.64 | 1:20 | 7.1 | 7.0 | MR2-W3A | 3.6 | -97.0 | 5.1 | -39.0 |
| | | 1:100 | 9.1 | 8.0 | | 5.9 | -53.6 | 7.8 | -16.2 |
| **PT3** | 0.67 | 1:20 | 1.9 | | BC2-J1 | 1.4 | -37.1 | 1.8 | -6.6 |
| | | 1:100 | 2.4 | | | 2.0 | -21.6 | 2.3 | -5.7 |
| **PT4** | 2.41 | 1:20 | 5.1 | | GC1-J5A | 3.6 | -43.0 | 4.9 | -5.0 |
| | | 1:100 | 6.6 | | | 5.6 | -17.2 | 7.2 | 8.9 |
| **PT5** | 34.59 | 1:20 | 40.6 | | BR1-J8A | 7.2 | N/A | 10.4 | N/A |
| | | 1:100 | 52.4 | | | 12.9 | N/A | 17.4 | N/A |

## 6.2 Deterministic Analysis

The 1:20-year and 1:100-year AEP flow estimates were simulated using a deterministic numerical model. The WRMD requires that for simulating the hydrological behaviour of the study area, the non-proprietary USACE HEC-HMS must be used. Additionally, the WRMD requires that the Geospatial Hydrological Modelling Extension of the HEC-HMS model, HEC-GeoHMS must be used for preparation of geometric data in Esri ArcGIS and for generation of the hydrological inputs for import into the HEC-HMS. Specifically, the WRMD requires the following regarding the HEC-GeoHMS application:

- Pre-processing Digital Elevation Model (DEM);
- Delineating the watershed and sub-watershed;
- Determining the watershed characteristics and parameters; and
- Creating the HEC-HMS project.

In the hydrological modelling, HEC-HMS (4.0) and HEC-GeoHMS 10 for ArcGIS 10 were used, which are the current latest versions of the software available.

### 6.2.1 Watershed DEM

LiDAR data provides a high accuracy and high resolution DEM. The LiDAR data obtained for this study covers the areas within the municipal boundary. The watershed areas falling outside the municipal boundary were not covered in the LiDAR survey. Attempts were made to merge together the available LiDAR data with the 1:50,000 Canadian Digital Elevation Data (CDED). The combined DEM resulting from this methodology was found to be unusable. For the hydrological modelling, the CDED data, which is commonly adopted as the baseline data for hydrological analysis, was used as the DEM. The grid size for this approach was 0.75 decimal seconds.

### 6.2.2 Establishment of Basin Models

The basin models were used to simulate the response of the various hydrological elements in a watershed to meteorological input. These hydrological elements typically included the watersheds and sub-watersheds, the storage sites, channel reaches, and diversions. Typically, the following steps were involved in the establishment of a basin model:

Selection of Locations with Required Flow: These included locations where flow would be required in the subsequent hydraulic modelling. Stream junctions within the IMA&SN were typically included where flow increase was expected. The lakes and ponds were included to account for the attenuation effect of these storage features on flood peaks. Additionally, the locations where the five pressure transducers were installed in the field programs and the location of hydrometric station 02ZM006 on Main River were included for model calibration purposes.

Delineation of Sub-watershed Boundaries: Watersheds and sub-watersheds were delineated using HEC-GeoHMS at the locations identified in the previous step.

Estimation of Hydrological Parameters: These included the drainage area of the watershed, the weighted CN for each of the sub-watersheds, and the lag time (a parameter used for simulating transformation of hydrographs by HEC-HMS). The weighted CN for a sub-watershed was determined using the georeferenced CN distribution map. The sub-watershed area and lag time were determined based on the underlying DEM and the CN distribution. These parameters were determined automatically by the HEC-GeoHMS tools.

Establishment of Other Model Elements: These included the lakes/ponds areas and stream reaches.

Creation of the HEC-HMS Project and Import into HEC-HMS: In this step, a HEC-HMS project was created in HEC-GeoHMS which was then imported into HEC-HMS.

Once imported into HEC-HMS, the basin models were reviewed for connectivity. Adjustments were made where necessary. The established basin models for the five study watersheds are presented in Figure 6.1 through 6.5. The modelling elements shown in these figures are W=sub-watershed; L=lake or pond; J=junction of stream to stream, or stream to storage, or storage to storage; R=stream reach; D=diversion.

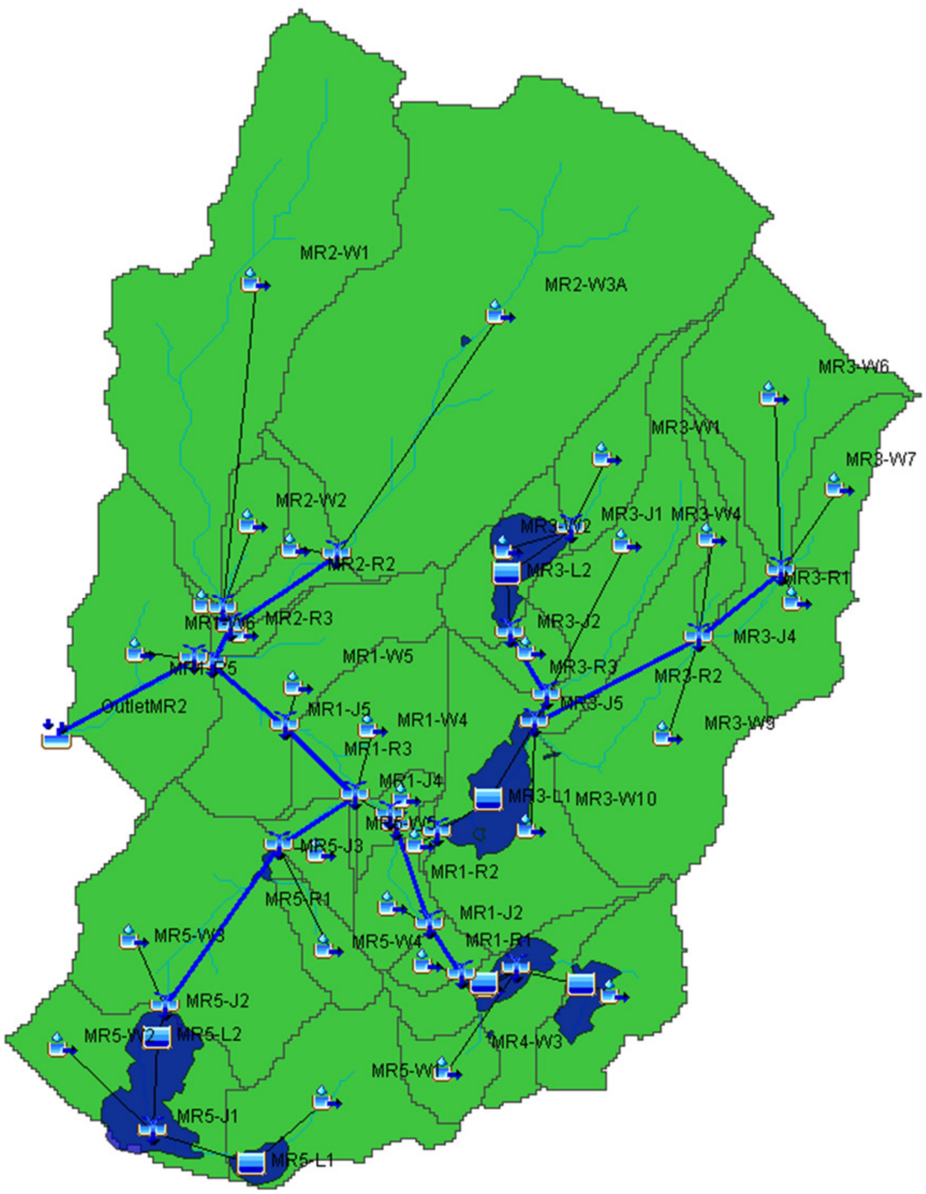



**Figure 6.1 Basin Model – Main River**

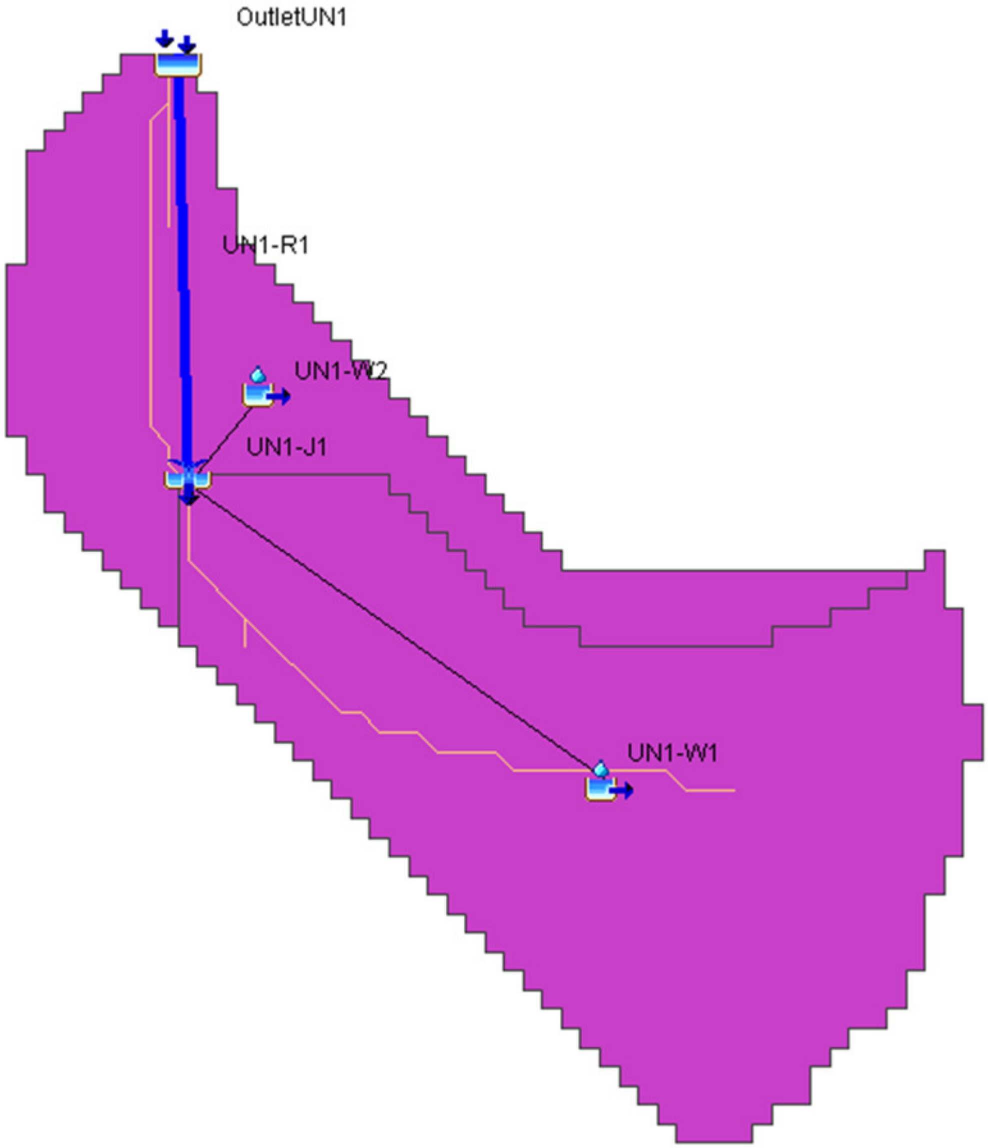



**Figure 6.2 Basin Model – Unnamed Stream**

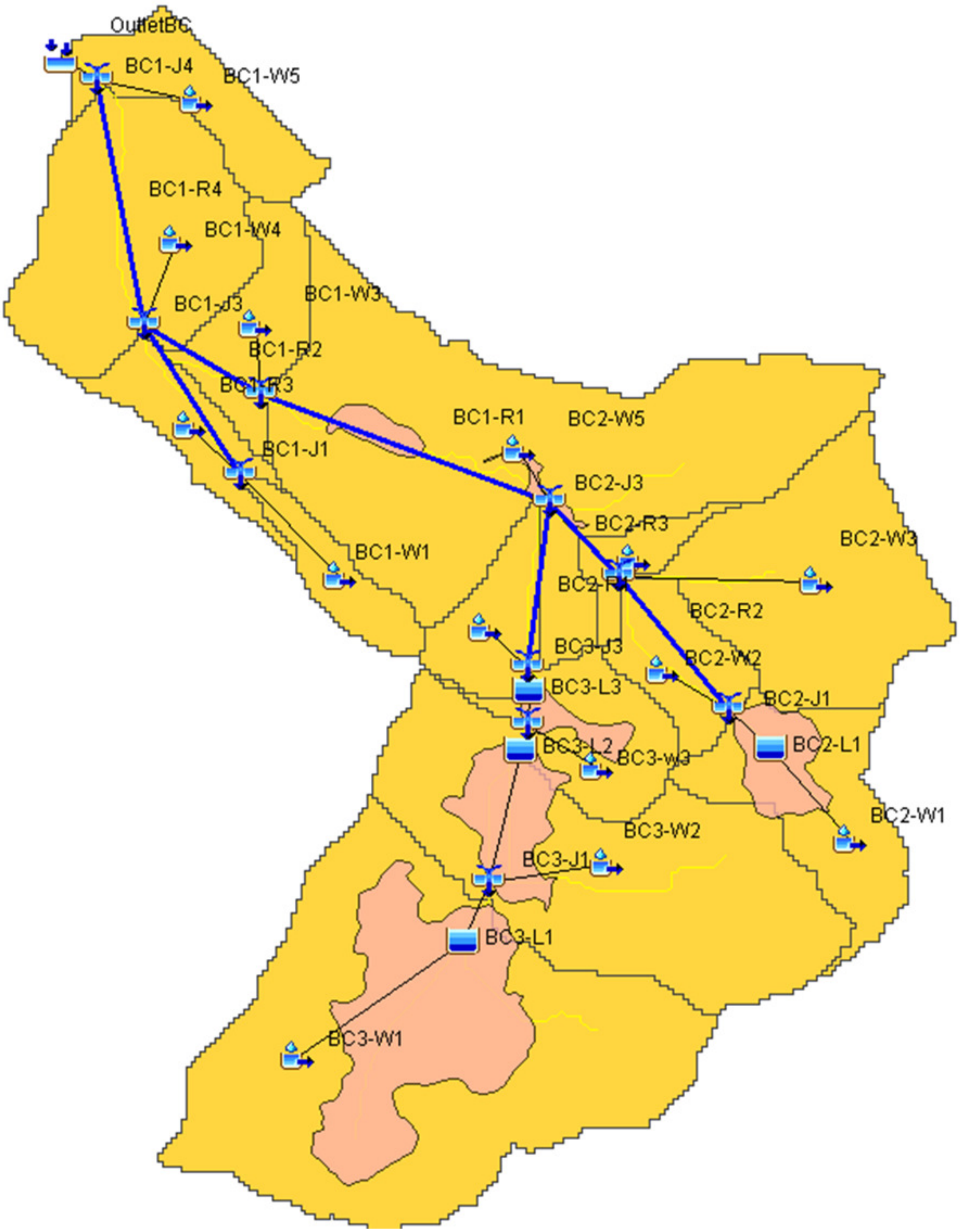



**Figure 6.3 Basin Model – Beachy Cove Brook**

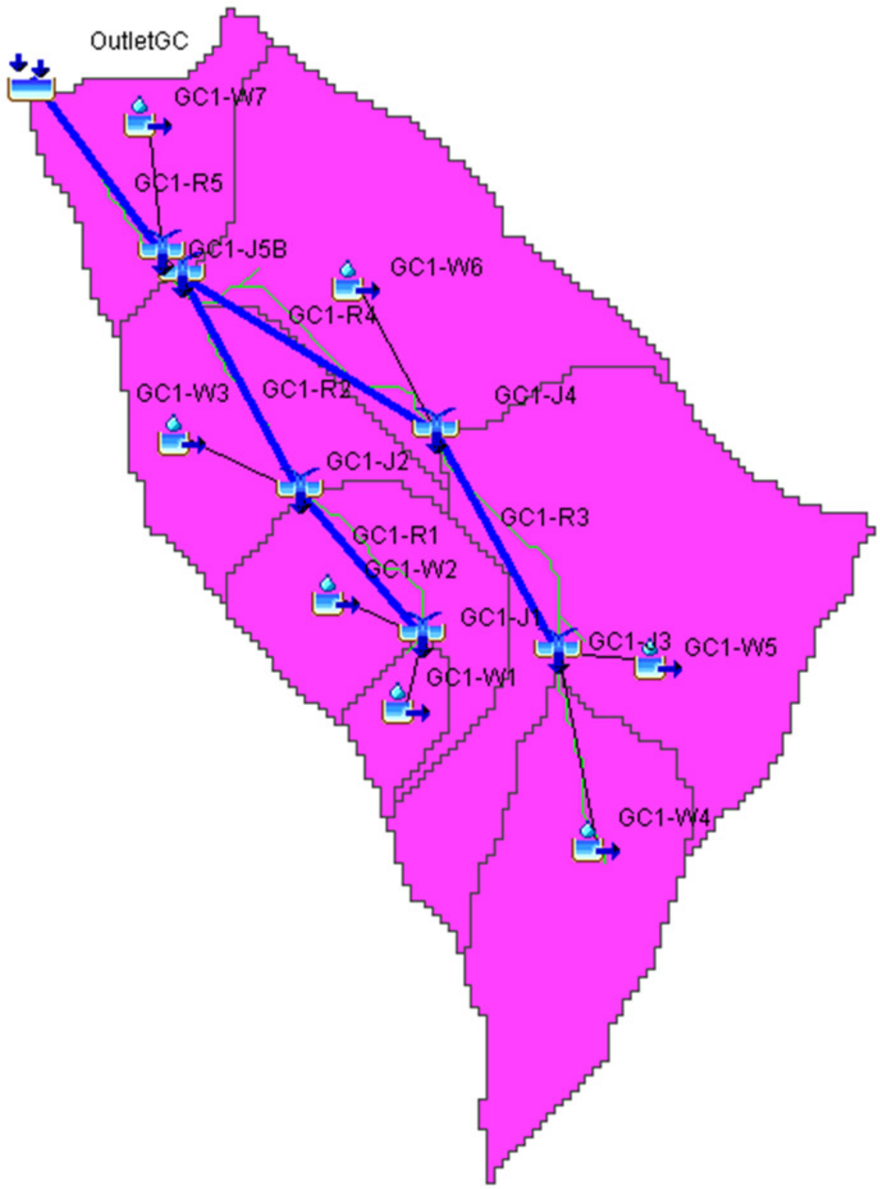



**Figure 6.4 Basin Model – Goat Cove Brook**

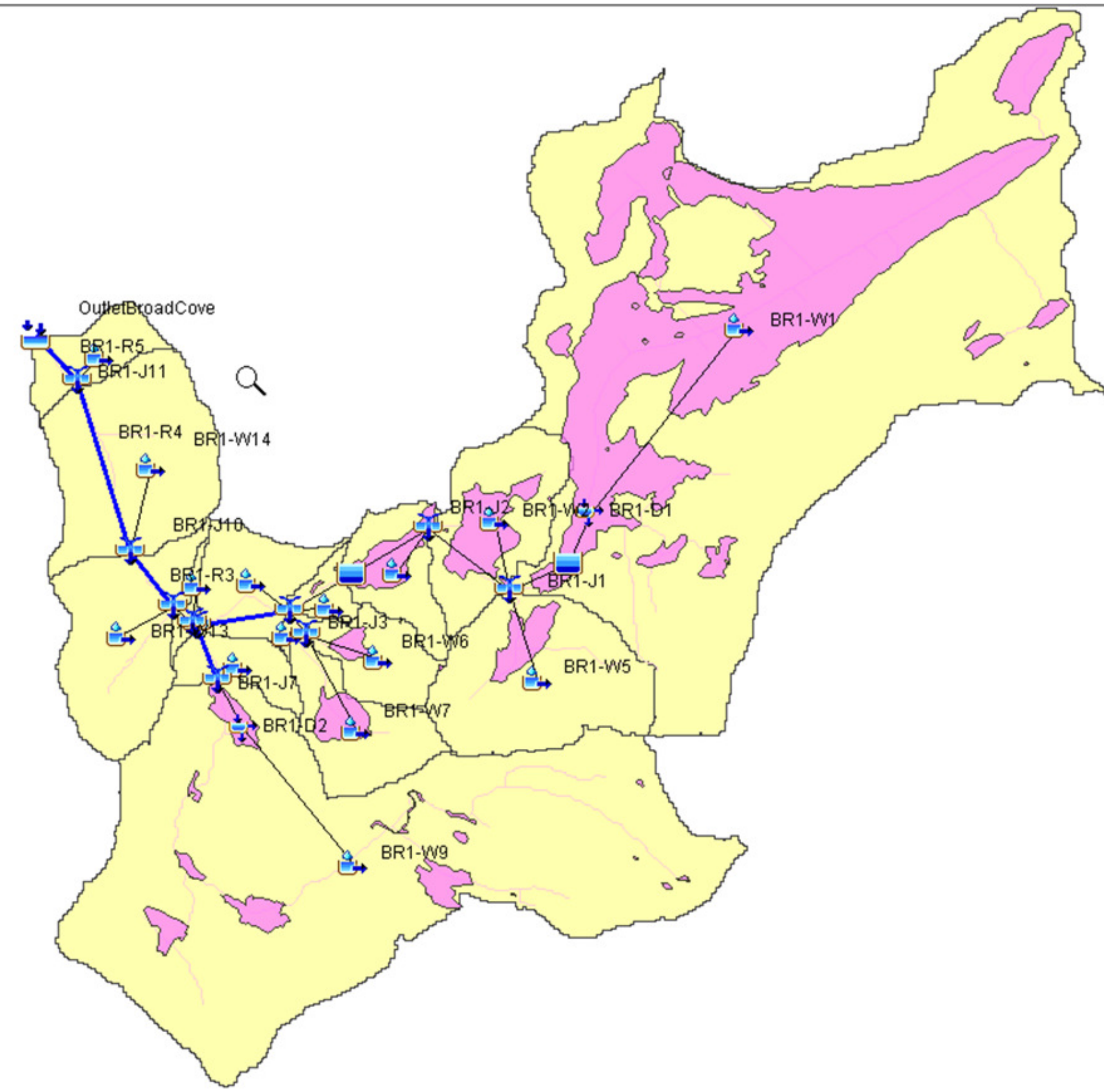



**Figure 6.5 Basin Model – Broad Cove River**

### 6.2.3 Storage and Discharge Characteristics for Lakes/Pond

There are numerous lakes and ponds in the study watersheds. One of the objectives of the hydrological modelling was to simulate the 1:20-year and 1:100-year AEP peak water levels in the lakes within the IMA&SN. Additionally, lakes and ponds can potentially provide significant attenuation (reduction) to peak flood flow downstream of these storage features. In order to simulate flood flow through the lakes and ponds, it was necessary to generate the elevation and storage curves as well as elevation and discharge relationships for these features.

Table 6.2 lists the lakes and ponds which have been incorporated into the HEC-HMS model. Elevation and storage relationships have been developed for these sites as provided in Appendix G. Most of the elevation and storage relationships were developed based on the LiDAR survey data (rather than the 1:50,000 CDED DEM). However, LiDAR coverage was not available for Windsor Lake, and the elevation storage curve for this lake was primarily based on the water surface area shown on the 1:50,000 map.

The discharges for five of the lakes contained in Table 6.2 are controlled by road crossings with culverts used for passing the flow. Discharges from these lakes are determined by the capacities of these culverts. The relationship between elevation and discharge (commonly referred to as rating curves) for each of these culverts was determined using the U.S. Federal Highway Administration (FHA) software HY-8, which is commonly used for sizing culverts for stream-crossings. This software has the capability to consider a large range of variations in the physical features associated with the culvert. The survey data for these culverts as well as LiDAR data were used as input to HY-8. The rating curves developed for these lakes and ponds are contained in Appendix G.

***Table 6.2 Lakes and Ponds Incorporated into the Hydrological Models***

| Watershed | Mapping Zones | Name | HEC-HMS ID | Discharge Control |
|---|---|---|---|---|
| Main River (MR) | MR3 | Millers Pond | MR3-L1 | Natural channel |
| | | Clements Pond | MR3-L2 | Culvert |
| | MR4 | Murrays Pond | MR4-L1 | Weir structure |
| | | Butlers Pond | MR4-L2 | Natural channel |
| | MR5 | Western Round Pond | MR5-L1 | Natural channel |
| | | Nearys Pond | MR5-L2 | Culvert |
| Beachy Cove Brook (BC) | BC2 | Olivers Pond | BC2-L1 | Natural channel |
| | BC3 | Hogans Pond | BC3-L1 | Culvert |
| | | Mitchells Pond North | BC3-L2 | Culvert |
| | | Hughs Pond | BC3-L3 | Culvert |
| Broad Cove River (BR) | | Windsor Lake | BR1-L1 | Crude Ogee Spillway |

The discharge for four of the lakes listed in Table 6.2 consists of natural channel. The discharge rating curves for these lakes and ponds were developed using the HEC-RAS model. LiDAR and survey data were used to create the geometry files for the channels. The Manning's roughness coefficient for the model was based on field survey records. The rating curves for these discharge channels are also contained in Appendix G.

The discharge from Murray's Pond is controlled by a weir structure. The discharge from Windsor Lake is controlled by a crude ogee concrete spillway. Discharges from these two storage sites were simulated using weir equations, and the development of rating curves was not necessary.

### 6.2.4 Flow Diversions

Windsor Lake, and to a lesser extent, the Little Power's Pond in the Broad Cove River watershed are used as the water supply sources for the City of St. John's. Records provided by the City of St. John's covering the period from 2005 to 2014 indicate that the monthly flow diversion for water supply from Windsor Lake ranged from 1.3 million $m^3$ to 2.6 million $m^3$ with an average of 1.6 million $m^3$ (calculated using the total withdraw volume divided by the number of month for the record period, or 0.62 $m^3/s$). The monthly flow diversion for water supply from the Little Power's Pond for the period from 2003 to 2014 ranged from zero to 0.9 million $m^3$ with an average of 0.15 million $m^3$ (0.058 $m^3/s$). The average intakes from these two sources were incorporated into the hydrological model.

### 6.2.5 Baseflow and Start Conditions

Baseflow generally contributes very little to peak flood flow during a large storm event. Nevertheless, a baseflow was estimated and incorporated into the hydrological model. This flow was adopted to be the average flow for the three month period from September to November, and was estimated to be approximately 0.04 $m^3/s/km^2$ based on the flow records for the hydrometric station of 02ZM006 and 02ZM016. The primary purpose of the hydrological modelling for this study was to estimate the peak flow in response to 24-hour rainfall event. Due to the relatively short duration of the hydrological simulation, the recession of the baseflow was not simulated. For initial conditions for lakes and ponds, it was assumed that the inflow was equal to the outflow at the beginning of the simulation and the water levels in the lakes and ponds were determined from the elevation and discharge relationships.

### 6.2.6 Rainfall Input

Environment Canada publishes IDF curves that are estimates of rainfall amounts for return frequencies between 1:20-years and 1:100-years and for durations of five minutes to 24 hours. The IDF data is the baseline data used by engineers and planners for determining the capacity of stormwater management infrastructure. The IDF information is typically prepared for meteorological stations with long and high resolution rainfall records. Near the study area, the IDF data is available for St. John's International Airport, which was prepared using data from 1949 to 1996. In the flood-risk-mapping study for the Town of Logy Bay-Middle Cove-Out Cove, the IDF information for St. John's was updated using rain gage data collected by the City of St. John's at Windsor Lake for the period from 2001-2010 (CBCL, 2012). Suitable data for this purpose was not available for the meteorological station at St. John's International Airport maintained by Environment Canada. Hydrologic modelling was completed using the updated IDF data by CBCL as shown in Table 6.3.

***Table 6.3 Rainfall Amounts in mm for Specified Durations and Frequencies***

| Duration | Frequency | | | |
|---|---|---|---|---|
| | Environment Canada | | CBCL (2012) | |
| | 20-yr | 100-yr | 20-yr | 100-yr |
| 5 min | 8.3 | 11.2 | 8.2 | 10.4 |
| 10 min | 11.9 | 15.7 | 11.9 | 15.0 |
| 15 min | 15.0 | 19.9 | 15.2 | 19.2 |
| 30 min | 20.8 | 27.2 | 22.6 | 28.5 |
| 1 h | 27.7 | 35.5 | 32.4 | 40.9 |
| 2 h | 40.2 | 53.1 | 46.8 | 59.8 |
| 6 h | 62.4 | 78.5 | 75.0 | 94.2 |
| 12 h | 76.5 | 94.5 | 96.0 | 121.2 |
| 24 h | 89.9 | 110.6 | 110.4 | 136.8 |

Simulations using the hydrologic models require the precipitation input to be provided in the form of rainfall hyetographs (rainfall distribution with time). The rainfall data as presented in Table 6.3 can be converted to rainfall hyetograph using the alternating block methodology. The 1:20-year and 1:100-year hyetographs developed based on the CBCL rainfall data presented in Table 6.3 using the alternating block method are shown in Figure 6.6.

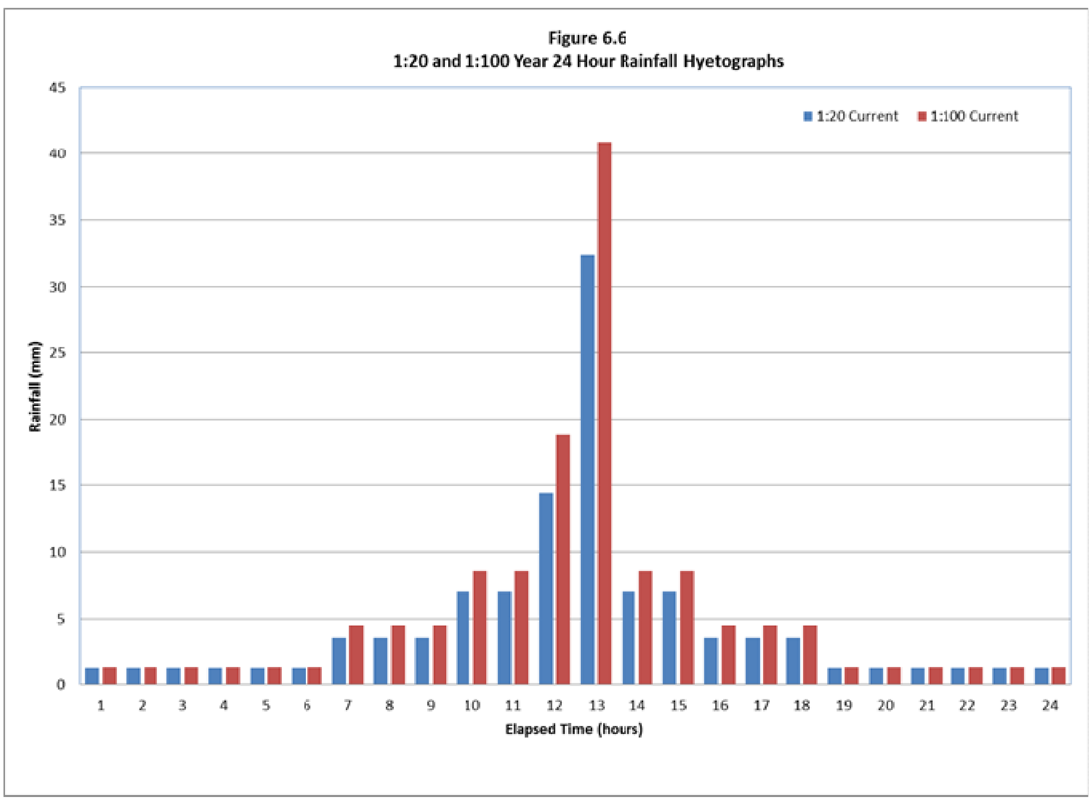



**Figure 6.6 1:20-Year and 1:100-Year Hourly Rainfall Hyetographs based on CBCL Data**

The 1:20-year and 1:100-year hyetographs were used in the hydrological model to simulate the 1:20-year and 1:100-year AEP peak flood flows. Additional rainfall data were used in the model calibration and verification. These included the following:

- Hourly rainfall records at the St. John's International Airport (Station No. 8403603) for the duration from September 20 to 22, 2010 when Hurricane Igor occurred (see Figure 6.7); and
- Hourly precipitation records at St. John's International Airport from September 23 to November 14, 2014 when the pressure transducers were deployed for the field program associated with this study.

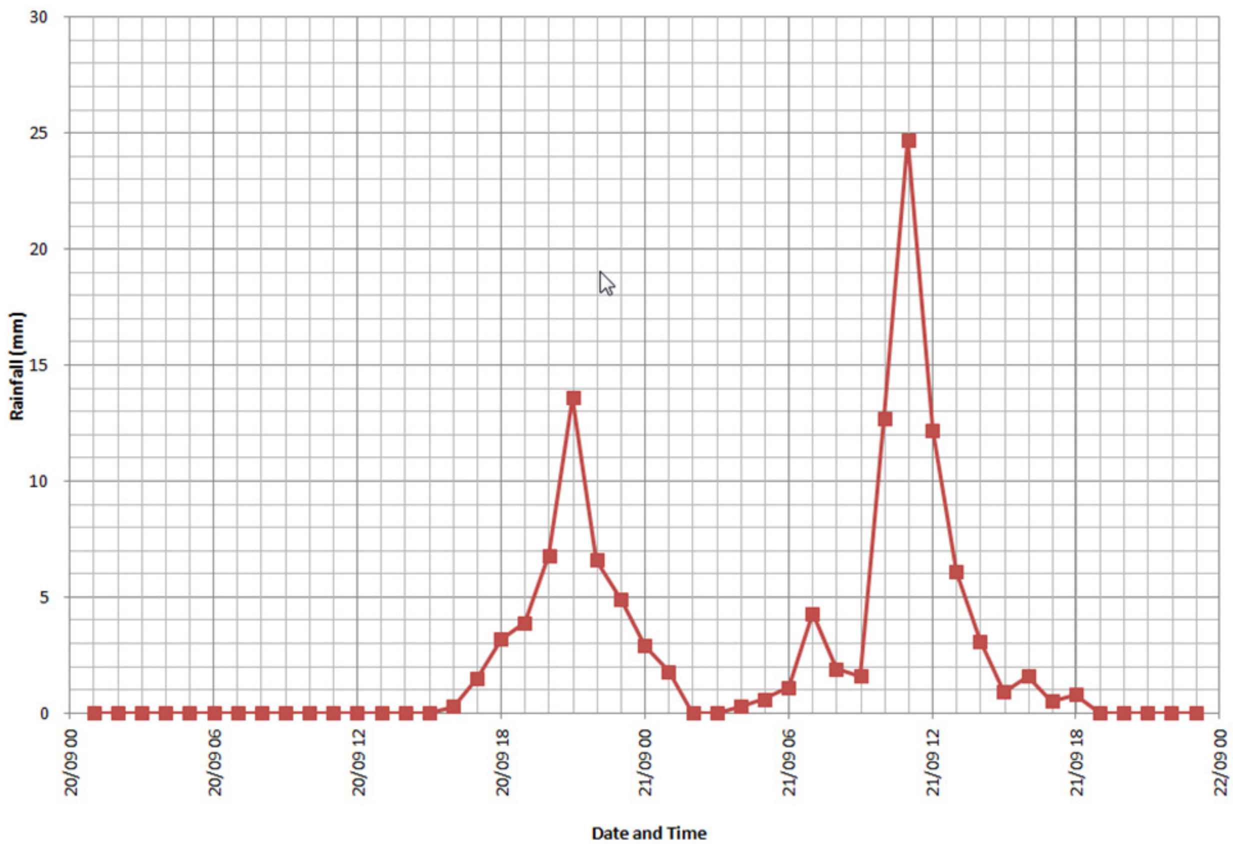



**Figure 6.7 Hourly Rainfall Hyetographs during Hurricane Igor in 2010**

### 6.2.7 Model Calibration and Verification

One of the objectives of the hydrological modelling was to simulate the peak flows at points of interest within the IMA&SN to be used as input in the subsequent hydraulic simulation. The runoff generation potential depicted by the parameter CN is an important variable contributing to peak flows. The other variables include the antecedent moisture condition in the watersheds and sub-watersheds, and the water levels in the lakes and ponds. All these variables are dynamic and change with time. Calibration of the hydrological model, in the conventional sense, will require detailed information on the distribution and variation of these variables. This was not considered to be feasible for this study. Nevertheless, a quasi-calibration was conducted to establish confidence that the peak flood flows simulated were consistent with the flow records in the general physiographic region under somewhat wet (thus conservative) conditions.

The following exercises were conducted for the verification of the hydrological models:

- Comparison with regional flood frequency analysis;
- Comparison with flow recorded for the Hydrometric Station 02ZM006 during Hurricane Igor in 2011; and
- Comparison with peak flow events during the field survey program in 2014.

Comparison with Regional Flood Frequency Analysis

The 1:20-year and 1:100-year AEP instantaneous peak flows obtained through the RFFA at the sites of the five pressure transducers are summarized in Table 6.1. The peak 1:20-year and 1:100-year peak flows simulated with the hydrological models using the hyetographs presented in Figure 6.6 and the CN distribution presented in Figure 5.4 are also summarized in Table 6.1. The 1:20-year and 1:100-year flows simulated with the models were consistently lower than the RFFA estimates, sometimes significantly. This appears to imply that the runoff generation potential from the study watersheds is higher than indicated by the CN distribution shown in Figure 5.4.

As indicated previously, the CN as shown in Figure 5.4 was based entirely on the drainage characteristics of the soil covering the study watersheds. However, much of the Avalon Peninsula was described as barren where the soil deposit in many areas is thin and discontinuous, while the underlying bedrock has characteristically low permeability. The unique geological condition of the study area dictates that, while the drainage characteristics of the overburden soil is important in determining the runoff potential, the thickness of the soil and the permeability of the underlying bedrock should also be taken into consideration. For the study watersheds, the permeability of the underlying bedrock likely causes an increase in the runoff potential.

To reflect the effect of the thin overburden deposit and the low permeability of the underlying bedrock on the runoff generation potential, the CNs estimated for all the sub-watersheds delineated in the hydrological models were increased by 10%. With this increase, the flows estimated for the pressure transducer sites were generally consistent with the flows estimated with RFFA.

Pressure Transducer 5 was located downstream of Windsor Lake, and the peak flow at this location is significantly influenced by the attenuation effect of Windsor Lake. As discussed previously, the RFFA equations cannot take into consideration abnormally large storage features in a watershed, and will overestimate the peak flow. Flow comparisons between RFFA and HEC-HMS modelling was not made for this location.

Hurricane Igor

Hurricane Igor was the most destructive tropical cyclone to strike the island of Newfoundland on record (http://en.wikipedia.org/wiki/Hurricane_Igor). The combination of a stationary front and significant moisture from Hurricane Igor resulted in unprecedented rainfall across parts of eastern Newfoundland, leading to widespread flooding. In Bonavista, more than 250 mm was estimated to have fallen between September 20 and 21, 2010. The Avalon Peninsula was probably spared the worst, with a total of 118 mm rain recorded for St. John's for the same period. The peak hourly precipitation for St. John's was recorded to be approximately 25 mm (less than 1:20-year event).

The peak flow simulated for the hydrometric station of 02ZM006 using the hydrological model during Hurricane Igor was 6.0 $m^3$/s. The peak flow recorded for this station during Hurricane Igor was 7.7 $m^3$/s, which was 28% higher than the simulated flow.

Peak Flow Events during the 2014 Field Program

Pressure transducers were deployed for the period from September 23 to November 14, 2014 at five locations. The recorded water levels at the five locations are contained in Appendix D. 15-minute flow records were obtained for three hydrometric stations in the vicinity of the project area, including the following:

- 02ZM006: Northeast Pond at Northeast Pond (within the Main River watershed);
- 02ZM008: Waterford River at Kilbride (approximately 12 km to the west of Portugal Cove); and
- 02ZM016: South River near Holyrood (approximately 35 km to the south of Portugal Cove).

The hydrometric station of 02ZM006 is located in the Main River watershed. The other two are located outside, but in close proximity to the study area. The flows for these three hydrometric stations were converted to unit-area basis for comparison. Two peak flow events were examined for the duration when the pressure transducers were deployed in the field programs, one occurred on October 19, 2014, and one occurred on November 3, 2014. The unit-area flows for these three stations are shown in Figure 6.8. Hourly rainfall data was also obtained from Environment Canada for the duration of the field programs. The hourly precipitation during the two peak flow events is also shown in Figure 6.8.

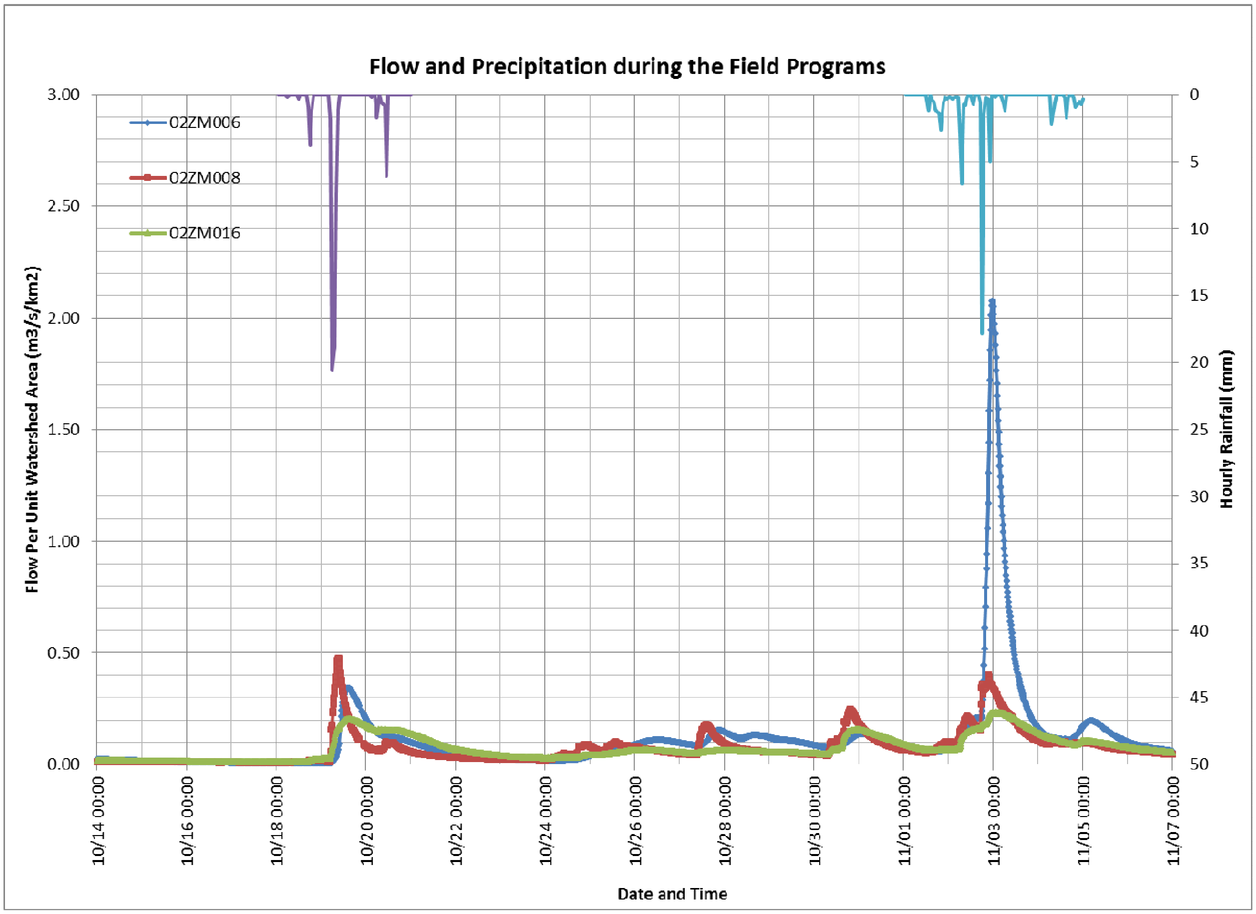



**Figure 6.8 Peak Flow Events and Precipitation during the Field Program**

Figure 6.8 indicates that the unit-area flows for the three hydrometric stations generally exhibit similar co-variations. The unit-area flows for the three hydrometric stations during the event of October 19, 2014 were comparable. However, the unit-area peak flow for the hydrometric station of 02ZM006 during the November 3, 2014 event was significantly higher than for the other two hydrometric stations and about six times higher than the peak flow experienced on October 19, 2014. The peak hourly rainfall recorded at St. John's International Airport (Climate Station 8403506) during the November 2-3, 2014 event was slightly lower than the peak rainfall during the October 19, 2014 event, which explains the exceptionally high peak flow experienced on November 3, 2014 for the hydrometric station 02ZM006.

Water Survey Canada (WSC) was consulted regarding the peak flow recorded for 02ZM006 during the November 3, 2014 event. WSC confirmed that this high flow was not due to an error. Other information obtained through the study collaborated with the exceptionally high peak flow experienced at 02ZM006 on November 3, 2014:

- the five pressure transducers deployed in the field programs all recorded significantly higher peaks on November 3 than on October 19, 2014; and
- the Town of PCSP reported that there was significant flooding during the November 3 storm and the extent of the flooding was comparable to Hurricane Igor in 2010 (Chris Milley, personal communication).

The significantly larger flood peak experienced in the project area during the November 3, 2014 event cannot be explained with the precipitation records in St. John's (Station #8403603). This leads to the conclusion that, during the rain event of November 3, 2014, the Portugal Cove area received significantly more rainfall than at the recording rain gauge site in St. John's. Therefore, the precipitation data recorded at St. John's during the November 3, 2014 event was not considered suitable for simulating the peak flows experienced in the Portugal Cove area.

A much smaller peak was recorded on October 19, 2014. The established hydrological models were based on the CNs for ARC III, which was conservative and suitable for simulating a large rainfall event. For small rainfall events, these models underestimate the infiltration and overestimate the peak flow. The peak runoff event recorded on October 19, 2014 was considered too small to be suitable for calibrating the hydrological models.

### 6.2.8 Sensitivity Analysis

The most significant input parameters for the hydrological modelling include the following:

- weighted CN for each of the sub-watersheds; and
- hyetographs for each of the sub-watersheds for 1:20-year and 1:100-year rainfall events.

The CN adopted in this hydrological modelling was based on wet soil conditions in order to be conservative. However, as discussed previously, there are uncertainties regarding the adopted CN values, which could result in higher values for CN than adopted in the model. One of the uncertainties is the effect of the thin soil cover over most of the study watersheds and the low permeability of the underlying bedrock.

For sensitivity analysis, the weighted CN values adopted in the model were increased by 10%, and the flows simulated for the sites of the five transducers were compared with the flows simulated with the adopted CN values. A summary of the comparison is provided in Table 6.4. It is seen that, with an increase of CN by 10%, the flows will increase by approximately 22 to 52%.

There was uncertainty/error with the estimate for the 1:20-year and 1:100-year rainfall hyetographs. This uncertainty can be reduced with the increased length of the data sequence. Environment Canada uses 95% confidence limits as a measure of this uncertainty/error. For example, for 24-hour 1:100-year rainfall the 95% confidence limits were reported to be +/- 19.2 mm, or +/- 17%.

For sensitivity analysis, the input hyetograph was increased by 10%, and the flows simulated for the sites of the five transducers were compared with the flows simulated with the adopted hyetograph. A summary of the comparison is provided in Table 6.4. It is seen that, with an increase of precipitation by 10%, the flows will increase by approximately 11 to 31%.

***Table 6.4 Summary of Sensitivity Analysis***

| Location | HEC-HMS Element | Drainage Area | Return Frequency | Flow with Adopted CN and Precipitations | Sensitivity of CN | | Sensitivity of IDF | |
|---|---|---|---|---|---|---|---|---|
| | | | | | Flow with 10% Increase of CN | Percentage Increase | Flow with 10% Increase of Precipitation | Percentage Increase |
| | | (km$^2$) | (yrs) | (m$^3$/s) | (m$^3$/s) | (%) | (m$^3$/s) | (%) |
| **PT1** | MR1-J6A | 17.00 | 1:20 | 25.2 | 35.8 | 42.1 | 31.3 | 24.2 |
| | | | 1:100 | 39.1 | 52.3 | 33.8 | 47.8 | 22.3 |
| **PT2** | MR1-J3A | 6.50 | 1:20 | 10.6 | 16.1 | 51.9 | 13.9 | 31.1 |
| | | | 1:100 | 17.2 | 23.8 | 38.4 | 21.8 | 26.7 |
| **02ZM006** | MR2-W3A | 3.64 | 1:20 | 5.1 | 6.9 | 35.3 | 6.1 | 19.6 |
| | | | 1:100 | 7.8 | 10.0 | 28.2 | 9.3 | 19.2 |
| **PT3** | BC2-J1 | 0.67 | 1:20 | 1.8 | 2.2 | 22.2 | 2.0 | 11.1 |
| | | | 1:100 | 2.3 | 2.8 | 21.7 | 2.6 | 13.0 |
| **PT4** | GC1-J5A | 2.41 | 1:20 | 4.9 | 6.5 | 32.7 | 5.8 | 18.4 |
| | | | 1:100 | 7.2 | 9.0 | 25.0 | 8.4 | 16.7 |
| **PT5** | BR1-J8A | 34.59 | 1:20 | 10.4 | 14.6 | 40.4 | 13.1 | 26.0 |
| | | | 1:100 | 17.4 | 22.7 | 30.5 | 21.2 | 21.8 |

## 6.3 Modelling Results

As indicated previously, one of the objectives of the hydrological modelling was to determine the 1:20-year and 1:100-year peak flows at points of interest, which will be used as input in the subsequent hydraulic modelling. HEC-HMS model generates inflow and outflow hydrographs (including peaks) for every hydrological element shown in the basin models (see Figures 6.1 to 6.5). This information is saved in an internal file and can be accessed automatically by the HEC-RAS model employed in the hydraulic modelling.

Another objective of the hydrological modelling was to estimate the 1:20-year and 1:100-year peak water levels in the lakes and ponds within the IMA&SN. The maximum predicted change in water level for a 1:20-year event and 1:100-year event is 0.1 m. A summary of the water levels is provided in Table 6.5.

**Table 6.5 Estimated 1:20-Year and 1:100-Year Water Levels in Lakes and Ponds**

| Watershed | Mapping Zones | Lake/Pond ID | Current Development | | Note |
|---|---|---|---|---|---|
| | | | 1:20 | 1:100 | |
| Main River (MR) | MR3 | MR3-L1 | 116.8 | 116.9 | Millers Pond |
| | | MR3-L2 | 149.7 | 149.8 | Clements Pond |
| | MR4 | MR4-L1 | 149.2 | 149.3 | Murrays Pond |
| | | MR4-L2 | 150.4 | 150.5 | Butlers Pond |
| | MR5 | MR5-L1 | 138.8 | 138.8 | Western Round Pond |
| | | MR5-L2 | 138.2 | 138.3 | Nearys Pond |
| Beachy Cove Brook (BC) | BC2 | BC2-L1 | 146.1 | 146.1 | Olivers Pond |
| | BC3 | BC3-L1 | 136.6 | 136.7 | Hogans Pond |
| | | BC3-L2 | 136.6 | 136.7 | Mitchells Pond North |
| | | BC3-L3 | 135.9 | 136.0 | Hughs Pond |

# 7.0 HYDRAULIC ANALYSIS

The objective of the hydraulic analysis was to simulate water surface profiles resulting from the 1:20-year and 1:100-year AEP flows estimated in Section 6.0. In the hydraulic analysis, water elevation profiles resulting from 1:20-year and 1:100-year flows were estimated for each of the streams listed in Table 3.1. The water level profiles for the streams were integrated with the water levels for the lakes and ponds. The computed water surface elevations were then used in conjunction with the LiDAR topographic data to delineate inundation zones within the IMA&SN.

The following sections describe the development and calibration of the hydraulic model, as well as simulation of various flood events.

## 7.1 Hydraulic Model Development

For a given location, the water level elevation in response to the peak flood flow is determined by the geometric characteristics (dimensions and resistance to flow) of the channel and flood plain as well as by the magnitude of the flow. The geometric characteristics of the channel and the magnitude of the peak flow are the basic input requirement in the hydraulic analysis. It is assumed that the flow is steady and equal to the 1:20-year or 1:100-year instantaneous peak flow.

The Request for Proposal (RFP) requires that for simulating the hydraulic behaviour of the streams, the latest version of the non-proprietary HEC-RAS and the Geo-RAS extension be used. HEC-RAS (USACE, 2002) is a hydraulic modelling computer program developed by the USACE to simulate water surface profiles for steady and gradually varied flow in open channel watercourses. The HEC-GeoRAS is the interface between HEC-RAS model and GIS database and is used for preparing geometric input to the HEC-RAS model, as well as for converting the HEC-RAS output into GIS mapping presentations. HEC-RAS is capable of modelling complicated networks with multiple reaches and tributaries. For this project, the HEC-GeoRAS 4.3.93 for ArcGIS 9.3 and HEC-RAS 4.1.0 were used in the hydraulic analysis, which were the current latest versions.

The simulation of water surface profiles using HEC-RAS and HEC-GeoRAS typically involves the following steps:

- Prepare geometric input using HEC-GeoRAS and high accuracy LiDAR and field survey data and import the geometric data into HEC-RAS;
- Prepare flows and boundary conditions for the HEC-RAS model;
- Model calibration, simulation and export of modelling results to HEC-GeoRAS; and
- Delineation of inundation zones using HEC-GeoRAS and GIS tools.

## 7.2 Preparation of Geometric Input

The geometric input to the HEC-RAS models for this project include the following:

- Cross-sections;
- In-line structures including culverts and bridges; and
- Manning's roughness coefficient.

### 7.2.1 Cross-Sections

Cross-sections associated with the stream-crossing structures (bridges and culverts) were surveyed during the field program. Additional cross-sections were also surveyed at predetermined locations between the structures. Surveys of these cross-sections typically extended a minimum of 5 m beyond the top of the banks. Details related to the locations of the cross-sections were discussed in Section 4.0 as well as in Appendix D. All these cross-section surveys included the channel geometry below the water surface.

The surveyed cross-sections associated with the stream-crossing structures as well as between the structures were incorporated into the models. The surveyed cross-sections were compared with the LiDAR data as they were being incorporated into the model. The survey data and the LiDAR data generally showed good agreement beyond the top of the banks and on the flood plains. However, some deviations between the two data sources were observed within the channel as expected.

The surveyed cross-sections generally extended approximately 5 m beyond the top of banks, and this was not sufficient for the inundation modelling. The LiDAR data was used to augment and extend the survey cross-sections along the potential flood plains.

The surveyed cross-sections were supplemented by additional cross-sections entirely based on the LiDAR data. The additional cross-sections were expected to significantly improve the resolution of the model simulations.

### 7.2.2 Stream-Crossing Structures

A list of stream-crossing structures was provided in Table 4.1, which was based on the stream network shown on 1:50,000 topographic map, and the road network shown on 1:2,500 municipal map. These structures were included in the field survey program or surveyed previously by NLCEL as discussed in Section 4.0.

The dimensions, elevations, and design details of the structures listed in Table 4.1 were extracted from the surveys and incorporated into the HEC-RAS models. These dimensions and elevations were verified using the LiDAR information to the extent feasible.

### 7.2.3 Dams and Flow Control Structures

The weir structure downstream of Murray's Pond is located within the IMA&SN. The effect of this structure on the water level in Murray's Pond was simulated in the hydrological modelling as discussed in Section 6.0. It was not necessary to incorporate this structure in the HEC-RAS model. The dam and spillway structure at the discharge of Windsor Lake is located outside of the IMA&SN, and simulation of this structure in the HEC-RAS model was not required.

### 7.2.4 Manning's Roughness Coefficient

Manning's roughness coefficient is a parameter measuring the resistance of the stream channel to flow. The higher the value of this parameter, the higher the resistance to flow, and therefore the higher the water level for the same flow.

Manning's roughness coefficient is determined by several factors, the most significant of which include composition of channel substrate materials, vegetation conditions in the channel and along the banks, and stream alignment. Information affecting the Manning's roughness coefficient was collected during the survey of the cross-sections and documented on the survey forms contained in Appendix D.

Table 7.1 summarizes Manning's roughness coefficient adopted for this study. It was designed to consider the incremental effects of the various factors affecting this parameter by taking into consideration the general characteristics of the streams in the project area. This parameter was estimated for the main channel and for the flood plain separately. Generally, the flood plain will have a higher roughness coefficient than the main channel.

For cross-sections surveyed in the field program, Manning's roughness coefficient was estimated based on the survey records using Table 7.1. For cross-sections generated from the LiDAR data, Manning's roughness coefficient was estimated based on adjacent surveyed cross-sections.

*Table 7.1 Manning's Roughness Coefficient*

| | | |
|---|---|---|
| **Manning's Roughness Coefficient for Main Channel** | **Channel Vegetation Condition** | |
| | Little vegetation | 0.030 |
| | Some vegetation | 0.035 |
| | Dense vegetation | 0.050 |
| | **Bank Vegetation Condition - Add** | |
| | Light brush/trees | 0.000 |
| | Medium brush/trees | 0.015 |
| | Heavy brush/trees | 0.030 |
| | **Channel Substrate Condition - Add** | |
| | Earth material (sand, silt, etc.) | 0.000 |
| | Bedrock, gravel, cobble and occasional boulders | 0.015 |
| | Cobbles with frequent large boulders | 0.030 |
| | **Channel Alignment Condition - Add** | |
| | Fairly regular, relatively straight and uniform | 0.000 |
| | Irregular, winding or sluggish | 0.020 |
| **Manning's Roughness Coefficient for Flood Plain** | **Flood Plain** | |
| | No to sparse vegetation | 0.035 |
| | Medium to dense grass | 0.050 |
| | Medium to dense brush | 0.100 |
| | Medium to dense trees | 0.150 |

Source: Based on Table 4.2 in Handbook of Steel Drainage & Highway Construction Products, American Iron and Steel Institute, 1984.

### 7.2.5 Preparation and Export of Geometric Data Using HEC-GeoRAS

All the geometric input discussed in the preceding sections was prepared using HEC-GeoRAS. The geometric files prepared were then exported to HEC-RAS model.

## 7.3 Flow and Boundary Conditions

### 7.3.1 Reach Flows

1:20-year and 1:100-year flows at locations of interest were simulated in the hydrological modelling and stored in an internal file. This file was accessed by HEC-RAS model automatically to enable the simulation to be executed.

### 7.3.2 Sea Level Conditions

All five watersheds included in this study eventually discharge into Conception Bay. Therefore, the water level in Conception Bay provides a boundary for the HEC-RAS model. However, it was discussed previously that the coast line in the project area is steep and a dominant portion of the IMA&SN is above the elevation where it can be influenced by the water level in Conception Bay (see Figure 2.4 for the Main River).

The components of the water level in Conception Bay are as follows:

- Tide: Tide is influenced by the movement of the moon and the sun, and is cyclical with fluctuation from low tide to peak tide to low tide during a 12.4-hour period.
- Storm Surge: Storm surge is caused by the wind blowing over the surface of the ocean, as well as by the low pressure associated with a weather system.
- Future sea-level rise due to climatic change.

Table 7.2 presents relevant tidal elevations for the study area extracted from the Canadian Hydrographic Chart for Conception Bay. Probable maximum storm surge is estimated from inspection of the 40-year return period hindcast values by Bernier and Thompson (2006) as illustrated in Figure 7.1. Future predictions for sea level rise are made based on predictions presented in Batterson and Liverman (2010).

***Table 7 2 Water Levels, Storm Surge, and Sea Level Rise Projections in Conception Bay***

| Description | Elevation (m, geodetic) |
|---|---|
| HHWMT (m) | 0.5 (1) |
| HHWLT (m) | 0.8 (1) |
| Probable Maximum Surge (m) (2) | 0.95 |
| Sea level rise 2020 (m) (3) | 0.06 |
| Sea level rise 2050 (m) (3) | 0.27 |
| Sea level rise 2080 (m) (3) | 0.63 |

**Notes:**

1. Source: Canadian Hydrographic Chart #4847.
2. Source: Figure 10 in Bernier and Thompson (2006).
3. Source: Table 3 and Figure 4 in Batterson and Liverman (2010); Zone 1 for Goulds and Petty Harbour.

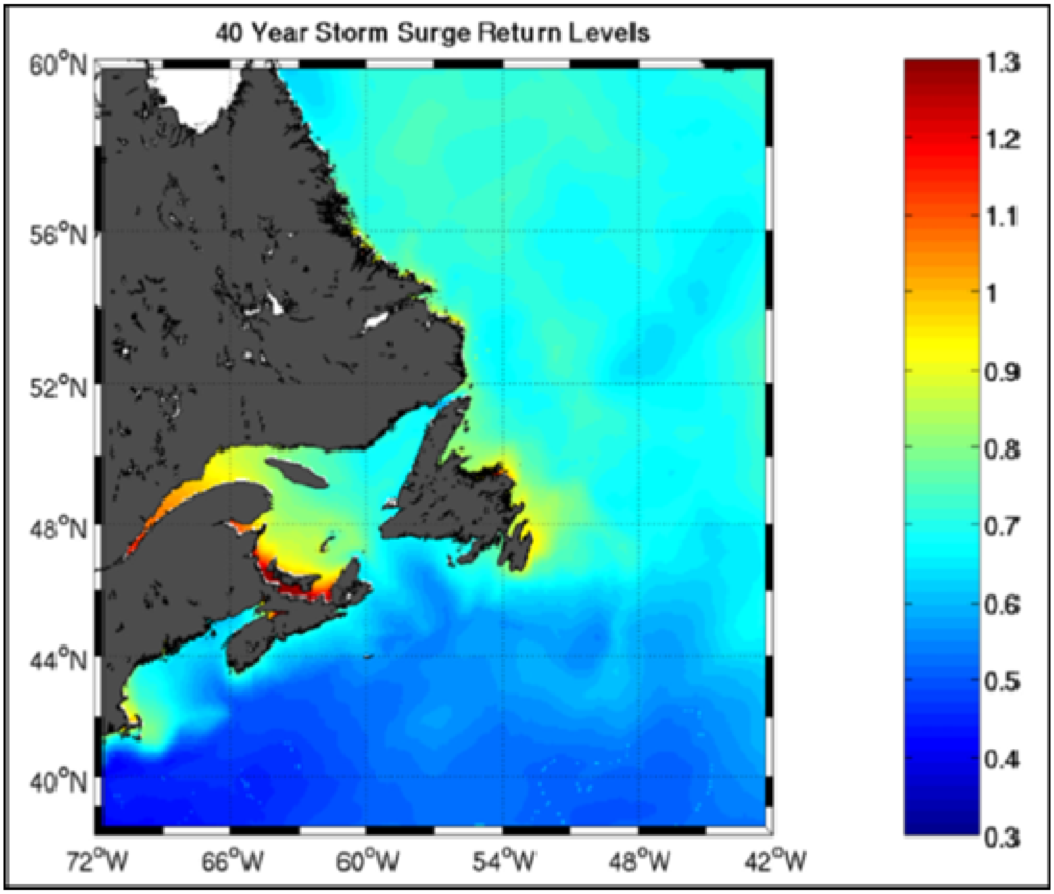
**Acronyms (from Forrester, 1983):**

MWL: is the height above chart datum of the mean of all hourly observations used for the tidal analysis and that particular place (DFO, 2012a), or, the average of all hourly water levels over the available period of record.

HHWMT: is higher high water, mean tide, which is the average of all the higher high waters from 19 years of predictions.

HHWLT: is higher high water, large tide, which is the average of the highest high waters, one from each of 19 years of predictions.

HHW: higher high water.




Source: Bernier and Thompson, 2006.

**Figure 7.1 40-Year Return Level of Extreme Storm Surges (metres)**

For the purposes of this study, the water level in Conception Bay was computed as the HHWLT of 0.80 m (geodetic) plus a storm surge of 0.95 m for existing conditions. This provides a combined total of 1.75 m which was used as the downstream boundary representative of existing conditions. This approach was adopted in the previous studies conducted for WRMD. However, it should be noted that tide fluctuation is independent of storm surge and peak flood flow from the watersheds, and a tide will subside from its peak within a short timeframe. Therefore, the probability of the peak tide occurring at the same time of extreme storm surge and peak flood flow from the watersheds is extremely low. This assumption is considered very conservative but has little bearing on the water surface profiles simulated.

## 7.4 Hydraulic Model Calibration/Validation

Manning's roughness coefficient is the single most important parameter requiring calibration. This parameter varies with location along a stream and with the water level. The calibration data for this parameter should ideally include flood water levels at representative locations along the streams being modelled. Suitable data for this purpose is not available. The pressure transducers deployed in the field program captured a significant high water level event on November 3, 2014, which should enable the model calibration to be conducted for the stream segments in the vicinity of the pressure transducers. However, due to the spatial variability of the Manning's roughness coefficient, model calibration for one stream segment cannot represent calibration for the other stream segments.

In the absence of suitable calibration data for the hydraulic model, Amec Foster Wheeler took the following measures to establish confidence of the hydraulic models and the selection of the values for the Manning's roughness coefficient:

- Comparison with the results from the previous inundation studies; and
- Consultation with the Town of PCSP regarding their experiences during significant historical flood events.

*Comparison with Results from Previous Inundation Studies:*
Two previous inundation studies have been conducted covering a portion of the IMA&SN identified for this study. These are as follows:

**Flood-risk Mapping Study of the Portugal Cove - St. Philip's, and Outer Cove, March 1996:** This study was commissioned by the then Newfoundland Department of Environment and conducted by CORETEC Incorporated in association with Davis Engineering & Associates Limited. This study included modelling to estimate the 1:20-year and 1:100-year AEP flood profiles and delineation of the inundation areas for the following areas:

- Main River from Bauline Line Extension to Churchill's Road; and
- Broad Cove River from approximately Dan's Road to just downstream of John's Road.

**Town of Portugal Cove – St. Philip's Storm Water Management Plan, Murray's Pond Brook/Main River and Broad Cove River, May, 2010:** This study was commissioned by the Town of PCSP and conducted by NLCEL. This study included delineation of 1:100-year AEP inundation zones for Main River from Murray's Pond Brook to its discharge into Conception Bay, and Broad Cove River from Little Power's Pond to its discharge into Conception Bay.

Flood zones were incorporated into the environmental protection map developed by the Town based on the historical flood studies. Digital shape files for these flood zones were obtained from the Town. The digital shape files showing the flood zones were compared with the flood zones presented in the two reports indicated above and some discrepancies were noted. For Broad Cove River, the flood zone from the 2010 NLCEL was not incorporated and the shape file provided by the Town appears to be based on the earlier 1996 study by CORETEC. The Town of Portugal Cove - St. Philip's was consulted and it was not clear how the discrepancies originated.

Figure 7.2 and Figure 7.3 compares the 1:100-year inundation areas delineated in this study with those adopted in the environmental protection map by the Town for Main River and Broad Cove River. The Manning's roughness coefficient was based on the field observations and the values summarized in Table 7.1 with no adjustment made. It is seen that the extent in width of the inundation zones simulated in this study is generally comparable with those from the previous studies. It is also observed that, due to the availability of high accuracy LiDAR data for this study, the inundation zones delineated shows significantly more details, and are more likely to be accurate.

**Consultation with the Experiences of the Town of PCSP:** Amec Foster Wheeler undertook an exercise to review the inundation zones developed from the hydraulic modelling with Mr. Chis Milley, Town Manager, who has intimate knowledge of the community and historical flooding. A section by section review was conducted, and the flooding conditions experienced during Hurricane Igor and during the November 3, 2014 flood events were used as a reference. The modelling results were found to be generally consistent with the flooding experiences in most areas of the watersheds. Mr. Milley identified a number of areas with potential discrepancies and/or deficiencies. Model input for these areas were reviewed and revised as necessary. A summary of the review of the 1:100-year inundation and delineation conducted with Mr. Chris Milley is provided in Appendix H.

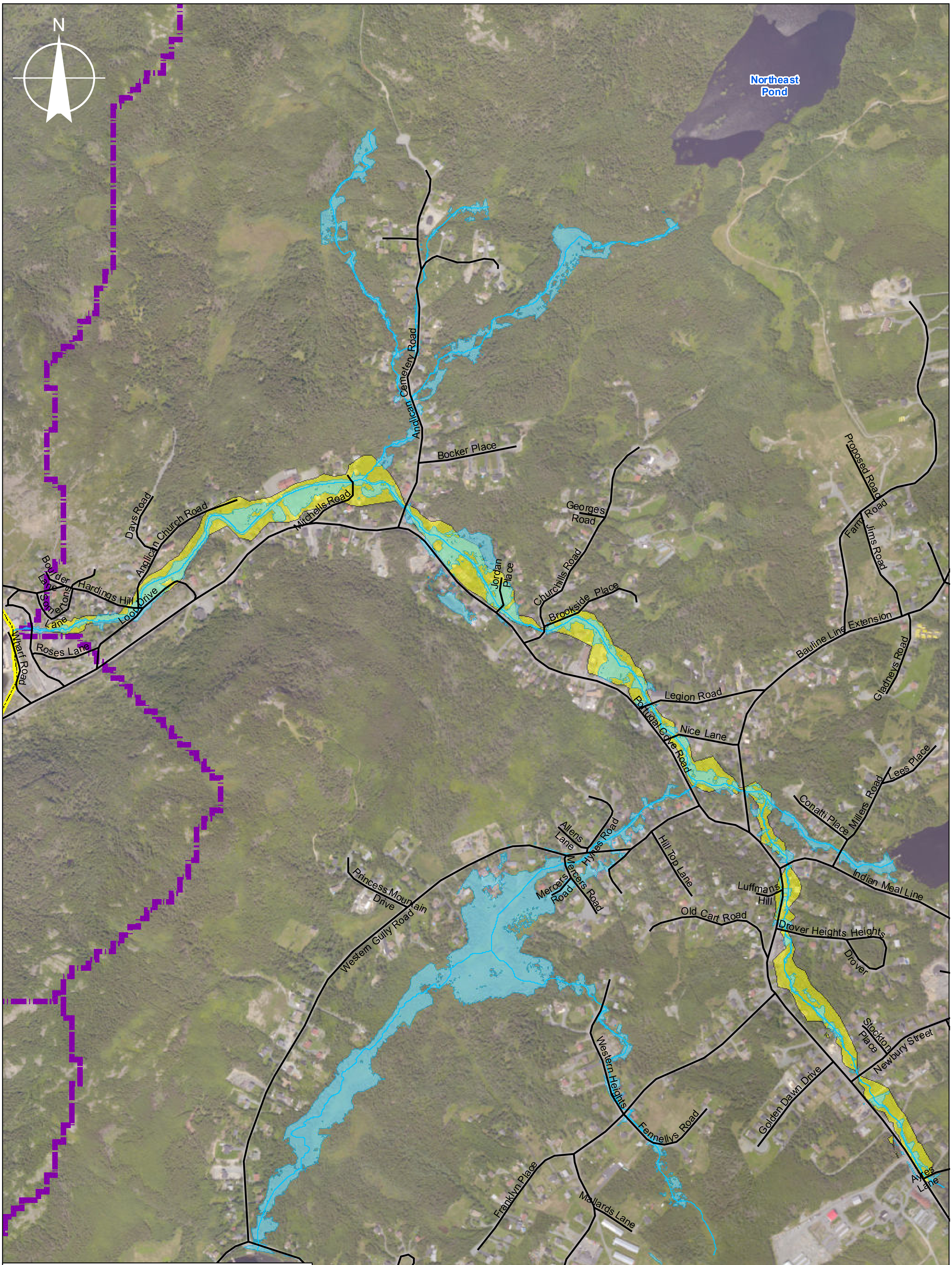
### 7.4.1 Sensitivity Analysis

Input into the hydraulic model includes channel geometry, Manning's roughness coefficient, and peak discharge. The channel geometry input was based on field and LiDAR surveys. There were uncertainties regarding the selection of Manning's roughness coefficient. There were also uncertainties regarding the 1:20-year and 1:100-year AEP peak flows adopted in the hydraulic models.

HEC-RAS uses the Manning's equation in its underlying simulations to calculate the water depth and water level elevation. The sensitivities of Manning's roughness coefficient and peak discharge on flow depth can be evaluated by reviewing Manning's equation. For natural streams with channel width significantly greater than depth, the Manning's equation can be written as follows:

$$D = (nQ/WS^{1/2})^{0.6}$$

Where: D = flow depth (m)
n is the Manning's roughness coefficient (dimensionless)
Q is the discharge rate ($m^3/s$)
W is the channel width (m)
S is the channel slope (m/m)




LEGEND:

- Roads
- Watercourse Centerline
- Watershed Boundary
- Historical 100 Year Flood Zones
- 100 Year Flood Zones by AMEC
- Municipal Boundary

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| CLIENT: WATER RESOURCES MANAGEMENT DIVISION<br>DEPARTMENT OF ENVIRONMENT AND CONSERVATION<br>GOVERNMENT OF NEWFOUNDLAND AND LABRADOR | 100 50 0 100 200 Meters<br>DATUM: NAD 83 CSRS | PROJECT: FLOOD RISK MAPPING STUDY<br>PORTUGAL COVE - ST. PHILIP'S | DWN BY: JT | DATE: MAY, 2015 |
| | | | CHK'D BY: HX | REV. NO: 0 |
| amec foster wheeler | PROJECTION: MTM ZONE 1<br>PROJECT NO: TE144022 | TITLE: COMPARISON OF 1:100 YEAR INUNDATION<br>ZONES WITH PREVIOUS STUDIES FOR MAIN RIVER | SCALE: 1:8,200 | FIGURE NO: 7.2 |

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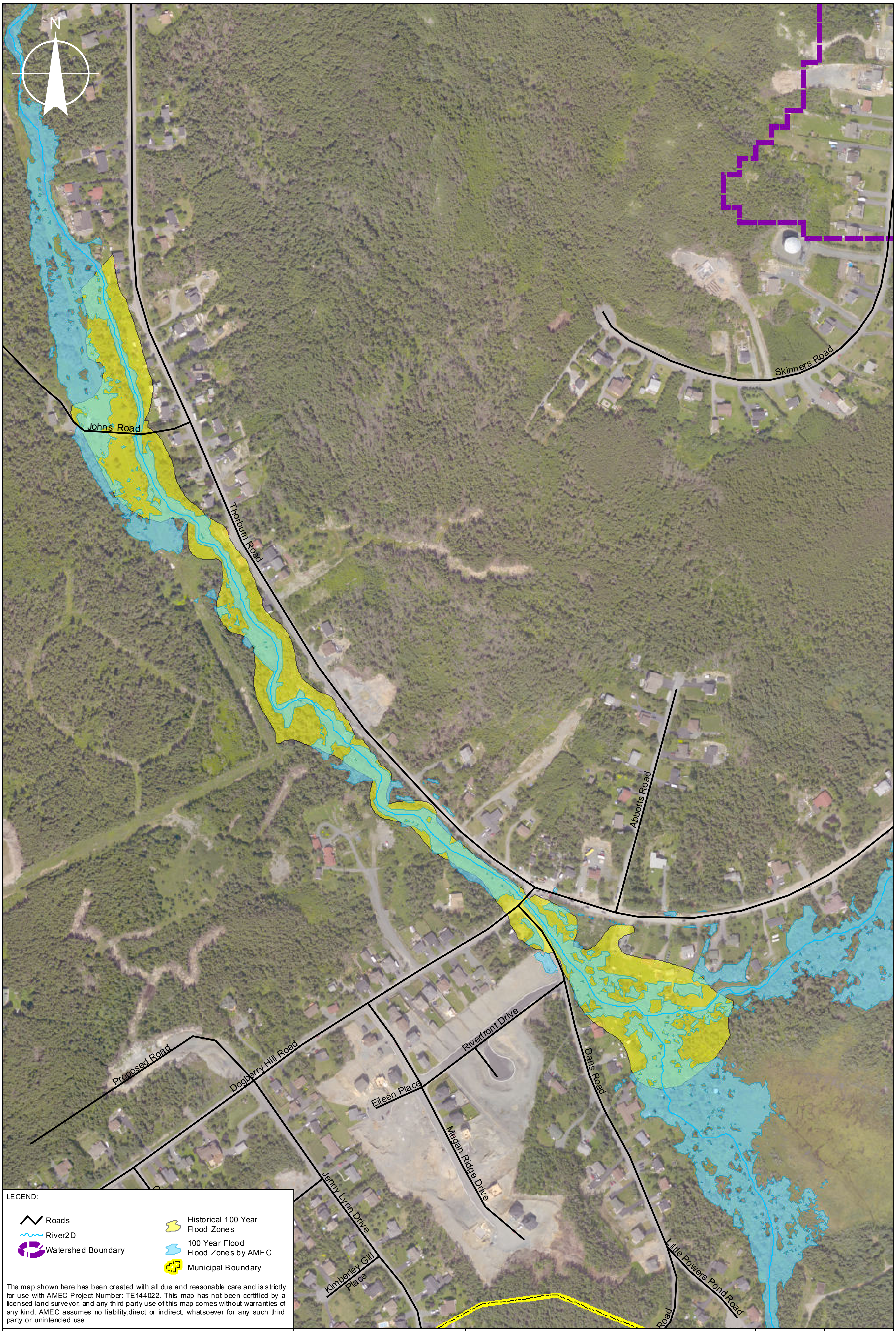
SOURCE: LEGEO LiDAR Imagery 2014, PCSP Municipal Topo

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| CLIENT: WATER RESOURCES MANAGEMENT DIVISION DEPARTMENT OF ENVIRONMENT AND CONSERVATION GOVERNMENT OF NEWFOUNDLAND AND LABRADOR | DATUM: NAD 83 CSRS | PROJECT: FLOOD RISK MAPPING STUDY PORTUGAL COVE - ST. PHILIP'S | DWN BY: JT | DATE: MAY, 2015 |
| | | | CHK'D BY: HX | REV. NO: 0 |
| amec foster wheeler | PROJECTION: MTM ZONE 1 | TITLE: COMPARISON OF 1:100 YEAR INUNDATION ZONES WITH PREVIOUS STUDIES FOR BROAD COVE RIVER | SCALE: 1:4,000 | FIGURE NO: 7.3 |
| | PROJECT NO: TE144022 | | | |

Path: G:\GIS\PROJECTS\TE144022_GovNL_PCSP_2014FloodRiskMappingStudy\MXD\20150721_Fig_7_3.mxd User: tanya.morehouse Date: 21/07/2015 SOURCE: LEGEO LiDAR Imagery 2014,PCSP Municipal Topo

In this equation, the channel width and slope are determined by channel geometry. This equation indicates that flow depth (and therefore water level elevation) is positively related to Manning's roughness coefficient and flow rate. Therefore, an increase in the value for these two parameters will result in an increase in flow depth. The equation also indicates that for a constant width and slope, the effect of the two parameters on flow depth is related to the exponent 0.6, and a change in the value for these two parameters will not result in proportionate change in flow depth. For example, an increase of 20% in either of the two parameters will result in an increase of only about 12% in flow depth if the channel width is relatively wide and constant, and the slope is constant.

To understand the sensitivities of Manning's roughness coefficient and flow discharge rate on flood water levels, it is necessary to understand the degree of uncertainties associated with these two parameters. Unfortunately, this information is typically lacking.

## 7.5 Modelling Results

Results from the hydraulic modelling are presented and discussed in Section 9.0, where the delineation of inundation zones for existing climatic conditions is compared with that for climatic change conditions.

## 8.0 CLIMATIC CHANGE AND EFFECT ON FLOODING RISK

WRMD requires an evaluation of the climate change condition as defined by the climate change IDF curves presented in the report "Projected Impacts of Climate Change for the Province of Newfoundland & Labrador" prepared by Dr. Joel Finnis (referred to as Finnis report in the following) and available at http://www.exec.gov.nl.ca/exec/ccee/publications/index.html. The Finnis report contains projected changes to 19 key climate indices through the integration of available observations and a collection of state-of-the-art regional climate model projections of the 21st century (2038-2070). Some of the findings presented in the report are highlighted below:

- Newfoundland and Labrador is expected to experience changes in temperature, precipitation, and sea level in the future as a result of climate change. These factors can influence the flood-risk faced by a community directly or indirectly.
- In Newfoundland, widespread significant increase in precipitation by mid-century is expected for winter and spring of about 9 mm to 27 mm per season. Significant changes are expected year-round on the Avalon Peninsula and the Great Northern Peninsula.
- A typical precipitation event is expected to become more intense under a warming climate.
- On the Avalon Peninsula, the number of days with precipitation greater than 10 mm is not expected to increase significantly by mid-century.
- In the St. John's area, extreme precipitation is expected to change little seasonally, except in the fall where there is a sharp increase due to the influences of the Atlantic hurricane season, available moisture, and storm track.

Of particular relevance is the effect of climatic change on the IDF curves. In this regard, the Finnis report (2012) acknowledges that estimating future precipitation for specific return period is a much more difficult problem than estimating changes in mean temperature or precipitation. This is particularly true for longer return period estimates, as uncertainty increases as the probability of an event decreases. Similarly, uncertainty increases as the period of record decreases (e.g. estimating the 100-year event from ~30 years is problematic, yet often done).

The Finnis report contains the projected IDF estimates (6/12/24 hours) based on the same data used in the development of the official Environment Canada IDF curves. For St. John's, this data covers the period from 1949 to1996. The Finnis report also contains 24-hour precipitation projections for various return frequencies using Adjusted and Homogenized Canadian Climate Data (AHCCD). The AHCCD data attempts to correct the errors associated with the precipitation measurements (e.g. undercatch due to wind) and may combine the data sequence from different meteorological stations to increase the length of the data sequence. For St. John's, data up to 2011 was used in the projection. A summary of the projected IDF estimates presented in the Finnis report, in comparison with the IDF data discussed in Section 6.0 to represent the current climatic conditions, is provided in Table 8.1.

**Table 8.1 Current and Projected Rainfall Amounts (mm)**

| Duration | Current IDF by Environment Canada | | Current IDF Updated by CBCL 2012 | | Projected Future IDF - Single Station | | Projected Future IDF - AHCCD | |
|---|---|---|---|---|---|---|---|---|
| | 20-yr | 100-yr | 20-yr | 100-yr | 20-yr | 100-yr | 20-yr | 100-yr |
| 5 min | 8.3 | 11.2 | 8.2 | 10.4 | | | | |
| 10 min | 11.9 | 15.7 | 11.9 | 15.0 | | | | |
| 15 min | 15 | 19.9 | 15.2 | 19.2 | | | | |
| 30 min | 20.8 | 27.2 | 22.6 | 28.5 | | | | |
| 1 h | 27.7 | 35.5 | 32.4 | 40.9 | | | | |
| 2 h | 40.2 | 53.1 | 46.8 | 59.8 | | | | |
| 6 h | 62.4 | 78.5 | 75 | 94.2 | 69.3 | 84.4 | | |
| 12 h | 76.5 | 94.5 | 96 | 121.2 | 85.4 | 102.9 | | |
| 24 h | 89.9 | 110.6 | 110.4 | 136.8 | 102.6 | 123.7 | 124.5 | 155.7 |

Note: IDF = Intensity-Duration-Frequency

The IDF projections for future conditions based on the records for the single station at St. John's International Airport is lower than the updated IDF data by CBCL representing current conditions. This is due in part to the length of precipitation records (i.e., from 1949 to 1996) used in the projections. The projections using AHCCD data showed an increase of 13% for 1:20-year precipitation and 14% for 1:100-year precipitation when compared with the CBCL update representing current conditions and used in this study. The data sequence associated with the AHCCD records (up to 2011) is comparable to that used in the CBCL update. For the purpose of this study, the IDF projections using the AHCCD data presented in the Finnis Report was used to represent future climatic conditions. The hyetographs used as input in the hydrological models to represent future climatic conditions are presented in Figure 8.1

The increase in IDF precipitation depth due to climatic change (13% for 1:20-year, 14% for 1:100-year for a 24-hour duration storm) is relatively moderate and within the error of IDF estimates. However, as acknowledged by Dr. Finnis in his report, potentially significant uncertainty in the IDF projections exists, as estimating future precipitations for specified return periods is problematic.

Table 8.2 compares the 1:20-year and 1:100-year AEP peak flows for current climate and projected future climate conditions in 2050 for the locations of the five pressure transducers, and the hydrometric station 02ZM006. It is seen that with the projected increase in IDF data due to climatic change, the increase in 1:20-year and 1:100-year flows range from 17 to 37%. When an exponent of 0.6 is applied to these flow increases, the water level increase can be estimated to range from 10 to 20%.

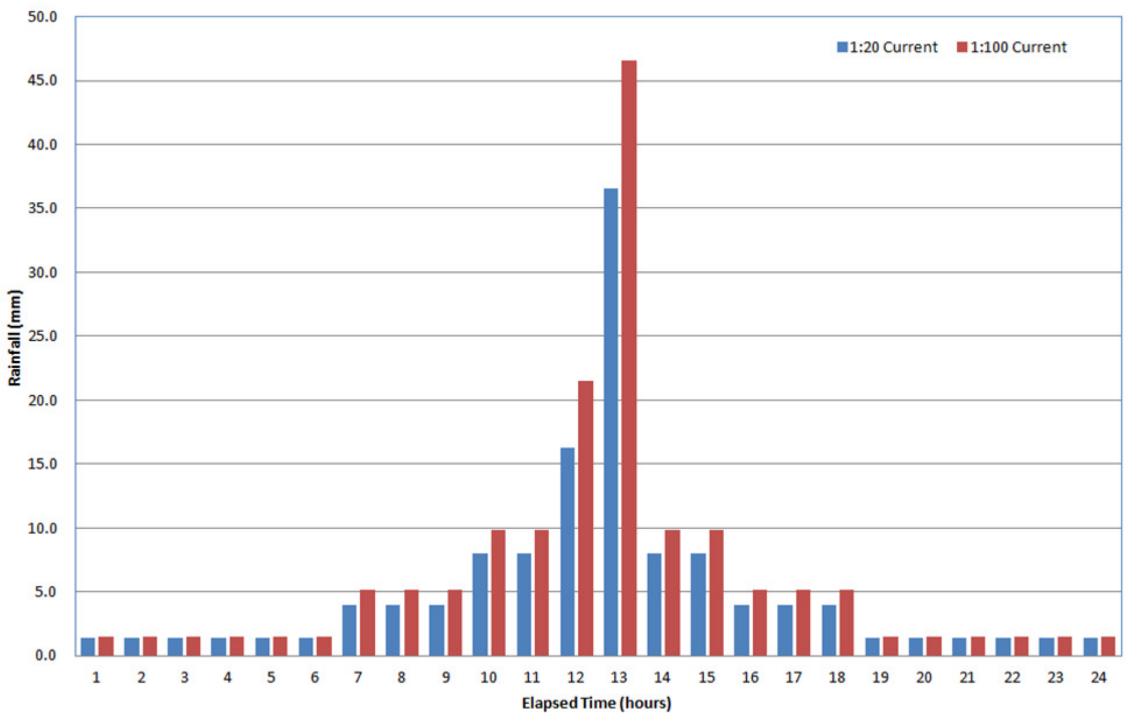


**Figure 8.1 Projected Future (2070) 1:20-Year and 1:100-Year 24-hour Rainfall Hyetographs**

***Table 8.2 Comparison of Peak Instantaneous Flows for Current and Future (2050) Climate Conditions***

| Location | HEC-HMS Element | Drainage Area | Return Frequency | Existing Climate | Projected Climate | Difference |
|---|---|---|---|---|---|---|
| | | (km$^2$) | (yrs) | (m$^3$/s) | (m$^3$/s) | (%) |
| **PT1** | MR1-J6A | 17.00 | 1:20 | 25.2 | 32.9 | 31 |
| | | | 1:100 | 39.1 | 50.9 | 30 |
| **PT2** | MR1-J3A | 6.50 | 1:20 | 10.6 | 14.5 | 37 |
| | | | 1:100 | 17.2 | 23.2 | 35 |
| **02ZM006** | MR2-W3A | 3.63 | 1:20 | 5.1 | 6.5 | 27 |
| | | | 1:100 | 7.8 | 9.9 | 27 |
| **PT3** | BC2-J1 | 0.67 | 1:20 | 1.8 | 2.1 | 17 |
| | | | 1:100 | 2.3 | 2.7 | 17 |
| **PT4** | GC1-J5A | 2.41 | 1:20 | 4.9 | 6.0 | 22 |
| | | | 1:100 | 7.2 | 8.9 | 24 |
| **PT5** | BR1-J8A | 34.59 | 1:20 | 10.4 | 13.9 | 34 |
| | | | 1:100 | 17.4 | 22.8 | 31 |

## 9.0 INUNDATION AND FLOOD HAZARD MAPPING

The established hydrological and hydraulic models discussed in Section 6.0 and 7.0 can be used for generating flood risk mapping in combination with GIS mapping tools. Figure 9.1 shows the flood risk zones for 1:20-year and 1:100-year AEP floods for current climate conditions. Figure 9.2 shows the flood risk zones for 1:20-year and 1:100-year floods for projected future climate conditions.

The WRMD requires a variety of mapping products to be generated. Table 9.1 provides a list of these mapping products (note, the mapping products for future development conditions were not included as this was not considered necessary because of relatively limited future development expected). In total, 14 map series are required at the 1:2,500 scale, three overview map series are also required.

In addition to flood risk mapping, the WRMD requires mapping produced for inundation depth, velocity distribution and flood hazard. The HEC-RAS model determines the water surface elevation at each of the cross sections within the IMA&SN, and flow depth can be determined by comparing the water surface elevation with the underlying LiDAR DEM. The HEC-RAS model adopted for this study is one dimensional which determines the average flow velocity at a cross section. However, this model has a feature that allows the flow at the cross sections to be distributed to the main channel as well as the left and right flood plains. This feature allows a velocity to be estimated for the main channel as well as for the left and right flood plains. The velocity distribution for this study was generated using this feature.

The flood hazard is dependent on velocity and flow depth. The WRMD requires that the matrix, as presented in Figure 9.3, be used for determining flood hazard class. According to this matrix, four classes of flood hazard exist: "Low" (green), "Moderate" (yellow), "Significant" (orange), and "Extreme" (red).

The mapping product series produced as part of this project are contained in Appendix I. The following are noted in the preparation of the mapping series:

- For producing the flood risk maps, HEC-GeoRAS interprets water level between two cross sections lineally. This may cause the flood risk at stream crossings not represented correctly. For example, if the water level is higher than the road surface elevation upstream, but lower than the road surface elevation downstream, HEC-GeoRAS will interpret that only the upstream portion of the road is flooded. In reality for this situation, the road will be overtopped and flooded. Flooding at the road crossings was manually reviewed by comparing the upstream water level with the road surface elevation, and revisions were made where necessary.
- Flood risk for the lakes was modelled in the HEC-HMS model and not in the HEC-RAS model. For producing the flood risk maps, the flood zones for the lakes and streams were combined manually.

- Inundation depth, velocity, and flood hazards maps were not produced for the lakes.
- As indicated previously, historical flood studies covered only two reaches of the Main River and Broad Cove River. Shape files showing the flood zones as adopted in the environmental protection map of the Town were obtained and used for representing the historical flood zones in preparing the comparison maps (Map Series C4 and C5). As noted previously, some discrepancies were noted between the shape files and flood zones presented in the reports of the historical flood studies. The origin of the discrepancies could not be confirmed.

The WRMD requires the integration of the modelling components developed in this study, including the HEC-HMS and HEC-RAS models, using the GIS Map to Map work flow. This integration enables the hydrological and hydraulic modelling to be completed in one operation in GIS platform. Output from the modelling can be incorporated directly into GIS mapping presentations. As part of this project, the Map to Map work flow was established for each of the five watersheds.


CLIENT:
**WATER RESOURCES MANAGEMENT DIVISION**
**DEPARTMENT OF ENVIRONMENT AND CONSERVATION**
**GOVERNMENT OF NEWFOUNDLAND AND LABRADOR**

amec foster wheeler

300 150 0 300 600 Meters

DATUM: NAD 83 CSRS

PROJECTION: MTM ZONE 1

PROJECT NO: TE144022

PROJECT:
**FLOOD RISK MAPPING STUDY**
**PORTUGAL COVE - ST. PHILIP'S**

TITLE:
**INUNDATION ZONES FOR 1:20 AND 1:100**
**FLOODS FOR CURRENT CLIMATE CONDITIONS**

| | |
|---|---|
| DWN BY: JT | DATE: MAY, 2015 |
| CHK'D BY: HX | REV. NO: 0 |
| SCALE: 1:35,000 | FIGURE NO: 9.1 |

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SOURCE: LEGEO LiDAR Imagery 2014,PCSP Municipal Topo


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| | | | CHK'D BY: HX | REV. NO: 0 |
| amec foster wheeler | PROJECTION: MTM ZONE 1<br>PROJECT NO: TE144022 | TITLE: INUNDATION ZONES FOR 1:20 AND 1:100<br>FLOODS FOR FUTURE CLIMATE CONDITIONS | SCALE: 1:35,000 | FIGURE NO: 9.2 |

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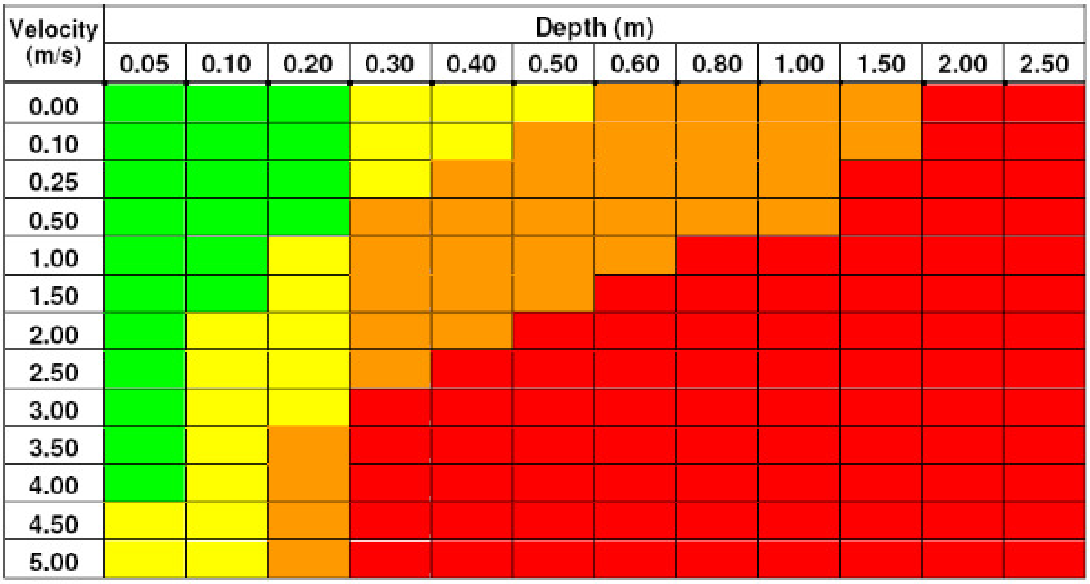
SOURCE: LEGEO LiDAR Imagery 2014,PCSP Municipal Topo

***Table 9.1 Summary of Inundation and Flood Hazard Mapping***

| Description | Map Scale | Map Background | AEP Event Presented | | Map Series ID |
|---|---|---|---|---|---|
| | | | 1:20 | 1:100 | |
| **Current Climate Condition** | | | | | |
| Flood risk | Overview | Orthophotography | Y | Y | A1 |
| Flood risk | 1:2,500 | Orthophotography | Y | Y | A2 |
| Flood risk | 1:2,500 | Community Mapping | Y | Y | A3 |
| Inundation depth | 1:2,500 | Community Mapping | Y | | A4 |
| Inundation depth | 1:2,500 | Community Mapping | | Y | A5 |
| Velocity distribution | 1:2,500 | Community Mapping | Y | | A6 |
| Velocity distribution | 1:2,500 | Community Mapping | | Y | A7 |
| Flood hazard | 1:2,500 | Community Mapping | Y | | A8 |
| Flood hazard | 1:2,500 | Community Mapping | | Y | A9 |
| **Projected Future Climate Condition** | | | | | |
| Flood risk | Overview | Orthophotography | Y | Y | B1 |
| Flood risk | 1:2,500 | Orthophotography | Y | Y | B2 |
| Flood risk | 1:2,500 | Community Mapping | Y | Y | B3 |
| **Comparisons** | | | | | |
| Flood risk for current climate vs. projected future climate | Overview | Orthophotography | Y | Y | C1 |
| Flood risk for current climate vs. projected future climate | 1:2,500 | Orthophotography | Y | Y | C2 |
| Flood risk for current climate vs. projected future climate | 1:2,500 | Community Mapping | Y | Y | C3 |
| Flood risk for current climate vs. historical flood zones | 1:2,500 | Orthophotography | Y | Y | C4 |
| Flood risk for current climate vs. historical flood zones | 1:2,500 | Community Mapping | Y | Y | C5 |




Uden, et al (2007)

**Figure 9.3 Velocity – Depth Flood Hazard Matrix**

# 10.0 CAPACITY OF THE STREAM-CROSSING STRUCTURES

The stream-crossing structures surveyed and included in the HEC-RAS models are summarized in Table 10.1. The types, important dimensions and elevations, and conditions of the structures are summarized in Table 10.1. Details of the stream-crossing structures are contained in Appendix D.

For the purpose of this study, it was assumed that the capacities of the stream-crossing structures were determined by the following criteria:

- the surface of the road at the crossing site will not be overtopped during the design storm event; and
- the backwater from the stream-crossing site will not cause flooding of upstream residences at the ground level.

There may be other criteria in determining the capacity of the stream-crossing structures (e.g., fish passage, sofit, etc.). These criteria have not been considered in this study. For the purpose of this study, the capacities of the stream-crossing structures were evaluated for passing the 1:20-year and 1:100-year AEP flows.

There were two elevations considered in evaluating the capacity of the stream-crossing structures:

- the road crest elevation at the stream-crossing site; and
- the ground elevation of the lowest upstream residences.

The lower of the two elevations was used in evaluating the capacities of the stream-crossing structures. The road crest elevation was surveyed in the field programs and summarized in Table 10.1. As part of this task, the elevation for the lowest upstream residence was identified and compared with the road crest elevation at the stream-crossing site. If the elevation for the residence was found to be lower than the road crest elevation, it was used to replace the road crest elevation for determining the capacities of the stream-crossing structures. It was found through this exercise that, generally, the capacities of the stream-crossing structures were governed by the crest elevation of the road.

The 1:20-year and 1:100-year AEP water surface elevations at the stream-crossing sites for current development and climate conditions are summarized in Table 10.1 in comparison with the road surface elevations. The flood elevation is highlighted when it is higher than the road crest elevation, and the capacity of the stream crossing structure is inadequate.

## Table 10.1 Comparison of Upstream Flood Water Levels and Top Elevation of the Road Crossing

| Stream Crossing ID | Structure Type | Structure Dimensions | Road Crest Elevation | Upstream Water Level | | Structure Condition |
|---|---|---|---|---|---|---|
| | | | | 1:20 | 1:100 | |
| **Main River** | | | | | | |
| MR-1 | Bridge | | 9.9 | 8.3 | 9.0 | |
| MR-2 | CSP arch with open bottom | Span: 6.2 m Rise: 1.8 m | 31.1 | **31.4** | **31.8** | Good condition, no blockages or damage noted. |
| MR-3 | Bridge | Span: 5.0 m Height: 1.4 m | 38.7 | **39.1** | **39.2** | |
| MR-4 | Bridge | | 41.6 | **42.0** | **42.7** | |
| MR-5 | CSP arch | Span: 1.6 m Rise: 1.1 m | 65.0 | 64.4 | 64.9 | Good condition, but has some notable blockages by boulders. |
| MR-6 | Bridge | | 67.6 | **67.8** | **67.8** | |
| MR-7 | CSP circular | Diameter: 0.8 m | 75.3 | **75.6** | **75.7** | Culvert is new and is in good condition. |
| MR-8 | CSP arch | Span: 1.65 m Rise 0.95 m | 92.0 | **92.2** | **92.4** | Good condition, no blockages or damage noted. |
| MR-9 | CSP arch | Span: 1.4 m Rise: 1.0 m | 98.5 | **98.6** | **98.7** | Good condition, no blockages or damage noted. |
| MR-13 | Bridge | | 53.6 | 52.0 | 53.1 | |
| MR-14 | C.S.P Arch Culvert | Span: 5.0 m Height: 2.4 m | 55.1 | **55.5** | **55.6** | |
| MR-15 | Bridge | | 60.1 | **60.4** | **60.5** | |
| MR-16 | Box Culvert | Span: 4.3 m Height: 1.7 m | 60.7 | **60.9** | **61.0** | |
| MR-17 | Bridge | | 66.5 | **67.1** | **67.3** | |
| MR-18 | Bridge | | 76.2 | 75.1 | 75.5 | |
| MR-19 | CSP arch with open bottom | Span: 6.0 m Rise: 1.4 m | 78.6 | **78.7** | **79.0** | Good condition, no blockages or damage noted. |
| MR-20 | Bridge | Span: 3.9 m Height: 2.2 m | 80.6 | 79.3 | 79.6 | |
| MR-21 | CSP arch with open bottom | Span: 5.7 m Rise: 1.5 m | 83.5 | 83.0 | 83.6 | Good condition, no blockages or damage noted. |
| MR-22 | Bridge | | 89.1 | 88.6 | 88.9 | |
| MR-23 | CSP arch | Span: 1.8 m Rise: 1.2 m | 109.0 | **109.1** | **109.2** | Some rusting, mostly on bottom. |
| MR-24 | CSP arch | Span: 1.8 m Rise: 1.2 m | 111.0 | 110.8 | 110.8 | Some rusting, mostly on bottom. |
| MR-25 | CSP circular | Diameter: 0.6 m | 125.8 | 125.6 | 125.9 | Bottom of culvert is rusting. |
| MR-26 | Corrugated HDPE circular | Diameter: 0.7 m | 144.4 | 143.8 | 144.3 | Culvert is in good condition but there is a buildup of silt in the culvert. |
| MR-28 | CSP arch with open bottom | Span: 4.0 m Rise: 2.0 m | 96.7 | 95.9 | 96.3 | Good condition, no blockages or damage noted. |
| MR-29 | Bridge | Span: 2.0 m Height: 0.9 m | 103.1 | 103.0 | **103.1** | |
| MR-30 | Bridge | Span: 2.3 m Height: 0.6 m | 104.5 | **105.5** | **105.5** | |
| MR-31 | CSP arch | Span: 1.4 m Rise: 1.0 m | 109.8 | 109.9 | 110.0 | Good condition, no blockages or damage noted. |
| MR-32 | C.S.P. Culvert | Diameter: 0.6 mm | 112.6 | **112.7** | **112.8** | |
| MR-33 | C.S.P. Culvert | Diameter: 0.6 mm | 113.0 | **113.3** | **113.3** | |
| MR-34 | C.S.P. Culvert | Diameter: 0.45 m (3) | 115.0 | **115.1** | **115.1** | |

**Table 10.1 Comparison of Upstream Flood Water Levels and Top Elevation of the Road Crossing**

| Stream Crossing ID | Structure Type | Structure Dimensions | Road Crest Elevation | Upstream Water Level | | Structure Condition |
|---|---|---|---|---|---|---|
| | | | | 1:20 | 1:100 | |
| MR-35 | CSP circular | Diameter: 1.2 m | 116.5 | 116.1 | 116.3 | Fair condition, large indent on top of culvert on downstream side. |
| MR-36 | Bridge | | 113.0 | **113.3** | **113.5** | |
| MR-37 | CSP open bottom arch | Span: 2.8 m Rise: 2.0 m | 142.0 | 140.5 | 140.5 | Good condition, no blockages or damage noted. |
| MR-38 | C.S.P. Culvert | Diameter: 0.45 m | 144.2 | **144.2** | **144.3** | |
| MR-39 | Culvert | Diameter: 3 @ 0.6 m | 147.2 | **147.3** | **147.3** | |
| MR-40 | CSP arch | Span: 1.2 m Rise: 0.8 m | 138.8 | **139.2** | **139.3** | Fair condition, minor rusting. Infilled with gravel on bottom. |
| MR-41 | C.S.P. Culvert | Diameter: 0.6 m | 148.5 | **148.6** | **148.7** | |
| MR-42 | Concrete Culvert | Diameter: 0.6 m | 148.7 | **148.8** | **148.8** | |
| MR-44 | CSP circular | Two culverts with 0.4 m and 0.5 m diameter. | 149.2 | **149.4** | **149.5** | Culverts appear to be partially blocked with mud/detritus at the upstream end. |
| MR-45 | Corrugated HDPE circular | Diameter: 0.60 m | 149.4 | **149.6** | **149.6** | Road over top of culvert is disintegrating. |
| MR-46 | CSP circular | Diameter: 0.6 m | 150.7 | **150.8** | **150.9** | |
| MR-48 | CSP circular | Diameter: 1.0 m | 149.2 | 148.6 | 148.7 | Good condition, no blockages or damage noted. |
| MR-49 | CSP circular | Diameter: 1.5 m | 132.7 | **132.8** | **132.9** | New culvert/good condition, no blockages or damage noted. |
| **Unnamed Stream** | | | | | | |
| UN-1 | CSP circular | Diameter: 1.45 m | 13.4 | 12.4 | 12.8 | Bottom of culvert rusting away. |
| UN-2 | CSP circular | Diameter: 1.0 m | | | | Long culvert removed from the model due to suspected non-performance. |
| UN-4 | CSP Circular | Diameter: 0.8 m | | | | Culvert is in good condition but there are signs of rusting along the bottom. There is a steel mesh trash rack located immediately upstream of the inlet (culvert removed from the model due to suspected non-performance of the culvert). |
| UN-5 | CSP/HDPE circular | Diameter: 0.6 m | | | | Culvert in good condition (driveway culvert not modelled) |
| UN-6 | CSP circular | Diameter: 0.9 m | 32.9 | 32.5 | 33.0 | Bottom of culvert badly rusted (portions are rusted away). The inlet of the culvert is blocked by grass clippings dumped by nearby residents. At the time of the survey there was no water flowing through the culvert. |
| UN-7 | CSP circular | Diameter: 0.45 m | 34.2 | **34.4** | **34.5** | Culvert is in good condition. |
| UN-8 | CSP circular | Diameter: 0.60 m | 37.1 | **37.2** | **37.3** | Culvert is in good condition. |

## Table 10.1 Comparison of Upstream Flood Water Levels and Top Elevation of the Road Crossing

| Stream Crossing ID | Structure Type | Structure Dimensions | Road Crest Elevation | Upstream Water Level | | Structure Condition |
|---|---|---|---|---|---|---|
| | | | | 1:20 | 1:100 | |
| UN-9 | CSP arch | Span: 1.2 m Rise: 0.8 m | 40.5 | 39.5 | 39.8 | Generally culvert is in good condition with some rust visible on the inside (primarily along the bottom). |
| **Beachy Cove Brook** | | | | | | |
| BE-1 | CSP circular | Diameter: 2.0 m | 26.5 | 25.6 | 26.1 | Good condition, no blockages or damage noted. |
| | CSP arch | Span: 1.8 m Rise: 1.4 m | 26.5 | 25.6 | 26.1 | Good condition, no blockages or damage noted. |
| BE-2 | CSP circular | Diameter: 0.5 m, 0.2 m | 120.7 | 120.4 | 120.5 | Edge of the DS side of the the corrugated culvert is bent and shows signs of rusting. There are no obvious signs (e.g. holes) of deterioration. |
| BE-3 | CSP circular | Diameter: 0.60 m | 122.9 | 121.5 | 120.7 | Bottom of culvert is rusted. |
| BE-5 | CSP circular | Diameter: 1.5 m | 134.6 | 132.8 | 133.0 | Bottom of culvert rusted. |
| BE-6 | CSP circular | Diameter: 2 @ 1.35 m | 136.2 | 135.4 | 135.6 | Good condition, slight rusting along bottom but no signs of deterioration. |
| BE-7 | Concrete box | Height: 0.9 Width: 3.05 m | 137.0 | 136.5 | 136.6 | Good condition with no signs of deterioration. |
| BE-8 | CSP arch | Span: 1.9m Rise: 1.2 m | 137.7 | 137.5 | 137.6 | Bottom of culvert is rusty but there were no perforations visible. |
| BE-9 | CSP Circular | Diameter: 2 @ 0.6 m | 138.7 | **139.1** | **139.2** | Culverts rusted along bottom but no sign of perforation. |
| BE-10 | CSP circular | Diameter: 0.8 m | 133.1 | **133.4** | **133.4** | Culvert is in good condition with minimal signs of rusting. |
| BE-11 | CSP circular | Diameter: 0.75 m | 137.0 | 136.7 | 136.7 | Culvert is in good condition with minimal signs of rusting. |
| BE-12 | CSP circular | Diameter: 0.8 m | 139.7 | **140.0** | **140.1** | Culvert is in good condition. |
| BE-13 | CSP arch | Diameter: 1.5 m | 141.8 | 140.2 | 140.4 | Culvert is in generally good condition but there is some surface rust. |
| BE-14 | CSP circular | Diameter: 0.76 | 152.82 | 152.3 | 152.4 | Good Condition. |
| BE-15 | CSP circular | Diameter: 0.50 | 155.30 | >155.3 | >155.3 | Bottom rusted. |
| BE-16 | CSP circular | | 137.3 | **137.5** | **137.6** | Culvert is in good condition. |
| BE-17 | CSP circular | Diameter: 0.6 m | 158.7 | 158.2 | 158.2 | Culvert is in good condition. |
| **Goat Cove Brook** | | | | | | |
| GC-1 | CSP arch | Span: 1.8 m Rise: 1.25 m | 78.2 | 76.7 | 77.2 | |
| GC-2 | CSP circular | Diameter: 1.2 m | 104.4 | 103.5 | 103.7 | Culvert is in good condition, but is bent on top. |

**Table 10.1 Comparison of Upstream Flood Water Levels and Top Elevation of the Road Crossing**

| Stream Crossing ID | Structure Type | Structure Dimensions | Road Crest Elevation | Upstream Water Level 1:20 | Upstream Water Level 1:100 | Structure Condition |
|---|---|---|---|---|---|---|
| GC-3 | CSP circular | Inlet Diameter: 1.05 m<br>Outlet Diameter: 0.9 m | 110.7 | 110.2 | 110.5 | Bottom of corrugated section rusted through near outlet. Appears that a portion of the culvert under the road way has been partially crushed and loose fill has fallen into the culvert. |
| GC-4 | CSP circular | Diameter: 0.6 m | 116.2 | 115.4 | 115.5 | Rock at outlet. |
| GC-5 | CSP arch | Span: 1.22 m Rise: 0.95 m | 116.7 | 115.7 | 115.9 | Culvert is rusty, especially bottom. Culvert appears to be partially crushed with fill visible through the sides of culvert. |
| GC-6 | CSP circular | Diameter: 0.9 m | 135.9 | 135.5 | 135.6 | Inlet end of culvert is visibly rusty. |
| **Broad Cove River** | | | | | | |
| BR-0 | Bridge | | 3.1 | 1.8 | 1.8 | |
| BR-2 | Bridge | | 39.8 | 37.8 | 38.1 | |
| BR-3 | Bridge | | 83.8 | 82.9 | 83.0 | |
| BR-5 | CSP open bottom arch | Approximate span of 7.5 m and rise of 1.66 m | 107.4 | 106.3 | 106.7 | Newly constructed structure (Nov. 2014) |
| BR-6 | Bridge | | 109.1 | **109.4** | **109.4** | |
| BR-7 | CSP circular | Diameter: 2 @ 1.5 m | 124.9 | 124.4 | 124.7 | Culvert is in good condition. |

There are several options to improve the capacity of a stream-crossing structure, including the following:

- increase the dimensions of the structure, to improve the hydraulics (e.g., installation of headwall and wing walls for a culvert), or both;
- replace the structure with a more hydraulically efficient structure; and
- allow for greater headwater by raising the road profile while avoiding the flooding of upstream third party properties.

Selection of the preferred option for improving the capacity of the stream-crossing structure will depend on a host of considerations that are beyond the scope of this project. In addition to water levels, other criteria must also be incorporated into the design process.

# 11.0 RECOMMENDATIONS

Key recommendations are as follows:

1. It is recommended that the GNL implement a program to survey the peak flood water levels following significant flood events. Proper documentation of these data will provide valuable information for future inundation studies, and for updating existing flood studies. A significant constraint for this inundation study is the lack of suitable calibration data for the hydraulic model. The most ideal calibration data for this purpose should consist of peak flood water levels in various areas of the watersheds. These data are best collected shortly after significant flood events.

2. It is recommended that standards be developed regarding the identification of inundation modelling areas and the associated stream network, especially when numerous small streams are involved. In this study, the perennial stream network shown on 1:50,000 topographic map was adopted as the modelling stream network. The WRMD may wish to adopt this as the standards, or other standards at its discretion.

3. It is recommended that the LiDAR survey should be required to cover the entire watershed areas to avoid the difficulties associated with working with different DEMs.

4. It is recommended that the CN for a particular land classification and soil type be given as a range, rather than a discrete value, to enable calibration of the hydrological model. Mechanisms could be developed to enable consideration of the effect of a shallow soil profile and the permeability of the underlying bedrock on the selection of the CN.

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